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Steel Bridges

Conceptual and Structural Design of Steel and Steel-Concrete Composite Bridges

Jean-Paul Lebet, Manfred A. Hirt

Translated from the French by
Graham Couchman

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Preface

This book is published by the EPFL Press, the English language imprint of the *Presses Polytechniques et Universitaires Romandes* (PPUR). It is one of a series of 25 books, three of which concern steel construction, published in French under the collective title of *Traité de Génie Civil* (TGC).

Volume 12 of the TGC is based on the courses given at the Swiss Federal Institute of Technology in Lausanne (EPFL), on the theoretical and applied research undertaken at the Steel Structures Institute (ICOM), and on contacts with industry. It deals with both the conceptual and structural design of steel and composite (steel-concrete) bridges and is compatible with the basic principles and design methods developed in Volume 10 of the TGC. Taken together, Volumes 10 to 12 of the TGC are useful to both students, as support for their courses and projects, and practicing engineers searching for as deep an understanding of the subject as possible. Their contents apply to the design of steel structures in general, and, in particular, to industrial and administrative buildings, and bridges.

The subject of bridges is so deep and wide that it is not possible to cover all types of bridge, and their details, in a volume such as this. Rather this book focuses primarily on beam bridges, emphasising the basis of their conceptual design and the fundamentals that must be considered in order to assure structural safety and serviceability, as well as highlighting the necessary design checks. The guidance can be extended and applied to other types of structure. The contents of this book deal first of all, and in detail, with road bridges, followed by chapters with specifics of railway bridges and bridges for pedestrians and cyclists.

The book is divided into five parts. The first part is a ***general introduction*** to bridges, illustrating specific ***terminology*** and giving a historical background for steel bridges. The second part considers the ***conceptual design*** of the primary structural elements and construction details for both steel and composite steel-concrete bridges. The different phases in the elaboration of a bridge construction project are described, with an emphasis on the qualities that a bridge should possess. In particular, this section highlights the relevance of the choice of materials and methods of erection to the basic conceptual design. The third part is dedicated to the ***analysis* and *design*** of the structural members of steel and composite bridges. It reminds the reader first of all of the key design principles and notes the different actions to be consider for a road bridge. It then identifies the checks that are needed to assure structural safety and serviceability. These checks are based on the principles contained in modern codes and standards from Switzerland (SIA) and Europe (Eurocodes). The fourth part deals with specific aspects of the conception and peculiarities of ***other types of bridge*** such as railway bridges, bridges for pedestrians and cyclists, and arch bridges. Particular attention is paid to consideration of the dynamic performance of bridges for pedestrians and cyclists. A method is provided for checking this behaviour for simple structures. The final part contains a ***numerical example*** for a composite bridge. It illustrates the important steps in the analysis and design of the structure in order to reinforce the theory with a numerical application of the various checks.

Acknowledgements

The French version of this book was the result of substantial work undertaken by numerous personalities linked to the Steel Structures Institute (ICOM). The authors would like to extend their warmest thanks to all who have participated. Particular acknowledgement is given to Michel Thomann for his assistance with the first draft of several chapters, notably those covering other types of bridge. The authors also warmly thank Joël Raoul for reading and commenting on the first version of the work. They equally thank Marcel Tschumi for his advice concerning the chapter dedicated to railway bridges. Their gratitude extends to

Yves Rey, Dimitrios Papastergiou, Michel Crisinel and Laurance Davaine for their advice and help concerning the numerical parts of the book.

The conceptual and structural design of steel and composite bridges could not be conveyed without a clear and precise graphic presentation and attractive page layout. The authors want to sincerely thank Claudio Leonardi for the great care he took with the preparation of the figures in this book.

Special thanks from the authors go to Graham Couchman, who accepted the mission to translate the book and used much personal engagement and competence to do it well.

The page layout and text corrections were carried out by Anne Kummli, and the book was proofread by Emily Lundin. Management of the production was overseen by Christophe Borlat of the PPUR. We hope that all these people, along with the manager of the EPFL Press, Frederick Fenter, and the Director of the PPUR, Olivier Babel, will accept our thanks for the patience, support and care they brought to preparation and realization of this book.

Lausanne, April 2013 *Jean-Paul Lebet and Manfred A. Hirt*

Photographs

The authors of this work would like to thank all those who made their photographs available in order to illustrate bridges of note.

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Contents

CONCEPTUAL DESIGN OF BRIDGES

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1 Introduction

Inclined leg composite bridge between Varone and Loèche (CH) over the Dala river.
Eng. Zwahlen & Mayr, Aigle and Zumofen & Glenz AG, Steg.
Photo Zwahlen & Mayr, Aigle.

1.1 Objectives

This book is one of a series published in French under the collective title, *Traité de Génie Civil* (TGC). Within the TGC series, this volume represents the third of three parts dedicated to steel structures, originally published as:

- Volume 10: *Construction métallique*/Notions fondamentales et méthodes de dimensionnement
- Volume 11: *Charpentes métalliques*/Conception et dimensionnement des halles et bâtiments.
- Volume 12: *Ponts en acier*/Conception et dimensionnement des ponts métalliques et mixtes acier-béton.

These volumes are addressed equally to students, for support in their courses and projects, and to practicing engineers seeking to deepen their understanding of the subject. Steel construction is considered in the context of single and multi storey industrial and administrative buildings and bridges. All three volumes are based on courses given by members of the Steel Structures Institute (ICOM) in the Swiss Federal Institute of Technology at Lausanne (EPFL), and on the course notes prepared earlier at ICOM [1.1] [1.2] [1.3] [1.4].

The objective of this twelfth volume is to present the theory behind the conceptual and structural design of steel and composite (steel-concrete) bridges. Given the breadth and complexity of the subject of bridges, it is clearly not possible in this single volume to cover all the details of all types of bridges. Choices had to be made. It was decided to focus this volume primarily on beam bridges with an emphasis on both their conceptual design and the fundamental considerations regarding structural safety and serviceability. The guidance nevertheless applies to other types of structures. The contents of this book deal first of all, and in detail, with road bridges. The principles are extended in later chapters to railway bridges as well as bridges for pedestrians and cyclists.

Ideas for bridge configuration are based on practical experience obtained not only in Switzerland [1.5], but equally on experience from other countries with long traditions of steel construction (USA [1.6], France [1.7], Great Britain [1.8] [1.9], and Germany [1.10]). Design principles are based on Swiss experience (SIA standards), in addition to European experience as reflected in the recent set of Eurocodes. An example of the configuration and design of a composite bridge, with numerical illustrations, is presented at the end of the book. It explicitly illustrates the theory contained in the book.

To understand and apply the contents of this volume, an understanding of the basic theory and design methods presented in Volume 10, in addition to certain aspects developed in Volume 11, is necessary.

1.2 Structure and Contents

This book comprises a total of 19 chapters grouped into five parts (Table 1.1) as follows:

- In the first part, *Introduction to Bridges*, the reader enters the world of bridges and becomes familiar with terminology specific to these structures. This facilitates understanding of the subsequent parts on the analysis and design of steel bridges. In addition to an introduction (Chapter 1), this part includes a classification and description of bridges and their constituent parts (Chapter 2), as well as an illustrated history of steel and composite bridges (Chapter 3).
- The second part concerns the *Conceptual Design of Bridges*. Having considered some basics regarding the conceptual design of a bridge and a bridge project (Chapter 4), this part considers structural forms used for bridges and the role of various structural elements (Chapter 5). Chapter 6 presents principal construction details for bridge beams and essential elements for a good concept,

Table 1.1 Layout of this book with division into Parts and Chapters.

Parts	Chapters
Introduction to Bridges	1. Introduction 2. Bridge Description 3. History of Steel and Composite Bridges
Conceptual Design of Bridges	4. Basis for Conceptual Design 5. Structural Forms for Bridges 6. Construction Details 7. Fabrication and Erection of the Steel Structure 8. Slabs of Composite Bridges
Analysis and Design (Beam Bridges)	9. Basis of Design 10. Loads and Actions 11. Internal Moments and Forces in Beam Bridges 12. Steel Beams 13. Composite Beams 14. Cross Bracing and Plan Bracing 15. Overall Stability
Other Types of Bridges	16. Railway Bridges 17. Bridges for Pedestrians and Cyclists 18. Arch Bridges
Numerical Example	19. Example of a Composite Bridge

or configuration. The final two chapters in this part concern the fabrication and erection of the steel structure (Chapter 7), as well as the conceptual design and construction of the concrete slab used in a composite bridge (Chapter 8). Although the second part of the book contains no calculations, it allows the engineer to conceive a structure for the preliminary sizing that is both realistic and sound.

- The third part covers the *Analysis and Design* of beam bridges. It covers the determination of the internal forces and the design checks necessary to assure structural safety and serviceability. Having reminded the reader of design principles (Chapter 9) and described various actions (Chapter 10), this part deals with the spatial behaviour of bridges and the calculation of internal forces (Chapter 11). The four subsequent chapters cover the checks to be carried out, firstly for steel beams (Chapter 12), then composite steel-concrete beams (Chapter 13). The final two chapters consider the checks associated with the cross bracing and plan bracing (Chapter 14) and the overall stability of the bridge (Chapter 15).
- The fourth part, dealing with *Other Types of Bridge*, adopts a more descriptive approach to the conceptual and structural design of particular bridges as a function of their use or form. This part considers railway bridges (Chapter 16), bridges for pedestrians and cyclists (Chapter 17), and arch bridges (Chapter 18).
- The fifth and final part, a *Numerical Example,* illustrates, by consideration of a composite road bridge, the key notations and principles covered in this book.

This book does not consider the configuration and design of infrastructure associated with bridges (abutments, piers, and foundations). These subjects are covered in Volume 9 of the TGC concerning concrete bridges.

1.3 Reference Documents

1.3.1 Standards and Recommendations

The theory given in this volume is based on fundamentals regarding the resistance of materials and statics, presented in Volumes 1 to 5 of the TGC, as well as those for steel construction, noted in Volumes 10 and 11. They are independent of any consideration of standards. The latter varies by country and does, however, influence design. For this book we have taken the Swiss design rules, as given in the standards produced by the Swiss Society of Engineers and Architects (SIA) in Zurich (www.sia.ch). The following standards have been used for reference:

- SIA 260 "Bases pour l'élaboration des projets de structures porteuses" (Basis of Structural Design) (2003),
- SIA 261 "Actions sur les structures porteuses" (Actions on Structures) (2003),
- SIA 262 "Construction en béton" (Concrete Structures) (2003),
- SIA 263 "Construction en acier" (Steel Structures) (2003),
- SIA 264 "Construction mixte acier-béton" (Composite Steel and Concrete Structures) (2003),
- SIA 269 "Bases pour la maintenance des structures porteuses" (Maintenance of Structures) (2009).

In some cases the SIA standards refer their reader to the Eurocodes, in particular concerning specific calculation methods (the SIA standards primarily present principles and rules). This book is therefore also based on the following documents published by CEN in Brussels (www.cenorm.be):

- EN 1990 Eurocode "Basis of Structural Design" (2002),
- EN 1991 Eurocode 1 "Actions on Structures" (2002-2006),
- EN 1992 Eurocode 2, Part 2 "Design of Concrete Structures – Concrete Bridges – Design and Detailing Rules" (2005),
- EN 1993 Eurocode 3, Part 2 "Design of Steel Structures – Steel Bridges" (2006),
- EN 1994 Eurocode 4, Part 2 "Design of Composite Steel and Concrete Structures – General Rules and Rules for Bridges" (2005).

Alongside the standards noted above, we also refer to certain publications from the Swiss Centre for Steel Construction (SZS) in Zurich (www.szs.ch). These documents serve as an invaluable reference for values of the characteristic properties of steel sections, for the resistance of connections (welds, bolts and shear studs) and joints, and for the resistance of members (considering the various types of buckling, composite cross sections, etc.). They also give guidance on conceptual design, fabrication, erection, and protection against the corrosion of steel structures. Key documents include:

- SZS B3 "Protection de surface des constructions métalliques" (Surface Protection of Steel Structures) (1992),
- SZS C4.1 "Tables de dimensionnement pour la construction métallique" (Structural Design Tables for Steel Construction) (2006),
- SZS C5 "Tables de construction" (Design Tables) (2005).

Neither the documents listed above (SIA standards, Eurocodes, SZS guides), nor the other volumes of the TGC are cited in the bibliography at the end of each chapter. This is because they are fundamental documents related to the complete book. However, the lists at the end of each chapter do contain all publications referred to in the text of that chapter, in square brackets, according to the order they appear in the chapter.

1.3.2 Other References

Alongside the books, standards, recommendations and tables noted above, this book refers to a large number of key books and periodicals dedicated to steel construction, covering all aspects of analysis and design, from concept development through to detailed design. While not pretending that the list is exhaustive, it nevertheless covers the principal references concerning steel and composite (steel-concrete) bridges.

Key Books

Construire en acier 2 [1.5] Book presenting examples of bridges constructed both in Switzerland and abroad. Edited by SZS in Zurich, Switzerland.

Bridge Engineering Handbook [1.6] American book that synthesises experience in the construction of bridges.

*Construction métallique et mixte acier-béto*n, Vol. 1 et 2 [1.7] Books covering both steel and composite (steel-concrete) construction published under the banner of the association for the promotion and teaching of steel construction (APK) in France.

Steel Designers' Manual [1.8] British manual of steel construction, UK.

European Steel Design Education Programme (ESDEP) [1.9] Volumes 12 and 15 in a series of 29, based on the Eurocodes and published by the Steel Construction Institute (SCI), UK.

Handbuch Brücken [1.10] Book concerning the basis of the configuration, design and maintenance of bridges.

Publications of International Associations

IABSE: The International Association of Bridge and Structural Engineering periodically publishes a review, themed publications on structures, and conference proceedings (www.iabse.org).

ECCS: European Convention for Constructional Steelwork, Brussels, Belgium. They publish guides, recommendations and manuals concerning the design and construction of steel structures, promotional booklets, etc. (www.steelconstruct.com).

CIDECT: The International Committee for Research and Technical Support for Hollow Section Structures publish technical guidance concerning tubular steel structures (www.cidect.com).

OTUA: Paris, France. Periodical and themed publications cover different questions concerning steel and its uses (www.otua.org).

SCI: The Steel Construction Institute, Ascot, UK, delivers numerous publications on the design and construction of steel structures, architecture, codes and standards, etc. (www.steel-sci.org).

Stahlbau Kalender: This annual series of manuals (since 1999) presents new subjects associated with the design of steel structures. Ernst & Sohn, Berlin, Germany (www.ernst-und-sohn.de).

Periodicals

Advantage Steel, Canadian Institute of Steel Construction, Willowdale, Ontario, Canada (www.cisc- icca.ca).

Bauen in Stahl – Construire en acier – Costruire in acciaio, until 2003, then from 2004 *steeldoc*, Periodicals from the Swiss Centre for Steel Construction (SZS), Zurich, Switzerland (www.szs.ch).

Bridge Design & Engineering, Editorial Office, Huntingdon House, U.K. (www.bridgeweb.com).

Bulletin Ponts Métalliques, OTUA, Paris-la-Défense, France (www.otua.org).

Construction métallique, Centre technique industriel de la construction métallique (CTICM), Saint Aubin, France (www.cticm.com).
Costruzioni Metalliche, ACS-ACAI Servizi Srl, Milan, Italie (www.acaiacs.it).
Journal of Bridge Engineering, American Society of Civil Engineers (ASCE), Reston, VA, USA (www.asce.org).
Journal of Constructional Steel Research, Elsevier Sciences Ltd, Oxford, U.K. (www.elsevier.com).
Modern Steel Construction, American Institute of Steel Construction (AISC), Chicago, Ill, USA (www.aisc.com).
New Steel Construction, The Steel Construction Institute, Ascot, U. K., and The British Constructional Steelwork Association Ltd, London, U.K. (www.steelconstruction.com).
Structural Engineering International, SEI, journal of the International Association for Bridge and Structural Engineering, AIPC, Zurich, Suisse (www.iabse.org).
Stahlbau, Ernst & Sohn GmbH, Berlin, Germany (www.stahlbau.ernst-und-sohn.de).
Stahlbau-Rundschau, Österreichischer Stahlbauverband, Vienna, Austria (www.stahlbauverband.at).

1.4 Conventions

1.4.1 Terminology and Typography

In conformance with the conventions used in other volumes of the TGC, the following terminology and typographical styles have been adopted:

- This book is divided into chapters (Chap.), sections (Sect.), and paragraphs (§).
- Unless integrated within the text, equations are numbered continuously by chapter, and identified by brackets, for example (2.4). An exception is that equations taken from the Eurocodes are not numbered.
- Numbering of the figures (Fig.) and tables (Tab.) is continuous within each chapter. For example, Figure 1.2 follows Table 1.1, and Table 6.14 follows Figure 6.13.
- *Plain italics* within the text are used to highlight certain terms, or complementary information coming from the Eurocodes, for quotations, or to indicate non-English terms and expressions. An entire line in plain italics indicates information coming from the Eurocodes.
- ***Bold italics*** are used when a new term is cited for the first time: this allows the reader to easily identify where the term is defined. All terms given in bold italics are recalled in the index at the end of the book.

1.4.2 Axes

In general the convention used for the axes of structural elements and bridges themselves is as follows (Fig. 1.2):

- x axis: axis along the length of an element,
- y axis: cross section horizontal axis,
- z axis: cross section vertical axis.

Displacements in the direction of the x, y and z axes are noted as u, v and w, respectively. For cross sections the conventions are as follows:

- y axis: axis parallel to the flanges (or the short side for rectangular hollow sections) or the horizontal axis for bridge cross sections,

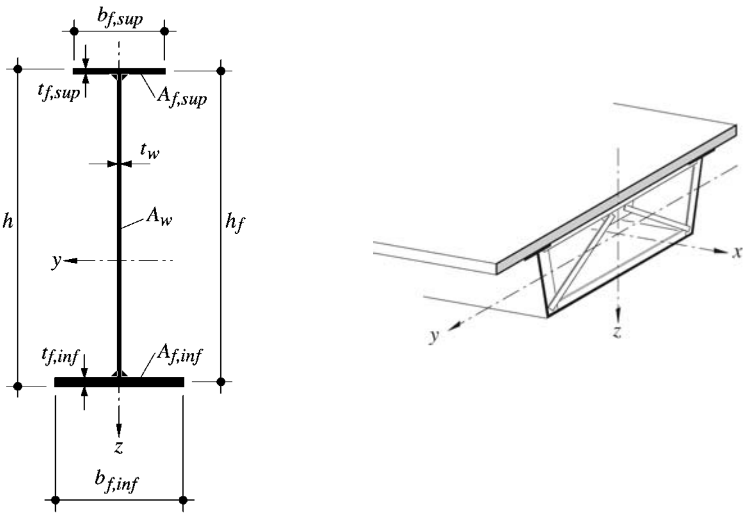

Fig. 1.2 Axis convention and notations for a plate girder cross section and a bridge.

- z axis: axis perpendicular to the flanges (or the short side for rectangular hollow sections) or the vertical axis for bridge cross sections.

For angles:

- y axis: axis parallel to the smaller flange,
- z axis: axis parallel to the larger flange,

and if necessary (for angles or other unsymmetrical sections):

- u axis: major axis, if it does not correspond to the y axis,
- v axis: minor axis, if it does not correspond to the z axis.

1.4.3 Notation and Signs

A detailed list of notations is given at the end of the book. It is generally in accordance with that of the Eurocodes, in SIA standards and SZS tables. The few exceptions are highlighted as such when they are used.

As far as signs are concerned, a tensile force is noted as positive, and compression is noted as negative. The sign convention for moments is in accordance with the normal rules for statics. However, these sign conventions should not be applied blindly in all cases, because, for example, some software uses different conventions. Also, certain interaction formulae require the use of absolute values for the forces. Such exceptions are highlighted in the text.

1.4.4 Units

This book is based on the (SI) international system of units, so the basic units are metres, kilograms and seconds. The Newton [N] is the unit of force corresponding to an acceleration of 1 m/s^2 applied to a mass of 1 kg. The following units are therefore used systematically throughout this book:

- length : millimetres (mm) or metres (m),
- concentrated load: kilo-Newtons (kN) or Newtons (N),
- distributed load : kilo-Newtons per square metre (kN/m^2) or kilo-Newtons per linear metre (kN/m),
- stress : Newtons per square millimetre (N/mm^2).

To avoid confusion over units, they are explicitly given in all the numerical examples. This approach has the added advantage of encouraging a certain rigour with calculations, which helps avoid errors of an order of magnitude in calculated results.

References

[1.1] ICOM, *Conception des structures métalliques/Partie A: Notions fondamentales et dimensionnement des éléments de construction métallique*, EPFL, ICOM-Construction métallique, Lausanne, 1987 (2e édition).

[1.2] ICOM, *Conception des structures métalliques/Partie C: Dimensionnement des halles et bâtiments*, EPFL, ICOM-Construction métallique, Lausanne, 1987 (2e édition).

[1.3] ICOM, *Conception des structures métalliques/Partie D: Dimensionnement des ponts*, EPFL, ICOM-Construction métallique, Lausanne, 1982 (2e édition).

[1.4] ICOM, *Conception des structures métalliques/Partie E: Dimensionnement plastique des ossatures*, EPFL, ICOM-Construction métallique, Lausanne, 1978.

[1.5] *Construire en acier 2*, Chambre suisse de la construction métallique, Zurich, 1962.

[1.6] CHEN, W.-F., DUAN, L., *Bridge Engineering Handbook*, edited by Chen, W.-F. and Duan, L., CRC Press, Boca Raton, FL, USA, 2000.

[1.7] APK, *Construction métallique et mixte acier-béton*, vol. 1: *Calcul et dimensionnement selon les Eurocodes 3 et 4*, vol. 2: *Conception et mise en œuvre*, Eyrolles, Paris, 1996.

[1.8] *Steel Designers' Manual* – 6th Edition, Steel Construction Institute, Ascot and Blackwell Science, Oxford, 2003.

[1.9] *European Steel Design Education Programme (ESDEP)*, 29 volumes, The Steel Construction Institute, Ascot (UK), 1995.

[1.10] MEHLHORN, G., *Handbuch Brücken; Entwerfen, Konstruieren, Berechnen, Bauen und Erhalten*, Springer Verlag, Berlin, 2007.

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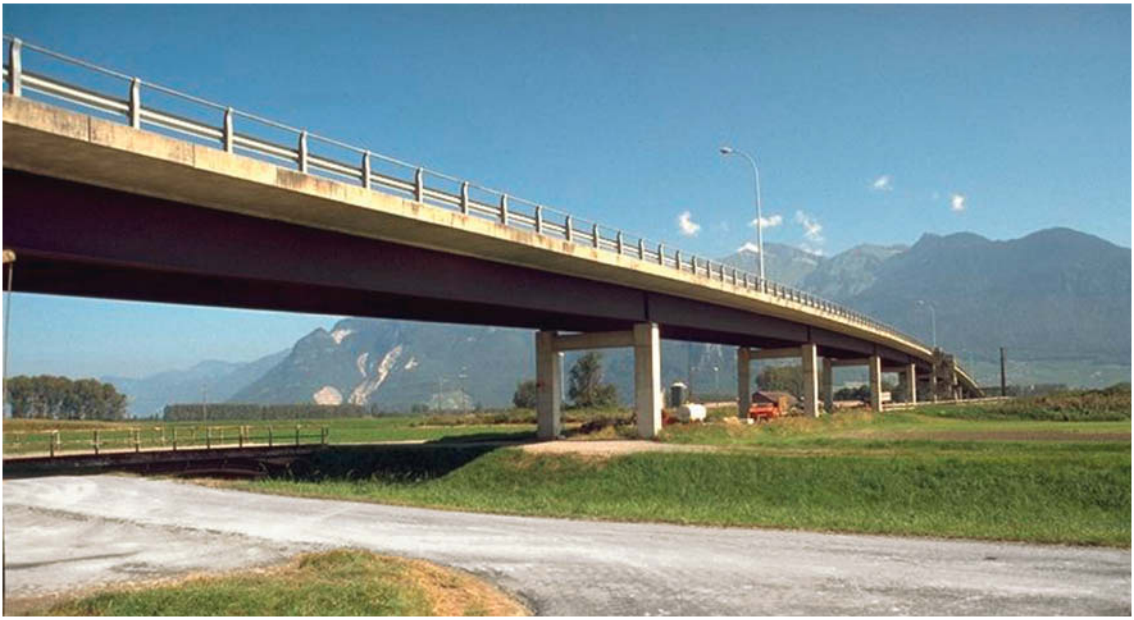
2 Bridge Description



Access viaduct to the freeway junction at Aigle (CH).
Eng. Piguet & Associés, ingénieurs-conseils SA, Lausanne.
Photo ICOM.

2.1 Introduction

A bridge is a spatial object whose purpose is to cross an obstacle (valley, water, road) with a communication route. Consequently a bridge should be able to transfer to its foundation the actions applied to it. In order to achieve this, the structure of a bridge will depend on the size of the obstacle to be crossed as well as the types and magnitudes of the actions. Given these factors, a number of options are available for the configuration of a bridge. This chapter describes both various types of bridges in terms of several classification criteria and the key structural elements that help a bridge satisfy its function. This chapter therefore aims to

- familiarise the reader with bridges and their terminology,
- characterise bridges according to certain classification criteria,
- describe the elements of a bridge and detail their functions. This also helps explain load paths from the point of application down to the foundations,
- summarise the other components of a bridge, identifying their functions and configuration.

This chapter therefore contains the basics necessary for a reader to tackle the chapters on analysis and design. It offers a complete vision of a bridge's characteristics and functions and enables subsequent detailed study of each element of a beam bridge, such as the slab, the main beams, the cross-bracing, or indeed the supports.

2.2 Classification Criteria for Bridges

Numerous criteria can be used to distinguish bridges. These criteria are important for the design engineer when deciding on appropriate hazard scenarios, load cases, design assumptions or calculation models as a function of the type of bridge. The classification is therefore more than merely of didactic interest; it is a genuine work and communication tool for the engineer. Principal criteria for the classification of a bridge are as follows:

- type of use,
- geometry,
- structural form,
- type of slab,
- cross section,
- slab position,
- erection method for the steel structure,
- slab construction.

2.2.1 Type of Use

Classification according to use includes primarily the following:

- road bridges,
- railway bridges,
- bridges for pedestrians and cyclists.

Within the category of ***road bridges*** one can also distinguish between freeway (motorway, autoroute) bridges and those used for regional and local roads. In Switzerland, for example, freeway bridges normally comprise two separate structures, each supporting traffic in one direction. Bridges on regional and local roads normally support traffic in two directions.

Railway bridges support either lines that are normal or small-gauge, or special lines, such as those for rack railways, funicular railways, or tramways. The principles for conceptual design, analysis and detailed design that are detailed in this book are, in general, valid for both road and railway bridges. However, supplementary rules specific to railway bridges are given in Chapter 16. Some bridges are planned to carry a mixture of both road and rail traffic. This is achieved either by separating the traffic onto two levels (Fig. 2.1), by dividing the deck laterally, or dividing the traffic according to both methods for major urban bridges carrying several types of traffic.

Bridges for pedestrians and cyclists, also simply called, *footbridges*, are a particular type of bridge for which the imposed loads are small compared with those acting on a road bridge. This peculiarity grants the designer great freedom with architectural and structural form. Chapter 17 deals specifically with this type of bridge.

A number of other types of bridges exist, such as those for waterways (including aqueducts) or bridges to uniquely support pipework. However, since such types are less common, they are not considered explicitly in this book.

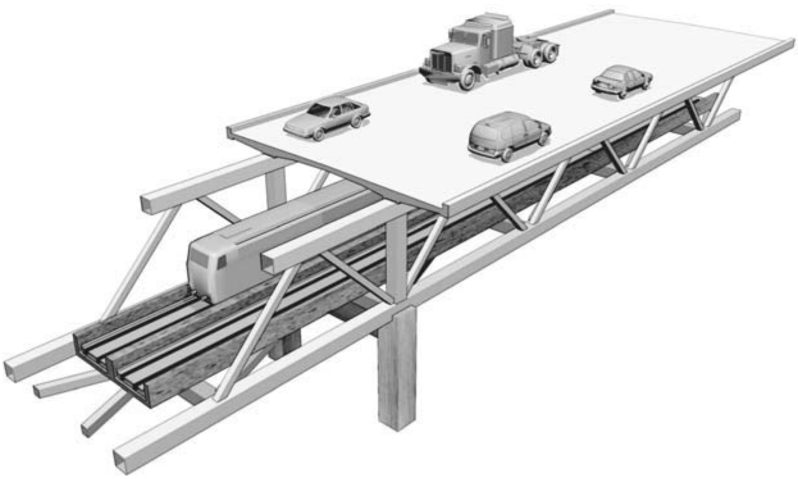


Fig. 2.1 Schematic of a bridge for mixed rail and road traffic use.

2.2.2 Geometry

Classification according to the plan of a bridge and the alignment of the supports allows distinction between three types of geometry, as shown in Figure 2.2:

- straight bridges,
- curved bridges,
- skew bridges.

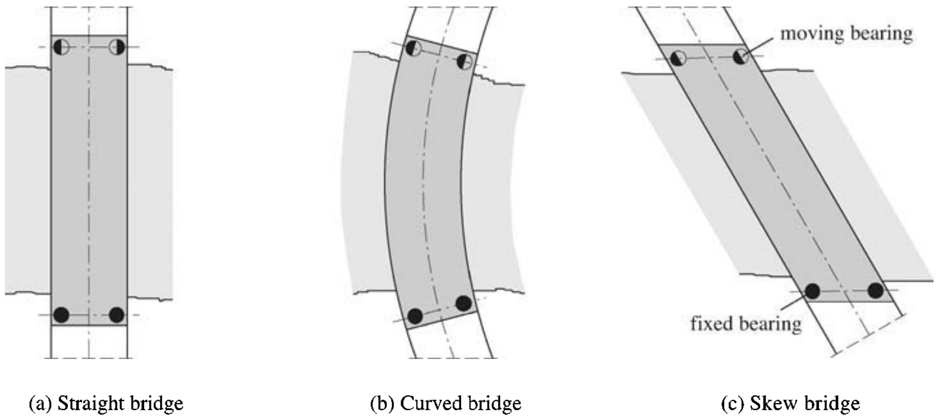


Fig. 2.2 Geometry on plan of a bridge.

The current trend for the configuration of bridges is to align their axes with the direction of the communication route they are carrying. As a result of this, bridges that are curved both in plan and in elevation are becoming more and more common. Increasingly, bridges cross a watercourse or a route at an angle, rather than adopting the more traditional option of being perpendicular.

A ***straight bridge***, viewed on plan, appears to be rectangular in form. Straight bridges are often the most economical and easiest to design, detail, and erect.

A ***curved bridge*** is constructed according to a curvilinear axis. It is necessary to precisely distinguish the various radii of curvature in both the horizontal and vertical planes. However, the influence of curvature in the vertical plane on the behaviour of the bridge is ignored in all but a few cases. Curvature may nevertheless complicate the fabrication of the bridge elements. In some cases with a curved bridge deck, the supporting elements (main beams) comprise a series of straight beams connected end-to-end to form a faceted curve.

A ***skew bridge***, be it straight or curved, is distinguished by having one or more lines of supports that are not perpendicular to the axis of the bridge.

2.2.3 Structural Form

A common classification is based on the structural form chosen for the bridge. The following four main types may be identified:

- beam bridges (using rolled sections, plate girders, trusses or box girders),
- arch bridges,
- inclined leg bridges,
- bridges supported by cables.

A ***beam bridge*** transfers vertical loads to the supports by bending the beams. This type of bridge is most often used for short and medium spans and, for the latter, composite steel-concrete construction is generally adopted. Beam bridges are the most economical, but the self-weight limits the span to around 150 m for railway bridges and 300 m for road bridges using a box girder. These span limits are more or less halved if the bridge comprises two plate girders. Trusses allow greater spans – up to around 500 m – thanks to a more efficient use of the steel, although the beam depth does increase. Large trusses are no longer considered to aesthetically pleasing, particularly in an urban setting.

Within the category of ***arch bridges***, one distinguishes further according to the structural form:

- three-pin arches,
- arches that are built-in at both ends, with or without a pin at the top of the arch,
- arches that are pinned at both ends, with or without a tie.

Although three-pin arches are indispensible in situations where support settlement is a problem, they are rarely used for bridges, because it is difficult to achieve a hinge at the top of the arch, and maintenance is costly. Arches that are built-in at both ends require very good ground for the foundations (rock), and this often limits their use to mountainous regions. Therefore, arches that are pinned at both ends are the most common; they can achieve spans of roughly 200 m with plate girders beams, or 550 m with trusses.

The structural principle behind arches is to transfer loads to the foundations by compression of the arches. Resisting the outward thrust at the base of the arch requires sound foundations. Tied arches (Fig. 2.3(a)) are an interesting option that carries the horizontal thrust of the arches in cases where ground conditions are inadequate. The tie normally comprises the bridge deck. Arch bridges are considered in more detail in Chapter 18.

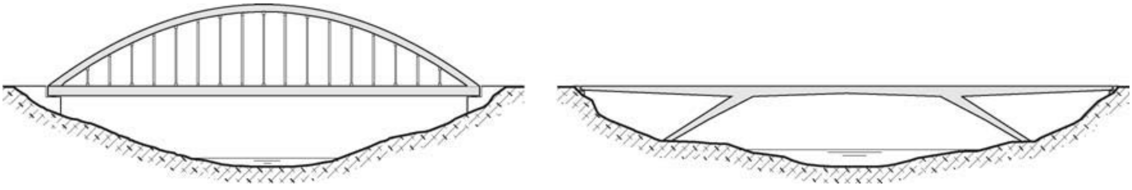
Inclined leg bridges (Fig. 2.3(b)) in some ways combine beam bridges and arch bridges, because they transfer loads to the foundations via a combination of bending and compression. In addition to bending, the central part of the span and the inclined legs are compressed as an arch would be. Thereby the foundations are subject to both horizontal and vertical forces. The two end-spans are beams subjected simply to bending.

Bridges supported by cables comprise:

- suspension bridges,
- cable stayed bridges.

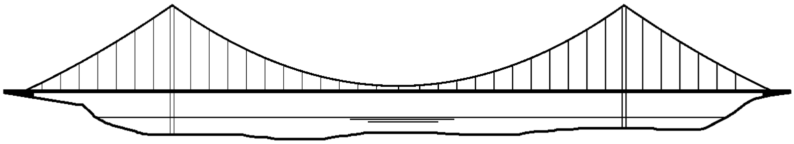
Suspension bridges (Fig. 2.4(a)) can achieve very long spans (for example, the Akashi Kaikyo bridge in Japan has a free span between pylons of 1991 m). The deck is suspended, via vertical cables, from one or several main cables, which that have a parabolic form. These main cables are supported by two pylons and are generally anchored in massive anchors at the abutments or, sometimes, in the deck of the bridge itself. Steel is very well suited to the construction of suspension bridges because it minimizes the self-weight of the structure. Suspension bridges with the longest spans all possess a steel deck (orthotropic deck).

Cable stayed bridges (Fig. 2.4(b)) fall into a number of sub-categories according to the form of their cables: harp, semi-harp or fan (Fig. 2.5). The horizontal component of the tension in the cables imposes

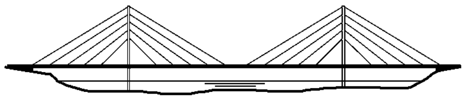


(a) Tied arch bridge (b) Inclined leg bridge

Fig. 2.3 Tied arch bridge and inclined leg bridge.

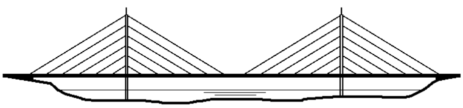


(a) Suspension bridge

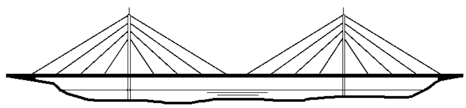


(b) Cable stayed bridge

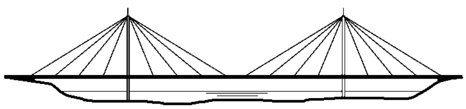
Fig. 2.4 Bridges with suspension cables.



Harp



Semi-harp



Fan

Fig. 2.5 Arrangement of cable stays.

substantial compression in the deck. Main spans between the pylons can approach 1100 m (for example, the Sutong Bridge in China has a free span between pylons of 1088 m).

The detailed study of bridges supported by cables goes beyond the scope of this book. Several books are dedicated exclusively to the configuration and design of cable-supported bridges, particularly [2.1] for suspension bridges and [2.2] for cable-stayed bridges.

2.2.4 Type of Slab

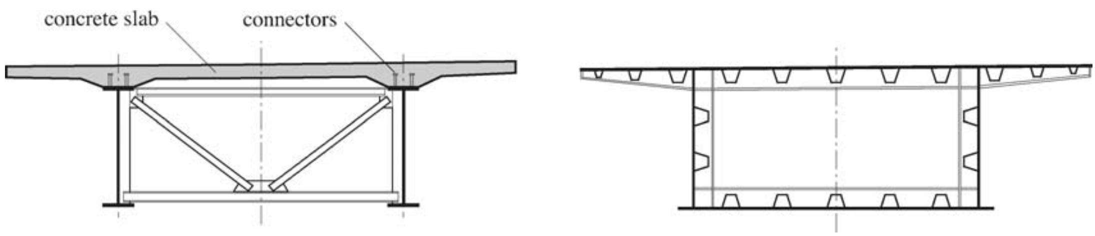
There are three main categories of slab:

- concrete slab connected to the steel structure,
- concrete slab not connected to the steel structure,
- orthotropic steel plates forming the slab.

This book focuses on steel-concrete composite bridges, for which the concrete slab is structurally connected to the steel structure that comprises the main beams (Fig. 2.6(a)). This connection means that the slab contributes to the bending resistance of the beams and the torsional resistance of the bridge.

Bridges in which the concrete slab is not connected to the steel beams use it simply as a rolling surface and to carry loads locally. The design of the steel beams should be carried out according to the guidance given in Chapters 11 and 12 of this volume. The design of the concrete slab itself, when not used as part of a composite structure, is treated in the TGC, volume 9.

Orthotropic decks (Fig. 2.6(b)) comprise steel plates stiffened both longitudinally and transversally at regular intervals on their lower surface. The stiffened plates are welded to the main beams, generally to form the upper flange. This type of slab differs from those in concrete in terms of its low self-weight and high fabrication cost. The use of orthotropic plates is therefore justified when self-weight becomes important relative to the imposed loads, which is the case for very long span bridges. It can also be justified when considering replacement of the concrete slab on an existing bridge in order to permit the structure to carry higher traffic loads. However, orthotropic slabs are susceptible to fatigue caused by direct application of concentrated loads (truck wheels) to the slab. Guidance on the conceptual design of orthotropic slabs is given in Section 6.7.



(a) Twin girder bridge with concrete slab (b) Box girder bridge with orthotropic steel deck

Fig. 2.6 Examples of bridge decks.

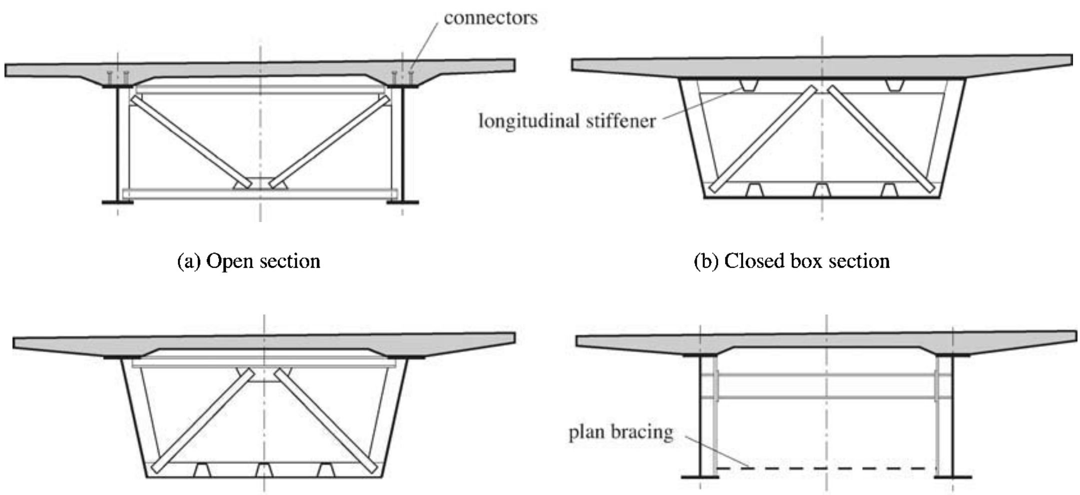
2.2.5 Cross Section

Classification according to the cross section type is critical to describing the torsional behaviour of the bridge. There are two basic types:

- open cross section,
- closed cross section.

Open cross sections (Fig. 2.7(a)) most often simply comprise either two main beams (twin girder) or a number of main beams (multi-girder). ***Closed cross sections*** may comprise a box that is completely made of steel (Fig. 2.7(b)), a steel U shaped section (Fig. 2.7(c)) or a twin girder section closed by lower plan bracing (Fig. 2.7(d)). In the latter two cases, the slab is connected to the steel section to form a closed cross section.

The distinction between open and closed cross sections is particularly useful when considering the way in which the bridge will resist torsion. This is considered in detail in Chapter 11. Open cross sections essentially resist in non-uniform (warping) torsion and have little torsional rigidity. Closed sections essentially resist in uniform (St-Venant) torsion and deform very little. A closed cross section is therefore advantageous for bridges subjected to significant torsion, for example curved bridges or bridges with substantial cantilevers to the slab.



(a) Open section (b) Closed box section

(c) U shaped steel section (d) Open section, closed by lower level plan bracing

Fig. 2.7 Types of cross section.

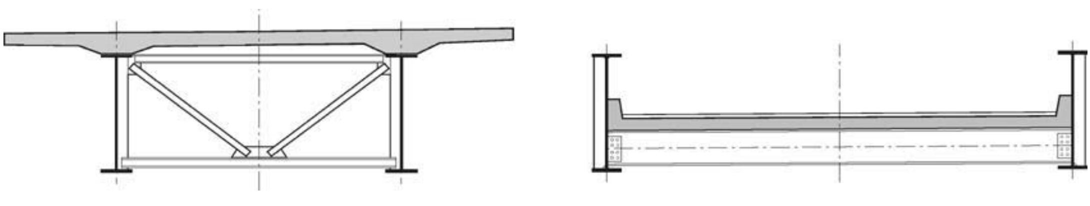
2.2.6 Slab Position

The slab may be positioned at different heights relative to the main beams. There are two basic choices:

- upper slab,
- lower slab.

The adoption of an ***upper slab*** (Fig. 2.8(a)) is currently the most typical choice. With such a configuration, the slab protects the supporting steel structure from the weather as well as, potentially, from vehicle impact. A lower slab cannot achieve this. An upper slab also provides the potential in the future to widen the rolling surface. For a given total width, the span of the slab between the main beams is less for an upper slab, which may therefore be thinner. In addition there is no need to introduce transverse secondary beams to transfer the loads, through bending, to the main beams. An upper slab is therefore the most economical solution for wide slabs (i.e., three or more lanes of traffic).

However, the total depth of the bridge deck is greater with an upper slab. When this depth must be limited, it is wise to use a ***lower slab*** (Fig. 2.8(b)), or a cross section with multi-beams or a box girder. When a



(a) Upper slab (b) Lower slab

Fig. 2.8 Position of the slab.

lower slab is used, the relative positions of the main beams may render noise barriers unnecessary. A lower slab is often used for cable supported or arch bridges, because it facilitates direct anchorage of the hangers or cables to the main beams.

2.2.7 Erection of the Steel Structure

Three main methods are used for the erection of the steel structure, namely:

- erection by crane from the ground,
- cantilever erection,
- erection by launching.

Other methods for erecting the steel structure do exist, but they are more specific than the three noted above. Examples exist of complete bridges that have been put into position from a watercourse using barges, or complete bridges that have been placed by either transverse sliding or rotation around an abutment. These methods are most frequently used when replacing an existing structure, when it is necessary to minimise the time that traffic flow is disrupted. The construction of the bridge from its constituent elements can take place without disturbing the traffic, and only the final positioning of the completed structure causes disturbance. Methods of erection for the steel structure are discussed in more detail in Section 7.5.

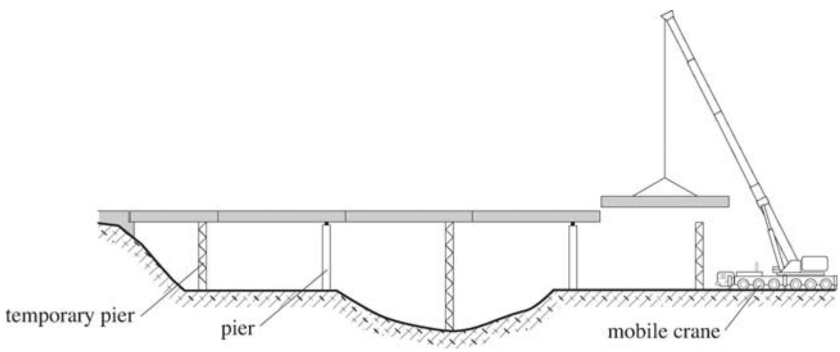


Fig. 2.9 Erection using a crane at ground level.

Erection by ***using a crane*** at ground level (Fig. 2.9) is suitable when the height of the bridge relative to the ground level is small (less than 15 m). Individual bridge elements are successively lifted using a mobile crane or a forklift, and are mounted on either the piers or on temporary supports. The elements are then welded to those already in place. The construction joints in the structure should be located away from the supports, in regions where stresses are lower.

Cantilever erection (Fig. 2.10) is used when the height of the bridge relative to the ground level is substantial, or when access to the valley floor is limited and prohibits erection by crane from ground level. Elements of the steel structure are positioned length by length, cantilevering out towards the next support. To avoid excessive stresses on the cantilever, it may be necessary to use temporary piers (assuming the height of the bridge above the ground is not too great). Incremental erection is often specified for bridges crossing an important watercourse, and is well suited to cable-stayed bridges.

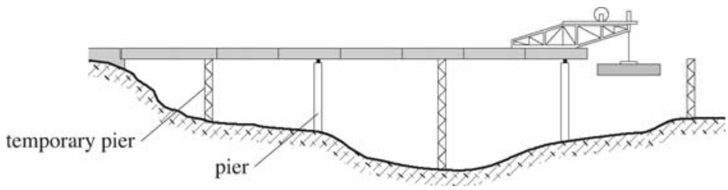


Fig. 2.10 Cantilever erection.

Erection by launching (Fig. 2.11) is used when conditions are similar to those that make cantilever erection attractive. Individual elements of the steel structure are joined in an area at one of the bridge ends and in line with the bridge axis. The completed subassembly is then progressively pushed or pulled into position from the abutments. To minimize potential problems during launching that result from the increasing cantilever, a lightweight nose section is often used to reduce the weight of the cantilever.

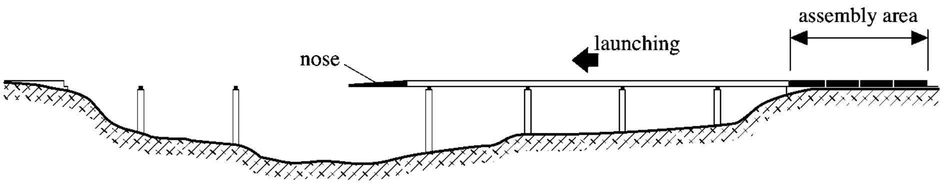


Fig. 2.11 Erection by launching.

2.2.8 Slab Construction

When a concrete slab is used, there are three basic methods to construct it on top of the steel beams, namely:

- slab cast in-situ,
- slab launched in stages,
- slab comprising precast elements.

For a ***slab cast in-situ***, the wet concrete is placed on either fixed formwork (fixed to the main beams) or moving formwork (a trolley that moves along the beams). During this operation the steel structure of beam bridges may be supported by temporary falsework (propped construction) or not (unpropped).

For a slab that is ***slid into place***, lengths of concrete slab are prefabricated in an area in line with the bridge axis. These lengths are then pushed along the structure into position on the steel beams. This method of slab construction is analogous to erection of the steel structure by launching.

For a slab ***comprising precast elements***, these elements are located on the steel structure one after the other, advancing from the end support(s). The precast elements are placed either by a crane on ground level or by a forklift on the completed parts of the bridge.

2.3 Structural Elements

This section describes the main structural elements used in a bridge and explains their function. First, it is necessary to distinguish between the components of the substructure and those of the superstructure. The substructure comprises those elements that support the bridge, such as the piers, abutments and foundations (Fig. 2.12). Consideration of these elements, constructed mainly from concrete, is not the subject of this book. The other structural elements form the superstructure. In the case of beam bridges, the junction between the substructure and the superstructure is at the level of the bearings.

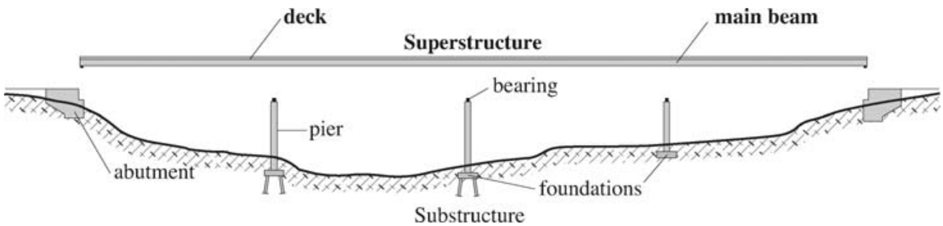


Fig. 2.12 Structural elements (substructure and superstructure).

2.3.1 Superstructure

Figure 2.13 shows schematically the different structural elements that make up the superstructure of a twin girder bridge. Individual elements are the ***slab***, the ***main beams*** with their ***shear connectors,*** and the ***cross bracing***. The ***plan bracing*** is also part of the superstructure; for the example in Figure 2.13, the slab, connected to the main beams, plays the role of plan bracing. It is worth noting that the slab and main beams taken together are also called the bridge ***deck***. The following paragraphs briefly describe these structural elements and their functions. For arch bridges the arches themselves with the hangers form part of the superstructure. The same can be said for bridges supported by cables, for which the pylons, main cables and hangers all form part of the superstructure.

An essential function of the *slab* is to transmit the traffic loads to the primary structural elements of the bridge. In Switzerland, for example, the slab is normally formed from reinforced concrete, which is sometimes prestressed either transversally or longitudinally. In Switzerland the slab is rarely made of steel. When structurally connected to the steel beams, the slab contributes to the overall behaviour of the bridge and may, according to the bridge design concept, also serve as plan bracing. Within the spans an upper slab – and at the intermediate supports, a lower slab – also serves as lateral support for the compression flanges

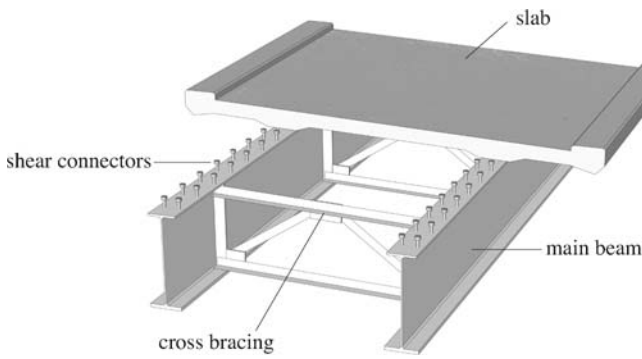


Fig. 2.13 Schematic view of the elements forming the superstructure of a composite twin girder bridge.

of the main beams, and in so doing, increases their lateral torsional buckling resistance. Slabs of composite beams are considered in more detail in Chapter 8.

The *main beams* are the longitudinal structural elements of the bridge. They transfer the loads coming from the slab to the supports by bending, by shear, and by torsion. The main beams may be rolled sections (used for example in multi-beam bridges of short span), plate girders, or trusses. Fabricated beams – be they I-section plate girders or box girders – must be stiffened to avoid buckling of the slender plates from which they are formed. Construction details for beams are considered in Chapter 5; design checks are given in Chapter 12 for steel beams and in Chapter 13 for composite (steel-concrete) beams.

The *cross bracing* is formed from planar elements perpendicular to the bridge axis which tie the main beams together. They fulfil two basic functions:

- prevent deformation of the bridge cross section,
- transfer the horizontal forces which act on the main beams (due to wind, effects of curvature) to the plan bracing.

The cross bracing (Fig. 2.14) may consist of a steel plate (diaphragm), a truss, or a frame. The functions of the cross bracing, the forces acting on it, and the erection requirements will dictate the cross bracing form to be adopted for a given bridge. Cross bracing is considered in more detail in Sections 5.6 (function, different types), 6.4 (construction details), and 14.3 (forces acting on the cross bracing).

In a composite steel-concrete bridge, the concrete slab is connected to the steel beams in such a way that the two act together to resist the actions. The *shear connectors* (Fig. 2.13) form this structural connection between the steel and concrete. Headed studs are the most common form of shear connector. Connection between the slab and beams is considered in Paragraph 8.3.3 as well as in Section 13.5.

The *plan bracing* ensures the lateral behaviour of the bridge by stiffening the primary structure in the horizontal plane. It transfers to the supports horizontal forces, primarily due to wind. The plan bracing normally comprises a truss formed using the main beams. For composite bridges, where the slab is structurally connected to the steel beams, the slab itself performs this function. During construction it may be

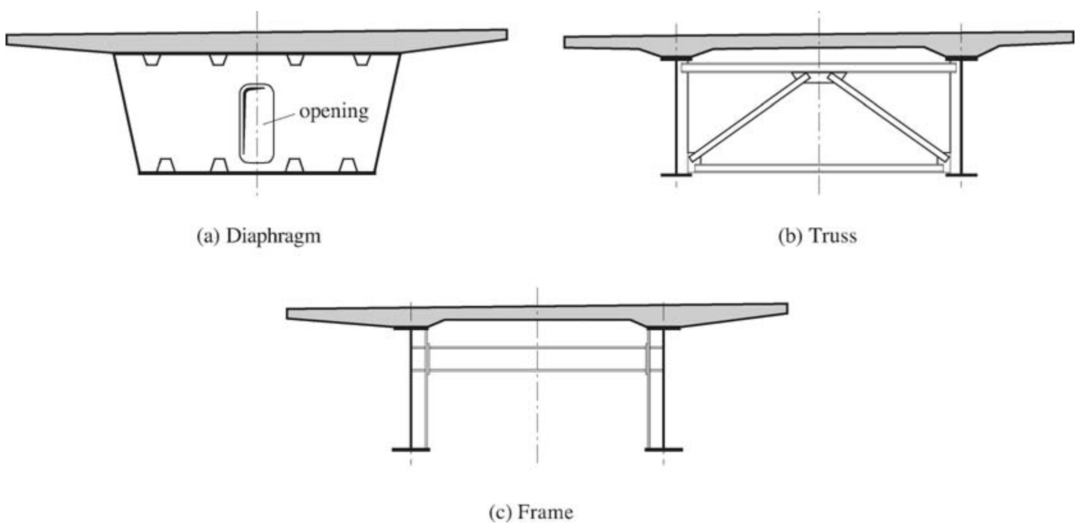


Fig. 2.14 Types of cross bracing.

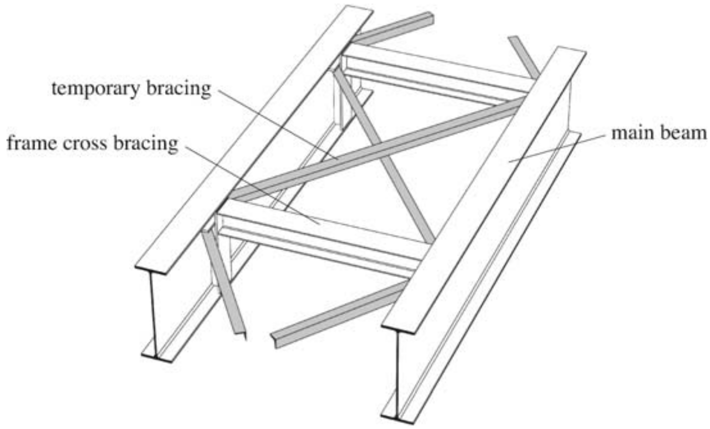


Fig. 2.15 Plan bracing for erection.

necessary to use temporary bracing to assure stability, as shown in Figure 2.15. Plan bracing is considered in more detail in Sections 5.7 (function, different types), 6.5 (construction details), and 14.4 (forces acting on the plan bracing).

2.3.2 Substructure

The substructure (Fig. 2.12) comprises the ***piers***, the ***abutments*** and the ***foundations***. These elements support the superstructure and transfer the actions down to the ground.

In Switzerland the *piers* are normally formed from reinforced concrete, which is sometimes prestressed, and less often from steel, which is used, for example, with inclined-leg bridges. The piers may be pinned at their bases but are more often built-in, and either free or pinned at their tops. The tops of the piers are rarely built-in with steel bridges. The choice of pier form, and the choice of their end conditions depend, amongst other things, on: the ground conditions, the overall stability of the bridge (Chapter 15), the dimensions of the piers, and the forces to which they are subjected.

The *abutments* are generally formed from reinforced concrete. These elements are located at the ends of the bridge; they assure its integration with the surrounding terrain. In some cases the abutments need to support the embankments and protect them from water, for example in the case of a bridge passing over a river. A concrete transition slab is normally supported by the abutment and extends several meters (typically 3 to 8 m) under the rolling surface. This slab allows a progressive transition between the road and the abutment in cases where there is settlement of the road. More detailed consideration of when a transition slab should be used, as well as guidance on their design, is given in [2.3]. The abutments are subject to soil and water pressures and vertical loading from the deck (support reaction). If the abutment forms a fixed support for the bridge, it will also be subject to horizontal actions (due to braking forces, wind, seismic actions, movements within the span).

The *foundations* transfer to the ground – by compression and by friction – the forces coming from the piers and abutments. One can distinguish between shallow (raft) and deep (piles, floating piles, caisson) foundations. Given the substantial loads that act on the piers, shallow foundations can only be considered in cases where the ground conditions are excellent, such as rock or very compact ground. Exceptions are relatively unimportant bridges, such as overpasses and footbridges.

2.4 Other Components

Other components necessary to the proper functioning of a bridge are the ***bearings***, ***expansion joints***, and ***water evacuation system***. The following paragraphs provide information regarding the function, configuration, and durability of these components. Further information, regularly updated to reflect current Swiss practice, is available in the document, *Details of Bridge Construction*, edited by the Federal Roads Office [2.3]. Additional information and references concerning these components is given in Section 6.8.

2.4.1 Bearings

The bearings are found at the interface between the superstructure and the substructure. They must transfer the vertical and horizontal forces from the superstructure to the piers and abutments while allowing the necessary movements of the superstructure. This double function is normally achieved using bearings designed and detailed to permit the envisaged movements (translation and/or rotation). Figure 2.16 shows an example of a bridge bearing designed both to allow longitudinal movement and provide fixity against transverse movement.

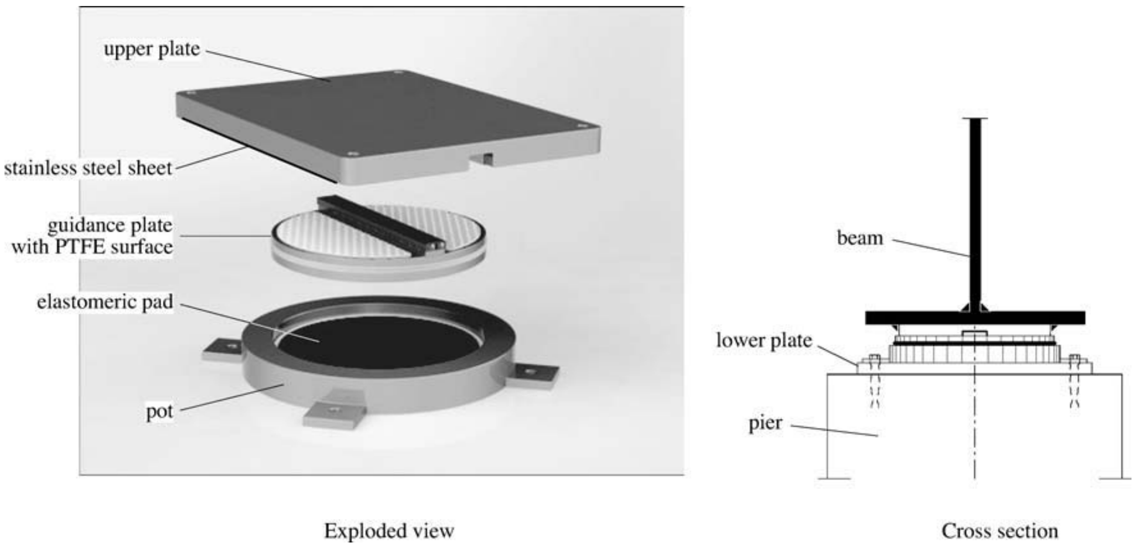


Fig. 2.16 Example of a "pot" bearing – free to move longitudinally but fixed laterally.

Bearings, particularly those allowing movements, generally have a design life that is shorter than that of the structure as a whole. It is necessary to check them regularly, to provide the necessary maintenance, and if necessary, to replace them. Bearings are particularly susceptible to the presence of water (standing water or water ingress). Failure of a bearing can result in harmful loading of the superstructure as well as the substructure. This is why it is necessary to consider, as part of the conceptual design of the bridge, both a means of easy replacement of the bearings as well as a maintenance programme for early identification of any deterioration.

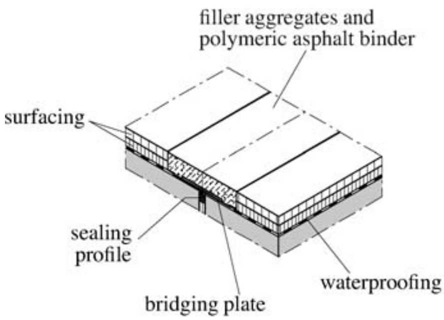
The bearings may be fixed – in which case they transfer horizontal forces to the substructure without any relative movement of the superstructure – or moving – allowing relative movement in the longitudinal and/or transverse directions. The choice of either fixed or moving bearings depends on the structural form of the superstructure conceived by the bridge designer. It is important to note that moving bearings do transmit some horizontal force, due to either friction or the stiffness of the elastomeric type. Forces due to friction and stiffness of bearings are considered in Section 10.7.

2.4.2 Road and Expansion Joints

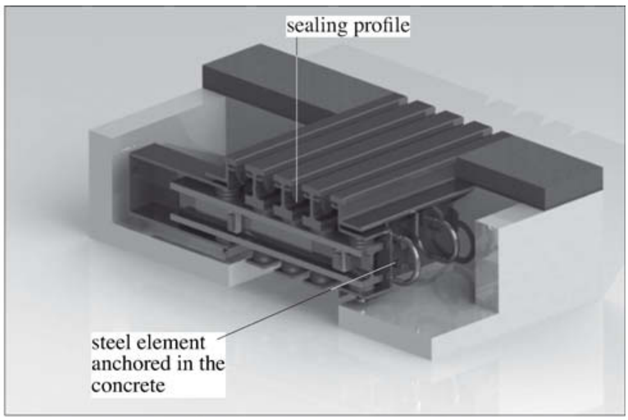
The road and expansion joints assure the continuity of the rolling surface between the deck and abutments, or between two separate parts of the deck. Above all they allow movement of the superstructure relative to the substructure. Examples include variations in length due to changes in temperature, or rotations that result from loads applied to the deck. The joints must also be able to support the vertical loads from the traffic.

The road and expansion joints can be conceived in a number of ways, according to the magnitude of the movements they must accommodate. Two specific groups are:

- joints formed from polymer modified bitumen (Fig 2.17(a)) used for small movements (expansion: 20 mm, shortening: 10mm);
- joints that include steel elements anchored in the concrete of the slab and the abutments. Figure 2.17(b) shows an example with compressible sections. This type of joint allows greater movements (up to ±1200 mm) and is generically known as an ***expansion joint***.



(a) Flexible road joint with polymeric asphalt
Movement : + 20 mm, – 10 mm



(b) Modular expansion joint with a number (n) of individual gaps, each with an elastomeric sealing profile
Mouvement : ± n × 40 mm

Fig. 2.17 Examples of a road joint.

The road and expansion joints are important for the correct functioning of the bridge; consequently, they require appropriate maintenance. They are subjected to wear and fatigue, mainly due to the effects of traffic (wheel actions), which means they have a limited design life. Their replacement is expensive, particularly for expansion joints. The damage that may be caused by their malfunctioning, for example by water that can then reach the bearings and/or supporting structure, is significant. Hence, the current trend is to reduce the number of expansion joints for a bridge.

The need for road and expansion joints depends above all on the distance between the fixed point on a bridge (§ 5.3.4) and its moving ends, and on the amount of traffic that is envisaged. When the distance is short, one can conceive a bridge with no joints at all and thereby reduce maintenance costs [2.4]. This solution is relatively recent; therefore, it is necessary to wait a few years to be certain that the stresses caused by the restrained movements within the span do not result in damage (cracking of the slab leading to corrosion of the reinforcement) that negates the benefits of avoiding joints. In addition, it is necessary to give extra consideration to the interaction between the structure and its foundations. For longer spans, bridge sections of 600 to 1000 meters have been built without intermediate joints.

2.4.3 Water Evacuation

To assure the durability of the bridge, an efficient and reliable way to evacuate rainwater must be conceived. Standing water on the rolling surface – as well as being dangerous for traffic (aquaplaning, increased braking distance, slippery surface) – can accelerate structural degradation. Defects or damage to the waterproofing layer present beneath the surfacing can lead to damage of the concrete due to either freeze-thaw action or chlorides in the water (for example de-icing salt). Standing water on the steel elements can also lead to corrosion.

It is therefore necessary to conceive a complete system for water evacuation (Fig. 2.18), considering both transverse and longitudinal directions. The transverse and longitudinal slopes on the surfacing, as well as the details of the system for water evacuation, must prevent local standing water. The evacuation system must also collect possible pollutant liquids rather than dispersing them into the local environment, for example an accidental spill of hydrocarbons on the bridge.

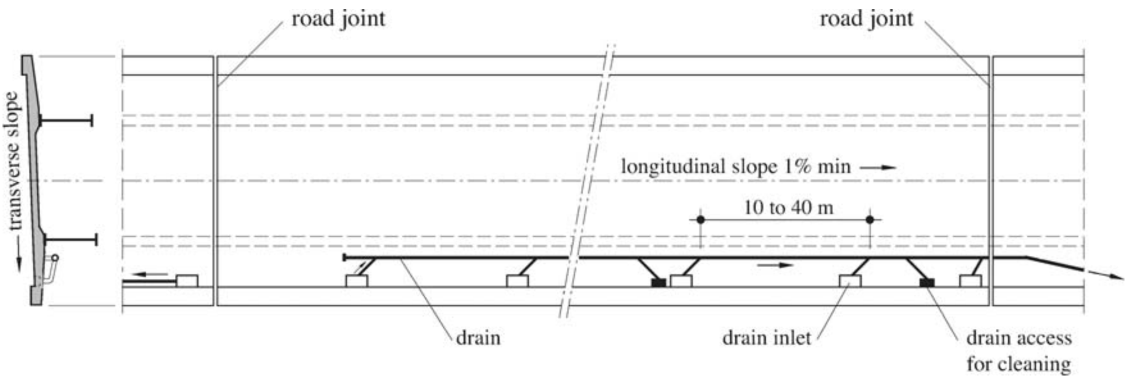


Fig. 2.18 Water evacuation system.

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3 History of Steel and Composite Bridges

Rail bridge over the Rhine between Koblenz (CH) and Waldshut (D), (1859).
Eng. Robert Gerwig.
Photo ICOM.

3.1 Introduction

This chapter directs the reader's attention to the numerous possibilities for steel bridges, and generates further interest with photographs of structures of note because:

- they represent a key stage in the development of bridge technology, either in terms of their configuration or their construction,
- they hold a record for length or span (at the time of writing),
- they are representative of steel or composite construction in a specific region.

The illustrations are presented in a sequence that follows the chronological development of steel bridges. However, it should be recognised that it is not possible, in a book of this type, to present all noteworthy bridges; therefore, the selection is not exhaustive. Readers seeking further information on the development of bridges should refer to the numerous books that exist on the subject, some of the most representative of which are noted at the end of this chapter.

3.2 History of Bridge Construction

This history of bridge construction, as well as the information in Section 3.3, is derived from the bibliography, internet sites on the subject, and personal visits.

The first bridges with a structure of load-bearing iron elements were suspension bridges. The idea of a suspended bridge is extremely old; the first known footbridge, using suspension chains, was constructed in China around 65 A.D. However, until the end of the 18th century, very few bridges were constructed from metal. The weight of the chains limited spans to around 20 m, and only the invention of chains made of articulated iron bars, known as eye bars and patented in England in 1817, allowed spans to substantially increase. The first important suspension bridge by the Englishman, Telford, was constructed over the Menai Straits (Fig. 3.1) in 1826 and is noteworthy for achieving a record span of 176 m. This bridge is still in service, the original iron chains having been replaced by articulated steel bars in 1938.

Fig. 3.1 Menai Straits Bridge (UK). Engineer Thomas Telford (Photo Mike Knapton).

Towards the end of the 18th century, in 1779 to be precise, the first cast iron bridge appeared over the Severn at Coalbrookdale in England. Conceived and constructed by a blacksmith named Abraham Darby, the bridge comprises five arches of 30 m span (Fig. 3.2). Other cast iron arch bridges were constructed at the end of the 18th century and the beginning of the 19th century, such as the bridge at Sunderland (Great Britain) with a span of 72 m (1796).

Fig. 3.2 Coalbrookdale Bridge (UK). Engineer Abraham Darby (Photo Kentaro Yamada).

Parallel with these developments suspension bridges progressed with the adoption of cables to replace chains, once and for all. This solution is credited to a Frenchman, Seguin. The first use of cables dates back to 1816, by an Englishman, Rees, while the Swiss, Dufour, undertook systematic testing of cables between 1823 and 1824. In 1823 he completed the first suspension bridge in continental Europe, the St-Antoine-Geneva footbridge for pedestrians, comprising two 40 m spans.

A record for the longest single span, indeed a record that stood for many years, was established in Europe by the suspension bridge at Fribourg (CH) with a span of 265 m (Fig. 3.3). This was constructed by a Frenchman, Joseph Chaley, in 1834 and demolished in 1930. Only one suspension bridge of this era is still in existence in continental Europe: the Pont de la Caille over the ravine des Usses in Savoy, credited to Belin in 1839, spanning 192 m (Fig. 3.4).

At the beginning of the 19th century, practical and robust procedures were developed for the industrial production of rolled plates (1830). These permitted more economical and easier construction of large structures by riveting. An important improvement for the cables was developed by a Frenchman, Arnodin, who perfected the fabrication of double-spiral (alternately wound) steel wire ropes around 1880, which replaced, once and for all, the previous solutions with parallel wires.

Cast iron, a relatively brittle material, did not lend itself to the construction of beam bridges. It was only towards the middle of the 19th century that the first examples of such bridges appeared, with the development of wrought iron (which is notably better in tension) for use on an industrial scale. One of the first

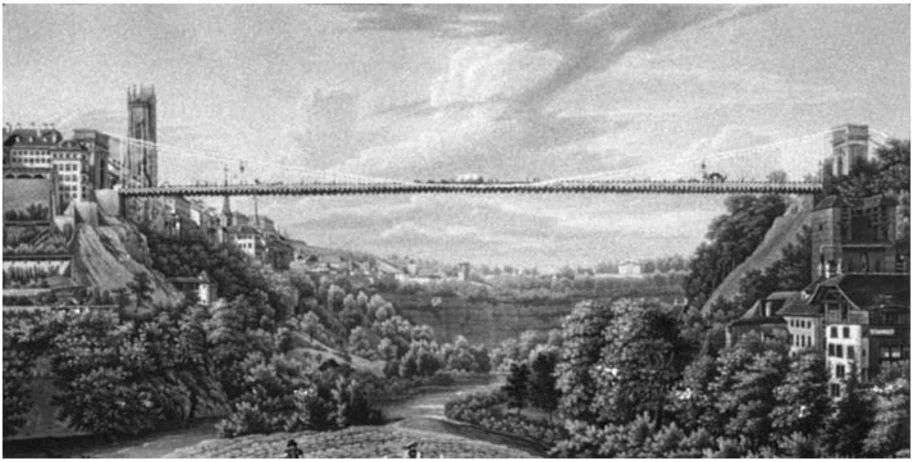

Fig. 3.3 Pont de Fribourg (CH). Engineer Joseph Chaley (Photo IBK, ETH – Zurich).

Fig. 3.4 Pont de la Caille (F). Engineer E. Belin (Photo ICOM).

great beam bridges was the Britannia in Wales, which entered service in 1850 (Fig. 3.5). With two main spans of 146 m, this beam bridge had a closed cross section in the form of a rectangular box inside which passed a railway line. It was replaced in 1971 by a bridge with a structure of steel truss arches.

Wrought iron had thereby replaced cast iron for large span arches. The most spectacular example of this form of construction was the Viaduc de Garabit, constructed by the team of Gustav Eiffel in 1884 (Fig. 3.6). The total length of 564 m includes a triangulated arch of 165 m span and rise of 52 m. It was erected using the cantilever method, spanning out from the supports.

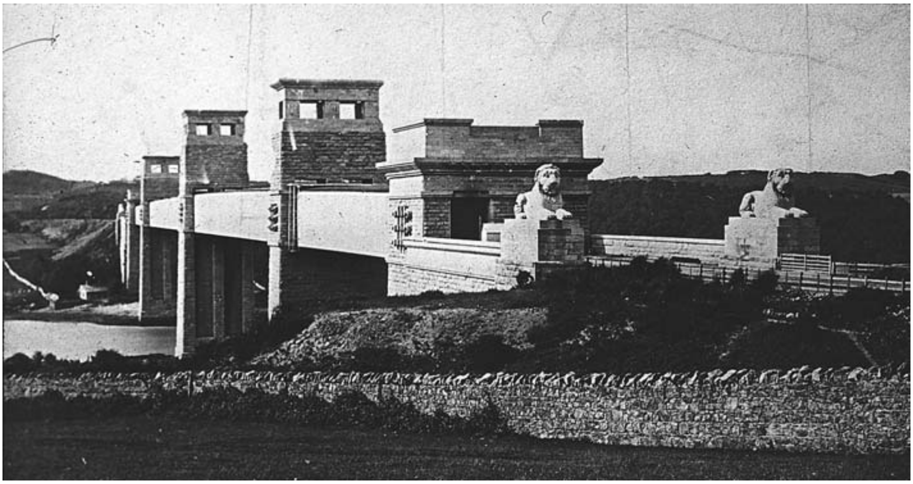

Fig. 3.5 Britannia Bridge (UK). Engineers William Fairbairn and Robert Stephenson (Photo IBK, ETH – Zurich).

Fig. 3.6 Viaduc de Garabit (F). Engineers Maurice Koechlin and Léon Boyer (Photo ICOM).

Fig. 3.7 Firth of Forth Bridge (UK). Engineers Benjamin Baker and John Fowler (Photo Robert McCulloch).

Development of the industrial fabrication of steel followed the invention of the Bessemer converter in 1856 and the Siemens-Martin process in 1864. Due to its mechanical properties, and in particular its improved tensile behaviour, steel went on to entirely replace both cast iron and wrought iron. The era of steel bridges in Europe began with the Firth of Forth (Fig. 3.7), which adopts truss beams, of variable depth and extremely rigid. Constructed between 1881 and 1890, this bridge comprises two central spans of 521 m and two side spans of 207 m each. The central spans comprise two cantilevers, each of 207 m, supporting between them a beam of 107 m span. This system, know simply as "cantilever", was subsequently adopted for a large number of similar bridges.

The first developments of long span bridges in the United States are credited to John A. Roebling. One of the most notable is the first suspension bridge crossing the deep gorge downstream of the Niagara Falls. Completed in 1855 this bridge had a span of 250 m and was constructed in two stages – firstly for a railway and secondly for horse drawn carriages. It was demolished in 1896.

During this period (1877) electric arc welding was discovered. Alongside the ever increasing ability to produce thicker steel plates, this new joining method allowed, later on in the second half of the twentieth century, the fabrication of the solid web beams (plate girders) widely used today.

John A. Roebling was also the father of the Brooklyn suspension bridge across the East river in New York, which entered service in 1883 (Fig. 3.8). Its span of 487 m set a world record at the time, and was achieved thanks to the first use of steel cables. This bridge was also original in the form of the cables, which combined suspension cables and cable stays in a form known as "hybrid".

For a long time the United States was known as the country of long span suspension bridges, particularly due to the works of the Swiss engineer, Othmar H. Ammann. He was the first to achieve a span exceeding 1000 m, with the George Washington Bridge, which crosses the Hudson River in New York (Fig. 3.9). This was inaugurated in 1932 with a span of 1067 m and had a second deck added in 1962. The magnificent Golden Gate Bridge was inaugurated five years later with a span of 1280 m (Fig. 3.10).

Specific studies of the aerodynamic performance of suspension bridges were undertaken following the collapse of the Tacoma Bridge, which was destroyed in 1940 due to the resonance of its deck in adverse wind conditions. These studies led to the adoption of decks comprising large truss box girders. In addition to their aerodynamic advantages, the use of such boxes facilitated the construction of twin deck bridges

Fig. 3.8 Brooklyn Bridge (USA). Engineers John A. Roebling then Washington A. Roebling (Photo Bojidar Yanev).

Fig. 3.9 George Washington Bridge (USA). Engineer Othmar H. Ammann (Photo Bojidar Yanev).

Fig. 3.10 Golden Gate Bridge (USA). Engineer Joseph B. Strauss (Photo Rich Niewiroski Jr.).

using beams between 10 m to 12 m deep, and carrying traffic on the upper and lower flanges of the box girders. The Verrazano-Narrows Bridge at the entrance to New York, also the work of Ammann, was conceived in this way and inaugurated in 1964 (Fig. 3.11). This suspension bridge held the world record until 1981 at a span of 1298 m.

That record for longest span was finally beaten in 1981 by the Humber Suspension Bridge in Great Britain (Fig. 3.12). This bridge has a span of 1410 m, and its deck adopted a new form, comprising a box girder with an aerodynamic shape to significantly reduce the effects of wind. However, this modern form of deck has not been used in Japan where large suspension bridges are still constructed using box girder trusses (mainly due to the need to separate railway and road traffic on double decks). Most of the long span suspension bridges in Japan link the islands of Honshu and Shikoku. One of them, the Akashi Kaikyo Bridge, has held the world record for longest span since its inauguration in 1998, with a central span of 1991 m (Fig. 3.13) [3.1].

In the world of arch bridges, it is worth considering some notable examples, such as the New River Gorge Bridge in the United States, constructed in 1977 with a span of 518 m (Fig. 3.14); the bridge at Bayonne in the United States (1931 and 504 m); and Sydney Harbour Bridge in Australia (1932 and 503 m). Since 2003 the Lupu Bridge in China spanning 550 m has held the record for a steel arch bridge (Fig. 3.15).

Fig. 3.11 Verrazano-Narrows Bridge (USA). Engineer Othmar H. Ammann (Photo Bojidar Yanev).

Fig. 3.12 Humber Bridge (UK). Engineers Gilbert Roberts and Bill Harvey (Photo ICOM).

Fig. 3.13 Akashi-Kaikyo Bridge (J). Engineer Honshu-Shikoku Bridge Authority (Photo Honshu-Shikoku Bridge Expressway Compagny).

Fig. 3.14 New River Gorge Bridge (USA). Engineer Clarence V. Knudsen (Photo Jason Galloway).

Of final note is the spectacular growth in the use of cable stayed bridges since the middle of the 20th century. Developments in materials (high strength steels), methods of calculation by computer, and the capacity of lifting equipment used for erection have all contributed to this trend. As an example, in 1957 the longest span for a cable stayed bridge was 260 m for the Theodor Heuss Bridge in Dusseldorf, Germany. By the end of the 1980s, the longest spans were between 400 and 500 m, which included numerous bridges in Thailand (Rama IX, 450m, 1987), Japan (Yokohama Bay, 460 m, 1989) and Canada (Annacis Island, 465 m, 1986). During the final years of the millennium, the record for longest span was toppled with increasing frequency: reaching 856 m in 1995 with the Pont de Normandie in France, then 890 m in 1999 with the Tatara Bridge in Japan (Fig. 3.16) [3.1]. Now cable stayed bridges are competing in the span range that was previously the exclusive domain of suspension bridges.

Fig. 3.15 Lupu Bridge (PRC). Engineer Shanghai Municipal Engineering Design Institute (Photo Yaojun Ge).

Fig. 3.16 Tatara Bridge (J). Engineer Honshu-Shikoku Bridge Authority (Photo Honshu-Shikoku Bridge Expressway Compagny).

Another evolution is the use of multiple span cable stayed bridges. The most notable example of this type of structure is the Viaduc de Millau in France (Fig. 3.17), which was opened in 2004. Conceived by Michel Virlogeux, it comprises eight cable stayed spans, of which six spans reach 342 m [3.2].

In Switzerland a number of noteworthy steel and composite bridges were constructed as part of the development of the freeway (autoroute) network. Examples include the composite bridge over the Veveyse near Vevey (1968), which comprises a 5 m deep steel box girder with spans of 58 m, 129 m and 111 m [3.3]. Additionally, there are the bridges over the Rhone at St Maurice (1986), which are cable stayed composite bridges spanning 100 m (Fig. 3.18). More recently, two important and innovative bridges have been constructed on the Yverdon-Berne section of freeway A1. The Viaduc des Vaux (1999) is a box girder

Fig. 3.17 Viaduc de Millau (F). Engineer Michel Virlogeux (Photo Daniel Jamme).

Fig. 3.18 Bridge over the Rhone at St Maurice (CH). Engineer René Walther (Photo ICOM).

composite bridge with spans of 130 m. It was launched despite its complex S shape geometry plan. The Viaduc de Lully (1995) is a composite bridge that adopts space frame steel trusses (Fig. 3.19) [3.4]. A particularity of this bridge is that the trusses are formed from thick walled tubes welded to each other without the use of gusset plates.

Fig. 3.19 Viaduc de Lully (CH). Engineer Hans-G. Dauner (Photo ICOM)

Europe is currently experiencing a kind of renaissance in steel and composite bridges of small to medium span. This puts an end to an era in which the vast majority of this type of structure was constructed using reinforced or prestressed concrete. Part of this change is due to improvements in methods of fabrication, such as automatic welding, and the numerically controlled cutting of plates. It is also due to the development of high strength steels with improved characteristics for welding. Developments in lifting equipment, for both the workshop and on site, now allow prefabrication of elements of considerable weight. The adoption of these larger elements can greatly simplify the work needed on site and thereby significantly reduce the time needed on site. For these various reasons, composite bridges, and in particular twin beam composite bridges, appear likely to continue to see widespread uptake for several decades in the world of short and medium span bridges. This will be true for both road and railway bridges. Already of note is a significant increase in the use of twin beam composite bridges in Japan since the beginning of the 2000s, and some very nice bridges on the TGV (high speed train) network in France [3.5 and 3.6].

3.3 Record Spans

This section presents a succinct summary of the evolution of record spans for different types of bridge across the centuries. The historic evolution is of more interest than the numbers themselves, given that the records increased rapidly, particularly for cable stayed and suspension bridges.

Figure 3.20 presents the evolution of span record for different types of bridges from 1800 to the present day. The types of bridges considered are box girder beam bridges, truss beam bridges, arch bridges, suspension bridges and cable stayed bridges. The figure shows several landmark structures for each type, and includes works that will push the records further and are at either the project stage or being constructed at the time of writing.

Box girder bridges had their glory years in the middle of the 20th century in Europe due to the post war need for reconstruction. Germany played a key role in the development of great steel bridges with variable depth beams, with several examples over the Rhine and its tributaries. Since the 1980s box girder bridges

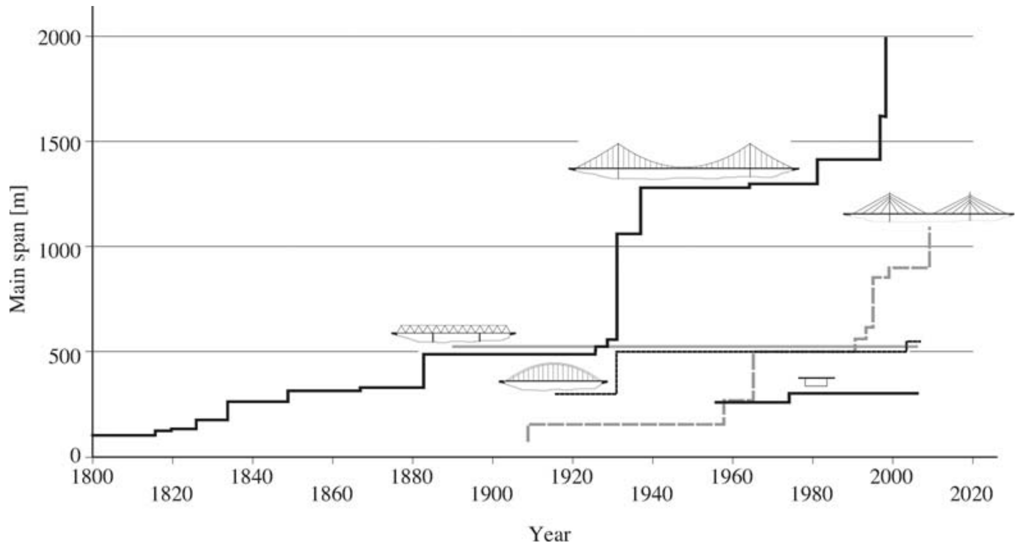

Fig. 3.20 Evolution of record spans from 1800 to the present day for different types of bridge.

have not been competitive with cable stayed bridges over these sorts of span, and are now primarily used for spans up to around 200 m. This leaves the world of long spans free for cable based solutions.

Unlike box girder bridges, truss beam bridges have mainly been constructed outside Europe, particularly in the United States during the development of their railway network. Here also, since around 1980, cable supported bridges have replaced trusses for long spans, mainly for economic reasons [3.7]. Large trusses are unlikely to see widespread use in the future because they are often judged as being of limited aesthetic appeal. The great truss bridges are often of significantly varying depth, as illustrated in Figure 3.7.

The category of arch bridges can be sub-divided into solid web arches and truss arches. Although the latter facilitate the achievement of long spans, today they have suffered the same fate as truss beams. Whereas solid web arches have seen a great renaissance in Europe in the form of tied arches (Fig. 2.3(a)), their spans are limited to around 150 m.

New world records for span are no longer possible for these first three types of bridges because cable stayed and suspension bridges are significantly more economic for spans over 400 m or so.

Suspension bridges have always dominated the world of long spans (more than 500 m). It is interesting to note that recent years have seen a spectacular jump in spanning ability, passing from 1410 m in 1981 to nearly 2000 m today. All the indications are that this trend will continue. For example, plans to cross the Messina straits to link Sicily to mainland Italy include the provision of a 3300 m span hybrid cable stayed suspension bridge.

As noted in Section 3.2, since around 1980, cable stayed bridges are the type that has seen the most spectacular development. Figure 3.20 illustrates this rapid evolution. A cable stayed bridge in China that was opened to traffic in 2008 has a span of 1088 m and forms part of the Sutong Bridge, that has a total length of 8206 m.

Fig. 3.21 Sutong Bridge (PRC). Engineer Jiangsu Provincial Communications Planning and Design Institute (Photo Yaojun Ge).

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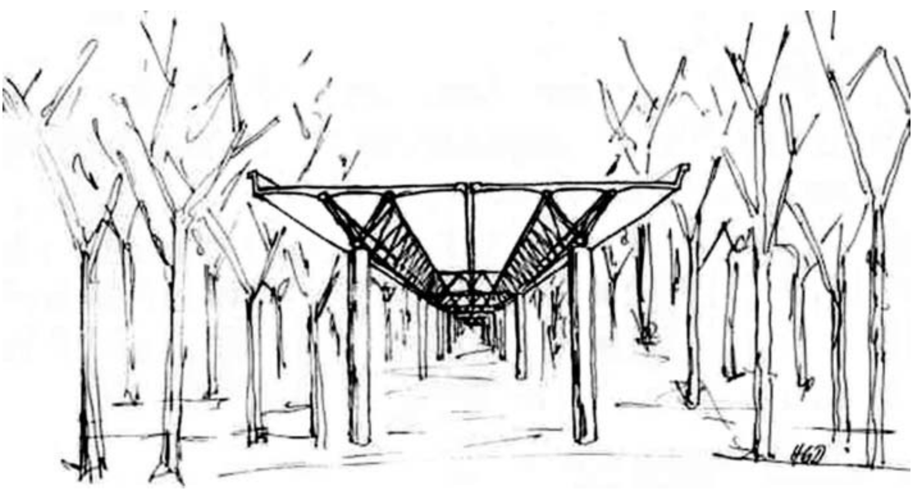
4 Basis for Conceptual Design



Viaduc de Lully (CH).
Sketch by Hans-G. Dauner, ingénieur civil, Aigle.

4.1 Introduction

During the elaboration of a bridge project, the development of an overall concept is an essential role of the engineer. The principal characteristics of the structure are defined and collectively categorised as the ***structural concept***.

This chapter describes the development of the concept by explaining the phases that the engineer should take and examining the basis for these phases. Figure 4.1 schematically shows the components that are the basis for the *structural concept*, and puts them in the context of the elaboration of a bridge project. The figure also highlights where these components are addressed in this book, either in this chapter or in parts of other chapters dedicated to the structural concept (Chap. 5-8) and to the ***analysis and design*** (Chap. 9-15) of beam bridges.

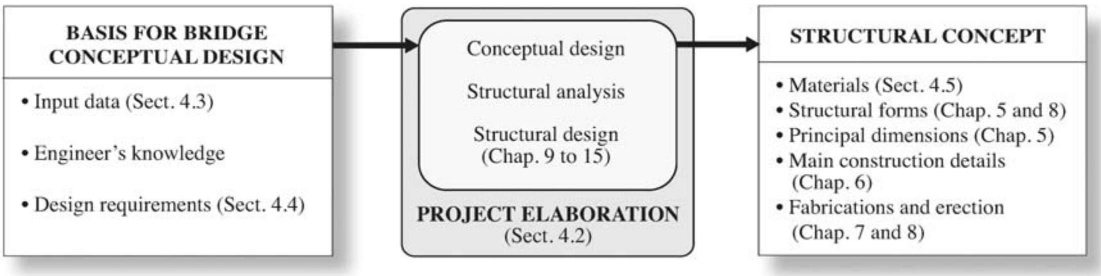


Fig. 4.1 Relationship between the basis for the bridge conceptual design and the structural concept.

Section 4.2 of this chapter describes the different roles typically performed by an engineer during the elaboration of a bridge project, in which the conceptual design phase plays a dominant role. Section 4.3 considers the ***input data*** needed to begin the conceptual design of a bridge. This input is defined by the client's representative and relates to both the *bridge* itself and the *site* in which it will be located.

There is a connection between Chapter 4 and Chapter 9 concerning the ***basis of design***. In particular, much of the project input data is also essential for the design. Therefore, it is compiled in the ***client's requirements*** (§ 9.3.1), a document created at the start of all construction projects on the basis of discussions with the client's representative. By considering both the input data and the performance requirements concerning both the bridge and site, the project leader establishes the ***basis of design*** (§ 9.3.2).

Section 4.4 lists and describes the qualities of a well conceived bridge, qualities that the engineer must always keep in mind to guide the concept development. In particular, the following qualities are essential:

- reliability,
- robustness,
- durability,
- aesthetics,
- economy.

Reliability is achieved primarily through the application of certain principles and design checks aimed at assuring serviceability and structural safety (Sect. 9.5 and 9.6). The other qualities are directly related to both the choices that are made and the respect of various criteria, for which rely the experience, competence and imagination of the engineer are pivotal.

It is worth emphasising that the aesthetic and economic aspects of a bridge can only be approached from an indicative point of view in this chapter and book. While some of the economic parameters discussed will remain relevant to both other parts of the world and other times, this will certainly not be the case for others, which vary as a function of local technological developments and a given economy (particularly the cost of labour). As for aesthetics, some objectively recognised principles are highlighted, but it is difficult to claim that one bridge solution is better than another, particularly when considering its interaction with the local environment.

Finally, during the conceptual design phase the properties and characteristics of the materials have a determining influence. They are outlined in Section 4.5, where aspects of particular relevance to bridges are developed. That section also covers the question of corrosion protection and solutions that can be applied to steel bridges.

To summarise, as far as the bridge conceptual design development is concerned:

- Chapter 5 covers the ***structural forms*** used for bridges, the ***typology*** of beam bridges, and ***preliminary sizing*** of their structural elements,
- Chapter 6 presents different solutions for steel bridge ***construction details***,
- Chapter 7 describes the main ***erection methods*** for the principal structural elements of a bridge,
- Chapter 8 covers the *structural concept* and detailing of ***slabs*** for composite (steel concrete) bridges, as well as their ***construction***.

4.2 Project Elaboration

The elaboration of a bridge project for a given site and use requires several phases from developing the concept right up to the execution of the works. These phases are shown schematically in Figure 4.2. They are numbered 1 to 5 and correspond to distinct activities. Each one has clearly defined objectives and specific outputs.

4.2.1 Preliminary Studies

The objective of the preliminary studies for a bridge project is to identify and study a range of possible solutions and choose those which warrant further consideration. This activity is particularly concerned with showing the overall feasibility of the project and estimating costs, prior to developing the project further.

After having visited the bridge site and considered the information supplied by the client's representative (Sect. 4.3), the engineer will start to outline and develop several possible solutions. Rarely does the combination of requirements for use and specific site conditions yield only one possible solution for a bridge.

Sketches of potential solutions are produced with a degree of precision that depends on the complexity of the project. The elaboration of each solution is an iterative process, and modifications are made at each step to reflect the various constraints of execution, ground conditions, aesthetics and so on.

The preliminary study phase is purely one of structural concept development, during which no calculations are made. Various sketches and drawings fix the general lines of the bridge, the structural form, the spans, etc. The size of key elements is estimated first on the basis of experience and the consideration of typical values of slenderness, and then on rudimentary calculations.

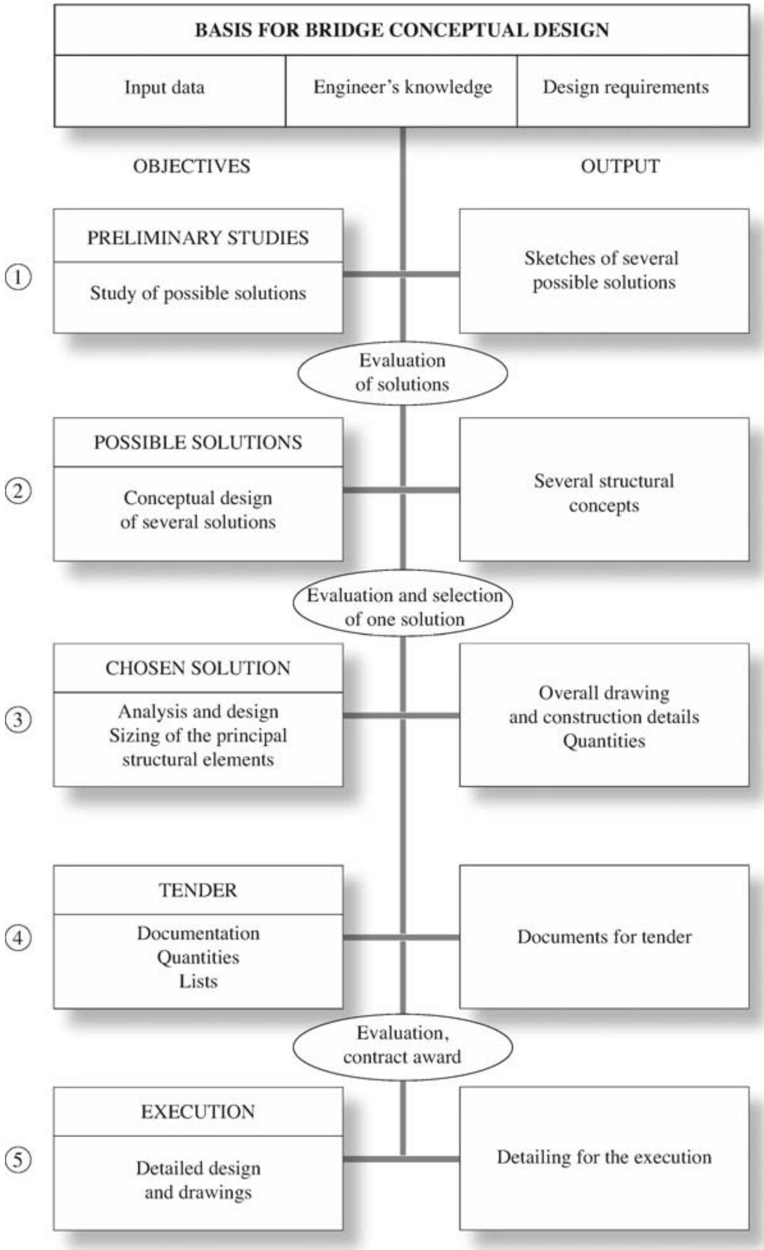


Fig. 4.2 Phases leading to the construction of a bridge.

The first phase ends with an evaluation of the feasibility of the various solutions that have been developed, and with the identification of one or more that warrant further study. The duration of this phase, the choice of how many possible solutions to take forward, and the organisation of the rest of the study clearly depend on the importance of the bridge.

4.2.2 Possible Solutions

During this phase work is pursued on one or more solutions taken forward from the preliminary studies, with the aim of identifying the definitive *structural concept*. Having already made certain key choices, additional choices are made to an extent that the required qualities of the bridge can be delivered (Sect. 4.4). The other purpose of this phase is to define, in a more detailed way, all the measures needed to assure both adequate performance during construction and service. The cost of the retained solutions can then be established based on initial cost estimates, previous professional experience, and input concerning other items, such as bearings, joints, and water evacuation.

Important parts of this phase concern the development and refinement of the design regarding serviceability and structural safety. When considering the basis of design, it is important to include both the erection and the final state. The size of the principal structural elements must be established more precisely, based on a preliminary design. The result of this phase takes the form of drawings showing key dimensions of the project and its essential characteristics, such as the overall structural form, the cross section, longitudinal and transverse load carrying structures, and major construction details. Each of the presented solutions has a technical log that justifies the choices made and describes the erection method to be adopted.

This phase may be considered the most important in the elaboration of a bridge project, because it includes fundamental choices that will influence the rest of the project. Numerous qualities of the completed bridge – notably those concerning robustness, durability, aesthetics and cost – depend on decisions made during the consideration of possible solutions. To facilitate placement and joining of the different structural elements during erection, major construction details and execution should not be neglected during this phase of concept development. These factors can also affect the ease of future inspection and maintenance.

During this study of possible solutions, the project leader must apply experience, imagination, and sound knowledge about not only different bridge types and their behaviour, but also materials and statics. Knowledge of the means, competence, and traditions of potential suppliers is equally necessary, and particularly needed when presented with fabrication or erection problems of unusual complexity. Such problems may necessitate devising solutions that are both original and economic.

The choice of a definitive structural concept is based on an evaluation of the possible solutions according to the following criteria:

- meeting the various input and use requirements,
- possibility and ease of execution,
- durability and maintenance,
- aesthetics,
- cost and duration of the work.

The weight given to the criteria varies according to the client's priorities and the specific location of the bridge.

4.2.3 Chosen Solution

The aim of developing the chosen solution is to prepare all the information that is necessary to invite construction offers. In particular this phase entails structural analysis and design of the structural elements, in order to prepare general plans of the chosen solution and drawings of representative construction details. A detailed programme for the construction works is also prepared during this phase.

An initial estimate of costs is then developed. The documentation developed during this phase is used for the public enquiry, to combat any objections, to gain approval from the relevant public authorities, and as the basis of the tender documents that will be issued to construction companies wishing to prepare bids.

4.2.4 Tender

In this phase work contracts are awarded to construction companies. This phase comprises preparation of the necessary set of documents that are issued as part of a public process. Responses from different companies are then compared and evaluated, taking into account a number of criteria related to the quality of the response, the work programme proposed, and the price. The works will be awarded to the companies that best meet the criteria set out in the tender documents.

4.2.5 Execution

During the execution project all the documents needed for the fabrication and erection of the structure are prepared. Activities include the detailed design of the structural elements and all the detailing that is necessary for execution, including workshop drawings, piece listings, etc. Also included is the elaboration of the quality control plan, which will be used to ensure that the execution conforms to the requirements, particularly those contained in the basis of design documentation.

After the site works have been completed, the engineer should use the drawings as the basis for "as-built" plans reflecting any changes made during execution. He should also ensure that the basis of design has been either satisfied or modified to reflect any decisions made during execution, particularly with regard to structural safety.

Finally, the engineer contributes to the inspection and maintenance plans supplied to the client. These documents identify any specifics to consider during inspection and maintenance of the particular bridge. They must note any measures identified in the basis of design that concern inspection, or that imply the need for regular maintenance of structural elements or mechanical apparatus. These plans enable the bridge owner to define the frequency and nature of inspection and maintenance, and thereby to start planning and financing these activities.

It is worth highlighting that the elaboration of a bridge project may be the subject of a competition, if the bridge is of a certain importance or has some other peculiarity. This is often the case for bridges to be built in an environment where aesthetic considerations dominate, for bridges that are anticipated to be of high cost and, generally, for bridges forming part of the communication routes network. In such cases several design offices, or consortia, undertake the first three phases, shown in Figure 4.2.

Typically, a professional establishes the cost of the bridge on a standard basis in order to be able to compare the offers of the different tenders. A jury then evaluates the different solutions, with particular reference to the criteria noted above (§ 4.2.2), and chooses the most suitable option. Although other types of competition are possible, the one as described above – known as a project competition – is the most common.

4.3 Input Data for a Bridge Project

The engineer charged with the elaboration of a bridge project requires input data to summarise the client's requirements, to define the basis of design, and to begin the conceptual design of the bridge. It comprises data relating to requirements for the bridge use, for the bridge itself, and also for the site where the bridge is to be located. Figure 4.3 shows the main input data that the engineer must obtain at the start of the project. These issues are discussed below.

INPUT DATA FOR A BRIDGE PROJECT

Requirements for use
- Use of the bridge, possible extensions
- Planed service life
- Maintenance requirements
- Special conditions for use
- Special conditions for erection
- Deadlines
- ...

Specifics for the bridge
- Alignment
- Longitudinal profile
- Transverse profile
- Site to be crossed
- ...

Specifics for the site
- Topography
- Geology
- Geotechnics
- Hydrology
- Climate and seismicity
- ...

Fig. 4.3 Input data for a bridge project.

4.3.1 Requirements for Use

Clearly the engineer must know the use for which the bridge is destined: road traffic, railway traffic, pedestrian traffic, or mixed traffic. He must know if he is required to conceive a single bridge for the totality of the traffic, or if two or more structures are needed (for example bridges on the Swiss freeway network adopt a separate bridge or a separate superstructure for each direction of traffic). The engineer must also be aware of the type and quantity of any additional installations, such as pipes or ducts, which will be carried by the bridge and can be particularly significant in an urban situation. Any requirement to carry exceptional traffic must also be highlighted. The configuration of the bridge can also be affected if future changes are already envisaged (widening of the roadway, removal of footpaths, etc.).

The planned service life (which is generally recommended in the relevant codes and standards) must be identified. This is particularly relevant to the estimation of the traffic that will use the bridge during its lifetime, as its volume and weight must be taken into account when considering the fatigue life of the steel structure (Sect. 12.7). It is necessary to formulate particular requirements, which can be quite detailed, as a function of the importance and location of the bridge (for example, the necessity to withstand a seismic event). Other requirements concern specific use (military traffic), or access to the structure to allow inspection and maintenance. They might also concern, for example, local access restrictions during construction, or protection of the water table. Finally, details of construction milestones – particularly the date for “opening to traffic” – must be defined.

4.3.2 Specifics for the Bridge

The bridge is defined in terms of its type of use, its alignment, and its longitudinal and transverse profiles. The functional inputs may be completed with particular requirements for the site to be crossed, such as any height or space restrictions, zones where piers cannot be placed, or even requirements related to the desired architectural quality.

Alignment

The alignment defines the geometry of the bridge axis. If it is part of a new line, particularly in a non-urban environment, the geometry should as far as is possible try to avoid excessive skews or radii of curvature. Such features may have a detrimental effect on the mechanical functioning of the bridge, or reduce the number of viable options for the type of bridge and method of erection. It is often found that relatively minor changes to the bridge orientation can allow worthwhile economies during execution or indeed the service life of the bridge. In an urban environment the line is often predetermined by the existing infrastructure, whether a new bridge is being created or the project is for a bridge replacement.

Longitudinal Profile

The longitudinal profile defines the line of the bridge in elevation. This line must take into account any restrictions associated with the obstacles to be crossed, in particular height requirements for any routes being traversed (including their possible developments). To satisfy both water run-off and architectural requirements, profiles that are either horizontal or in the form of a basin are to be avoided. Noticeable discontinuities in the slope of the bridge must also be avoided, using transition curves to achieve the desired variations in profile.

Transverse Profile

The transverse profile defines the width of the bridge. This depends on the type of use, as it defines the number of lanes or tracks, and the presence of footpaths or other specifics. The client's representative often defines the edge details, parapets, or crash barriers. Although aligned with the transverse profile requirements, the configuration of the bridge cross section must also take into account possible future evolutions in the bridge use. This is particularly relevant if widening of the deck is envisaged. If mixed traffic will be present, for example road and pedestrian, consideration should be given as to whether the road traffic could temporarily use the space normally reserved for pedestrians, or whether permanent barriers will be envisaged to separate the different types of traffic. A further consideration with mixed traffic – private or public – is whether the traffic will be completely separated or will use the same space, including potential future developments.

Site to Be Crossed

If the site to be crossed includes communication routes, then any height and space restrictions associated with the type of traffic using these routes must be defined. Serious consideration should be given to any future development projects planned for these routes, including widening. Height restrictions [to respect above the route being crossed] should take into account variations in longitudinal profile along the route, as well as future resurfacing of the route and any uncertainties associated with settlement of the new bridge. When a waterway is being crossed, consideration must be given to the acceptability of placing piers in the water, and any restrictive effects of such piers on the water flow must be clarified. Finally, any requirements associated with the integration of the bridge into the environment, including its architectural qualities, must be identified.

4.3.3 Specifics for the Site

The main input data associated with the site concerns the topography of the region, its geology and associated geotechnical data, hydrological information when an important waterway is being crossed, and actions concerning the climate and seismicity for the region. Although numerical values for these actions are generally presented in relevant codes and standards (in Switzerland SIA 261), in some cases it is necessary to consider site-specific values (for example, the bridge orientation relative to the dominant wind direction, or the effects of wind turbulence in a valley).

Topography

The topography of the site concerns the configuration of the ground. It should define the position of any access routes that are either existing or planned for the construction, as well as areas available for site installations or storage. This information is essential given the influence it may have on the options available for the bridge type and erection method. Before starting any work, it is imperative that the project manager visits the site. This visit will furnish information not only concerning the specific topography, but also the local environment in terms of degree of urbanisation, type of vegetation, etc. These elements must be taken into account to ensure that the future bridge will integrate well into its local environment.

Geology

The geological input defines the nature of the ground. Data are obtained from surveys (drilling, bore-holes) either near or at the location of the future bridge. These data are important not only to identify the best positions for the pier and abutment foundations, but also to determine the most appropriate form of the foundations (shallow or deep), and these may influence the bridge type. The geological data must particularly include information concerning the orientation of, and cracks in the bedrock layers, the extent of any zones at risk (falling rocks, avalanches, etc.), the activities of any possible slip zones, as well as the presence of subterranean faults or watercourses.

Geotechnics

The geotechnical input data, based on tests either on site or in the laboratory, concern those ground properties necessary to calculate its resistance and deformability. This information is needed to define the depth of the foundations and predict the movements and settlement of the terrain. The project manager must also know the level of the water table, as this will enable him to determine the most appropriate method for construction of the foundations. The initial geotechnical data will not explicitly relate to the exact locations of the foundations. The data will come from the interpretation, by a soil mechanics expert, of information obtained over the length of the site. According to the importance of the bridge, or the difficulty of the terrain, one or more specific tests, at or in the vicinity of the proposed pier and abutment positions, may be advisable to avoid the wrong foundation concept and any costly surprises during execution.

Hydrology

When the bridge is to cross an important waterway, it is necessary to know the different water levels, as well as the periods of high and low water, as these may influence the foundation concept or the location of temporary piers to be placed in the waterway during execution. In addition to knowing the transverse profile of the water course, it is important to know the lines of the banks and their evolution over time; this information should take into account any planned modifications to the banks and effects of natural erosion. If it is possible to place piers in the water course itself, then information concerning the rate of flow and possible water borne bodies is needed, as well as information to evaluate the depth of possible scour.

Climate and Seismicity

For most bridges, sufficient input information concerning climate and seismicity can be found in the codes and standards dealing with actions on structures. Climatic information includes:

- wind actions,
- temperature,
- snow.

For particularly slender bridges, such as those spanning deep valleys, or for cable stayed or suspension bridges, it is often necessary to carry out specific studies to determine their dynamic behaviour under the effect of wind or during an earthquake.

Snow is normally only considered for covered bridges, or bridges situated on routes that are not kept clear, or perhaps for very specific conditions during execution. Climatic and seismic actions are described in more detail in Chapter 10, which is dedicated to actions on bridges.

4.4 Design Requirements

During the conceptual design of a bridge, the engineer must choose those solutions that will best provide the structure with the following qualities:

- reliability,
- robustness,
- durability,
- aesthetics,
- economy.

4.4.1 Reliability

The reliability of a bridge relates to its ability to satisfy the design limit states (§ 9.3.5) for the planned service life. In order to guarantee the reliability of a structure, it is necessary to take into account uncertainties associated with: the definition of the actions, the modelling of the structure, the determination of the action effects, and the resistance of the materials. The reliability is expressed in terms of the probability of limit states being exceeded and is assured by verification of the *design criteria* for the *serviceability limit state* (Sect. 9.5) and for the *ultimate limit state* (Sect. 9.6).

This trait of reliability, which is common to all construction, should be based on a clear concept for the design principles. In Switzerland these design principles are defined in the standard SIA 260 *Basis of structural design*, that serves as a reference document for those codes and standards associated with each construction material. This basis is described in detail in Chapter 2 of Volume 10 of the TGC, and in Chapter 9 of this book.

Reliability is not the only quality that a structure must possess in order for it to be deemed satisfactory. For any given structure several solutions may meet the requirements of reliability. However, only some of these solutions will be acceptable once other fundamental requirements concerning *robustness*, *durability*, *aesthetics,* and *economy* are fulfilled. All of these should be taken into account when developing the bridge concept, especially during the preliminary studies (§ 4.2.1) and consideration of those possible solutions (§ 4.2.2) for which choices are sketched out and comparisons made.

4.4.2 Robustness

According to the definition given in the standard SIA 260, the "robustness of a structure is its ability to limit the extent of damage or failure to something that is proportionate to the cause of that damage or failure". In other words, if a structure is locally damaged by an unforeseen, event but is nevertheless able to continue to carry out its function for some time (because there is no sudden collapse), that structure may be deemed robust. Unforeseen events may be actions that are difficult to predict, for example in terms of magnitude, including collisions, explosions, and sabotage. Other types of unforeseen events include failure of a key structural element, errors in the design, or inaccuracies in the execution. Therefore, the aim of designing a robust structure is to limit the consequences of unforeseen events to an extent that, where possible, total collapse of the structure is avoided.

This notion of robustness is illustrated in Figure 4.4 by considering three different structural forms for a three span bridge. All three solutions theoretically possess the same level of reliability, provided that the beams are correctly sized to meet the requirements for the serviceability and ultimate limit states. If an unforeseen event leads to the failure of a local section in the central span, this would result in collapse of that span for both the solution comprising two cantilever beams with a suspended beam (Fig. 4.4(a)) and that comprising the series of simple spans (Fig. 4.4(b)); however, the continuous beam (Fig. 4.4(c)) would not collapse. Therefore, the continuous beam represents a more robust solution than the other two. Its central span might well be more flexible after the failure of a local section, but it would not create catastrophic consequences for persons either on or under the bridge.

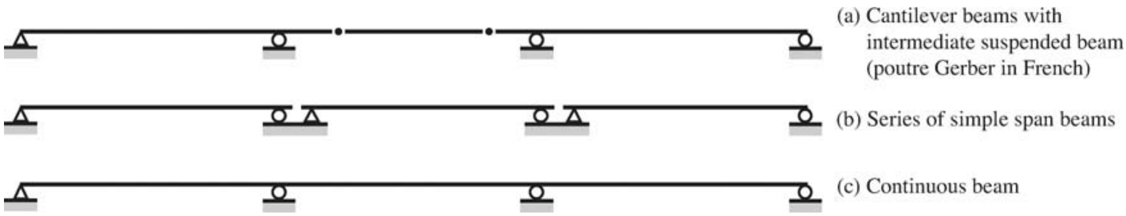


Fig. 4.4 Examples of non-robust systems (a) and (b), and a robust system (c).

To conceive a robust structure, the designer must give particular consideration to the following aspects:

- Conceiving a structure such that the loads can follow other paths through the structure, should one of the structural elements fail (indeterminate structures, the idea of redundancy), even if this resulted in substantial deformations. For example a cable stayed bridge should be able to accommodate the unexpected failure of a cable by redistribution of forces in the deck and the other cables. Favouring the use of ductile materials for the structural elements and connections.
- Using structural forms that are stable and insensitive to ground movements, such as those resulting from seismic events (§ 10.6.1) or unforeseen settlements.
- Using structural forms that are insensitive to inaccuracies during execution.

- Providing protection to resist accidental actions (for example crash barriers, buffers).
- Providing easy access so that the entire structure can be inspected and any damage or unforeseen behaviour identified for remedial interventions.

In the world of bridges, robustness is of great importance for structures situated on routes that have high levels of traffic. It is also essential for bridges that are of key strategic importance, and therefore must absolutely function to facilitate evacuation of a region following a catastrophe (lifeline).

4.4.3 Durability

A bridge is said to be durable if its overall configuration and its detailing satisfy the requirements of the serviceability and ultimate limit states, under the expected actions, when respecting the planned inspection and maintenance programmes. The notion of durability is associated with the planned service life for the structure and its components. It is worth distinguishing between the design life of elements that do not require maintenance (structural elements made from weathering steel, concrete elements), and those for which maintenance, or even replacement at regular intervals, is planned (surfacing, paint, expansion joints, bearings). For the latter, the design life proposed in the basis of design influences the choice of construction details in terms of how they facilitate replacements and the materials used.

Assuring durability during the course of the planned design life is primarily dependent on the following:

- the choice of materials and their adequate corrosion protection,
- the choice of good construction details,
- careful execution of the details,
- an overall bridge concept, and detailing concepts, that facilitate observations, inspections and interventions.

During the conceptual design of steel and composite bridges, the main aspects associated with durability that an engineer should consider are: the choice of materials (Sect. 4.5), the corrosion protection (§ 4.5.5), and the primary detailing of the steel structure (Chap. 6) and slab (Chap. 8).

If decisions are made sufficiently early, then all considerations concerning the configuration and detailing of the structure can have a noticeably beneficial effect on durability. Neglecting or forgetting such considerations during the conceptual design phase can lead to costly on-site improvisations that may result in reduced quality, reduced durability, and substantial difficulties in maintaining the bridge.

In its document entitled, “Bridge construction details” [4.1], the Swiss Federal Roads Office proposes examples covering various types of bridge equipment.

4.4.4 Aesthetics

Although it cannot be measured or quantified, aesthetic value is counted amongst the important qualities of a bridge. This value is based on an overall judgement that takes into account the local environment, the structural form of the bridge, expression of the details, the colour, as well as other parameters, according to the specific case. Several aspects of aesthetics, particularly those regarding the integration of the bridge into the site and the form and proportions of the bridge, are touched on below. Consideration of these notions during the conceptual design phase avoids errors that could have a negative influence on the aesthetic qualities of the bridge. More comprehensive information on the aesthetics of bridges is given in a number of works covering the subject, for example [4.3 to 4.12].

The aesthetic aspects of a bridge should be taken into account from the very beginning of the studies. It is wrong to think that a few last minute changes to details can bring the harmony that is necessary, as it is the overall lines of the bridge that dictate whether the visual experience is positive or negative. It is true that aesthetic considerations will have an effect on the economy of the bridge, but any increase associated with this aspect is likely to be minor (1% to 2% of the total cost), unless a particularly extravagant aesthetic concept is desired.

Integration into the Site

A bridge is normally designed to last one hundred years in a natural or manmade environment. Its presence makes a lasting mark that shows the evolution of society. It should therefore be an agreeable addition to the local environment. Therefore, one of the first qualities a bridge must exhibit concerns how it integrates into its surroundings. It must not upset the equilibrium of the surroundings; it must either adapt to or indeed reinforce this equilibrium. Depending on the type of bridge and the site, it integrates (is subordinate to the surroundings), or it makes a strong statement. In the latter case the bridge must possess an intrinsic beauty that works within its surroundings. It is not acceptable to create a bridge that is spectacular in itself but spoils its local environment.

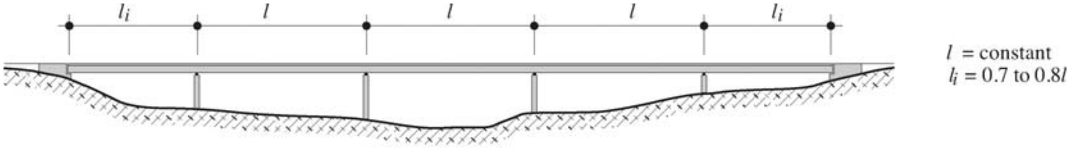
To be in harmony with its surroundings, a bridge must not only possess harmonious proportions, but also appear to be in equilibrium; its structure must demonstrate order and coherence. In tranquil surroundings these proportions should allow the bridge to blend in through its lightness and transparency, whereas in other cases, a bridge can be used to underline and reinforce more spectacular surroundings. The designer must take seriously this aspect of integration with the surroundings. A priority should be placed on viewing the prospective site from a number of perspectives. Consideration of how well a bridge will integrate can be given effectively using photo-montage or graphical software tools.

Form and Proportions

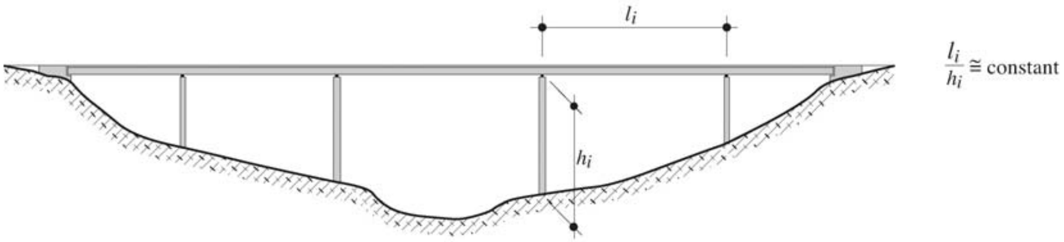
The choice of harmonious proportions between the different elements gives a bridge its qualities of transparency and lightness, rather than it appearing excessively massive. The proportions are a function of the relative size of the structural elements and their repetition in space, the interactions between solid and open surfaces, the interaction between the dimensions of the piers and deck, between the thickness of the deck and its span length, the width and height of the piers, and the interplay between light and dark surfaces, shadow and sunlight.

Bridges should demonstrate order, equilibrium and regularity. Abrupt changes in the dimensions of the structural elements, cross sections, or spans should be avoided. Numerous changes in alignment of the structural elements, stops, breaks or discontinuous curves in the lines of the bridge give an appearance of disorder. The basic philosophy must always be to search for regularity and symmetry. In the case of a bridge that is a constant height above ground, it is desirable to have equal spans from an aesthetic point of view just as much as from a construction point of view (Fig. 4.5(a)). When the height above ground is diminishing, then a gradual reduction in spans has a favourable effect (Fig. 4.5(b)). Symmetry, spans in equilibrium, and similar cross sections for the beams and piers give order, simplicity and clarity, and give the bridge as a whole unity both homogeneity and harmony. An observer should be able to identify intuitively the load paths within the structure.

The choice of spans and their variation is important. The length of the spans dictates the depth of the beams, which could form obtrusive screens for bridges near to the ground, if the ratio between the height above ground to beam depth is less than four. For example, this implies that for a beam depth that is 1/20



(a) Bridge of constant height above ground



(b) Bridge of variable height above ground

Fig. 4.5 Forms and proportions of beam bridges.

of the span, on a bridge that is situated on average 8 m above the ground, the maximum span is 40 m. Also for bridges close to ground level, both square openings and rectangular openings (elongated in the vertical sense) are considered unsightly.

For bridges that are high above ground level, the depth of the beams, and therefore the dimensions of the spans, are of less importance. The slenderness of the deck is defined in terms of the relationship between the visible length of the bridge and the visible depth of the deck, which is influenced by the width of the deck and the position of the observer. For long spans (> 80 m), beams of variable depth look more aesthetically pleasing than straight lines, which can have a rigidity in conflict with nature. Piers that are tapered, rather than having parallel sides, have a greater lightness and elegance.

If the bridge is long and the deck wide (> 12 m), then two piers per support line are reasonable; unfortunately, this can appear, from certain points of view, to be a forest of piers, and this impression is even greater with two parallel bridges. This could lead the designer to propose piers that are as slender as possible; however, such a response can undermine the perception of a bridge's safety and stability.

When a bridge only has a few spans, then it is better, from an aesthetic point of view, to make these an odd number rather than an even number. For example, when looking at a bridge with two spans, the eye is first drawn to the central pier, to the detriment of seeing the bridge as a whole. Therefore, wherever possible it is preferable to conceive a bridge (be it a beam bridge or one with inclined legs) with three spans (Fig. 4.6).

For a bridge to be considered beautiful, it is also important that its detailing is carefully carried out, particularly regarding visible details. For example, the look of a bridge can be ruined by water stains or other dirty marks that result from failure of either the deck's water-proofing or the water evacuation system. Post-construction fitting of elements such as anti-noise barriers, pedestrian walkways, or ducts (for water, electric cables, fibre optics, etc.) can have a detrimental effect on a bridge's lightness.

Of equal importance are the choices of colour for the steel elements, the texture of the concrete elements such as the edges and parapets, and the faces of the abutments and piers. For steel bridges that are to be painted, matt colours are preferable to those with a gloss finish, as the latter tend to highlight any inaccuracies in the flatness of the plates, etc. The natural colours of the materials are often preferable to artificial colours.

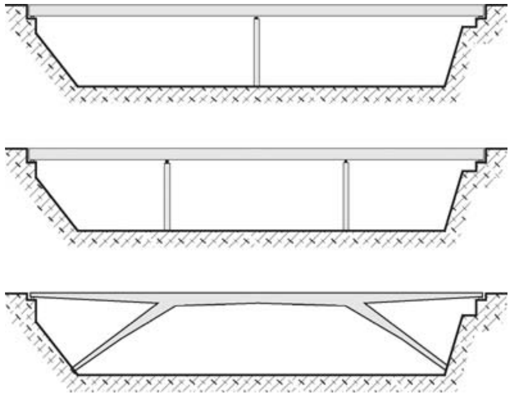


Fig. 4.6 Aesthetics of two and three span bridges.

The aesthetics of a bridge cannot be formulated in terms of rules. For any given situation several aesthetic solutions may exist, all of which may be equally defensible. It is up to the designer to search these out, using creativity, intuition, and taste for beauty. The few principles given here may be useful to help avoid certain errors when it comes to getting the aesthetics right.

4.4.5 Economy

Choosing the right type of structure, number and length of the spans, and its location can positively influence the cost of a bridge or viaduct. In the case of a project that is part of a new communication line, and which is the result of a compromise between often conflicting requirements (optimisation of the line, protection of the natural and built environments, impact study), the location of a bridge is determined by this line. However, this location is normally open to minor adjustments that can improve the economy of the project. For example, minor adjustments to the location may profit from improved foundation conditions, or modifications to the geometry may facilitate the execution method or reduce the skew. Clearly, any decisions must be made at the very start of the project's elaboration.

The biggest project savings come from a good choice of bridge type and its general configuration. Evaluations should be made considering a number of options. The savings to be made during this conceptual design phase are much more significant than those that can be achieved through subsequent structural design calculations. The choice of bridge type should be based on all the functional requirements, as well as those imposed by the surroundings. Both the total length of the bridge and its longest span play a determining role in this choice and in the options for execution. The total length of the bridge affects the volume of any embankments and the dimensions of the abutments. Increasing bridge length is sometimes both more economic and more aesthetically pleasing than adopting large abutments.

The choice of spans should take into account the foundation conditions, the options for execution, and the aesthetics of the structure. During the conceptual design phase, the engineer must consider the choice of longitudinal and transverse structural forms, which are a function of the length of the bridge, the slenderness of the piers, and the foundation conditions. Configuration of the cross section should take into account the intended bridge use, the span dimensions, and the options for execution and aesthetics. Any planned widening of the bridge in the future should also be taken into account during this phase; investing a little more can be very advantageous when it comes to future modifications.

Good construction details chosen early in the project elaboration that take into account execution, durability, and facilities for maintenance (inspection, upkeep, refurbishment, etc.) can also have a determining effect on the project economy. Anything forgotten or insufficiently considered, or decisions made in haste during execution tend to reduce the quality of the bridge and increase costs.

The cost of a bridge is not simply the cost associated with its design and execution. Additional costs accompany its use and maintenance, the cost of money (interest, amortization, inflation), and the cost of demolition and recycling (life cycle costs).

Other variables from the general economic situation can influence the economy of a project. This is particularly relevant for the costs of materials and construction, which are influenced by the business of construction companies and their resources.

From an economic point of view, it is generally preferable to choose solutions that are simple and well known. It may be beneficial to increase the amount of material in order to decrease the amount of work (for example, by increasing the thickness of a beam web in order to reduce the number of stiffeners), but this depends on the relative costs of material and labour. Any factor that contributes to a structural concept that is both simple and proven helps to guarantee ease of execution, durability and economy. If experimenting with innovative technical solutions, then the advantages and possible downsides must be critically examined and the choice justified for the particular solution.

Finally, if the project is to be delivered on time and to cost, the organisational structure needed to realise a bridge project, in particular the methods for dissemination of information and decisions, is essential.

Regarding certain economic aspects of bridge construction: on average, the relative costs of freeway roads, bridges, and tunnels are one to three to five. Annual maintenance costs are in the order of 1.0% to 1.2% of the construction costs. The division of construction costs between the different parts of a bridge depends on its location, the complexity of the structural form, and the foundation conditions. For a composite steel-concrete bridge with average spans, excluding the design and management costs, the following ranges can be assumed:

- Substructure: 25% to 40%
- Superstructure: 40% to 60%
- Site Installations: 6% to 8%
- Other Components: 10% to 15%

4.5 Choice of Materials and Their Properties

The choice of materials for a bridge is of primary importance in order to assure its durability within a reasonable maintenance regime. The right choices of materials and material characteristics depend in particular on:

- The facilities available for fabrication, transport, assembly and erection of the construction elements. For example, the choice of a low strength grade of steel will lead to structural members that are voluminous, therefore heavy, more difficult to transport, and possibly requiring large volumes of weld for their assembly.
- Steels with only average weldability may require special precautions during the execution of any site welded joints.
- Concrete that is badly specified may result in difficulties during placement.
- The risk of brittle fracture affecting the performance of the structure.
- The ability of the structure to resist the effects of degradation with time, for example, the corrosion resistance of the steel and the concrete's resistance to chemical and atmospheric agents.

Section 4.5 considers the choice of steel for bridge construction and its corrosion protection. As far as the choice of concrete for the foundations, piers, abutments and slab is concerned, reference should be made to TGC Volume 10, Paragraph 3.3.4 and TGC Volumes 8 and 9.

4.5.1 Steel Grade and Quality

The principles of steel making, as well as a description of the different products used in steel construction, are given in TGC Volume 10, Section 3.2. The mechanical characteristics of steel, and the main tests used to determine these characteristics, are also described in TGC Volume 10, Paragraph 3.3.1. Structural steels are particularly characterised by:

- The ***grade***, which is defined by the yield strength (S355 for f_y = 355 N/mm^2),
- The ***quality***, characterised by the resistance of the steel to bending by impact (according to a resilience, or Charpy V-notch, test) as an indicator of the resistance to brittle fracture; to some degree the quality may also give an indication of the weldability of the steel.

When choosing steel for use in a bridge, its quality is of particular importance, as discussed below. Much progress has been made in recent years concerning the process of plate rolling, and this has resulted in steels (known as thermomechanically rolled steels) that are interesting in terms of their fabrication, their weldability and their resistance to brittle fracture.

The designation of steels is based on the system defined in the European standard EN 10 027-1 [4.12]. The different products used in the construction of steel bridges are defined in the following European standards:

- EN 10 025-2: Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy structural steels [4.13],
- EN 10 025-3: Hot rolled products of structural steels – Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels [4.14],
- EN 10 025-4: Hot rolled products of structural steels – Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels [4.15],
- EN 10 025-5: Hot rolled products of structural steels – Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance [4.16].

The same steel definitions are used in the 2003 edition of SIA 263.

Steel Grade

For rolled products the steel grade is defined by the yield strength, expressed in N/mm^2, corresponding to a plate that is less than 16 mm thick. This number is preceded by the letter S (initial for the English word *Structural steel*) in order to distinguish structural steels from other categories of steel, such as those used for mechanical applications (letter E from ***Engineering** steel*) or steel used in reinforced concrete (letter B from *reinforcing **Bar***). The yield strength f_y as defined in the standards (a value which corresponds to the upper limit of yielding R_{eh} measured in a standardised tension test) is a guaranteed minimum value for the steel.

In the world of bridges, it is very common to use plate girders, and the reduction in yield strength must be taken into account as the thickness of the plates used to form the girders increases. This reduction is related to the fact that for thicker plates the rolling temperature is higher and the cooling less rapid. This leads to less regular graining of the steel. Table 4.7 shows how yield strength reduces as a function of plate thickness and grade of steel according to EN 10 025. Note that for plate thicknesses greater than 80 mm, this table is not applicable for thermomechanically rolled steels, which can achieve higher yield strengths [4.15].

Table 4.7 Yield strength f_y in N/mm^2 as a function of plate thickness.

	$t \leq 16$ mm	$16 < t \leq 40$	$40 < t \leq 63$	$63 < t \leq 80$	$80 < t \leq 100$	$100 < t \leq 150$
S235	235	225	215	215	215	195
S355	355	345	335	325	315	295
S420	420	400	390	370	360	340
S460	460	440	430	410	400	380

The SIA standards offer a simplified grading of the reduction of yield strength with plate thickness:

- Up to $t = 40$ mm, use f_y for $t \leq 16$ mm as given in Table 4.7,
- For t between 40 mm and 100 mm, use f_y for 40 mm $\leq t \leq$ 63 mm as given in Table 4.7.

When steel grades S420 and above are used in bridges, they are normally referred to as "high strength steels" (HSS).

Steel Quality

The notion of steel quality is used to define the particularities of the material's resistance to bending by impact of a test specimen containing a notch (Charpy test), which is an indication of its resistance to brittle fracture. This type of failure must be avoided because it can occur at low temperatures, even when the loading is relatively light. It is a form of failure associated with almost no plastic deformation. The risk of brittle fracture of a structural element is heightened when:

- there is a presence of high tensile stresses, or multiaxial tensile stresses; this aspect of multiaxiality increases with plate thickness (state of plane strain),
- there is a structural defect which provokes strain concentrations. This could be caused by anomalies in the weld (TGC Vol. 10, § 7.3.4) or a sudden change in section,
- there is a presence of high tensile residual stresses,
- the rate of load application, and therefore strain rate, is high,
- the service temperature is low,
- a high steel grade is used (for a given steel quality).

Some or all of these conditions are often present in steel bridges; therefore, careful consideration must be given to the phenomenon of brittle fracture.

The behaviour (resistance) of steel with regard to brittle fracture is expressed in terms of its ***toughness***, or capacity to absorb the energy needed to propagate a crack. Toughness is defined by the critical value K_c of the stress intensity factor, which is a material constant (TGC Vol. 10, § 13.3.5). This notion of toughness comes from the fracture mechanics theory, which allows the stresses in the region of the crack tip to be analysed and to determine how harmful the crack is. When the stress intensity factor K (being a function of the geometry and dimensions of the crack as well as the applied stresses) exceeds K_c, and the conditions are such that the material is below the transition temperature, then brittle fracture will occur. The toughness

K_c of steel can be determined using a test specimen, which is cracked by fatigue prior to testing. However, such tests require a considerable time investment and therefore are rarely administered.

To classify a given steel in terms of its resistance to brittle fracture at an industrial scale, one uses a bending test by impact, or resilience test (TGC Vol. 10, § 3.3.1), which allows measurement of the rupture energy of a standardised specimen with a V-notch (Charpy impact test). This rupture energy, or resilience, is expressed in Joules or Joules/cm^2 and is a function of the temperature and rate of load application. Several empirical relationships exist between the resilience and the toughness of steel (TGC Vol. 10, § 13.3.5). By taking into account the service conditions of a bridge in its environment and knowing an appropriate empirical relationship between resilience and toughness, and for a defined critical flaw size, it is possible to calculate, for example, the maximum plate thicknesses that may be used (see Tab. 4.9, § 4.5.4).

Standards define steel quality classes by fixing a minimum, or guaranteed, value of rupture energy (in bending by impact) for a specified test temperature. The quality is designated using letters and numbers as indicated in Table 4.8.

Table 4.8 Definition of steel quality according to EN 10025.

Standard	Notation	Rupture energy in Joules	Test temperature in °C
EN 10 025-2 Non-alloy steels	J	27	
	K	40	
	R		+20
	0		0
	2		–20
EN 10 025-3 Fine grain steels	–	40	–20
	L	27	–50

As an example, steel quality K2 indicates a guaranteed value of 40 Joules at –20 °C. Steels generally used, in increasing order of quality, are JR, J0, J2, K2 for non-alloyed steels and N or NL for fine grain steels (M or ML for thermomechanical steels). The weldability of the steel improves from quality class JR to J2.

Delivery State

Steels of quality J0 or less are delivered untreated after rolling and are identified as **+AR,** according to EN 10 025-2. If they have undergone a process of normalisation, then they are identified by **+N** in this standard. The normalisation process aims to regularise the structure of the metal and refine the grains to improve the tensile resistance and toughness of the material. By definition, fine grain steels, according to EN 10 025-3, are all delivered in a normalised state and identified by the letter **N**. All steels used in bridges are generally normalised. So-called thermomechanical steels are subject to a specific rolling process (see below) and are identified by the letter **M** according to EN 10 025-4. These symbols are added to those defining the steel quality.

4.5.2 Weldability

Weldability is not a mechanical characteristic of steel like tensile resistance or resilience; it cannot be quantified, but is rather based on a qualitative judgement of the aptitude of the metal to be welded to another piece via an intermediary metal (electrode). An indicative measure of the weldability of a metal is given by its carbon equivalent value; steels with low carbon equivalent offer improved weldability.

Welded joints made of steels possessing good weldability can possess the same characteristics as the base metal. To obtain this quality it is particularly necessary to avoid brittle areas of weld in the heat affected zone (HAZ). Without going into details, the likelihood of brittle zones increases when the carbon equivalent of the base metal increases, and there is rapid cooling.

A lack of weldability manifests itself primarily by cracking in the HAZ, known as cold cracking. The likelihood of such cracking increases in brittle zones, when there is a presence of tensile stresses and hydrogen. These conditions may be found:

- if a steel with a high carbon equivalent is cooled too rapidly after welding, then brittle zones may appear; cooling of the HAZ is more rapid when the plates to be assembled are thicker because of the greater thermal conduction,
- when welding results in residual stresses (TGC Vol. 10, § 7.3.3) in the HAZ due to restrained thermal movements; these tensile residual stresses will be greater when the plates to be assembled are thicker,
- when hydrogen is present in the weld due to moisture in the flux of the electrodes or, to a lesser extent, humidity in the atmosphere.

The three factors noted above may coexist during the fabrication of girders from thick plates and, particularly, when executing welded joints on site.

The following precautions are generally taken to avoid cold cracking:

- Preheating by blowtorch (or combined series of torches) of the zone around the joint prior to welding to reduce the cooling speed of the HAZ. On site, the regions of welded joints are sheltered in order to maintain a higher ambient temperature and provide protection from the wind,
- Developing weld processes that reduce the magnitude of residual stresses,
- Avoiding hydrogen getting to the weld during fusion by drying the electrodes in an oven prior to use.

Preheating and, above all, postheating of the welded zone have a beneficial effect on the elimination of hydrogen from the HAZ. Preheating is always needed for steel grades S355 and above; however, due to their low carbon equivalent, thermomechanical steels, as described below, do not need preheating.

4.5.3 Thermomechanically Rolled Steels

Thick plates destined for fabrication into main beams are generally delivered in a normalised state. This is a stable state obtained by annealing and/or rolling to give characteristics to the final product that are essentially linked to the chemical composition of the steel.

The recent development of very powerful rolling machines and equipment for accelerated cooling, accompanied by systems to permit accurate process control, has allowed rolling phases to be developed which result in the direct production of plates with the required mechanical properties without needing to resort to subsequent thermal treatments. This is known as thermomechanical rolling [4.17].

Steels that are manufactured using this process, which has seen commercial application since the beginning of the 1990s, possess an optimal structure not only regarding the fineness of the grains, but also their morphology and distribution of the precipitates. Thermomechanical steels differ from traditional normalised steels in the following ways:

- for the same mechanical properties, they require less carbon and other hardening elements (lower carbon equivalent value),
- for the same chemical composition, they have superior mechanical properties.

The chemical composition of thermomechanical steels gives them a very low susceptibility to cold cracking after welding, and this leads to a simplification of weld procedures. In particular:

- for the majority of cases, preheating in the area of the connection to be welded is not needed, provided the ambient temperature is above 5 °C,
- in most cases there is no need to respect a minimum temperature between weld passes.

These simplifications concerning welding procedures for thermomechanical steels result in certain advantages concerning:

- the cost of the weld material,
- the cost of operations concerned with heating, and the control of temperature,
- the cost of remedial work,
- the conditions required when welding outdoors,
- the quality of the weld in terms of cold cracking.

The result of these advantages is an easier use of high strength steels (S460M) in plate girders, in particular in regions where the structural members are highly stressed.

4.5.4 Steels Typically Used in Bridge Construction

The steel most commonly used for key structural members such as main beams is grade S355, delivered in a normalised state. It is designated S355J2 + N or J355K2 + N for non-alloyed steels (EN 10 025-2), and S355N or S355NL for fine grain steels (EN 10 025-3). When thermomechanical steels are used, they are designated S355M or S355ML (EN 10 025-4). Higher strength steels (S460) are of interest in highly stressed regions of continuous beams, such as over intermediate supports. In such locations a reduction in flange thickness of up to 30% can be achieved compared to when the flanges are formed from S355N steel. In addition, the volume of weld material needed to join the flanges is reduced. This allows improved productivity during welding. Steel grades inferior to S355 are not used in the construction of bridges, except perhaps for secondary elements that are only lightly stressed.

Regarding brittle fracture, by applying the methods of fracture mechanics, it is possible to define maximum allowable plate thicknesses as a function of the bridge use and environment (considering the stresses acting on the plate and its temperature). Table 4.9 gives values of maximum plate thickness for different grades and qualities of steel, for different stress levels σ_{Ed}, and for two levels of atmospheric temperature T_{Ed}. This table is taken from EN 1993-1-10. In this approach T_{Ed} is the temperature that governs the toughness of the material, which is taken as an accidental action with its lowest value at the location being considered. The stress σ_{Ed} is taken as the value accompanying this accidental action, and can be assumed equal to the value for frequent actions (TGC vol. 10, eq. 2.23). It is possible to use linear interpolation for values within the bounds given in Table 4.9. Steel producers or distributers should be consulted for guidance on available plate thicknesses.

Table 4.9 Maximum thickness t [mm] according to EN 10025, as a function of the temperature T_{Ed} and the stress σ_{Ed} in the plate.

Standard	Grade	Quality	T_{Ed}			
			0 °C	30 °C	0 °C	30 °C
			$\sigma_{Ed} = 0.50 f_y(t)$		$\sigma_{Ed} = 0.75 f_y(t)$	
EN 10025-2 Non-alloy steels	S235	J0	105	65	75	40
		J2	145	90	105	60
	S355	J0	80	45	50	25
		J2	110	65	75	40
		K2	135	80	90	50
EN 10025-3 and 4 Fine grain steels N (normalized) M (thermomechanical)	S355	N, M	135	80	90	50
		NL, ML	180	110	130	75
	S460	N, M	110	65	70	40
		NL, ML	155	95	105	60

$f_y(t)$: Yield strength as a function of plate thickness, according to Table 4.7.

4.5.5 Corrosion Protection of Steel

To assure the durability of a bridge, it is necessary firstly to protect the steel structure against corrosion, and secondly to maintain this protection regularly. Corrosion can take place when the relative air humidity is around 60%, or less when chlorides are present in the atmosphere. The speed of corrosion is a function of the aggressivity of the environment. Two main methods are used to protect steel bridges against corrosion:

- protection using paint,
- use of steel with improved anti-corrosion characteristics, known as weathering steel.

The use of paint is the most common form of protection, even today, and despite the fact that steels with improved resistance to atmospheric corrosion (*weathering steels*) have been available for a number of years. These can be used instead of any paint. Whatever solution is used to avoid corrosion, it is essential that regular checks are made, especially in the early years of a structure's life, to confirm that either the paint protection is effective or that the surface layer of the weathering steel is developing correctly.

As far as protection of any bridge is concerned, it is worth remembering the fundamental principle that any water present must be allowed to run off without obstruction. When configuring the construction details, all areas that could constitute a "moisture trap" (see Sect. 6.2) should be avoided. Examples include significant horizontal surfaces, edge upstands, slots, corners or angles in which water could stagnate and not evaporate easily. Holes and cut outs, for example in the stiffeners, should be incorporated in order to allow flow of water.

Protection by Paint

Paint systems used to protect steel against corrosion general comprise:

- a base layer,
- an intermediate layer,
- a finishing layer.

The base layer protects against corrosion and facilitates adherence of the subsequent layers. It makes use of pigments, for example zinc chromate or zinc powder, to inhibit the corrosion process. This layer is applied in one or two passes in the fabrication shop. To ensure that the base layer sticks well, the steel surface must be carefully cleaned. It is important to clear the surface of all traces of dirt, rust, carbon deposits, oil, grease and weld splashes. Typically, steel shot is used in the fabrication shop to blast the surfaces and achieve this cleanliness. Shot blasting can achieve different degrees of surface preparation. Normally preparation Sa 2 1/2 according to [4.18] is required. This involves a careful scouring of the surface, which should present, after shot blasting, a metallic colour that is more or less uniform. This operation results in a roughened surface to which the base layer may be applied.

Prior to the application of the subsequent layers, the length of time during which the base layer can be exposed to the elements, while still retaining its properties, varies between 6 and 18 months, depending on its type and thickness. Care should be taken to maintain this layer over areas that are affected by the erection process (bolts, welds), including any areas of paint that are damaged during transportation or during erection itself.

The main purpose of the intermediate layer is to increase the total build-up (thickness) of the paint protection. Given the current cost of labour, there is a trend to using fewer layers, and increasing the thickness of the other layers eliminates the need for the intermediate layer.

The finishing layer, which is normally applied on site, must be compatible with the layer(s) beneath it. It must also be unaffected by the effects of the atmosphere (UV); to achieve this it sometimes contains lamellar pigments, such as micaceous iron oxide or flakes of aluminium. The finishing layer also plays a decorative role and gives the coating its surface properties, such as colour, lustre and texture.

Both applying the complete coating in accordance with good practice and obtaining the correct thickness of covering are essential to achieving the desired level of protection. The type of coating and thickness required are a function of the atmospheric exposure of the structure and the atmospheric conditions (categories of corrosivity C1-low to C5-very high). The total thickness of the coating will vary between 120 and 300 μm; the thickness for a layer is 30 μm minimum and can be up to 60 μm for a single pass. Document B3, edited by SZS, and the technical guidance SIA 2002 [4.19] contain additional information on the coating thickness necessary as a function of the aggressiveness of the environment. They include consideration of the different types of paint, conditions for application, and control of the process. In the European context protection by paint is covered by the standard SN EN ISO 12944 [4.20].

The expected life of a coating, before either total or partial replacement, will be at least 30 years, provided the coating was properly applied in the first place and regular inspection has allowed areas of local damage to be repaired. Partial or total replacement of the coating requires a serious amount of both work and equipment to ensure that no environmental damage occurs during removal of the old layers of paint.

Weathering Steels

Unlike traditional steels, those that are lightly alloyed (P, Cu, Cr, Ni, Mo) present a good resistance to atmospheric corrosion. This improved resistance to corrosion is due to the formation of a compact self-protecting oxide layer (the patina) on the surface of the material. This layer is practically waterproof, adheres well, and is tough, being of a dark brown colour and prohibiting oxidation of the remaining material. Such steels, therefore, do not require paint to protect them. They are identified by the letter **W** according to [4.16]. The patina develops very quickly once the material is exposed to the atmosphere, then the speed of corrosion reduces down to practically zero after several decades.

Figure 4.10 shows the development of the thickness of the patina for coupons of plate exposed over a number of years [4.21]. This clearly shows the trend for the layer to build up rapidly at the beginning of the

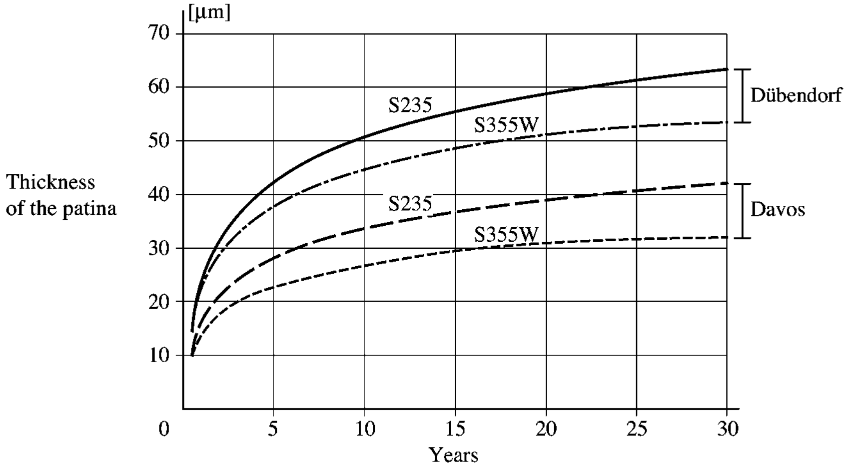


Fig. 4.10 Examples of the thickness development of a weather resistant patina.

exposure, and that this rate of build up slows to practically zero for weathering steels. It is also clear that the thickness of the oxide layer is greater in a polluted environment (Dübendorf) than in an Alpine environment (Davos). The curves also show that the difference in the development of the oxide layer (at least for these locations in Switzerland) is not that significant between weathering steel and traditional steel. However, the difference is that for traditional steels, the process of corrosion does not stop.

In Switzerland, weathering steels first started to be used for bridges in the 1970s. While observations of existing structures [4.21] have highlighted several conceptual errors in the use of such steels that should be avoided in the future, they have also confirmed that corrosion does effectively diminish with time. However, the real loss of steel due to corrosion is difficult to estimate with any precision, because it is strongly dependent on the aggressiveness and characteristics of the environment. Indicative values over the first 10 years of exposure are that the loss from each exposed face is of the order of 0.04 to 0.10 mm in a rural environment, and 0.15 to 0.25 mm in a very aggressive industrial environment. The average reduction per decade over the first 50 years will be of the order of 0.05 mm. Certain codes and standards, for example DAST [4.22], suggest increasing the nominal thickness of plates in order to allow for this loss of thickness with time (particularly for thin plates used as part of main beams).

One of the essential requirements to assure the correct development of the protective patina is alternate dry and wet phases. The continuous presence of water or general humidity on the steel structure over long periods results in pronounced corrosion in the areas concerned. Another important condition to be taken into account is that the presence of chlorides in the atmosphere is incompatible with the use of weathering steels. When considering the points noted above alongside experience gained from existing bridges, the correct use of weathering steel should comply with the following rules:

- Environment
  - do not use weathering steel in locations less than 500 m from the sea or in regions where the influence of the sea is significant (fog or wind blowing from the sea),
  - do not use weathering steel in regions with fog likely to contain salt (for example, a bridge crossing a heavily trafficked freeway that is frequently subjected to de-icing salts),
  - do not use weathering steel less than 1 m above ground level (vegetation) or less than 3 m above a river,

   - do not use weathering steel in regions that are directly affected by particularly aggressive industrial environments.
- Construction details to be used with weathering steel
   - plates must be greater than 5 mm thick; for the web and flanges of main beams, they should be at least 10 mm,
   - the layout of elements should allow sufficient ventilation of the exposed surfaces (Fig. 4.11(a)),
   - the layout of each element should avoid the run-off of iron oxides on to the neighbouring elements, for example by using returns that prevent rusty water running directly over the surface of bridge elements, particularly concrete piers, supporting beams made from weathering steel (Fig. 4.11(b)),
   - piers should also be protected from run-off of rusty water during bridge construction.

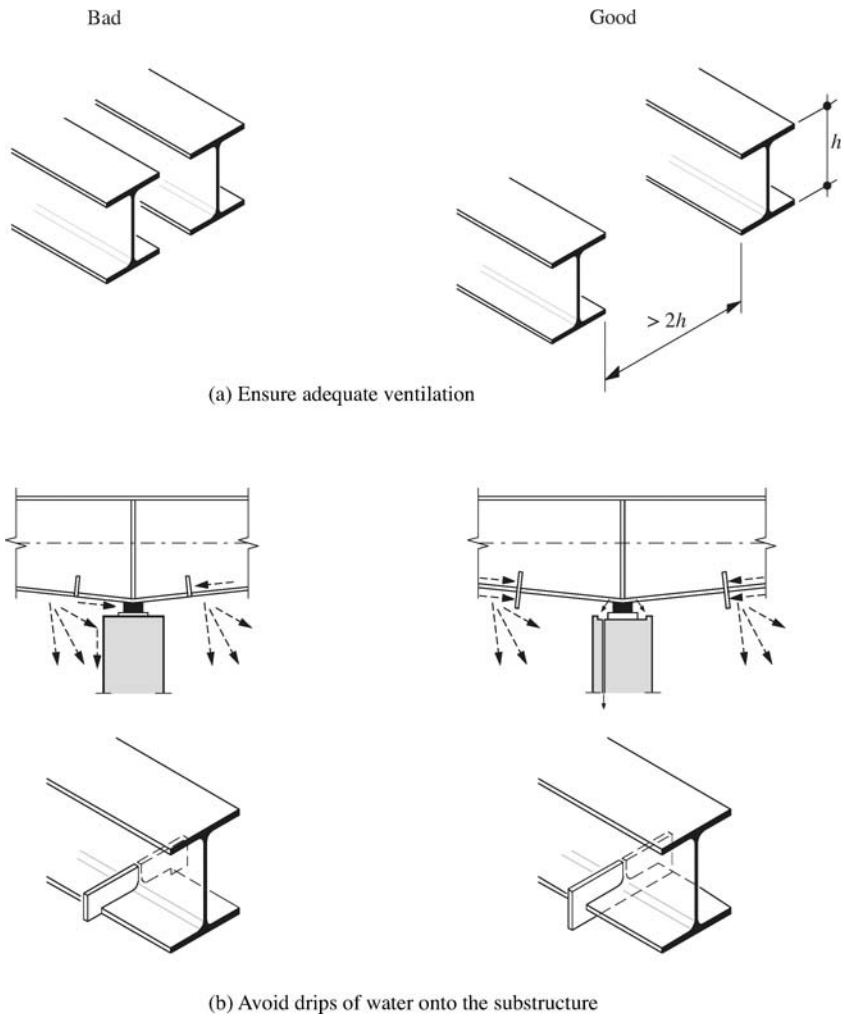


Fig. 4.11 Examples of construction details appropriate for use with weathering steels.

- Appearance
  - in order to obtain a uniform patina, it is important that the surfaces are carefully shot-blasted,
  - for composite bridges, any concrete that splashes or runs onto the exposed steel plates during construction of the slab must be cleaned, or, even better, should be avoided in the first place,
  - condensation, which can appear on certain parts of the steel structure, can lead to a variable surface colour and texture (nevertheless, it is impossible to avoid this problem using simple measures).
- Inspection
  - an inspection regime that is appropriate for the state of the surfaces must be implemented, notably in order to clean the steel structure (accumulation of dirt and dust, vegetation), to avoid long term exposure to standing water, and to manage, and thereby assure, the correct functioning of ducts for water evacuation (notably inside box girders).

Cost studies have shown that the use of weathering steel may already be beneficial during the execution phase (saving up to 10%). The extra cost of weathering steel compared to traditional steel, even if extra material is taken into account, is already compensated for by the savings in the cost of initial painting. This advantage is even greater if one takes into account the costs of maintenance and renewal of the paint during the service life of the bridge. The use of weathering steel is even more justifiable when access to the steel structure for re-painting is difficult. Clearly, the particular measures that must be taken to avoid polluting the environment and to protect the workers during cleaning and painting are also avoided when weathering steel is used.

From an aesthetic point of view, the dark brown colour of the protective patina is generally well accepted, particularly in rural environments. Finally, the lifecycle impact of the bridge is reduced when weathering steel is chosen over paint-protected steel [4.21].

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5 Structural Forms for Bridges

Truss girders composed of tubular members for the Viaduc de Lully (CH).
Eng. Bureau d'ingénieurs DIC, Aigle.
Photo Bureau des autoroutes, canton de Fribourg.

5.1 Introduction

A bridge is a structure that allows one or more lines of communication to cross an obstacle, such as a valley, a river or other lines of communication. Its primary function is to support the traffic actions relating to the lines it is carrying, and to transfer them to the foundations. In addition to the imposed loads, the structure must also support its own self-weight and that of any additional components it carries. It must also be able to resist any actions coming from its surrounding environment, notably wind, snow, temperature changes and those resulting from seismic events.

The bridge picks up and transfers vertical loads and horizontal forces by means of its *structural form*, which is conceived to be durable in resisting these actions, providing adequate structural safety, and meeting the service requirements (Chap. 9). The load carrying structure of a bridge, normally just referred to as its ***structure***, comprises all the elements (Fig. 5.1) that fulfil a function in transferring the vertical loads and horizontal forces to the foundations. A bridge's structure is spatial; therefore, under loading it behaves in a three dimensional way. However, during the design process, from conceptual design right through to detailed design, an engineer will generally break down the three dimensional structure into a number of planar systems. This simplification is justified for many structural forms used for bridges.

The aim of this chapter is to present and explain the main concepts behind different structural forms and the functions that are fulfilled by the various elements comprising the structure. Firstly, Section 5.2 illustrates the way in which actions are transferred from their point of application to the supports. Next, the different structural forms are explained, considering the longitudinal structure (Sect. 5.3) and transverse structure (Sect. 5.4). Because this book is primarily focused on bridges using *main beams*, the configuration

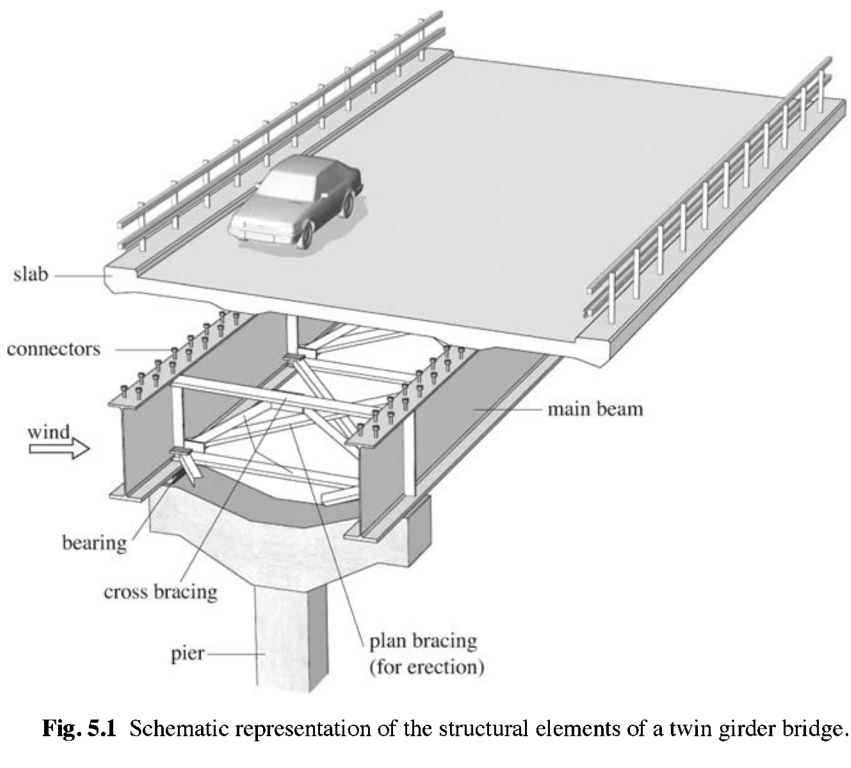


Fig. 5.1 Schematic representation of the structural elements of a twin girder bridge.

and function of their *cross sections* are considered in detail in Section 5.5, and *cross bracing* is considered in Section 5.6. The way in which *plan bracing* is used to transfer horizontal loads to the foundations is considered in Section 5.7.

In order to avoid overloading this chapter, specifics relating to the concept development of structural forms for railway bridges, bridges for pedestrians and cyclists, and arch bridges are given in Chapters 16, 17 and 18, respectively. Several publications related to the concept development of structural forms for bridges are noted in references [5.1 to 5.3].

5.2 Load Transfer

In general a bridge is subjected to a range of both permanent and variable actions. These may be broken down into vertical loads and horizontal forces. The horizontal forces act either along the axis of the structure or perpendicular to it. Figure 5.2 shows the three types of variable action to which a bridge may be subjected:

- vertical actions, either distributed q or concentrated Q, which represent the imposed loads,
- transverse horizontal actions, either distributed q_T to represent forces such as wind, or concentrated Q_T (which might represent the effect of a seismic event),
- longitudinal horizontal actions Q_L to represent, for example, the effect of traffic braking or accelerating, or the effects of a seismic event.

To these variable actions are added the permanent actions (not illustrated in Figure 5.2) which act vertically and represent the self-weight of the structure and items such as the deck surfacing and any additional components such as bearings.

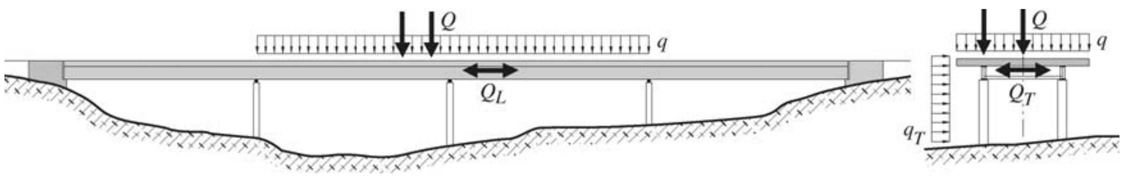


Fig. 5.2 Types of variable actions on a bridge.

The totality of the permanent and variable actions must be transferred to the bridge foundations, at the pier and abutment locations, by the bridge structure. This structure in general comprises a *deck* and whatever is supporting the deck (*piers*, *arch*, *cables*). The structure must be conceived to resist the three types of action described above, and the self-weight.

As explained at the beginning of this chapter, a bridge's structure is spatial, and to facilitate its analysis and design, an engineer will generally break it down into a collection of interrelated planar systems. Figure 5.3(a) shows schematically the beam bridge of Figure 5.1, broken down into a series of planes with two main beams, cross bracing, and plan bracing in the form of the deck.

Below, we consider first the load path for a vertical load Q acting eccentrically on the deck (Fig. 5.3(b)). Because the deck is supported on two main beams, the vertical load is distributed between these beams. More precisely, the transverse distribution of the load into the main beams, via planes B and C, depends on

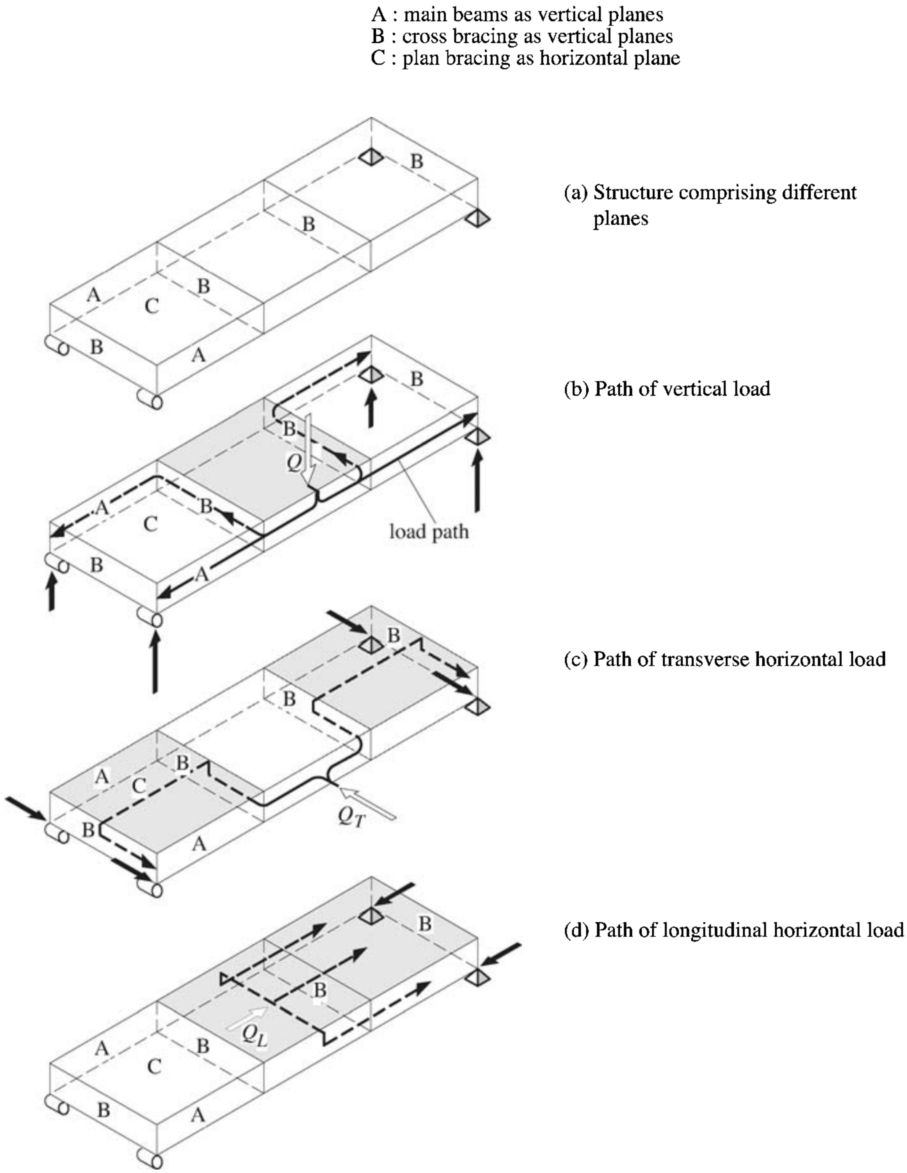


Fig. 5.3 Decomposition of the structure into planes of resistance and load paths.

the cross section adopted for the bridge (Sect. 5.5). It also depends on the torsional stiffness of the elements that form the cross section. The reaction of the deck is transferred by the main beams in planes A acting in vertical bending and shear. The main beams are supported by the piers and abutments which take the vertical load and transfer it to the foundations. In plane A, one talks of the bridge's longitudinal structural form and its structure for supporting vertical loads. Different configurations for the longitudinal structural form are presented in Section 5.3.

Figure 5.3(c) shows the load path taken by a transverse horizontal force Q_T. This force is applied transversely to the beams that are perpendicular to plane A. It is transferred to those elements that stiffen the beams horizontally, such as the plan bracing or the slab in plane C. The transfer of the horizontal force through the plan bracing is partly facilitated by the cross bracing found in planes B. The horizontal forces acting on the plan bracing (which is in a horizontal plane) are transferred to the supports by bending and shear in the bracing. In this plane, the plan bracing is supported by the piers and abutments, which transfer the reactions from the bracing to the foundations. If the plan bracing is not in the same plane as the beam supports, then the reactions in the bracing are transferred to the supports via the cross bracing in planes B, located over the piers and abutments. This eccentricity of the plan bracing relative to the supports results in vertical reactions at the supports (Fig. 5.18), in addition to the horizontal support reactions. In plane C one talks about the transverse structural form and the structure for carrying the horizontal forces (understood to mean acting perpendicularly to the longitudinal axis of the structure). The transverse structural form is considered in more detail in Paragraph 5.4.2.

Figure 5.3(d) shows a longitudinal horizontal force Q_L acting along the axis of the bridge, which acts like a normal force applied to the slab and beams for them to transfer this force to the fixed point of the bridge. This is the fixed point in relation to the longitudinal direction of the bridge, located either directly over an abutment or comprising a number of piers that are fixed to the beam (bridge on flexible piers). The location of the fixed point of a bridge will be considered in more detail in Paragraph 5.3.4.

5.3 Longitudinal Structural Form

The choice and configuration of the longitudinal structure of a bridge are primarily a function of the size of the obstacle to be crossed, the length of the spans, the accessibility of the location, and the possible meth-

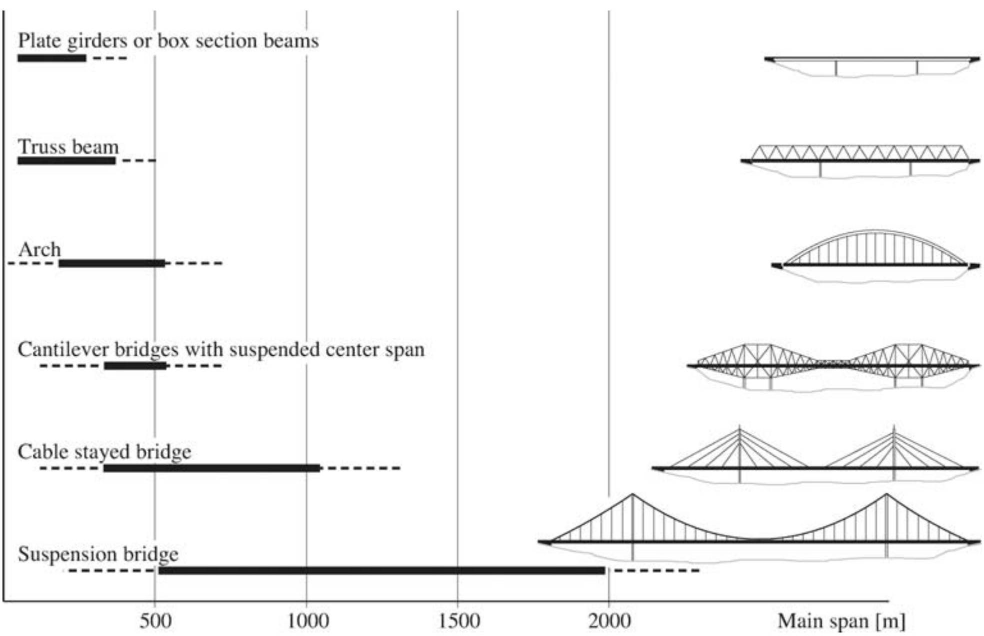


Fig. 5.4 Main types of longitudinal structural form and corresponding main span ranges.

ods of execution. Although for a given span several forms of structure are often possible, it is certainly not the case that all structural forms can achieve the same span range (mainly for technical and economical reasons). Figure 5.4 shows different types of longitudinal structural form, with their respective typical span ranges.

5.3.1 Influence of Span

As the span increases, the effects of the self-weight of a bridge relative to the effects of all the other actions increases. Consequently, the imposed load that a structure can support diminishes as the span increases. The theoretical limiting span for a bridge is that for which the structure can only just support its self-weight, leaving no capacity to carry any imposed loads. This limit is primarily a function of the characteristics of the materials used and the form of the longitudinal structure. However, practical spans remain well below this limit for three reasons:

- the bridge must be able to support imposed loads in addition to its own self-weight and other permanent actions,
- it must be economically reasonable,
- to give an overall pleasing aesthetic appearance, the choice of spans must take into account the height of the bridge above ground, the slenderness of the superstructure (deck), and the location of the bridge in its surroundings (§ 4.4.4).

The span ranges shown in Figure 5.4 for various structural forms are not fixed. They vary with time and from country to country, depending on the relative costs of labour, primary materials, and available technology.

For short span bridges a structural form based on main beams is the most traditional solution. This option comprises beams with a solid web, either plate girders or box girders, supported on piers and abutments and which provides resistance essentially through bending and shear (Fig. 5.5(a)) and, in the case of box girders, additional resistance through torsion. As the span length increases, such beams are less effective due to their significant self-weight and the fact that the cross section is not well utilised in bending except for the extreme fibres (those furthest from the neutral axis). Better use can be made of the material by adopting members that act in either tension or compression. For this reason as spans increase, solid web beams are often replaced by truss beams (Fig. 5.5(b)).

If the obstacle to be crossed is steep sided and deep, or if access is difficult, an interesting option can be an arch from which the deck is supported or suspended. However, this is only the case where ground conditions permit a simple solution for resisting the compression forces at the ends of the arch. Arches are also of interest for simple, relatively short spans, when the deck is suspended from the arch, and the arch is tied (Fig. 5.5(c)). With such a solution the horizontal component of the compression in the arch can be resisted by the deck, which fulfils the function of a tie. A bridge with inclined legs is another interesting structural solution for crossing a valley that is steep sided but of limited width. This form of structure then behaves somewhere between a beam and an arch, because it provides resistance through both bending and compression (Fig. 5.5(d)).

As spans increase structural forms based on the use of cables become advantageous. The deck is suspended by cables acting in tension to transfer the loads into a pylon for cable stayed bridges (Fig. 5.6(a)), or into a primary cable, in the case of suspension bridges (Fig. 5.6(b)). These cable based solutions support the deck in a flexible way, whereas for shorter spans (when main beams are adopted), the deck is supported rigidly on piers and abutments.

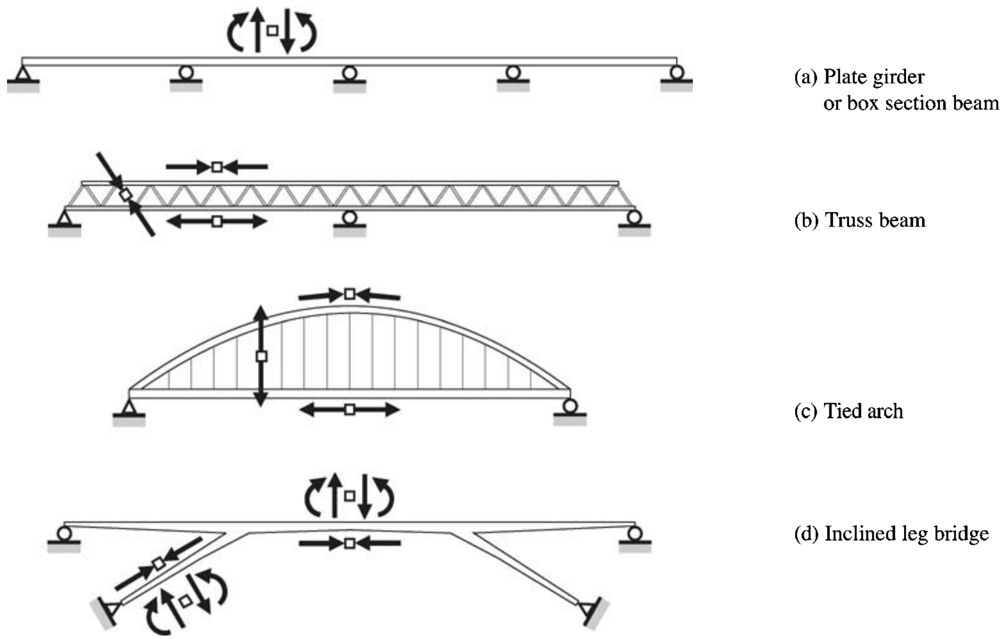


Fig. 5.5 Different structural forms.

The choice of the longitudinal structural form, the length of the spans, and the type of foundations depend on so many different factors that the first choice made by the engineer will not necessarily be the best solution. Several solutions should be considered during the conceptual design phase (structural forms, spans) and during the preliminary studies (§ 4.2.1) in order to identify the best options to take forward for further development.

For the chosen structural form, the engineer should use his experience to evaluate the influence on cost (in terms of cost per square metre of deck) for different span lengths as a function of foundation type. This is simply illustrated in Figure 5.7. The cost of the superstructure increases with span because more

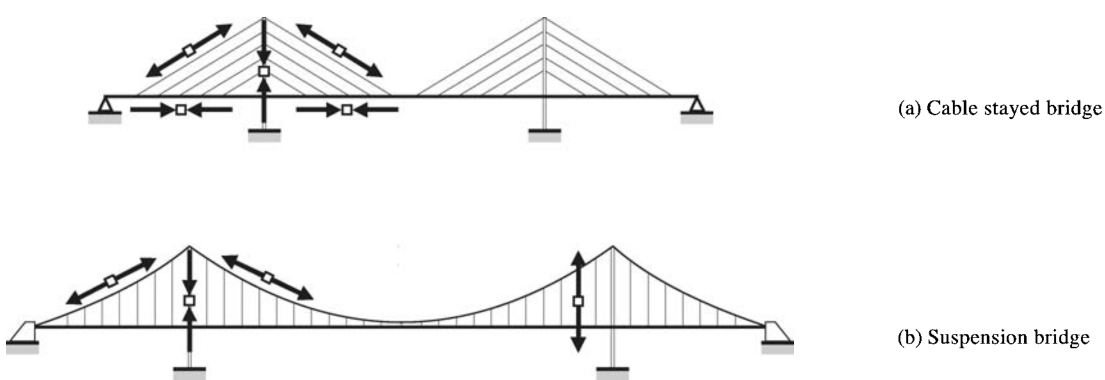


Fig. 5.6 Structural forms with supporting cables.

material is needed to resist the actions. However, the total foundation costs decrease as spans increase because fewer foundations are needed, and a certain percentage increase in their load carrying capacity does not result in the same percentage increase in their cost. By combining the costs of the superstructure and substructure, it is possible to define the most favourable range of spans. This type of reasoning can also be applied to other factors.

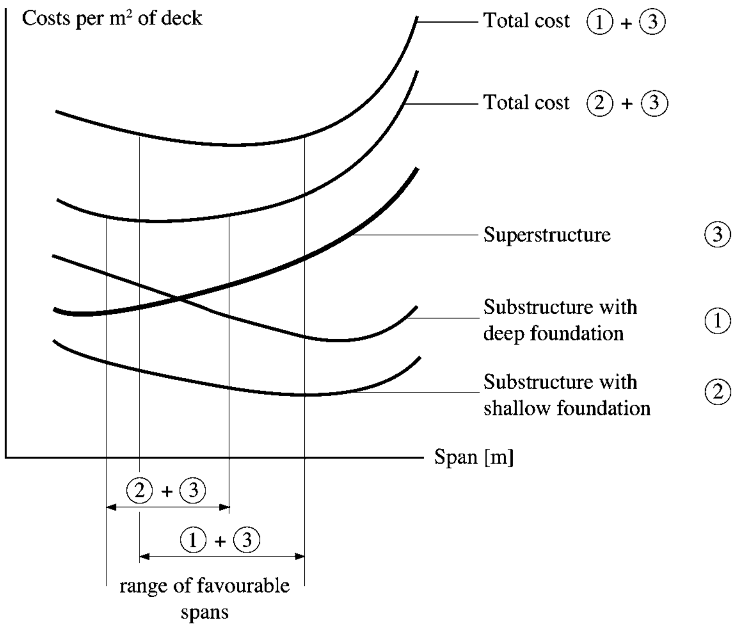


Fig. 5.7 Example of cost variation as a function of span and foundation type.

5.3.2 Plate Girder or Box Girder Beam Bridges

The span range for beam bridges varies from a few metres to 300 m for box girders, and up to 500 m for long-span trusses (Fig. 5.4). Bridges adopting solid web open section (I section) main beams can achieve maximum spans of the order of 125 m. Beyond this, box girders normally become a more advantageous option.

Beam bridges are the most common option for short and medium spans. The bridge cross section may comprise one or several beams (Sect. 5.5), which may be rolled sections, plate girders, box girders, or trusses. The beam transfers the forces coming from the slab to the supports in bending and shear. The beam depth h is linked to the main span l by the ratio l/h, termed the ***slenderness*** of the beam. Table 5.8 lists average values of slenderness for plate girders, box girders, and trusses comprising simple or continuous beams, as used for road bridges.

Typical slenderness values for beams mean that, for road bridges, rolled sections (which have a maximum depth of the order of 1000 mm) can only span up to around 25 m. Beyond this, plate girders must be used. These have the advantage that they can be made to measure, for example to suit the stresses to which they will be subjected. To reflect the applied bending and shear, the cross sectional areas of the flanges

Table 5.8 Average span to depth ratios l/h (slenderness) for steel beams used in road bridges.

Type of Beam	Structural Form	
	Simple Beam	Continuous Beam
Plate girder	12 to 18	20 to 28
Box girder	20 to 25	25 to 30
Truss	10 to 12	12 to 16

and web vary between regions along the length of the span. However, the minimum thickness of the web should be limited to 10 mm to assure durability and facilitate handling of the plates during fabrication. The flanges should be chosen so that their slenderness (the ratio between their width and thickness) does not lead to problems of local instability (local buckling) that would limit their contribution to the bending resistance of the beam cross section (§ 12.2.3).

Plate girders can also be designed to have a variable depth. Locally increased depth is of particular interest in regions above intermediate supports for long span beams. The increased stiffness (which depends on the second moment of area) of the beam attracts moment to these regions and correspondingly reduces the moments in the spans, permitting the beam dimensions and therefore self-weight to be reduced in the central parts of long spans. This solution also often improves the aesthetics of the bridge, and it is particularly relevant when two short spans sit at either end of a long span, as shown in Figure 5.9.

Variable depth beams can achieve a slenderness of between 40 and 50 in span, and of the order of 20 to 25 around the intermediate supports. While a variable depth bridge is interesting for a totally steel structure (beams and deck), this is not the case for a composite solution. A composite section resists bending most effectively when subject to positive moments, which means in the spans, so it is not beneficial to reduce the positive bending moments in the spans by conceiving a variable depth solution as described above.

A study of around seventy continuous composite bridges (mainly twin girder bridges), built or planned in Switzerland, has allowed an empirical relationship to be developed for the slenderness of the steel beams used. The relationship, which is a function of the span and the slab width, is given in equation (5.1). For modern box girder continuous composite bridges, slenderness varies between 25 and 30.

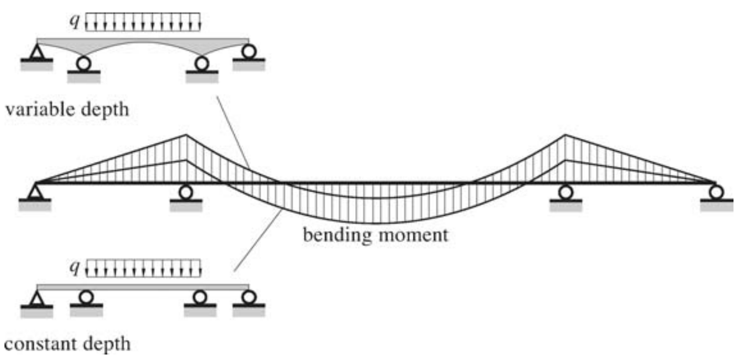


Fig. 5.9 Bending moment due to a uniformly distributed load q for a beam of variable depth, compared to that for a beam of constant depth.

$$\frac{l}{h} = 20 + \frac{l-30}{5} - \frac{2b-12}{2.5} \tag{5.1}$$

l : maximum span of the beam [m]
h : depth of the steel beam [m]
$2b$: total slab width [m]

The cross sections of plate girders used in composite bridges are not doubly symmetric (like rolled sections). The slab, which is connected to the steel beams, fulfils the function of the upper flange, thereby allowing the area of the upper steel flange to be reduced. This means that the area of the upper steel flange will be less than that of the lower steel flange. Above intermediate supports the concrete slab is subject to tension and will crack. In these regions the steel reinforcement in the slab effectively contributes to the upper flange. However, the reinforcement is less effective in fulfilling this function than is the concrete when in compression. Indicative values (based on Swiss bridges) of the average ratios between the area of the web A_w, the upper flange $A_{f,sup}$ and lower flange $A_{f,inf}$ and the total area A_{tot} of the beam in a composite bridge are:

- $A_{f,sup}/A_{tot}$: 25% at the supports and 20% in span
- A_w/A_{tot} : 35% at the supports and 40% in span
- $A_{f,inf}/A_{tot}$: 40%

These ratios are applicable to bridges in which the slab is supported on top of the steel beams. When the slab is placed between the beams, the latter will have a cross section that is rather single symmetric.

For this same survey of planned or constructed Swiss bridges, with the slab supported by twin plate girders (Fig. 5.1), typical dimensions for the webs and flanges for spans between 30 and 100 m are given in Table 5.10.

Table 5.10 Web and flange dimensions for plate girders used in composite bridges [mm].

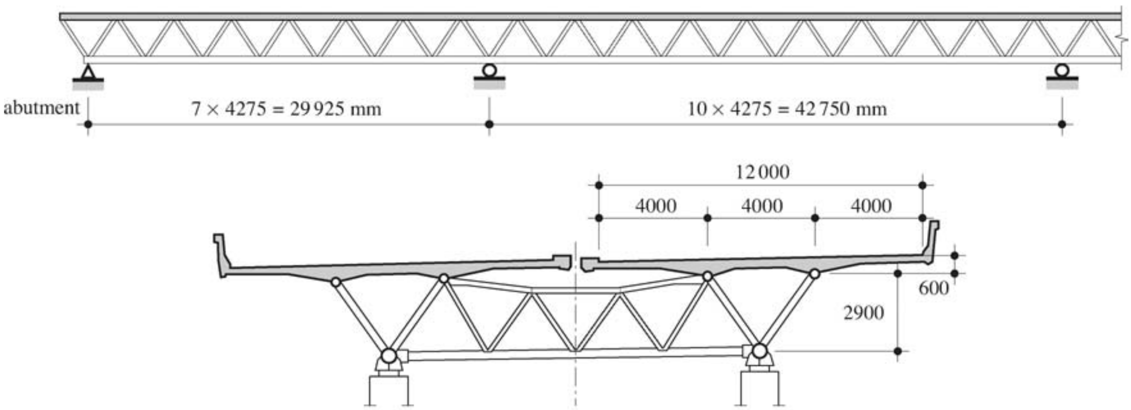
Dimension		Notation	In Span	At Support
Thickness of	the top flange	$t_{f,sup}$	15 to 40	20 to 70
	the bottom flange	$t_{f,inf}$	20 to 70	40 to 90
	the web	t_w	10 to 18	12 to 22
Width of	the top flange	$b_{f,sup}$	300 to 700	300 to 1200
	the bottom flange	$b_{f,inf}$	400 to 1200	500 to 1400

Typical slenderness h_w/t_w for the webs is of the order of 100 to 150 at the supports and 200 in span. For long span solutions the flanges may be up to 150 mm thick and, for plates of this size, the engineer must consider issues associated with brittle fracture (§ 4.5.1), and availability of the required grade of steel from the steel producers.

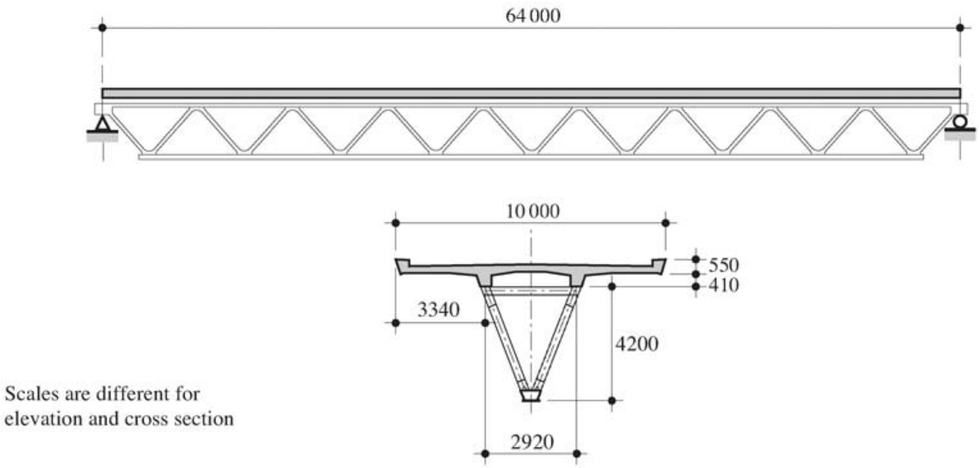
For box girders the slenderness of the flanges is an important issue, because the width of the flange may vary from around 1.0 m for small, narrow boxes, up to several metres (§ 5.5.2). This means that, when the flange is in compression, it must be stabilised by welded stiffeners to ensure it is totally effective in its contribution to the bending resistance of the section. The stiffeners may be in the form of flat plates, angles, or box sections. Typical thicknesses for the plates used to form the webs and flanges of box girders, for composite bridges with spans varying between 45 and 150 m, are given in Table 5.11.

Table 5.11 Web and flange thicknesses for box girder beams [mm].

Dimension	Notation	In Span	At Support
Thickness of the top flange	$t_{f,sup}$	16 to 28	24 to 40
the bottom flange	$t_{f,inf}$	10 to 28	24 to 50
the web	t_w	10 to 14	14 to 22



(a) Viaduc de Lully (CH)



(b) Road bridge over the Hagneck Canal (CH)

Fig. 5.12 Recent examples of truss bridges (Engineer DIC, Aigle).

5.3.3 Truss Beam Bridges

Truss bridges are currently seeing something of a revival. This renewed interest in trusses for road bridges, even for those of short span, is due to the aesthetic lightness of these structures as well as technological advances in the use of tubular sections and the ability to form welded joints between them. Modern road bridges typically adopt Warren trusses (simple V truss), which often form part of a composite solution in conjunction with a concrete slab.

Figure 5.12 shows two examples of composite bridges with a structure that comprises a truss of triangular cross section formed from Warren trusses in two planes. Stability against overturning of the beam is achieved by linking together two bridges (one for each direction of traffic) via cross bracing at the piers in one case (Fig. 5.12(a)), and by supporting the upper chords of the simple truss at the abutments in the other case (Fig. 5.12(b)). In both cases the two planar trusses comprise upper and lower chords and diagonals, all of tubular cross section, but they differ in the configuration of the tubes. The Viaduc de Lully adopts circular tubes with great wall thickness that were hot rolled. They are welded directly to each other without the use of gusset plates. For the bridge over the Hagneck canal, the tubes are of rectangular cross section, and were fabricated by welding together plates. Examples of joint detailing for tubular members used in trusses are given in Section 6.6.

5.3.4 Longitudinal Structural Form of Beam Bridges

The beams are supported at the abutments and piers by bearings (§ 2.4.1). The bearings may permit movement (sliding bearings, roller bearings, etc.) in the longitudinal sense of the beams relative to their supports, or be a fixed point between the beams and the piers or abutments. Similarly, the bearings may be either fixed or allow relative movement in the transverse sense. Embedding the steel beams into the concrete piers to form rigid connections is not a detail normally adopted in Switzerland. Typical solutions are such that the supports allow rotation between the steel beams and the supporting piers and abutments.

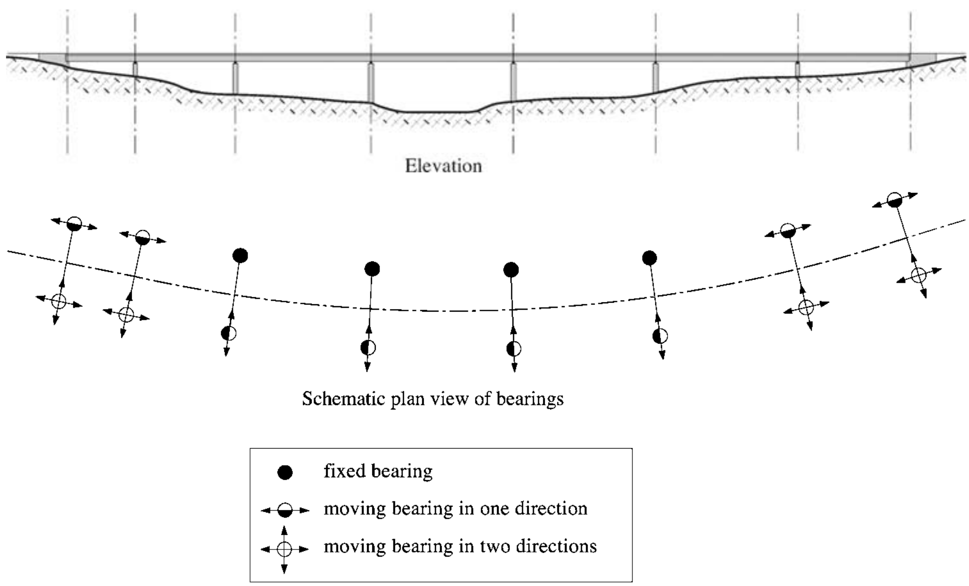


Fig. 5.13 Example of fixed and moving bearings with their schematic representation for a bridge on flexible piers.

Figure 5.13 shows an example of the collection of fixed and moving bearings present between the beams and the piers and abutments for a bridge on flexible piers (this concept is explained below). This figure also indicates how the bearings should be shown schematically on a plan drawing in order to clearly communicate their purpose.

Horizontal forces acting longitudinally (Fig. 5.3(d)) are transferred to the foundations via the fixed bearings over the relevant piers and/or abutments. If an abutment supports a bearing, which is fixed in the longitudinal direction, then the fixed point of the bridge is located at this abutment. If several piers support fixed bearings, but neither of the abutment does, the fixed point of the bridge (for example, for movement due to uniform variations in temperature) depends on several factors. These include the flexibility of the piers and the ability of the ground to resist the effects transferred by the beam.

To illustrate the idea of the fixed point F of a bridge, Figure 5.14 shows two examples of typical longitudinal structural forms:

- bridge with a fixed point at an abutment,
- bridge on flexible piers.

The first solution is generally adopted for short bridges. A bearing, which is fixed in the longitudinal direction, is located at one of the abutments, and the other bearings allow movement in the longitudinal sense (Fig. 5.14(a)). If the bridge is supported on tall piers, it is possible that the bearings at the top of these piers may even be fixed. In such cases, the lengthening or shortening of the bridge due to uniform variations in temperature, for example, results in movement at the tops of the piers. However, the piers are capable of accommodating these movements, because for short bridges the movements are small, and due to their height, the piers are flexible in terms of this direction of movement.

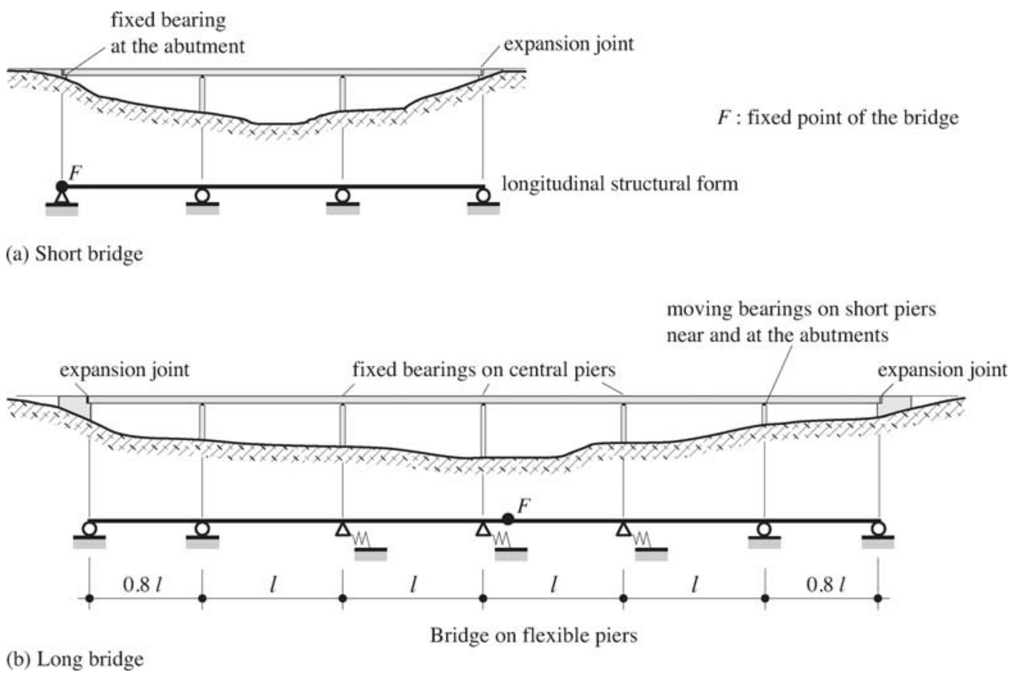


Fig. 5.14 Longitudinal structural form for a bridge, and the location of the fixed point F.

If the bridge is long, and the fixed point is located at an abutment, it will probably be necessary to include a movement joint in one of the spans in order to limit the relative movements that the bearings, or the tall piers, must accommodate. However, if possible, it is desirable to eliminate road and expansion joints, given that they may adversely affect the durability of the bridge local to the joint position, and that the continuity of the beam will be interrupted. For long bridges another solution to consider is that of a ***bridge on flexible piers***.

A bridge on flexible piers is called that because it does not have a longitudinal fixed point that can be physically defined (Fig. 5.14(b)). This concept for long bridges adopts bearings that allow longitudinal movement at each abutment. An expansion joint is thus located at each abutment. The tallest piers (generally those in the central region of the bridge) are topped by fixed bearings, and the shortest piers (those nearest to the abutments) are topped by bearings that allow longitudinal movement. The central piers, that is those that are fixed to the beams, react like springs when subjected to the movements caused by variations in ambient temperature. In such a case the fixed point is located at the centre of flexural stiffness (in the longitudinal sense for the bridge) of those piers that support a fixed bearing.

The expansion of a bridge on flexible piers, due to variations in temperature, varies by position along the bridge. It is proportional to its distance from the fixed point. Tall, slender piers topped by fixed bearings see only small longitudinal movements, because they are close to the fixed point. The piers, therefore, are subjected only to minor bending stresses due to this effect, more so because they are slender and therefore not rigid in resisting the movement. The piers furthest from the fixed point (those near the abutments) are generally also the shortest and therefore the stiffest in terms of bending. However, they are not subject to longitudinal movements (except the small amounts that result from friction in sliding bearings), because they are topped by bearings that can accommodate such movements. In this way bridges that are over a kilometre in length can be built without any road and expansion joints other than those at the abutments. A certain minimal number of piers with fixed bearings must be adopted in order to assure the longitudinal stability of the structure. This aspect of design checking is considered in Chapter 15.

For both types of bridge, on flexible piers and short bridges supported on slender piers, the principle should be to conceive a solution having a majority of fixed bearings, given that they are more durable and require less maintenance than bearings that allow movement (and in so doing are more delicate).

While the concept for the structure of a long bridge is being developed, it is beneficial to consider spans that are of equal length. Such a solution facilitates fabrication of the main beams, which should be identical. With that said, wherever possible it is preferable to adopt end spans that are around 80% of the length of the central spans, so that the cross sectional dimensions of the beams used in the end spans will be similar to those used in the other spans. Such a solution also results in improved aesthetics.

5.3.5 Curvature in Plan

For a straight bridge, variations of the ambient temperature lead to variations in the length of the bridge along its axis. When a bridge is curved in plan, such temperature variations result not only in changes in the bridge length, but also in variations in the radius of curvature. Consequently, movements no longer take place along the axis of the bridge, but rather along the axis of the chord that relates each support to the fixed point of the bridge.

Figure 5.15(a) shows expansion of a two-span curved bridge with the fixed point located at an abutment. It is clear that the movements at the bearing positions are not in the direction of the bridge axis. In addition, the expansion joint experiences not only a change in its opening width, but also relative transverse movement between its two sides.

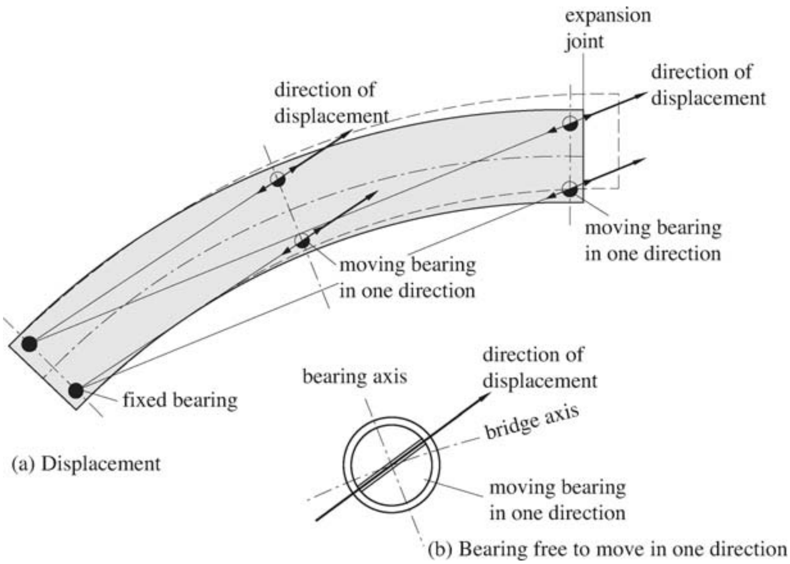


Fig. 5.15 Displacement of a curved bridge subject to a temperature increase and movements at the bearings.

In such a situation, and if the bearings only allow movement in the direction of the bridge axis, then they will experience and transfer transverse forces. If these transverse forces are small, and the bearing is designed to accommodate them, then it can simply be orientated to allow movements in line with the bridge axis. However, for most curved bridges the movement bearings are orientated according to the directions of displacement, as shown in Figure 5.15(b). It is then necessary to design expansion joints that allow transverse movement of the bridge relative to the abutment.

5.4 Transverse Structural Form

5.4.1 Plan Bracing

In the transverse direction the structure must transfer the horizontal forces that act perpendicular to the bridge axis, down to the foundations (Fig. 5.3(c)). The engineer must therefore develop a structure that provides resistance in the horizontal plane, using plan bracing. For many bridge solutions the answer to this question of plan bracing can generally be described as a truss located in a horizontal plane, as shown by the example in Figure 5.16. The plane of the bracing may be located either at the top or the bottom of the bridge cross section when cross bracing in the form of a truss is present, or at the level of the cross girders when the cross bracing takes the form of moment resisting frames.

Figure 5.17 shows the way in which horizontal wind forces are transferred to the plan bracing, via a structural system that includes the cross bracing in the span. This figure illustrates the transfer of wind forces into the plan bracing that is located at the bottom of the bridge cross section, in the same plane as the lower chords of the truss cross bracing (the same principles apply when the plan bracing is located at the top of the cross section). The wind pressure acts on the web of the main beam, and it is normally assumed that half the wind force is applied directly to the plan bracing, and the other half is applied via the cross bracing.

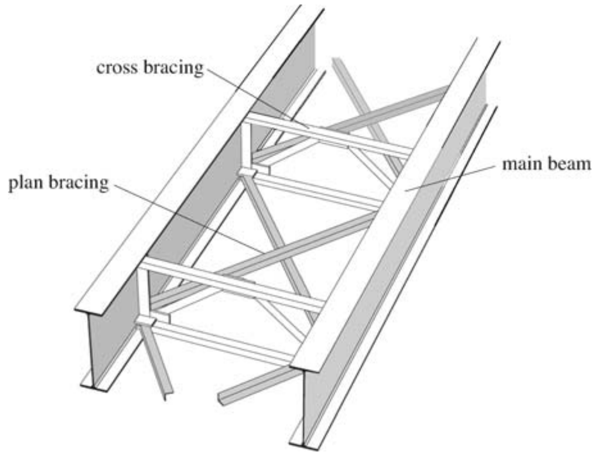


Fig. 5.16 Example of bracing for a twin girder bridge.

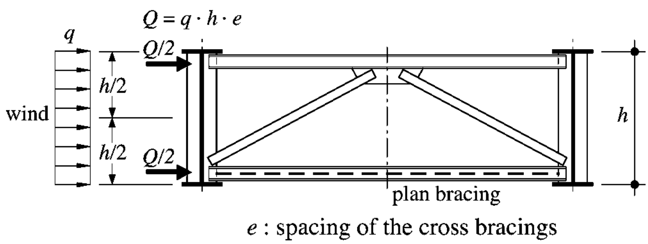


Fig. 5.17 Introduction of forces due to wind into the lower plan bracing via the cross bracing.

In the case of a composite bridge, the slab that is structurally connected to the steel beams fulfils the function of the plan bracing. However, when the slab is not present during erection, temporary steel plan bracing is typically required.

The plan bracing is supported at the bridge piers and abutments. The support reactions from the plan bracing are transferred to the piers and abutments by bearings, either directly (if the plane of the bracing is close to that of the bearings) or via the cross bracing (if the plane of the plan bracing does not coincide with that of the bearings). The latter situation arises for composite bridges when the slab fulfils the function of the plan bracing (Fig. 5.18).

5.4.2 Transverse Structural Form of Beam Bridges

So that support reactions R_H from the plan bracing are transferred to the piers and abutments, it is necessary that at least one of the two bearings on each support be fixed transversally; that way they provide lateral support (Fig. 5.13). For a twin girder bridge, Figure 5.18 illustrates two examples: one with two fixed bearings, and one with a single fixed bearing plus a bearing that allows transverse movements. The horizontal and vertical support reactions that balance the reaction R_H, which is transferred through the cross bracing at the support, are also illustrated in this figure. When two or more bearings above a pier or abutment are fixed transversally, the cross section of the bridge is not totally free to deform, and this

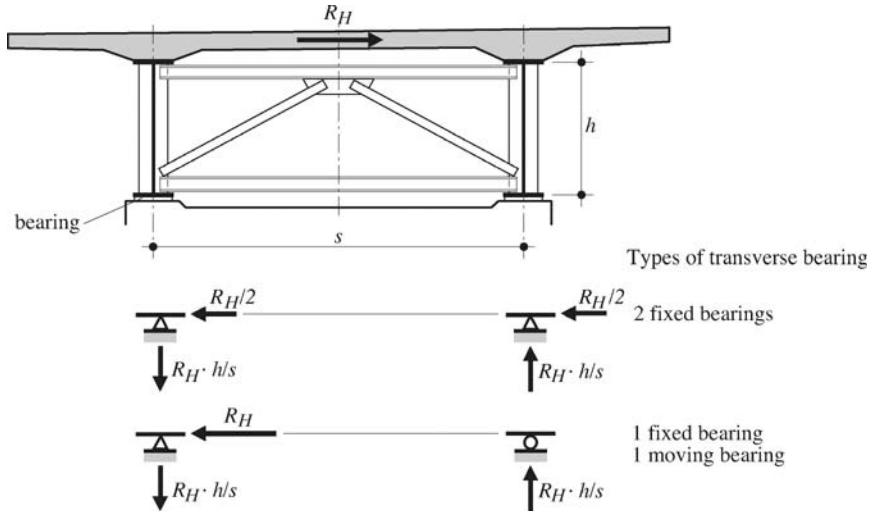


Fig. 5.18 Transfer of the horizontal wind force R_H from the slab to the supports.

can result in certain forces (for example when there are temperature differences between the deck and the piers). If two or more of the total number of bridge bearings are fixed transversally, then the reactions from the plan bracing will be distributed between the various fixed supports below the beams.

The magnitude of the support reactions R_H depends on the flexural stiffness of the piers, transversally to the bridge axis, (Fig. 5.19). In reality, as far as the transverse direction is concerned, only the abutments represent fixed supports for the plan bracing; whereas the piers, due to their form being only more-or-less stiff, provide elastic supports.

Figure 5.19 presents several examples of piers and structural forms for the plan bracing (in the horizontal plane) that correspond to the assumptions made about the stiffness of the piers, as far as their function in supporting the plan bracing is concerned. These examples are for a three-span bridge. If the superstructure is supported on massive piers (Fig. 5.19(a)), the intermediate supports for the plan bracing may be taken as fixed. On the other hand if the deck is supported on slender piers (Fig. 5.19(c)), it can be assumed that the piers do not provide any transverse support, hence only the abutments support the plan bracing. For a situation between these two extremes (Fig. 5.19(b)), one should take into account the effective stiffness of the piers in defining a spring stiffness of the intermediate transverse supports.

5.5 Types of Cross Section

Beam bridges, which comprise main beams and a slab, have a cross section (§ 2.2.5) that is described as either:

- open, or
- closed.

A cross section is described as open when it is formed from two or more I section beams (rolled or fabricated), trusses, or beams in the form of small box sections. A cross section is described as closed when it comprises a single or multi-cell box section. The distinction between open and closed cross sections is

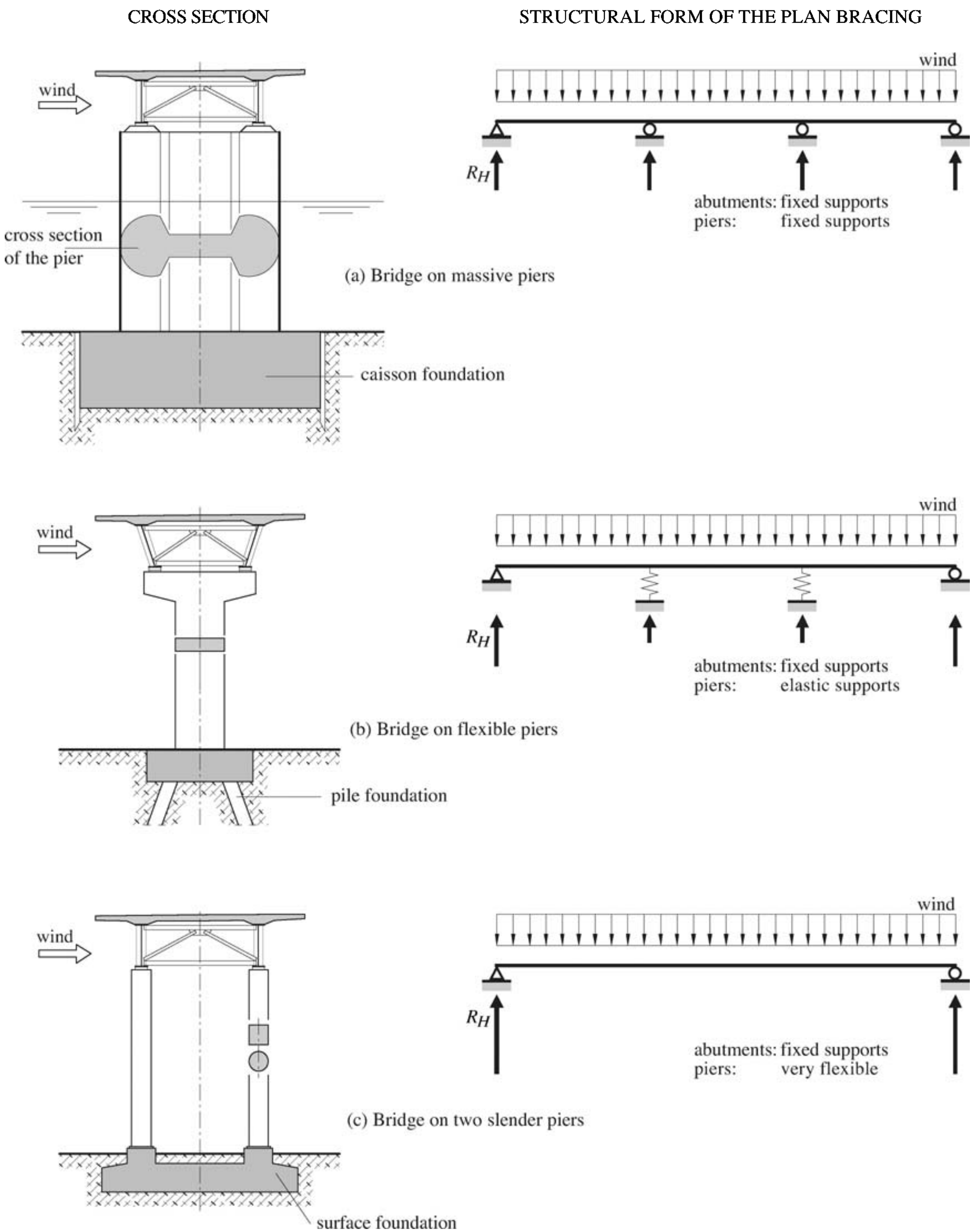


Fig. 5.19 Different plan bracing systems as a function of the transverse stiffness of the piers.

particularly relevant to the way in which sections resist actions that are applied eccentrically to the bridge axis, because of the way torsional moments are resisted.

5.5.1 Open Cross Sections

Figure 5.20 shows examples of the most common forms of bridges with an open cross section, namely twin girder and multi girder solutions. The twin girder solution (Fig. 5.20(a)) is the simplest form of a composite bridge. The cross section comprises a concrete slab structurally connected to two steel beams. This solution is common for composite bridges with a slab width less than around 13 m, which is the width needed for a freeway bridge to carry two lanes of traffic (forming a single carriageway). If the slab is larger than this, then a twin girder bridge will require a thicker concrete slab in order to resist transverse (relative to the bridge axis) bending. This will clearly increase the weight of the slab, which will probably also require transverse prestress. The spacing of the two main beams is normally more-or-less equal to b (say, between 1.0 and 1.1b), where b is equal to half the width of the bridge slab. This spacing is chosen in order to equalise the positive and negative transverse bending moments acting on the slab. This form of cross section is appropriate for spans up to around 125 m. For longer spans it is necessary to reduce the self-weight of the deck by replacing the concrete with a steel orthotropic deck (§ 2.2.4), or perhaps to adopt a box girder solution, since it is better able to resist eccentric loading (Sect. 11.4).

When the slab is very wide, or the depth available for the structure is limited, it may be necessary to adopt a multi-girder solution for the bridge (Fig. 5.20(b)). For this type of solution, which is uncommon in Switzerland, the beams are normally rolled sections at around 3 m centres.

Small, narrow box sections (Fig. 5.20(c)) may be used instead of I sections, in particular when headroom is limited. Such a solution is still classified as an open cross section even though the torsional stiffness of the boxes affects the transverse distribution of the loads (§ 11.5.2). Both the flexural stiffness about the minor axis and the torsional stiffness of these small box sections exceed those of I sections. For bridges that are close to the ground, these attributes can avoid the need to install temporary plan bracing as well as cross bracing (at least in the spans). In such a situation the cross section itself fulfils the function of the

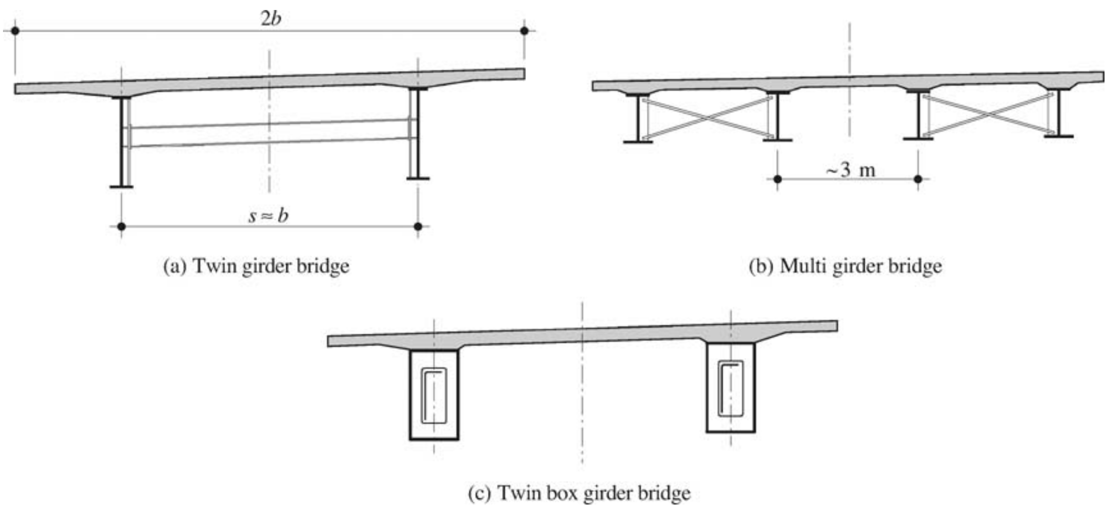


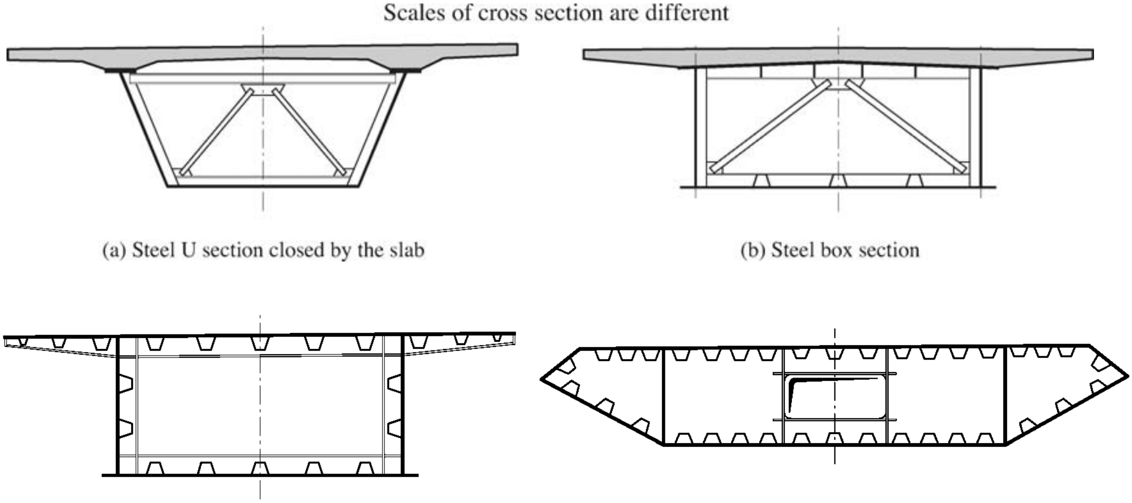
Fig. 5.20 Examples of open cross section.

cross bracing in the form of a moment resisting frame (the slab forms the cross girder and the webs of the boxes, which are built into the slab, form the "uprights").

5.5.2 Closed Cross Sections

When the cross section of the bridge comprises a box section, it is described as having a closed cross section. For composite bridges and medium spans, the closed section is formed from a U shaped steel section, also known as an *open steel box section*, onto which the concrete slab is connected to close the box (Fig. 5.21(a)). For long spans, and depending on the erection method, it may be advantageous to create a *closed steel box section* prior to casting of the slab. Such a cross section is shown in Figure 5.21(b). The slab is connected to the upper flanges of the box section, which is stiffened both longitudinally and transversally and fulfils the function of formwork for the concrete. For very long spans an *orthotropic deck* replaces the concrete slab in order to reduce the self-weight of the structural elements (Fig. 5.21(c)). For long span cable supported bridges, the box girder must adopt an aerodynamic form, rather than being a simple rectangle, in order to improve the behaviour of the bridge under wind loading (Fig. 5.21(d)).

Scales of cross section are different



(a) Steel U section closed by the slab

(b) Steel box section

(c) Box with an orthotropic deck

(d) Multiple box section for cable stayed or suspension bridges

Fig. 5.21 Examples of closed cross section.

The most common form for a closed steel box girder is rectangular (Fig. 5.21(b)), and the box normally only has one cell. For composite bridges with an open box, this is generally of trapezoidal cross section (Fig. 5.21(a)). The inclination of the webs relative to vertical should not exceed 20 to 25 degrees, otherwise special measures must be taken to ensure that the geometry is maintained during concreting of the slab. One of the reasons for choosing a trapezoidal form is to offer supports to the slab that are appropriately spaced to best equalise the positive and negative transverse bending moments. This geometry also allows the width of the lower flange of the box to be reduced, thereby reducing the number of longitudinal stiffeners needed to ensure the flange is totally effective in compression when the beam is subject to negative moments (Sect. 12.2). It is worth noting that it is also possible to create a lower flange that is totally effective in compression by placing on it a concrete slab, which is connected to the steel plate using shear studs. Assuming it is part of a composite bridge, one then obtains a cross section that is doubly composite in the regions above the intermediate supports.

A closed box section can also be created using trusses, rather than plates, to form the walls of the box. Therefore, a bridge comprising two vertical trusses and two levels of truss plan bracing is also described as a bridge with a closed cross section, and indeed behaves as one. The beam cross sections shown in Figure 5.12, comprising two planes of truss and a slab, represent closed cross sections of a triangular form.

A twin girder composite bridge with a truss plan bracing placed between the lower flanges of the main beams also forms a closed cross section. The addition of such plan bracing to an open section modifies the way in which the bridge responds to eccentric loading. A cross section that is closed in this way possesses torsional stiffness that is much greater than that of the basic open section.

A closed cross section also possesses significant stiffness in horizontal bending and is therefore able to resist and transfer horizontal wind forces. When an open box section solution is adopted, a closed steel section is nevertheless often needed during erection to resist the wind loads (depending on the method of erection). The closed steel section can be formed by using temporary plan bracing at the top of the box to fulfil the function of the slab prior to concreting (Sect. 5.7).

The substantial torsional stiffness of closed sections allows spans of up to 300 m to be achieved. At the top end of this span range, the bridges are formed totally from steel, with an orthotropic deck. Because of their structural properties, closed sections also offer advantageous solutions for curved bridges, which are subject to additional torsional effects due to their curvature in plan.

5.6 Cross Bracing

The main beams, the horizontal plan bracing, and the cross bracing (which is orientated perpendicularly to the beam axis) form the three planes of resistance of the spatial structure of a bridge (Fig. 5.3(a)). Cross bracing is located both above the piers and abutments and at equidistant points within the spans. Section 5.6 describes the different functions of the cross bracing, different design options for the bracing, and the various issues to consider during conceptual design.

5.6.1 Functions of the Cross Bracing

The cross bracing fulfils a number of functions as part of the load carrying structure of a bridge. In addition to those principal functions described in Paragraph 2.3.1, and depending on the configuration and manner of execution of the bridge, the bracing may fulfil a number of supplementary functions. The various functions of the cross bracing are considered below:

- The cross bracing ensures that the cross sectional shape of a bridge does not distort under loading, which means that the analysis can be based on the assumption that the dimensions of a given cross section are constant (§ 11.2.1). As far as this function is concerned, the cross bracing is only lightly stressed for a twin girder bridge, because the torsional stiffness of I beams about their longitudinal axis is small. For such bridges subjected to eccentric loading, the cross sectional shape of the bridge is easily maintained by the cross bracing (§ 14.3.1). For bridges with a closed cross section, maintaining the form of the box implies that the cross bracing in the spans transfers any torsional load into the box section, which resists these loads by means of a shear flow (§ 14.3.2).

- For an open cross section bridge, the cross bracing in the spans transfers to the slab and/or plan bracing some of the wind forces acting on the webs of the main beams (Fig. 5.17). At the pier and abutment locations, the cross bracing transfers the horizontal forces from the plan bracing to the supports (in cases where the plan bracing is located above the plane of the supports) as shown in Figure 5.18. At the pier and abutment locations, the cross bracing for a box girder bridge must fulfil an additional function of transferring the torsional moment to the supports, assuming they are designed as rigid supports for the beam in torsion. When fulfilling this function the cross bracing will be more highly loaded than the bracing in a twin girder bridge.
- The cross bracing provides lateral restraint to the compression flanges of the main I beams and thereby contributes to increased resistance to lateral torsional buckling of the beams by reducing their effective length. The forces applied to the cross bracing when performing this function (§ 14.2.3) are transferred to the plan bracing. The elastic stiffness that the cross bracing exhibits as lateral support depends on the type and stiffness of the bracing, and should be taken into account when determining the effective length for lateral torsional buckling of the beams (§ 12.2.4).
- The cross bracing of curved bridges resists the deviation forces in the flanges, be they in compression or tension, and transfers the torsional forces due to curvature into the main beams (§ 14.2.4).
- The cross bracing may serve to introduce local forces from jacks used to lift the bridge in order to repair or replace bearings (§ 14.2.5).
- The cross bracing may be used to support ducts and pipes and, when the slab is cast in situ, it may support the formwork between the webs of the main beams.
- The cross bracing assures the geometry and stability of the overall structure during execution – both erection of the steelwork and construction of the slab.

For straight, open cross section bridges, the cross bracing will be at 6 m to 10 m centres. For closed cross section bridges, typical spacing of the cross bracing is between three and four times the depth of the section. For curved bridges generally closer spacing is adopted than that found in straight bridges. The cross bracing above piers and abutments is used to transfer to the supports forces in the plan bracing resulting from wind and torsional moments acting on box girders. Given that this bracing is more highly loaded than that in the spans, the cross bracing at the piers and abutments tends to be stronger.

5.6.2 Types of Cross Bracing

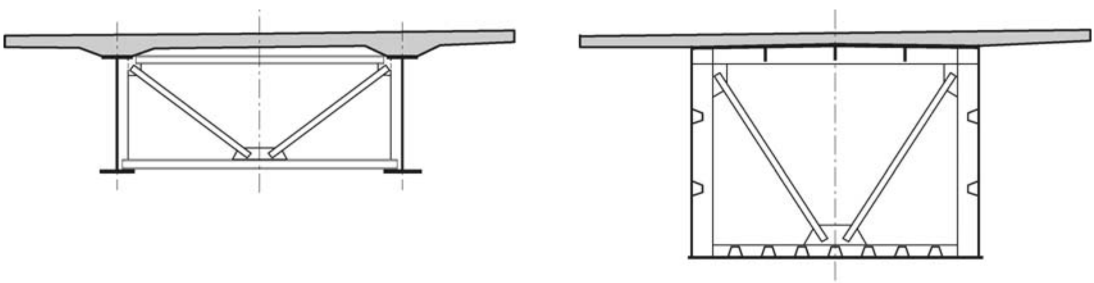
Three types of cross bracing are used, and they may be distinguished according to their form:

- ***truss cross bracing***,
- ***frame cross bracing***,
- ***deep beam*** or ***diaphragm cross bracing***.

The choice of cross bracing depends on a number of criteria, which vary in importance according to the method of erection, the type of bridge cross section, the length of the spans, and the location of the bridge. The main aspects to consider during conceptual design of the cross bracing concern the facilities and equipment that are available for erection, the magnitude of forces, the voids that are needed to accommodate cables and ducts, and the personnel for both construction of the concrete slab formwork and maintenance of the bridge. Finally, and depending on the location of the bridge, the aesthetic of the cross bracing may also need to be considered. Currently in Switzerland the most common solution for the cross bracing, which is also generally the most cost-effective solution for twin girder bridges, is the use of frame cross bracing.

Truss Cross Bracing

Truss cross bracing is normally in the form of a K, and occasionally in the form of an X. Figure 5.22 shows two examples of truss cross bracing, one for a twin girder bridge and the other for a box girder bridge. For the open cross section, the bracing comprises top and bottom chords, diagonals, and vertical stiffeners to the main beams, which form the uprights of the truss. For the closed cross section, the bracing comprises diagonals plus the four box stiffeners forming a frame within the box. Such bracing may also be inverted, with the diagonals meeting at the top of the section. This option is often adopted for cross bracing within the span, particularly if the top chord is also used to support the slab formwork for a twin girder bridge. Truss cross bracing, with the diagonals meeting in the middle of the bottom chord, serves an additional purpose in that the diagonals form a point of lateral support for this member to prevent its in-plane buckling. This form can be of interest for cross bracing at supports, where stresses are high and the bottom chord can be subject to relatively high compression (depending on the transverse structural form at the supports).



(a) Open section (b) Closed section

Fig. 5.22 Truss cross bracing.

Different types of steel section may be used to form the truss members. The most common are rolled angles (LNP), used either singularly or in pairs, rolled channels (U), and hollow section tubes (RHR or ROR). The truss members may be connected together by either bolting or welding, and may be connected directly or via gusset plates to the main beam stiffeners. Choices will depend on the sizes of the different elements (§ 6.4.2).

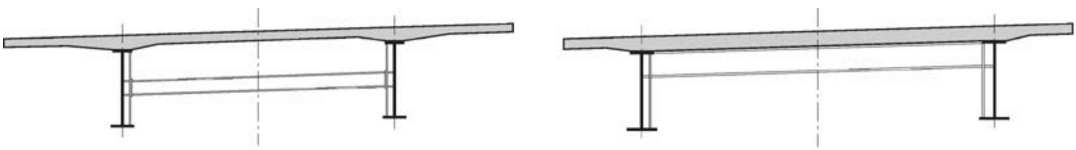
Truss cross bracing is very stiff (in resisting shear) in plan (§ 14.5.2). It is mainly used for the cross bracing in the spans of box girders. Because frame bracing requires less labour, for twin girder bridges it is often preferred.

Frame Cross Bracing

Frame cross bracing comprises a cross girder (Fig. 5.23) rigidly connected to uprights that also have the function of vertical web stiffeners for the main beams. The cross girders are normally rolled H or I sections

(HE or IP) or, if the loads are high, plate girders. The girders may be either welded or bolted to the uprights (§ 6.4.1). In order to assure adequate stiffness of the cross bracing, the girders should normally be at least 300 mm deep. Cross girders are generally located around the mid depth of the main beams (Fig. 5.23(a)), or perhaps closer to the slab, if they are also to act as formwork support during concreting. If the slab is to be prestressed transversally after its connection to the main beams, the cross girders will be less affected by the transverse compression forces from the prestressing, when they are placed lower down in the cross section.

For some forms of bridge cross section, the cross girders allow the distance between the main beams to be increased by creating lines of transverse support beneath the slab (Fig. 5.23(b)). These intermediate slab supports reduce local bending. In such situations the distance between them is reduced to 3 m or 4 m. The local bending moments in the slab are highly dependent on this spacing, so it is worth investigating different spacings in order to ensure the best possible behaviour of the slab in both local bending and bending of the whole structure. The cross girder may also be extended beyond the main beams in order to support the slab overhangs. Supporting these cantilevers is particularly recommended near to expansion joints, because it can lead to significant reductions in the vertical deflections of the slab, and thereby facilitate good performance of the joint (Fig. 6.14).



(a) Cross girder at mid depth (b) Cross girder also supporting the slab

Fig. 5.23 Frame cross bracing.

The use of frame cross bracings results in a cross section with some flexibility in terms of resisting horizontal forces, and they are therefore mainly used with open cross section bridges. However, they can also be used with box sections, when the individual bracing members form an internal frame that is welded to the webs and flanges of the box. Frame bracing is less commonly used (than trusses) with box sections, because the frame's in-plane flexibility may not provide sufficient guarantee that the shape of the box will be maintained under loading (Fig. 6.15).

Diaphragm Cross Bracing

Deep beams or diaphragms comprise a solid web beam that is of similar depth to the main beams (Fig. 5.24(a)) for bridges of open cross section. For box girder bridges the diaphragm is made of a stiffened plate that fills the whole of the cross section (Fig. 5.24(b)). This form of cross bracing is mainly used for long span box girder bridges, specifically above supports and abutments where the cross bracing is highly stressed.

Within a box section, the diaphragm is welded around the total inside perimeter of the box. An opening is needed to allow movement of personnel for inspection and maintenance inside the box. Diaphragms, which are primarily loaded in shear, are typically stiffened (particularly around any openings) and are extremely stiff in their plane.

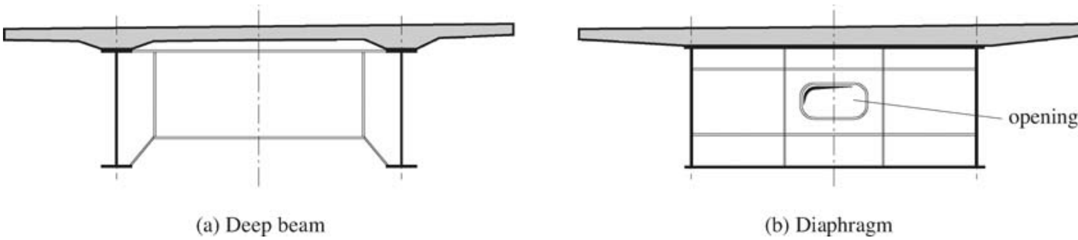


Fig. 5.24 Cross bracing using deep beams or diaphragms.

5.7 Plan Bracing

5.7.1 Functions of the Plan Bracing

The plan bracing forms the plane of resistance to horizontal forces acting perpendicularly to the bridge axis (Fig. 5.3(a)). It effectively forms a beam in the horizontal plane, which is supported by the piers and abutments (§ 5.4.1). Its primary function is to transfer the wind forces that act on the bridge to the supports. It also helps to maintain the geometry of the structure during erection of the steelwork and construction of the slab.

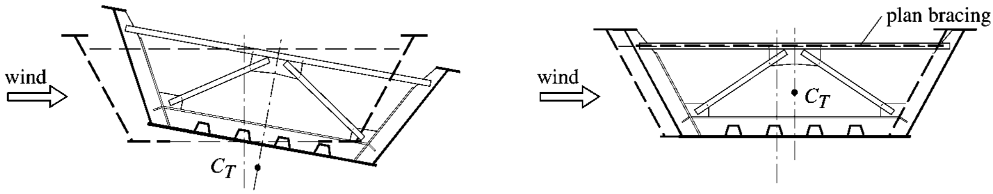
For composite bridges the slab is connected to the main beams and fulfils the function of the plan bracing when the bridge is in its final state. However, during erection of the steel structure of a composite bridge, *temporary bracing* in the form of a truss is often needed in order to assure the resistance and lateral stability of the main beams.

For a beam bridge, the chords of the plan bracing truss are the main beams of the structure. The "uprights" of the plan bracing truss are often elements of the cross bracing, either a chord (in the case of truss cross bracing) or a cross girder (in the case of frame cross bracing).

During erection by crane of minor bridges close to the ground, and when the beams are of limited depth, plan bracing is not always needed since construction of the slab results in only small lateral forces in the steel structure. This is also the case for bridges constructed using small box girders, which are capable of resisting small wind loads. On the other hand, temporary plan bracing is essential during erection by launching of the steel structure in order to assure its lateral stability. This is equally true of open sections and open U shaped box sections. In the latter case, the plan bracing is used to close the box, and has the effect of changing the position of the shear centre C_T (TGC, vol. 10, § 4.5.2) and considerably increasing the torsional stiffness. Figure 5.25 illustrates how the position of the shear centre changes, and also shows the displacement and rotation of the section subjected to horizontal forces. This figure also shows that the deformations of the beam are reduced when the section is closed by the upper plan bracing.

For open sections the temporary plan bracing is normally placed at the same level as the cross girders or, when the cross bracing is in the form of trusses, at the level of their upper or lower chords. Following construction of the slab and its shear connection to the steel structure, the temporary bracing may be removed or left in place, depending on the costs associated with its removal.

In addition to the horizontal forces of wind loading, the plan bracing, in association with the cross bracing, resists the forces resulting from stabilisation of the compression flanges of the main beams against lateral torsional buckling. Depending on its form, the plan bracing may be fixed directly to the flanges of the main beams, and the connection points may then be taken as fixed supports when considering lateral torsional buckling of the compression members.



(a) Without plan bracing (b) With plan bracing

Fig. 5.25 Influence of the upper plan bracing on the position of the shear centre C_T and the deformation of an open section.

5.7.2 Types of Plan Bracing

Several options exist for the form of truss plan bracing for a bridge. Figure 5.26 shows schematically the most common structural forms used for beam bridges, namely bracing in the shape of an X, a diamond, or a K. As noted above, the chords of the plan bracing trusses are formed by the main beams and the "uprights" by the cross girders (or the chords) of the truss cross bracing.

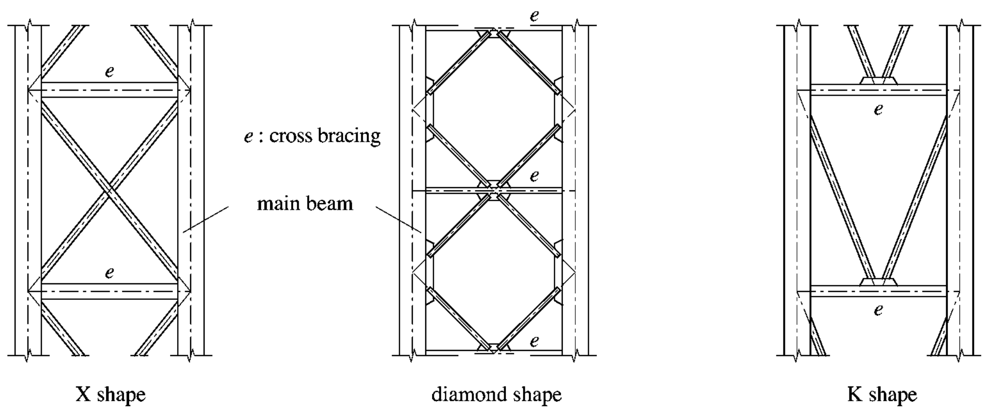


Fig. 5.26 Structural forms of plan bracing.

The choice of plan bracing may be dictated by conditions associated with the bridge execution, by the magnitude of the internal forces, by the method of steelwork erection, or by the magnitude of the expected vertical deflections of the main beams to which the bracing is attached. According to the shape of the bracing and its position within the cross section depth, longitudinal deformations of the main beams (due to bending) can stress the elements of the plan bracing, if they are not free to follow these deformations (§ 14.4.2). The magnitude of the so-called "parasitic" or "secondary" forces depends on the shape of the plan bracing, the size of the chords relative to the diagonals, and the location of the plane of the bracing within the beam depth. If the bracing is located near to the neutral axis of a beam, then in bending the parasitic forces will be negligible.

Out of the three most common structural forms of bracing (Fig. 5.26), X shape bracing is the most susceptible to substantial "parasitic forces". These forces are less significant for diamond bracing and negligible for K bracing. Therefore, K bracing is often the most appropriate solution, and indeed this is often the reason why this form of plan bracing is found in railway bridges (in which the main beams, be they solid web or trusses, are often highly stressed by the traffic loads). Once the form of the plan bracing has been decided upon, any parasitic forces must be calculated and taken into account. However, all three forms of bracing may be considered if the plan bracing is only needed for the erection phase, because there will be no traffic loads acting on the beams, and therefore the deformation of the beams will be small.

The plan bracing diagonals are typically rolled products, single or paired angles (LNP), channel sections (U), tee sections (T) or hollow section tubes (RHS or ROR). For temporary bracing which is only lightly stressed, round bars may also be considered.

For steel (non-composite) bridges, the plan bracing and its joint and connection details should be carefully studied to ensure it can continue to meet its functional requirements (fatigue safety) throughout the service life of the structure. This is particularly the case for railway bridges of a classic form. For such structures three types of plan bracing, each having a different function, are necessary:

- primary plan bracing, needed to provide the bridge with resistance to transverse horizontal forces,
- secondary plan bracing, needed to transfer the lateral horizontal forces acting on the rails (nosing forces) to the cross bracing (which itself forms the "uprights" of the primary plan bracing),
- special plan bracing, needed to resist braking forces.

Chapter 16 is dedicated to railway bridges and considers these subjects in detail (§ 16.2.1).

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6 Construction Details

Cross bracing in the box girder of the Viaduc de Millau (F).
Eng. Michel Virlogeux, France and Bureau d'études Greisch, Belgique.
Photo ICOM.

6.1 Introduction

Steel and composite bridges with spans in excess of around 20 metres generally comprise plate girders for the main beams. These made to measure steel beams are a collection of plates that are welded together to form I or H sections (Fig. 6.1) or boxes. Because these sections are formed from plates that are thin relative to their other dimensions (this is particularly the case for web plates), they must be stiffened. Often, vertical and longitudinal stiffeners will be needed to assure the necessary resistance as well as to guarantee the correct functioning of the beam. The welds that connect the individual plates must be detailed and executed in an appropriate manner, and they must possess sufficient resistance to transfer the local forces.

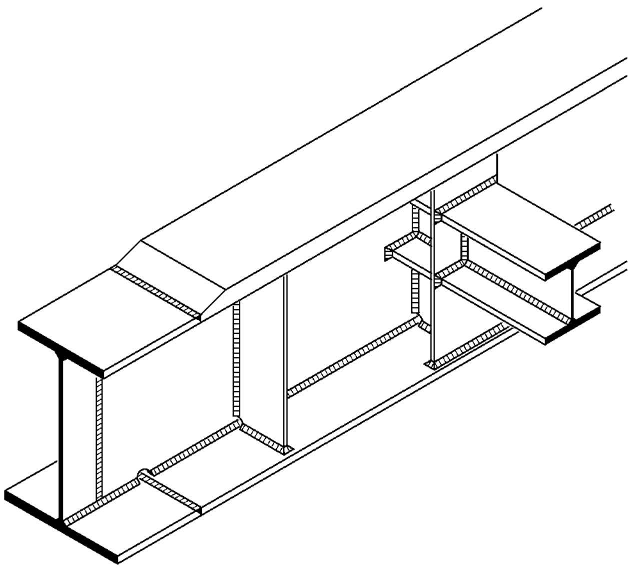


Fig. 6.1 Plate girder with stiffeners and frame cross girder.

This chapter considers the conceptual design of good construction details for the steelwork that forms the load carrying structure of a bridge. Section 6.2 begins by presenting general concepts for detailing to avoid, in particular a continual presence of standing water or moisture on the steel structure. Section 6.3 considers concepts for the detailing of plate girders, covering both the workshop fabrication and work on site.

When several beams are used to support the bridge slab, they are effectively joined together by means of cross bracing (Sect. 5.6). Cross bracing is also needed for box girders. Section 6.4 covers detailing of the cross bracing, including its attachment to the beams. Section 6.5 provides guidance on detailing of the horizontal plan bracing between the main beams. Some details associated with the assembly of elements to form a truss are described in Section 6.6.

Even though they are rarely used for new short and medium span bridges, decks formed from orthotropic steel plates are seeing renewed interest for renovation projects and for the widening of existing bridges. Therefore some guidance on their configuration and detailing is presented in Section 6.7. Finally, Section 6.8 refers to the various additional components needed for a bridge, such as bearings and road and expansion joints.

The construction details covered by this chapter are not exhaustive. However, they do describe details currently used in Europe and are appropriate for a reasonably equipped modern fabrication shop. According to the traditions and preferences of individual steelwork contractors, some of the detailing concepts may differ. In any case, they must meet the relevant basic requirements set out below.

The European Convention for Constructional Steelwork (ECCS) [6.1] has published recommendations for good design practice of construction details with respect to their fatigue behaviour.

6.2 Detailing of Bridges

The overall conception of a steel or composite bridge, as well as the configuration of the construction details, must always take into account the fact that a bridge is an outdoors structure that will be subjected to variable climatic conditions, such as rain, wind and temperature variations. As already discussed in Paragraph 4.5.5, the steel structure must be protected against the effects of moisture as well as water, which must not be allowed to accumulate and remain on the steel components. A range of details may be considered in order to achieve a good solution to these problems.

Figure 6.2 shows several construction details – both good and bad – aimed at avoiding moisture and standing water. This figure is not exhaustive, but illustrates principles to be considered when developing similar details. The examples of good detailing to avoid moisture and standing water apply both to structures formed from weathering steel, and those where the steelwork will be protected by paint (even though such steelwork is normally well protected). Inspections of existing painted steel structures have highlighted that corrosion often occurs in areas that are exposed to constant moisture or standing water, and where there is insufficient natural ventilation.

A commentary on the examples shown in Figure 6.2 is given below:

- A general consideration for welded joints is that care should be taken to avoid leaving open any gaps between plates or at their ends. Such gaps may occur when welding is undertaken from one side only. Moisture can form in gaps, which are both difficult to protect with paint and difficult to ventilate adequately (Fig. 6.2(a)).
- Water on the flanges of plate girders must be able to run off. Water may be present on the flange surfaces either because rain is driven by the wind against the girders, or, as is often the case, moisture condenses on the cold webs at dawn and runs down onto the flanges. Often the flanges will have a longitudinal slope (bridge deck not horizontal, variable depth beam, precamber), and the water will run down this slope. When it comes to a vertical stiffener, the water will accumulate at this point, if the detailing of the stiffener does not allow the water to flow past it. This problem is exacerbated by the fact that dust also tends to collect in such locations (Fig. 6.2(b)). For box girders the presence of water on the top of the bottom flange can be avoided by extending the webs to form drip features.
- Rain water is evacuated from the bridge surface using ductwork that, for aesthetic reasons, is located within a box girder. If any defects occur in this ductwork, such as at the joints, water can accumulate on the lower flange of the section. To avoid such build ups, holes should be drilled in the stiffeners and the flange to facilitate drainage (Fig. 6.2(c)).
- Where possible it is recommended to shelter the beams from rain water. Slabs with cantilevers that are longer than the depth of the beams achieve this (Fig. 6.2(d)).
- Another measure to keep water away from the steel beams is drip features formed at the edges of the concrete slab (Fig. 6.2(e)).
- According to the form of surfacing and the waterproofing of the slab, drainage ducts or decompression spouts may pass through the slab. Their exits beneath the slab, and their joints to

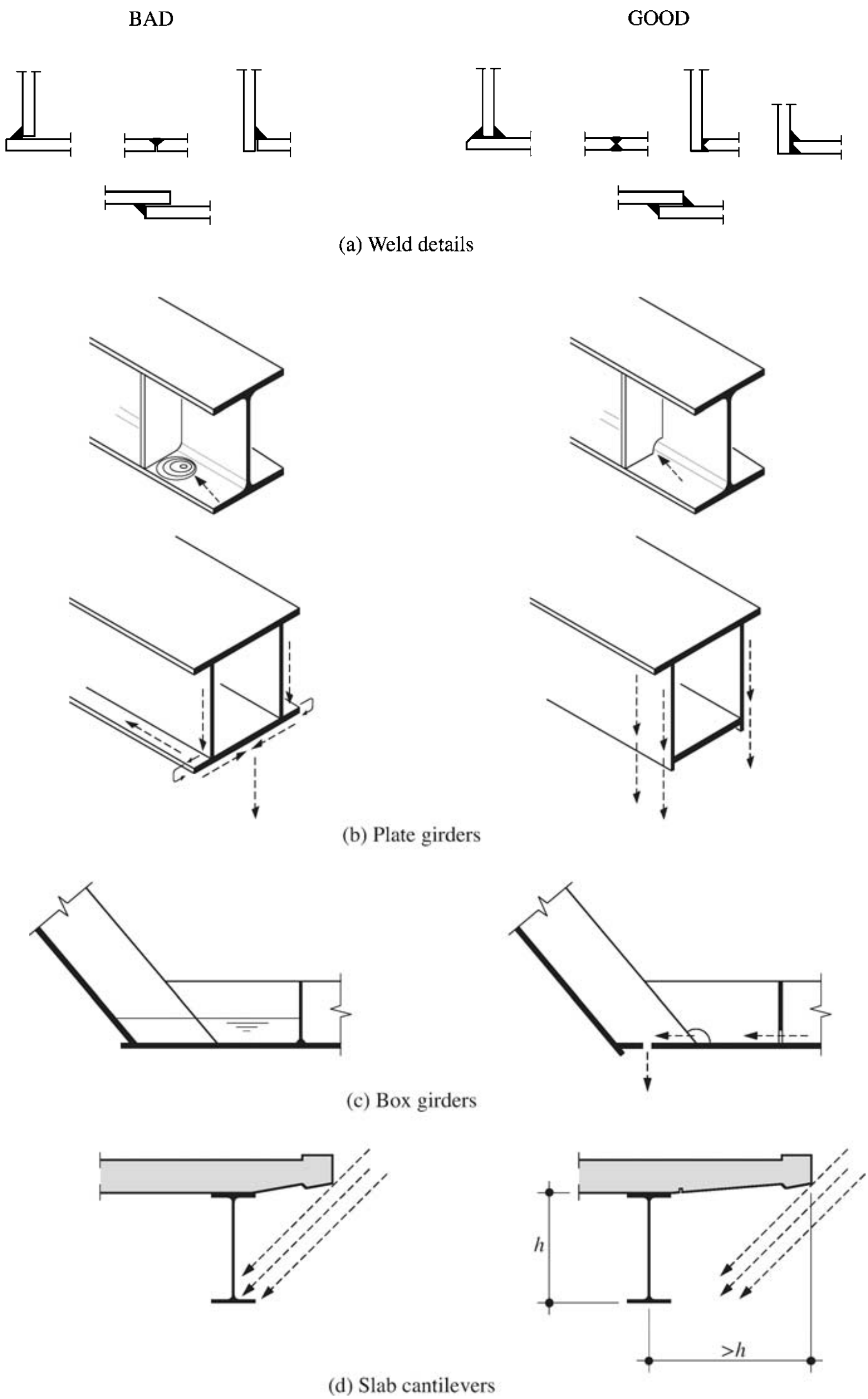
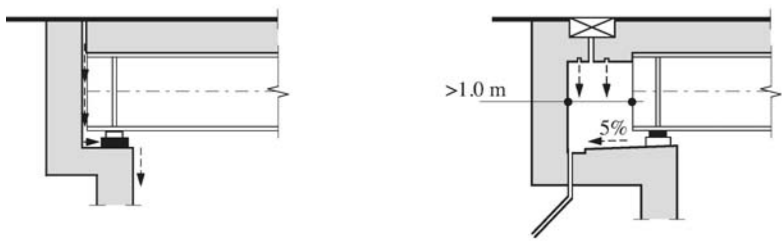


Fig. 6.2 Examples of good and bad construction details to avoid standing water and moisture.

(e) Slab edges

(f) Surfacing drainage



(g) Detailing of abutments

Fig. 6.2 (cont.) Examples of good and bad construction details to avoid standing water and moisture.

collection ducts, must be carefully detailed and not located above elements of the steel structure (Fig. 6.2(f)).

- The abutments should be designed in a way that will facilitate adequate natural ventilation of the steel structure. They must also be designed to collect any water that passes through road joints or defective expansion joints in the surface, even if the latter is supposedly waterproof (Fig. 6.2(g)). It is also worth adding that good abutment design will facilitate human access to permit inspection of the ends of the steel beams and the bearings.

6.3 Plate Girders

6.3.1 Weld Details

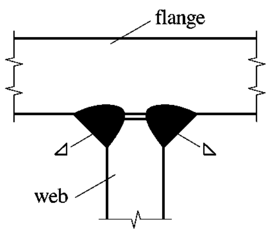
Plate girders comprise plates of different thicknesses that are connected together by welding. For a beam of constant depth, the dimensions of the flanges (width and thickness) and the thickness of the web will vary along the span as a function of the bending moments and shear forces defined by the analysis of the beam. In general, variations in dimensions between two adjacent plates are accommodated by grinding or machining of the thicker/wider plate, so that the plate dimensions on either side of a welded joint are the same. The plates are described as having a discontinuous variation (Fig. 6.1). It is possible to avoid the need for a welded joint between two plates of differing thickness by using plates that are rolled with a varying thickness along their length, which are known as *longitudinally profiled plates*.

It is appropriate to distinguish between the longitudinal welds between the flanges and the web, and the transverse welds at locations where the plate dimensions change and/or erection joints are welded on site.

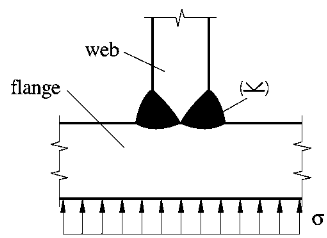
Welds Between the Flanges and the Web

The flange to web welds of a plate girder normally comprise fillet welds that are made in the fabrication shop using automated procedures (TGC Vol. 10, § 7.3.1). These welds, which generally have a throat thickness between 5 mm and 8 mm, are made using the submerged arc process. This process only allows welding in a more-or-less horizontal position, which means it is necessary to turn the beam in order to weld the web to the upper flange. Assuming it is done correctly, fillet welds formed in this way achieve partial penetration, and this may be considered when calculating the weld resistance (TGC Vol. 10, § 7.4.2).

Figure 6.3 shows examples of welds between a flange and the web of a plate girder. Figure 6.3(a) illustrates that there will be a zone between the plates and the two runs of weld that is not welded. Such a situation is not acceptable when there will be significant forces acting perpendicularly to the surface of the flange. When such forces will be present the weld must have full penetration, to assure good transfer of load between the plates. This is the case, for example, in the regions of the beam supports where a full penetration K weld will allow good transfer of the support reaction to the web via the welded connection (Fig. 6.3(b)).



(a) In general:
partial penetration fillet welds



(b) Support region:
full penetration K weld

Fig. 6.3 Examples of web to flange welds.

Changes of Section

Wherever there is a change in the width or thickness of a flange, or the thickness of a web, it is necessary to connect two plates by means of a transverse, or *butt weld*. Wherever possible, and particularly for erection joints welded on site, the locations of changes in section should be kept away from zones that are highly stressed (for example, above intermediate supports or at mid-span). The number of changes of section along the length of a beam, or the length of zones of constant dimensions, depends on achieving a good compromise between optimal utilisation of the material and the cost associated with the work needed to form the joints. Currently in Europe, where the cost of labour is relatively high, the trend is to limit the number of changes of section.

Changes in the sections of the flanges and the web need not necessarily occur at the same location along the length of a beam. Connecting two adjacent flange or adjacent web plates is achieved using full penetration welds in order to guarantee perfect continuity of the beam section, and good transfer of both normal and shear stresses acting on the section. Continuity of the flanges is normally achieved using X welds when they are formed in the fabrication shop, or V welds when they are made on site. Where backing is not possible, the welds are made from one side followed by grinding or gouging the root to sound metal for completion (by welding) from the other side.

From a conceptual point of view, changes in flange thickness (Fig. 6.4) can be achieved in two ways:

- the depth of the web can be kept constant, which results in a steel beam with mildly varying overall depth (Fig. 6.4(a)). Although this solution is practical for the fabrication shop, it is not compatible with some methods of erecting the steelwork, such as launching, or of constructing the concrete slab, such as sliding it into place,
- the overall beam depth can be maintained constant with a web that is of variable depth (Fig. 6.4(b)). This is the most common solution, given the options for bridge erection that it facilitates and the better appearance.

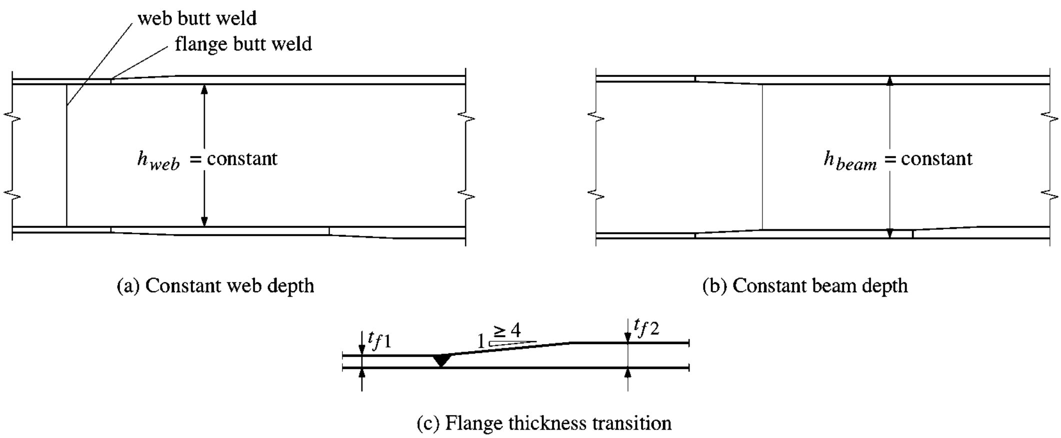


Fig. 6.4 Details of changes in plate thickness for flanges and webs.

The connection between flange plates of differing thickness is achieved by machining the thicker plate to assure a smooth transfer of forces (Fig. 6.4(c)). The slope in such a transition zone should be less than 1:4 to assure good performance, including adequate fatigue resistance.

Independent of any changes in section, consideration should be given to the erection joints that must be achieved on site to join individual lengths of beam. One issue with such welds is that they are formed in more adverse conditions than those in the fabrication shop (wind, temperature, moisture). A second issue is that the beam cannot be turned to ensure the weld is made with the beam in an optimal position. The number of welds that must be made on site should therefore be reduced to a minimum. The number of erection joints that will be needed clearly depends on factors such as site access and transportation from the fabrication shop to the site (which influence the size and weight of the individual beam elements). It also depends on the on site options for lifting and locating individual pieces of the steel structure.

Figure 6.5 shows an example of a beam erection joint. Prior to and during welding, the two parts must be firmly held in place relative to each other, to assure the geometry of the beam and to maintain the gap between the two plates that is needed to form the weld correctly (which depends on the weld type and plate thicknesses). For the example shown in Figure 6.5, bolted splice plates that will be removed after welding temporarily hold the plates together. Other options for temporary holding in place are available, and the choice will vary according to the fabricator's preferences.

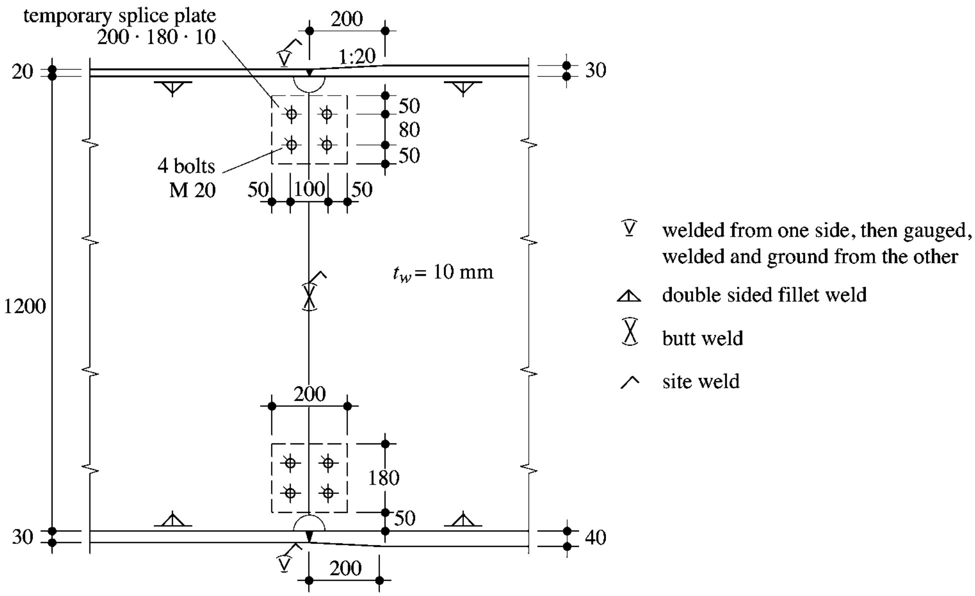


Fig. 6.5 Example of a site welded erection joint.

To reduce stress concentrations resulting from the intersection of three welds, cope holes (also called mouse holes) are often cut into the web to coincide with the location of the transverse flange welds. Another option for reducing stress concentrations due to crossing welds is to displace longitudinally, by several tens of centimetres, the web joint relative to the flange joint. For this option a cope hole is often cut into the web where it coincides with a transverse flange weld made on site, not only to facilitate forming the welds (root welding of the upper flange plates), but also to facilitate their checking via ultrasound.

If suitably qualified labour is not available on site for welding, the erection joints of a beam may also be bolted. A bolted joint is achieved using splice plates for the web and flanges (TGC Vol. 10, § 9.4.2), and often requires a considerable number of bolts for plate girders used in bridges. In Europe the preference tends to be to create welded joints, as they achieve greater continuity of the beams and have a better appearance.

6.3.2 Stiffeners

The function of the stiffeners that are welded to the webs and flanges of plate girders varies according to both their location along the length of a beam and their orientation. Their orientation may be parallel to the longitudinal beam axis, in which case they are called longitudinal stiffeners, or perpendicular to this axis, in the case of transverse stiffeners. For I section plate girders, the transverse stiffeners are also known as vertical stiffeners. As their name implies, the funtion of stiffeners is to stiffen the plate to which they are welded in order to improve the effectiveness of the plate in contributing to the section resistance. When fulfilling this function they serve as lateral supports for the plate, thereby stabilising it against out-of-plane deformations and preventing local buckling of the plate when it is subject to compression (Sect. 12.6). Transverse stiffeners placed above the beam supports fulfil an additional function, which is to help transfer the support reactions into the beam.

Figure 6.6 shows an example of a box girder with both transverse and longitudinal stiffening of its flanges. The region of the box shown in this figure is representative of that which would be found in a negative moment region near an intermediate support. The longitudinal stiffening of the lower flange, which will be subject to compression, is clearly more important than that of the upper flange, which will be in tension. In the final state the upper flange will be connected to the concrete slab by shear studs to ensure composite action, so the longitudinal stiffening of the upper flange is only needed during erection, when the self-weight of the concrete acts, but before composite interaction is achieved. When the slab is cast in situ, the upper steel flange acts as formwork for the concrete and must resist both global and local effects of the concrete.

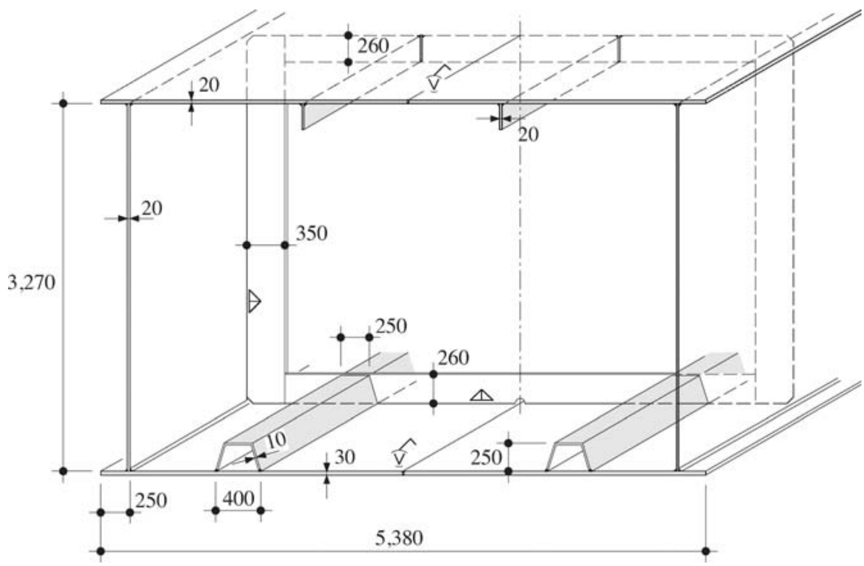


Fig. 6.6 Example of stiffening of a box girder.

Longitudinal Stiffeners

The function of the longitudinal stiffeners welded to a flange is to ensure that the flange contributes effectively to the ultimate bending resistance of the beam. In the case of slender I beams, the flanges are nevertheless normally designed to be sufficiently compact (§ 12.2.3) to be totally effective in resisting bending, making the addition of longitudinal stiffeners unnecessary. However, for box girders the flanges are generally more slender, and longitudinal stiffeners are needed.

The addition of longitudinal stiffeners welded to a web in order to make it completely effective in resisting bending is not generally economical. The extra cost of the labour needed to achieve a marginal increase in bending resistance (work associated with welding the stiffeners, and accommodating intersections with transverse stiffeners) is generally more than the cost of slightly increasing the flange thickness in order to give the same improvement in section performance. Consequently, for plate girders the area of web that contributes to the bending resistance is defined in terms of effective width (§ 12.2.5).

However, longitudinal stiffeners welded to a web are necessary if that part of the web is very slender and subject to compression. In that case, the web slenderness is reduced by the longitudinal stiffeners, which resist the small out-of-plane movements of the web (web breathing) created by traffic loads (§ 12.7.3). If these movements are excessive, over the long term they can lead to fatigue cracking in the regions of the welds between the web in compression and the flange or vertical stiffeners.

Longitudinal stiffeners, like transverse stiffeners, are typically attached using fillet welds. The stiffeners are usually one-sided and located between the main beams, or inside box girders, so they are not visible when the bridge is viewed from afar. Stiffeners may be formed from flats, Tee sections, or angles, all of which are open sections with no torsional stiffness (Fig. 6.7(a)). Alternatively, they may be of a form that, once connected to the plate, becomes a closed section (Fig. 6.7(b)). In the latter case the stiffeners do possess torsional stiffness and can therefore provide some rotational restraint to the plate, rather than being just a simple support.

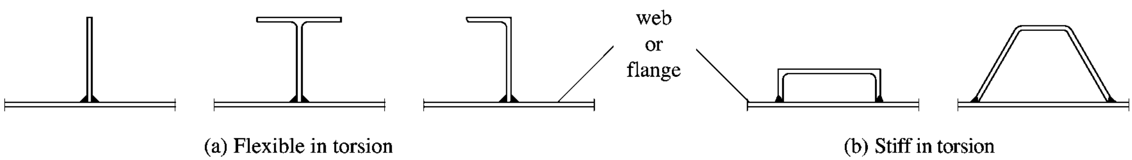


Fig. 6.7 Longitudinal stiffeners.

Transverse Stiffeners

Transverse stiffeners welded to the web of a plate girder enhance the shear resistance of the beam (Sect. 12.3). The resistance is greater when these vertical stiffeners are closely spaced. Therefore, transverse stiffeners are more closely spaced in the support regions, where the shear force is greater than in the spans (Fig. 6.8). In general, the spacing of the stiffeners in the support regions is roughly equal to the depth of the beam, whereas in the span, the increased spacing may even be as great as the distance between the cross bracing – of which the stiffeners are an integral part.

The *intermediate transverse stiffeners*, namely those located in the span, are normally fillet welded to one side of the web only (between the main beams). They generally comprise either flat plates or Tee sections.

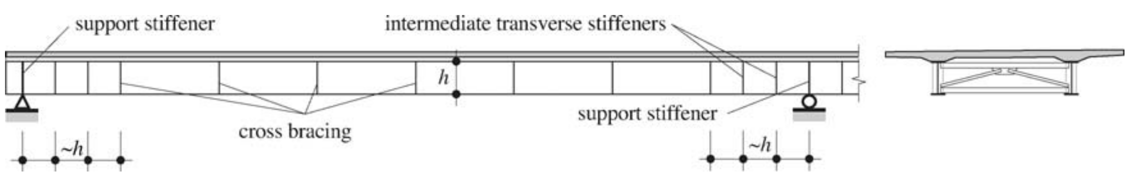


Fig. 6.8 Example of transverse stiffening of a plate girder.

In addition to being welded to the web, they are welded at their ends to the two flanges. Even though welded transversally, such a connection to a flange in tension is not the best connection detail as far as fatigue resistance is concerned (Sect. 12.7). However, it is better than leaving the ends of the stiffeners unattached (Fig. 6.9). This is because under traffic loading, any area of web left free in the gap between the end of a stiffener and a flange would be subject to small out-of-plane movements. These small movements provoke increased stresses in that region of web, which could lead to fatigue cracking, as has been seen on several existing bridges.

If the upper flange is not stressed perpendicular to its plane by variable actions, then the weld between a transverse stiffener and an upper flange may be made using a partial penetration fillet weld. For composite bridges, once the slab is fixed to the upper flanges of the main beams, traffic loading can lead to local stresses that cause fatigue cracking in such details, so it is indeed preferable to join the stiffeners to the upper flange using full penetration welds.

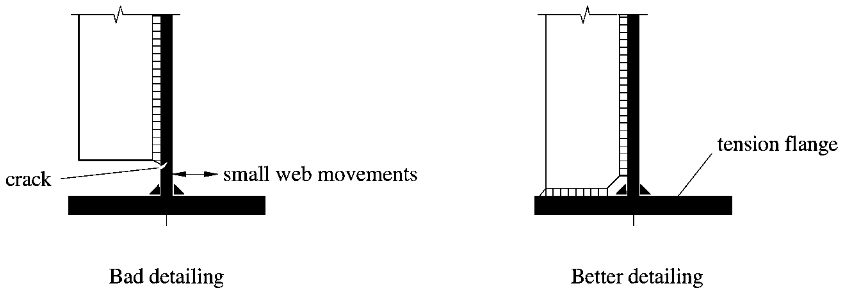


Fig. 6.9 Connection of transverse stiffeners to a flange subject to tension.

For transverse stiffeners formed from Tee sections, such as those that also fulfil the function of the uprights in frame cross bracing, the connection between the flange of the Tee and the tension flange of the beam itself is a detail that has diminishing fatigue resistance as the length of the Tee flange increases (Sect. 12.7). If necessary, the flange of the Tee may be cut back so that only its web is welded to the beam flange. Figure 6.10 shows examples of connecting a stiffener to the tension flange. Cutting back the flange of the Tee stiffener facilitates manual welding of its web. To allow effective welding around its total perimeter, the depth of the web of the stiffener should be less than the width of the flange outstand.

When the beam takes the form of a box section, it is also necessary to consider transverse stiffeners welded to the flanges (Fig. 6.6). One of their different functions is to guarantee the shear resistance of the flanges, which are stressed by shear flow in the closed section. They also serve as intermediate transverse supports for the longitudinal stiffeners attached to the compression flange. When both transverse and longitudinal stiffeners are welded to the same plate, consideration should be given to where they intersect. Two solutions are possible:

- Adopt continuous longitudinal stiffeners with transverse stiffeners that are cut to the form of the longitudinal stiffeners. Such a solution allows automatic welding of the longitudinal stiffeners, but requires the transverse stiffeners to be cut in such a way that they can properly fit around the longitudinal stiffeners.
- The longitudinal stiffeners may be discontinuous where they cross each transverse stiffener. While this option avoids the need to cut the transverse stiffeners, it often means that it is not possible to achieve satisfactory automatic welding of the longitudinal stiffeners (between the transverse stiffeners). In addition, restrained shrinkage during cooling of the welds at the ends of the longitudinal stiffeners results in tensile residual stresses in the welds.

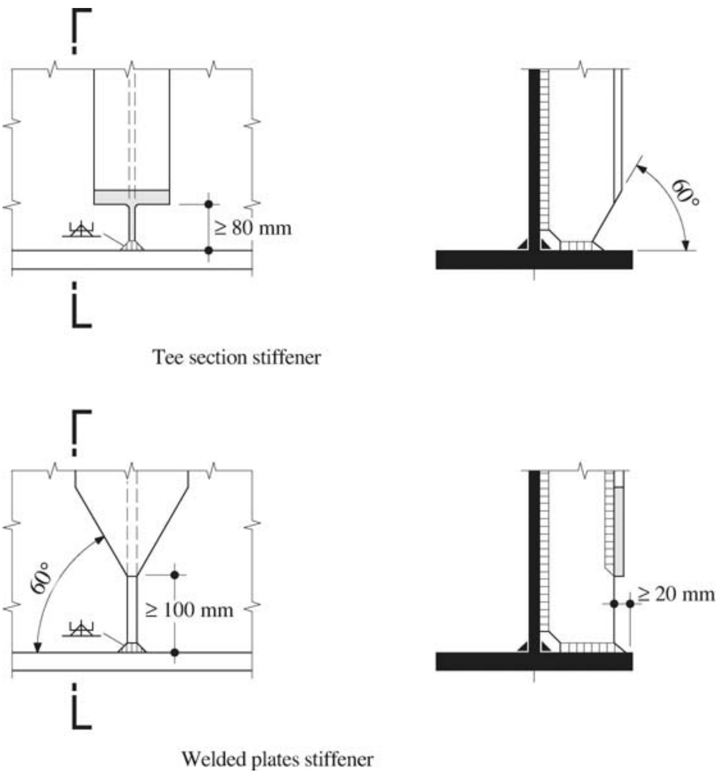


Fig. 6.10 Example of the connection of a transverse Tee stiffener to a flange subject to tension.

The appropriateness of one of these two options depends on the relative number of longitudinal and transverse stiffeners, as well as the preferences of a given fabricator. However, the first solution is generally preferable because it allows fatigue sensitive cruciform joints to be avoided.

Support stiffeners are normally located on both sides of a web, both to facilitate centring of the support reaction when transferred to the beam and to avoid introducing bending moments due to asymmetric stiffening relative to the plane of the web. These stiffeners are connected to the web by fillet welds. In the region immediately above the lower flange, to achieve a good transfer of the support reaction between the flange and stiffener, it is better to use a full penetration weld. For connecting the stiffener to the upper flange, as for intermediate stiffeners of a composite bridge, a full penetration weld is also preferable. Above the intermediate supports of a composite bridge, it is often possible to weld the flange of a Tee section support stiffener to the upper tension flange of the beam. In this region of a beam, the variation in tensile stresses due to fatigue loading is smaller than that in the lower flange in span.

The choice of cross section for the support stiffeners (Fig. 6.11) depends on the magnitude of the support reaction. The simplest solution, for small bridges, comprises two flat plates (Fig. 6.11(a)). If the support is free to move in the longitudinal direction, it is better to adopt two Tee section stiffeners. These contribute to good out-of-plane stiffness for the flange, which is helpful when the support reaction is not exactly aligned with the stiffeners. However, the lower flange in a support region is often a low point of a beam due to its camber, or indeed because the beam is of variable depth. In such a location, the use of Tee section stiffeners may result in dirt and water build up in that area. To avoid such a water trap, which would adversely

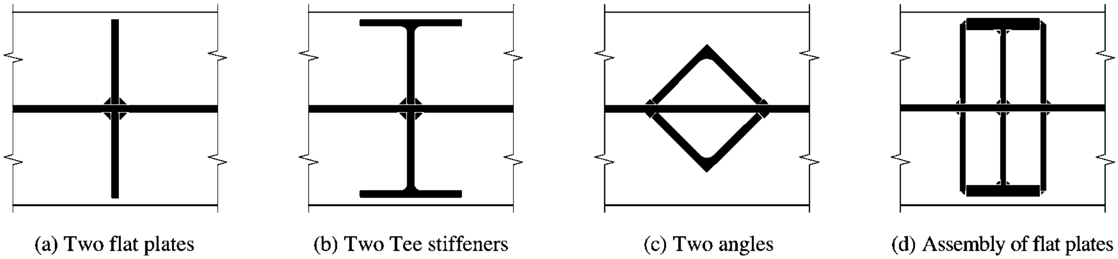


Fig. 6.11 Examples of support stiffeners (cross section).

affect the durability of the beam, closed section stiffeners may be adopted to allow water to run away (Fig. 6.11(c)). Finally, to allow the transfer of large support reactions, it may be necessary to adopt more complicated stiffeners fabricated from flat plates (Fig. 6.11(d)). When such a solution is adopted, careful consideration must be given to the sequence of welding to assure high quality fabrication of the stiffener.

6.4 Cross Bracing

This section presents construction details for cross bracing of bridges formed from either I section or box section main beams. The types of cross bracing that may be used with these bridges, and their particular function in contributing to the overall performance of the structure, are described in Section 5.6.

In general, one distinguishes between the cross bracing *in span* or *at supports* according to its location. The latter is more highly stressed, and is therefore bigger than the bracing adopted in the spans (Sect. 14.3). Cross bracing at the supports may also fulfil the function of temporary support for the main beams when the bridge is being lifted, to change or repair the bearings. At the abutments the cross bracing is often extended beyond the beams in order to provide support for the deck slab. Such a design guarantees the geometry needed at the slab ends to ensure correct functioning of the road and expansion joints. In the following descriptions of three types of cross bracing, distinction is made where the conceptual design for the bracing in span and at the supports is different.

6.4.1 Frame Cross Bracing

Whether at support or in span, frame cross bracing comprises a girder (cross girder) and two uprights. The uprights function also as vertical stiffeners, and in support regions, they help with the load transfer of the support reaction into the beam web. The cross girder must be rigidly connected to the uprights to achieve the necessary frame action. The cross girders are normally erected as pieces on site and connected to the uprights by welds or bolts. Figure 6.12 shows an example of a welded frame bracing at a support and in span for a composite bridge with 50 m spans. The cross sections of the cross girders and uprights may be formed from welded plates or rolled sections (Fig. 6.12(a)), Tees for the uprights, and I sections for the cross girders (Fig. 6.12(b)).

The connections between the cross girder and uprights are normally stiffened using flat plates. These stiffeners help transfer the connection forces and are welded to the uprights and main beam webs using full penetration welds (Fig. 6.12(a)). For frame bracing in span, when the uprights are not symmetric relative to the beam webs, the stiffeners may be stopped short of the webs to facilitate fabrication (Detail 2 in Fig. 6.12(b)). The frame connection stiffeners may also serve as gusset plates for bolted connection of the diagonals needed for any temporary plan bracing (Fig. 6.13).

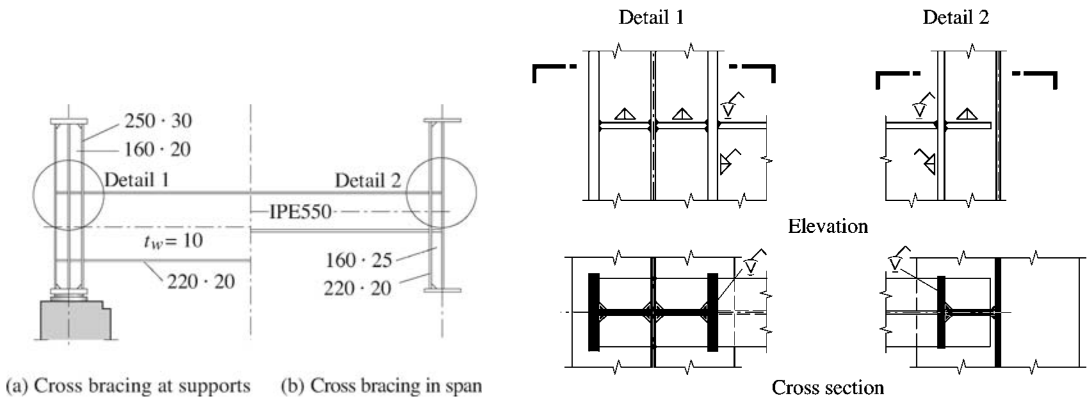


Fig. 6.12 Examples of details for a welded frame cross bracing.

For *welded bracing* the connection between the cross girder flanges and the uprights should comprise full penetration butt welds to assure good transfer of forces. For *bolted bracing* the cross girders should have end plates attached using full penetration welds. The end plates will subsequently be bolted to the flanges of the frame uprights using pre-loaded high strength bolts (Fig. 6.13). To transfer the bending moments in the frame connections, the end plates will normally be extended both above and below the cross girders, given that the bending moments may be either positive or negative depending on wind direction (with wind being the dominant action affecting them). Due to fabrication and erection tolerances,

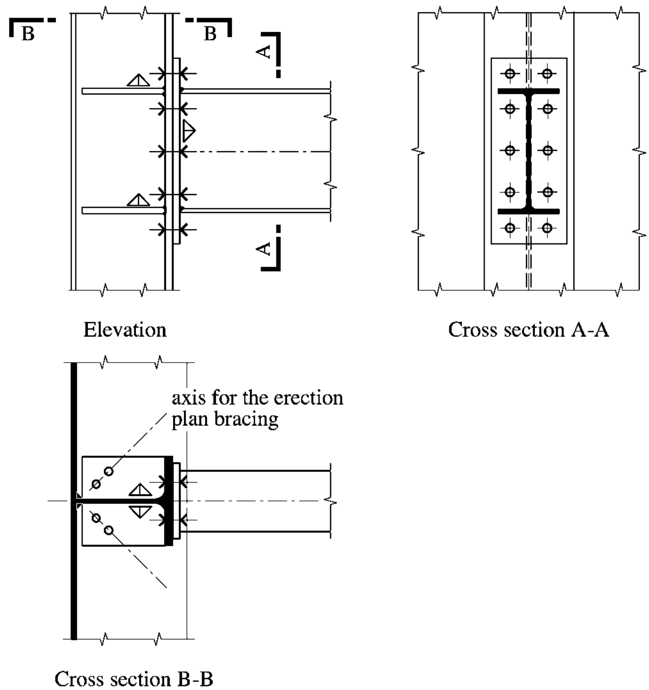


Fig. 6.13 Examples of details for a bolted frame cross bracing.

bolted bracing is easier to erect on site than welded bracing. However, because plates will not be perfectly flat, gaps may be present between the end plates and uprights with which they are in contact. These can allow water to enter and thereby compromise the durability of the joint.

For in span frame bracing, while the magnitude of the moments and forces given by the structural analysis may suggest that small rolled sections could suffice for the cross girders, the frame must have sufficient stiffness, as far as lateral deformation of the flanges is concerned. For this reason sections bigger than IPE300 are normally used. The positioning of cross girders within the depth of a bridge cross section is decided more by considerations of execution and performance in service than by resistance requirements. It is a function of factors such as supporting the slab formwork, for service walkways, or ducts for water or cables. It will be necessary to have access to the upper surfaces of the upper flanges of the cross girders for maintenance. The cross girders should be placed at mid depth of the main beams in order to provide efficient lateral support to the compression flanges of the main beams, to prevent lateral torsional buckling under both positive and negative moments, and considering both the erection and final states. Finally, in cases where the slab has transverse prestressing, the cross girders will be less affected by this force if they are located lower down in the cross section, away from the slab.

When the cross girders of frame bracing are joined to the slab of a composite bridge, they also support the self-weight of the slab, the permanent loads, and the actions due to traffic. Such cross girders are often more closely spaced when they support the slab. The latter may be thinner and of constant thickness if the *cross girders* extend either side of the main beams to support the slab cantilevers. This idea of supporting the slab over its full width can be extended to the cross bracing at the bridge ends in the regions of the road and expansion joints. In such locations the cross bracing has an additional function, namely that of guaranteeing the geometry of the slab ends, to assure correct functioning of the joints. Figure 6.14 shows an example of such cross bracing at an abutment. In this particular case the importance of the cross girder actually leads to the adoption of a deep beam, or diaphragm (§ 6.4.3).

When it is necessary to repair or replace the bearings, the cross bracing may be used as a support for the lifting jacks. Transfering the jacking forces to the beams via the cross bracing requires strengthening in the form of a stiffener welded above the intended jack position. Figures 6.14 and 6.15 show examples of such reinforcement details.

For box girders, the frame bracing takes the form of two girders and two uprights, for a closed box, or a single lower girder for a U shaped section. Figure 6.15 shows two examples of frame bracing for a U shaped box. For a composite bridge the open frame bracing is closed once the slab is in place. For a closed steel box, the frame bracing is implicitly closed.

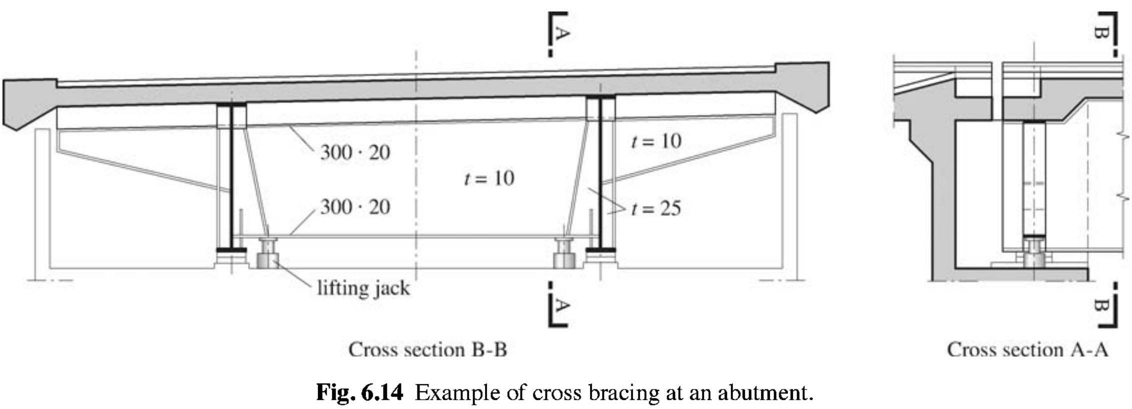


Fig. 6.14 Example of cross bracing at an abutment.

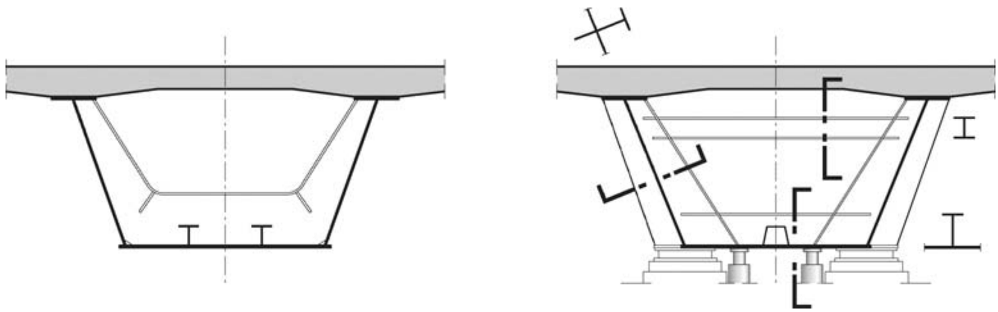


Fig. 6.15 Examples of frame cross bracing for a U shaped steel section.

6.4.2 Truss Cross Bracing

Truss cross bracing (Fig. 6.16) comprises steel sections for the chords and diagonals when the uprights are integral with the main beam webs, and fulfil the additional functions of web stiffening and transfer of the support reactions (for cross bracing at supports). Depending on the magnitude of the forces, the chords and

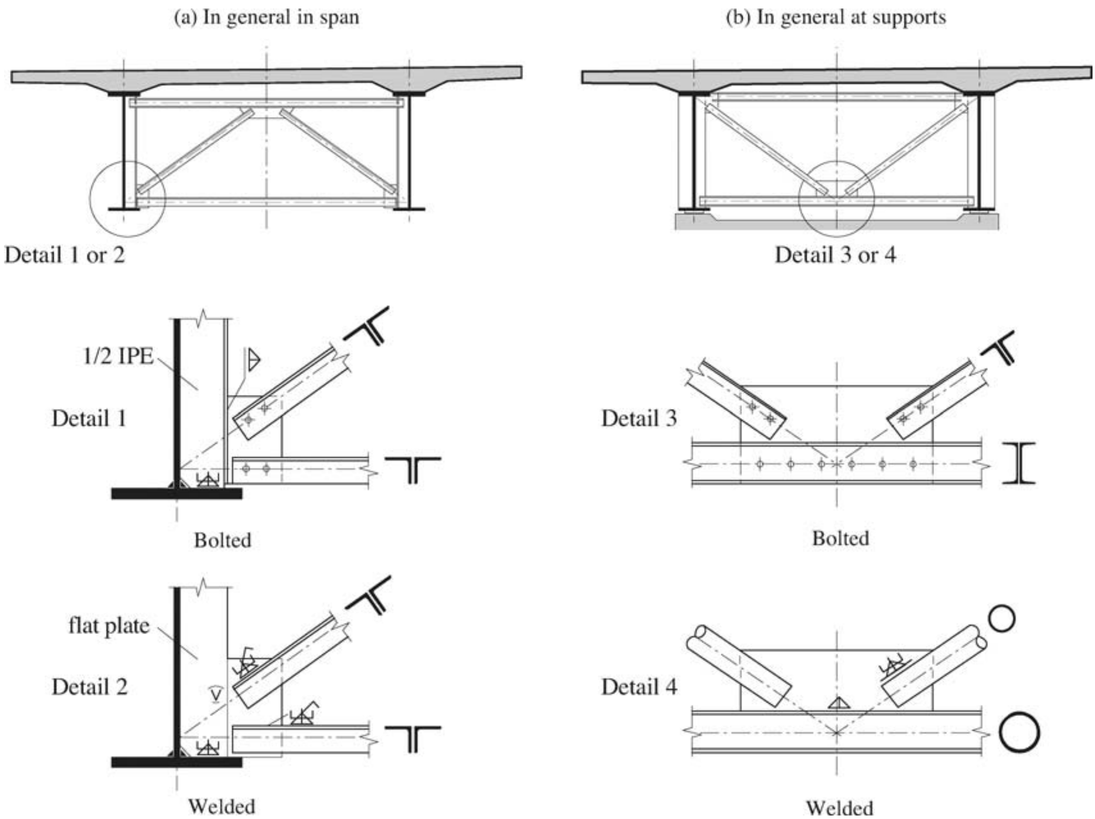


Fig. 6.16 Examples of truss cross bracing for open sections with either bolted or welded connections.

diagonals will be either single or double angles, channels (U section), or hollow section tubes. Truss bracing is often fixed to the main plate girders on site by bolting. Figure 6.16 shows schematically two forms of truss bracing for an open cross section bridge. The form shown in Figure 6.16(a) is often used in span if the upper chord also serves as support for the slab formwork. The inverted form (Fig. 6.16(b)) may be used at supports to enhance resistance against transverse buckling of the highly compressed lower chord.

Figure 6.16 also shows some details for the cross bracing connections. Because the cross bracing at a support may also fulfil the function of support for the jacks during lifting of the bridge, it must be designed such that it can resist the reactions from the jacks. Figure 6.17 shows an example of how this can be achieved.

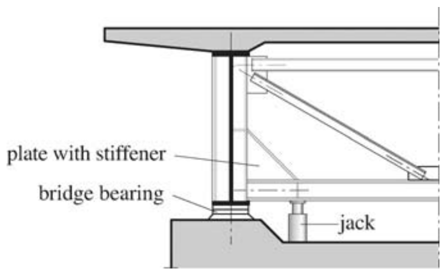


Fig. 6.17 Jacking details for lifting the bridge and for bearing replacement.

For box section beams, truss bracing is normally welded to the transverse stiffeners. The box is transported to the site either as a complete cross section or as two half cross sections. Figure 6.18 shows an example of truss bracing for a box fabricated as two halves for separate transportation, then welded together on site.

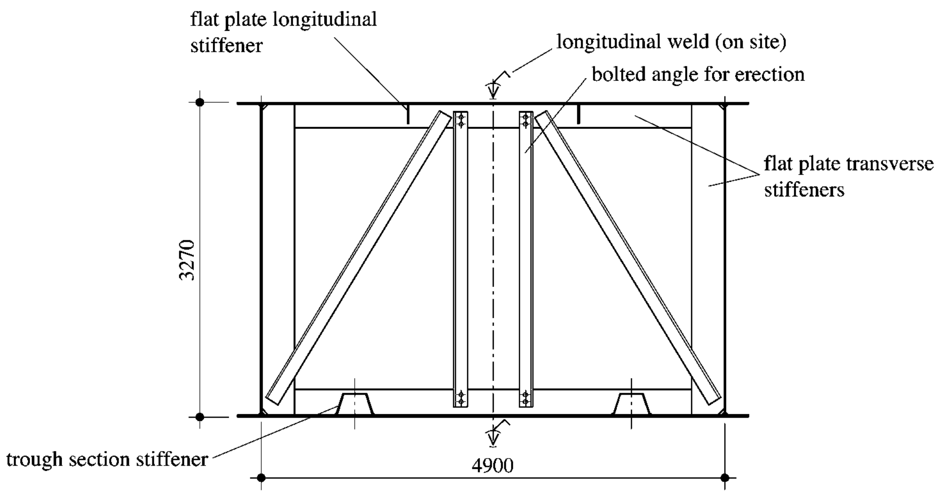


Fig. 6.18 Truss cross bracing in a box girder.

6.4.3 Diaphragm Cross Bracing

A diaphragm consists of a stiffened plate, welded around its total perimeter to the inside of a box section. Obviously, such a solution is only used for box girders. However, if a deep beam is used that occupies virtually the whole of the cross sectional area of an open section bridge, this is also known as a diaphragm (Fig. 6.14). Diaphragms in box girders are primarily used at support locations, where the internal forces acting on the cross bracing are greatest. Diaphragms are also commonly used for box girders that are shallow or narrow, because other forms of cross bracing are not well suited to such geometrical constraints. It is necessary to leave an opening in the diaphragms to allow inspection inside the box. Local stiffening will be needed around such openings, and in locations where the bridge will be subjected to jacking forces. General stiffening will also be needed to assure the shear resistance of the diaphragm. Figure 6.19 shows an example of a diaphragm.

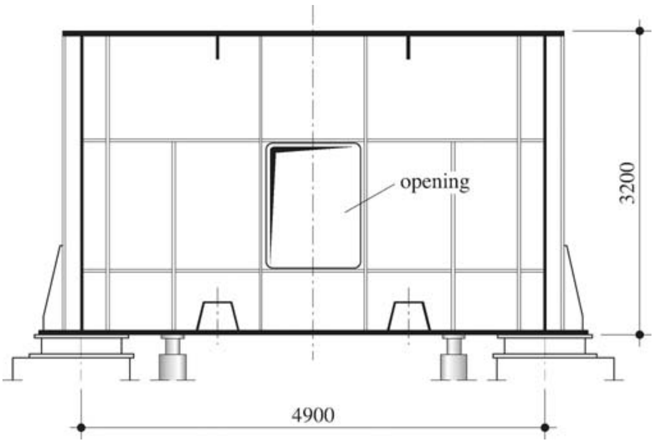


Fig. 6.19 Example of a diaphragm in a box girder.

6.5 Plan Bracing

Plan bracing is needed to transfer the horizontal forces that act transversally to the beam axis. For composite bridges the concrete slab plays this role, and no plan bracing is required in the final state. However, temporary plan bracing is needed during erection. This temporary bracing is normally removed after construction of the concrete slab and its connection to the steel structure.

The plan bracing is located between the main beams, generally at the level of one of the chords when truss cross bracing is used, or at the level of the cross girders when frame cross bracing is adopted. In the latter case the cross girders also form the "uprights" of the plan bracing system. The diagonals in the temporary plan bracing are normally in the form of a St Andrews cross (X), with the diagonals formed from angles. The angles are bolted to gusset plates welded to the uprights of the cross bracing (Fig. 6.20). If there is sufficient space, they may be bolted directly to the flat plate stiffeners, which are used to reinforce the connections of cross girders in frame cross bracing (Fig. 6.13).

Different forms of plan bracing, as well as different types of diagonal, are considered in Paragraph 5.7.2. For (non-composite) steel bridges, the plan bracing is not only used during erection, but remains functional during the whole life of the bridge. The diagonals may be subject to significant parasitic forces, depending on the form of the plan bracing and its position in the bridge cross section (§ 14.4.2).

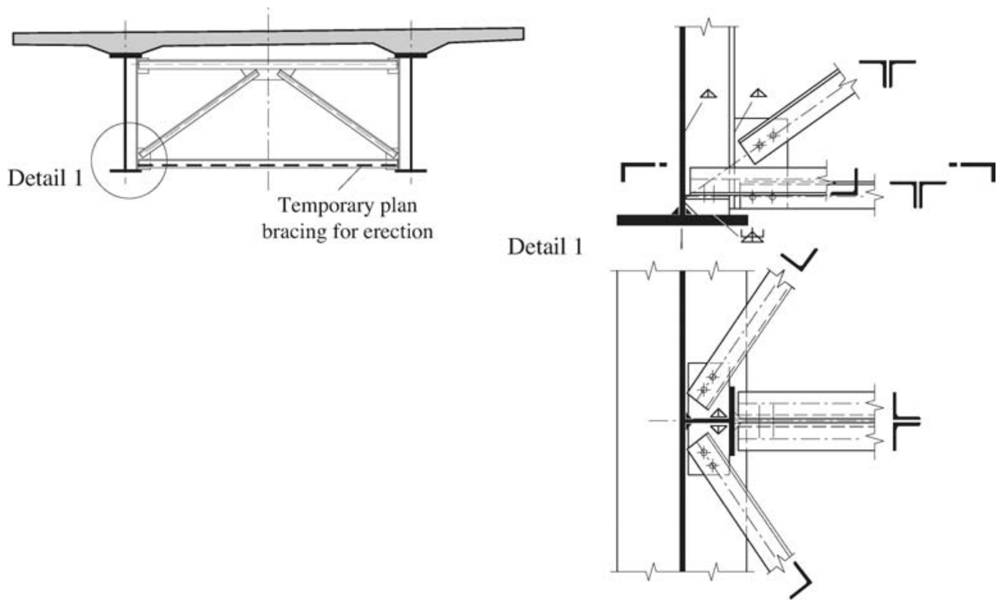


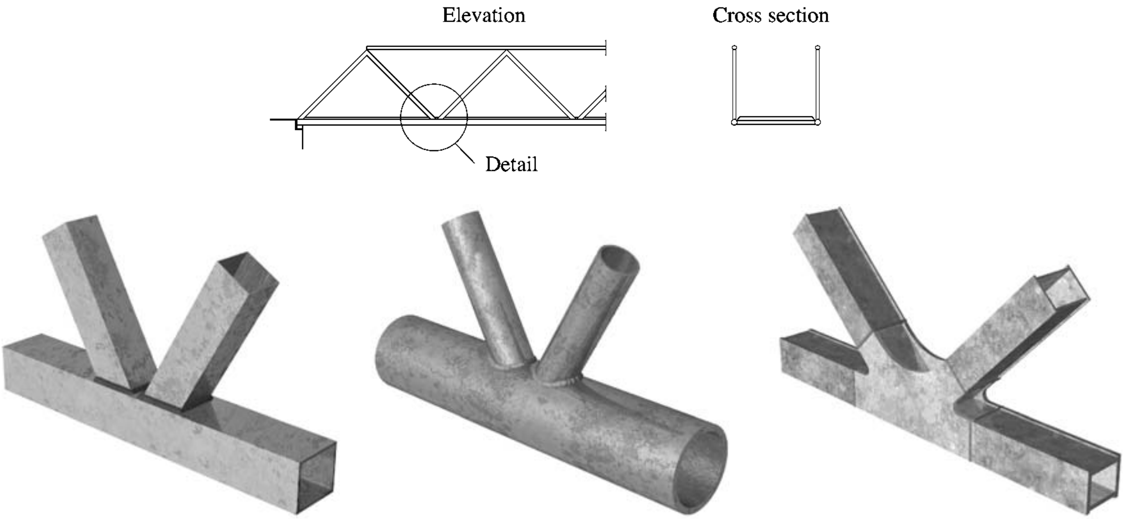
Fig. 6.20 Example of plan bracing in the form of a St Andrews cross for the erection stage, connected to the main beams.

6.6 Truss Beams

In terms of their general configuration, trusses used for road and railway bridges are no different from those adopted for other types of structures (TGC Vol. 10, Sect. 5.7 and TGC Vol. 11, Sect. 12.3). However, they are different in terms of the dimensions of the members forming the truss, which are heavier.

Generally, the truss members are formed from hollow sections, be they circular or rectangular, which may be fabricated from plates. In terms of maintenance and durability, hollow sections are preferable to open I sections. In particular, I sections allow dirt to easily build up and standing water to accumulate at the truss joints. Both of these are potentially harmful to the bridge's durability. Figure 6.21 shows three examples of members forming a Warren truss, which is a type of truss commonly used for road bridges and pedestrian footbridges. When Warren trusses are used for railway bridges, they are designed to include uprights. Figure 6.21(a) shows the detail of a K node between rectangular hollow sections, and is an example from a pedestrian bridge. Figure 6.21(b) shows the same detail for thick walled circular hollow sections, such as might be used for a road bridge. Figure 6.21(c) shows the same detail for fabricated hollow sections.

For bridges, the joints between the chords, the diagonals and the uprights must all be carefully considered in order to ensure a good transfer of forces and, in particular, minimise stress concentrations and ensure good fatigue resistance. As far as fatigue resistance is concerned, welded gusset plates used to transfer forces between the diagonals and chords, often found in trusses of building frames (TGC Vol. 11, Fig. 12.7(c)), are a poor choice of construction detail. For bridges, lengthy longitudinal welds to tension chords can be avoided by choosing a solution such as that shown in Figure 6.21(c). In this detail the gusset is cut from a plate such that it has rounded corners, which will improve the flow of stresses; also, this detail eliminates all longitudinal welding to the chord. The gusset plate forms part of the web of the chord. The welds between the members of the truss and the node should be full penetration butt welds.



(a) Rectangular hollow sections (b) Circular hollow sections (c) Welded box sections

Fig. 6.21 Joint details of a Warren truss.

Connections between hollow sections that are welded together without gusset plates (Fig. 6.21(a) and (b)) are valued for their simplicity and appearance. However, such connections do require considerable work to precisely cut the ends of the members to sometimes complex profiles. The ends must also be bevelled to allow full penetration welding. Because of the complex geometry, all welding must be done manually. The fatigue resistance of such details, notably in the region between welds for two diagonals to a chord, is still an important research issue.

An alternative way of connecting hollow section members without using gusset plates is to replace the entire node with a cast steel node. Figure 6.22 shows a view and a cross section of a K node formed around a steel casting. In this example the cuts at the ends of the truss members, and the welds, are much simpler. Butt welds are used around the perimeter of the node.

Trusses offer great flexibility of choice to the designer, who can play with the layout of the truss members and indeed adopt either a planar or spatial solution. Figure 6.23 shows a space truss supporting the

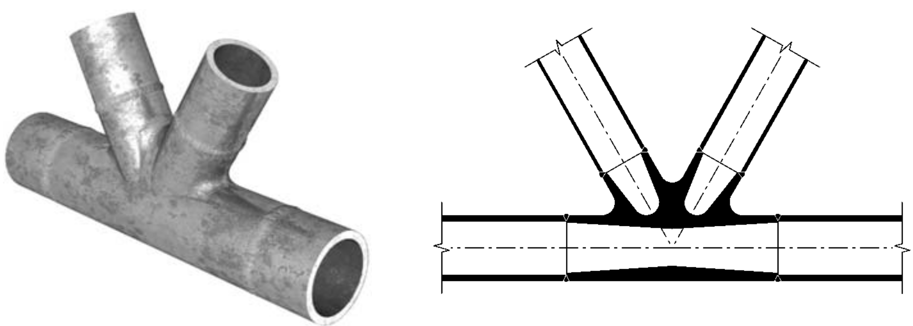


Fig. 6.22 Cast steel K node.

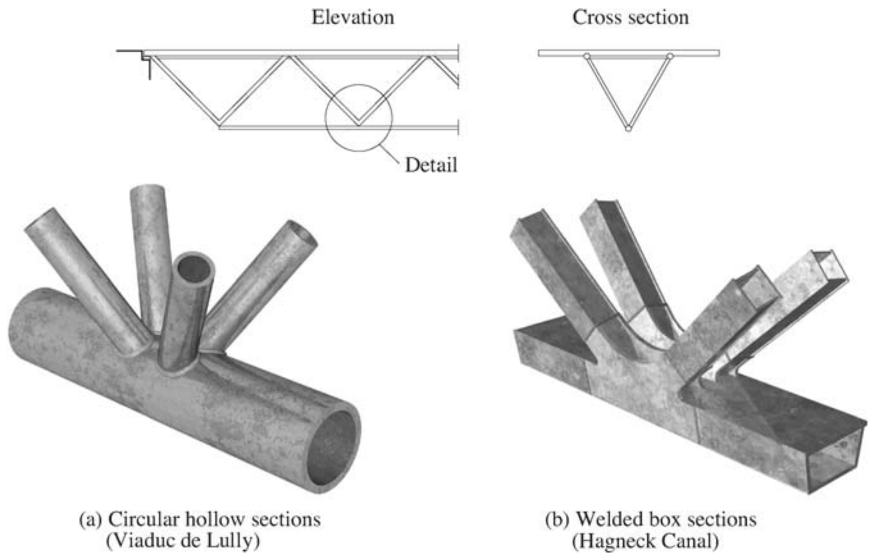


Fig. 6.23 Space frame truss and details of a double K node.

slab of a road bridge, as well as two examples of double K node details. One of these examples is for a truss formed from circular hollow sections (Fig. 6.23(a)), and the other is for hollow sections fabricated from flat plates (Fig. 6.23(b)).

6.7 Orthotropic Deck

The orthotropic deck of a bridge comprises a steel plate that is stiffened on its lower face by both longitudinal and transverse welded stiffeners. This plate also forms the upper flange of the plate or box girders (Fig. 6.24). Therefore, an orthotropic deck is subject to stresses resulting from global bridge behaviour, as it is part of both the longitudinal load carrying system and local behaviour in transferring concentrated actions, such as heavy wheel loads. In addition, the plate and its stiffeners are subject to important fatigue loading, because each wheel passage represents, locally, a load cycle. Because of this, the weld details for an orthotropic deck should be carefully detailed and fabricated.

The deck plate is stiffened in two orthogonal directions (Fig. 6.24) to pick up concentrated wheel loads and transfer them, by bending, to the main beams. The longitudinal stiffeners may be of an open cross section, such as flat plates or angles, or box sections. The latter possess greater torsional stiffness and thus provide better stiffening to the plate, thereby reducing its vertical deformation and reducing local bending stresses due to concentrated actions. Modern bridges commonly use box section longitudinal stiffeners for their decks. The box section is obtained by a trough welded to the plate. The troughs are formed by bending plates that are typically 6 mm or 8 mm thick. Transverse stiffening is normally achieved using elements of the cross bracing, which are welded to the plate. This element is either a diaphragm, when that solution is adopted for the cross bracing, or a cross girder when frame bracing is used. Alternatively, it may be just a simple transverse stiffener located between the cross bracings, as necessary.

The plate thickness for an orthotropic deck is usually 12 mm or more (EN 1994-2). The plate should not be too thin, particularly under the traffic lanes, in order to provide sufficient local bending stiffness.

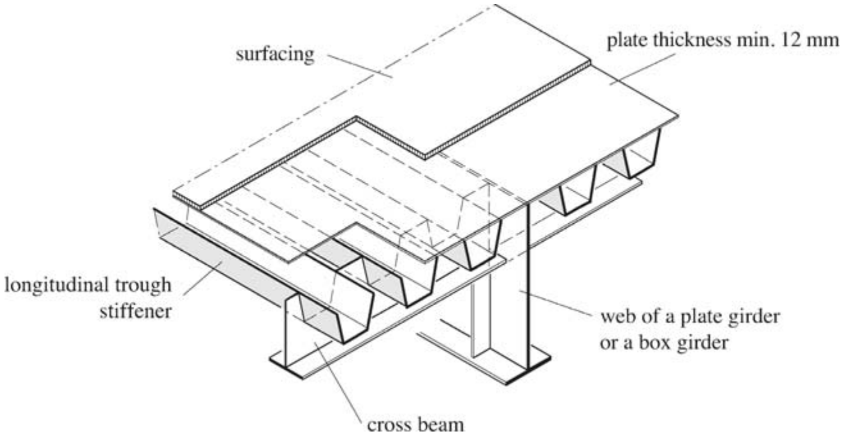


Fig. 6.24 Example of an orthotropic steel deck.

Fatigue stresses are then reduced, and the fatigue resistance is thereby increased. In addition, when local deformations are smaller, the surfacing adhering to the plate will last longer. Longitudinal stiffeners in the form of troughs are generally spaced at 300 mm centres (Fig. 6.25). They have a depth of 250 mm to 300 mm and a width between their webs of 300 mm, which reduces with depth, to a width of 100 mm to 150 mm. The longitudinal stiffeners are supported by the transverse stiffeners, which are normally spaced about 4 m apart.

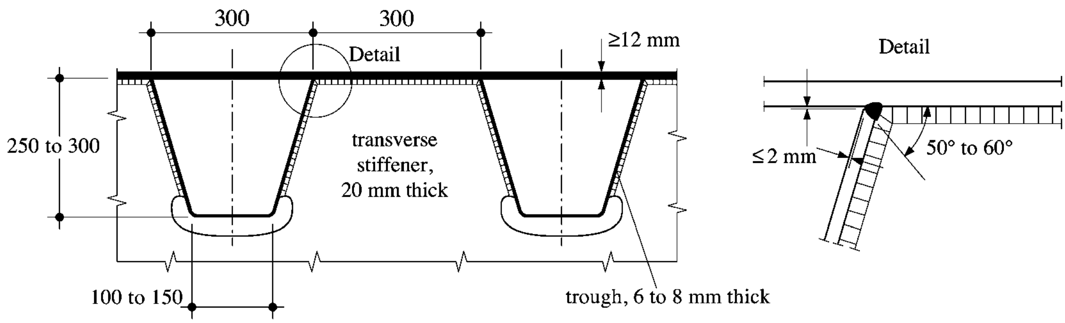


Fig. 6.25 Examples of attaching trough shaped stiffeners to the deck plate and transverse stiffener.

Negative experiences concerning the fatigue resistance of orthotropic decks have led to the preferred solution of using continuous longitudinal stiffening (troughs) that passes through the transverse stiffeners (rather than using discontinuous longitudinal stiffeners that are welded to either side of the transverse stiffeners). At the intersections of the troughs with the transverse stiffeners, the latter are coped so that the troughs can pass through the openings that are formed. Normally only the webs of the longitudinal stiffeners are welded to the transverse stiffeners, whereas their bases are not welded; this avoids fatigue cracking in that zone (which is very susceptible to fatigue). The cut-outs in the webs of the transverse stiffeners have rounded corners to reduce stress concentrations (Fig. 6.25). The web openings must also be sufficiently large to permit weld returns, thereby avoiding detrimental notches and facilitating application of the corrosion protection. EN 1994-2, which covers the design of bridges, contains guidance on the detailing of orthotropic decks.

Orthotropic decks are protected by a waterproofing layer and surfacing that often comprises bituminous concrete with a total thickness between 60 mm and 80 mm. Both the surfacing and the waterproofing layer must be flexible and well adhered to the plate. Although thicker surfacing helps to reduce the bending fatigue stresses in the plate, it also increases the weight of the deck, and this goes against the desired aim of reduced self-weight that is the main driver for using an orthotropic deck. Other types of surfacing may be used (hot asphalt, composite materials, etc). However, they must be able to achieve good interaction with the plate, and their influence on the distribution of local stresses in the plate must be understood.

6.8 Other Components

Other components used in bridges, such as road and expansion joints and bearings, are described in Section 2.4. The size of these components, and the construction details adopted to attach them to the structure itself, depend on the forces to be transferred, the movements to be accommodated, and the type of components specified. The designers choose these components based on offers from potential suppliers [6.2 to 6.5]. The components' descriptions must contain information regarding their placement and attachment to the structure. The expansion joints must assure a waterproof solution, and the choice of bearings must prevent dust and dirt accumulation.

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7 Fabrication and Erection of the Steel Structure

Erection of the Viaduc des Vaux near Yvonand (CH).
Eng. Realini + Bader & Ass. ingénieurs-conseils, SA, Lausanne and Giacomini + Jolliet, ingénieurs civils & Ass. SA, Lutry.
Photo Bureau des autoroutes, canton de Vaud.

7.1 Introduction

Fabrication comprises all the operations undertaken in the workshop that are necessary for the production of the steel structure. Starting from flat plates and rolled sections, the fabrication process in particular includes cutting of plates, sawing of rolled sections, drilling, grinding, and connecting by either welds or bolts. The structural elements produced in these processes, and – where necessary – also protected against corrosion by paint, are then transported to the site. The means that are available for *transportation*, and the accessibility of the site, dictate the maximum size and weight of the individual structural elements to be joined on site.

Erection covers all the operations carried out on site that are needed to construct the steelwork. The method of erection is dictated by the site conditions, by the available cranes, and by the experience of the organisation that will carry out the erection. Careful thought should be given to the method of erection; such consideration should start at the very beginning of the bridge project, so that any impacts of the method on the structural concept, and its detailing, are identified sufficiently early. The method chosen for the erection should aim to achieve optimal construction of the bridge, in terms of assuring the safety of the workforce on site and controlling the costs.

A study of the history of construction reveals that numerous accidents have occurred during erection, even though this lasts for a much shorter period than the service life of a structure. These accidents have often cost a number of lives, as well as causing material damage. The designer must be conscious of his responsibilities, and vigilant to ensure that the various erection phases are not only carefully considered and clearly defined, but also strictly adhered to by those carrying out the work on site.

Sections 7.2 to 7.4 cover the fabrication of the steel elements in the workshop, their transportation, and joining together on site. These sections are followed by Section 7.5, which describes in more detail the different erection methods for the steel structure. The potential impacts of the erection method on the conceptual and structural design are also explained in this section. Finally, Section 7.6 describes the tolerances to be considered for both fabrication and erection. Construction of the concrete slab of a composite steel-concrete bridge is considered in Section 8.4.

7.2 Fabrication in the Workshop

7.2.1 Receiving and Preparing the Plates

The first step in fabricating a steel bridge is to order the materials: plates, rolled sections, and connectors (welding material and bolts). It is worth noting that the availability of the steel, and therefore the time for delivery, may vary considerably depending on the state of the market and the required grade and quality. The characteristics of the steel, as described in Paragraph 4.5.1, must be precisely specified. Plates are delivered with a certificate that guarantees both the chemical composition and mechanical properties of the steel. Upon delivery the fabricator marks the plates with a number cross-referenced to the certificate to assure traceability using the organisation's quality procedures. If required, tests can be carried out to check whether the material delivered corresponds to the specified grade and quality. Currently, the tests most commonly used are tensile tests, to check grade, and resilience tests, to check quality (TGC Vol. 10, § 3.3.1).

For the fabrication of both plate girders and box girders, the plates are cut to the required dimensions, generally using flame cutting. To facilitate welding it is necessary to chamfer the plate edges either by grinding or, if possible, as part of the cutting process for thick plates. If bolted joints are adopted, then the necessary holes must be drilled. Punching to form these holes, unless followed by reaming, is not acceptable for components that will be subject to variable loading.

7.2.2 Fabrication of the Structural Elements

It is easier to achieve good quality welded joints in the fabrication shop than on site, given the better working conditions (for example: inside working is not affected by the weather, there is the possibility to turn elements so they are in the best possible position for welding, and all working can be carried out at ground level). It is also possible to achieve a more consistent weld quality in the shop, because the welds can be formed automatically using welding robots. Automatic welding methods are of particular interest for the long runs of weld needed to fabricate plate girders.

According to the complexity of the steel structure, and if site bolted joints are to be used, it may be wise to undertake a preliminary erection in the workshop to check that the geometry and means of adjustment are correct and adequate. This operation is known as a *trial erection*. It means that significant corrections or adjustments to the structure, which are normally expensive, can be avoided on site. When the erection joints are welded, it is not necessary to undertake a trial erection, as there is normally enough play between the plates to achieve the correct geometry for the structure.

7.2.3 Welds

Welds contain imperfections (TGC Vol 10, § 7.3.4) of varying sizes that may, according to their size, lead to a weakening of the beam (fatigue, brittle fracture). In order to guarantee the required weld quality, it is necessary that they are made by competent personnel using qualified weld procedures (SIA 263/1). Additionally, there must be checking procedures in place to guarantee the welds are properly made according to the requirements. The frequency of checking, and locations where checks must be made, are a function of the weld quality classification. The quality class depends primarily on the levels of stress to which the weld will be subjected, and the importance of the structural element under consideration. Classification ranges from class A, for exceptional cases with very stringent requirements, to class D, for lightly loaded structural welds (SIA 263/1).

The most basic form of weld checking is a visual inspection. Next, according to the quality class, comes a non-destructive test using magnetic particles, dye penetration, ultrasound or x-ray inspection. The standard SIA 263/1 provides details of appropriate tests to carry out as a function of the weld quality classification and the implicit requirements.

7.2.4 Corrosion Protection

At the end of the fabrication process, the steel members are cleaned by shot blasting then quickly protected against corrosion. Shop applied corrosion protection comprises, as a minimum, a base layer of paint. In some cases all the paint layers may be applied in the shop (§ 4.5.5). Any plate edges that are to be connected by welding on site should be protected from the paint using plastic tape. When weathering steels are used, there is no need to paint the steelwork at all.

7.3 Transportation

Once fabricated, the structural elements are transported to site, where they are erected into position. The maximum dimensions and weight of the shop fabricated elements may be limited by the capacity of the fabrication shop, the means of transportation, site access, or the method and equipment to be used for erection. As far as transportation is concerned, the main factors influencing the size and weight of individual components are:

- rail: headroom limitations on the rail network, load limits on the wagons, the length of the wagons,
- road: headroom under bridges, load carrying capacity of available vehicles, load limits on any bridges on the access routes, the ease of access (straightness, width) of the route,
- boat: the load carrying capacity of boats able to navigate the access route, headroom limitations under bridges, etc.

For rail transportation the weight limit for normal wagons is between 500 kN and 700 kN. In exceptional cases, special wagons can be used to transport greater loads. The maximum length that can be transported by rail is around 20 m. Although many steelwork fabrication shops are linked to the rail network, the same is not true of construction sites. Rail transport is rarely used, therefore, without complementary road transport. In such cases the more restrictive of the limits concerning both rail and road transportation will dictate piece size and weight. Additionally, combined rail and road transportation necessitates extra handling, when compared with transportation by road alone.

On the Swiss road network, for example, it is normally possible to transport items within the following limits without the need for special authorisation:

- length: 30 m
- width: 3 m
- height: 4 m
- total weight: 440 kN

If only freeways are used, the weight limit is increased to 500 kN. For loads exceeding these values special transport authorisation is required and, depending on how far the limits are exceeded, the load will need to be accompanied by a security vehicle or by the police. Depending on the size and weight of an exceptional load, it may be necessary to close the route either entirely, or to traffic in one direction.

The maximum height of a road convoy is dictated by the headroom under bridges and within tunnels. On main roads the minimum headroom is 4.5 m. The maximum weight that can be transported may also be limited by any bridges to be crossed. In Switzerland some bridges have been designed considering SIA 261/1 load model 3. The total weight for such road convoys may be 4800 kN. Reference [7.1] contains guidance appropriate for such convoys.

In Switzerland road transportation is the most common. In European countries where the network of waterways is more developed (France, Germany, Netherlands), the use of barges can be an interesting alternative to rail or road for long journeys (§ 7.5.5).

Other means of transportation may be envisaged according to the accessibility of the site. For mountainous regions air transportation, by means of helicopter, may be the best or the only alternative. The capacity of a powerful helicopter is around 40 kN, which clearly places a very strict limit on the size of steel elements. The use of airships may also be envisaged for exceptional cases in the future.

7.4 Site Assembly

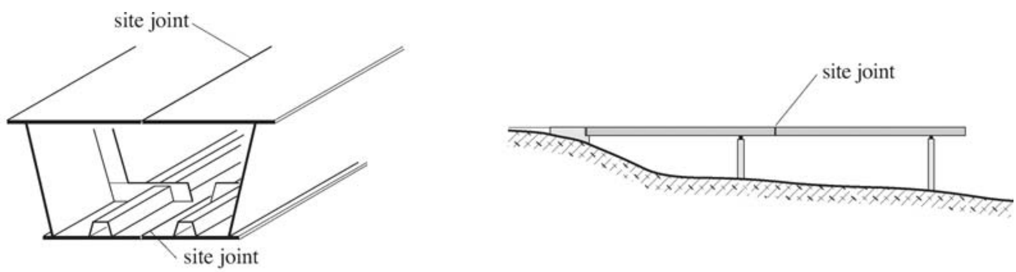
Joints to be made on site require the presence of suitably qualified labour. Even so, such joints are likely to be of poorer quality than those made in the fabrication shop. It is therefore wise to limit the number of site joints as much as possible. Detailed study of the different options for dividing the structure to minimise site joining is essential for the elaboration of a bridge project.

In Switzerland, as in a number of other European countries, welding of site joints is preferred because it results in completely monolithic behaviour of the finished structure, simplifies the operation of site painting, and results in a more uniform appearance. Welded joints also avoid trapping water or humidity between plates, as can happen, for example, when bolted splice plates are used. Bolted joints are only used for the site joints in plate girders in exceptional circumstances. However, they are frequently used to join the cross and plan bracing to the main beams.

Normally, welding butt joints between lengths of main beams on site requires the construction of a shelter to shield the welding operation from the weather. This is one aspect of the ways in which site welding is normally carried out in more adverse conditions than those found in the shop, and this can affect the weld quality. Because of this, more careful checking of the welds is needed, generally using ultrasound. All of this means that site welded joints are more expensive than those formed in the fabrication shop, so it is beneficial to limit their number.

The use of large steelwork sub-assemblies not only results in fewer erection joints, but also facilitates better execution. Therefore, it should be encouraged, as long as the transportation limits discussed in Section 7.3 are respected. Clearly, the lifting demands on site for large and heavy pieces are greater, and can lead to more significant stresses in the sub-assemblies. It is important to consider all aspects related to lifting early in the project elaboration, so they are properly taken into account in the bridge's conceptual design and detailing. It should also be remembered that site joints should be located in lightly loaded regions of the structure.

Figure 7.1 shows two examples of site joints between structural elements, indicating the positions of the joints. In the joining of two half boxes, shown in Figure 7.1(a), the joint welds are located in the middle of the box flanges, where the shear stresses are low. In the joining of main beams, shown in Figure 7.1(b), the joint is positioned in a region where the self-weight bending stresses are low. One should avoid placing a transverse joint in a beam at mid-span, or above an intermediate support.



(a) Joint between two half boxes

(b) Joint between beam sections

Fig. 7.1 Examples of site joints between elements of the steel structure.

For site joints, good access should be planned to facilitate the work of the welders, which is already difficult enough when working outdoors. In particular, the number of overhead welds should be minimised, as they require great care and are more tiring to make. Weld sequences must also be respected in order to limit residual stresses and distortions due to welding. Preheating of the weld regions, which is often required (§ 4.5.2), is helpful in limiting this phenomenon.

When site welding is carried out, it is necessary to plan means by which the pieces to be joined together are held in place. For example, this could mean using bolted splice plates, which are removed after welding. Figure 7.2 shows such an example of a site joint for a plate girder (see also Fig. 6.5). This clearly

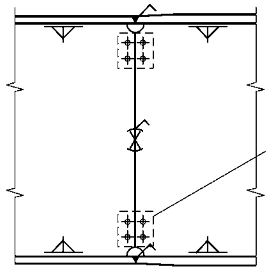


Fig. 7.2 Site welded joint with temporary bolted splice plates.

shows the temporary bolted splice plates and the site welds. Alternative details for holding together plates that are to be welded are possible, and will depend on the preferences of the chosen steelwork contractor.

If components specifically needed for the erection are fixed to the steel structure, for example temporary plan bracing, they should be removed once they are no longer needed. For structures that are susceptible to fatigue (railway bridges in particular), great care is needed to ensure that any temporary components needed for erection (and welded to the structure) do not constitute details that will reduce its fatigue resistance. If this is the case, then they should be removed after erection, with subsequent careful grinding of the welded zones.

7.5 Erection of the Steel Structure

The geographical and topographical characteristics of a bridge location influence the method used for erection of the steel structure. Considering only beam bridges, the principal methods of erection are:

- erection by crane from the ground,
- cantilever erection,
- erection by launching,
- placement of the complete bridge (or of large bridge elements).

These methods are described in more detail in Paragraphs 7.5.2 to 7.5.5. The technical difficulties associated with each method are highlighted, as well as the impact of the erection phases on the internal moments and forces acting on the steel structure. Paragraph 7.5.1 describes the fundamental characteristics of steelwork erection that are common to all the methods and that must be carefully considered when studying the issue of erection.

Other erection methods may also be considered on a case by case basis. Certain methods have been developed to respond to the specific needs and characteristics of some types of bridge. For example, placement of bridge elements using a trolley suspended from a cableway above the bridge is of interest for suspension bridges, because the pylons of the final structure can be used to support the cableway. Specific techniques have also been developed for arch and cable stayed bridges. Section 18.4 deals with this subject for arch bridges.

7.5.1 Specifics of Steel Erection

Today's erection of steel bridges is characterised by a small number of highly qualified personnel on site and a strict supervision of the operations. The erection of a bridge's steelwork is a particularly critical phase as far as the overall stability of the structure and the safety of personnel are concerned. One cannot overemphasise the need for a *detailed study of the impacts of the erection method* based on the conceptual and structural design of the structure.

Due to the increasing capacity of lifting equipment, the current trend is to avoid the need for falsework, which is generally expensive, by increasing the size of the steelwork pieces and making maximum use of those parts of the bridge already in place to support those that follow. A consequence of this is that, during erection, parts of the steel structure may be subject to stresses from its self-weight that are higher than those they will experience in service. This effect is more pronounced if the *structural form of the load carrying structure during erection is different* from that in the final state. Such is particularly the case for bridges that are launched or erected by the cantilever method.

When a bridge is in service, the probability of the potential actions occurring simultaneously and in an extreme way is small. This is not the case during erection, when the various actions do tend to occur together, and their numerical value assumed in design is more easily exceeded. A detailed study of the erection load combinations, as well as diligent control of each erection phase, are required because the probability of an unforeseen or accidental action is much higher than when the bridge is in service (concentration of lifting equipment, manoeuvring of large structural pieces, etc.). All these reasons mean that, as much as possible, the potential *impacts of the erection method* on the load carrying structure should be considered in the earliest design studies. Only by doing this can the conceptual design of the bridge be sufficiently influenced.

Finally, it is likely that elements needed to *guarantee the stability* of certain parts of the structure in its final state will not yet be in position during the erection process. This situation must be addressed either by planning temporary support elements, or by ensuring through other means (design calculations, checks, inspection) that structural safety in terms of this aspect of stability is guaranteed throughout the erection. An example is the concrete slab of a composite bridge, which provides lateral support to the main beams' compression flanges (to prevent lateral torsional buckling), but which is only in place and effective after erection of the steelwork and construction of the slab. It may also be the case for plan bracing, which only becomes effective at the end of the steelwork erection programme. It is also necessary to check elements of the steel structure for their resistance to local buckling when subjected to concentrated forces (fixing points during erection, introduction of forces into the web during launching). The *overall stability* of the load carrying structure during its placement (against overturning, uplift, etc.) must be guaranteed. There may be a lack of stability during erection due to, for example, the absence of some of the self-weight that has a stabilising effect. It is also important to check that the various pieces of steelwork can adequately resist the various handling operations to which they will be subjected (lifting, transportation, assembly).

7.5.2 Erection by Crane from the Ground

Erection by crane (Fig. 7.3) is achieved by lifting the various steelwork elements – lengths of main beam, cross bracing, plan bracing – from the ground using a crane. This is an advantageous means of erecting the steel structure because it requires less equipment on site and a reduced amount of labour. However, it does require good accessibility around and on the site, and is only suitable for bridges relatively close to the ground (up to approximately 15 m).

The erection may take place either with or without intermediate (temporary) supports. If such supports (props, falsework) are used, they may be either continuous or discrete. Continuous falsework requires a significant investment in material; this is why such an approach is rarely used today. It should only be considered in specific cases where it is impossible to use heavy lifting equipment to place large pieces of the structure. This is the case when there is insufficient access for such equipment, and/or the space available for constructing steel sub-assemblies on the ground is limited.

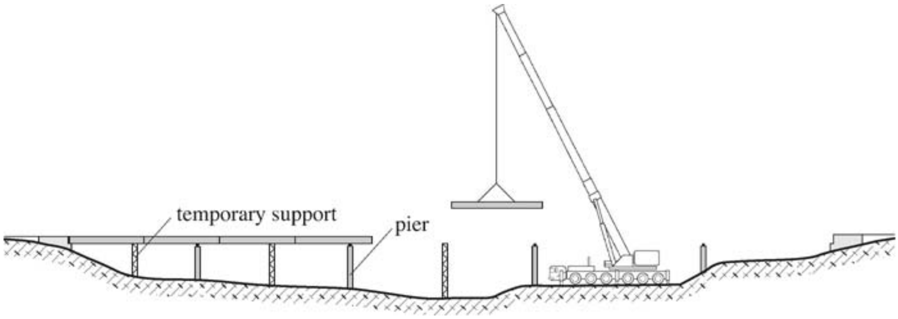


Fig. 7.3 Erection of the steel structure by crane, with additional temporary pier supports.

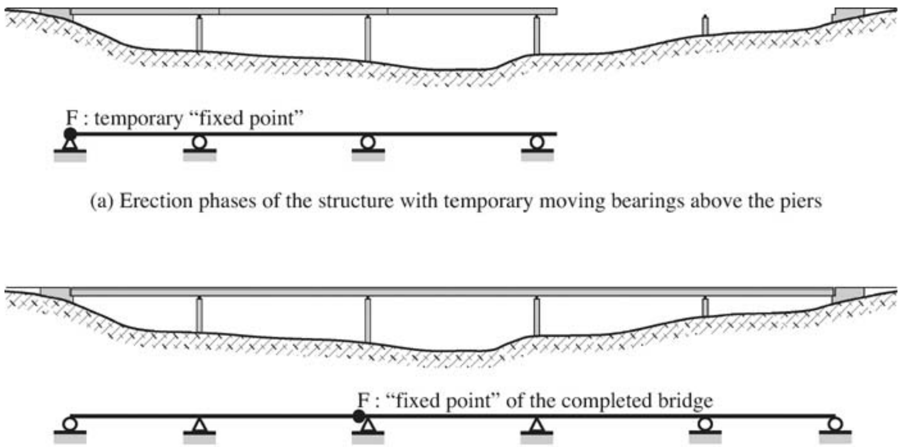
The evolution of heavy lifting equipment has meant that when temporary supports are needed, they tend to be a small number of discrete props rather than continuous falsework (Fig. 7.3). The bending moments that can be resisted at the bases of the telescopic arms of mobile cranes currently found in Switzerland can be as high as 15 000 kNm, which means that, for example, a weight of 100 tonnes can be carried at a distance of 15 m. If the spans are sufficiently short, and the size of the individual steelwork pieces allows it, the need for temporary supports can be avoided.

Influence of Erection on the Design Solution

The length of beam elements that can be erected from the ground is generally limited (maximum 30 to 40 m). Consequently, the stresses that these elements are subjected to during erection are not critical, providing lateral torsional buckling is avoided. However, the designer must ensure that the elements can resist the local forces introduced during handling and transportation (introduction of concentrated forces, lifting points, etc.). It is also necessary to develop an erection programme that ensures the partly built structure remains stable at all times, for example under wind loading or impact loading from a crane.

For a beam bridge that is supported on several piers, and with a longitudinal structural form that makes it a bridge on flexible piers (§ 5.3.4), erection of the steel structure by crane can result in significant stressing of the piers (Fig. 7.4). For such a bridge the steel beams are normally erected continuously working away from an abutment. To hold them in place longitudinally, they are temporarily fixed to this abutment during the erection process. A variation in temperature ΔT will lengthen (or shorten) the steel beams, and in so doing drag any piers that support a fixed bearing with them. Such movements at the pier heads will result in bending moments at the pier bases that exceed those they will experience when the bridge is in service. This issue is exacerbated because during erection, the steel structure will be directly exposed to the sun and temperature variations may reach 50 °C (steel temperatures exceeding +60 °C have been measured). Also, bending at the pier bases may be particularly problematic because the axial forces that accompany the moments are small during erection, so the tensile stresses due to the bending may be sufficient to crack concrete piers.

To avoid such moments and forces, it may be wise to leave the bearings between the beams and piers temporarily free to move during erection (Fig. 7.4(a)). In the final state the beams are fixed to the central piers, while the abutments, and any short piers near to the abutments, support moving bearings (Fig. 7.4(b)). To reduce the bending moments in the piers during erection, an alternative solution is to adopt temporary cable stays. This partially prevents changes in beam length, although as a consequence of this, normal forces arise in the steel structure.



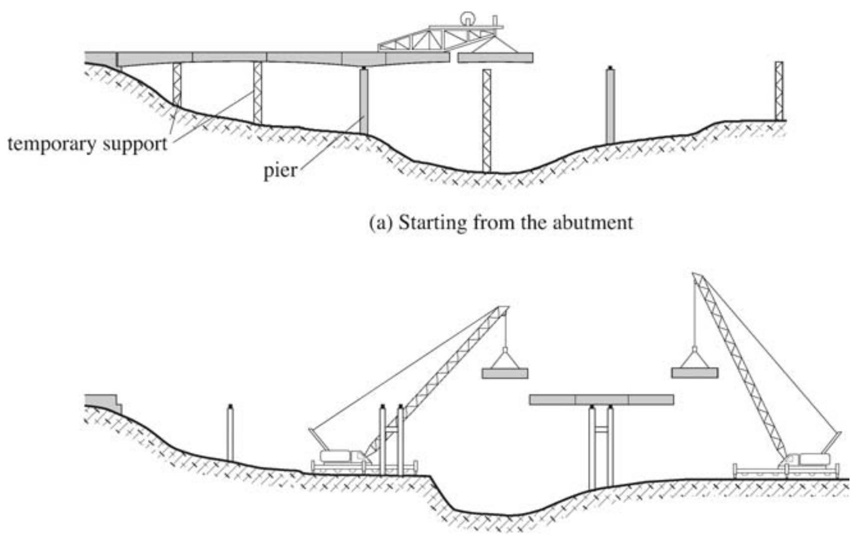
(a) Erection phases of the structure with temporary moving bearings above the piers

(b) Final state of the bridge on flexible piers

Fig. 7.4 Fixing of bearings adapted to suit the bridge construction sequence.

7.5.3 Cantilever Erection

Cantilever erection means forming the main beams by constructing cantilevers that extend away from the piers by joining together successive pieces. Joining the two opposing cantilevers at mid-span achieves the continuous beam. This method of erection is particularly suited to long span bridges (> 100 m), and those far above ground or water level. It is often used above navigable waterways because the bridge elements



(a) Starting from the abutment

(b) Symmetrically, from either side of a pier

Fig. 7.5 Cantilever erection.

can be brought in by barge and then lifted into position. The method is well adapted to cope with all types of bridge alignment, and bridges formed from beams with significant variations in depth.

Cantilever erection can also be considered for construction in a single direction starting from an abutment. For example, the first span of a bridge, lifted into place from the ground using a crane and a temporary support, can serve as counterweight to the next span cantilevering out (Fig. 7.5(a)). If a span is too big, then temporary supports can be envisaged. Symmetric cantilever erection away from piers (Fig. 7.5(b)) requires the steel structure to be rigidly connected to the piers. The pieces of cantilever are lifted into place, either by a crane on the ground or by lifting equipment located on the structure already built.

Influence of Erection on the Design Solution

The biggest difficulties associated with the cantilever erection method concern maintaining the bridge shape and direction (horizontal as well as vertical). To be able to join together two opposing pieces of bridge by cantilevering from adjacent piers, significant and accurate precambering must be planned to compensate for the deflections of the cantilevers under their own self-weight (Fig. 7.6). Additional precambering must be considered to compensate for subsequent deflections due to the weight of concrete and any other permanent loads. Precambering of the different pieces to be erected in this way must also take into account any curvature in plan and possibly torsion of the structure.

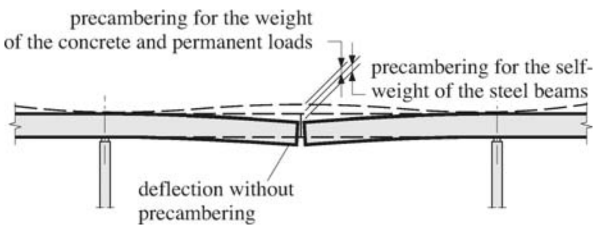


Fig. 7.6 Deflection of cantilevers due to their self-weight, and precambering to address this problem.

During cantilever erection the steel structure may be highly stressed in bending, due to its self-weight. It may also be subjected to additional loads, such as the weight of lifting equipment, or buggies used to move pieces into location from existing parts of the structure. It is important for the designer to consider the longitudinal distribution of self-weight of the steel and actions associated with erection, not only to determine their impact in terms of stresses but also deflections, and to check overall stability against overturning. If the stresses in the supporting sections are excessive, it is possible to use temporary supports below, or cable stays above, to support the steel structure (Fig. 7.7). Finally, the webs of box girders or main beams must be able to resist the local concentrated forces from the mobile lifting equipment used to transport the pieces during placement.

For the check of structural safety in the horizontal plane, it is necessary to consider wind acing on the cantilevers. Temporary plan bracing is typically required to strengthen the beam sections against horizontal bending, or torsion in the case of open U shaped box girders. Therefore, closed box section solutions are better suited to this method of erection.

The types of bridges constructed using cantilever erection are often those that are high above the ground, therefore with slender piers. During erection the structural form of each pier is that of a cantilever built in at its base and free at its head. The effective buckling length of the pier is therefore twice its system

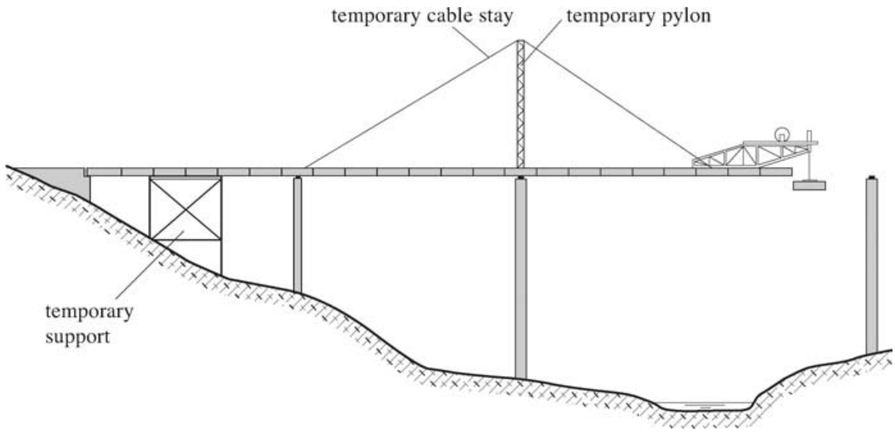


Fig. 7.7 Cantilever erection with temporary cable stays.

length, necessitating a careful study of its stability during erection. When cantilever erection proceeds symmetrically from a pier (Fig. 7.5(b)), the piers may also be subjected to torsional moments about their vertical axes, due to lateral wind that only blows on one cantilever. Additionally, the piers are subject to bending due to transverse wind loads, and bending due to any asymmetric loading of the two cantilevers (lifting equipment, other construction equipment, different phases of erection). The states of stress in the piers during cantilever erection are complex, and require detailed and careful study of their resistance and their stability.

When calculating the forces acting on the cantilevers and piers during erection, the designer can consider a characteristic value for the wind force that is lower than that proscribed for the bridge in its final

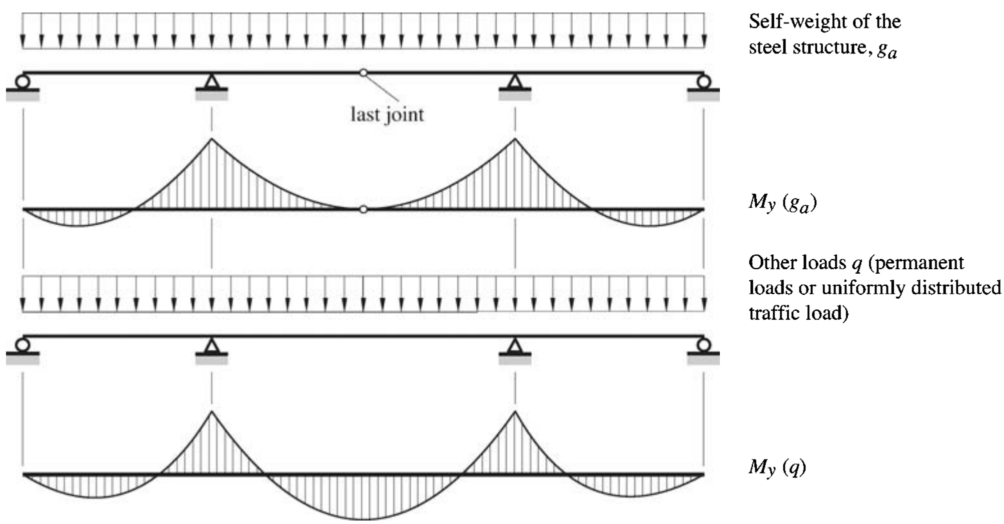


Fig. 7.8 Structural forms at different stages during cantilever erection and corresponding bending moments.

state, because of the limited duration of the various erection phases. In the absence of any guidance from codes and standards, it is essential to discuss a lower wind force during erection with the client's representative. Paragraph 10.4.1 gives some guidance on this subject.

Prior to joining together two opposing cantilevers, they are subjected to the self-weight of the steel structure (Fig. 7.8). Therefore, the bending moment diagram $M_y(g_a)$ is not that of a continuous beam. On the other hand, the weight of the concrete, other permanent actions, and variable actions will be applied to the continuous beam. In order to check the structural safety of the bridge, it is therefore necessary to superimpose the stresses calculated for each action, and to consider the corresponding structural form.

7.5.4 Erection by Launching

This method of construction of the steel structure comprises assembling elements of the structure in an area that is in line with the bridge axis, and located at one or both ends. As the elements are joined together piece by piece, the structure is pulled or pushed, in stages, into its final position (Fig. 7.9). Currently this method is used to launch structures with spans up to 150 m. Greater spans can be accommodated, but the resulting cantilevers require cable stays for support. Launching of the steelwork is adopted when lifting equipment cannot access the whole length of the bridge, or when the height of the bridge above the ground becomes too great for lifting by cranes on the ground.

Launching in this way has the advantage of allowing all the steelwork elements to be assembled on the ground in the assembly area, which means that work can be undertaken in more suitable conditions than may be the case with alternative erection methods. This is particularly important as regards welding operations. The adoption of erection by launching is limited, however, by the following requirements:

- sufficient space is available behind an abutment and in line with the bridge axis for steelwork assembly,
- the bridge must be either straight or curved in plan with a constant radius if it is to be launched from a single abutment,
- the bridge may be straight and curved if launched from both abutments, but the transition zone between straight and curved lengths must not be too abrupt,
- preferably, the main beams should be of constant depth, because the lower surface of the steel structure should be planar (the rolling plane). However, many structures of variable depth have been launched in this way, using temporary devices to compensate for depth variations during launching,
- for long span bridges it is preferable to have a closed cross section (box or U shaped section closed by temporary plan bracing) to ensure sufficient bending and torsional stiffness in the horizontal plane.

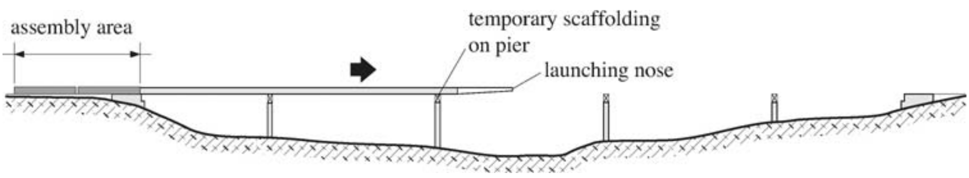


Fig. 7.9 Erection by launching of the steel structure.

Cantilever Deflections

In order to reduce the self-weight of the cantilever during launching, a lightweight structure, often comprising a truss to form a temporary *launching nose*, is placed at the leading edge of the steelwork. To compensate for the cantilever deflection as it approaches a pier, the launching nose is equipped either with an ad-hoc system for lifting, or takes the form of a beak (Fig. 7.10(a)). However, profiling the launching nose in this way is only an appropriate solution when the bridge is launched from the roadway level (see below), and for small spans. When these limitations are not satisfied, the deflections become too big, and it is necessary to implement a system for lifting the cantilever, such as using cables located above each pier (Fig. 7.10(b)). Adopting temporary cable stays (Fig. 7.10(c)) can substantially reduce both the cantilever deflection and the negative moment above the pier.

The steel structure can be pushed over the leading abutment with the lower flanges of the main beams positioned more-or-less at the level of the roadway of the finished bridge (Fig. 7.11(a)), or directly at the level of the definitive pier bearings (Fig. 7.11(b)). For the first option it is necessary to place temporary support, corresponding to the beam depth, on top of each pier to allow the launched structure to slide into

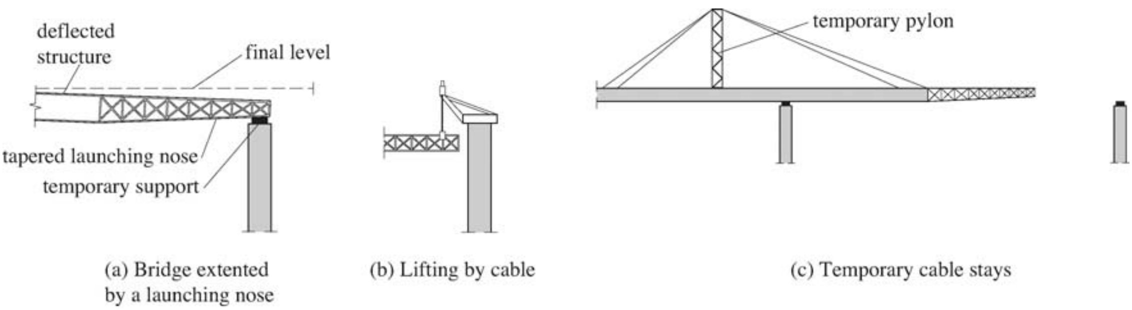


Fig. 7.10 Compensation for the deflection of a launched cantilever as it arrives at a pier.

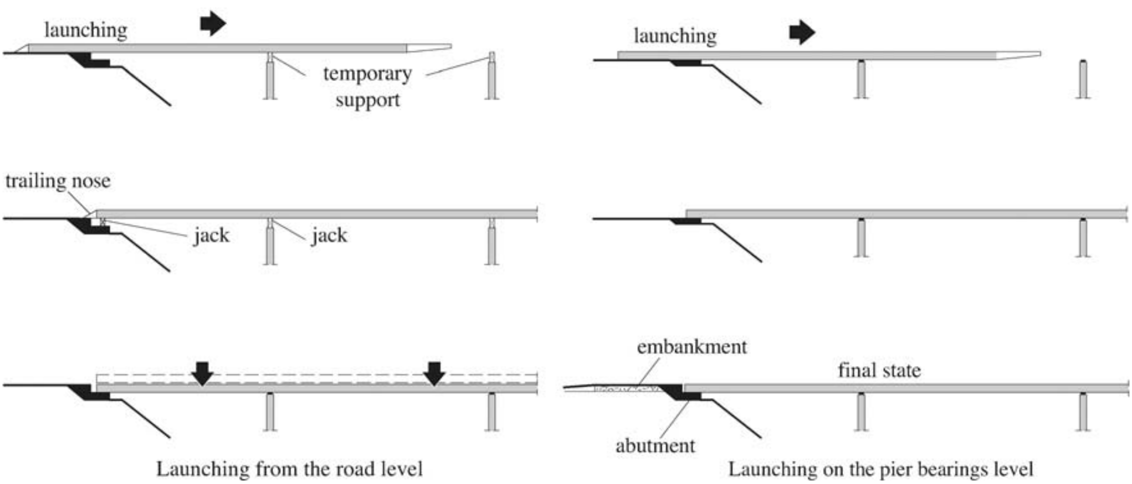


Fig. 7.11 Positions of the beams during launching.

place. Once the complete steel structure has been launched into position, it is then necessary to lower the bridge into its final position (which is often a delicate operation). It is wise to adopt a “nose” section on the trailing edge of the bridge to aid placement. The jacks used for lowering the bridge are then placed below the beams, and the trailing nose is demounted.

When the second option described above is adopted, the bridge is effectively pushed out already in its final level. Once it has been completely launched, all that remains to do is to slightly lift the beams, so that the temporary sliding bearings can be replaced by the definitive bearings. However, this option means that concreting of the abutment walls, and backfilling behind the abutment, can only take place after launching the steelwork.

To keep the weight down, typically the steel structure alone is launched. It is nevertheless possible to launch the structure with the concrete slab already in place on part of the bridge. Such a solution is attractive if part of the bridge will not subsequently be easily accessible for concreting; the slab can be more readily formed in this way and the completed composite structure launched. Even when this solution is adopted, normally the cantilever part does not include the slab during launching.

Equipment for Moving and Sliding

Even though launching of the steel structure is an erection method that is well proven, it still requires careful execution. It necessitates equipment to move the steelwork over the pier heads, and an assembly area that often must allow for the steel to be manoeuvred into its final position on the supports after launching. There must also be adequate provision to push, or pull, the entire structure. This is normally achieved by:

- pulling by cables, with 6 to 8 strands,
- hydraulic jacks with a stroke of 1 to 3 m.

Whether pushing or pulling, it is essential to plan a way to hold or block the steelwork so that its movement can be controlled at all times. This may also mean being able to move the structure backwards.

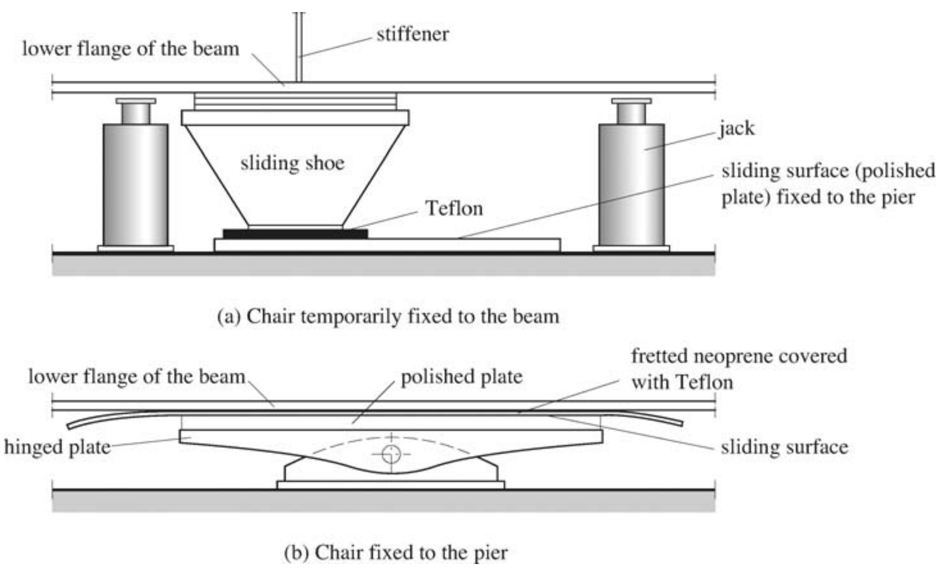


Fig. 7.12 Launching chairs.

Launching chairs (Fig. 7.12) distribute the support reactions into the beams and facilitate moving of the structure over the piers.

Traditionally formed from a series of rollers with a guidance mechanism, launching chairs have evolved into devices with sliding blocks that allow higher loads to be supported than is possible with rollers. The term *launching chairs* is used to describe a range of solutions comprising a shoe, or block, which slides over a polished surface with an interstitial layer covered by Teflon. Two alternatives are normally used:

- Figure 7.12(a) shows a solution where the chair is fixed to the beam and therefore moves with it. An advantage of this option is that it allows the block to be temporarily fixed under a beam stiffener, meaning that the support reaction is transferred into a sufficiently strong part of the beam. Using pickup jacks the chair is then moved from stiffener to stiffener to take the support reactions. This operation requires the launching to be interrupted, and so adds time to the operation.
- Figure 7.12(b) shows a solution where the chair is fixed to a pier, including a hinge (as is normally used). Sliding is facilitated by layers of fretted neoprene, with the sliding surface covered in Teflon. In order to distribute the support reaction over a certain length, the chair can be designed to be longer than is typical for the solution described above. The presence of a hinge allows the chair to follow the rotation of the beam, which varies depending on the phase of launching. This solution allows more rapid launching than the alternative described above, but does require the main beam webs to have sufficient resistance between stiffeners to accept the support reactions (see below). This type of launching chair is by far the most commonly used today.

At the assembly area, and above each pier, there is a transverse guide normally comprising vertical axis rollers that guides the edges of the bottom flanges of the main beams. These guides not only guarantee that the steel structure moves out in the correct direction, but also guarantee that the concentrated support reactions transferred by the chairs are correctly centred and introduced into the beam webs. This guidance ensures both that the beam flanges are not deformed out of plane, and that there are no unforeseen moments or forces transferred to the webs.

The speed of launching may attain 10 m per hour, although it will vary between sites and between one method of launching and another for the same bridge. Clearly, the way of introducing the movement affects the speed, but other factors can be equally as influential, such as the type of launching chair, and the lateral wind and sunshine falling on the structure. Launching must be stopped if the bridge behaviour starts to differ from that which has been predicted (deflections, support reactions). Values to allow checking of behaviour must be monitored continuously during each phase of the launch. For a horizontal bridge the force necessary to achieve movement of the steel structure is of the order of 3% to 10% of the self-weight. The friction characteristics of Teflon supports are considered in detail in Section 10.7.

As far as is possible, and depending on the bridge orientation, launching a steel structure in excessively sunny conditions should be avoided. Sunshine acting laterally effectively produces a horizontal temperature gradient in the bridge cross section that can lead to a horizontal displacement of the cantilever tip such that tying-up with the next pier is no longer possible. It may also result in internal forces and horizontal support reactions that are incompatible with the equipment used for sliding and transverse guiding.

In principle, launching should only take place when lateral wind is not significant. When the wind speed exceeds an agreed value, then the start of launching should be delayed, or a launch in progress should be stopped, or in extreme cases, the steel structure should be brought back to reduce the cantilever length. Lateral cable staying of the beam cantilever may be necessary, either at the end of the launching or, when launching has to be stopped, to limit the stresses and deflections under wind load.

Safety Against Overturning

Launching requires more calculation effort than any of the other options for erecting bridge steelwork. It is necessary to consider a substantial number of erection phases in order to determine, with sufficient precision, the internal moment and force envelopes, and thereby stresses, for all the construction phases.

It is clearly important to constantly check that the *safety against overturning* of the launched structure is guaranteed, taking account of the self-weight of the steel structure and any counterweight acting to hold it down. This can be illustrated by considering the example of a three span bridge (Fig. 7.13). Just before the launching nose reaches pier 2, the safety against overturning (toppling) of the structure may not be guaranteed. If necessary, one can place a counter weight at the trailing end of the structure (abutment 1) to assure stability. The principles of checking safety against overturning are described in Paragraph 9.6.2 and Chapter 15.

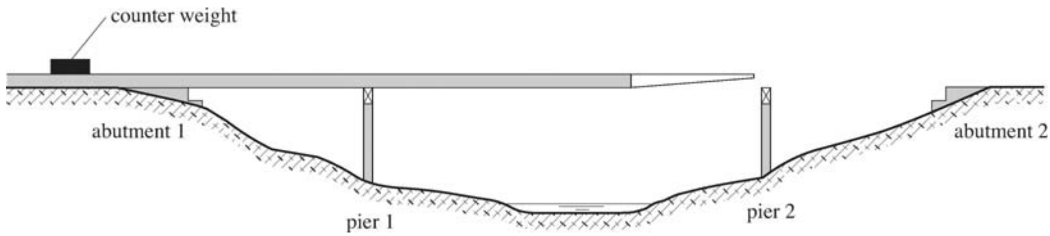


Fig. 7.13 Safety against overturning.

Stressing of the Main Beams

Launching puts in motion the entire structure and leads (in each section of the main beams) to states of stress that are totally different from those the beams will experience when the bridge is in its final state. A detailed calculation of the internal moments and forces is therefore needed for each phase of the launch. Of particular interest are the bending moments and shear forces, and the introduction of the support reactions into the main beams. Negative bending moments due to the self weight of the launched structure can be very significant. For the example of a three span bridge, Figure 7.14 shows the envelope of negative and positive bending moments for a steel structure launched without a launching nose. It also shows part of the envelope for negative moments assuming a 20 m long launching nose, with half the weight of the equivalent bridge steelwork. The diagram for the final state bending moments is also shown in Figure 7.14.

The cross sections at mid-span, once the bridge is completed, are stressed by large negative moments during launching. This is particularly the case for the first two spans to be launched. It is also clear that the use of a launching nose reduces some of the negative moments considerably, for example at cross section A, which is very lightly stressed when the bridge is in its final position. Although the use of a launching nose actually increases the moments in the cross section above pier 2, this is not a problem because this cross section must be designed to resist considerable bending moments due to the presence of other actions.

This example emphasises that the choice of the launching nose, in terms of weight and length, has a significant influence on the distribution and magnitude of the internal moments and forces that occur during launching. The length of the nose should be chosen in such a way that a steel structure designed to resist the moments and forces in the final state can also resist, for the majority of cross section positions, those moments and forces that occur during launching. A launching nose having a length between one quarter and one third of the longest span is typically used today.

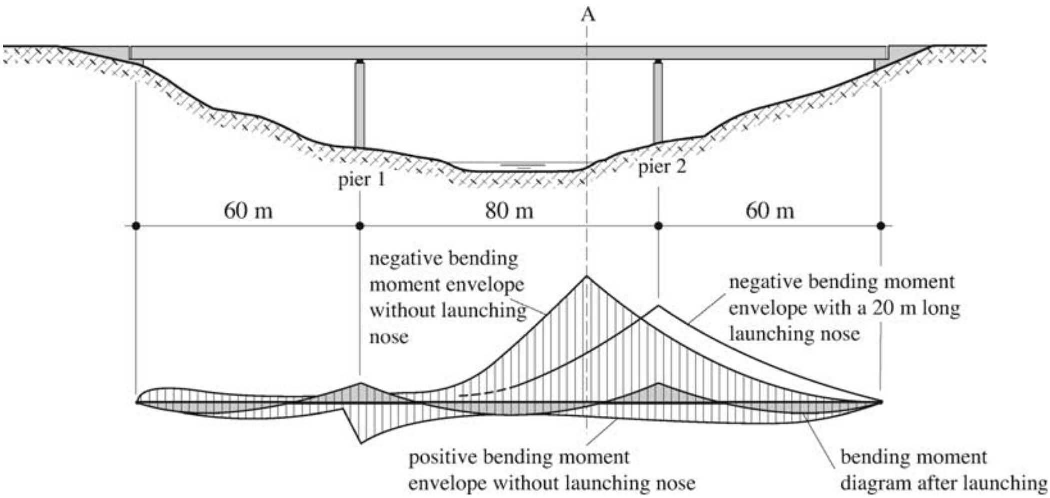


Fig. 7.14 Examples of bending moment envelopes during launching.

During launching the different steel beam cross sections will move over the piers; therefore, they must all be capable of resisting the concentrated support reactions. While in the final state the support reactions are transferred to the beam webs via stiffeners, this is not the case during launching (Fig. 7.15). The resistance of a web to local buckling under concentrated load, or the so-called *patch load* resistance, may govern the design, when no vertical stiffener is present. One of the functions of launching chairs is clearly to distribute longitudinally such substantial concentrated loads. Measuring the support reactions during launching enables the site team to check whether the real values are in agreement with the predicted values. If the measured values are different, then changes must be made to ensure that the reactions are sufficiently distributed to avoid web buckling. This could be achieved by modifying the level of the launching chairs using jacks. If necessary, the resistance of the webs to concentrated loads must be improved locally, either by increasing the web thickness or by welding a longitudinal stiffener to the lower part of the web.

Deflections of the steel structure during launching should be measured and controlled to avoid difficulties when the launching nose passes over, or is lifted onto, the piers. If necessary, the height of the

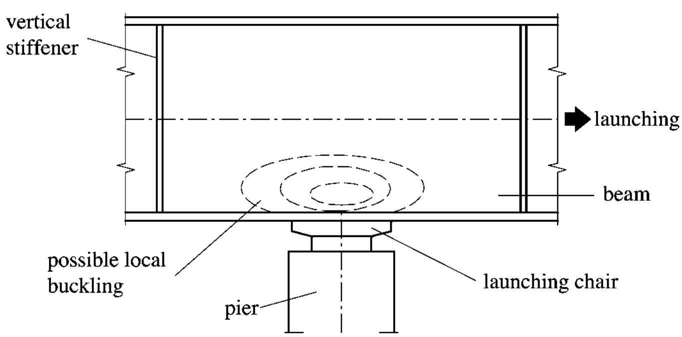


Fig. 7.15 Support reaction during launching, when introduced into the beam between stiffeners.

launching chairs can be modified using jacks to allow passage of the nose section. The lateral guides located in the assembly area and on top of each pier must be designed to resist the horizontal forces resulting from any lateral wind.

When a curved bridge is being launched, it is necessary to take into account the effects of the curvature on both the internal moments and forces and on the deformations (different support reactions, rotation of cross sections). These curvature effects, which can be both substantial and significant, are discussed in Section 11.7.

Stressing of the Piers

Like the bridge deck, the *substructure* of a bridge is also subjected to specific moments and forces during launching of the steelwork. These must be studied in detail, particularly given that the piers may be slender. The forces acting on the piers during launching are the longitudinal friction of the superstructure on the launching chairs, and the horizontal component of the forces that can be exerted by the tapered launching nose coming into contact with the piers. These forces must be considered, taking into account that, during erection, the piers are independent of the superstructure. This means that their structural form is different, and specifically that the buckling length is different from that in the final state (Fig. 7.16).

The friction forces that develop during launching must be resisted by bending of the piers. The resulting bending moments may govern the pier design because the accompanying normal forces in the piers, due to the self-weight of the steel structure alone, are minimal. To reduce the bending moments in the piers during launching, it is possible to provide cable stays fixed to the pier heads. If the piers are very tall, such cable staying is problematic. However, it is possible to eliminate friction forces by adopting a means of moving the bridge at the head of each pier. Such means must be perfectly synchronised between all piers, which requires quite specific methods of control and coordination.

When the deflections of the cantilever are compensated for by the form of the launching nose, an additional horizontal force H, due to the taper of the nose, acts on the piers. When the leading edge of the nose, with a taper α, passes over a pier, it creates a support reaction R (Fig. 7.17). Because this reaction is not vertical, it also pushes against the pier, resulting in supplementary bending of the latter. In this situation, depending on the slenderness and stiffness of the piers, it may also be necessary to consider cable stays.

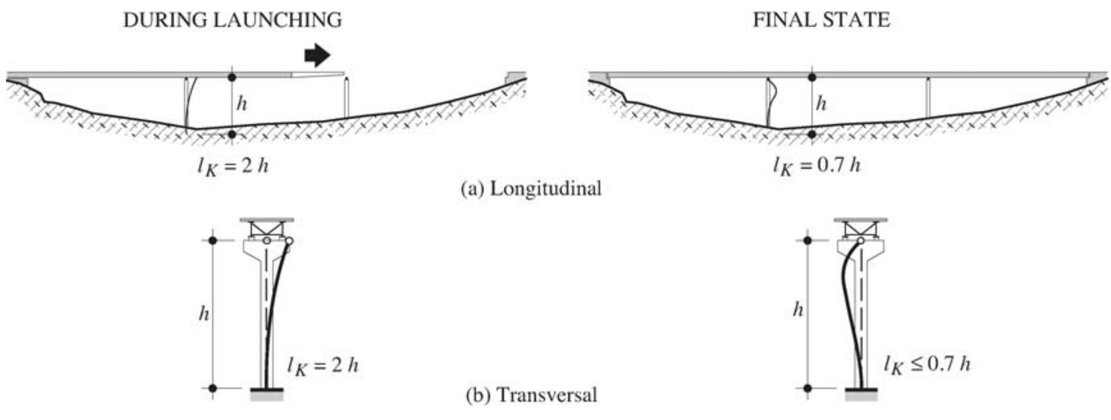


Fig. 7.16 Effective buckling lengths of the piers (during launching, and in the final state).

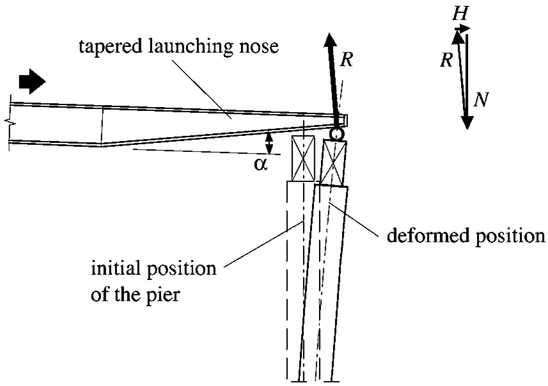


Fig. 7.17 Passage of the tapered launching nose over a pier.

7.5.5 Placement of the Complete Bridge or of Large Bridge Elements

It is possible to manoeuvre the complete steel structure into place or, for large bridges, to erect sub structures weighing hundreds of tonnes. The most commonly used methods are moving the bridge into place with the aid of barges, by transverse sliding, or placement by rotation.

When the bridge crosses a navigable stretch of water, the steel structure can be placed on barges for placement. For example, if the bridge is constructed on a bank that is in line with the axis of its final placement, one end of the bridge can be placed on a barge that serves as a float (Fig. 7.18). The bridge is then moved into position by pushing the barge towards the abutment located on the opposite bank.

If the bridge is assembled in an area further away from its final position, then the complete structure can be placed on barges to transport it from the assembly area to its final location. A box girder bridge may be floated into position by towing, once it has been made watertight. The box is lifted using cables linked to jacks fixed to the piers and abutments, then placed in the correct position.

It is also possible to slide the steel structure transversally (Fig 7.19) into position. This method is often used when a new bridge replaces an existing structure. The new bridge can be assembled with minimal disturbance to the traffic on the existing structure. The new bridge is erected alongside the existing structure on temporary piers and abutments. Traffic is then diverted onto the new structure, so that the old structure can be deconstructed. Total interruption to the traffic is brief; generally the transfer takes place over one night, which is sufficient to allow the new structure to be slid onto the refurbished piers and abutments. Then the new structure can be opened to traffic and the temporary piers and abutments removed.

Another method of bridge placement over a waterway is that of rotation about one of its extremities. If there is sufficient space along the bank, then the entire bridge is constructed parallel to the river and rotated into

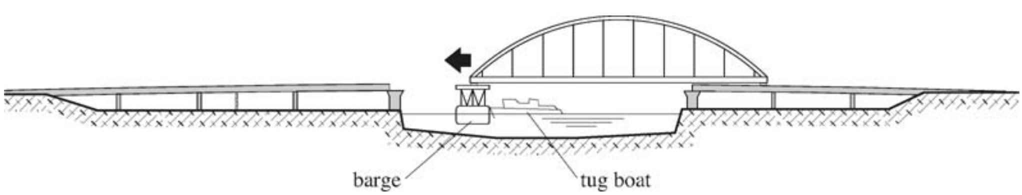


Fig. 7.18 Launching over a navigable river or canal, using a barge.

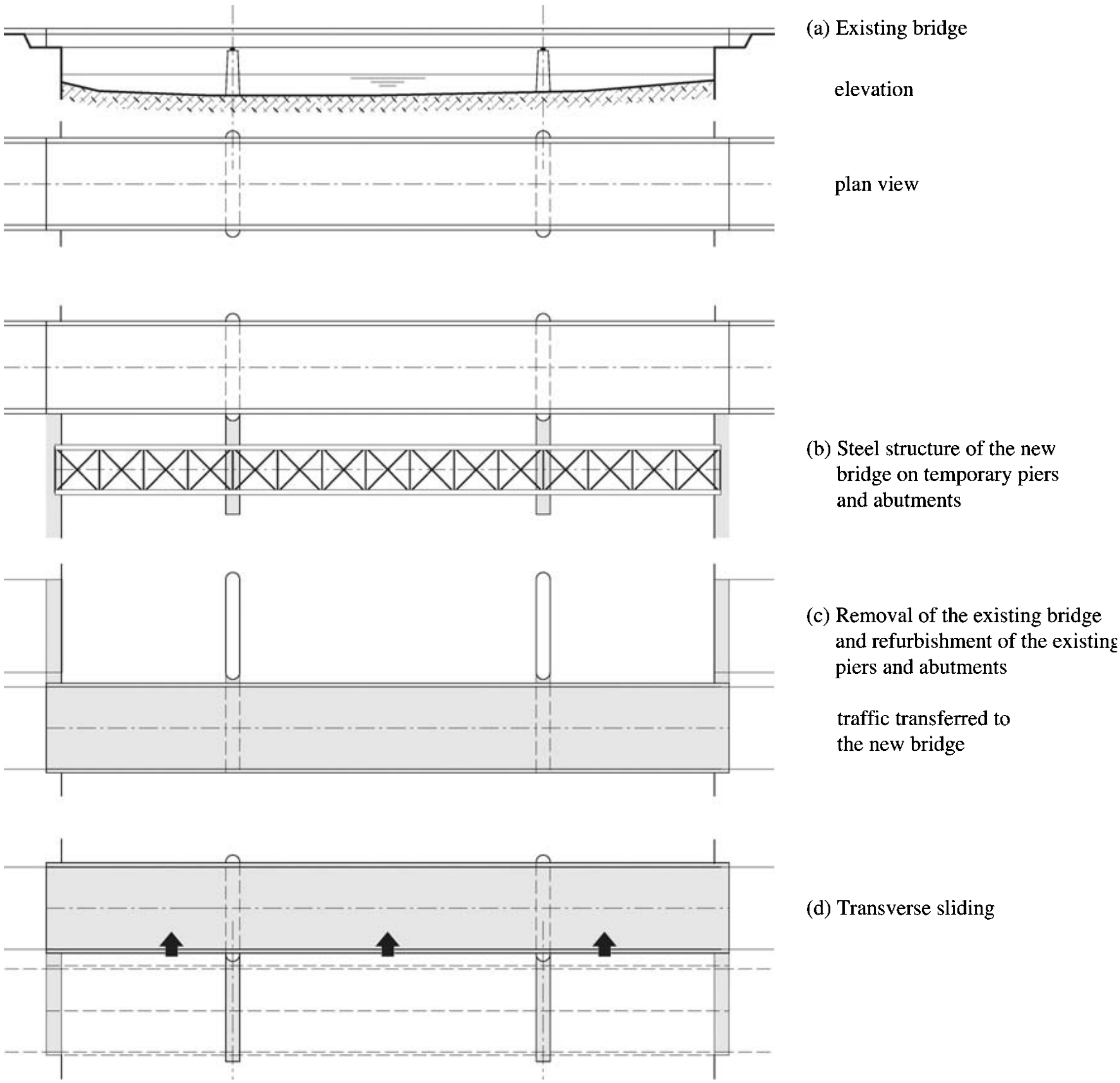


Fig. 7.19 Bridge replacement by transverse sliding.

place about a vertical axis through the abutment. This method is of considerable interest because the erection of the steel structure may take place entirely on the ground, giving easier access to the whole of the assembly area.

7.6 Tolerances

Fabrication Tolerances

The considerable size of the steel elements and the machines used in fabrication shops do not allow the theoretical dimensions of the elements to be perfectly respected. To ensure that the fabrication of the various steel elements will be satisfactory, codes and standards fix *fabrication tolerances* between the theoretical values and those deemed acceptable. These tolerances are chosen to provide the best possible compromise between the dimensional requirements for execution and what can practically be achieved during fabrication. They allow the geometry of the elements leaving the fabrication shop to be controlled, and they also influence the degree of precision needed during execution. Figure 7.20 shows some examples of fabrication tolerances for a plate girder. Fabrication tolerances for plate girders are defined in Switzerland in the standard SIA 263/1, while those for rolled sections are given in tables, for example SZS C5. Similar tolerances can be found in the Eurocodes.

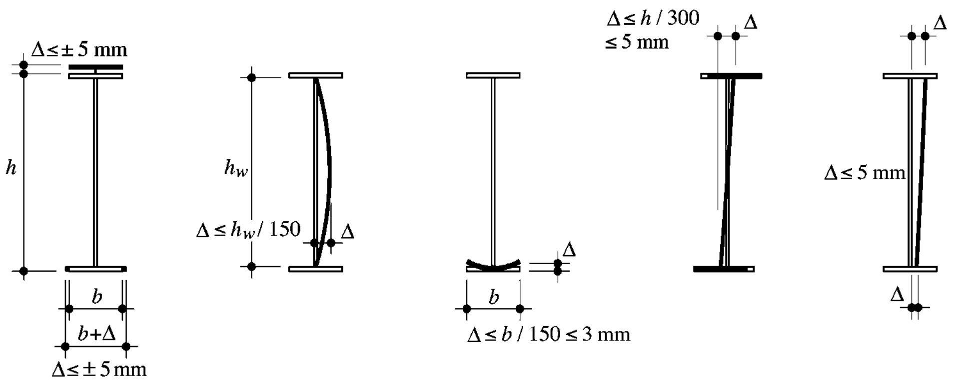


Fig. 7.20 Examples of fabrication tolerances for the cross section of a plate girder.

Erection and Construction Tolerances

The overall geometry of the bridge must be respected on site during the assembly and placement of the steelwork elements. *Erection tolerances* must be fixed before starting the works on site and controlled during erection. These tolerances are primarily associated with the alignment of pieces, the distance between the main beams, and the connections between the flanges of adjacent beam lengths. Limiting values for the deviations between the theoretical positions of the structural elements and their actual positions are also given in SIA 263/1.

For a road bridge, *construction tolerances* related to the geometry and contours of the roadway must be fixed by the relevant authorities. Their aim is to assure the comfort of the road user. These tolerances may govern, in particular, when an orthotropic deck is used, as the surfacing is of only limited thickness. When a concrete slab is used, the tolerances associated with the roadway may still have a direct impact on the fabrication of the steel elements, because they may require precambers that influence the geometry of the steelwork during its fabrication. Construction tolerances are also fixed to assure adequate waterproofing and surfacing, as well as placement of various bridge components [7.2].

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8 Slabs of Composite Bridges

Precast slab elements used for the bridge at Dättwil (CH).
Eng. Bureau d'ingénieurs DIC, Aigle.
Photo DIC, Aigle.

8.1 Introduction

A road bridge comprises either a concrete slab connected to the steel beams, or an orthotropic steel deck. The conceptual design of orthotropic decks, including some construction details, was considered in Section 6.7. Given that this book is more specifically concerned with steel-concrete composite bridges, this entire chapter is dedicated to the slabs of composite bridges in order to consider the main aspects of their conceptual design and configuration.

Sections 8.2 and 8.3 present general aspects of the configuration of reinforced and prestressed concrete slabs. They consider the principles and construction details that govern the design of these elements, without going into the details of ordinary reinforced concrete slab design (which can be found in TGC Volume 9).

Section 8.4 describes methods for the construction of concrete slabs, including concrete that is cast in-situ, a slab that is launched in stages and a slab composed of precast elements (Fig. 8.1). It highlights the advantages and disadvantages of each method, and describes their particular fields of application. It also touches on the impact of the slab construction method on the conceptual design of the steel structure.

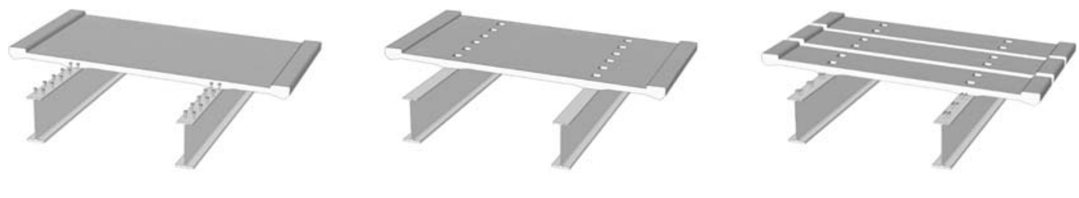


Fig. 8.1 Slabs of composite steel-concrete bridges.

Section 8.5 considers concrete slabs from the point of view of cracking, which may develop either during their construction or, with time, in service. Detailed consideration is given to slabs that are cast in-situ on the steel structure and, particularly, the question of cracking that can occur when the concrete is freshly cast. Finally, Section 8.6 considers the question of longitudinal prestressing of composite bridge slabs and presents various methods of achieving prestress, as well as how the prestressing forces evolve with time.

8.2 Slab Design

In terms of its general configuration and many of its construction details, the slab of a composite bridge is similar to that of a (reinforced or prestressed) concrete bridge. Slabs for these two types of bridge are distinguished essentially by the way they are connected to the beams, which not only support them but also locally influence their design. For all options the slabs have an identical function, namely to resist and transfer the traffic loads and other actions that act on them. For a composite bridge, because of the two different materials that constitute the deck, the various effects of the long term behaviour of concrete are discussed in Section 13.2.

8.2.1 Functions of the Slab

The primary function of a bridge slab is to offer a surface that supports the imposed loads (traffic) and other actions on the bridge. The slab, which is generally covered with a waterproof layer and surfacing, must also support various bridge components. These can include crash barriers, kerbs or other means of separating pedestrians from traffic, and in some cases lighting masts, signalling gantries or electricity lines on a railway bridge. The slab must therefore be designed for the following functions:

- to resist concentrated traffic actions, both vertical and horizontal, as well as those acting on barriers or guard rails,
- to transfer the effects of these actions to the longitudinal structural elements such as the main beams (I sections or box sections).

For a steel-concrete composite bridge, the slab also has the following functions:

- to contribute to the longitudinal bending resistance of the bridge,
- to act as horizontal plan bracing to transfer transverse forces to the piers and abutments,
- to provide stability against lateral torsional buckling by restraining the upper compression flange in span of plate girders or U shaped open box sections.

Because the slab will not yet exist during erection of the steel structure, the functions noted above will need to be performed by other structural elements during that phase of the execution (Sect. 5.6 and 5.7).

8.2.2 Typical Dimensions

The slab must resist the actions due to traffic and transfer them to the main beams, or other structural elements that support it. These actions, which may be either concentrated or distributed, act both vertically and horizontally and are defined in Section 10.3. They are transferred transversally to the longitudinal beams by local bending of the slab, which is generally assumed to be simply supported on the steel beams. Figure 8.2 shows the cross section of a twin girder composite bridge, with typical main dimensions of the slab.

The cross section shown in Figure 8.2 is typical for a Swiss freeway bridge, with a slab that is generally between 12 m and 13 m wide and is supported on two steel beams. In order to limit the thickness and thereby the weight of the slab, the spacing of the main beams is chosen to equalise the transverse bending moments (positive between the beams and negative over them). This implies that the beam spacing will be around 50% to 55% of the total width of the slab. When this is achieved the slab thickness will generally be variable, with at least 300 mm above the beams and a minimum of 250 mm between them. The slab thickness above each beam may differ in order to give its surface the necessary slope (cross fall), particularly in the case of curved bridges. Given the requirements associated with durability of the slab and concrete cover of the reinforcing bars, the minimum thickness for a bridge slab should not be less than 240 mm, even at the ends of cantilevers.

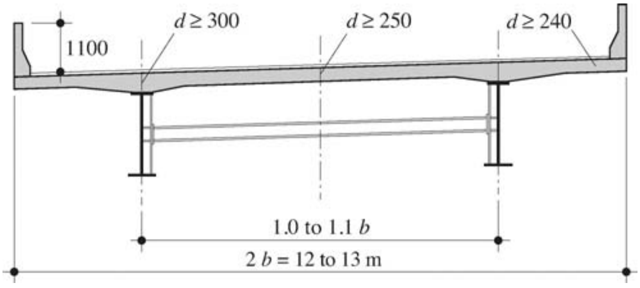


Fig. 8.2 Dimensions of the slab of a composite road bridge with two main beams.

For wider slabs it may be necessary to adopt additional longitudinal beams, to form a multi girder solution. It is also possible to provide additional support to the slab, for example from the (frame) cross bracing, when the cross girders are placed in the upper part of the cross section (§ 5.6.2). These cross girders may be designed to only support the slab between the beams, or may also support the cantilevers when the slab is wide (Fig. 8.3). The cross girders are normally connected structurally to the slab and therefore contribute to its transverse bending resistance. They are typically at 4 m centres, allowing the slab to have a constant thickness of around 240 mm.

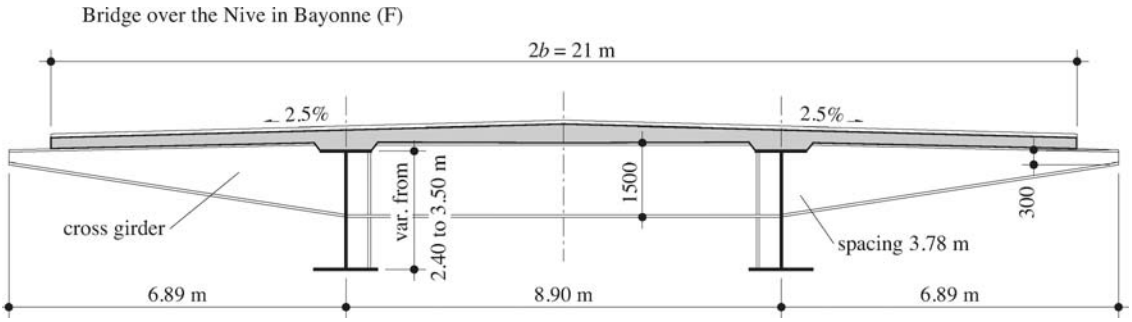


Fig. 8.3 Example of the cross section of a composite bridge with cross girders and cantilever girders [8.1].

The presence of cross girders in the upper part of the cross section can be a nuisance during concreting of the slab, in particular when mobile formwork is adopted. Rather than adopting a very wide slab to support all the traffic lanes, it is often preferred, notably in Switzerland for freeway bridges, to build two separate bridges. Such a solution also has advantages during maintenance and repair of the slab.

The width of the slab depends on the type of use of the bridge. Its thickness and transverse reinforcement depend on the internal moments and forces due to the self-weight of the concrete and surfacing, plus the concentrated and distributed traffic loads. In span, the longitudinal reinforcement is determined considering criteria for minimum reinforcement (§ 13.7.3), and is of the order of 0.75% to 1.0% of the cross sectional area of the slab. Above intermediate supports (piers), where the slab is in tension, the reinforcement must ensure that crack widths remain below a specified minimum value. However, in such regions the percentage reinforcement in the slab is often fixed at a default value of between 1% and 2% for composite bridges. At the intermediate supports the longitudinal reinforcement contributes to the bending resistance of the composite beams.

8.3 Construction Details

8.3.1 Waterproofing and Surfacing

In many countries within Europe, the concrete slab is covered with waterproofing and surfacing. The function of the *waterproofing* is to protect the concrete from aggressive agents such as water, de-icing salts and exhaust products from vehicles. The waterproofing is an essential component to ensure the durability of the slab. Examples of failed waterproofing – notably at joints at the slab edges, or where the waterproofing has been incorrectly placed – are numerous. Moreover, the consequences for the slab are often disastrous, with substantial remediation costs (partial or total slab replacement, traffic disruption). The surfacing protects the waterproof layer against the various actions due to traffic and provides a wearing surface on top of the slab.

The waterproofing must satisfy numerous requirements, notably:

- be watertight,
- accommodate cracking of the slab,
- resist aggressive agents,
- adapt to the concrete surface conditions,
- accommodate a range of temperatures without degradation,
- resist impacts and puncturing.

The waterproofing may be either floating, which means it is separated from the concrete slab, or completely adhered to the slab. When it is adhered to the slab, the waterproofing system will include a tack coat. When a floating solution is adopted, the system includes a waterproofing layer and a protective layer. Adhered waterproofing systems are formed from layers of either polymer-modified bitumen or liquid polymer. Floating systems are formed from mastic asphalt. The protective layer normally comprises poured asphalt, which must adhere in a durable manner over the whole surface of the waterproofing. A protective layer formed from a compacted bituminous mixture should only be considered for bridges on secondary circulation routes. Currently, adhered waterproofing systems are preferred because any failure can be more easily addressed locally; however, they do require great care during placement.

Details for the waterproofing at the slab edges and against expansion joints, and details needed for the run off of surface water, must be carefully considered and executed in order to assure the durability of the layer. Figure 8.4 shows an example of a waterproofing detail at a slab edge. The upstand may be additionally clamped in place using a steel clamping band. Figure 8.5 shows a detail of a connection between a waterproofing layer and an expansion joint, achieved using two layers of poured asphalt.

Depending on the specifics of a given bridge, the *surfacing* must satisfy various requirements, which concern, for example:

- resistance to deformation under traffic loading,
- surface roughness, to ensure that vehicle tyres will grip,
- drainage of rain water (porous asphalt),
- noise.

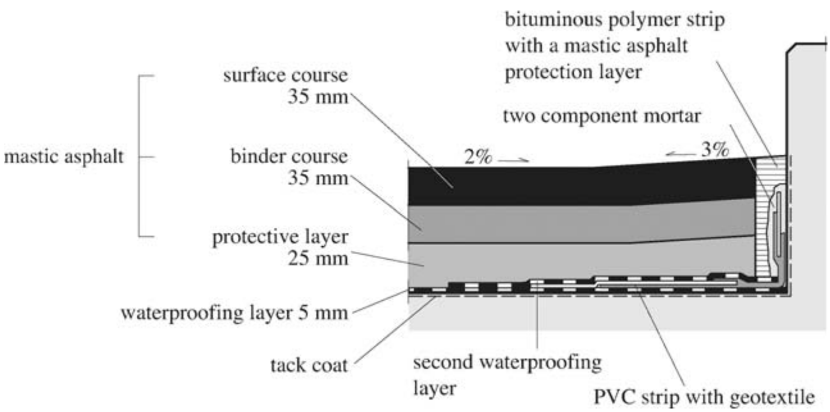


Fig. 8.4 Example of waterproofing with a slab edge treatment [8.2].

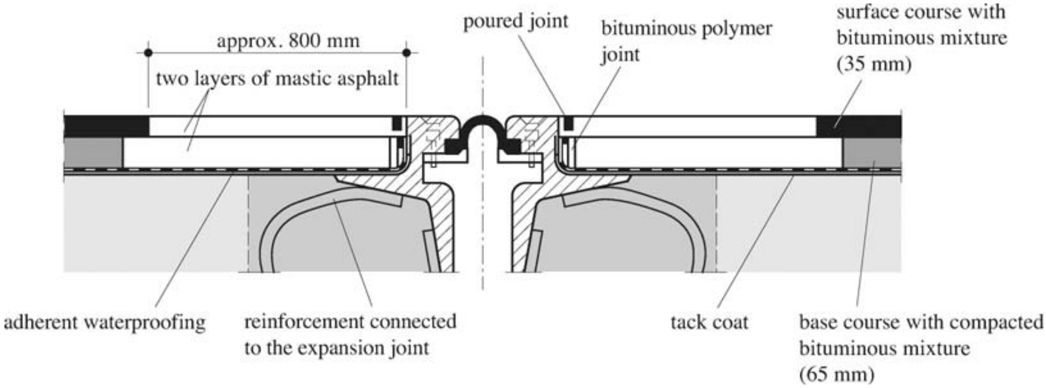


Fig. 8.5 Example of waterproofing with an expansion joint [8.2].

As far as the surfacing is concerned, one may distinguish between a bituminous mixture and a mastic asphalt, which is placed in two or three layers. The different layers of the surfacing are often called the base and binder courses, which build up the necessary thickness, and the wearing course. The total thickness of the combined surfacing and waterproofing system is normally around 100 mm. Figures 8.4 and 8.5 show two examples of the composition of the surfacing. Descriptions of waterproofing systems, surfacing, the components from which they are formed, appropriate tests for the materials, methods for laying them, and various requirements and checks are defined, for example, in the standard VSS 640 450 [8.3].

8.3.2 Edge Beams and Parapets

The edges of a bridge slab serve as support for any components aimed at containing the traffic, for example crash barriers or noise barriers. They must also be formed in a way that will prevent water runoff on to the underside of the slab, in order to protect the longitudinal beams. The edge beams should also facilitate appropriate detailing and execution of the slab waterproofing (§ 8.3.1). Because the edge beams are clearly visible to onlookers, they must not adversely affect the appearance of the bridge.

The choice and configuration of the edge beams, and the means of traffic containment, are primarily a function of the type of road, the objectives of protecting those things beneath the bridge, the length of the bridge, and any requirements for noise protection. Generally the client's representative specifies, as part of the project requirements, the type of edge detail and any containment requirements, according to the specifics of the bridge in question. Figure 8.6 shows two examples of slab edge details for freeway bridges (including principles of the reinforcement arrangement): an edge beam with crash barriers and a structural parapet used to assure safety.

Because of their location, the slab edges are permanently exposed to not only the effects of the weather but also chemical attack from de-icing salts. The concrete that is used to form them, therefore, must have good resistance to chemical attack, and the reinforcement should have sufficient cover to prevent corrosion.

Normally, the edges are concreted after the slab, and then rigidly connected to it. They must be able to resist any horizontal forces, such as those from vehicle impacts (§ 10.6.2), and transfer them to the slab itself. The edges are not considered as structural elements as far as the longitudinal behaviour of the bridge is concerned, because during refurbishment or replacement they would be unable to fulfil this function.

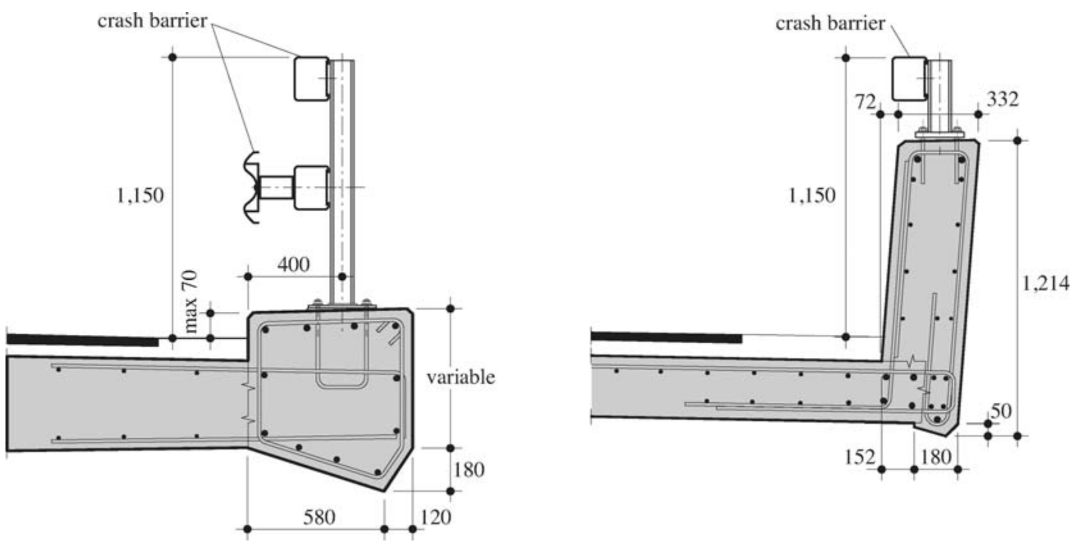


Fig. 8.6 Examples of edge beam and parapet details.

8.3.3 Slab to Steel Connection

To form a composite bridge, it is necessary for the concrete slab to be structurally connected to the steelwork. This is achieved using mechanical connection elements, known as ***connectors***, because the natural adhesion of the concrete to the steel surfaces is too weak and insufficiently robust. Given this mechanical connection the concrete slab adds to the bending resistance of the steel beams, with the two materials acting together as a steel-concrete composite section. The connectors must be capable of resisting both slip and uplift of the slab, relative to the steelwork. Different types of connector may be used:

- *Studs*: connectors, which comprise a shaft with a rounded head to prevent the slab from separating from the steel beam. Studs used in bridge construction are at least 150 mm long and generally of 22 mm diameter. Studs are flexible connectors, classified as ductile when subject to shear, which allows the longitudinal shear forces to be redistributed.
- *Perforated plates*: connectors, which are formed from a plate that has large diameter perforations through which reinforcing bars pass. Resistance is provided by a combination of friction of the concrete along the plate and interlock with the perforations and reinforcement. These connectors exhibit ductile behaviour.
- *Blocks*: rigid connectors, which are formed from a welded steel profile (Tee or angle), and which do not allow redistribution of longitudinal shear.
- *Anchors*: ductile connectors (often formed from reinforcing bars), which act in tension and permit redistribution of longitudinal shear.

The most common forms of steel-concrete connectors for modern composite bridges are studs and perforated plates. The other types of connector noted above were frequently used in the 1960s and 1970s for composite bridges. When precast slabs have been used, the connection has sometimes been achieved by *friction* between the concrete slab and steel members, using vertical prestressing to create this friction by clamping the slab to the steel beams with preloaded bolts.

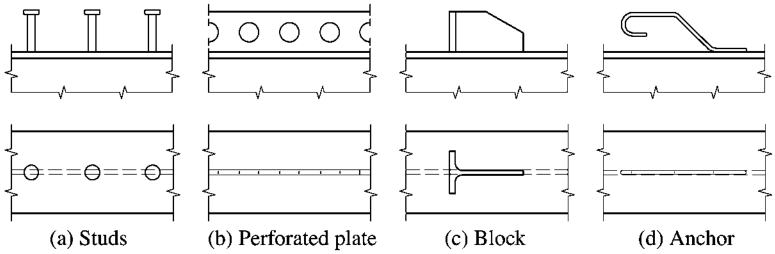


Fig. 8.7 Types of shear connector.

Today the most widely used form of connector is headed studs, due to their convenience and ease of placement, which is achieved by electric arc welding, using a special tool. They are generally welded in place in the fabrication shop; however, depending on the method of construction of the slab, the studs may be welded on site. This requires the provision of sufficient electrical power. In terms of their behaviour, because they are flexible when subjected to shear, the studs allow a good redistribution of longitudinal shear forces between them. This is a requirement if the resistance of the composite beams is to be determined using plastic design. Headed studs also have the advantage of providing the same resistance independent of the shear force direction.

In order to achieve correct location of the studs and sufficient concrete encasement to assure correct behaviour of the connection, the detailing requirements illustrated in Figure 8.8 must be respected. The spacing of the studs in the direction of the longitudinal shear must not be less than $5d_D$ (d_D is the stud diameter) or more than 800 mm. However, according to the EN 1994 (Eurocode 4), when the compression flange is restrained against lateral torsional buckling by the slab, the longitudinal spacing of the connectors must not exceed $22t_f\sqrt{235/f_y}$ (where t_f is the thickness of the flange and f_y the yield strength of the steel beam). In the direction transverse to the shear force, the spacing of the connectors should not be less than $2.5d_D$. Studs welded to a tension flange, and subjected to fatigue loading, should not have a diameter greater than $1.5t_f$.

The completion of the steel to concrete connection will be delayed, according to the method of construction of the concrete slab, for example if it is launched in stages by sliding, or if precast slabs are used, or for some types of prestressing of the slab. In such cases it is helpful to leave voids in the slab above the steel beams. These voids, typically around 300 mm × 300 mm, are at one metre spacing, and the steel to

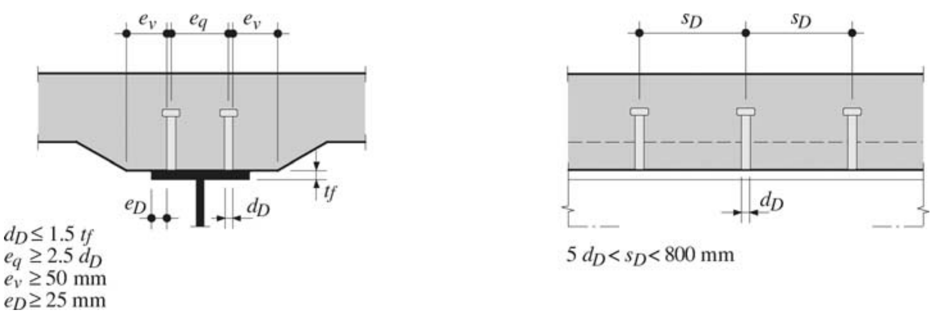


Fig. 8.8 Placement of shear studs.

concrete connection is achieved by studs that are grouped within the void areas and subsequently embedded in low shrink concrete. Normally, the number of studs in each group varies between 10 and 16, depending on the location along the beam. Figures 8.14, 8.15 and 8.16 illustrate the connection using grouped studs for the case of a slab that is launched in stages.

The behaviour of a connector under load cannot be determined by simple calculation. Both its static and fatigue resistance are derived from so-called *push-out* tests. These are standardised so that the behaviour of each type of connector can be determined experimentally. Design values of stud resistance are given in Paragraph 13.5.2.

8.4 Construction of the Concrete Slab

Once the steel structure is totally or, in some cases, partially erected, the concrete slab is constructed. The three main methods for achieving this are described in this section, namely:

- slab cast in-situ,
- slab launched in stages,
- slab composed of precast elements.

8.4.1 Slab Cast In-situ

Today, the most common option for constructing the slab is to cast the concrete in-situ. This traditional method is well adapted to some particular types of slab (skew supports, variable curvature, variable geometry). Two construction methods may be distinguished according to the type of formwork used. The slab may be cast on fixed formwork, which is a method used mainly for bridges of limited length, or it may be cast on mobile formwork.

Fixed Formwork

When the slab is cast in-situ on fixed formwork, the following options may be considered:

- formwork on falsework supported from the ground,
- formwork fixed to the steel structure,
- formwork made of thin concrete planks.

For the first option the formwork is supported on continuous falsework that rests on the ground. The downside of this solution is that it requires an extensive falsework system, and it is therefore only viable for bridges that are reasonably close to the ground. A benefit is that the wet weight of concrete puts virtually no load onto the steel structure. Also, upon striking the formwork, the self-weight of the concrete is directly taken by the steel-concrete composite beams.

When the bridge is high above ground, it may be advantageous to fix the formwork to the steel structure itself. For this solution it is necessary to check that the forces transferred by the formwork can be introduced into the steel structure without either exceeding its resistance or creating instability. It is necessary to consider the method of fixing the formwork to the beams (drilling through plates or temporary fixings welded to the beam, etc.) sufficiently early such that any work needed may be carried out in the shop during fabrication of the beams, rather than requiring additional work on site. When the formwork is fixed to the steel structure, the self-weight of both the formwork and the concrete act on the steel beams. To avoid overstressing the steel structure during casting of the concrete, the designer may specify localised temporary supports of the beams, to be removed after the concrete has set, and it has been structurally connected to the

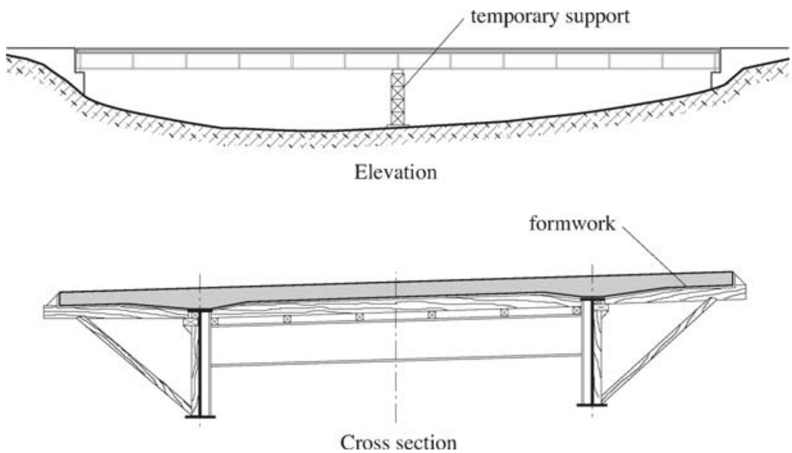


Fig. 8.9 Formwork fixed to the steel structure.

steel beams. When temporary supports are used, the self-weight of the concrete is directly supported by the steel beams. Figure 8.9 shows an example of formwork that is fixed to the steel structure, with temporary supports at mid-span.

Formwork for the slab can also be achieved using thin reinforced *concrete planks*. These concrete planks, which are typically 80 mm to 100 mm thick, are placed directly onto the steel beams. Voids may be left in the planks where they are to pass over groups of studs, and reinforcement in the slabs assures their continuity across zones that are left open for the studs. The slab concrete is then cast in-situ on the planks to create a monolithic system and connect the steel and concrete together. The planks contribute to the transverse bending resistance of the bridge deck, whereas in the longitudinal sense, because of the joint between them, they do not contribute. The self-weight of both the planks and the in-situ concrete act on the steel beams. This type of solution using planks is adopted for composite bridges when the spacing between the steel beams is relatively small, as is the case for some multi beam bridges, or when cross girders are used to support the slab. Figure 8.10 shows an example of formwork using planks for a bridge with a narrow deck.

Rather than adopting concrete planks for the formwork, cold formed steel ribbed decking may be used, particularly if the spacing of the beams is small (2 m to 3 m). The deck acts as lost formwork and is not considered to contribute to the slab resistance. This is because it is difficult to guarantee the performance of

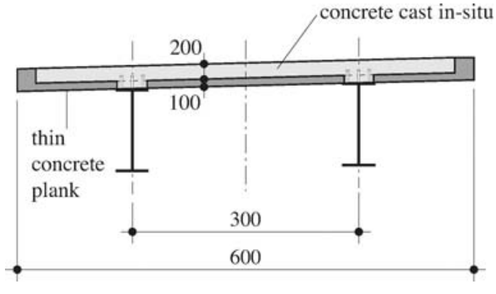


Fig. 8.10 Formwork using thin concrete planks.

thin plates over time, as their durability may be quickly compromised in the sort of aggressive atmosphere which may exist in the vicinity of a bridge.

Placing of the concrete onto fixed formwork easily allows discontinuous casting of the slab. Because the structural connection between the steel and concrete is effective once the concrete has set (studs welded before concreting), it is advantageous to cast the concrete first of all in the spans and then at the intermediate supports (Fig. 8.11). This process reduces the tensile forces in the slab at the supports for formwork fixed to the steel structure. Consequently, as compared to sequential concreting, the likely transverse cracking of the slab at the intermediate supports is reduced.

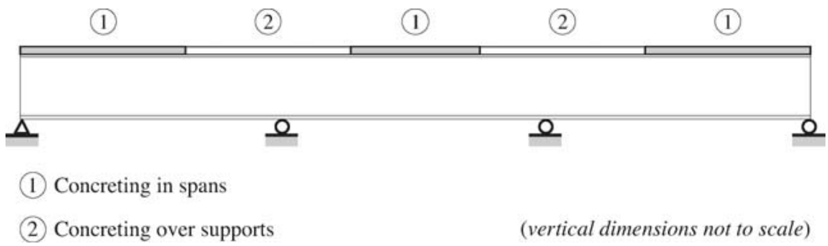


Fig. 8.11 Schematic of concreting stages using fixed formwork, intended to reduce tensile forces in the slab over intermediate supports.

Mobile Formwork

For long bridges that are high above the ground, it is advantageous to cast the slab in-situ using formwork that is supported by a trolley and can move along the steel structure (Fig. 8.12). This solution can be adopted for bridges with a geometry and a transverse cross section that are more-or-less constant. During casting of the concrete, the trolley supports the formwork for the slab cantilevers by means of hangers. For striking, the hangers are withdrawn, and the formwork for the cantilevers is rotated away from the slab. The formwork trolley moves on rails that are supported on the main beams' upper flanges ahead of the area being cast, and on the slab behind the area having been cast. The formwork between the steel beams is often supported on the cross bracing, on which it also moves by sliding. The need to move the formwork should be taken into account during the conceptual design of the bridge cross section, so that the cross bracing is located in an appropriate position to facilitate these operations. Also, the cross bracing must be designed so that it can support this load case.

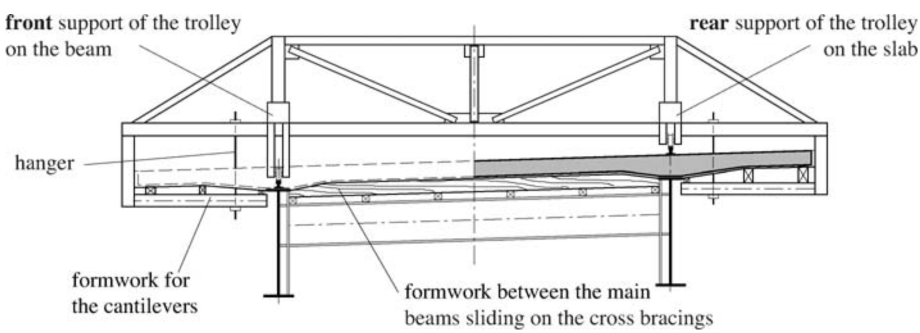


Fig. 8.12 Example of mobile formwork.

When the slab is cast in-situ on mobile formwork, composite interaction begins as soon as the concrete has hydrated. As when fixed formwork is used, *span before pier concreting,* or *span by span concreting,* should be adopted to limit the development of tensile stresses in the concrete above the supports. A downside is that both options require numerous forward and backwards movements of the formwork trolley. The impact of the concreting sequence on the transverse cracking of the slab is discussed in Paragraph 8.5.3.

If the slab is post tensioned longitudinally (Sect. 8.6), and the prestress is only applied to the slab and not to the steelwork, it is necessary to create voids around the groups of studs during casting of the concrete. That way, composite interaction is not achieved as soon as the concrete hydrates, and the concreting can be continuous. When this solution is adopted, there is less likelihood of concrete cracking at the intermediate supports, especially since the longitudinal post tensioning can be quickly introduced – from the third day after concreting.

When mobile formwork is used, the programme for concreting usually allows a length of concrete slab between 15 m and 25 m to be cast in one week. The following sequence is normally adopted:

- Monday and Tuesday: strike the formwork, introduce prestress if necessary, move the trolley and position it in the next location,
- Wednesday and Thursday: fix the reinforcement and any ducts for post tensioning,
- Friday: cast the concrete slab.

8.4.2 Slab Launched in Stages

This method for constructing the concrete slab is analogous to that for launching the steel structure. Sections of slab are cast in an area where there is fixed formwork, and then pushed or pulled onto the steel structure (Fig. 8.13). The main advantage of this method of sliding is that it allows the production of a continuous slab at a reasonably high speed, and needs only limited equipment on site. However, it does require precise execution, including constant checking that the specified tolerances are being respected. Once the sliding has been completed, the studs, which will enable the steel to concrete connection, are welded to the upper flanges in voids that have been left in the slab.

The sliding operation takes place in phases, on a weekly cycle, as follows:

- one length of slab is cast in the concreting area,
- after the concrete has hydrated and thereby gained sufficient strength (3 to 4 days), strike the formwork either by dropping it away or by lifting the slab from it with jacks,
- the new section, which is joined to the preceding sections by longitudinal reinforcing bars left ready for this purpose, is slid onto the steel structure along with the preceding sections, again using jacks,
- the formwork is then repositioned, and the next length of slab can be readied so the cycle recommences.

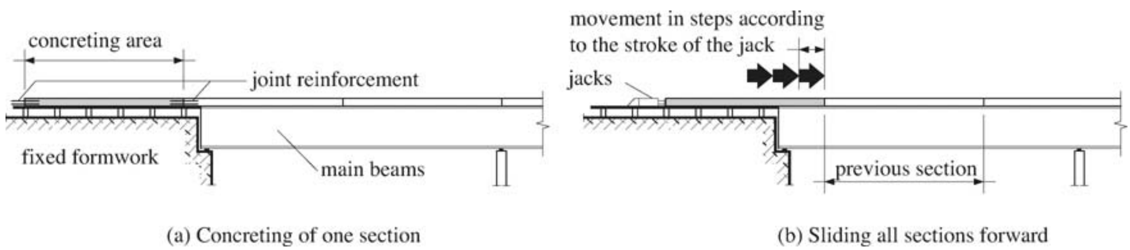


Fig. 8.13 Construction of the slab launched in stages.

Equipment for Concreting

A range of equipment is needed in the concreting area, including hydraulic jacks. The individual sections of slab have a length between 15 m and 25 m. Normally the concreting area is located at one of the bridge ends, either on the steel structure itself or behind it (Fig. 8.13(a)). For a bridge form that rises gradually to its mid-point, it may be advantageous to cast the slab in the middle of the bridge and slide it either continuously or, alternately, in the direction of the abutments. This gives the option to double the speed of slab construction, but it does require strengthening the steel beams in the span or spans where concreting is to take place.

Moving the Slab into Place

The slab may be moved in two ways, namely pushing or pulling:

- hydraulic jacks, with a stroke of around 1 m, are fixed to each of the main beams when the slab is pushed (Fig. 8.13(b)),
- a winch is used to allow continuous movement when the slab is pulled into place. While this solution is good from the point of view of guiding the slab, the movement is jumpy, due to both friction and the long cables used for winching.

When this method is used, temporary plan bracing in the upper part of the steel cross section is essential to guarantee the overall stability of the steel structure. The same is true of cross bracing, which does not interfere with the advancement of the slab as it slides along the tops of the upper flanges of the main beams. It is possible to push a slab weighing up to 3000 or 4000 tonnes, and with a length up to 600 m.

Equipment for Sliding

Cast iron or steel sliding shoes, generally at 2 m spacing longitudinally, reduce the friction between the slab and the steel beams during sliding. They are placed in the voids that are subsequently used to allow welding of the shear studs (Fig. 8.14). Such an arrangement allows the dynamic coefficient of friction to be reduced to around 18%. Because such a reduction is not by itself sufficient, the slip surface is also lubricated with slightly damp graphite, and this allows the dynamic coefficient to be reduced to around 6% (this value was measured on the bridge at the Aigle freeway junction). However, coefficients of friction measured during sliding are variable.

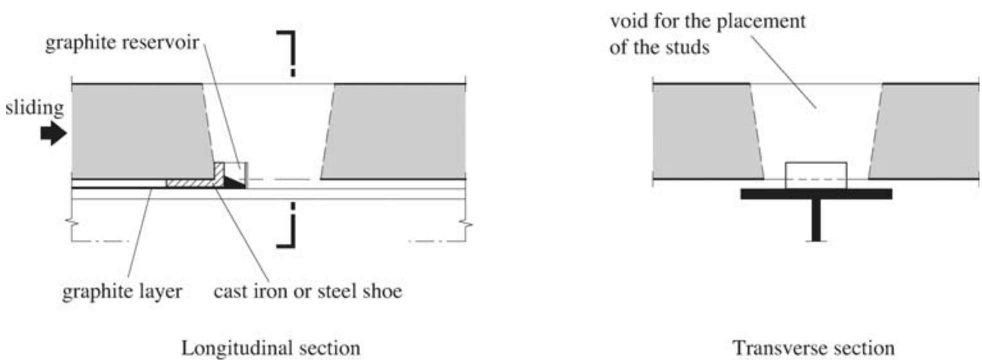


Fig. 8.14 Cast iron or steel sliding shoe with a graphite reservoir.

Guiding the Slab

During the operation of sliding, shoes are placed on the upper flanges of the main beams to help the slab slide over them. Measures must be taken to guide the slab laterally while it is in motion, so there is no lateral movement that would result in eccentric concentrated loading from the shoes onto the steel beams. Lateral guidance is normally achieved using vertical axis rollers, which are fixed to the steel structure above the piers. These rollers may push against either plates attached to the slab specifically for the purpose of lateral guidance (Fig. 8.15), or the slab edges themselves. In the latter case the rollers are fixed to the steel beams using a system of trusses.

All questions related to guidance, tolerances, and retention of the slab must be thoughtfully considered and answered during the conceptual design of the bridge, so that the advancement of the slab during sliding is always under control.

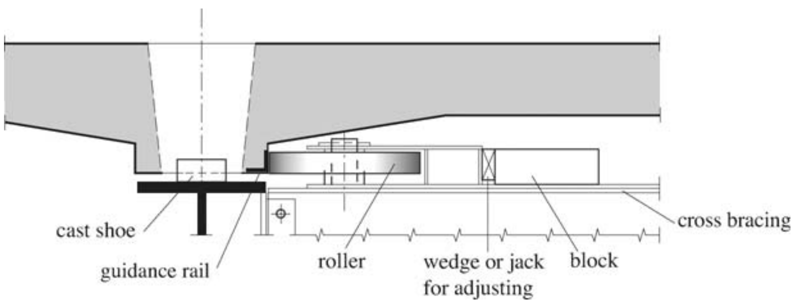
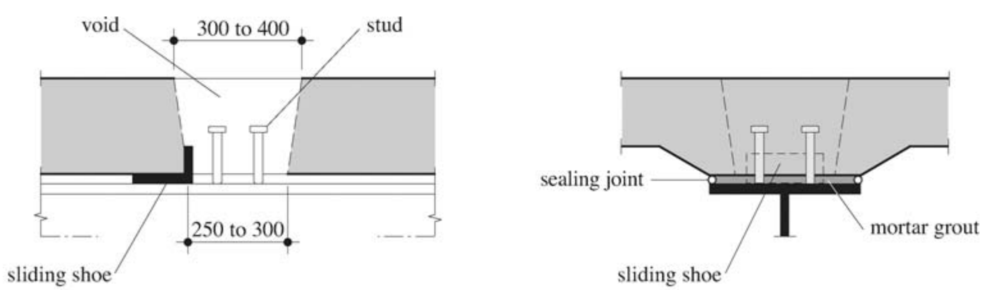


Fig. 8.15 Example of a device for lateral guidance of the slab.

Steel-concrete Connection

Once the entire slab has been slid into place, the studs are welded to the upper flanges of the steel beams using a special welding device. This takes place in the voids that are left open at around 1 m centres for exactly this purpose (Fig. 8.16(a)). Structural connection between the steel and the concrete is then achieved by pouring special low shrink concrete into the voids. The voids must be sufficiently large to respect the minimum spacing requirements between the studs. The concrete, which is placed in the voids, has a greater resistance than that used for the slab in general; this guarantees transfer of the longitudinal shear forces, which are localised because the studs are grouped.



(a) Longitudinal section, studs welded onto the flange

(b) Transverse section after filling the voids

Fig. 8.16 Connection between the concrete slab and the steel flange after placement of the slab.

Because the shoes used to facilitate sliding are left in place, it is also necessary to fill in the space between the steel and the concrete slab to protect the upper steel flanges against corrosion. Typically, mortar is injected into the space between the beam flange and the slab, since it is bounded by sealing joints to prevent the ingress of water (Fig. 8.16(b)).

8.4.3 Precast Elements

It is possible to streamline the construction of the slab even further by adopting slab elements that are prefabricated (precast), either in a factory or on site, then transported and placed on the steel beams. Normally, these elements are the full slab width and around 2 m long, weighing between 15 tonnes and 20 tonnes. They are cast in formwork (moulds) that allow any slab geometry to be achieved with precision. Precast slabs are formed including voids, generally at 1 m centres, to facilitate subsequent creation of the steel to concrete connection using studs set out in groups. Figure 8.17 shows the principles of deck construction using precast slab elements.

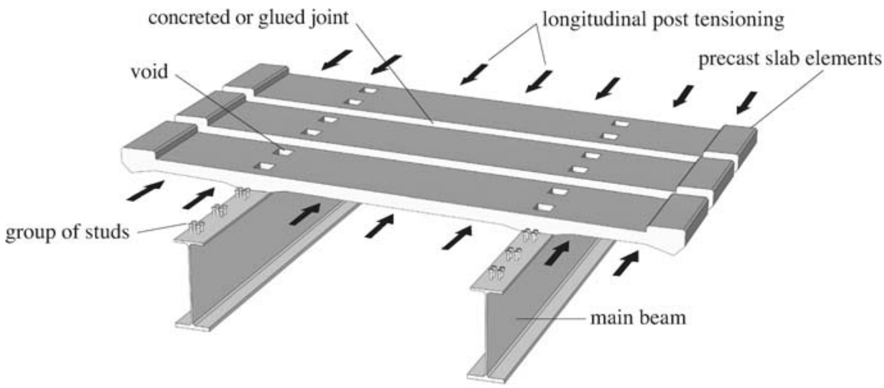


Fig. 8.17 Construction principles for a slab comprising precast elements.

Like launching the slab in stages by sliding it over the steel beams, this method is associated with rapid execution that requires limited equipment on site. Unlike sliding, the shear connectors can be welded to the beams in the fabrication shop prior to placing the precast elements. However, this method does have one considerable disadvantage, namely the slab joints. The numerous transverse joints between the precast elements constitute locations where, over time, durability may be reduced. This is particularly so when the joints are not in permanent compression. Experience shows that slabs comprising precast elements have often shown signs of degradation due to cracking of the joints, accompanied by tearing of the waterproof layer and a presence of water. Adopting longitudinal prestressing by post tensioning of the slab avoids such problems.

There are essentially two distinct ways of forming the transverse joints. The traditional option is to adopt concreted joints, and a more recent alternative is glued joints.

Concreted Joints

In order to form concreted joints, the edges of the precast elements are detailed in such a way that they will act as formwork for the joint (Fig. 8.18(a)). Reinforcement is included in the joint to provide continuity for the starter bars protruding from the precast elements, and to carry the slab shear forces to which the joint is subjected. Any ducts for longitudinal post tensioning are jointed across the gap.

The principles of construction of a slab using precast elements with concreted joints are set out below:

- The precast elements are placed on the steel beams using a mobile crane, which is located either on the ground (when the bridge is relatively close to the ground) or placed on that part of the slab that has already been lifted into place on the steel structure.
- The reinforcement in the joints is completed, and the post tensioning ducts are jointed.
- The transverse joints between the individual precast slabs are concreted following placement of all elements, and the continuity of the slab is thereby assured.
- Longitudinal post tensioning is applied to the slab. It acts solely on the slab because, at this stage, the concrete is not yet connected to the steel structure.
- The voids around the groups of studs are filled with low shrink concrete.

Fabrication tolerances must be strictly respected so that succeeding slab elements can be put in place without difficulty. An accumulation of exceeded tolerances could result, for example, in an inability to place some precast slab elements over the groups of studs.

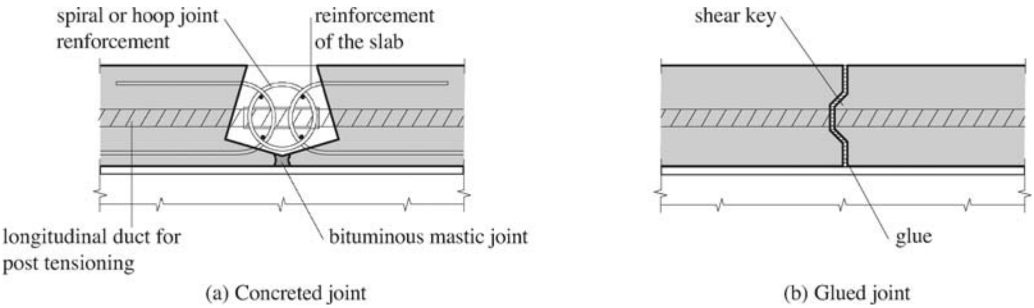


Fig. 8.18 Longitudinal sections of joints in precast slabs.

Glued Joints

Glued joints facilitate more rapid construction of the slab. When this type of joint is adopted, the face of each precast slab element is detailed to include the shear keys, which marry up precisely with the form of the face of the preceding element (Fig. 8.18(b)). Because there is no reinforcement crossing the joint, it is essential to use longitudinal post tensioning to ensure that the joint will be in a permanent state of compression.

The principles of construction of a slab with glued joints are similar to those for one with concreted joints. The only difference is that the joints are glued in sequence as the precast elements are positioned. Temporary longitudinal post tensioning is applied to the slab to help with successful connection of the glued joints.

To reduce the construction time still further, the studs in groups may be replaced by alternative methods of shear connection such as gluing of the precast elements onto the steel beams, or other methods that do not require voids to be left in the slab.

Influence of Tolerances

The use of precast elements for the slab lends itself to twin girder bridges, because any small deviations in level between the two flanges onto which the slab sits are insignificant. On the contrary, for a multi girder bridge, it is more difficult to ensure that the slab will be correctly supported on all the beams.

Typically, the precast elements are not placed directly onto the beam flanges; instead, they sit on interstitial gaskets (sealing joints) placed at the flange edges. The purpose of these gaskets is to compensate for any imperfections in the concrete elements, and with the flanges, to ensure the slab is regularly supported. Precast elements may also be placed on the steelwork via the intermediary of Teflon cleats to facilitate longitudinal movement of the slab during jointing and post tensioning. The gaps between the slab and flanges are subsequently filled by mortar injection (in a similar way to that shown in Figure 8.16(b)).

Finally, any deviations in the formwork and non-uniform creep of the precast elements can lead to differences in level at the slab joints, which may be as much as 10 mm to 20 mm at the tips of cantilevers. Such defects must be avoided, as they spoil the appearance of the visible slab edges.

8.4.4 Influence of the Slab Construction Method on the Bridge Design

Normally, with the notable exception of slabs that are cast in-situ on fixed formwork supported by falsework on the ground, the self-weight of the slab acts on the steel structure alone. This is the most unfavourable situation for the main beams as far as lateral torsional buckling is concerned (upper flange in compression in the span). During concreting of the slab, during sliding or placement of precast elements, the compression flange is not restrained laterally, and this constitutes a hazard scenario that must be taken into consideration. Once the concrete has hydrated, or the voids around the groups of studs have been filled and the concrete has hydrated, the flanges are held laterally, and lateral torsional buckling cannot occur.

The sequence of slab construction may also impact the support reactions applied to the steel beams. Depending on the relative span lengths and the stages used to form the slab, it is possible for negative support reactions to occur, with a resulting tendency for uplift of the beams from the supports. It is particularly important to check for this phenomenon of uplift at the abutments, for end spans that are short relative to their neighbour, and to do so as well during the phase when the slab is being placed in the neighbouring span.

Slab Cast In-situ

The advantage of this method of slab construction is that composite interaction between the steel and concrete is achieved as soon as the concrete has hydrated. When the slab is cast in sections on mobile formwork, the composite cross section thus created is able to resist some of the self-weight of the slab concreted in the subsequent phase. This evolution of the resisting cross sections (from bare steel to composite), as a function of the progression of the slab and the hydrating of the concrete, has an impact on the deflections and the internal moments and forces that develop in the beams during slab construction (§ 13.3.1).

When determining the *deflections*, it is particularly important to take into account the evolution of the resisting cross sections as the slab is cast. Accuracy is needed in order to define the precambering to be applied to the steel beams during fabrication and to compensate for the self-weight of the load carrying structure. Depending on how much the calculations are simplified, the predicted deflections may be considerably different from the real deflections. For example, Figure 8.19 illustrates the effect of the evolution of the resisting cross sections for a continuous beam with the slab cast in-situ in stages going from one end of the bridge to the other. The figure shows deflections that have been calculated according to three different assumptions:

- the self-weight of the slab is applied simultaneously to the whole of the steel structure, so the steelwork alone resists this loading,
- the progression of composite interaction is taken into account, but without considering the creep of the concrete (n = constant),
- the progression of composite interaction is taken into account as well as the effects of creep, which are considered in a detailed manner for each phase of concreting (n = variable)

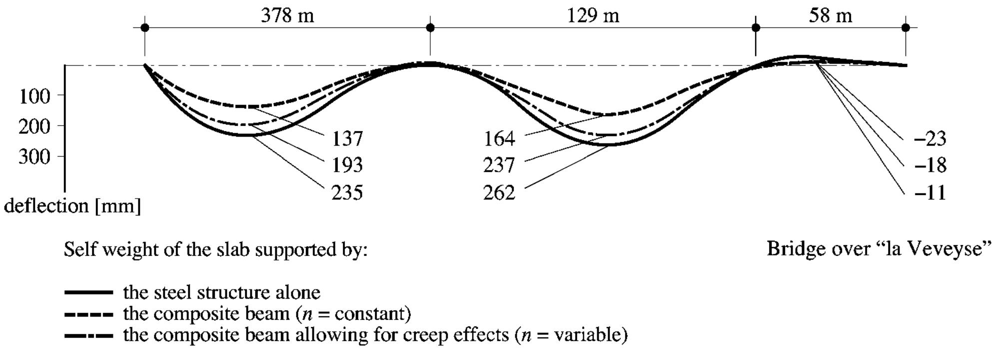


Fig. 8.19 Deflection due to the slab weight according to several calculation models.

These calculation assumptions, which go from the most simple to the most detailed, result in calculated deflections for the steel beam which may differ substantially, depending on how correctly the concreting stages and progression of composite interaction are taken into account. The first of the three options leads to an overestimation of the deflections, because only the steel structure is assumed to contribute to the stiffness. The second option leads to an underestimate of the deflections, because the effects of creep, which effectively unloads the composite section and loads the steel structure, are not taken into account. The third option gives values that approach the real deflections, and on which the precambering should be based.

When calculating the *internal moments and forces* during construction of the slab, it is also important to distinguish between each phase, with different sets of actions and corresponding different resisting cross sections. The loads associated with a section being concreted (self-weight of concrete and formwork) are applied to the steel structure in zones where the slab does not yet exist, and to the composite structure in zones where the concrete has hydrated (Sect. 13.3). The modular ratio n between the steel and concrete must be determined for each section as a function of the age of the concrete.

Slab Launched in Stages

When the slab is launched in stages by sliding it on the steel beams, it is necessary to determine the internal forces, the support reactions, and the deflections of the steel structure for each stage of progression of the slab. The steel beams alone support the self-weight of the slab, because composite interaction only takes effect once the slab has been fully slid into place and the steel to concrete connections created. As an example, Figure 8.20 shows the development of bending moments as launching progresses. It can be seen from this example that the bending moments due to sliding the slab are different in both form and magnitude from those corresponding to the final state, when the slab is in its definitive position. Therefore, when considering structural safety during slab construction, it is necessary to take these effects into account.

It was noted in Paragraph 8.4.2 (Fig. 8.14) that the slab is supported on cast iron or steel sliding shoes generally placed every two metres or so on each of the steel beams. The designer must take into account the concentrated loads that these shoes apply to the steel structure (of the order of 100 kN). Firstly, the webs of the main beams must be checked for resistance to local buckling under these loads. Secondly, checks must be carried out to ensure that the beams will not fail in lateral torsional buckling, given that the upper flanges are in compression in the spans and only restrained laterally during slab construction at points where cross bracing is present.

When this method is adopted for slab construction, the steel to concrete connection only takes place after completion of the slab and filling of the voids. A favourable consequence of this is that, in theory, much of the shrinkage of the concrete slab will have already taken place before the connection is made. This shrinkage will have been able to develop freely, without stressing either the steel beams or the concrete slab. Depending on the time interval between casting the slab and connecting it to the steel beams, it may be advantageous to take into account the reduction in shrinkage when calculating the stresses in the slab.

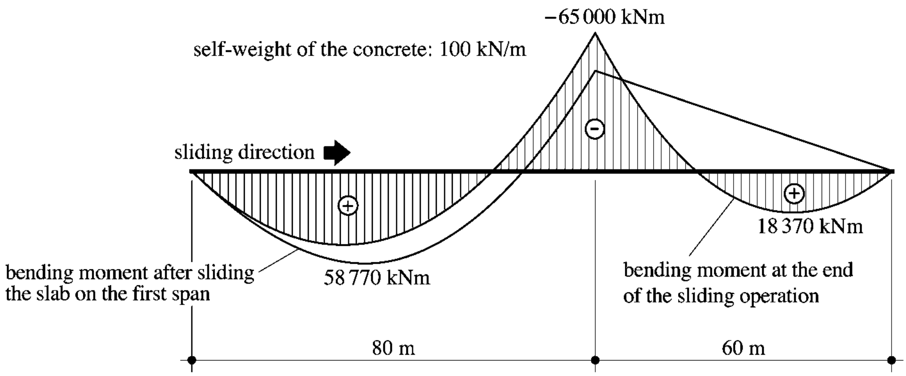


Fig. 8.20 Bending moments in the steel structure due to sliding of the concrete slab.

If the area set aside for concreting is located on the steel structure itself (at an end, or in a span), this extra loading must be taken into account when designing the steelwork. The extra load comes mainly from the formwork and the equipment that is used to lower the formwork after casting and slide the slab.

Precast Slab Elements

The construction loading, which results from placing the precast elements, is applied to the steel structure alone, because the composite connection is only formed after the entire slab has been completed. If the precast elements are placed using a crane located on the ground, it is possible to limit the stresses imposed on the steel structure by choosing an appropriate sequence. If they are placed using a crane that sits on the bridge itself, the slab must be constructed using a continuous sequence, and the resultant construction stresses are the same as when the slab is slid into place. Clearly, the weight of the mobile crane must be taken into account as an additional construction load. The slab elements must be designed to be able to resist the stresses due to handling and transportation, as well as any construction loads prior to achieving continuity of the slab.

8.4.5 Influence of the Slab Construction Method on the Pier Loads

The choice of the method for placing the concrete is not of any great significance in terms of the construction stage stresses applied to the piers (with the exception of uplift). Slab construction does not result in horizontal stresses, except perhaps when the slab is slid into place (depending on the equipment used for lateral guidance of the slab). In the case of a bridge on flexible piers (Chap. 15), the vertical loads due to self-weight of the slab may have an impact on the overall stability of the bridge during construction. Longitudinal stability must be checked for each phase. Finally, for an inclined leg bridge, the construction of the slab results in horizontal longitudinal forces, which must carefully be taken into account.

8.5 Cracking of the Slab

8.5.1 Causes of Cracking

The durability of concrete bridge slabs, be they reinforced, prestressed or composite, is a subject that has troubled engineers and bridge project managers for many years. It is inevitable that all concrete structures will be subject to the development of cracks. This cracking, associated with carbonation and the presence of water containing chlorides within the concrete, has always been considered as one of the drivers of degradation of concrete structures. It is known, however, that with crack widths of less than 0.4 mm [8.4], the cracks themselves only represent one of the parameters playing a role in this degradation. It is reasonably easy to control the width of cracks by using an appropriate percentage of reinforcement.

One of the dominant factors concerning the protection of concrete slabs, and therefore limiting corrosion of the reinforcement, is the compaction of the concrete. Also, when a good waterproofing layer is specified and carefully placed, then water will be unable to reach concrete cracks that develop with time. Even so, despite ensuring the concrete is well compacted and the waterproofing well detailed and placed, in order to guarantee the long term performance of the bridge slab, it is wise to avoid excessive cracking of the slab. It is possible to limit cracking of the slab, in particular transverse cracking, by taking adequate precautions during the conceptual design of the bridge. To help designers choose the most appropriate means to limit cracking, the phenomena that lead to concrete cracking are considered below.

Cracking occurs when the tensile stresses in the slab reach the concrete tensile resistance. The tensile stresses in the slab are due to the combination of actions due to traffic, permanent actions, temperature, and various changes of the concrete, such as shrinkage. Numerical values resulting from an analysis of the tensile stresses present in the slab of a composite bridge, which is assumed to be homogenous concrete, are summarised in Table 8.21 for two composite bridges with different span lengths. The tensile stresses due to various actions are given in chronological order of their effect on the slab. The values are appropriate for a slab, which is directly connected to the steel beams, and with the concrete cast using mobile formwork. For both examples, concreting is assumed to progress from one end of the bridge to the other.

The effect of temperature (§ 13.2.3), which normally results in both tensile and compressive stresses, is not given in Table 8.21, because this analysis is essentially concerned with tensile stresses, which act over the total slab depth. The stresses associated with traffic are those for a 25 tonne truck crossing the bridge. The stresses associated with shrinkage are based on a shrinkage strain of 0.015%.

Table 8.21 illustrates that the most important tensile stresses are present after the end of concreting the slab, in other words after all concreting stages and once the concrete has hydrated. These two initial effects represent more than 50% of the total tensile stresses for a 30 m bridge (2.4 N/mm^2), and more than 60% for a bridge of 80 m (4.5 N/mm^2). Considering the tensile resistance of the concrete at this stage (2.0 to 3.0 N/mm^2), the magnitude of these tensile stresses means transverse cracking will be likely for a 30 m span bridge, and certain for an 80 m span bridge.

Table 8.21 Average tensile stresses in a concrete slab assumed to be homogeneous [N/mm^2].

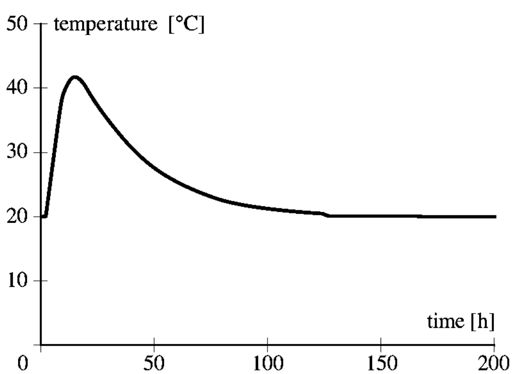
Origin	Span Length 30 m	Span Length 80 m
Concrete hydration	0.6	1.8
Concreting from one end to the other	1.8	2.7
Surfacing	0.8	1.3
Traffic	0.3	0.1
Drying shrinkage	0.8	1.4
Total	4.3	7.3

Therefore, in order to reduce cracking over time, it is essential to limit the tensile stresses during slab construction. The designer should try to limit the effects of *concrete hydration*, and to define *sequences of concreting* that are adequate (§ 8.5.3) when the concreting is cast in-situ and composite connection is achieved as the concrete hydrates.

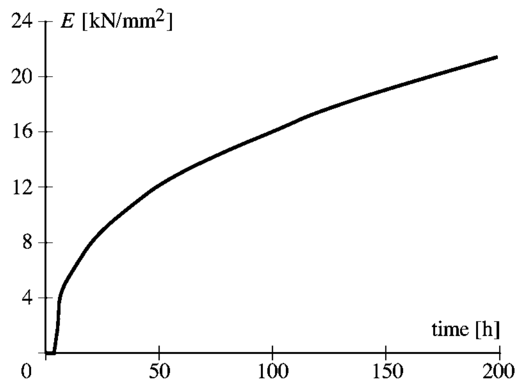
8.5.2 Effects of Concrete Hydration

The thermal behaviour of concrete as it hydrates is characterised by a phase of increasing temperature, lasting around 10 hours, followed by a phase of cooling, lasting around 200 hours. The mechanical properties of the concrete change as it hydrates, with an increase in the elastic modulus between the phases of warming and cooling. Figure 8.22(a) shows an example of the evolution of the temperature measured in a concrete slab as it hydrates. Figure 8.22(b) shows schematically an example of the development of the elastic modulus of concrete as it hydrates [8.5].

If deformations of the concrete during the warming and cooling phases of its hydration are prevented by a structural element, then stresses develop in the concrete. In particular, during cooling of the concrete, tensile stresses develop in the slab. In composite bridges, the elements that prevent deformations are the main beams when the composite connection is made as the concrete hydrates. In such cases the hydration of the concrete can lead to tensile stresses in the slab, which are close to the tensile resistance of the



(a) Evolution of temperature



(b) Evolution of the elastic modulus

Fig. 8.22 Behaviour of concrete during hydration.

concrete. The magnitude of these tensile stresses depends on the degree to which the slab is restrained by the steel beams.

This effect of restraint imposed on the slab by the steel beams can be evaluated in a simplified way by considering an elastic modulus that is constant, but has a different value for the phases of warming and cooling during hydration of the concrete. Figure 8.23 shows for composite bridges the importance of the effects of hydration of the cement, which is associated with hydration of the concrete. It shows, in a schematic way, the stresses that are created in a composite beam under the effects of a warming then cooling phase of the slab, with a temperature change of $\Delta T = 25$ °C for each phase.

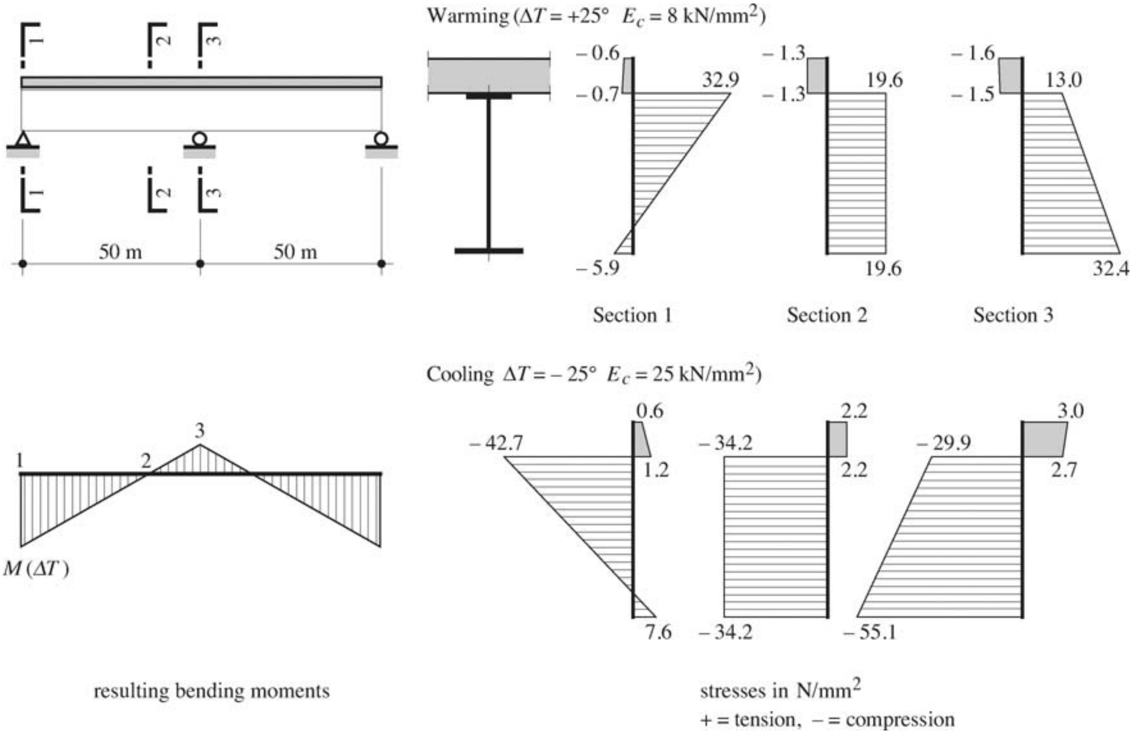


Fig. 8.23 Stresses in the composite cross sections 1 to 3 during hydrating of the concrete.

For the structural form shown in Figure 8.23 (two span beam), the slab is compressed during its warming, and subject to tension during the cooling phase. Stresses in the cross sections are calculated, taking into account the bending moments and normal forces due to ΔT, as well as the corresponding redundant bending moments. The differences between the two states, 200 hours after the concrete has begun to hydrate, are tensile stresses, which result from the increase in the elastic modulus (Fig. 8.22(b)). For this example, which is representative of a composite beam in a twin girder bridge with spans of around 50 m, the tensile stresses in the slab due to hydration of the concrete are between 0.9 and 1.4 N/mm^2 above the central support (cross sections 2 to 3). Such values are noteworthy when compared with the tensile resistance of freshly cast concrete (1.8 to 2.5 N/mm^2 after 200 hours).

For a composite bridge the degree to which the slab is restrained by the steel beams may be defined using the so-called *retention coefficient* n_A, which is simply the ratio between the area of the steel beam and that of the concrete slab [8.6].

$$n_A = \frac{A_a}{A_c} \tag{8.1}$$

n_A	:	retention coefficient
A_a	:	area of the steel beam
A_c	:	area of the concrete slab

A small retention coefficient n_A may correspond to a steel beam of small cross sectional area which therefore only provides a small amount of restraint to the slab. A large value of the retention coefficient n_A reflects a large steel beam which provides a high degree of restraint against deformations of the slab. Calculations of the retention coefficient n_A for a number of existing bridges has shown that it is almost linearly proportional to the span, for cross sections at intermediate supports and in span (Tab. 8.24).

This proportionality allows the retention coefficient n_A to be related in a simplified way to the span of a composite bridge. The interdependence is particularly interesting when evaluating the global effects of the concrete hydration on cracking of the slab as a function of span, although it should always be remembered that the correspondence is only indicative.

Laboratory tests, measurements taken during the execution of bridges, and numerical simulations have all shown [8.6] that tensile stresses σ_c develop in the slab during hydration of the concrete. The magnitude of these stresses depends on the retention coefficient, as shown in Table 8.24.

Table 8.24 Tensile stresses in the slab during hydrating as a function of span.

Span [m]	$n_A = A_a/A_c$	σ_c [N/mm^2]
30	0.05	0.5 à 1.0
50	0.08	1.0 à 1.5
80	0.12	1.5 à 2.1

These tensile stresses develop over the total length of the slab and can therefore lead, for bridges of medium to long span, to general transverse cracking. In order to minimise this effect, it is necessary to either cool the concrete during concreting, or to use a concrete mix that only warms up slightly. Either option needs to allow the tensile stresses to remain below 1.0 N/mm^2 for a retention coefficient n_A of 0.12 [8.6]. If neither of these measures is adopted, it is necessary to place "anti-crack" reinforcement, sufficient to prevent transverse cracking, over the entire length of the slab. This is particularly the case for composite bridges having a retention coefficient n_A greater than 0.08.

Simplified calculations, assuming the concrete slab is fixed to a rigid support, allow the tensile stresses that develop in the slab after hydration of the concrete to be determined. Ignoring the effects of bending moments, and only considering equilibrium of the normal forces in the composite cross section, the tensile stress σ_c in the slab can be evaluated using the following equation:

$$\sigma_c = \frac{\alpha_T \cdot n_A^2 \cdot \Delta T \cdot E_a^2 \cdot (E^*_{c2} - E^*_{c1})}{(n_A \cdot E_a + E^*_{c2})(n_A \cdot E_a + E^*_{c1})} \tag{8.2}$$

σ_c	:	tensile stresses in the slab at the end of concrete hydration
α_T	:	coefficient of expansion of the concrete, which is assumed to be the same as that of steel ($\alpha_T = 1 \cdot 10^{-5}$)
n_A	:	retention coefficient (A_a/A_c)
ΔT	:	difference in temperature between that of the concrete and that of the environment during hydration
E_a	:	elastic modulus of steel
E^*_{c1}	:	adjusted average elastic modulus of concrete during the warming phase
E^*_{c2}	:	adjusted average elastic modulus of concrete during the cooling phase

The adjusted average elastic moduli for the concrete E^*_{c1} and E^*_{c2} are assumed to be constant, but different between the warming and cooling phases. These moduli are adjusted to take into account creep according to TGC Volume 8, and according to [8.7]. For the types of concrete normally used in composite bridges, the following values may be adopted when using equation (8.2):

- $E^*_{c1} = 6$ kN/mm^2,
- $E^*_{c2} = 25$ kN/mm^2,
- $\Delta T = 25$ °C.

The agreement between calculations made using this simplified equation and numerical simulations, as well as measurements from tests, is very good. Differences are slight, given the complexity of the phenomenon. This method means that simple calculations can be used to identify cases that could result in elevated tensile stresses in the slab, and therefore require the study of specific preventative measures to limit their values.

8.5.3 Influence of the Concreting Sequence

When the slab is cast in-situ and already connected to the steel structure during concrete hydration, the concreting operations may introduce tensile stresses in the slab above intermediate supports, which are sufficiently large (Tab. 8.21) to contribute to transverse cracking of the slab. This cracking will be more likely when the concrete is freshly cast and the slab is already subject to tension due to hydration of the concrete (§ 8.5.2).

Placing of the slab in this way is normally carried out in stages using a concreting trolley or, in cases where there is easy access under the bridge or the number of spans is limited, using formwork fixed to the steel beams. Movement of the concreting trolley or formwork between concreting stages is not always a straightforward operation, so effort should be made to minimise the number of moves needed for these elements. There are several ways in which the concreting stages can be organised, such as those illustrated in Figure 8.25. These are *sequential concreting*, *span before pier concreting*, and *span by span concreting*.

Sequential concreting (Fig. 8.25(a)) is the most logical because the trolley always moves in the same direction and only over short distances (between 15 m and 25 m). However, this option is unfavourable from the point of view of stresses in the slab, particularly in the regions of the intermediate supports. Effectively, while the regions above the intermediate supports are already concreted (stages 3 then 6), the placing of fresh concrete in the following spans (stages 4 and 5, then 7 and 8) creates tensile stresses in the slab, which can be quite high in the support zones (Tab. 8.21).

Casting the mid-span sections before the pier sections (Fig. 8.25(b)) allows these tensile stresses at the supports to be avoided, because the regions in the span are cast before the support regions themselves.

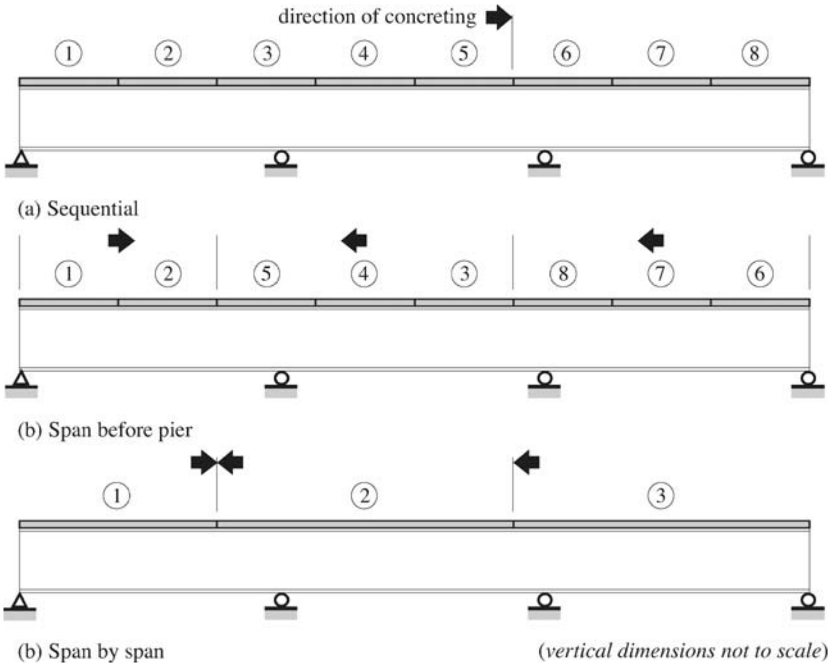


Fig. 8.25 Schematic of concreting stages for a slab that is cast in-situ.

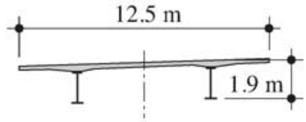
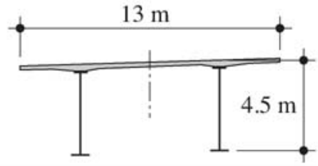
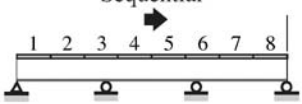
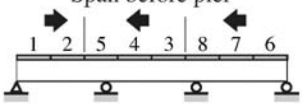
However, this method does require movements of the formwork elements that are not easy, notably because the trolley must pass over zones that have already been concreted, for example stages 3 and 4 when moving from stages 5 to 6.

An alternative to casting the spans before the support regions (Fig. 8.25(c)) avoids these trolley movements over concreted regions and, by increasing the length of the concreting stages, limits the number of formwork displacements. Concreting of the large stages, in 2 and 3, achieves these objectives. However, this method does require the casting of long lengths of slab, and is in principle only viable for important bridges that justify the amortization of the cost of the concreting trolley. Concreting in long lengths also implies the need to guarantee both the concrete supply and the means of placing it. This method may be particularly beneficial for long bridges that have only short to medium spans.

Generally, sequential concreting and span before pier concreting are the most widely adopted methods. In order to compare these two options for concreting, Table 8.26 presents stresses calculated for the mid-depth fibre of the slab, resulting from the concreting operations for two continuous bridges with three identical spans of 30 and 80 m, respectively. The calculations take into account the movements of the concreting trolley and the casting of the concrete as a function of the chosen method. Stresses in the slab are calculated at the first intermediate support, after all concreting phases, taking into account the effects of creep and the variable elastic modulus of the concrete.

Comparison of the results illustrates the benefits of casting the span sections before the pier sections, rather than sequential concreting, as far as the stresses at the supports are concerned. The former method even creates compressive stresses in the slab at the intermediate supports for the two sections presented in Table 8.26. The results are particularly interesting for the bridge with 80 m spans. In this case, sequential

Table 8.26 Comparison of stresses in the slab above the first intermediate support as a function of the method of concreting.

Cross section	 12.5 m; 1.9 m	 13 m; 4.5 m
Span	$l = 30$ m	$l = 80$ m
Retention coefficient	$n_A = 0.04$	$n_A = 0.12$
Sequential 	$\sigma_c = 1.8$ N/mm^2 (tension)	$\sigma_c = 2.7$ N/mm^2
Span before pier 	$\sigma_c = -0.2$ N/mm^2 (compression)	$\sigma_c = -0.5$ N/mm^2

concreting results in tensile stresses in the slab of the order of 2.7 N/mm^2, which could lead to significant cracking, while concreting the spans first results in a slight pre-compression at the supports (−0.5 N/mm^2).

Although concreting the span sections first limits tensile stresses at the supports, span regions that have already been concreted are subjected to tensile stresses during concreting of the subsequent support regions. This is also true for sequential concreting. The consequences of this, however, are minor, as the span sections are subsequently subjected to compression due to the permanent actions.

With the aim of limiting tensile stresses in the slab, it is also possible to use temporary supports. Figure 8.27 shows the procedure that is used for this approach.

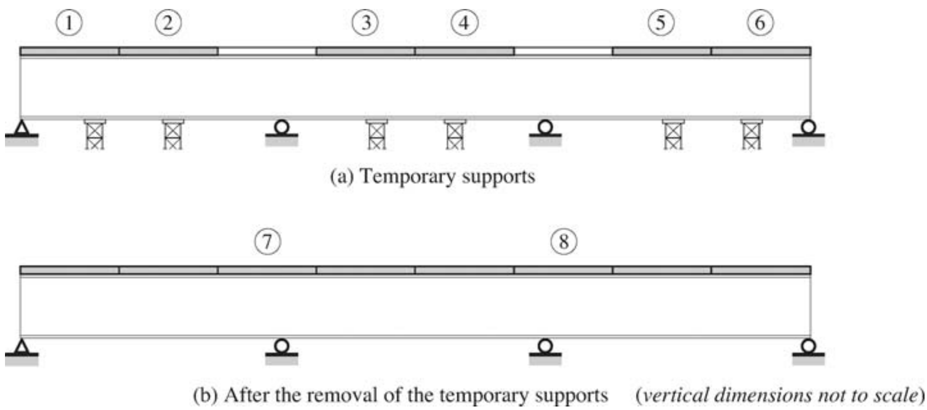


Fig. 8.27 Concreting stages with temporary supports (props).

Concreting the slab when the beams are on temporary supports consists firstly of concreting the span regions above the temporary supports (phases 1 to 6). The temporary supports are removed once the concrete has hydrated, and so those regions act as composite sections with the slab in compression. Concreting of the support regions then takes place. Adoption of this method not only allows some of the self-weight of the concrete in the spans to be resisted by composite cross sections, it also reduces the tensile stresses at the support regions. Although this method often allows the size of the steel beams to be reduced, its use is limited by complications to the execution resulting from the need to apply temporary supports. Also, concreting using temporary supports is only applicable in the particular case of bridges that are at limited height above the ground.

8.6 Longitudinal Prestressing

Longitudinal prestressing of the slab is one of the options available for improving the durability of a composite bridge. It should be considered, however, only as supplementary to good conceptual design and construction of the slab and its reinforcement [8.8], including compaction of the concrete and adequate waterproofing. The objective of longitudinal prestressing is to avoid transverse cracks forming in the concrete slab. In general, it is desirable for the slab to remain, over the long term, in a state of longitudinal compression under the effects of the permanent actions. It may even be desirable, according to the specifics of a given bridge, for the slab to remain longitudinally compressed under either a portion or all of the variable actions.

For composite bridges with the slab cast in-situ, the use of longitudinal prestressing should only be considered for long span structures (high value of n_A). Such structures are the most likely to suffer from transverse cracking, given the origin of the tensile stresses that may develop in the slab of a composite bridge (Tab. 8.21), and assuming other precautions have not been taken during concreting (§ 8.5.2 and § 8.5.3). On the other hand, longitudinal prestressing is always justified when precast slab elements are used, and is essential when the joints between these elements are glued (§ 8.4.3).

Whether or not to use longitudinal prestressing for a composite bridge revolves around the following questions:

- What will be the long term effectiveness of the prestressing, given the losses that occur due to the long term concrete behaviour?
- At what age should the longitudinal prestress be introduced? Late application means that losses with time will be reduced, but on the other hand, tensile stresses develop in an in-situ slab at a young age, meaning prestressing should ideally take place as early as possible.
- Is it better to put the slab or the composite beams into a state of prestress? Only putting the slab into a state of prestress means specific construction details must be adopted to delay the composite connection (voids around groups of studs). On the other hand, prestressing of the composite beams on the whole requires greater levels of prestress to achieve the same effect.
- What impact will prestressing have on the cost of the bridge?

Answers to these questions are given in the following paragraphs. They also provide information that should enable the designer to decide, for a given bridge, whether or not prestressing is appropriate and of interest.

8.6.1 Choice of Prestressing Method

The losses of longitudinal compressive prestress, due to the concrete behaviour (shrinkage, creep) as well as the transfer over the long term of the prestressing force from the slab into the steel beams, mean that it is often difficult to justify prestressing the slab of a composite bridge. An additional problem is that calculations to estimate losses are complex and often inaccurate. A study of several methods used for prestressing has analysed the losses of stress with time, as a function of the age at the time of prestressing, the age of the concrete at the time of forming the composite connection, shrinkage of the concrete, and the amount of prestress introduced [8.4]. The study compared the different methods with reference to the losses. The results given below concern the methods defined in Figure 8.28.

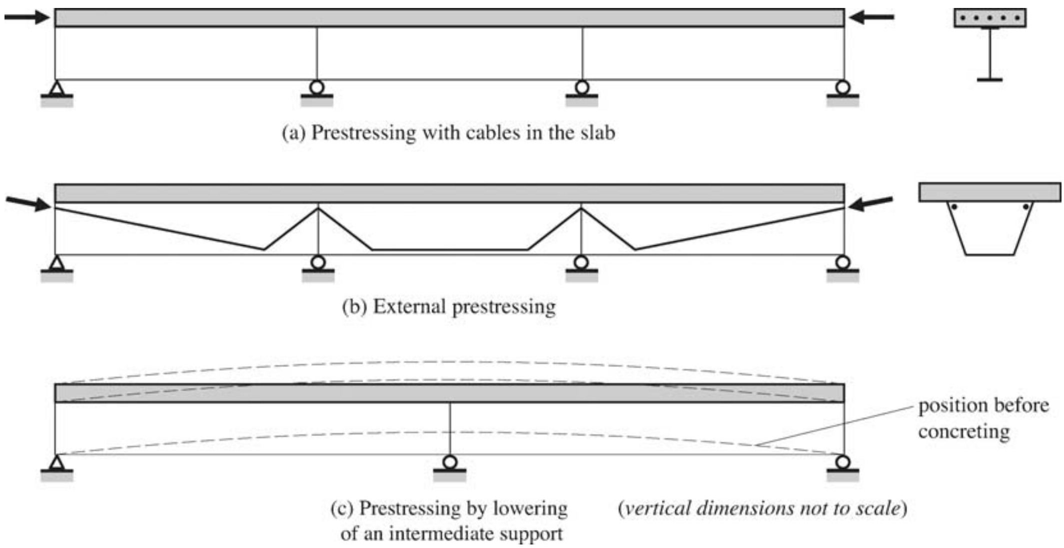


Fig. 8.28 Schematic representation of different methods for longitudinal prestressing.

The prestressing method shown in Figure 8.28(a) comprises placing ***cables centrally*** in the slab, which may be cast in-situ, slid into place, or formed from precast elements. These cables are then put into tension either before or after the composite connection is formed between the slab and steel beams. When the cables are tensioned before the slab is connected to the beams, the prestress is only applied to the slab. This has the advantage of requiring a smaller prestressing force than that needed when the composite connection is formed first. On the other hand, prestressing after the connection has been formed between the slab and steel beams avoids the need of voids in the slab around the groups of studs. The slab is then more homogenous.

Figure 8.28(b) shows schematically a method known as ***external prestressing*** because the cables are not placed within the concrete. They instead are fixed to the steel beams, and the geometric lines of the cables allow them to introduce bending moments that oppose the moments due to the permanent loads and traffic actions. An additional advantage of this method is that external cables can be easily inspected and replaced, if need be. A disadvantage is that the construction details needed to introduce the force and the changes of direction add to the fabrication costs of the steelwork and require suitable fatigue resistance.

The ***lowering of intermediate supports*** to put prestress into the slab is shown in Figure 8.28(c). This method comprises lifting the steel structure (or erecting it with a precamber) above the intermediate supports prior to concreting the slab. Subsequent lowering at the supports after hydrating introduces compression into the slab. This method is mainly used for two and three span bridges, particularly for flyovers.

Beyond three spans it is complicated to manage and carry out the lowering operation at the supports, and the effect of prestressing is smaller. Examples exist where prestressing by lowering of the supports has been combined with prestressing by post tensioning ducted cables, after making the composite connection, so that some of the prestress losses after the lowering operation are compensated.

The ***loss of prestress*** for a longitudinally prestressed composite beam is determined by calculating the drop in compressive stress in the slab at an internal support after 10 000 days. Figure 8.29 shows schematically how the value of stress in the slab σ_c changes with time. This can also be defined in terms of the variation $\Delta\sigma_c$, which expresses the loss of compressive stress as a percentage:

$$\Delta\sigma_c = \frac{\sigma_{c0} - \sigma_{ct}}{\sigma_{c0}} \cdot 100\% \tag{8.3}$$

σ_{c0} : initial compressive stress in the slab at an intermediate support at the time of prestressing, which is the average initial stress, taking into account the sources of immediate loss (friction on the cables, movements of the cables),

σ_{ct} : compressive stress in the slab after 10 000 days, taking into account relaxation of the steel cables, shrinkage, creep, and the transfer of prestressing force from the concrete slab to the steel beam that is due to the concrete behaviour over time.

A value of $\Delta\sigma_c$ approaching 100% indicates that the initial compressive stress at the intermediate support has practically disappeared with time.

Calculations have been made for a number of typical composite bridges with spans of 30 m, 50 m, and 80 m. The curves shown in Figure 8.30, which are taken from [8.6], show the loss of stress $\Delta\sigma_c$ as a function of span for two different times of prestressing using cables in the slab. The value t_0 indicates the age of the concrete at the time when the concrete to steel connection was made (for post tensioning after 7 days), while t_p is the age of the concrete at the time of prestressing for slabs connected immediately upon concreting.

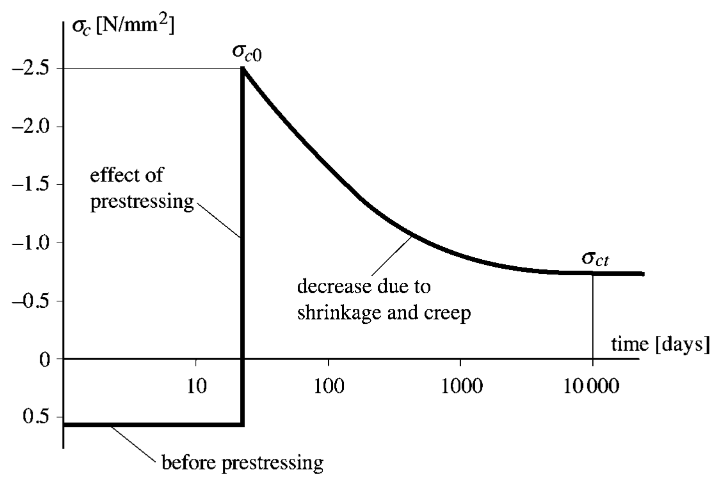


Fig. 8.29 Schematic of the evolution of the stress in the slab above a support as a function of time.

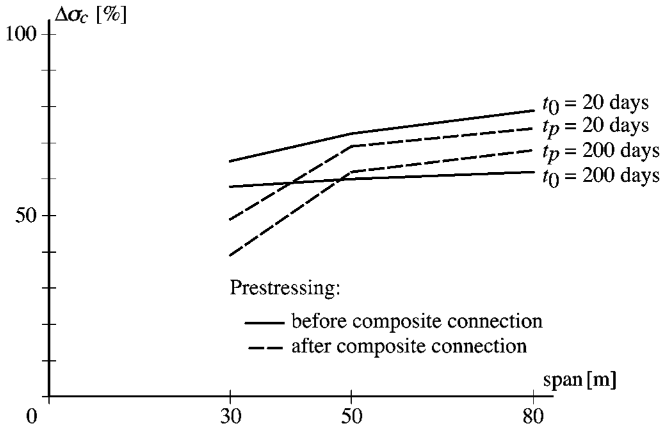


Fig. 8.30 Loss of prestress $\Delta\sigma_c$ as a function of the span for the case of post tensioning using cables in the slab.

The following comments can be made on Figure 8.30:

- prestress losses increase as the span increases,
- for prestressing prior to composite connection, the loss is greater if that connection is made rapidly (t_0 = 20 days) after prestressing,
- for prestressing after the composite connection is made, the sooner the prestressing takes place (t_p = 20 days), the greater the losses,
- generally, once spans exceed 50 m, then, whatever timing is used for prestressing, the loss of compression in the slab will be more than 60% after 10 000 days.

The values shown in Figure 8.30 were calculated for a shrinkage coefficient of 0.02%. Losses of prestress would be greater for larger values of this coefficient (for a shrinkage coefficient of 0.03%, the values of $\Delta\sigma_c$ shown in figure 8.30 increase by around 15%). Also, the calculations were based on a value of $\sigma_{c0} = -2.5$ N/mm^2. For an initial compression of twice this amount, the losses are smaller ($\Delta\sigma_c$ reduces by around 10%). Effectively, while for such cases the loss of compression is greater in absolute terms, the loss relative to the initial value of stress (eq. 8.3) is normally less. Finally, for composite bridges that are prestressed by lowering an intermediate support, the losses of prestress are substantial, and even total for long spans ($\Delta\sigma_c \approx 100\%$).

In practical terms, for a composite bridge with a slab that is cast in-situ, the choice of method used to apply longitudinal prestress to the slab should take into account the likely occurrence of transverse cracking. This is most likely for long span bridges (Tab. 8.21). Clearly, it is necessary to consider prestress losses when determining the initial compression to put into the slab. The designer must also take into account whether the prestress is to be applied to the slab alone (prestress before the composite connection), or to the whole composite section. There are also some parameters for the designer to consider that work against each other. For example, it is desirable to prestress the slab as soon as possible because tensile stresses develop in the concrete at an early stage. However, the sooner the prestress is applied, the greater are the losses. Finally, it is worth repeating that prestressing is a measure that should be considered to be supplementary in terms of reducing cracking of the slab, and other measures taken during concreting can be just as effective and less costly.

Table 8.31 summarises the different options for longitudinal prestressing of a slab. It includes recommendations for prestressing methods that may be used depending on the spans of a given bridge, based on the expected losses of each method and in order to remain economically attractive.

Table 8.31 Examples for the choice of prestressing methods.

	Span		
Prestressing method	30 m	50 m	80 m
With cables in the slab, before composite action	Avoid	Conceivable	Recommended
External cables, after composite action	Expensive	Conceivable	Conceivable
With cables in the slab, after composite action	Recommended	Conceivable	Expensive
By lowering of intermediate supports, for 2 to 3 spans	Recommended	Conceivable	Avoid

Of particular note from Table 8.31 are the recommendations that longitudinal prestressing is introduced into the slab after forming the composite connection when the steel beams are relatively small (short spans), and before forming this connection for bridges where the steel beams are heavier (long spans). This preference to prestress the entire composite section for short spans comes from the economic savings that can be made by not having to form voids around the groups of studs. Those savings outweigh the additional costs associated with the greater prestressing force that is needed.

8.6.2 Simplified Method for Calculating Prestress Losses

While Table 8.31 recommends prestressing methods for different situations, it does not allow a numerical value for the loss of prestress in a composite bridge over time to be calculated. However, it is possible to obtain a good estimate of the loss of compression in the slab at the intermediate supports using a simplified model. The scope of application of this model is limited to the prestressing methods that are described in Table 8.31. With reference to the method of calculating the different effects as given in TGC, Volume 8, and several other references [8.9], [8.10], it is possible to use the following equation for $\Delta\sigma_c$. This is equation (8.3) but neglects, when considering equilibrium of the internal forces, the minor influence of the bending moments:

$$\Delta\sigma_c = \frac{n_A \cdot E_a \cdot (\varphi \cdot \sigma_{c0} + \varepsilon_{cs} \cdot E_{c0})}{\sigma_{c0} \cdot (n_A \cdot E_a + n_A \cdot E_a \cdot \chi \cdot \varphi + E_{c0})} \cdot 100\% \tag{8.4}$$

n_A	:	retention coefficient (A_a/A_c)
A_a	:	area of the steel beam
A_c	:	area of the concrete slab
E_a	:	elastic modulus of steel
E_{c0}	:	elastic modulus of the concrete at the instant when prestress is applied
σ_{c0}	:	initial compressive stress in the slab at the supports after introduction of the prestress
ϕ	:	creep coefficient
ε_{cs}	:	strain due to shrinkage
χ	:	ageing coefficient according to [8.8] and TGC, Volume 8

The parameters in equation (8.4) concerning the long term behaviour of concrete should be determined based on values given in [8.11] or the standard SIA 262. For the final value of shrinkage, measurements undertaken by ICOM [8.12] have shown that the value given in the standards is slightly greater than values measured in-situ. According to reference [8.13] final values of shrinkage were close to 0.02% for measurements taken in-situ on specimens.

Values determined using equation (8.4) have been compared with the results of simulations using numerical modelling. Differences between the two were less than 15%, which is small considering the various unknowns concerning the behaviour of the materials (ageing, creep, final value of shrinkage). It can be concluded that it is possible to use equation (8.4) to provide a simple estimate of the loss of compression in the slab of a composite section, at the supports, without the need to undertake complex numerical modelling in the early stages of design.

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9 Basis of Design

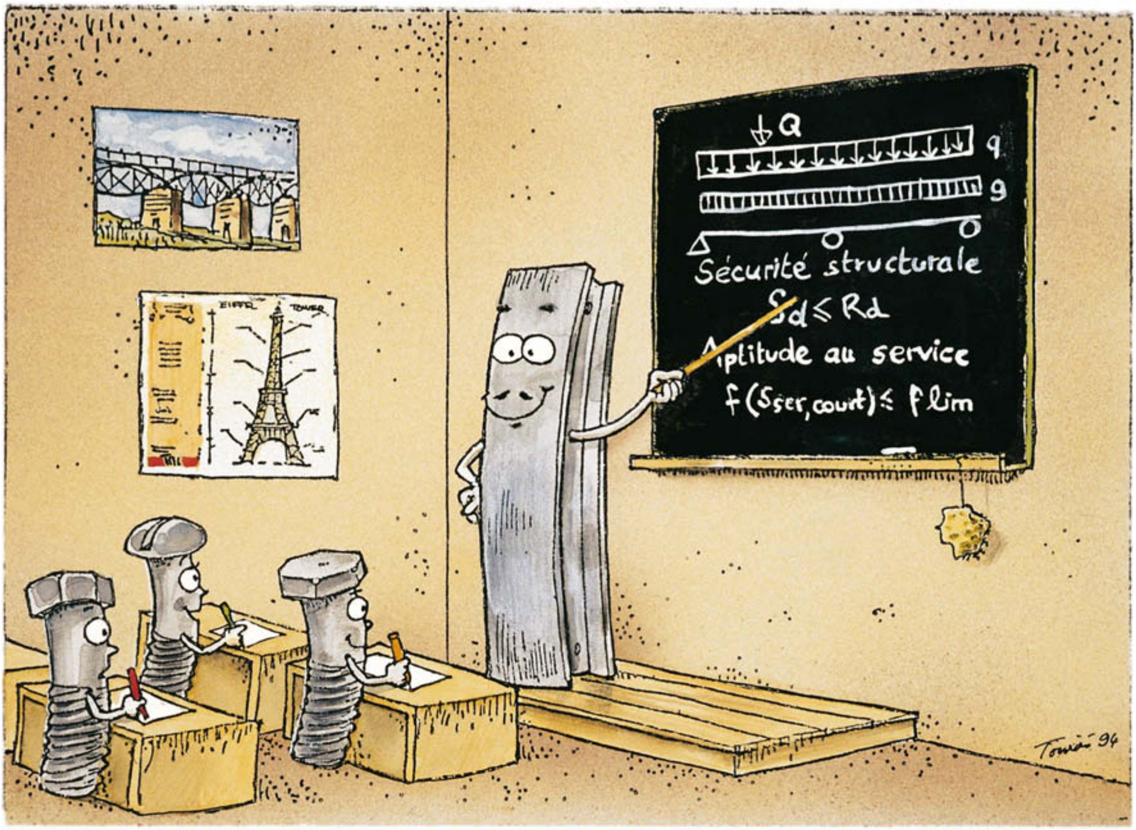

Illustration by Thomas Mikulas, Le Mont-sur-Lausanne.

9.1 Introduction

When designing a structure, certain principles must be applied concerning the checks needed to guarantee both its serviceability and its structural safety. While the chapters that follow (Chap. 10 to 15) consider the actions, the structural analysis and the structural design of steel and composite bridges, the aim of this chapter is to recall the basic principles, which are also presented in TGC Volume 10, Chapter 2.

Section 9.2 recalls basics and organisational documents that are relevant to all forms of construction. In particular, it presents the various phases during the life of a structure. Consideration of these phases leads to the development of documents covering the input data, the assumptions, the decisions made, the calculations, and the drawings that are needed to transfer information to the various parties involved in the life of a bridge.

Section 9.3 describes in more detail elaboration of a project that concerns the conceptual design of the load carrying structure and its construction details, preliminary sizing, structural analysis, and then leading on to the structural design (Fig. 4.1). This section also identifies the contents of the principal documents associated with the project elaboration, namely the client's requirements and the basis of design. The project phase that is described in a general manner in this section is illustrated with more detail, and as specifically applied to bridge projects, in Section 4.2.

Section 9.4 summarises the terminology associated with loads and actions and is useful as an aid to understanding the sections that follow it. Sections 9.5 and 9.6 recall the principles of the various verifications as well as the requirements concerning, respectively, serviceability and structural safety.

9.2 Bridge Life Cycle and Documentation

For all construction projects the aims of the designer, generally speaking, may be defined as follows:

- provide the client with a structure that satisfies expectations for its use,
- guarantee users an adequate level of structural safety,
- provide a structure that is durable, from the points of view of both its serviceability and its structural safety.

The design of the load carrying structure constitutes an important phase during the elaboration of a project as far as the ability to achieve these aims is concerned. It is one of the ways that the serviceability and structural safety requirements are satisfied, and results in:

- sizing of the elements that form the load carrying structure,
- defining the material qualities and properties,
- confirming and completing the construction details conceived during the conceptual design, including the preliminary sizing.

It is not possible to design, execute, and use a bridge correctly unless all those individuals and organisations involved in the whole life of the bridge have knowledge of the requirements and measures to be taken to guarantee its serviceability and structural safety. Good transfer of information between the different parties can only be achieved if the roles of each one are clear and coordinated. Considering both the phases in the life of a structure and the necessary organisational documents (Fig. 9.1) highlights these aspects.

This Figure 9.1 shows those aspects that should be considered in order to guarantee serviceability and structural safety:

- the different parties involved, namely the client's representative, and the key authors of the project such as the architect, the engineer, perhaps some specialists and some administrators,

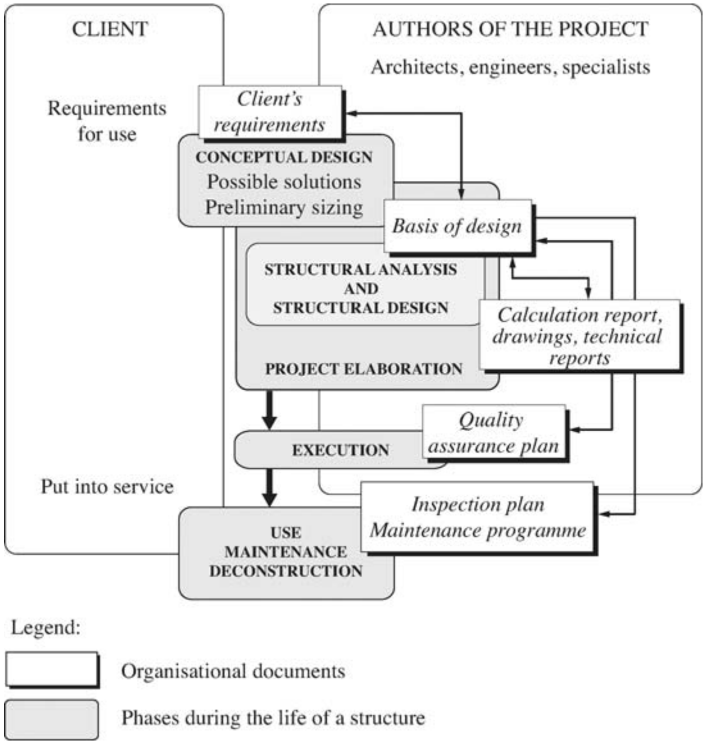

Fig. 9.1 Schematic representation of the steps and phases during the life cycle of a bridge, and the corresponding organisational documents.

- different phases of the project elaboration, namely the conceptual design including preliminary sizing, structural analysis and structural design, as well as execution, use, maintenance and eventually deconstruction,
- various documents in which the decisions made are recorded, such as the client's requirements, the basis of design, the calculations, the drawings and technical reports, the quality assurance plan for the execution, the inspection plan and maintenance programme. All these documents contain information that must be clearly stated in order to guarantee serviceability and structural safety.

The different phases illustrated in Figure 9.1, and the corresponding documents, are as follows:

- At the start of any construction project, the dialogue must be established between the client's representative and the architect/engineer. This dialogue must allow the aims of the client, concerning use of the structure, to be communicated. Similarly, it must cover all the various requirements and limitations concerning the project elaboration, its execution and use. These items are included in the ***client's requirements***, which must be written in such a way as to be easily understood by the client's representative and contain all the decisions that have been taken and agreed.
- The authors of the project are then responsible for translating these requirements into technical terms for use by construction professionals and presenting them in a document known as the ***basis***

of design. The content of this document necessarily evolves; it must be corrected and completed as the various phases of the project elaboration, and the execution, progress.

- During the ***project elaboration*** (Sect. 4.2), the client's requirements and the basis of design are key reference documents for use by the engineer undertaking the ***conceptual design***. This phase, which includes preliminary sizing, results in the definition of the ***structural concept*** that describes, in particular, the structural form, the construction method, key dimensions and sizes, the characteristics of the materials and the main construction details.
- The engineer then moves on to the ***structural analysis*** and the ***structural design***. The structural analysis allows the behaviour of the load carrying structure, when subjected to the various actions, to be quantified. The structural design determines the final sizes of the structural components by considering the action effects on deformations and resistance of the structure. This is done by applying the principles of checking for serviceability and structural safety.
- During the structural analysis and structural design, the engineer produces calculations, drawings and technical reports. These documents serve as the basis for the ***execution***. In addition, a ***quality assurance plan*** is produced to provide the means by which it can be assured that the execution of the structure complies with the stated requirements.
- An ***inspection plan*** and a ***maintenance programme*** are provided to the client's representative, so that when the bridge is in service he has the necessary information to respect the contents of the client's requirements and basis of design concerning serviceability and structural safety.
- During the ***use*** of a bridge, the client's representative is responsible for respecting the guidance given in the maintenance programme so that durability can be assured. Also, he must take whatever measures are necessary to ***maintain*** the structure in a state that is appropriate for its planned service life.

9.3 Project Elaboration

9.3.1 Client's Requirements

At the start of any project, the client's representative and the main authors of the project must together define the client's requirements (Fig. 9.1). This is a document that compiles the various requirements for use of the structure, expressed in a way that can be understood by the client's representative. All parties concerned should sign this document, which should contain, for example:

- the general aims for use of the bridge (type of traffic for a road bridge, railway bridge, pedestrian bridge or mixed use), its position with regard to the communication route of which it is part, and any limitations to the bridge (Sect. 4.3),
- the planned service life,
- specific needs in service and for maintenance, for example concerning waterproofing, surfacing and noise protection,
- the context and requirements of any third parties, such as users of communications routes that are crossed by a bridge, any access limitations, or needs for protection during execution,
- specific objectives of the client's representative concerning, for example, the use of certain materials or the deadline for putting the bridge into service,
- the aims of any protective measures, and any special risks, such as those related to fire and flooding,
- any particular requirements taken from codes and standards, such as those relating to the use of a bridge by special transport.

9.3.2 Basis of Design

The basis of design document for a project is effectively a translation of the client's requirements into technical language suitable for use by the project authors. It contains details of the considerations given and decisions made during the course of the project elaboration and execution, with regard to serviceability and structural safety. Therefore, the basis of design constitutes the key document used by the project authors and various specialists to inform each other of their calculations, the quality assurance plan for execution, and the inspection plan and maintenance programme for the use and maintenance of the bridge.

The extent and contents of the basis of design depend on the importance of the bridge and any particular dangers it faces. Within the document, those considerations related to serviceability are distinguished from those related to structural safety. It is important that these two are not mixed up.

Serviceability

As far as serviceability is concerned, the basis of design particularly contains information related to the following:

- the planned service life of the bridge,
- the intended use,
- requirements related to those uses concerning functionality, comfort, and the appearance of the bridge,
- measures that are planned to ensure that these requirements are met,
- the main assumptions behind the calculations.

In some countries the ***planned service life*** is related to the time needed to amortise the construction costs. Defining this life for a bridge determines the anticipated volume of traffic, which is needed when checking fatigue resistance. The planned service life is not the same as the length of guarantee, which is a legal requirement. However, it is of primary importance, because it also serves as a basis for planning measures such as periodic inspections, maintenance, and replacing of parts of the bridge, as needed.

As far as serviceability is concerned, the basis of design includes above all a synthesis of the ***requirements for use*** and the measures to take in order to satisfy the various needs of the client's representative. For bridges these primarily concern the appearance, the waterproofing of the deck, the resistance to ice and corrosion, as well as the resistance to wear and tear under normal conditions of use. They also concern the functioning of any mechanical components. Specific requirements may be defined by the client's representative regarding deflections of the structure, concrete crack widths, and vibrations of the structure. In the absence of more specific information, the values recommended in codes and standards apply.

The uses that are agreed with the client's representative are translated into specific ***states of use*** in the basis of design. The *states of use* represent different situations in which serviceability must be guaranteed, covering situations that are *anticipated* during the planned service life of the bridge. In order to identify the different *states of use* to be considered, the actions to which the structure will be subjected must be analysed, and their possible effects on the structure must be investigated. During this analysis it is particularly important to understand what specific requirements must be satisfied to guarantee serviceability. Having done this, it is relatively easy to focus attention on appropriate measures. The establishment of the states of use, and their detailed consideration by the engineer, therefore constitute the basis on which the measures to assure serviceability are defined.

The ***measures*** to be taken to guarantee serviceability may be:

- choosing appropriate materials (Sect. 4.5),
- careful choice of construction details (Chap. 6),

- checking by calculation (Sect. 9.5),
- execution that is carried out carefully and in agreement with the drawings, etc.,
- appropriate inspection and maintenance.

In many cases, careful detailing and an appropriate choice of materials are sufficient to guarantee serviceability of a bridge. Also, checking by calculations is only one measure amongst several.

Structural Safety
The amount of consideration given to structural safety depends mainly on the intended uses and importance of the bridge, as well as its positioning in the local environment. The results of these considerations, as described in the basis of design, contain guidance on the following:

- the hazard scenarios,
- measures to be taken to guarantee structural safety,
- the ground conditions to be taken into consideration,
- the main calculation assumptions, concerning modelling of the load carrying structure,
- the accepted risks.

The ***hazard scenarios*** are derived from an analysis of *extreme* situations that may apply to a load carrying structure and cover one or more of the following:

- *actions*: deviations from the values that have been considered, exceptional actions due to the location of the bridge in its surrounding environment, ground behaviour,
- *resistance*: deviations from the values considered, effects of fatigue and corrosion, overall stability (overturning, uplift, sliding).

An appreciation of the risks associated with different actions and resistances, and the possibility of the actions occurring concurrently, lies at the heart of safety considerations. The engineer should take into account all the extreme situations that could occur, considering both the structure itself and the numerous actions, as a result of the functions of the bridge and its position within the local environment. His analysis should cover both the execution of the bridge and its use in service. He should envisage, list, and understand all the risks to which the bridge may be exposed, but only retain those that are significant. The outcome of this work will comprise a number of hazard scenarios, each of which will be characterised by a preponderant risk. The establishment of the hazard scenarios and their detailed consideration by the engineer constitute the basis for defining the structural safety measures.

Placed into four categories, the ***measures*** to be taken comprise:

- control the cause of the risk (for example by elimination, reduction, or reducing its impact),
- plan for inspections, controlling, alarm systems,
- design the structure to have sufficient load carrying resistance (Sect. 9.6),
- accept a given risk (impact by an aeroplane, for example).

The process of calculation and design may identify the need to reconsider some of the measures initially planned, or even the need to complement them with additional measures. Design calculations are only one of numerous measures available to guarantee structural safety.

In addition to identifying the hazards and the measures to be taken, the following items normally figure in the basis of design:

- the actions that have been considered and/or fixed (for example, traffic loads or wind),

- the material properties that have been taken into account and/or prescribed,
- some numerical results, or other values, that are of particular use during the design process or for the transfer of information (for example support reactions onto the foundations).

9.3.3 Conceptual Design

The conceptual design is the first phase of the elaboration of a project (Fig. 9.1). The main aims of this phase are to:

- choose the type of load carrying structure (beam, arch or supported by cables) (Sect. 5.3), its structural form (§ 5.3.4 and § 5.4.2), and the cross section type (Sect. 5.5),
- choose those principal dimensions of the bridge that are not defined as part of the client's requirements (for example the number of spans, the length of the spans, the depth of the deck or the height of the pylons),
- choose the structural materials and their properties (Sect. 4.5),
- define the principal construction details (Chap. 6),
- choose the erection method (Chap. 7).

Taken together these various choices form the *structural concept* (Fig. 4.1), and can also be compiled in terms of *preliminary studies* (§ 4.2.2). The chosen structural concept does not represent a unique solution to the problem, but rather the solution that is deemed to be the best from amongst the *possible solutions* that the project authors have considered.

Conceptual design is an iterative design process, which progresses according to subjective ideas and decisions that are based on the experiences and creativity of the project authors. For example, the span lengths and dimensions of the deck are often based on the slenderness of existing beams (§ 5.3.2), or they come from the results of simple preliminary design calculations.

As far as bridges are concerned, the final choice of the structural concept is normally based on an analysis of all the possible solutions, considering multiple criteria. In the case of important bridges, these solutions often come from different design teams, and are judged as part of a competition. Comparison of the various solutions will normally take into account the following:

- the feasibility of the proposed solution, and its compliance with the defined requirements (§ 4.4.1 and § 4.4.2),
- durability (§ 4.4.3),
- appearance, and the integration of the bridge into its surroundings (§ 4.4.4),
- aspects associated with maintenance,
- the construction deadlines and the costs (§ 4.4.5).

9.3.4 Structural Analysis

The structural analysis and structural design together form that part of the process dedicated to calculations used to define the load carrying structure. The structural analysis (Fig. 9.2) allows the behaviour of the load carrying structure, when subjected to various actions, to be predicted by means of a ***structural model*** (Sect. 11.2). This model translates the reality of the structure, which can have varying degrees of complexity, into a physical model that reflects the input geometry, material properties, and foundation soil conditions. The model used must be appropriate for the complexity of the real situation, so that the structural behaviour is correctly predicted, and all those effects that may influence this behaviour are included (for example a bar model or a model based on finite elements).

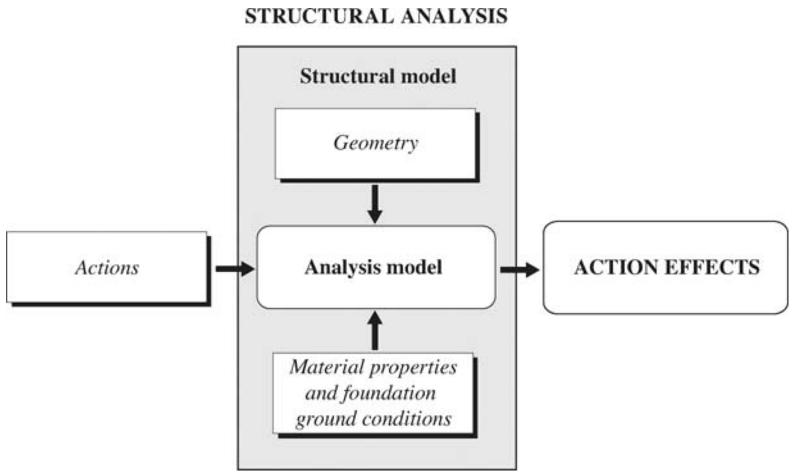

Fig. 9.2 Schematic representation of the structural analysis.

The ***action effects***, such as the internal moments and forces, the stresses, deformations and displacements, represent the response of the structure and are defined in the results of the structural analysis. They are calculated using a model that takes into account the material behaviour and respects equilibrium and kinematics. This ***analysis model*** may assume elastic or elasto-plastic behaviour of a structure that is assumed either not to deform, or to deform, under loading (these are known as first and second order analyses, respectively). The choice of the analysis model depends on the complexity of the structure and the desired accuracy of the results.

9.3.5 Structural Design

The structural design (Fig. 9.3) both determines the required sizes and material properties of the load carrying elements and refines and completes the preliminary construction detailing. It is one of the measures needed to guarantee the serviceability and structural safety of the bridge, based on the study of the states of use and hazard scenarios.

Structural design is carried out for the serviceability and ultimate limit states taking into account appropriate ***design criteria***. It determines the sizes of the load carrying elements by means of ***checks*** against serviceability (Sect. 9.5) and structural safety (Sect. 9.6) requirements. This is achieved, for example, by comparing the action effects with the allowable deflections or resistance of the structural elements. These checks consider ***design values*** for both actions and resistances. These values are based on the concept of ***partial factors***, which differ according to whether serviceability or ultimate limit states are being considered.

When carrying out the various checks, the action effects are grouped together within ***load cases***. Each case is characterised by a ***leading action*** and ***accompanying variable actions***, which are factored to be compatible with the leading action.

If calculations are the means of determining the adequacy of each of the structural elements in terms of the states of use and hazard scenarios, then for each limit state, and for each design criterion, a corresponding load case must be defined. The definition of these combined action effects is a result of consideration of all those actions that might act on a given element at any given time.

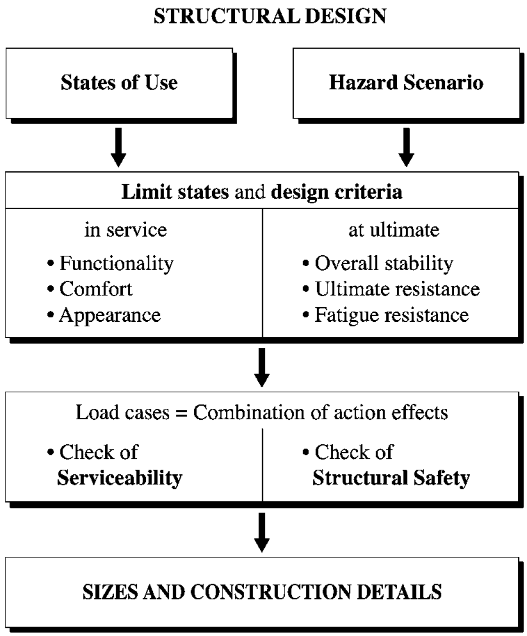

Fig. 9.3 Schematic representation of structural design.

Load Cases

In order to define a load case for a *serviceability limit state*, it is necessary, for the *specific state of use*, to consider the *probable actions in service* and distinguish between those that are permanent and those that are variable. For the variable actions, depending on their magnitude, a distinction may be made between those that will be rare and those that will be frequent in service. The definition of the load case will be a function of these consideration plus the *requirements in service* that need to be checked. For example, for the bridge to maintain an acceptable appearance during service, one might need to check the deflections of the beams solely under permanent loading. To guarantee the comfort of users, the designer may need to check deflections, or a criterion associated with dynamic response, solely under the rare or frequent traffic loads. For this case the permanent loads play no part.

When considering *structural safety* for the *hazard scenario*, the load case will be characterised by its *leading action*, by one or more *variable actions*, which are known as accompanying, and by *permanent actions* that are always present. The leading action is considered in the load case with its maximum value and most unfavourable effect, as far as the load carrying structure, or element, is concerned. The following principle is used to identify the accompanying actions: when an action is considered to be leading, which means it acts with extreme intensity on the structure, the other variable actions that are present will undoubtedly have intensities well below their maximum values. This principle is illustrated in Figure 2.9 of TGC Volume 10, with an example of the possible variations in time of wind and snow actions. For a bridge, an analogous situation is the simultaneous presence of traffic loads and the effects of wind and snow. Typically, snow is not considered as an accompanying action for a bridge, because while the bridge is open to traffic, the snow is cleared away.

9.4 Actions

Generally speaking an *action* may be mechanical (concentrated or distributed), physical (temperature, moisture, shrinkage), or chemical (salt, alkaline solutions). The response of the structure to these actions is expressed in terms of the *action effects*. These effects are internal moments and forces (normal force, bending moment, torsion, shear force), stresses, support reactions, deformations, displacements or a range of other physical and chemical reactions.

As far as structural design is concerned, an action F may take several values. The main distinction to be made is between the representative and design values. The ***representative value*** F_{rep} is normally conservative and based on a statistical evaluation, for example the characteristic value Q_k of a variable action corresponds to a given fractile value or a defined probability of occurrence. The representative value is sometimes based on a nominal value when there is insufficient data for statistical analysis, or on an average value when the magnitude of the action varies very little with space and time. This is the case for self-weight expressed by G_k.

The ***design value*** F_d is the numerical value that is used to calculate the action effects when carrying out the structural analysis and structural design. A different design value is used for calculations at the serviceability and ultimate limit states. Design values take into account aspects of variability and simplified modelling, and are normally determined by multiplying the characteristic value by a load factor.

Generally the design value of an action effect E_d is determined as follows, assuming the actions and their effects behave linearly:

$$E_d = E\{F_d, a_d\} = E\{\gamma_F F_{rep}, a_d\} \tag{9.1}$$

F_{rep}	:	representative value of an action (G_k for permanent loads and Q_k for variable loads)
γ_F	:	load factor
a_d	:	design value of a geometrical variable (dimension of a section, span etc.), normally taken as the nominal value given on the drawings.

The ***load factor*** γ_F depends on the type of action (self-weight, variable action) and the type of check (serviceability, structural safety). It takes into account both uncertainties in the numerical value of the action, and uncertainties in how the model predicts the effects of the action. Load factors are defined in Sections 9.5 and 9.6.

When identifying and defining those load cases needed for the design, three types of action are distinguished according to their nature:

- *Permanent actions G,* which normally act throughout the life of the structure. This category mainly comprises the self-weights of both the structural and non-structural elements (edge beams, surfacing, ballast). Any prestressing force P, which acts on a structural element after tensioning but then throughout the bridge life is normally accounted for in the resistance calculations. However, the local effects of prestressing, such as the introduction or deviation of the force, should be considered as permanent actions.
- *Variable actions Q* (actions due to traffic and climatic actions), which may have an intensity that varies substantially with time.
- *Accidental actions A* (seismic actions, impact or derailment), which have a very high intensity but only act over a very short period of time and have a low probability of occurring. Accidental actions are often given in codes and standards directly with their design value A_d.

During checks of either serviceability or structural safety, the different actions are combined into load cases that correspond to the different states of use and hazard scenarios. The combinations take into account the different limit states and design criteria to be checked. The checks normally consider the action effects. Because the load cases must take into account the reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions, their design values are multiplied by reduction factors. The definition of actions and their numerical values for bridges are considered in Chapter 10.

9.5 Verification of the Serviceability Limit States (SLS)

9.5.1 Principles

For those states of use that must be checked by calculation, it is necessary to show that the load carrying structure behaves in a way that is within the limits defined by the client's representative or specified in codes and standards. The checks cover the ***serviceability limit states,*** which concern:

- the functionality of the load carrying structure, in other words its ability to satisfy the desired requirements (for example deflections within acceptable limits in the case of bridges),
- the comfort of users (for example avoidance of vibrations of the structure which could create unwanted physiological effects),
- the appearance of the bridge (for example avoidance of excessive cracking of the slab for composite bridges, or deflections due to self-weight).

Generally speaking, for each of these different limit states, checking the serviceability comprises demonstrating that the following *design criterion* is satisfied:

$$E_d \leq C_d \tag{9.2}$$

E_d : design value of the action effect for the serviceability limit state under consideration
C_d : corresponding serviceability limit, defined either in the basis of design or in a code or standard.

For checking serviceability, the design value of the action effect E_d in equation (9.1) is determined with a load factor of unity:

$$\gamma_F = 1.0 \tag{9.3}$$

The design value of the action effect E_d is determined for a load case that comprises either a single action or a combination of several actions that may occur simultaneously, according to the serviceability limit state being considered.

9.5.2 Load Cases

To carry out a check according to equation (9.2), the standard SIA 260[1] distinguishes three types of load case as a function of the duration of application of the variable actions: *rare*, *frequent* and *quasi-permanent*

[1] In most cases, the standard SIA 160 is identical, or at least similar in its principles, to the Eurocode "Basis of design", except for the serviceability checks, where the SIA standard relies on the intelligent application of ideas by the designer.

(TGC Vol. 10, § 2.6.2, TGC Vol. 11, § 6.2.3). The reduction coefficients ψ that are associated with the representative values of the variable actions allow their different numerical values to be characterised. The type of load case and the type of action (permanent or variable) to be taken into account in each particular case depend on the serviceability limit state that is to be satisfied and on the consequences of the action effects. The variable action effects can be reversible or irreversible.

Considering road bridges as an example for the serviceability limit state concerned with user comfort, the design criterion relates to limiting the deflections w_{31} under the frequent (second subscript = 1) variable (first subscript = 3) load case:

$$w_{31}(\psi_1 Q_{k1}) \leq \frac{l}{500} \tag{9.4}$$

w_{31}	:	deflection under traffic loading
ψ_1	:	reduction factor applied to the representative value for traffic loading for the frequent load case ($\psi_1 = 0.75$ according to table 6 of annex B of the standard SIA 260)
Q_{k1}	:	characteristic value of the traffic action, load model 1 (§ 10.3.1)
l	:	span being considered

For this serviceability limit state considering user comfort, it is necessary to check the reversible deflections that occur due to the frequent load case concerning traffic. Checking the serviceability limit state considering appearance that concerns the permanent condition of the bridge, the quasi-permanent load case is applied when checking for the reversible deflections:

$$w_2(G_k) \leq \frac{l}{700} - w_0 \tag{9.5}$$

w_2	:	long term deflection under the effect of the permanent actions (self-weights of the structural and non-structural elements), including shrinkage and creep
G_k	:	characteristic value of the corresponding permanent actions
w_0	:	precamber of the beam

There are no variable actions included in equation (9.5) because, for this limit state, which concerns the appearance of the bridge, only the permanent actions have a long term effect. The SIA standard assumes that there will be no traffic permanently present on the bridge, so $\psi = 0$ (from table 6 of annex B of SIA 260). The precamber w_0 is normally calculated considering the permanent loads and a small part of the traffic loads (to be agreed with the client's representative), so that in the permanent state, the bridge exhibits a small upwards camber, which is deemed to be aesthetically pleasing.

In its annexes B and E the standard SIA 260 details the types of load case to consider as well as the different reduction factors ψ. Nevertheless, each specific serviceability limit state must be discussed and agreed with the client's representative during the development of the client's requirements.

9.5.3 Serviceability Limits

In order to carry out a check according to (9.2), the serviceability limit C_d must be defined as a function of the serviceability limit state under consideration. Limits are generally given in codes and standards as a function of the different criteria to be checked. For bridges, SIA 260 recommends limiting values for deflections and vibrations. However, it is important to add that these are indicative values, which should only be used when no other limits have been agreed with the client's representative and noted in the client's requirements.

Deflections

By way of an example, indicative deflection limits, as proposed in the SIA standards, are summarised in Table 9.4. The variable l denotes the span, or double the length of a cantilever. For railway bridges deflection limits to guarantee the comfort of the users depends on the span and the speed of the train. These indicative values concern situations where the action effects have reversible consequences.

Table 9.4 Indicative limiting values for deflections, and corresponding load cases (SIA standards).

Bridge Type	Serviceability Limit State	Load Case	
		frequent	quasi permanent
Road bridge	Functionality – vertical deflection in relation with the road joint (expansion joint)	5 mm[a]	
	Comfort	$l/500$[b]	
	Appearance		$l/700$[a]
Railway bridge	Functionality (ballasted tracks) – deflection in span $v < 80$ km/h 80 km/h $\leq v \leq$ 200 km/h	 $l/800$[c] $l/(15v\text{-}400)$[c]	
	– twist of the railway track (Fig. 16.12) $v \leq 120$ km/h $v > 120$ km/h	 1.0 mrad/m[c] 0.7 mrad/m[c]	
	– vertical deflection of the upper surface of the deck in relation with a neighbouring structure (abutment or other deck) $v \leq 160$ km/h $v > 160$ km/h	 3 mm[c] 2 mm[c]	
	Appearance		$l/700$[a]
Footbridge	Functionality – vertical deflection in relation with the road joint (expansion joint) – deflection in span	 5 mm[a]	 $l/700$[a]
	Comfort	$l/600$[b]	
	Appearance		$l/700$[a]

[a] deflection after the deduction of a possible precamber but including the effects of creep and shrinkage

[b] deflection due to load model 1

[c] deflection due to load model 1 and if necessary due to load model 2 (up to two loaded tracks, characteristic values including the dynamic amplification)

Vibrations

Bridges are subjected to dynamic actions (traffic, wind), and therefore are susceptible to ***vibrations***. Such vibrations may limit the use of the bridge or have a negative effect on the comfort of the bridge users. The dynamic behaviour of a bridge is not an easy subject to study, because parameters associated with the interaction between the dynamic actions and the response of the structure come into play. However, the question of dynamics must be carefully considered for bridges that are sensitive to oscillations induced by the wind, such as cable stayed and suspension bridges, or specific types of bridge such as those for high speed trains or for pedestrians.

Bridges designed for exclusive use by pedestrians warrant specific attention because they are normally lightweight, and so can be set in motion by users or the wind. The standard SIA 260 notes that for such bridges, natural frequencies between 1.6 Hz and 4.5 Hz should be avoided for vertical oscillations, as should natural frequencies less than 1.3 Hz for transverse horizontal oscillations. This subject is considered in more detail in Section 17.4.

Vibrations of a bridge can lead to resonance if the frequency of a repetitive variable action corresponds to one or other of the natural frequencies of the bridge. Resonance can lead to failure of some of the structural members, or even collapse of the bridge, and should be analysed and included in the checks for structural safety.

9.6 Verification of the Ultimate Limit States (ULS)

9.6.1 Principles

Checking structural safety by means of calculations must be carried out for each of the relevant hazard scenarios. That comprises comparing the design values of the action effects with design values for the resistance of the load carrying elements. For this check the standard SIA 260 distinguishes between four ***ultimate limit states***:

- Type 1: overall stability of the structure (sliding, overturning, uplift).
- Type 2: ultimate resistance of the load carrying structure or one of its elements (cross section resistance, instability or formation of a mechanism).
- Type 3: ultimate resistance of the foundation soil (sliding of the ground, failure of an embankment, soil failure).
- Type 4: fatigue resistance of the load carrying structure or one of its members.

For the ultimate limit state type 1, checking structural safety is based on showing that the following *design criterion* is satisfied:

$$E_{d,\,dst} \leq E_{d,\,stb} \tag{9.6}$$

$E_{d,dst}$: design value of destabilising action effects
$E_{d,stb}$: design value of stabilising action effects

For ultimate limit states type 2 and 3, structural safety is assured when the following *design criterion* is satisfied:

$$E_d \leq R_d \tag{9.7}$$

E_d : design value of the action effects
R_d : design value of the ultimate resistance (§ 9.6.3)

The design value of the action effects E_d is derived from a load case comprising a combination of several actions that may act simultaneously, which differ according to the ultimate limit state and the corresponding design criterion to be satisfied.

9.6.2 Load Cases

Limit State Type 1

For this ultimate limit state, it is necessary to define a load case for each bridge or part of the bridge. The designer must distinguish between stabilising and destabilising actions, and place variable actions in their most unfavourable position according to the given stability case. Actions are present on both sides of the inequality (9.6) with their design values, but with load factors that differ depending on whether the action is stabilising or destabilising. These load factors are given in standard SIA 260, of which an extract is given in Table 9.5. When self-weight is at the same time both stabilising and destabilising, it is nevertheless considered with the same load factor. This situation arises, for example, with the self-weight of a continuous beam, when checking the stability of the beam against uplift at a support. The self-weight is not considered with load factors of $\gamma_{G,sup}$ and $\gamma_{G,inf}$ according to whether the weight is favourable or unfavourable to uplift, but rather one or the other of these factors is simply applied to the total self-weight.

Limit States Type 2 and 3

For these limit states the design value of the action effects may be defined by the following general expression:

$$E_d = E\{\gamma_G G_k, \gamma_{Q1} Q_{k1}, \psi_{0i} Q_{ki}, a_d\} \tag{9.8}$$

γ_G : load factor for the permanent actions
γ_{Q1} : load factor for the leading variable action
ψ_{0i} : reduction factor for the accompanying variable action Q_{ki} (rare value hence subscript 0 of a variable action i)
a_d : design value of a geometrical variable (cross section dimensions, span, etc), generally taken as the nominal value given on the drawings

The considerations noted in Paragraph 9.3.5 concerning the sizing of a given structural member allow the load case corresponding to expression (9.8) to be identified. It is normally sufficient to only take into

Table 9.5 Load factors γ_F for type 1 and 2 ultimate limit states (SIA 260).

Actions	γ_F	Ultimate Limit State	
		Type 1	Type 2
Permanent Actions			
– unfavorable	$\gamma_{G,sup}$	1.10	1.35
– favorable	$\gamma_{G,inf}$	0.90	0.80
Variable Actions			
– in general	γ_Q	1.50	
– railway traffic	γ_Q	1.45	

account one accompanying variable action together with the leading action. It is very rare for several independent accompanying actions to act in a significant way at the same time as the leading action occurs at its maximum value and in its most unfavourable position.

The load factors γ_G and γ_Q depend on the type of action and the type of ultimate limit state. Table 9.5 summarises the numerical values of these load factors for checking ultimate limit states type 1 and 2, taken from the standard SIA 260. This standard also proposes load factors for ultimate limit state type 3 (foundation soil). For the permanent actions, G_k is multiplied either by $\gamma_{G,sup}$ or by $\gamma_{G,inf}$, depending on whether the effect of the whole is unfavourable or favourable. The reduction factors ψ_{0i} are given, for bridges, in annexes B and E of the standard SIA260.

Accidental Actions

For the limit state with *accidental* actions, which is a particular case of ultimate limit states type 1 to 3, the design value of the action effects is expressed by the following relationship:

$$E_d = E\{G_k, A_d, \psi_{2i}Q_{ki}, a_d\} \tag{9.9}$$

A_d : design value of the accidental action

ψ_{2i} : reduction factor for the variable action Q_{ki} that accompanies the accidental action (quasi-permanent action hence subscript 2 and variable action i)

Limit State Type 4

For this limit state that concerns *fatigue*, guidance on the definition of load cases to consider is given in the standard SIA 263, as well as Section 12.7 of this book.

9.6.3 Design Resistance

The following expression is generally used in the SIA standards to calculate the design value of resistance R_d:

$$R_d = \frac{\eta R_k}{\gamma_M} \tag{9.10}$$

with the ***resistance factor*** γ_M expressed as:

$$\gamma_M = \gamma_R \gamma_m \tag{9.11}$$

γ_R : partial factor taking into account uncertainties in the resistance model

γ_m : partial factor for a structural material property, taking into account unfavourable deviations from the characteristic value

η : correction factor inherent to the material (for example for the resistance or the elastic modulus of concrete)

The ultimate resistance R (either in the form of its characteristic value R_k or design value R_d) may be determined according to the rules given in Chapter 12 onwards of this book, or those given in the various standards covering structures. Checking of structural safety according to expression (9.7) may be carried out in terms of the collapse load of a mechanism (q), the internal moments and forces (M, T, N, V), or the stresses (σ, τ), according to the analysis model that is used. The different structural codes and standards also define values for the resistance factor γ_M. In the SIA standards, and generally in the Eurocodes, these values are as follows:

- γ_{M0} = 1.05 for cross section resistance of steel structures,
- γ_{M1} = 1.05 for the stability of steel structures,
- γ_{M2} = 1.25 for the resistance of joints and net cross sections,
- γ_{M0} = 1.50 for concrete properties,
- γ_{M0} = 1.15 for reinforcing steel properties.

For the sake of simplicity, a number of the SIA standards do not differentiate between γ_{M0} and γ_{M1}, leading to resistance factors with the following notation, which is adopted for the rest of this book:

- γ_a = 1.05 for structural steel,
- γ_{ap} = 1.05 for steel used in profiled decking,
- γ_c = 1.50 for concrete,
- γ_s = 1.15 for reinforcing steel,
- γ_v = 1.25 for connection elements,
- γ_{Mf} = 1.00 to 1.35 for fatigue.

Fatigue resistance is determined according to the concept that is described in detail in TGC Volume 10, Chapter 13. The application of this concept is given in Section 12.7 of this book.

In conclusion, one should remember that checking structural safety is not only needed for the *structure in its final state*, throughout the planned service life, but also for all the *construction phases*. The likelihood of either partial or total collapse of the structure is greatest during construction, because some very unfavourable load cases are often present. Also, the structural form may be very different from that of the final structure.

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10 Loads and Actions

Load case: traffic as the leading action.

10.1 Introduction

The chapter describes the loads and actions that should be considered when designing a bridge. In most countries these loads and actions are defined in the relevant codes and standards, for example in Switzerland in standard SIA 261. This chapter does not aim to faithfully reproduce the contents of that standard, but rather presents complementary information concerning the loads and actions to take into account when applying the principles of design. The following loads and actions are described:

- Section 10.2: Permanent loads and long term effects,
- Section 10.3: Traffic loads,
- Section 10.4: Climatic actions,
- Section 10.5: Actions during construction,
- Section 10.6: Accidental actions,
- Section 10.7: Frictional and restraint forces from bearings.

Additional, and often more detailed, information on actions may be found in the Eurocodes, particularly EN 1991 "Actions on structures", in Part 2 concerning traffic loads on bridges. The characteristic values of loads given in these documents should normally only be used in combination with the corresponding load factors, as well as with the resistance models and factors given in the Eurocodes.

When assessing an existing bridge, one may use reduced traffic loads and even a reduced self-weight, given the better knowledge of the actions affecting the bridge and the role it plays as part of a communication route. Detailed guidance on the assessment of existing bridges may be found in [10.1]. A worked example showing the numerical application of the actions when designing a bridge is given in Chapter 19.

10.2 Permanent Loads and Long Term Effects

10.2.1 Self-weight of the Structure

The self-weight of the load carrying structure is represented by its characteristic value G_k, which is determined by considering the dimensions given on the drawings and the average densities of the materials

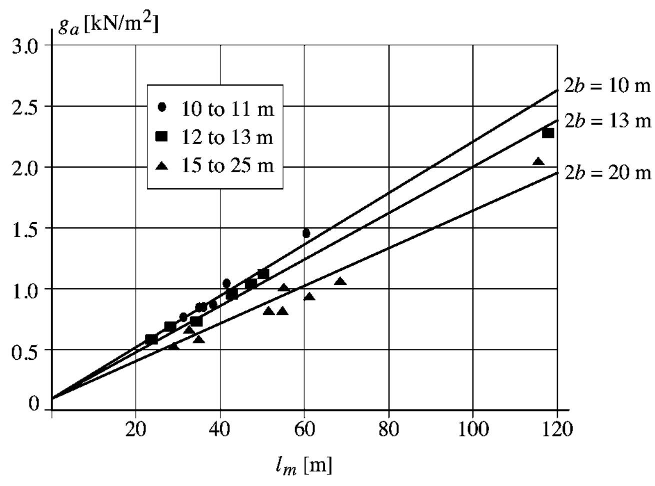


Fig. 10.1 Estimate of the self-weight of the steelwork of a twin girder composite bridge.

used. Because this numerical value is not known in advance, it must be estimated at the start of the structural design. For steel bridges the self-weight is mainly due to the steelwork; for composite bridges the self-weight of the concrete slab is significant and must be added.

The estimated self-weight of the main beams either is based on the experience of the designer or is given by empirical rules. This estimate must be checked and, if need be, adjusted as the design progresses. A statistical analysis of the self-weight of bridge steelwork (main beams, stiffeners, cross bracings and plan bracing) has enabled a relationship (10.1) to be established between the average length of the spans l_m and the weight g_a of the steel as a function of the deck width. This empirical relationship is based on the study of around thirty twin girder composite bridges constructed in Switzerland. Figure 10.1 shows graphically the relationship for three slab widths $2b$, and includes specific points for the bridges that were included in the study.

As an example, the steelwork of a twin girder composite bridge with a 13 m wide slab and 50 m spans weighs approximately 1.05 kN/m^2, or approximately 13 kN/m for the two girders. By comparison, the self-weight of the concrete slab of such a bridge, with an average thickness of 300 mm, weighs around 100 kN/m, and this varies very little with the spans:

$$g_a = 0.1 + \frac{0.02 l_m}{0.6 + 0.035(2b)} \tag{10.1}$$

g_a	:	self weight of the steelwork in kN/m^2
$2b$	:	slab width in m
l_m	:	average span length, $l_m = \Sigma\, l_i^2 / l_{tot}$ in m
l_i	:	length of span i in m ($i = 1 \ldots n$)
l_{tot}	:	total length of the bridge in m, $l_{tot} = \Sigma\, l_i$

Equation (10.1) gives good results for twin girder bridges that are either straight or slightly curved, and with spans less than 60 m. For longer spans the predicted value of g_a may be reduced by around 5%. For box girder and multi girder bridges, as well as those that are curved, equation (10.1) may still be used, but its accuracy will vary considerably from case to case. The designer must remember that the estimate of self-weight that is made as part of the preliminary studies should be checked and, if necessary, modified as the design progresses.

The self-weight of the load carrying structure should be applied with a constant load factor and independent of the influence line. Either an upper or a lower value is applied to the whole self-weight depending on whether, as a whole, it is unfavourable or favourable for the specific check being carried out. Table 9.5 gives load factors to apply for limit state type 1 (overall stability: sliding, overturning or uplift) and type 2 (ultimate resistance of the load carrying structure).

For beams of varying depth, the distribution of self-weight can vary substantially along the span, and the distribution of the steel must be taken into account when determining the internal moments and forces and the support reactions. During erection the self-weight of the structure, and particularly its distribution, must be carefully evaluated because it is the only action present, other than wind and anything due to erection equipment.

10.2.2 Self-weight of the Non-structural Elements

The self-weight of the non-structural elements of a bridge generally comprises that of any fixed components, such as: road surfacing, footpaths, slab edges, parapets, crash barriers, ducts or pipework, noise barriers, or systems for lighting or signalling.

The characteristic value g_k of the self-weight of non-structural elements is given by the product of their theoretical dimensions and their average density, or may be defined by the supplier. The following may be considered as indicative values:

- A surfacing thickness of 10 cm (which is typical in Switzerland) equates to a load of 2.4 kN/m^2. The designer may either plan measures to ensure that the thickness is not changed over the life of the bridge, or make an allowance in the design calculations for the extra weight of an increased thickness applied during the life.
- For a heavy concrete parapet of the type shown in Figure 8.6, the self-weight is around 10 kN/m.
- When heavy concrete parapets are not required, steel crash barriers should be specified, with a self-weight of around 1 kN/m.
- The weight of ducts and pipework is normally insignificant. Except in unusual cases the weight of water in the ducts may also be neglected.

10.2.3 Creep, Shrinkage and Prestress

For steel-concrete composite bridges the effects of shrinkage, creep and, where used, prestressing should be taken into account as soon as the reinforced or prestressed concrete slab is structurally connected to the steel structure. The effects of shrinkage and creep are considered in Section 13.2, which covers those specific actions that are particular to composite bridges. Introduction of prestress into the steel beams is considered in Paragraph 13.5.4, while losses of prestress were considered in Section 8.6.

10.2.4 Support Settlements

Whether or not support settlements need to be taken into account when designing for structural safety depends on the structural form and method of analysis that are adopted. If the form is statically determinate, then settlements have no effect on the internal moments and forces, thus on checking for structural safety. If the structural form is redundant, then each section must be able to resist the supplementary moments and forces to which it is subjected. This means that when an elastic analysis is used to determine the internal moments and forces, which is the most common approach, the settlements should be taken into account.

Support settlements should always be taken into account when checking serviceability. They must not exceed a certain limiting value, so that the bridge elevation is not excessively distorted; this limit is set by the designer either independently or in agreement with the client's representative.

So that any structurally or aesthetically unacceptable support settlement can be corrected, the designer should plan details that will allow the bearings to be moved back into their original positions. Such detailing is discussed in Section 6.4. These details also allow any malfunctioning bearings to be replaced.

10.2.5 Ground and Water Actions

Horizontal ground and water pressures must be taken into account when designing the foundations and abutments. They may be governing, particularly when checking the stability of these elements. The rules of soil mechanics allow the magnitude of the pressure to be determined. Standard SIA 261 dedicates a whole chapter to the calculation of the effects of the foundation ground on the structure. Appropriate load factors are defined in the standard SIA 260.

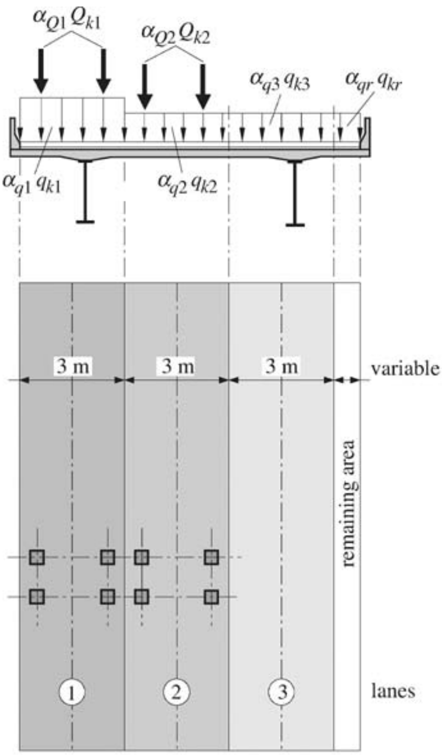
10.3 Traffic Loads

Depending on its intended use, a bridge supports either road traffic, railway traffic, pedestrians or a combination of these. The variable loads and actions corresponding to these uses are described below. Imposed loads for bridges supporting pipes, water ducts, gas or electricity lines are not covered in this section. For these cases the designer should refer to the relevant authority for the definition of such loads.

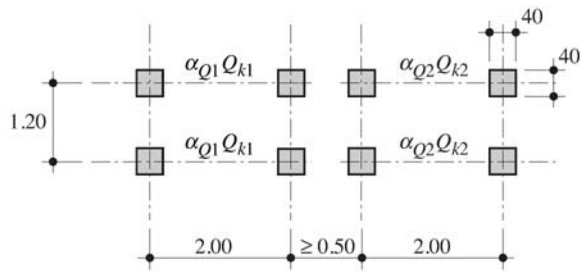
10.3.1 Road Bridges

Road traffic passing over a bridge results in vertical loads and horizontal forces being applied to the structure. The characteristic values for the different loads are based on a combination of traffic measurements and numerical simulations. In the standard SIA 261, the real traffic on a bridge is represented by load model 1, which has the same effect on the structure as the real traffic. This model defines the geometry of two groups of two axles to represent the concentrated traffic loads, and defines different surfaces that are uniformly loaded to represent traffic. Load model 1 is illustrated in Figure 10.2.

When considering this model, the portion of the deck surface that is accessible to traffic is divided into *notional lanes* onto which the uniformly distributed and concentrated loads are applied. Each of the notional lanes 1 and 2 are subject to a distributed load and a group of two axles. Lane 1 is the most heavily loaded



Model 1



Geometry of groups of axles

Notional lane i	Q_{ki} [kN]	q_{ki} [kN/m^2]
1	300	9.0
2	200	2.5
3	–	2.5
Remaining area (r)	–	2.5

Characteristic values

Fig. 10.2 Traffic load model 1 and numerical values according to standard SIA 261.

(line of trucks), whereas the loads acting on lane 2 represent a mixture of trucks and cars. Notional lane 3 and the remaining surface area are loaded by a uniformly distributed load that essentially represents cars alone.

Division of the Surface into Notional Lanes

The surface is divided into notional lanes that are each 3 m wide. This width determines the number of lanes to be considered. The remainder of the surface is known as *remaining area*. The designer should remember that the deck width to be divided up comprises the total width that is potentially accessible to road traffic. That means any footpaths are part of this width unless they are separated from the traffic lanes by some form of fixed and immovable system. When the total width is between 5.4 m and 6 m, two notional lanes, of equal width, are considered. If the width is less than 5.4 m, then only one single notional lane of 3 m width is considered, the rest being taken as remaining area.

According to Eurocode 1, when a bridge supports two-way traffic with no fixed and permanent separation between the two, then the total width, including that used for the central reservation, should be taken into account when calculating the number of notional lanes. When a bridge supports two-way traffic *with* fixed and permanent separation, or when the accessible surface is in two parts on *two independent decks*, then each of the two separate widths is considered separately when defining the number of notional lanes.

Positioning of Load Model 1

When determining the longitudinal internal moments and forces, the two groups of axles should be centred transversally within the notional lanes. When the loads are to be positioned to analyse their local effects on the slab, each of the two sets of axles may be moved off-centre within their notional lanes. However, a minimum distance of 0.5 m between the two groups must always be respected when the two groups of axles are placed at the same cross section.

Lane 1, which is the most heavily loaded, should be placed in the most unfavourable transverse position for the structural element being considered. Lane 2 is not necessarily placed next to lane 1 transversally. Longitudinally, the loads are placed considering the influence lines of the internal moment or force being considered, to provoke the maximum effect. Figures 11.26 and 11.27 show an example of positioning loads to obtain the maximum bending moment in the case of a composite twin-girder bridge.

According to Eurocode 1, if the deck supports two-way traffic, then unique numbering of the notional lanes should be adopted, even if there is fixed and immovable separation between the two. In other words, there will only be one lane 1, the most heavily loaded, to take into account and placed in the most unfavourable position for the structural member being considered.

If the two directions of traffic are supported on independent decks, then the notional lanes on each deck are numbered and loaded independently from each other. That said, unique numbering of the lanes is defined when designing the piers and abutments if the two decks are supported on a common substructure.

Characteristic Values

Characteristic values of road traffic loads have been determined from an analysis of data taken in several European countries. The values were calculated on the basis of a probability of being exceeded annually of 0.001, and are adjusted using the coefficients α_{Qi} and α_{qi} (Fig. 10.2). These coefficients for the traffic load model are determined nationally. In Switzerland the value of the α_i coefficients is normally 0.9, although they may be reduced to as little as 0.65 for routes of minor importance or when the roadway is narrow. In some particular cases they may also be greater than 0.9 (high proportion of heavy traffic, frequent traffic jams). Any reduction or increase in the α_i coefficients must be agreed with the relevant authority.

When the bridge lies on a communication route that is open to exceptional loads, an additional load model must be considered. This model (model 3) represents heavy convoys, and is described in the standard SIA 261/1. Load model 2, defined in the Eurocodes to represent a very heavy axle load, is not included in the standard SIA 261.

Load model 1 is also used to define *fatigue stresses*, but in this case, it is only necessary to consider the axle loads $\alpha_{Qi}Q_{k1}$ of notional lane 1. This lane is placed transversally to correspond to the effective traffic lane. The axles are placed longitudinally on the bridge so that the stress range, for the cross section being considered, is at a maximum. The Eurocode proposes other traffic load models that may be considered for more detailed checks of fatigue safety. When there is two-way traffic, or for bridges with several separate lanes, the effect of simultaneous load in several lanes acting on the structure is considered using the factor λ_4 (§ 12.7.2).

Dynamic Amplification

The dynamic behaviour of a bridge is very complex. It is influenced by numerous parameters, amongst which are the dynamic characteristics of the structure (natural frequency and damping), the surface roughness, the traffic characteristics (geometry, distribution of loads, etc.), the characteristics of the vehicles (suspension, damping, natural frequencies, etc.), and the speed of the traffic. Normally, it is impossible to introduce all these parameters into the analysis of the structure. For simplicity the dynamic effects of the traffic loads are allowed for by applying a dynamic amplification to the elastic response. This amplification is not necessarily the same when checking structural safety as it is when checking fatigue resistance. Its value may also be modified if the designer has good knowledge of the traffic characteristics and of the role of the bridge in its communication route, as is the case when assessing existing structures.

Standard SIA 261, like the Eurocode, implicitly considers dynamic amplification in the characteristic values of axle loads, with a value of around 1.8. However, it does not take into account the more significant dynamic effects in the regions of road and expansion joints. For this reason it is necessary to additionally increase the loads from axle groups using a dynamic amplification $\Phi = 1.3$ within a distance of 3 m from the joint, to calculate the corresponding (local) moments and forces.

For both road and railway bridges, the issue of resonance may be ignored, as the conditions that can lead to such a phenomenon do not normally occur for such structures. On the other hand, they are taken into account for footbridges, which are often lightweight and therefore more susceptible to vibrations (Sect. 17.4).

Horizontal Forces

The horizontal effects of road traffic are the longitudinal forces due to braking and accelerating. The effects due to centrifugal force with curved bridges are normally ignored in the case of road traffic (SIA 261 Art. 10.2.5.1). The forces due to acceleration QA and braking QB of vehicles are proportional to the load magnitude in load model 1 in notional lane 1. The forces act at the level of the road surface and, according to both the SIA standards and the Eurocodes, their characteristic values are determined using the following equation:

$$QA_k = QB_k = 1.2\alpha_{Q1}Q_{k1} + 0.1\alpha_{q1}q_{k1}b_1 l \leq 900 \text{ kN} \tag{10.2}$$

b_1 : width of notional lane 1 (normally 3 m)
l : distance between the expansion joints for the length of structure under consideration

The forces due to impact of vehicles against crash barriers, parapets, or bridge piers are taken as accidental actions and are considered in Paragraph 10.6.2. Structural elements that may be subject to impact forces should be given particular attention in terms of the measures to take to guarantee their structural safety.

10.3.2 Other Types of Bridge

Loads due to railway traffic as well as imposed loads acting on bridges for pedestrians and cyclists are described in the following chapters: railway bridges in Chapter 16, and bridges for pedestrians and cyclists in Chapter 17.

10.4 Climatic Actions

10.4.1 Wind

In the Final State

The wind forces acting on a bridge deck and its piers depend on the form and size of the elements. Determining these forces requires consideration of the dynamic wind pressure, the positioning of the bridge, its geometry and its sensitivity to dynamic effects. Table 63 of standard SIA 261 contains numerical values of force coefficients that allow the different geometries of bridge deck cross sections to be taken into account. It also indicates the positions where the horizontal and vertical wind forces should be applied to the deck. The wind may result in torsion acting on the main beams, depending on the relative positions of the line of application of the wind forces and the centre of rotation of the bridge cross section.

EN 1991 (Eurocode 1), in particular its Part 1-4, contains extremely detailed guidance that encourages the designer to determine (with apparently great precision) the wind forces acting on a bridge. The usefulness and even accuracy of such comprehensive, and therefore long calculations must be questioned. It is almost impossible, given the number of parameters, to model the effect of wind on a bridge in a realistic manner for each particular case. For the vast majority of bridges, wind has only a very limited impact on the sizes of the structural members, in the final state. An estimate of the magnitude of wind actions, such as that proposed in the SIA standard, seems sufficient and appropriate.

For certain bridges, for example those that are high above the ground (> 100 m) or bridges with cables, specific studies of the static and dynamic effects of wind are recommended. Such studies are also recommended in certain situations (for example launching of long span beam bridges) where the local topography can affect the wind conditions.

During Erection

During erection the wind may be a leading action, particularly when the bridge is constructed by launching or incrementally. Because the duration of exposure to the wind during an erection phase is limited – from several hours to several months, the characteristic value of the wind pressure may be reduced, because the probability of an extreme wind occurring is less than when the bridge is completed. Such a reduction is not explicitly recognised in standards, so agreement with the client's representative is absolutely essential. Two cases may be considered:

- *Erection of short duration*. The erection phases under consideration are limited to a few hours duration, and means of intervention to make the structure safe can also be implemented in a few hours (blocking the supports, cable stays to secure cantilevers, etc.). In such cases one can fix, as a function of wind measurements taken for the region, a limiting wind speed, which is used as a design value. Using meteorological information it is normally possible to know 48 hours in advance if conditions will be suitably favourable to authorise the erection. If the predicted wind speed is less than the limiting value, then the erection can go ahead. If the prediction turns out to be inaccurate, then erection can be stopped and the partially erected structure made safe in a few hours. If the predicted wind speed exceeds the limiting value, then the erection can be postponed. Using meteorological information in this way is particularly suitable when launching the steel structure. Clearly, the lower the limiting value of wind speed, the more days when erection is not possible. It is therefore beneficial to find a compromise between the progress of the works and the costs for erection. As an example, for launching the Viaduc de Vaux situated alongside Lake Neuchatel on Freeway N1, the limiting wind speed was fixed at 60 km/h, which corresponds to a dynamic wind pressure of 0.17 kN/m^2 (the minimum reference value according to the standard is 0.9 kN/m^2).

- *Erection of long duration*. Sometimes the duration of the erection phases may be too long – from several weeks to several months – to be covered by meteorological information, or the measures needed to ensure structural safety may take more than a few hours. In such cases a reduced value of wind pressure may be chosen as a function of the probability of the chosen wind speed being exceeded, based on a statistical evaluation of wind speeds measured in the proximity of the bridge.

10.4.2 Temperature

Variations of Temperature

Daily and annual variations of air temperature and degree of exposure to the sun are variable actions that result in more-or-less uniform heating of a structure. The resulting variations in temperature within a bridge depend on its location and orientation, its cross sectional form, its material characteristics and its ventilation. Generally speaking the variation in temperature, independent of the cross section and with particular regard to the deck, may be broken down into the following (Fig. 10.3):

- uniform temperature variation ΔT_1,
- linear temperature variation (gradient) ΔT_2, which may be broken down further into a vertical variation ΔT_{2y} and a horizontal variation ΔT_{2z},
- nonlinear temperature variation.

The *uniform variation* of temperature in a bridge is a result of the annual variation in air temperature. Characteristic values of the uniform temperature variation ΔT_1 are referenced to the average temperature in a given location, which is around 10 °C for the Swiss plane, and are quantified in the standard SIA 261:

- for steel bridges: ± 30 °C
- for composite bridges: ± 25 °C

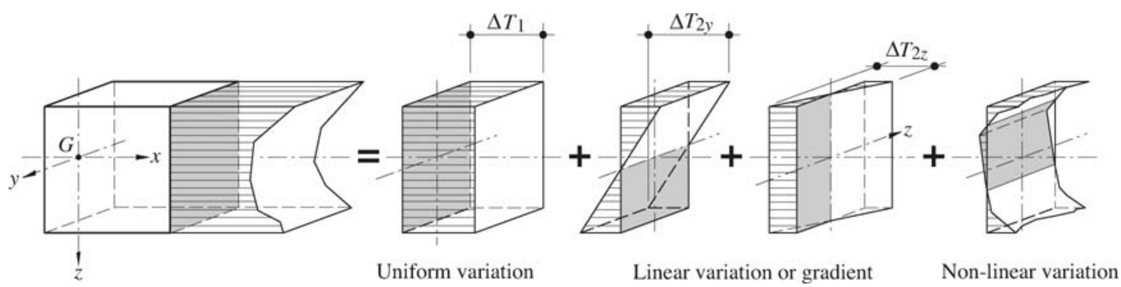


Fig. 10.3 Principle of decomposition of temperature variation in a cross section.

The *linear variation* ΔT_2 that develops in a cross section of the deck is a result of the daily warming and cooling due to varying exposure to the sun. The vertical variation is characterised by the temperature difference between the top and bottom of the cross section (steel bridges) or slab (composite bridges). Standard SIA 261 quantifies characteristic values for ΔT_{2y} as:

- for steel bridges: + 10 °C (upper surface warmer) or –6 °C (upper surface cooler),
- for composite bridges: + 12 °C (upper surface warmer) or –4 °C (upper surface cooler) across the thickness of the concrete slab: the temperature of the steel beams is assumed to be uniform.

Depending on sun exposure, for example for sun shining on one side only, there may be a horizontal variation in temperature ΔT_{2z} across the width of the deck. However, standard SIA 261 does not give any values for this particular situation.

The idea of linear temperature variations and the corresponding numerical values given in standards are only models that are more-or-less representative of reality. In some cases real temperatures measured on a bridge may be considerably different from the values given in the standards. As an example, Figure 10.4 shows the vertical variation in temperature proposed in the SIA standard and measured in-situ, when the temperature gradient in the slab is at its greatest, for a beam cross section in a composite bridge. For this example the difference in stresses acting on the steel cross section of the composite beam is significant. Such differences bring into question the usefulness of calculations based on the codified models, particularly given that such calculations can be quite complex. Further information is given in Paragraph 13.2.3, which considers temperature effects for composite bridges.

Also, the internal effects of temperature, as for all internal effects due to imposed deformations, may be ignored in some cases when checking structural safety. Paragraph 13.4.2 considers this issue in more detail for the analysis of composite beams.

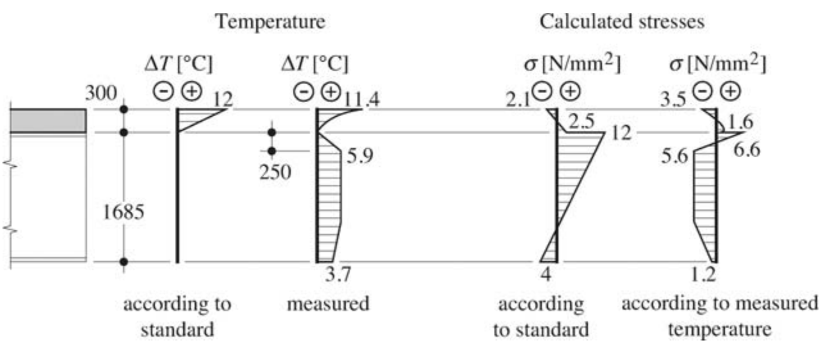


Fig. 10.4 Comparison between a measured temperature gradient ΔT_2 and the gradient proposed in SIA 261, and the corresponding calculated stresses.

Temperature Effects

Materials deform as a result of temperature change. If the load carrying structure is free to move, then overall deformations (lengthening or shortening, curvature) occur. If, on the other hand, the structure is restrained (statically indeterminate, also known as redundant), then thermal stresses will develop, leading to normal forces, bending moments, and shear forces, thus creating stresses across the section.

When the bridge deck is free to expand, the *uniform variation* of temperature ΔT_1 results in longitudinal deformations that can be accommodated by the expansion joints. Deformations Δl that are due to thermal effects may be calculated using the following equation:

$$\Delta l = \alpha_T \cdot l \cdot \Delta T_1 \tag{10.3}$$

α_T	:	coefficient of thermal expansion (for steel and concrete 10^{-5}/°C)
l	:	length of the element being considered
ΔT_1	:	uniform temperature variation

As an example, for a bridge constructed from steel and concrete, both assumed to have a coefficient of thermal expansion α_T of 10^{-5}/°C, a one degree change in temperature results in a longitudinal deformation of 1 mm per 100 m length. Uniform variations of temperature in a bridge, which correspond to annual variations in temperature, are of particular relevance when considering serviceability of the expansion joints and moving bearings.

When calculating both the displacements that must be accommodated by the expansion joints and the dimensions of the moving bearings, the Swiss Federal Roads Office [10.2] suggests increasing the values of the uniform temperature variation to be used in equation (10.3) by ± 10 °C. This additional variation is intended to allow for any differences in temperature between that during initial setting up of a component, which is normally carried out in the workshop, and the real temperature when it is located on site. If the ambient temperature during placement of the equipment on site is substantially different from 10 °C (for example, the annual average for the Swiss plane), this must be taken into account by modifying the values given above. Additionally, the sum of these two temperature variations must be increased by 50% to assure sufficient capacity for movement during the whole life of the bridge. If this movement is either totally or partially restrained, then normal forces will develop in the bridge deck and, unless certain provisions are made, be transferred to the piers and abutments. Any restraint due to the small difference in the coefficients of thermal expansion of concrete and steel may be ignored.

The result of a *linear temperature variation* ΔT_2 is to make beams curve upwards when the thermal gradient is positive (higher temperature on the upper surface), or downwards under a negative thermal gradient. If such movement is restricted (as is the case for continuous beams), then support reactions develop in the piers and abutments, and these lead to bending moments (and shear forces) in the superstructure. The same effects can arise if the cross section comprises several materials that are joined together and possess different coefficients of thermal expansion.

The beams deform laterally under the influence of a *horizontal temperature variation* if the horizontal structural form is determinate. This effect must be taken into account if the horizontal gradient is substantial during, for example, launching of the steel structure, such that the launching nose approaches the piers as planned. If the structure is indeterminate in the horizontal plane, lateral forces will develop in the supports and piers, and these will result in bending moments about a vertical axis.

A *non-linear variation* in temperature across the section results in a state of stresses that is in equilibrium. Normally such stresses are ignored during structural design. If this is not acceptable, then a reasonable estimation of the stresses can only be made knowing an exact distribution of temperature in the beams.

10.4.3 Snow

Snow is not normally taken into account when designing a bridge. However, for bridges on communication routes in regions of heavy snowfall, the load carrying structure is checked against traffic loads combined with half the snow load or, if it is more unfavourable, the full snow load with no associated traffic. The design value of the snow load q_s may be taken from the standard SIA 261, or other appropriate standards, taking into account local snow conditions.

10.5 Actions During Construction

Large forces may develop in the load carrying structure during construction of a bridge as a result of the combined effects of self-weight and imposed loads. The latter are due to construction equipment, temporary storage of materials, and/or structural components temporarily placed on the structure itself. Whilst it may be possible to calculate the self-weight and its longitudinal distribution with reasonable accuracy for each construction phase, the same cannot be said for the imposed loads. The standards do not generally include a load model to cover the loads due to concreting, the weight of labour, and accumulations of material and/or components (although EN 1991-6 does specify a minimum value). It is up to the engineer to estimate the numerical values of these actions, and to carefully take them into account in the basis of design, notably in the quality assurance plan for controlling execution. The same is true for the larger items of construction equipment such as cranes, be they mobile or fixed, and formwork trolleys.

The types of action and hazard scenarios to consider during construction are very much a function of the methods to be used for steelwork erection and slab construction. Specific types of actions may be present depending on the methods envisaged. Different hazard scenarios and their associated actions are presented in Chapter 7 (erection of the steel structure) as well as in Section 8.4 (construction of the concrete slab). It is also necessary to take into account particular actions which act on the piers during erection of the steelwork, and which may be very different from those when the bridge is being used in its final state.

It is necessary to adopt appropriate surveillance procedures to ensure adherence to assumptions made during the structural design, and defined in the basis of design, concerning things such as locations for storing materials on the structure or dimensions of cantilevers. These procedures must be clearly explained in the quality assurance plan that is used for controlling execution.

10.6 Accidental Actions

10.6.1 Seismic Actions

In Europe, as far as the standard is concerned, a seismic event is considered to be an accidental action. The principles for checking bridges against seismic events cover checks for both structural safety and serviceability, although the latter only apply to one category of bridge. This category covers bridges that are considered to be of great importance to a community, and must therefore be able to fulfil their function following a seismic event. A bridge in a rural location that sees very little use clearly is of less importance than one that is on the only communication route serving the whole of a valley. While in both cases one would design against collapse, significant damage making the former temporarily unusable would be tolerated, although for the latter, anything other than insignificant damage that does not affect serviceability would be unacceptable. In the first case it would only be necessary to check structural safety for the case of a seismic event, whereas for the second, it would also be necessary to check serviceability.

Therefore, the classification of a bridge is a function of its importance. This is achieved by placing the structure into one of three classes (CO). The criteria used to establish the class are: the importance of the bridge as far as organising rescue services is concerned, the average level of use, the potential for damage and associated risks to the environment [10.3]. Standard SIA 261 gives examples of bridges that fall into each class. Each class is characterised by an *importance factor* γ_f which magnifies the seismic actions for design.

As with most accidental actions, measures other than design calculations, such as choosing adequate construction details, are often the most effective solutions to the problem. A number of conceptual options aimed specifically at improving the behaviour of structures subject to a seismic event are given in the SIA standard. Depending on the class of the bridge, these measures may be either recommended or mandatory (and are a function of its geographic location, which places it in a certain zone of seismic risk (Z)). These different zones do affect the value of horizontal acceleration that is used in calculations. The measures to be taken also depend on the nature of the ground, which influences how the seismic effects are transferred to the piers and abutments.

During a seismic event a bridge is subjected to horizontal excitations in the longitudinal and transverse directions of the bridge, as well as vertical excitations, that are transferred by ground movements. In the case of a beam bridge, the results of a seismic event may include the following:

- superstructure falling off its supports,
- damage to the bearings, particularly fixed bearings,
- damage to the abutments,
- damage to the piers,
- damage to the road and expansion joints.

In all circumstances the superstructure falling off its supports must be avoided, whereas it is normally possible to repair damage to the bearings, abutments, piers and joints. As noted above, the designer should take appropriate measures during the bridge conceptual design phase and plan adequate construction details for the supports and piers.

Bridge Conceptual Design and Detailing against Seismic Actions

The bridge must be able to absorb forces from a seismic event by elastic and plastic deformations of the piers. The superstructure normally behaves elastically during an earthquake. Bridges that are long, continuous, and free from expansion joints are normally better because any intermediate joint represents a weak spot, as far as seismic behaviour is concerned. Continuous beams with spans that are more-or-less equal are better than those with irregular spans. Bridges on flexible piers perform better in their longitudinal direction than those where there is a fixed point on one of the abutments, because systems that are used to provide fixity cannot accommodate the enormous horizontal forces that develop during a seismic event. More detailed guidance on bridge conceptual design and construction detailing adapted to the effects of seismic events may be found in [10.4].

In order to assure the safety of a bridge, even if the supports are designed to resist the effects of a seismic event, it is obligatory to plan measures that will guarantee such an event does not result in the

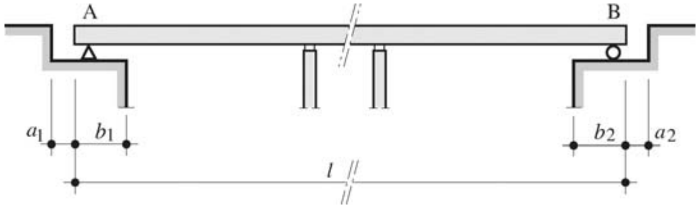


Fig. 10.5 Dimensions for the support zones of a bridge (in order to avoid the superstructure falling off its supports).

superstructure falling off its supports. One such measure, in the longitudinal sense, is to adopt minimum dimensions for the support tables, including the distances between the beam ends and abutment faces. These dimensions, notably b_1 and b_2 for bridges with a fixed support at an abutment, and b_2 for the moving bearings on the abutments of a bridge on flexible piers, are illustrated in Figure 10.5. The numerical values of these dimensions are based on considerations of the movement of the ground relative to the superstructure during a seismic event. Therefore, they are a function of the seismic risk zone in which the bridge is located, and the nature of the ground (defined by the class of ground) on which it is built.

The dimensions b_1 and b_2 of the support tables are defined by equations (10.4) to (10.6) of Eurocode 8 [10.5]; the same definitions are given in [10.6]. These definitions are slightly different from those given in the standard SIA 261 (2003), which are based on an earlier draft of Eurocode 8. The values of b_1 and b_2 are defined as follows:

- for bridges with a fixed support (at point A):

$$b_1 \geq 0.2 \text{ m} + a_2 + \frac{2l}{l_g} u_{gd} \leq 0.2 \text{ m} + a_2 + 2u_{gd} \tag{10.4}$$

$$b_2 \geq 0.2 \text{ m} + a_1 + \frac{2l}{l_g} u_{gd} \leq 0.2 \text{ m} + a_1 + 2u_{gd} \tag{10.5}$$

- for bridges on flexible piers:

$$b_2 \geq 0.2 \text{ m} + \left(1.3 + \frac{2l}{l_g}\right) u_{gd} \leq 0.2 \text{ m} + 3.3u_{gd} \tag{10.6}$$

l : length of the bridge when it has a fixed point at an abutment, or distance between the abutment and the theoretical fixed point when the bridge is on flexible piers

l_g : reference length, which depends on the ground type according to Table 10.6. This is the length beyond which the ground movement may be considered as non-correlated, that is, the maximum displacement between the abutment and the pier foundations is $2u_{gd}$

u_{gd} : design value of ground displacement given in Table 10.7 for class CO I; for classes CO II and CO III the values in this table should be multiplied by γ_f, with a value of 1.2 and 1.4 respectively.

Table 10.6 Reference length l_g as a function of the ground type (according to EN 1998).

Ground Type	A	B	C	D	E
Length l_g [m]	600	500	400	300	500

By way of example, Table 10.8 gives support table lengths derived using equations (10.4) to (10.6). They were calculated for a bridge with a fixed support on one of its abutments, and for a bridge on flexible piers, for bridge class CO III and for unfavourable ground type (class D: fine unconsolidated sand). Given the construction details needed to install the bearings, these values, calculated for $a_1 = a_2 = 1.0$ m, can be easily respected.

Table 10.7 Design value u_{gd} [mm] for the ground movement for CO I [10.6].

Ground Type		Zone Z1	Zone Z2	Zone Z3a	Zone Z3b
A	Rock or other rock like geological formation, including at most 5 m of weaker material at the surface	20	40	50	60
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres (30 m) in thickness, characterised by a gradual increase of mechanical properties with depth	40	60	80	100
C	Deep deposits of dense or medium dense sand, gravel, or stiff clay with thickness from several tens (30 m) to many hundreds of metres	50	70	90	110
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil with thickness from several tens of metres (30 m)	60	110	140	170
E	A soil profile consisting of a surface alluvium layer of type C or D and thickness varying between about 5 m and 30 m, underlain by stiffer material of type A or B	40	70	90	110

Table 10.8 Examples of minimum values for the dimensions b_1 and b_2 (CO III, $a_1 = a_2 = 1.0$ m).

Zone of Seismic Risk (SIA 261)	Unfavourable Ground Type D (According to SIA 261)			
	$b_1$and b_2 [m] fixed point on an abutment		b_2 [m] bridge on flexible piers	
	$l = 50$ m	$l = 200$ m	$l = 200$ m	$l = 500$ m
Z1	1.23	1.32	0.44	0.50
Z2	1.25	1.40	0.60	0.70
Z3a	1.27	1.46	0.72	0.85
Z3b	1.28	1.52	0.84	1.00

There is much less danger of a bridge superstructure falling off its supports in the transverse direction, because the presence of cross bracing at the supports, and the torsional resistance of the bridge cross section, help prevent such a fall. Nevertheless, in zones of high seismicity and depending on the choice of bridge cross section, it may be necessary to include a means of lateral retention to prevent the beams falling.

Verifications

For all bridges it is necessary to check structural safety with regard to seismic actions by means of calculation. The checks primarily concern the piers, the support zones, and, where appropriate, the fixed bearings. The beams themselves do not normally require checking by calculation, neither for the horizontal nor vertical forces.

Checking serviceability is only necessary for bridges that are class CO III. In particular such checks concern the function of the expansion joints and the displacements of moving bearings, which must be guaranteed for half the design value of the seismic accidental action that is used for checking structural safety (but with $\gamma_f = 1.4$).

For design against seismic actions, it is recommended to use a method that assures adequate deformation capacity (ductile behaviour). The plastic hinges in the structure are located, designed, and detailed so that they will be sufficiently ductile during a seismic event. The other parts of the structure are strengthened to ensure that they remain elastic when the plastic zones reach their effective ultimate resistance. As far as bridges are concerned, such a design approach concerns the piers, with plastic zones generally located at the base of those piers that are fixed to the bridge deck. The superstructure itself must remain elastic during a seismic event.

Two principal methods are used to determine the forces due to a seismic event: one using equivalent forces and one using a response spectrum. The first method replaces the dynamic seismic effects with equivalent static forces, which are determined considering the ground accelerations and the mass of the structure. The latter comprises a dynamic analysis of a linear elastic model of the structure. Both methods are described in detail in [10.7].

The use of equivalent forces is the most appropriate for bridges. The structure may be modelled as a single degree of freedom system comprising a mass, a spring and a damper. The superstructure, assumed to be rigid transversally, is effectively the location of the vast majority of the mass, and the piers assure horizontal stability. The method of equivalent forces is presented in the standard SIA 261. However, certain types of structure (arch bridges or the masts of cable supported bridges) often do require detailed calculation of the dynamic effects resulting from a seismic event.

Accompanying Actions

According to the standard SIA 260, when considering accidental actions, it is necessary to take into account the quasi-permanent accompanying variable actions $\psi_{2i}Q_{ki}$ with the accidental action A_d, in addition to the permanent actions. For bridges $\psi_{2i} = 0$, this means that when checking against a seismic event, there is no need to also include the traffic loads in the design calculations.

10.6.2 Impact

Impact against a structure is considered to be an accidental action in the standard SIA 261. It may be the result of a road vehicle, railway vehicle, boat, aeroplane, crane, or rock fall.

For such accidental actions the best approach is to first consider preventative measures that affect the origin of the danger and to eliminate or reduce the possibility of an impact (for example by modifying the positioning of the piers relative to the communication route under a bridge). Other means of protecting the structure may be appropriate, such as steel mesh (nets) retaining rocks, concrete parapets in front of the piers, or crash barriers. Also, the use of a redundant structural form (with alternative load paths) can prevent total collapse, even if parts of the structure are destroyed.

When it is not possible to reduce the hazard sufficiently by measures that act at its origin, or by adopting protective measures, then the structural members must provide sufficient resistance against the impact. Calculations must take into account the mass and speed of the colliding object, and the elastic and plastic deformation capacities of both the object and the structural member under consideration. In standard SIA 261, impact actions are modelled as equivalent static forces.

Impacts Due to Road Traffic

Impacts of road traffic must be considered if a load carrying structure is located near a communication route. The standard defines a distance beyond which such impacts may be neglected, namely 3 m within towns and 10 m elsewhere. When designing the load carrying elements of a bridge, one should distinguish between frontal and lateral impacts against the element. Also, the impacting object may be either the chassis of a heavy goods vehicle, or an element of bodywork, or even the cargo being carried by the vehicle. Both the standard SIA 261 and the directives of the Federal Roads Office [10.8] distinguish between the following types of impact, which are shown schematically in Figure 10.9:

a) frontal impact of a vehicle (chassis) against a pier, with a line of action at an angle α relative to the direction of the traffic,
b) lateral impact of a vehicle (chassis) against a wall or parapet, with a line of action that is assumed to be perpendicular to the axis of the route,
c) frontal impact of either the bodywork or vehicle cargo against a pier,
d) lateral impact of either the bodywork or vehicle cargo against a wall or parapet,
e) frontal impact of either the bodywork or vehicle cargo against an element of the superstructure above the route.

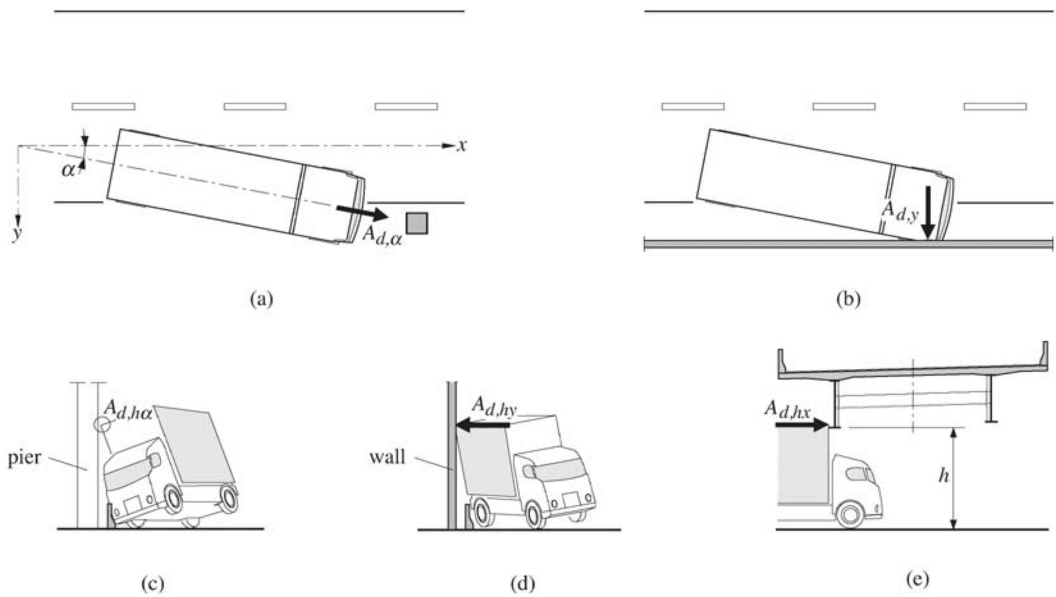


Fig. 10.9 Schematic of possible types of impact and the notation used for the resulting forces.

Base Values

Design values for the different types of impact have been determined on a probabilistic basis. In particular, directive [10.8] proposes base values Q_0 for different impact types, which enable the design values to be derived. The base values given in this directive generally correspond to the design values given in the standard SIA 261. However, the design values given in the directive (see Table 10.10) are generally smaller than the conservative design values given in the standard, and take into account additional parameters which describe the impact. The base values are given in Table 10.10, which also describes the size of the impact area for the equivalent horizontal forces, and their position.

Table 10.10 Base values Q_0 used to determine design values of impact forces, and details of application of these forces according to [10.8].

	(a) Frontal Impact (chassis)	(b) Lateral Impact (chassis)	(c) Frontal Impact (bodywork or cargo)	(d) Lateral Impact (bodywork or cargo)	(e) Frontal Impact, against a Structural Element
	$Q_{0,\alpha}$[a] [kN]	$Q_{0,y}$ [kN]	$Q_{0,h\alpha}$[a] [kN]	$Q_{0,hy}$ [kN]	$Q_{0,hx}$ [kN]
Freeways	1500	600	500	200	750
Main roads, v = 80 km/h	1000	400	333	133	500
Roads within town	500	200	150	60	250
Impact area	0.4 m × 1.5[b] m				concentrated
Height above the road of the centre of the impact area[c]	0.75 m to 1.5 m		1.5 m to 4.0 m		against the lower flange

[a] force acting with an angle between 0 and 30°, the most unfavourable angle has to be considered,
[b] if the width of the impacted structural member is less than 1.50 m, consider the width of the member,
[c] if the structural member is located on a slope or down below the roadway, the height of application is measured from the foot of the structural member.

Design Values

Outside towns, for main roads and freeways, the design value of the equivalent impact force is a function of the distance between the load carrying structure and the edge of the road, the volume and mix of traffic, and whether or not there is a traffic containment system in place to protect the structure. Design values are determined using equations (10.7) and (10.8).

For *frontal or lateral impact of the chassis* of a heavy goods vehicle, or its bodywork, against a pier or wall:

$$A_d = \psi_s \cdot \psi_v \cdot \psi_r \cdot Q_0 \qquad (10.7)$$

Q_0 : base value, from table 10.10

ψ_s : reduction factor to take into account the distance between the load carrying structure and the edge of the road, from Figure 10.11(a). When the ground is not flat this factor must be modified. Directive [10.8] gives guidance on this subject

ψ_v : magnification factor to take into account the traffic; directive [10.8] gives guidance on this subject, but it is worth noting that for a normal proportion (6%) of heavy traffic in the total volume this factor is equal to 1.0 for an average daily traffic of 20 000 vehicles, and 1.35 for 60 000 vehicles

ψ_r : reduction factor to take into account traffic containment systems

When traffic containment systems such as concrete crash barriers are located between the road and the load carrying structure (Fig. 10.9(c) and (d)) the value of $\Psi_r = 0$. However, if the system is placed less than two metres away from the structure, one must still take into account possible impact against piers and walls from the bodywork or vehicle cargo. When crash barriers are planned as the means of traffic containment, then the reduction coefficient is a function of the type of barrier and its distance from the load carrying structure [10.8].

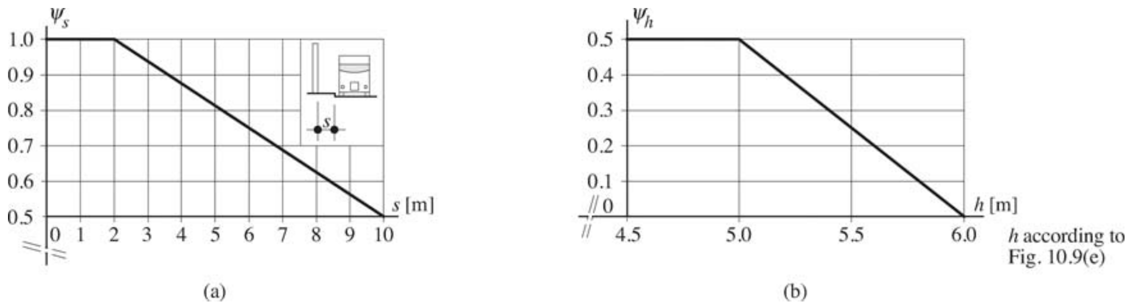


Fig. 10.11 Reduction coefficient ψ_s as a function of the distance s (for flat terrain) and reduction coefficient ψ_h as a function of headroom h.

For a *frontal impact of the bodywork* or vehicle cargo against the *superstructure* of a bridge:

$$A_{d,\,hx} = \psi_h \cdot Q_{0,\,hx} \tag{10.8}$$

ψ_h : reduction factor (Fig. 10.11(b)) to take into account the headroom according to Figure 10.9(e)

Within towns, the design value of the equivalent impact force may be taken as equal to the base value according to Table 10.10. According to [10.8], in such cases the values should be taken as indicative only. Specific details of the locality, such as the exposure of the load carrying structure, embankments, and curves in the road should be taken into account to either increase or reduce these indicative values. For frontal or lateral impacts of a vehicle chassis, the standard SIA 261 suggests that design values should be taken as the base values increased by 50% to recognise that the risk to the structure in case of impact is high.

Accompanying Actions

According to the standard SIA 260, for the "impact" hazard scenario, the designer must decide if an accompanying variable action with a frequent value $\psi_1 Q_k$ must be taken into account in conjunction with the accidental action A_d and the permanent actions. For the case of an impact against a bridge parapet, or a bridge pier, it is reasonable to assume that accompanying traffic may exist on the bridge when the route

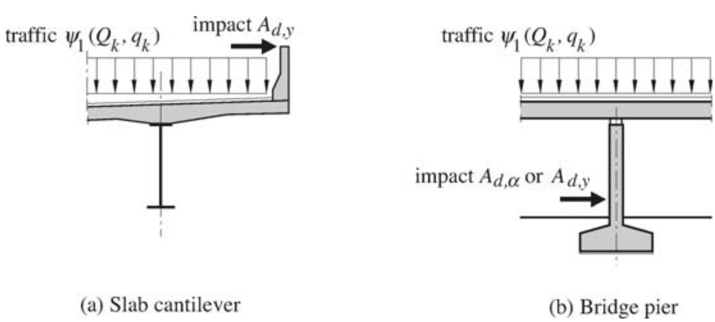


Fig. 10.12 Examples of the hazard scenario "impact" with accompanying traffic loading.

is subject to high traffic volumes. Figure 10.12 shows these two cases, for a slab cantilever and a bridge pier respectively. The road traffic is represented by load model 1, according to SIA 261, and is taken as accompanying with $\psi_1 = 0.75$.

Other Sources of Impact

Impacts due to trains are considered in Paragraph 16.3.5. Impacts from other types of vehicles and objects should be defined in consultation with the relevant authorities or, if that is not possible, with the client's representative. Information concerning rock fall onto bridges is contained in the standard SIA 261/1 and [10.9].

10.7 Frictional and Restraint Forces from Bearings

Frictional forces and those due to restraint of the bearings comprise horizontal actions that either the superstructure of a bridge imposes on the substructure, or conversely the substructure imposes on the superstructure, due to friction or resistance to deformation of the bearings. The numerical value of such actions is a function of the type of bearing, be it moving (sliding or rolling) or deformable. Forces transferred to the substructure by fixed bearings are considered in Chapter 15.

10.7.1 Sliding or Roller Bearings

For sliding bearings – pot or linear steel bearings – the sliding surface comprises a layer of PTFE (Polytetrafluoroethylene or Teflon®). This sliding surface, in addition to being very durable, has the advantage of possessing a small coefficient of friction μ. The frictional forces that develop above the bearings during movement of the superstructure are proportional to the support reaction:

$$Q_{f,d} = \mu N_d \tag{10.9}$$

$Q_{f,d}$	:	design value of the frictional force
μ	:	coefficient of friction
N_d	:	design value of the normal force acting on the bearing (support reaction)

The coefficient of friction μ reduces to between 0.06 and 0.03 when the applied stress is between 10 and 30 N/mm^2. Such low values of friction will only be achieved in practice if the bearings are properly maintained. Much higher values will apply if the sliding surface is heavily contaminated. For roller bearings, which are less commonly used now than in the past, the coefficient of friction is 0.025 for rollers made from special high strength steel.

10.7.2 Deformable Bearings

Deformable bearings, which are for example fretted elastomers, or "Neoprene", comprise a rectangular or circular block of elastomer that is sheared between steel plates. The force that develops in these bearings is known as a *restraint force*, because it acts in the opposite direction to the displacement that causes it. It is proportional to the displacement u, the shear modulus of the elastomer G, and the dimensions of the bearing. For this type of bearing, the displacement u should not exceed 0.7 nt, where n is the number of layers of elastomer, each of thickness t. Also, a minimum load equal to a stress of 2 to 3 N/mm^2 on the bearing surface is needed to transfer the restraint forces to the structure. If the stress is lower than this, the bearing must be mechanically fixed to the load carrying structure. The restraint force may be calculated using the following relationship:

$$Q_{rap,\,d} = \frac{AG}{nt}u_d \tag{10.10}$$

u_d	:	design value of the beam displacement above the support
A	:	contact area of the bearing
G	:	shear modulus of the elastomer ($G \cong 0.8$ N/mm^2)
n	:	number of layers of elastomer
t	:	thickness of one layer of elastomer

More detailed guidance on the design of bearings is given in the German standard DIN 4141 [10.10] and in manufacturers' literature.

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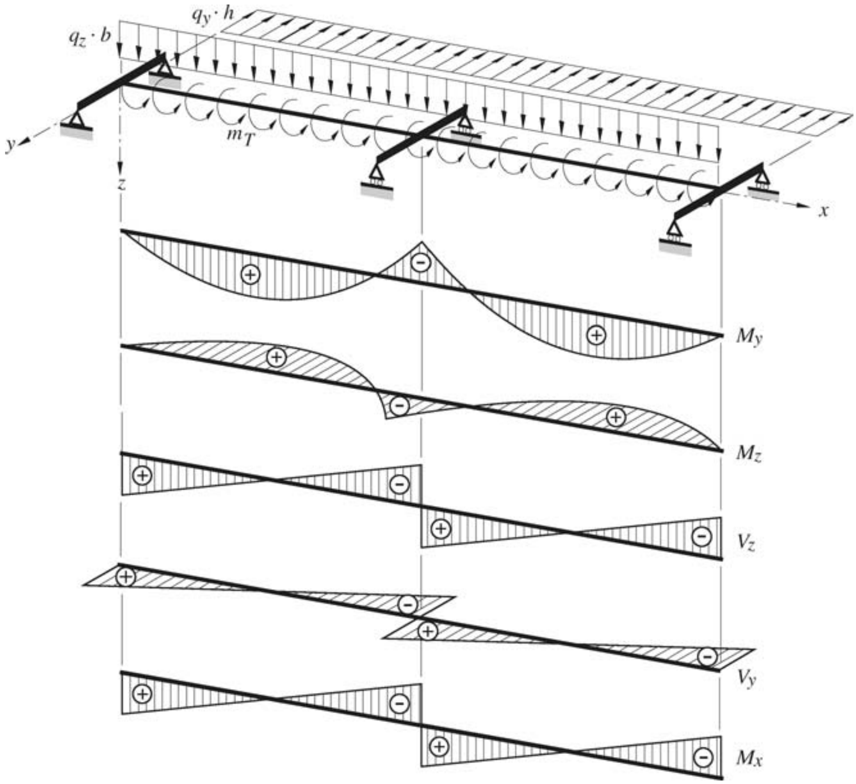
11 Internal Moments and Forces in Beam Bridges



11.1 Introduction

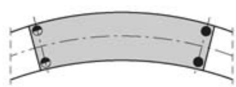
This chapter considers the elastic calculation of the *internal moments and forces* in beam bridges. Their load carrying structures are not only subject to bending moments and the corresponding shear forces, but also to torsion, which can be significant in the analysis of a bridge. For beam bridges torsion may be due to either eccentric actions (Sect. 5.5) or the geometry of the bridge (skew, curved).

The rules of statics presented in TGC Volumes 1 and 2 allow the internal moments and forces (M_y, M_z, M_x, V_y, V_z, N) to be calculated for beams that are either straight or slightly curved. This chapter covers in more detail the specifics of bridges when calculating these moments and forces (loads and actions, *modelling of the structure*, internal moments and forces) in addition to the statics for curved and skew bridges.

The distinction between types of bridge *cross sections* (closed or open) is particularly useful and necessary when calculating the internal moments and forces or the stresses acting on a given cross section. The primary mode of resisting torsion (uniform or non-uniform) is fundamentally different for these two types of cross section, and it has a big impact on the calculation methods.

This chapter is structured as follows: Section 11.2 gives a short summary of the calculation assumptions and modelling that are normally applied to straight beam bridges, basic for this type of structure. The importance of considering torsion when designing bridges is emphasised. The theory of torsion, and particularly non-uniform torsion (which is partly treated in TGC Volume 10, Paragraph 4.5.3), is considered in Section 11.3. The sections that follow – 11.4 to 11.7 – show the calculation of internal moments and forces for straight, skew and curved bridges, as shown in Table 11.1.

Table 11.1 Structure and contents of this chapter.

Type of Bridge	Straight Bridge		Skew Bridge		Curved Bridge	
Plan view						
Cross section	closed	open	closed	open	closed	open
Section / Paragraph	11.4	11.5	11.6.2	11.6.3	11.7.3	11.7.5

Calculation of the internal moments and forces in bridge beams is normally based on an *analysis model* that assumes the load carrying structure exhibits *elastic behaviour*, in other words it does not recognise redistribution of moments and forces following possible local plastification of a section (formation of a plastic hinge). Effectively, the slenderness of the flanges and/or parts of the web in compression of plate girders mean that local instability phenomena such as local buckling (TGC Vol. 10, Chap. 12) appear before plastification of the section. The rotation capacity of slender sections is therefore strictly limited, and this does not allow redistribution of the internal moments and forces. An exception is the span section of composite bridges, where the steel beams are only really subject to tensile stresses [11.1]. Allowance for plastic redistribution in such cases is discussed in Paragraph 13.4.2.

Even though engineers are now equipped with powerful computers and software that can easily calculate the internal moments and forces in a structure, an understanding of statics and knowledge of different analytical approaches are essential for the whole process – from conceptual to structural design. With this in mind, *simplified methods* are presented in this chapter; on the one hand, they allow preliminary sizing of structural members, and on the other, they enable the assessment of numerical computer output.

11.2 Modelling of Beam Bridges

11.2.1 Structural Model

Bridges are structures comprising planar elements that form a stable three-dimensional body. Equilibrium of this body is assured by connection to the ground via bearings, piers, abutments, and the bridge foundations. Therefore, bridges are spatial structures (Fig. 11.2(a)) subjected to external loads and actions and limited in their movements by their support conditions. The calculation of the internal moments and forces is an important step in quantifying the effects of the different actions from their point of application down to the supports (Sect. 5.2). Different models may be used to facilitate these calculations; the choice of model depends on the complexity of the structure, the available calculation tools, and the aims of the process. For a beam bridge, whether it is formed from a box girder or two or more I sections, the structural model that is normally used comprises the load carrying structure as a bar located at the centre of gravity of the box cross section or the I sections. This model is analysed using simple structural analysis (Fig. 11.2(b)). Such modelling is acceptable if the following conditions are satisfied:

- the length of the beam is substantially greater than the cross section dimensions (width and depth),
- the bridge cross section does not distort as a result of beam deflections,
- shear deflections are negligible, and
- stresses are proportional to deformations.

Plate girders and box girders, forming the cross sections of bridges that are either steel or composite, are fabricated by welding together plates that are of relatively limited thickness (§ 5.3.2). These structural elements are flexible out of plane and cannot alone guarantee that the bridge's cross section will not distort.

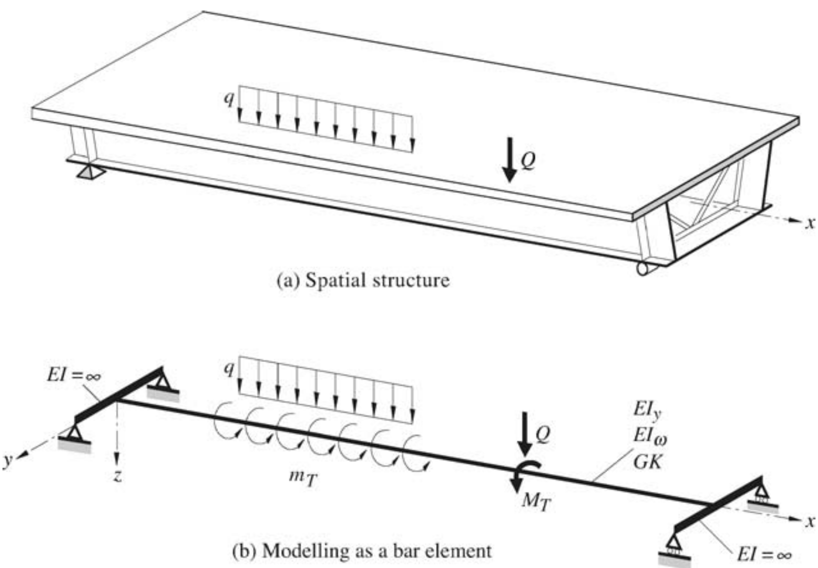


Fig. 11.2 Modelling of a beam bridge.

Hence, it is necessary to introduce cross bracing orientated perpendicular to the bridge axis to stiffen the cross section (§ 5.6.1). For a beam bridge with cross bracing present, it is assumed that the cross section will not distort; therefore, simple analysis using a bar model can be applied. The discussions below assume this.

Generally speaking the bar or bars in a bridge model must resist the bending moments, M_y and M_z, the corresponding shear forces, V_z and V_y, and the torsional moments, M_x. These internal moments and forces are calculated starting with the loads which act on the bar. However, the definition of these loads is based, first of all, on a transverse analysis of the bridge cross section. Only after this has been carried out can the internal moments and forces be determined from a longitudinal analysis of the beam. The following definitions should be noted:

- ***torque*** $\boldsymbol{M_T}$ or $\boldsymbol{m_T}$, the applied torsion, be it concentrated or distributed respectively, coming from the analysis of the loads and actions acting on the bridge cross section and which are applied to the beam,
- ***torsional moment*** $\boldsymbol{M_x}$, the internal torsional moment coming from the longitudinal analysis of the bar,
- ***torsional moment resistance*** $\boldsymbol{T}$, the resistance against the torsional moment M_x in each cross section of the beam.

Consider, as an example, a bridge that is subject to vertical loads q_z acting on half the road width, plus horizontal forces q_y acting over the beam depth (Fig. 11.3(a)). The *transverse* analysis (Fig. 11.3(b)) is considered first. If the assumptions of linear elastic behaviour (material and geometry) are satisfied, then the principles of load resolution and superposition apply (TGC Vol. 2, § 2.7.4). It follows that the distributed actions q_z and q_y acting on the structure can be resolved into two line loads $q_z \cdot b$ and $q_y \cdot h$, producing bending about the y and z axes, respectively. If these two forces do not act through the shear centre C_T, then a torque m_T will develop and act on the beam as a result of the eccentricities of the loads $q_z \cdot b$ and $q_y \cdot h$. This torque will result in torsional moments M_x about the x axis.

Once the loads have been resolved with respect to the shear centre of the cross section, the *longitudinal* model for the bar (Fig. 11.3(c)) and its structural form can be considered. This longitudinal analysis should be carried out as follows:

- in the vertical plane x-z to determine the bending moments M_y and shear V_z,
- in the horizontal plane x-y to determine the bending moments M_z and shear V_y,
- along the x axis to determine the torsional moments M_x.

The structural forms in the vertical and horizontal planes depend on the type of bearings (moving or fixed) and the piers (stiff or flexible). These things are particularly relevant as far as the transverse structural form is concerned (§ 5.4.2).

The principles of superposition mean that at the end of the analysis, the effects (internal moments and forces, stresses) of the different loads, resolved about the shear centre, can be added together to facilitate the numerous design checks.

Saint-Venant's principle (TGC Vol. 2, § 2.7.5) suggests that the local effects of a concentrated load are limited to a length equal to one or two times the largest dimension of the cross section. This means that for a slender bar, the local effects are practically negligible beyond the zone of load introduction.

More complex analysis models must sometimes be used in order to take into account the three-dimensional behaviour of bridges. This is particularly the case when the structural elements are such that local effects from load introduction are substantial relative to the global effects calculated using a bar model, or when cross bracing is too widely spaced (or not present) to allow the use of a simple model. In particular,

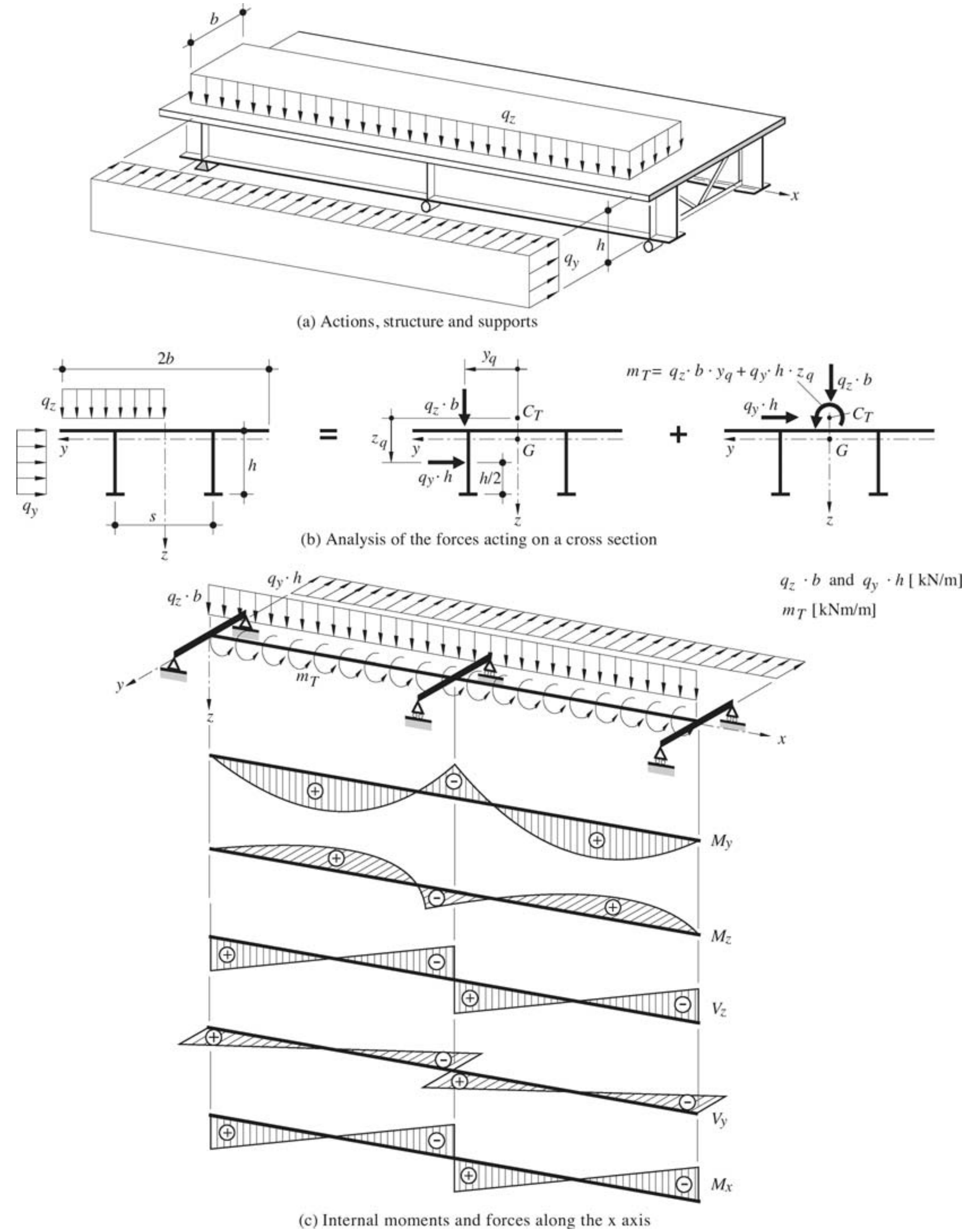


Fig. 11.3 Analysis of the forces acting on the transverse cross section of a bridge and the internal forces along the longitudinal axis.

if distortion of the bridge cross section cannot be prevented, other types of model should be adopted. Examples include models based on *folded plate theory* [11.2], which assume the load carrying structure is formed from different plates and slabs joined together at their edges and loaded either out of or in-plane. Other more complicated models may be used, for example based on the theory of long and thin shell elements developed by Vlassov [11.3].

Numerical methods become essential when the geometry of the structure or the loads become very complex, or when one wishes to take into account non-linear behaviour (non-linear material or geometry), or for structural forms that are highly indeterminate (redundant) and cannot be resolved using simple analysis methods.

11.2.2 Bending Moments

Elastic Analysis (EE) and (EER)

The elastic calculation of ***internal moments and forces*** is covered comprehensively in TGC Volume 1. Such an elastic analysis is normally used for steel and steel-concrete composite bridges. The limited ductility of the beam sections used for steel and composite bridges does not normally allow the plastic moment resistance of the sections to be reached. In particular, for plate girders subject to either negative or positive bending, or composite beams subject to negative bending, instability of the plates in compression reduces the ductility of the section (§ 12.2.1). Also, a beam's resistance is often limited by its lateral torsional buckling resistance (§ 12.2.4).

An elastic calculation of the internal moments and forces must take into account the varying second moments of area of the different cross sections along the length of the beam, which result from changes in plate thickness and/or beam depth. It must also take into account any non-linear material effects. For composite bridges, cracking of the concrete slab above intermediate supports results in a redistribution of moments and forces from the supports into the spans when the structural form is indeterminate. Rigorous calculation of this phenomenon cannot be carried out directly, because the second moment of area of each cross section is a function of the bending moment to which it is subject, and the distribution of moments depends on the relative stiffness. Paragraph 13.3.2 considers how cracked concrete should be taken into account in an elastic calculation of the bending moments along a composite beam.

Elasto-plastic Analysis (EP)

When plastic resistances are considered, the bending moments given by an elastic analysis are redistributed away from regions where the section has entered into elasto-plastic behaviour toward regions that remain elastic. However, this redistribution is not normally considered in an engineer's calculations. In particular, when considering the plastic resistance of a composite beam in the span (sagging), the bending moments that redistribute to the intermediate supports are not taken into account. The limits of application of this calculation method, which is known as EP for composite beams, are discussed in Paragraph 13.4.2.

As far as steel beams are concerned, it is also necessary to satisfy certain conditions before an EP calculation can be carried out. Given in the standard SIA 263, they include:

- the cross sections in span must be at least class 2 (TGC Vol. 10, § 12.3.2),
- under the same load conditions, the moment at an intermediate support should not exceed 90% of the elastic resistance of the section divided by γ_a, and
- the beam must be restrained against lateral torsional buckling.

When carrying out an elastic analysis to determine the internal moments and forces for checking against plastic behaviour of the cross sectional resistances in span (EP method), and assuming elastic behaviour at the intermediate supports, certain aspects of behaviour may be ignored. These include the loading history, as far as the bending moments are concerned, and the effects of any imposed deformations, such as shrinkage or those due to temperature changes that show up when checking structural safety. This is clearly not the case when elastic cross sectional resistances are considered (EE or EER method), when all such effects must be taken into account. In particular, when checking against the elastic resistance of a composite cross section, it is necessary to calculate the different bending moments that are applied to the different resisting sections (§ 13.4.3).

Plastic Analysis (PP)
In some cases it is possible to use plastic analysis to determine the internal moments and forces, which are then checked against plastic cross sectional resistances. Examples are multi-girder bridges using rolled sections that are Class 1 according to the standard slenderness rules, and simply supported composite beams, where the girders are primarily subject to tension.

11.2.3 Shear Force

Shear forces along the length of a beam are determined using the normal rules of statics (TGC Vol. 1). Normally, the vertical stiffness of the piers in compression may be assumed to be infinite, so there is no need to consider the beams as supported on elastic foundations (springs). The relative stiffness of different piers may be considered when software is used; indeed, this is essential when the deck supports are genuinely flexible (cable stayed and suspension bridges, arches).

The distribution of shear forces between different beams (two webs for a twin-girder bridge or box girder, several webs for a multi girder bridge or multi cell box section) depends on the means of resisting torsion (uniform or non-uniform). This distribution will be studied in Paragraph 11.5.2, particularly for open section bridges. For a box section bridge, the vertical load is supported equally by each of the webs, independent of the transverse position of the load on the structure.

11.2.4 Torsional Moments

Resolving the Loads with Respect to the Shear Centre
The shear centre, which is sometimes also known as the centre of rotation, is the point in a cross section such that if a load is applied with its line of action passing through this point, it produces no torsion, and therefore no rotation, of the cross section. For a closed section this point may be close to the centre of gravity of the cross section. However, for an open section the two points are distinctly different. TGC Volume 2, Section 9.9, contains further information on determining the position of the shear centre.

Resolving (into an equivalent set of moments and forces) the loads acting on a structure with respect to its shear centre, presented in Section 11.2.1, allows the ***torque*** acting on the structure, be it concentrated M_T or distributed m_T, to be calculated (Fig. 11.2). A straight bridge may be subjected to torque when it is acted upon by loads which are applied eccentrically to the shear centres of its cross sections. The analysis of these load cases is carried out assuming the principle of superposition (TGC Vol. 2, § 2.7.4), by *resolving* the eccentric load into a centric force and a torque (Fig. 11.3(b)). The vertical load $q_z \cdot b$, which acts at distance y_q from the shear centre of the cross section, and the horizontal force $q_y \cdot h$, acting at a distance z_q, are therefore resolved into two centric forces plus a torque: $m_T = q_z \cdot b \cdot y_q + q_y \cdot h \cdot z_q$. This method allows

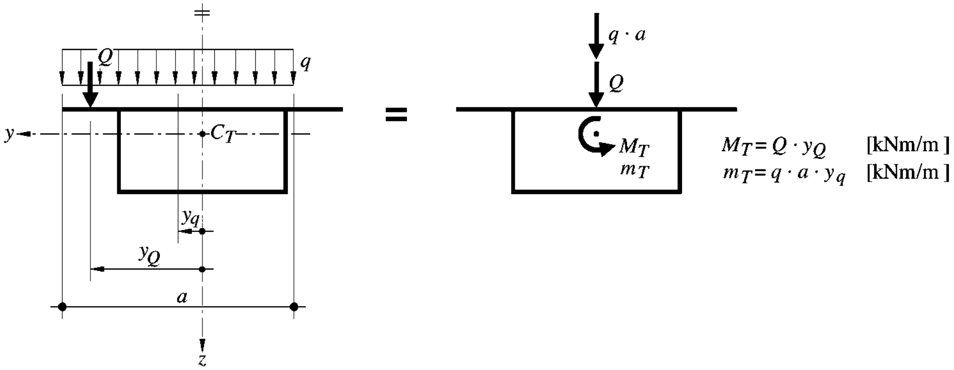


Fig. 11.4 Resolving of loads to those acting at the shear centre.

any type of load to be represented as a vertical and/or horizontal load that is centred and only produces bending, and a torque that only results in torsion along the length of the bar. Figure 11.4 shows another example of resolving loads, this time about the shear centre of a closed section.

A concentrated torque M_T, or distributed torque m_T, results in torsional moments M_x acting along the x axis of the bar. These torsional moments are balanced in the various cross sections by torsional moment resistances T (§ 11.3.1).

Closed Cross Sections

When the cross section of a bridge has a closed form, the bridge may be considered to be a bar that essentially resists in *uniform torsion* (§ 11.3.2). Determining the torsional moments M_x along the axis of a straight bridge subject to either concentrated or distributed torque is a question of considering equilibrium for a determinate structure, and equilibrium plus compatibility for an indeterminate structure. The equations of equilibrium, and the relationships between applied torques and the torsional moment diagram, are analogous to those governing the relationship between applied vertical loads and shear force.

Consider, as an example, an element of a straight and prismatic beam of length dx subject to a distributed torque m_T (fig. 11.5). Equilibrium of this element is described by Equation (11.1):

$$-M_x + m_T \mathrm{d}x + (M_x + \mathrm{d}M_x) = 0 \quad \Rightarrow \quad -\frac{\mathrm{d}M_x}{\mathrm{d}x} = m_T \tag{11.1}$$

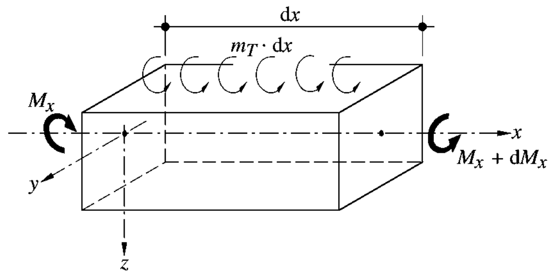


Fig. 11.5 Equilibrium of a beam element of length dx subject to torsional loading.

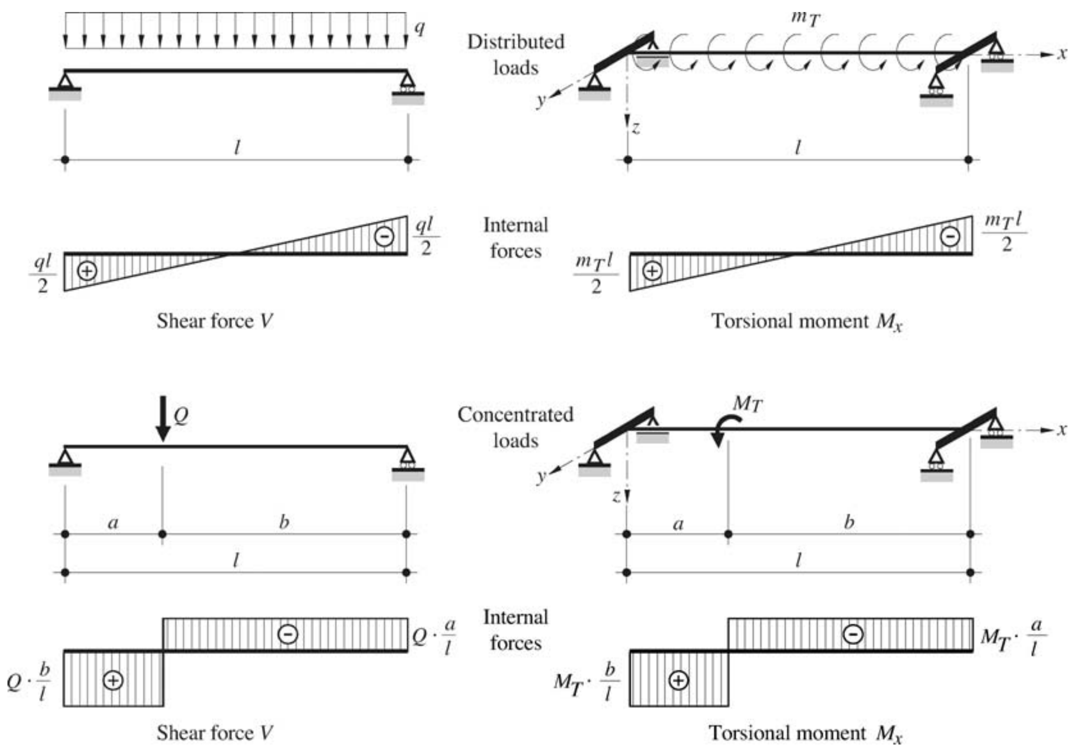


Fig. 11.6 Analogy between shear force V and torsional moment M_x diagrams.

The integration of this differential equation brings into play a constant that is a function of the boundary conditions: the support conditions. It should be noted that for a uniformly distributed torque m_T the torsional moment diagram is linear, whereas it has a discontinuity at the point of application of a concentrated torque M_T. The analogy with a shear force diagram is clear (Fig. 11.6).

For a determinate beam, knowledge of the support conditions is enough to calculate the complete torsional moment diagram (equilibrium). When the structure is indeterminate – which is nearly always the case for bridges because the ends are normally built-in with respect to torsion by supports that are perpendicular to the bridge axis or on a skew (certainly not parallel) – one or more conditions of kinematic compatibility must be considered.

For continuous beams where all the intermediate supports are rigid with respect to torsion, the loading on one span does not influence the torsional moment diagrams in the adjacent spans: there is no continuity. Such a structure is effectively a series of “simple beams with regard to torsion”; therefore, it may be considered separately. However, if the supports do not provide full restraint to torsion, but are rather flexible (slender piers) or only partially built-in (skew supports), more detailed calculations will be needed. Reference may be made to the analytical method presented by Kollbrunner and Basler in [11.2] Chapters 3.3 and 4.2.

Figure 11.7 presents the general case of a torsional moment diagram for a bar that is built-in against torsion and subject to uniformly distributed torque acting on a length c of the beam.

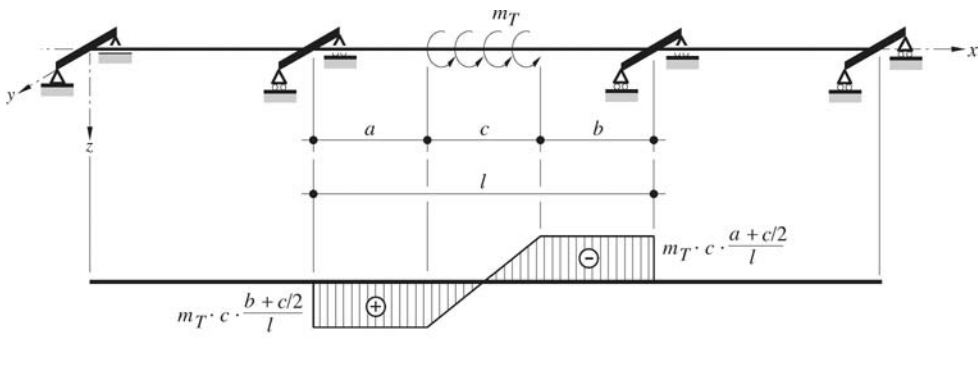


Fig. 11.7 Example of a torsional moment diagram for a bar with uniform torsion resistance.

Open Cross Sections

As discussed above (§ 11.3.1), the torsional moment M_x along a bar is resisted by two torsional modes, namely uniform torsion T_v (§ 11.3.2) and non-uniform torsion T_w (§ 11.3.3). When the bridge has an open cross section, resistance is essentially provided in *non-uniform torsion*, and separate calculations of the internal moments and forces resulting from the bending from the torsional moments is perfectly possible. However, the means by which open sections resist torsion makes the calculation of the torsional moments

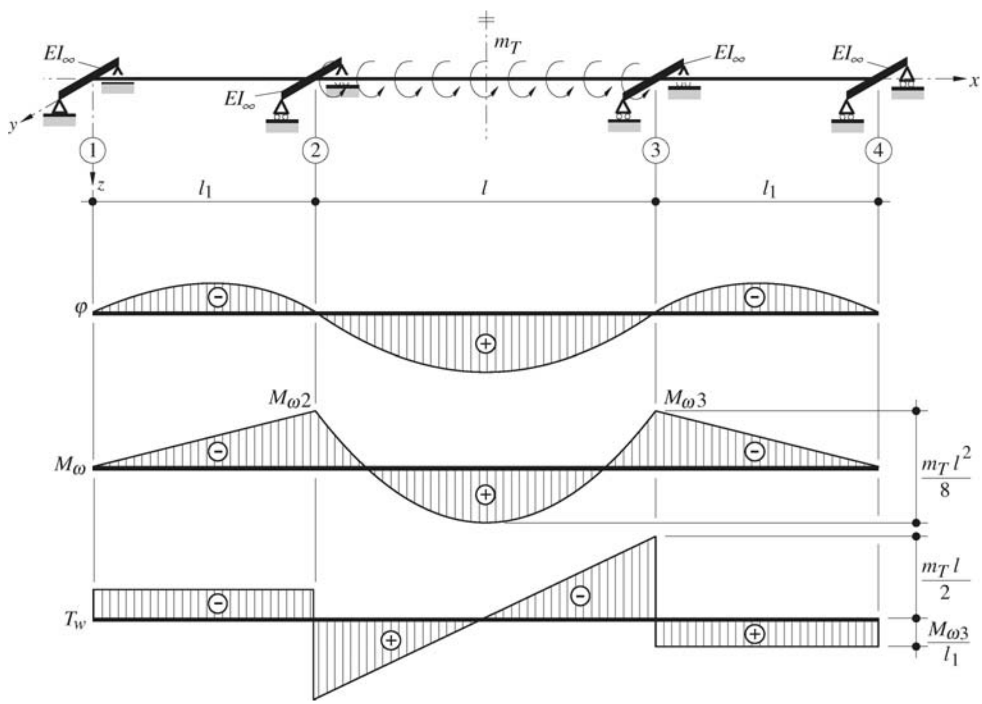


Fig. 11.8 Rotation, warping, and torsional moment resistance for a continuous beam subject to torsional loading.

less useful than for closed sections. Because resistance in non-uniform torsion translates into either normal or shear stresses in the section, it is beneficial to use a calculation tool, namely the *transverse influence line* of the load, to calculate the internal moments and forces in an open cross section (§ 11.5.2). These become simply bending moments and corresponding shear forces.

The method described above must be changed when beams resist in combined (also known as mixed) torsion. This is torsion that combines uniform and non-uniform torsion resistance. The analogy with bending still exists, but the warping continuity of the sections over intermediate supports implies a continuity of the warping moments (sometimes known as *bimoments*) M_{ω} in the unloaded spans. The non-uniform torsional moment resistance (also known as the *warping torsional moment*) T_w is derived from M_{ω}, and there exists an effective continuity of torsional moments, even if the supports are truly built-in against torsion and thus prevent all *rotation* about the x axis but not *warping*.

Figure 11.8 shows this situation for a beam that is continuous over three spans, with a cross section that resists in combined torsion, subject to uniformly distributed torque applied in the central span. The figure shows the rotation φ of the section about the x axis, the warping moment M_{ω}, and the non-uniform torsional moment resistance T_w. It can be seen that it is not only the loaded span that is affected by this torque. For a closed section, which resists essentially in uniform torsion, only the loaded span resists the torque.

11.3 Torsion

The resistance of a cross section to torsion is considered in TGC Vol. 10, Section 4.5. The aim of this section is to provide additional information that is both useful and necessary for the analysis of beam bridges.

11.3.1 Reminder

Torques (torsional loads) may be applied to a bridge by actions that are eccentric with respect to the shear centre of the cross section of the bridge, such as traffic, wind or asymmetric self weight. When the bridge is curved or skew, they may also arise due to centred loads when the bridge is curved or skew.

Torques that may be either concentrated M_T or distributed m_T, acting on a bar element, provoke a torsional moment M_x and are balanced, if one isolates a length of bar, by torsional moments corresponding to two modes of resistance. These modes are resistance in uniform torsion T_v and non-uniform torsion T_w:

- The first mode of resistance comprises a uniform shear flow within the cross section, corresponding to the uniform torsional moment (also known as the *Saint-Venant torsional moment*) T_v. This is the principal mode for *closed* cross sections. Resistance in *uniform torsion* is identified by the subscript v (TGC Vol. 10, § 4.5.2).
- The second mode of resistance comprises normal longitudinal stresses σ_w and shear stresses τ_w, resulting from the variation in normal stresses, corresponding to a *non-uniform* torsional moment T_w. This is also known as the warping torsional moment, and is the primary mode of resistance for *open* cross sections. Resistance in non-uniform torsion is identified by the subscript w for warping (TGC Vol. 10, § 4.5.3).

Therefore, the resistance to a torsional moment comprises the sum of two modes:

$$T = T_v + T_w \tag{11.2}$$

T : torsional moment resistance
T_v : resistance in uniform torsion
T_w : resistance in non-uniform torsion

Generally, a bar subject to a torsional moment that resists simultaneously in uniform and non-uniform torsion is *combined torsion*. Which of the two modes is preponderant is determined primarily by the cross sectional form of the structure. Consideration is given below to box girder bridges, which are closed sections and resist primarily in uniform torsion, and twin-girder bridges having open sections that resist primarily in non-uniform torsion.

11.3.2 Uniform Torsion

Certain requirements must be met to ensure that the equations given below, based on the theory of uniform torsion, are valid. These requirements are those of the theory of slender bars:

- deformations are small,
- the material is continuous, homogenous, isotropic and linear elastic, and
- cross sections maintain their shape and do not distort.

The deformation of a bar subjected to a torsional moment is characterised by the angle of rotation $\varphi(x)$ of a section at a distance x from the origin. The variation of this rotation over a short length dx of the bar is related to the torsional moment resistance $T_v(x)$ according to the following equation:

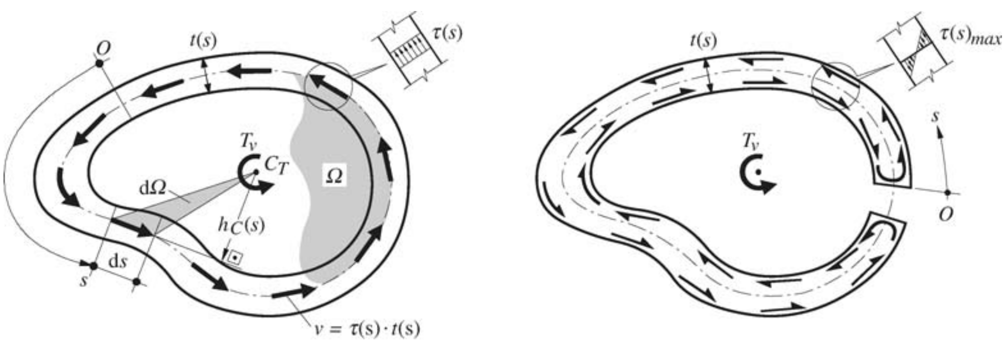
$$\varphi'(x) = \frac{\mathrm{d}\varphi(x)}{\mathrm{d}x} = \frac{T_v(x)}{GK} \tag{11.3}$$

$T_v(x)$	:	torsional moment at a distance x (abscissa)
G	:	shear modulus of the material
K	:	uniform torsion constant

Figure 11.9 shows, for both closed and open thin walled sections, resistance in uniform torsion by means of shear stresses τ in a closed shear flow v.

For a closed thin walled section, the relationship between the torsional moment resistance T_v and the stresses $\tau(s)$ may be expressed in a general manner according to Equation (11.4). The different variables used in this equation are defined in Figure 11.9.

$$T_v = \int \tau(s)t(s)h_C(s)\mathrm{d}s \tag{11.4}$$



(a) Closed section (b) Open section

Fig. 11.9 Shear flow corresponding to uniform torsion resistance.

Determining the magnitude and distribution of the shear stresses can only be done in a simple way for bars that are either circular or annular. For other types of cross section, the equations derived from elastic theory become very complex (TGC Vol. 3). Therefore, it is convenient to use the membrane analogy of Prandtl, as this provides a simple representation of the distribution of the stresses. The results that flow from this analogy are given below, firstly for closed sections, then for open sections.

Closed Thin Walled Sections

Figure 11.9(a) shows this form of cross section, which is typical for steel box girders. Thanks to the membrane analogy, one can easily obtain the general expression for shear flow v [N/m] (11.5):

$$v = \frac{T_v}{2\Omega} = \text{constant} \tag{11.5}$$

Ω : area defined by the median line of the closed section.

If the walls are assumed to be thin relative to their length, then the shear stress $\tau(s)$ may be assumed to be uniform over this thickness t. Equation (11.6), which is sometimes known as the *Bredt formula*, illustrates this relationship. The maximum shear stress occurs where the section wall is thinnest.

$$\tau(s) = \frac{v}{t(s)} = \frac{T_v}{2\Omega t(s)} \tag{11.6}$$

The cross section of a box girder bridge may be idealised as a closed section, and the torsional stiffness of the cantilevers may be neglected if they are thin (orthotropic deck). The theory outlined above may then be used without restriction. The case of a composite box girder (thin walled steel box plus concrete slab of moderate thickness) is treated later in this book.

Open Thin Walled Sections

Figure 11.9(b) shows this form of cross section. The value of shear stress $\tau(s)$ in an open thin walled section may be calculated at any location, but the membrane analogy indicates that it will be zero on the median line and maximum at the edges of the walls. The shear stress $\tau(s)$ is determined at each point around the cross section, as defined by its curvilinear coordinate s, according to Equation (11.7):

$$\tau(s) = \frac{T_v}{K}t(s) \tag{11.7}$$

When the open section is that of a composite twin-girder bridge formed from a collection of plates of width h and thickness t, plus a concrete slab of width $2b$ and thickness h_c, the uniform torsion constant (with respect to steel) of the section K_{eq} is defined by the following relationship:

$$K_{eq} = \frac{1}{3}\sum ht^3 + \frac{1}{3}\frac{(2b)}{m}h_c^3 \tag{11.8}$$

m : modular ratio for shear, $m = G_a/G_c$

G_a, G_c : shear modulus for steel and concrete respectively (for concrete, use the modulus corresponding to the action being considered, which means generally the modulus relating to short term loading, such as traffic)

The shear stresses acting in each of the materials are calculated as follows:

$$\tau_a = \frac{T_v}{K_{eq}}t, \qquad \tau_c = \frac{T_v}{m \cdot K_{eq}}h_c \tag{11.9}$$

It may be seen that, for a given material, the maximum shear stress occurs where the element walls are thickest. The maximum stress $\tau_{a,max}$ in the steel may be higher than that in the reinforced concrete slab $\tau_{c,max}$, even if the thickness of the steel plates is less, because the shear stresses are proportional to the torsional stiffness G_iK_i of the elements of each material in the section.

Section Closed by a Concrete Slab

Composite box girder bridges (Fig. 11.10) are typical of this type of section where, despite its moderate thickness, the torsional stiffness of the concrete slab itself contributes to the torsional stiffness of the section as a whole. The uniform torsional moment T_v comprises a moment in the box $T_{v(f)}$ (subscript f for the French word *fermé* meaning closed) and a moment in the slab $T_{v(o)}$ (subscript o for open), which is considered to be an element of moderate thickness. The moments are proportional to the rigidities GK_f and GK_o, considered with respect to steel using a modular ratio m. Consider equilibrium in the section:

$$T_v = T_{v(f)} + T_{v(o)} \tag{11.10}$$

Also taking into account that rotation of the section about its x axis is assumed to take place without distortion of the cross section, $\varphi(x) = \varphi_f(x) = \varphi_o(x)$, the distribution of torsional moments is as follows:

$$T_{v(f)} = \frac{K_f}{K_o + K_f} \cdot T_v \qquad T_{v(o)} = \frac{K_o}{K_o + K_f} \cdot T_v \tag{11.11}$$

$$K_f = \frac{4\Omega^2}{2\frac{h}{t_w} + \frac{s}{t_f} + m\frac{s}{h_c}} \qquad K_o = \frac{1}{3}\frac{(2b)}{m}h_c^3 \tag{11.12}$$

h	:	average depth of the closed section
s	:	width of the box section
t_w	:	thickness of the webs of the box
t_f	:	thickness of the bottom flange of the box
$2b$	:	concrete slab width
h_c	:	thickness of the concrete slab

Figure 11.10 also shows the shear stresses that correspond to the resistance in uniform torsion. In the part of the concrete slab that closes the box, the total shear stresses τ can be calculated using Equation (11.13). They are maximal at the top and the bottom of the concrete slab.

$$\tau = \tau_f + \tau_o = \frac{T_{v(f)}}{2\Omega h_c} + \frac{T_{v(o)}}{mK_o}h_c \tag{11.13}$$

τ_f	:	shear stresses due to $T_{v(f)}$ in the closed section
τ_o	:	shear stresses due to $T_{v(o)}$ in the open section

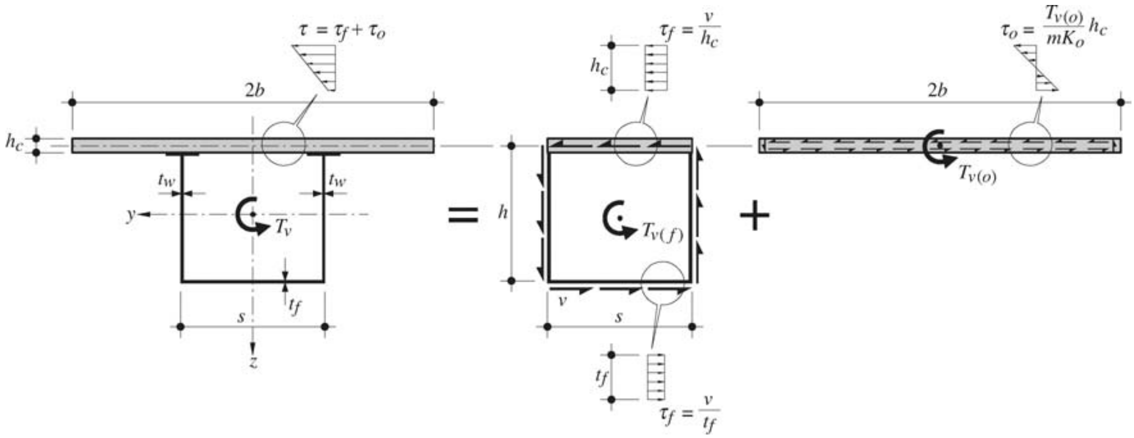


Fig. 11.10 Closed composite cross section resisting in uniform torsion.

Within both the webs and the bottom flange of the steel box, the shear forces are calculated using the first term in Equation (11.13) and the corresponding thicknesses t_w and t_f. In the slab cantilevers the shear stress is calculated using the second term in Equation (11.13).

11.3.3 Non-uniform Torsion

This mode of resistance can only develop if the cross section wishes to warp when subjected to a torsional moment, and this warping is prevented or limited by adequate support conditions. Reference [11.2] contains more information concerning the modes of resisting torsion of different cross sections according to their geometry.

A comprehensive study of non-uniform torsion is outside the scope of the present book, however the subject will be presented in the context of its relevance to bridge design. More information on non-uniform torsion is given in [11.2], as well as in TGC Volume 10, Paragraph 4.5.3.

Differential Equations

In addition to the requirements given in Paragraph 11.3.2, the following assumptions are made:

- the normal longitudinal stresses are uniform over the thickness of the walls (thin walls),
- the shear stresses are assumed to be uniform over the thickness of the walls, and
- shear deformations can be ignored.

As a reminder of the theory, consider an I-section bar that is built-in at one end and subjected to a torsional moment at its free end (Fig. 11.11(a)). Because warping of the section is partially prevented, the bar resists partially in non-uniform torsion. The translations $v(x)$ of the upper and lower flanges correspond to their displacements due to in-plane bending moments M_τ (Fig. 11.11(b)). The normal σ_w and shear τ_w stresses, which are related to the translations of the flanges, represent the stress state due to the non-uniform mode of torsional resistance (Fig. 11.11(c) and (d)).

The resultant of the shear stresses in each flange is equivalent to a force V_τ (Fig. 11.11(b)), and the couple of these two forces is the moment resistance in non-uniform torsion T_w, which is expressed by the following equation:

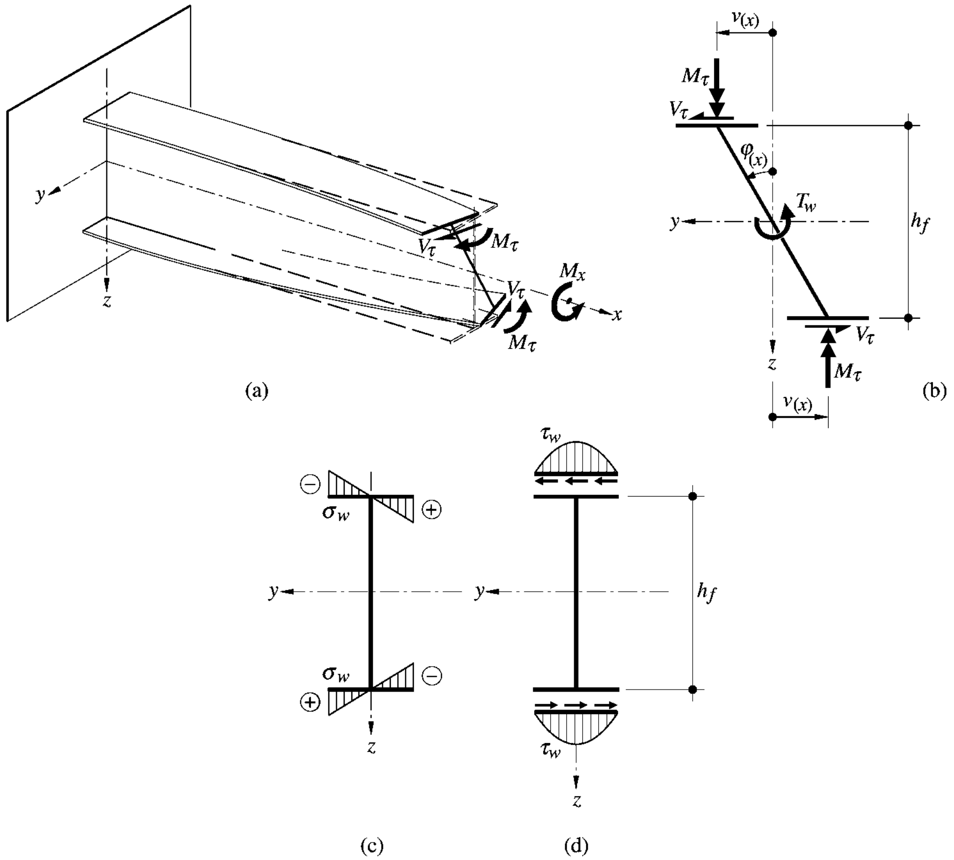


Fig. 11.11 Non-uniform torsion resistance: (a) deformation of the beam, (b) deformation of the section, (c) normal stresses, and (d) shear stresses.

$$T_w = V_\tau h_f \tag{11.14}$$

h_f : depth measured between the mid-thickness of the flanges

It is noteworthy that the cross section is only subjected to a torsional moment M_x, and this is balanced by the two opposing local bending moments M_τ. These moments also create normal stresses σ_w in the section. To characterise this stress state, which is the result of restricting warping of the cross sections, it is useful to define the warping moment, also known as the *bimoment*, M_ω. In the case of an I section, the warping moment is defined by the couple of the two bending moments M_τ.

$$M_\omega = M_\tau h_f \tag{11.15}$$

By analogy with simple bending it is possible to demonstrate that the shear forces V_τ acting in each flange of the beam are obtained from the derivative of the bending moments M_τ acting on each flange. The differential equation concerning the bending deformation of each flange is expressed as follows:

$$\frac{d^2v(x)}{dx^2} = -\frac{M_\tau}{EI_{fz}} \tag{11.16}$$

$v(x)$: deformation of the flange with respect to the y axis
I_{fz} : second moment of area of the flange about the z axis

Along with the hypothesis that the cross sections do not distort (rigid body rotation, Fig. 11.11(b)):

$$v(x) = \varphi(x)\frac{h_f}{2} \tag{11.17}$$

One can determine the differential equation for the shear force V_τ:

$$V_\tau = \frac{dM_\tau}{dx} = -\frac{d^3v(x)}{dx^3}EI_{fz} = -\frac{d^3\varphi(x)}{dx^3}EI_{fz}\frac{h_f}{2} \tag{11.18}$$

Equation (11.14) can then be reformulated as follows:

$$T_w = -\frac{d^3\varphi(x)}{dx^3}EI_{zf}\frac{h_f^2}{2} \tag{11.19}$$

Simplification can be achieved by considering the sectorial second moment of area I_ω, which for an I section (the general case is treated later) gives:

$$I_\omega = I_{fz}\frac{h_f^2}{2} \tag{11.20}$$

The following differential equation for non-uniform torsion is then obtained:

$$T_w = -\frac{d^3\varphi(x)}{dx^3}EI_\omega \tag{11.21}$$

With (11.16), (11.17), and (11.20), it is also possible to express a differential equation for the warping moment:

$$M_\omega = -\frac{d^2\varphi(x)}{dx^2}EI_\omega \tag{11.22}$$

Analogy with Bending

The torsional moment resistance T_w is therefore derived, with respect to x, from the warping moment M_ω. This is logical because V_τ is the derivative of the bending moment M_τ. Also, the variation in torsional moment along a length dx of a bar is equal to the torque m_T acting along dx, and this torque is itself directly proportional to a load q at eccentricity y_q. This situation for non-uniform torsion is very analogous, in terms of the equilibrium between loads and internal moments and forces, to that of bending, as illustrated in Table 11.12.

The diagram of warping moments M_ω may also be derived from that for torsional moment resistance T_w by integration, or by analogy with bending for a structural form that is analogous in bending and non-uniform torsion.

Table 11.12 Analogy between bending and non-uniform torsion.

Bending		**Non-uniform Torsion**	
Vertical load	$q_z = -\,\mathrm{d}V_z/\mathrm{d}x = -\,\mathrm{d}^2M_y/\mathrm{d}x^2$	Torque	$m_T = -\,\mathrm{d}T_w/\mathrm{d}x = -\,\mathrm{d}^2M_\omega/\mathrm{d}x^2$
Deflection	$w(x)$	Rotation	$\varphi(x)$
Shear force	$V_z = \mathrm{d}M_y/\mathrm{d}x$	Torsional moment	$T_w = \mathrm{d}M_\omega/\mathrm{d}x$
Bending moment	$M_y = -\,EI_y\,\mathrm{d}^2w(x)/\mathrm{d}x^2$	Warping moment	$M_\omega = -\,EI_\omega\,\mathrm{d}^2\varphi(x)/\mathrm{d}x^2$

Calculation of Stresses

The distribution of the stresses σ_w may be determined by considering the analogy with bending. For an I section the moments M_τ shown in figure 11.11(b) lead to stresses in each flange, for $z = -\,h_f/2$, of:

$$\sigma_w(y) = \frac{M_\tau}{I_{fz}}y = \frac{M_\omega}{h_f}\cdot\frac{h_f^2/2}{I_\omega}y = \frac{M_\omega}{I_\omega}\cdot\frac{h_f\cdot y}{2} = \frac{M_\omega}{I_\omega}\omega(y) \tag{11.23}$$

w : normalised sectorial coordinate of the section [m^2]

For the example of a beam that is built-in at one end (Fig. 11.11(a)), the warping moment M_ω has a negative value when the torsional moment resistance T_w is positive. Because the magnitudes of M_ω and I_ω are constant for a given section, the distribution of stresses σ_w only depends on the sectorial coordinate $w(y, z)$:

$$\sigma_w(y, z) = \frac{M_\omega}{I_\omega}\omega(y, z) \tag{11.24}$$

The normalised sectorial coordinate and the sectorial second moment of area I_ω are defined below for the general case applicable to any form of open cross section.

The analogy with bending may also be used when considering shear stresses τ_w, as shown in Figure 11.11 (d). Equation (11.25) allows the distribution of these stresses to be calculated for an I section (using the first equality) and for the general case (second equality).

$$\tau_w(y, z) = \frac{V_\tau S_z(y, z)}{I_{fz}t(y, z)} = \frac{T_w S_\omega(y, z)}{I_\omega t(y, z)} \tag{11.25}$$

S_z : first moment of area of the part of the flange under consideration with respect to the z axis
t : flange thickness
S_ω : sectorial first moment of the part of the flange under consideration (Equation (11.29))

Sectorial Coordinates

The sectorial coordinate Ω [m^2] of the section, which should not be confused with the cross section area of the closed section as defined in 11.3.2, is the basis for the normalised sectorial coordinate and, indeed, all the other geometric variables defined below. It is defined in Figure 11.13, with origin O and curvilinear abscissa s:

$$\Omega(s) = \int_O^s h_C(s)\mathrm{d}s \tag{11.26}$$

The normalised sectorial coordinate ω is defined by the following relationship, A being the area of the cross section:

$$\omega(s) = \Omega(s) - \Omega(s)_{\text{moy}} = \Omega(s) - \frac{1}{A}\int_A \Omega(s)\mathrm{d}A \tag{11.27}$$

This normalised sectorial coordinate differs from the sectorial coordinate Ω in the choice of centre of gravity of the elements $w \cdot \mathrm{d}A$ as the origin $\overline{O}$ of the curvilinear abscissa $\bar{s}$ (Fig. 11.13). This choice means that, by definition, one has:

$$\int_A \omega(\bar{s})\mathrm{d}A = 0 \tag{11.28}$$

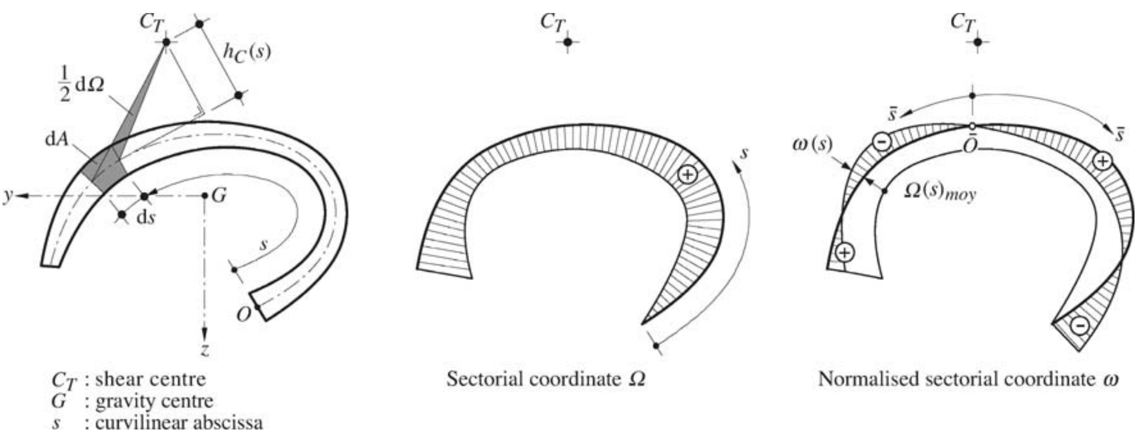


Fig. 11.13 Sectorial coordinate Ω and normalised sectorial coordinate ω of an open thin walled cross section.

The sectorial first moment of area is defined by:

$$S_\omega(s) = \int_O^s \omega(s)\mathrm{d}A \tag{11.29}$$

The sectorial second moment of area is defined by:

$$I_\omega = \int_A \omega^2(s)\mathrm{d}A \tag{11.30}$$

The normalised sectorial coordinate ω and the sectorial first moment of area S_ω are shown schematically in Figure 11.14 for the example of a twin-girder bridge. The diagram of *normalised* sectorial coordinates ω is directly proportional to the stress σ_w according to Equation (11.24), while the sectorial first moment of area S_ω is proportional to τ_w according to Equation (11.25).

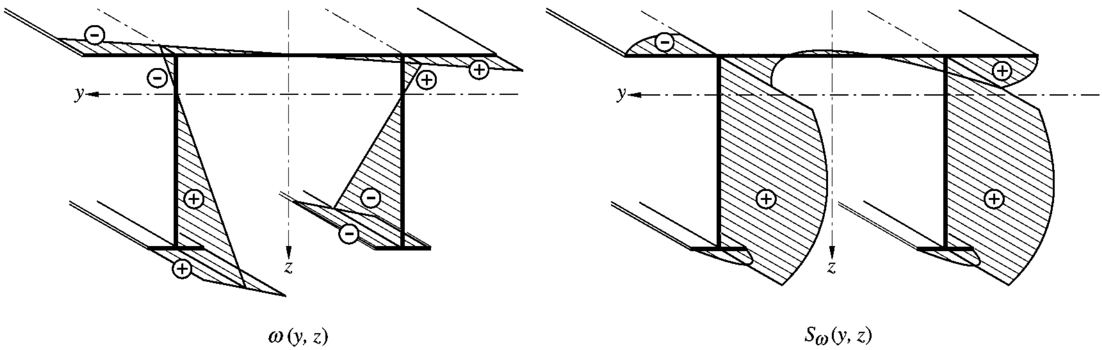


Fig. 11.14 Normalised sectorial coordinate ω and sectorial static moments S_ω of a bridge with an open section.

11.3.4 Combined Torsion

As noted in Paragraph 11.3.1, a bar resists simultaneously in uniform and non-uniform torsion, or, in other words, it resists in combined torsion. The relative importance of each mode of resistance is determined by the structural form, the support conditions, and the relative stiffness of the bar in each mode.

For a given applied torque, a *closed* section distorts much less than an open section. Therefore, a closed section, which is very stiff against uniform torsion, twists very little in order to develop the shear stresses that are needed to balance the torsional moment in uniform torsion. The normal and shear stresses resulting from the small amount of warping restraint are very small.

On the other hand, an *open* section – relative to the support conditions and the continuity at supports that generally limit this movement – requires considerable distortion to develop the necessary stresses in uniform torsion. However, restricting warping of the cross section results in significant normal stresses σ_ω and hence shear stresses τ_ω due to non-uniform torsion, while the stresses due to uniform torsion remain small.

However, an open section can resist primarily in uniform torsion if it has the appropriate support conditions, for example if the section is completely free to warp so that normal stresses σ_ω cannot develop. On the other hand, a closed section that has very little freedom to warp (section situated close to a perfectly built-in support) will resist primarily in non-uniform torsion.

Consideration of the general case for combined torsion allows the governing differential equation to be defined. Combining Equations (11.2), (11.3), and (11.21) leads to expression (11.31), which relates the torsional moment T to the rotation φ of the bar:

$$T = T_v + T_w = GK\frac{d\varphi(x)}{dx} - EI_\omega\frac{d^3\varphi(x)}{dx^3} \tag{11.31}$$

Taking the results of equilibrium considerations for a small bar element, according to (11.1), and replacing M_x by T:

$$m_T = -\frac{dT}{dx} \tag{11.32}$$

Introducing (11.32) into the derivative of (11.31) leads to the differential equation for *combined* torsion:

$$m_T = -GK\frac{d^2\varphi(x)}{dx^2} + EI_\omega\frac{d^4\varphi(x)}{dx^4} \tag{11.33}$$

This is an inhomogeneous fourth order equation with constant coefficients. For simple structural forms and loads, it can be resolved analytically. For a simple beam, four boundary conditions are needed, such as the imposed displacements, the support conditions with respect to torsion, or the torsional moment resistance T defined at a number of sections. The two components of the torsional resistance, namely T_v and T_w, can be derived from the expression for rotation φ using (11.31). For more complex cases design curves can be used to determine the contributions of uniform and non-uniform torsion as a function of the bar geometry (length and cross section) and the loading [11.2]. An example of such design curves is shown in Figure 11.15. The parameter κ is a characteristic of the stiffness of the bar in uniform torsion GK relative to the stiffness in non-uniform torsion EI_ω.

$$\kappa = l\sqrt{\frac{GK}{EI_\omega}} \tag{11.34}$$

l : length of bar between two torsional supports

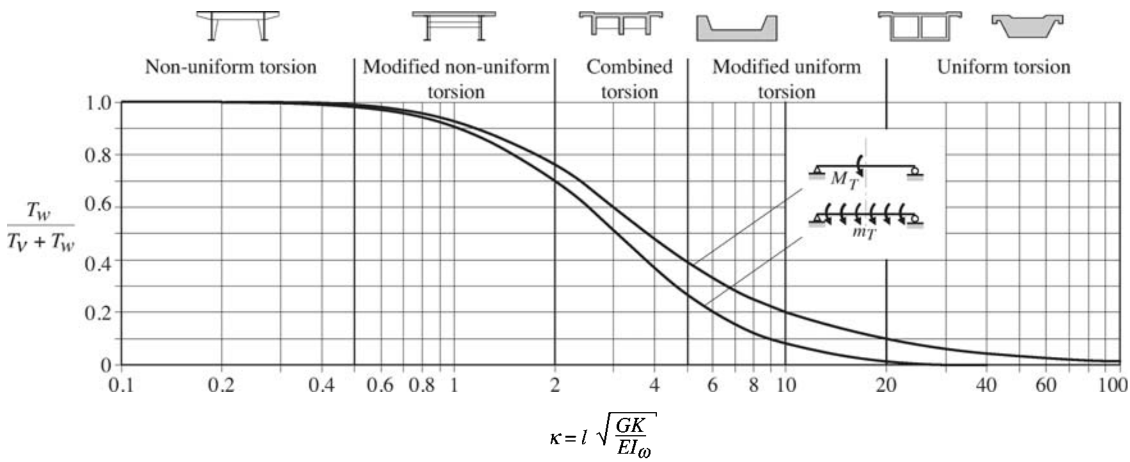


Fig. 11.15 Contribution of the warping moment resistance T_w to the total torsional resistance [11.2].

It is clear from Figure 11.15 that short bars with an open cross section will resist primarily in non-uniform torsion, while long bars with closed sections will resist primarily in uniform torsion. Between these two extremes, resistance will be in combined torsion. In the following sections, composite box girder bridges with appropriate cross bracing (§ 11.4.1) will be assumed to resist in uniform torsion. Twin-girder open section composite bridges will be assumed to resist primarily in non-uniform torsion, although the resistance of the slab in uniform torsion will be taken into account (§ 11.5.1). Therefore, they will be analysed as structures that resist in combined torsion, or more precisely modified non-uniform torsion.

More complex theories, such as that of Benscoter presented in [11.4], take into account the deformation energy due to the shearing that occurs during warping. This allows the real behaviour of bridges that resist in combined torsion to be analysed more accurately than is possible with the theory presented above. Benscoter's theory also gives a more complete insight into the way in which a structure resists torsion, which allows the limits of uniform torsion (Saint-Venant's theory) and non-uniform torsion (warping theory) to be better defined.

11.4 Straight Bridges with a Closed Cross Section

The transverse cross section of a so-called closed section bridge comprises either a steel box that is completely closed, or a U-shaped steel box that is described as "open" but is in fact closed on its upper face by a concrete slab (§ 5.5.2). This type of bridge is particularly good at resisting torsional moments, primarily in uniform torsion, and due to its high torsional stiffness deforms very little under this type of loading.

11.4.1 Torsional Behaviour

Bridges with a closed cross section when subject to a torsional moment resist primarily in uniform torsion, with a constant flow of shear stress in the box walls. Because the box is not generally circular, it is essential that cross bracing is present in order to mobilise this mode of torsional resistance. The cross bracing fulfils an essential function in ensuring that the shape of the cross section will not distort under loading, which is a necessary condition if application of the theory of uniform torsion is to be valid. If the cross bracings are too widely spaced, or even missing completely, the cross section will distort substantially and in a way that can only be realistically modelled using theories that are more complex than the simple statics of bars. Such models take into account the three dimensional behaviour of the structure, as, for example, the theory of folded shells and plates. To justify the use of statics of bars, and Saint-Venant's torsion theory, there must be sufficient cross bracing along the length of the beam (§ 5.6.1).

Figure 11.16 shows the deformation of a box section at the cross section where an eccentric load Q is introduced. The cross section is at the same time assumed to be completely resistant to distortion (it cannot deform), or totally free to distort in the absence of cross bracing. This figure also shows the distribution

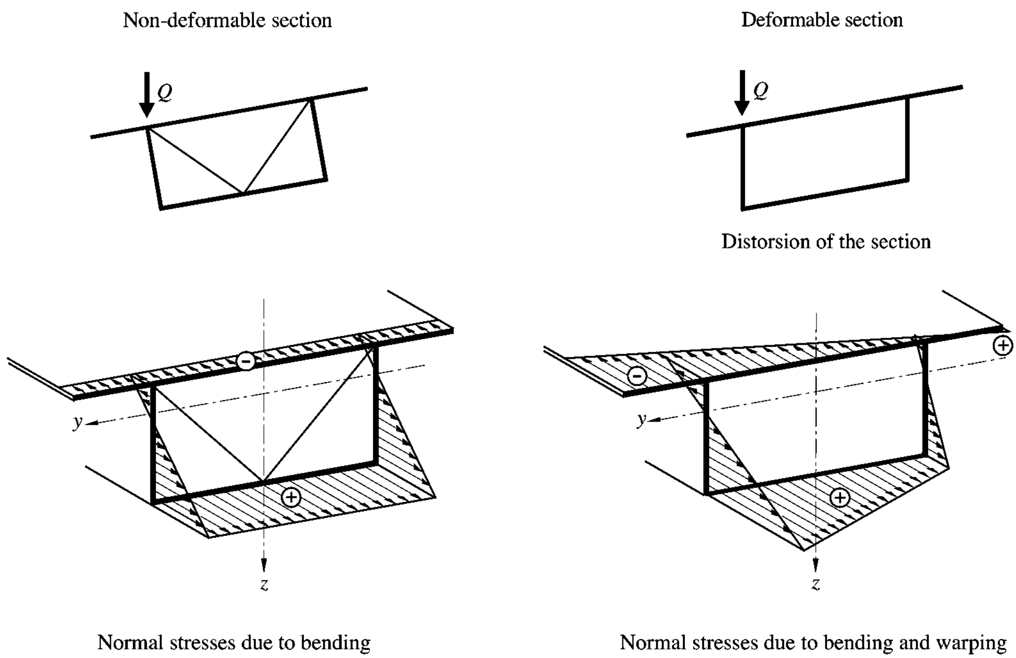


Fig. 11.16 Influence of cross bracing on the deformability of a closed section and the corresponding normal stresses.

of normal stresses acting in the box walls for these two extreme cases. It can be clearly seen that a section devoid of cross bracing to prevent its distortion will, indeed, distort and develop associated normal stresses due to warping. Consequently, the distribution of normal stresses will differ from those due to bending (calculated using simple statics). In reality, for a box section bridge with typical cross bracing, the normal stresses will be close to those for a section that does not distort, even though there is a certain amount of distortion present. This distortion will be very limited if there is a sufficient number of cross braces, each having sufficient in-plane stiffness. This phenomenon may then be ignored when designing the beam (§ 14.5.2).

Once the torsional moments M_x are known along the length of the beam, Equations (11.6) and (11.13) may be used to calculate the shear stresses resulting from the shear flow in each part of the box section, by setting M_x equal to T_v. The designer must not forget to add to these shear forces those that are due to vertical shear (due to bending) acting on the box section webs.

11.4.2 Calculation of the Internal Moments and Forces

Section 11.2, covering the modelling of beam bridges, contains all the information needed to calculate the bending moments and shear forces for this type of structure. The torsional moments that arise due to loads or forces that act eccentrically to the longitudinal axis of the bridge may be calculated using the information given in Section 11.2.4. The beam supports will provide torsional restraint if the box section is supported on at least two points for a given cross section.

The longitudinal and transverse distribution of the imposed loads, used to calculate the maximum values of the internal moments (torsion and bending), may be different, depending on which particular effect is being considered. Figure 11.17 shows the different positioning of a distributed load q and the internal moments, of which the maximum values are not necessarily at the same cross sections:

- The *torsional* moment is greatest at the supports. The slab is loaded over its half width b (for this example each line of support is assumed to be a torsional support).
- The *bending* moment is greatest at the intermediate supports. It may also be seen that for a box girder, it makes no difference whether one calculates the bending moments for a half box or the whole box, considering load over either half the slab width or its total width (Fig. 11.17).

For a *skew* bridge the effects of torsion and bending are influenced by the fact that the beam is partially built-in as a result of the skew at supports. Paragraph 11.6.2 discusses how this effect should be taken into account.

When a bridge is *curved*, only the shear forces can be calculated as they would be for a rectilinear bridge. The bending and torsional moments are both affected by the curvature of the structure. The effects of curvature on the internal moments and forces is considered in Paragraph 11.7.3.

11.5 Straight Bridges with an Open Cross Section

The cross section of a bridge with a so-called open cross section comprises two or more beams. These are normally plate girders with solid webs (§ 5.5.1). Because such types of structure resist primarily in non-uniform torsion when subjected to eccentric loads, the main difficulty during analysis is to determine the sectorial characteristics of the cross section (§ 11.3.3). In the remainder of this Section 11.5, particular attention is paid to twin-girder composite bridges, since they represent a major proportion of the open section bridges in Europe.

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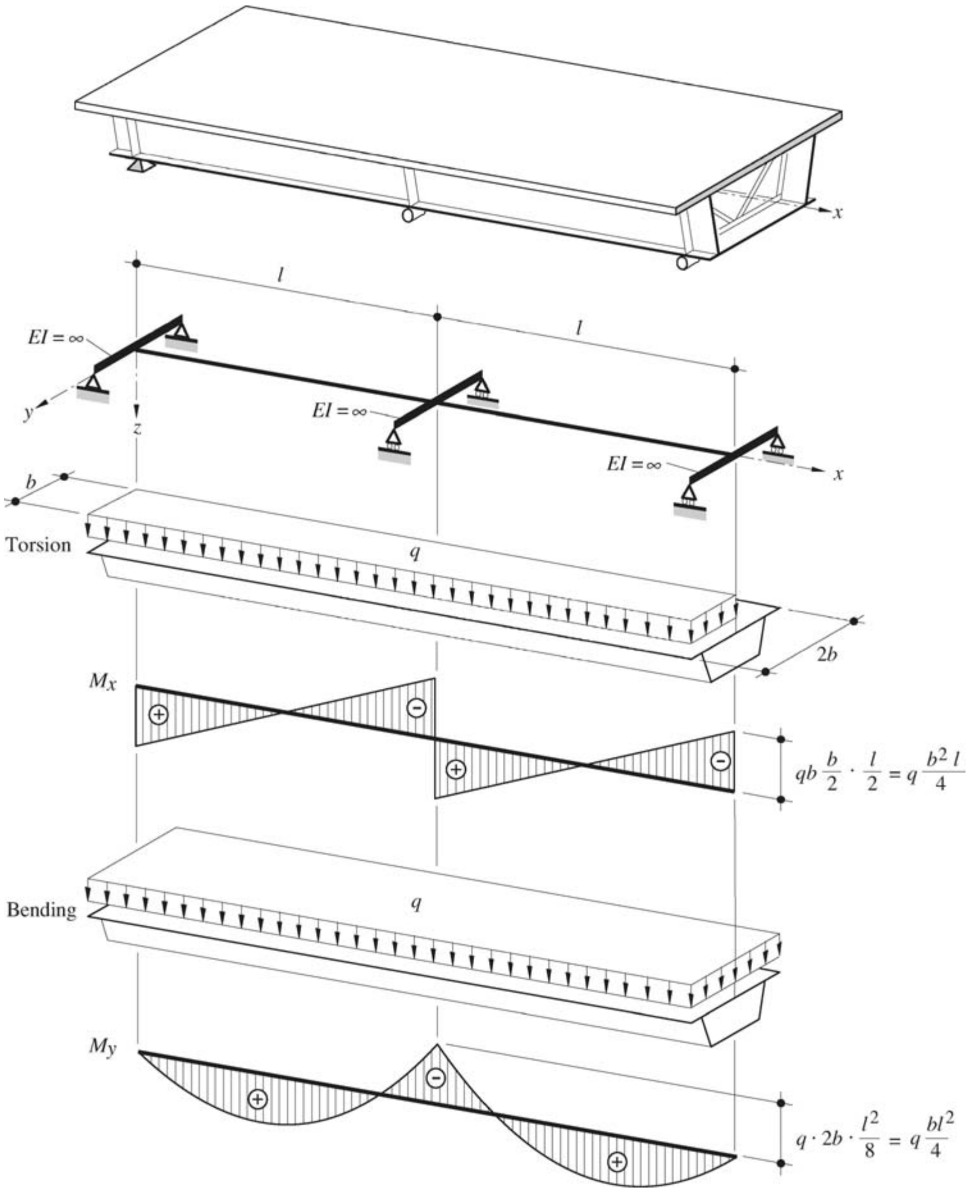

Fig. 11.17 Position of loads and diagrams of torsional and bending moments for a closed section bridge.

In order to avoid the need to calculate the sectorial characteristics (S_ω, I_ω), use will be made of the ***transverse influence line*** for the loads. This simplification allows an equivalent load to be defined which results in the same bending stresses in the beam cross sections as those resulting from bending and non-uniform torsion of the open section. This question is initially considered in Paragraph 11.5.2, allowing subsequent definition of where loads should be placed on the structure prior to its analysis in bending.

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12 Steel beams

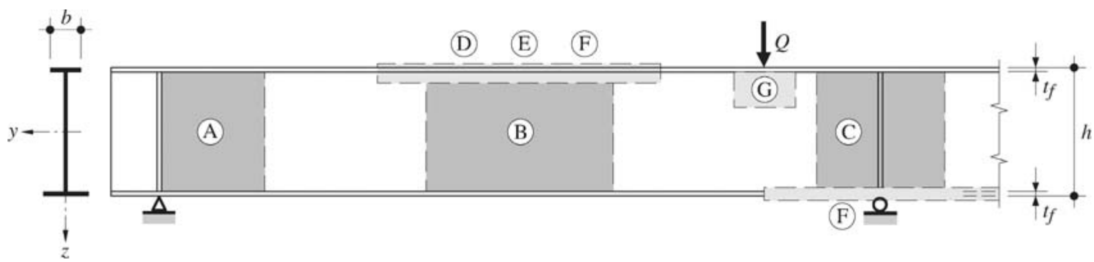
Box-girder beam bridge, Viaduc du Bois de Rosset, near Avenches (CH).
Eng. Bureau d'ingénieurs DIC, Aigle.
Photo ICOM.

12.1 Introduction

The main beams and other major structural elements used in bridges are fabricated plate girders or box girders, trusses, or even rolled sections for short span beams. This chapter deals exclusively with the structural design of fabricated main beams. The design of trusses is covered in TGC Volume 10, Section 5.7, and TGC Volume 11, Section 12.3. Rolled sections are considered in TGC Volume 10, Section 5.3.

The aim of this chapter is to define the ultimate resistance of plate girders and box girders, subject to bending and torsion. Emphasis is given to checking structural safety, whereas serviceability is covered in Chapter 13 in the context of composite steel-concrete bridges.

Several failure modes may be envisaged for a continuous plate girder. Figure 12.1 shows the various important regions along the length of such a beam, where design checks are needed. However, every cross section of a beam must satisfy the conditions associated with structural safety.



(A) Shear resistance

(B) Bending resistance

(C) Resistance to combined bending and shear

(D) Vertical buckling of the compression flange into the web

(E) Rotational buckling of the compression flange (local buckling)

(F) Lateral torsional buckling of the beam (lateral buckling of the compression flange)

(G) Resistance to point load (*patch loading*)

◯ Fatigue resistance at every construction detail under repeated loads

Fig. 12.1 Important regions of a plate girder.

First of all, the various possible failure modes for a beam are examined along with corresponding checks that must be executed. Resistance to bending moment (Sect. 12.2) and shear force (Sect. 12.3), as well as the interaction between the two (Sect. 12.4) are then considered. Theories for lateral torsional buckling and local buckling of thin plates, which are highly relevant for this type of beam, are covered in TGC Volume 10, Chapters 11 and 12. The theoretical basis for each of these stability phenomena is not repeated here. However, a model for shear resistance of plate girders is developed in this chapter.

The structural design of transverse and longitudinal stiffeners is also covered since these elements are an integral part of plate girders (Sect. 12.6). They represent structural elements that are indispensible for mobilising the post-buckling resistance of thin plates subject to compression and shear.

Checking positions of concentrated load introduction (Sect. 12.5) and checking fatigue safety (Sect. 12.7) complete the list of things that a designer must consider when assuring the use of a plate girder in a bridge application. The principles and particularities of the fatigue behaviour of steel structures, as well as the basis of fatigue checks, are covered in TGC Volume 10, Chapter 13, and are not repeated in this book.

The design checks (verifications) associated with these various phenomena are described in the remainder of this chapter, so that it reflects a logical sequence for the designer. Having developed the concept for the load carrying structure of a beam bridge, the engineer ensures, first of all, that the dimensions of the webs and flanges fall within the limits dictated by: the ability of the web to resist buckling of the compression flange into it, local buckling of the flange, and lateral torsional buckling of the beam. These forms of instability are likely to develop before the ultimate resistance of the section to a bending moment and/or shear can develop. The designer then determines the dimensions of the stiffeners or, if they are not needed, looks at the issue of concentrated load introduction. He finally ensures that the steel beam provides an acceptable level of safety against fatigue.

To complete this chapter, Section 12.8 covers some particular aspects of box girders, where the calculation of its resistance and the associated design checks differ from those for plate girders.

12.2 Bending Resistance

12.2.1 Introduction

When the web and compression flanges of a plate girder satisfy both the limiting slenderness criteria and limits for lateral torsional buckling (class 3 section and EE calculation method according to the standard SIA 263), the ultimate bending resistance M_R is equal to the elastic resistance M_{el} as defined in TGC Volume 10, Section 4.3.

$$M_R = M_{el} = f_y W_{el} \tag{12.1}$$

f_y : yield strength of the structural steel
W_{el} : elastic section modulus

For the singly symmetric cross sections that are typically used in bridges, $W_{el,y}$ corresponds to the elastic modulus relative to the y axis, calculated relative to the centre of gravity of the flange furthest away from the neutral axis. Strictly speaking the elastic section modulus W should be calculated relative to the most extreme fibre from the neutral axis, but for plate girders it is acceptable to consider the mid-thickness of the flanges. Even though the *EE calculation* method is adopted, this simplification means that some plastification at the flange extremities is tolerated.

For plate girders that satisfy the slenderness limits for class 1 or 2 sections, the ultimate bending resistance may be determined, as for rolled sections, using the guidance given in TGC Volume 10, Chapters 4 and 5. An *EP calculation* method is adopted.

When either the web or the compression flanges of a plate girder are too slender, and belong to class 4, the ultimate bending resistance is defined using an elastic calculation, but with a reduced area for either the web or the compression flanges. An *EER calculation* method is adopted. The principles of using a reduced area for members in compression are based on local buckling theory, and the hypotheses proposed by von Karman, as presented in TGC Volume 10, Chapter 12.

In addition to the fact that the bending resistance may be limited by local buckling of either the compression flange or the part of the web in compression, other instability phenomena may occur before the resistance M_{el} (Fig. 12.2) can be reached. These mainly concern the compression flange:

- ***vertical buckling*** of the compression flange into the web,
- ***rotational buckling*** of the compression flange (local buckling),
- ***lateral torsional buckling*** of the beam (lateral buckling of the compression flange).

How these three instability phenomena may be taken into account is considered below, before going on to consider how to check the bending resistance itself of a plate girder.

12.2.2 Vertical Buckling of the Compression Flange into the Web

When a beam is subjected to bending, the vertical curvature gives rise to transverse forces in the flanges. These forces are due to the change in direction of the normal forces N_f acting in the flanges (Fig. 12.2(a)). These forces, which are known as *deviation forces*, act perpendicularly to the flanges and result in compression stresses σ_z in the web. These compression stresses σ_z, which are assumed uniform for an element of beam of length dx and illustrated in Figure 12.2(a), are defined as follows:

$$\sigma_z = \frac{\sigma_f\, A_f}{r\, t_w} \tag{12.2}$$

σ_f : compression stress in the web
A_f : area of the compression flange
r : radius of curvature
t_w : web thickness

By making the following simplifying assumptions:

- the compression flange is totally plastified when instability occurs: $\sigma_f = f_y$
- the cross section of the pate girder is symmetric: $r = h_f / 2\varepsilon_y$
- the residual compressive stresses in the compression flange are equal to $0.5 f_y$, which means that the compression flange must be able to support a strain of: $\varepsilon_y = 1.5\, f_y / E$

Then Equation (12.2) becomes:

$$\sigma_z = 3 \cdot \frac{f_y^2}{E} \cdot \frac{A_f}{A_w} \tag{12.3}$$

A_f : area of the compression flange
A_w : "conventional" web area: $A_w = h_f t_w$
f_y : yield strength of the compression flange steel

If the web is particularly slender, it may buckle under the compression stresses σ_z and the compression flange, which is then no longer supported vertically by the web, and may buckle into the web. Considering the web to be a plate in compression, which behaves like a strut of length equal to the web depth h_f (Euler buckling of a column ignoring material and geometric imperfections), the critical stress σ_{cr} is given by:

$$\sigma_{cr} = \frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{h_f}\right)^2 \tag{12.4}$$

h_f : depth measured between the mid-thickness of the flanges
υ : Poisson's ratio, $\upsilon = 0.3$ for steel

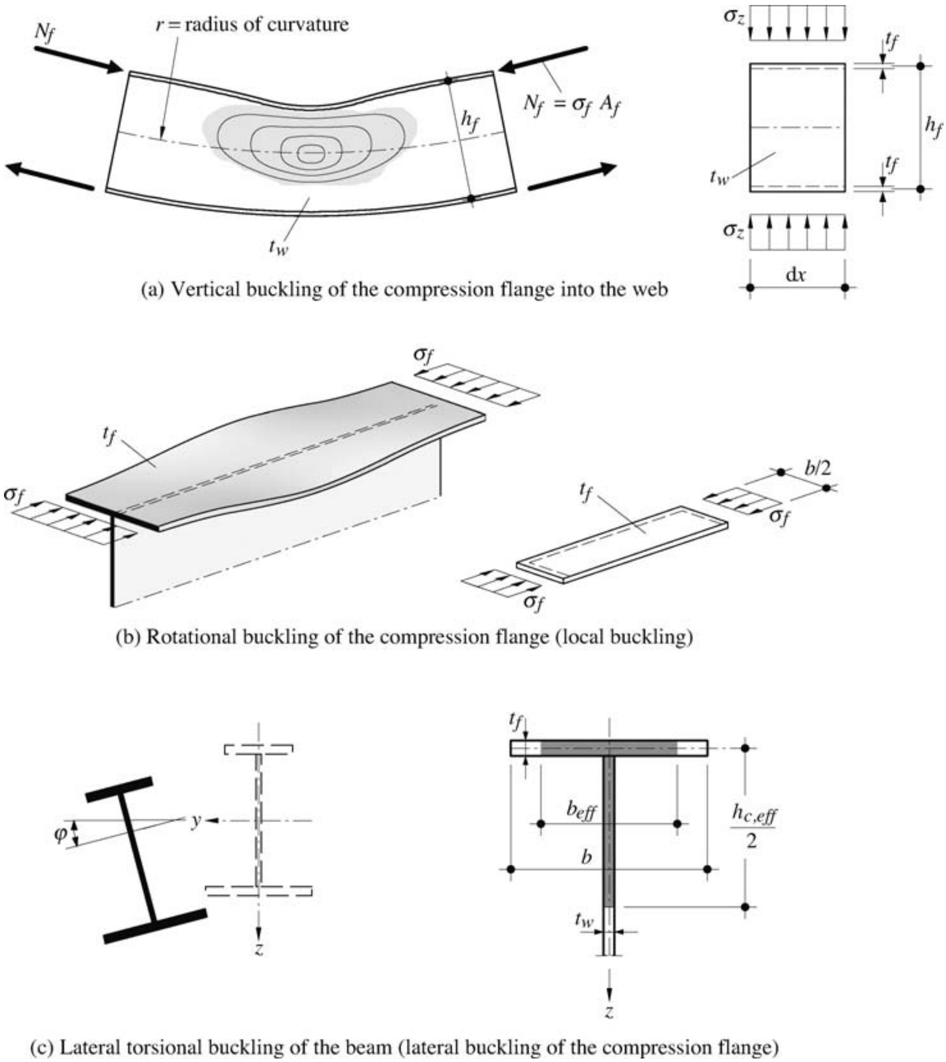


Fig. 12.2 Possible instability phenomena for a flange in compression.

To avoid failure by buckling of the compression flange into the web, the following condition must be satisfied: $\sigma_z \leq \sigma_{cr.}$ Considering this condition in combination with Equations (12.4) and (12.3), a slenderness limit for the web can be determined:

$$\frac{h_f}{t_w} \leq 0.55 \frac{E}{f_y} \sqrt{\frac{A_w}{A_f}} \tag{12.5}$$

Assuming a minimum ratio of 0.5 between the area of the web A_w and that of the flange A_f (practically the ratio is rarely less than this amount), Equation (12.5) becomes:

$$\frac{h_f}{t_w} \leq 0.40\frac{E}{f_y} \tag{12.6}$$

Resulting in:

- $h_f/t_w \leq 360$ for S235 steel,
- $h_f/t_w \leq 240$ for S355 steel.

Similar values of slenderness limits for webs in compression are specified in most codes and standards. They are valid for straight plate girders, free from loading applied in the plane of the web, and with no longitudinal stiffeners. When longitudinal stiffeners are present, as a simplification, these limiting values may be applied to each web panel.

12.2.3 Rotational Buckling of the Compression Flange

When the compression flange of a plate girder is particularly slender, it may fail by rotating about its connection to the web, as shown schematically in Figure 12.2(b). Because of the major role the flanges play in the bending resistance of a beam, it is essential to ensure that the geometry of these elements is such that instability does not occur, such that the total area of the compression flange can participate in the bending resistance of the beam.

Failure by rotation of a compression flange can be modelled as local buckling of a thin plate in compression that has two free edges and a pinned connection to the web. Because the web to which this flange is connected is normally slender itself, it should not be considered as providing any rotational restraint. Each half flange should therefore be taken as hinged where it is joined to the web, so the buckling coefficient k is taken as 0.426 (TGC Vol. 10, Table 12.7). It may be assumed that buckling by rotation will not govern if the ratio between the half flange width $b/2$ and its thickness t_f is such that yield can be reached in the whole of the flange (in other words, the total area of the compression flange contributes to the bending resistance). In this case, the calculated effective width $b_{eff}/2$, therefore, must be at least equal to, or bigger than, the half width $b/2$ of the flange. This can be expressed in applying, by analogy, Equation (12.24) from TGC Volume 10 with the condition that $\overline{\lambda_p} \leq 0.9$

$$\frac{(b/2)}{t_f} \leq 0.9\frac{\sqrt{k}}{1.052}\sqrt{\frac{E}{f_y}} \tag{12.7}$$

$\overline{\lambda_p}$	:	non-dimensional slenderness, $\overline{\lambda_p} = \sqrt{f_y/\sigma_{cr}}$
σ_{cr}	:	elastic critical buckling stress from TGC Volume 10, Equation (12.8)

from which the following expression is determined:

$$\frac{(b/2)}{t_f} \leq 0.56\sqrt{\frac{E}{f_y}} \tag{12.8}$$

Resulting in:

- $(b/2)/t_f \leq 17$ for S235 steel,
- $(b/2)/t_f \leq 14$ for S355 steel.

If the slenderness of the compression flange falls outside these limits, which would imply an inappropriate choice of dimensions, then only the effective area of the flange is taken into account when considering lateral torsional buckling and bending resistance.

The effective width $b_{eff}/2$ is defined as:

$$\frac{b_{eff}}{2} = 0.56\sqrt{\frac{E}{f_y}} \cdot t_f \leq \frac{b}{2} \tag{12.9}$$

12.2.4 Lateral Torsional Buckling of the Beam

Reminder of the Theory

The principles developed in TGC Volume 10, Section 11.3, concerning the ultimate resistance of a beam to lateral torsional buckling, may be applied to plate girders. Plate girders are slender and of open cross section, meaning they have limited resistance in uniform torsion (Saint-Venant); hence, they resist primarily in non-uniform torsion (§ 11.3.3). During lateral torsional buckling of such a beam, both the lateral displacement of the compression flange and the corresponding rotation of the cross section (Fig. 12.2 (c)) mobilise this resistance. This means that the stiffness of the section in uniform torsion is generally ignored when considering lateral torsional buckling of the beam. In other words, as the compression flange displaces laterally, the component of resistance in uniform torsion σ_{Dv} may be ignored relative to the component of resistance in non-uniform torsion σ_{Dw}.

Lateral torsional buckling of plate girders, therefore, may be considered as lateral buckling of a strut ($\sigma_{crD} = \sigma_{Dw}$) that has a cross section comprising the effective area of the compression flange and part of the web (12.12). The buckling length is taken as the distance l_D between those supports that prevent lateral movement of the flange. The cross bracing between the main beams may be considered to provide such support, although depending on the in-plane stiffness of these braces, they may not provide full restraint but rather only partial support to the flange. In such cases the lateral torsional buckling length is greater than the distance between adjacent sets of cross bracing. This complication is considered in greater detail at the end of this paragraph.

Because the normal force in the flange usually varies along its length, the lateral torsional buckling length is reduced using a factor η, which allows for the variation in bending moment between the lateral supports (TGC Vol. 10, Fig. 11.16). The reduced buckling length l_K is expressed as:

$$l_K = l_D / \sqrt{\eta} \tag{12.10}$$

The component σ_{Dw} (non-uniform torsion) of the elastic critical stress for lateral torsional buckling is equal to the elastic critical buckling stress of the part of the beam in compression (TGC Vol. 10, Sect. 10.2). It is defined as:

$$\sigma_{crD} = \sigma_{Dw} = \frac{\pi^2 E}{\lambda_K^2} \tag{12.11}$$

λ_K : slenderness of the part of the beam in compression, $\lambda_K = l_K / i_D$

The slenderness λ_K, therefore, corresponds to the slenderness of a strut with a cross sectional area given by:

$$A_D = b_{eff} \cdot t_f + \frac{h_{c,eff}}{2} \cdot t_w \tag{12.12}$$

b_{eff} : effective width of the compression flange, of thickness t_f, according to (12.9)
$h_{c,eff}$: effective depth of the web in compression, with thickness t_w, according to (12.28)

This area comprises the effective area of the compression flange plus the effective part of the web adjacent to the compression flange. This depth of web should not exceed one third of the total depth of web in compression:

$$\frac{h_{c,\,eff}}{2} \leq \frac{h_c}{3} \tag{12.13}$$

The second moment of area of the strut I_D, which is taken into account through the radius of gyration $i_D = \sqrt{I_D/A_D}$ corresponds to the second moment of area about the z axis of the area A_D defined above (see also Fig. 12.2 (c)).

Lateral Torsional Buckling Resistance

The ***lateral torsional buckling stress*** σ_D to consider when calculating the resistance M_D of a plate girder used in a bridge must take into account the structural and geometric imperfections, as well as the yield strength of the steel. It is defined by the following equation:

$$\sigma_D = \chi_D\, f_y \tag{12.14}$$

χ_D : lateral torsional buckling reduction factor, defined by Equation (12.16)

The lateral torsional buckling stress σ_D, which is less than the yield strength of the steel f_y, is used to limit the bending resistance of the beam to its lateral torsional buckling resistance. This stress is often used when checking class 4 plate girders, which have an ultimate bending resistance that is effectively limited by local instabilities, such as local buckling. The stress to be used in such calculations is derived from an addition of stresses determined for different effective cross sections depending on the type of action. This is the case, for example, when checking structural safety of a composite bridge above the intermediate supports (§ 13.4.4). For other cases, the lateral torsional buckling moment resistance M_D is used directly. It is defined as:

$$M_D = \sigma_D\, W_{c,\,eff} \tag{12.15}$$

$W_{c,eff}$: elastic section modulus for the compression flange of the effective cross section according to (12.33)

The reduction factor χ_D is calculated in a way that is analogous to overall buckling:

$$\chi_D = \frac{1}{\Phi_D + \sqrt{\Phi_D^2 - \bar{\lambda}_D^2}} \tag{12.16}$$

Φ_D : factor that takes into account the imperfections of the section (initial geometric imperfections, residual stresses, variations in yield strength) and the non-dimensional slenderness with regard to lateral torsional buckling $\bar{\lambda}_D$:

$$\Phi_D = 0.5[1 + \alpha_D(\bar{\lambda}_D - 0.2) + \bar{\lambda}_D^2\,] \tag{12.17}$$

α_D : imperfection factor that has a value of 0.49 for welded sections, corresponding to buckling curve c

According to the standard SIA 263, the reduction factor χ_D has a value of 1.0 if $\bar{\lambda}_D \leq 0.4$, whereas according to the Eurocodes, this value is only applicable for $\bar{\lambda}_D \leq 0.2$.

In the case of a plate girder (class 4 section), for which resistance in uniform torsion is ignored, the non-dimensional slenderness for lateral torsional buckling $\bar{\lambda}_D$ is given by:

$$\bar{\lambda}_D = \sqrt{\frac{f_y}{\sigma_{crD}}} = \frac{\lambda_K}{\pi}\sqrt{\frac{f_y}{E}} \tag{12.18}$$

If the cross section of a plate girder satisfies the requirements of a class 1, 2, or 3 section, in other words, if the compression elements are totally effective in contributing to the resistance, then the component of resistance in uniform torsion may be taken into account when determining the lateral torsional buckling resistance.

Defining the lateral torsional buckling stress using the reduction factor χ_D (12.14) recognises that the buckling curves, which relate this factor to the non-dimensional slenderness $\bar{\lambda}_D$ (12.16), have been calibrated against numerous tests. The same is true for overall buckling, as the curves are semi-empirical (TGC Vol. 10, § 10.3.3). However, at the time of writing, tests on plate girders as used in bridges have been insufficient. As a result the relevant codes and standards take a prudent approach for such sections in proposing the use of conservative curves, namely curve c as far as the SIA standards are concerned. The Eurocodes go even further by proposing curve d for beams having a total depth that is greater than twice the width of the compression flange.

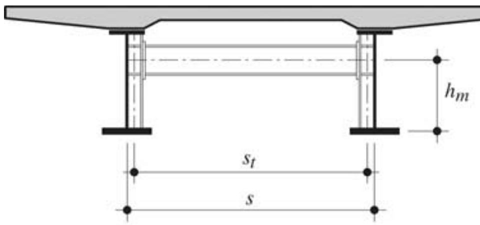
Numerical simulations using the finite element method (taking into account initial geometric imperfections and residual stresses due to welding and cutting [12.1]) have shown that lateral buckling behaviour is favourably influenced by tensile residual stresses due to the cutting of plates. These tensile stresses are found at the plate edges and, therefore, contribute to improving the lateral buckling behaviour of compression flanges. These favourable effects are the subject of current research, but are not yet included in the design checks proposed in the relevant codes and standards.

Influence of the Stiffness of the Lateral Supports to the Compression Flange

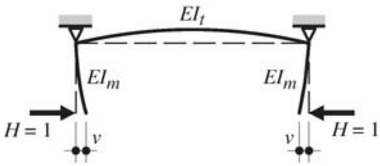
The cross bracing, which links the main beams of a bridge together, should not automatically be assumed to provide full lateral support to the compression flanges. The flexibility of the cross bracing should be taken into account when defining the effective length for lateral torsional buckling and, consequently, when calculating the critical buckling stress. Normally, when frame cross bracing is adopted as shown in Figure 12.3(a), the lateral supports to the compression parts of the main beams should not be considered rigid, but rather as elastic supports (springs). This situation may arise in the negative moment regions of a bridge of open cross section (where the lower part of the beams is in compression), or in the positive moment regions of a bridge with a lower slab (trough bridge) or a composite bridge during erection. In the last two cases, the upper flanges are in compression and supported elastically by the cross bracing.

For the cases noted above, the issue of lateral torsional buckling is effectively one of overall buckling of a continuous compression strut on spring supports. In general the strut may be described as being subjected to a compressive force that varies along its length (variable bending moment and variable second moment of area of the beam), with elastic (spring) supports provided by the cross bracing. These supports may have different spring constants. This is a relatively complex lateral torsional buckling problem, but nevertheless one which software can resolve in a satisfactory way. Simplified solutions have been developed, for example that by Engesser [12.2], by making the following simplifying assumptions:

- the strut is of constant cross section,
- the compressive axial force is constant along the length of the strut,
- the elastic supports all have the same spring constant, and
- the strut is laterally fixed at its ends, although it may rotate about its z axis.



(a) Cross section of an open section bridge with frame cross bracing



(b) Flexural stiffness of the frame

Fig. 12.3 Elastic support of lower flanges in compression by frame cross bracing.

Solving the differential equations that describe this stability problem allows the critical value of the axial force for a strut on localised elastic supports to be defined. The spring constant K for each cross bracing is assumed to be distributed along the length of the bar, with a value $k = K/e$ where e represents the spacing of the cross bracing. The critical value of the axial force is given by:

$$N_{cr} = 2\sqrt{kEI_D} \qquad (12.19)$$

Using this value of axial force as a starting point, it is practical to define a corresponding lateral torsional buckling length by equating (12.19) to the equation that expresses the critical load for the basic case of elastic buckling of a pin-ended column:

$$2\sqrt{kEI_D} = \frac{\pi^2 EI_D}{l_D^2} \qquad (12.20)$$

This enables calculation of the lateral torsional buckling length of a compression member that is supported elastically by the cross bracing:

$$l_D = \sqrt[4]{\frac{\pi^4}{4} EI_D e v} \geq e \qquad (12.21)$$

EI_D	:	bending stiffness of a strut of area A_D (eq. (12.12))
e	:	spacing of the elastic supports (cross bracing)
v	:	displacement of the lateral support for a unit load $H = 1$ acting in the direction of the support (Fig. 12.3(b)), $v = 1/K$

A buckling length that is less than the distance e between the elastic supports could, in some cases, be predicted by Equation (12.21). This is not, however, a possible solution, and in such cases l_D is taken as equal to e. The buckling length calculated using expression (12.21) allows the reduced buckling length to be determined using (12.10), and the critical stress using (12.11).

For compression flanges of small slenderness, Equation (12.21) gives lateral torsional buckling lengths that are excessive because the plastic deformations that the strut may be subjected to reduce its bending stiffness. For such cases, a more realistic approach is to replace the elastic modulus E by a reduced modulus E_{red} given by the following expression:

$$E_{red} = \bar{\lambda}_D^2 \frac{\sigma_D}{f_y} E \tag{12.22}$$

$\bar{\lambda}_D$: non-dimensional slenderness for lateral torsional buckling
σ_D : lateral torsional buckling stress

Using a reduced modulus requires an iterative approach. Starting with a choice of l_D, values for $\bar{\lambda}_D$ (12.18) and σ_D (12.14) are calculated. The initial choice of l_D is then compared with the value given by (12.21), and so on.

The displacement v can be calculated using the equations given in Table 14.8 for frame cross bracing. The greatest value of v, considering those for symmetric and antisymmetric lateral torsional buckling, governs. In the case of a trough bridge, or when the cross girder is located in the upper part of the cross section, the displacement v may be calculated to a good approximation using the following relationship, with notation as defined in Figure 12.3(b):

$$v = \frac{h_m^3}{3EI_m} + \frac{h_m^2 s_t}{2EI_t} \tag{12.23}$$

The uprights of the cross bracing are subject to horizontal forces corresponding to their function as lateral supports for the compression members. Although this force is theoretically zero for a perfect strut, numerical analyses that include initial imperfections of the compression member show that the force needed to provide lateral restraint is around 1% of the compression force acting in the strut itself. For bridges this force corresponds to the axial force acting in the flange plus part of the web, which has an area defined by Equation (12.12).

Other Methods for Checking Lateral Torsional Buckling

In the simplified method described above for calculating the critical load for lateral torsional buckling, a significant approximation is that no account is taken of variations in cross section dimensions and forces along the span. Both types of variation are commonly found in bridges. Eurocode 3 proposes defining the non-dimensional slenderness in a more general way by making use of minimum amplification factors for the design loads. According to the Eurocode, the non-dimensional slenderness is defined as follows:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \tag{12.24}$$

$\alpha_{ult,k}$: minimum amplification factor to apply to the design loads in order to reach the characteristic resistance of the cross section, without taking into account lateral torsional buckling
$\alpha_{cr,op}$: minimum amplification factor to apply to the design loads in order to reach the elastic critical resistance to lateral buckling of the strut under compression

The critical stress, or the factor $\alpha_{cr,op}$, may be calculated using software that permits variations in the geometry of the compression flange, and the support conditions due to the cross bracing, to be easily introduced. Once the non-dimensional slenderness $\bar{\lambda}_{op}$ is known, then the reduction factor is calculated using (12.16), replacing $\bar{\lambda}_D$ by $\bar{\lambda}_{op}$. This then allows the lateral torsional buckling stress (12.14) and the lateral torsional buckling moment resistance (12.15) to be calculated.

Finally, it should be noted that the general method of a second order elastic analysis of the load carrying structure, combined with checks on cross sectional resistances, may always be used. This method usually requires a finite element program that allows equivalent geometric imperfections of the member to be introduced, and performs a step-by-step calculation that takes into account second order effects. It is generally assumed that the form of these geometric imperfections corresponds to one of the instability modes of the system and, according to the Eurocode, the imperfection has a magnitude of $l/150$ (buckling curve d), where l is the distance between points of inflection for the deformed shape corresponding to the instability being considered.

12.2.5 Local Buckling of the Web

The distribution of normal stresses, as measured in a test on a singly symmetric plate girder with a thin web subject to bending moment, is shown in Figure 12.4 [12.3]. When the bending moment is small, the distribution of stresses over the depth of the web is linear, because the stresses are proportional to distance from the neutral axis (elastic pre-buckling behaviour according to the Navier-Bernoulli law, Fig. 12.4(a)). As the bending moment increases, the part of the web in compression deforms laterally (local buckling, Fig. 12.4(b)) and sheds the stresses it can no longer support into the flange. This redistribution of stresses means they are no longer linearly distributed (post-buckling behaviour, Fig. 12.4(a)).

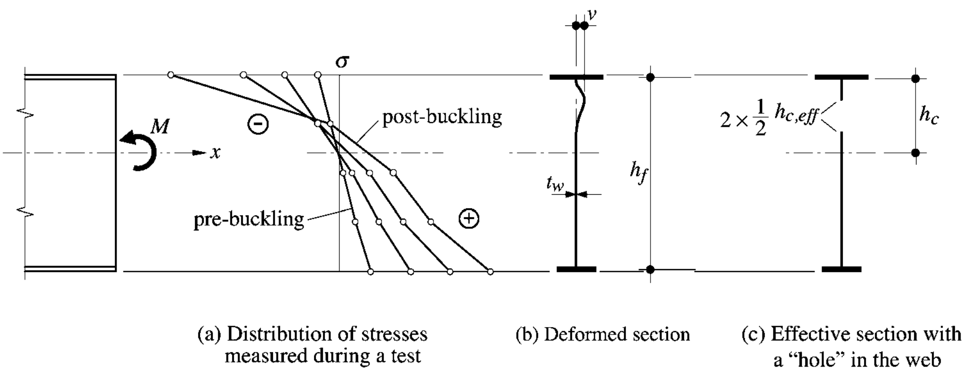


Fig. 12.4 Distribution of measured stresses in a plate girder with a slender web [12.3].

To study the bending moment resistance of a beam comprising thin plate elements, and taking into account the redistribution of stresses noted above, the so-called ***effective width*** method is used. This is the method that was considered in TGC Volume 10, Section 12.3, for the study of plates loaded in compression. It is assumed that the web in compression only resists over an effective depth $h_{c,eff}$, which is distributed half adjacent to the compression flange and half adjacent to the neutral axis (Fig. 12.4(c)).

Doubly Symmetric Section

The slenderness limit within which the total web contributes to the bending resistance may be calculated. This is the limit for which the effective depth $h_{c,eff}$ is equal to the depth in compression h_c, and is calculated using Equation (12.7) with reference to the total web depth h_f:

$$\frac{h_f}{t_w} \le 0.9\frac{\sqrt{k}}{1.052}\sqrt{\frac{E}{f_y}} \tag{12.25}$$

A doubly symmetric section subject to a bending moment is stressed the same in compression as it is in tension, and the buckling coefficient k equals 23.9, therefore:

$$\frac{h_f}{t_w} \le 4.2\sqrt{\frac{E}{f_y}}, \tag{12.26}$$

resulting in:

- $h_f/t_w \le 126$ for S235 steel,
- $h_f/t_w \le 102$ for S355 steel.

If the web slenderness exceeds these limits, which are the same as those given in Table 9 of the standard SIA 263, only the effective depth $h_{c,eff}$ is taken into account when calculating the bending moment resistance. This *effective depth* is defined by the inequality (12.25) which becomes, for a doubly symmetric section with $h_c = h_f/2$:

$$h_{c,\,eff} = 2.1\sqrt{\frac{E}{f_y}} \cdot t_w \le h_c \tag{12.27}$$

Singly Symmetric Sections

Equation (12.27) is only valid for doubly symmetric sections. Normally, plate girders are singly symmetric, and the following equation is used to calculate the effective depth:

$$h_{c,\,eff} = 0.9\frac{h_c}{\lambda_P} = 0.86\sqrt{k}\sqrt{\frac{E}{f_y}} \cdot \frac{h_c}{h_f}t_w \le h_c \tag{12.28}$$

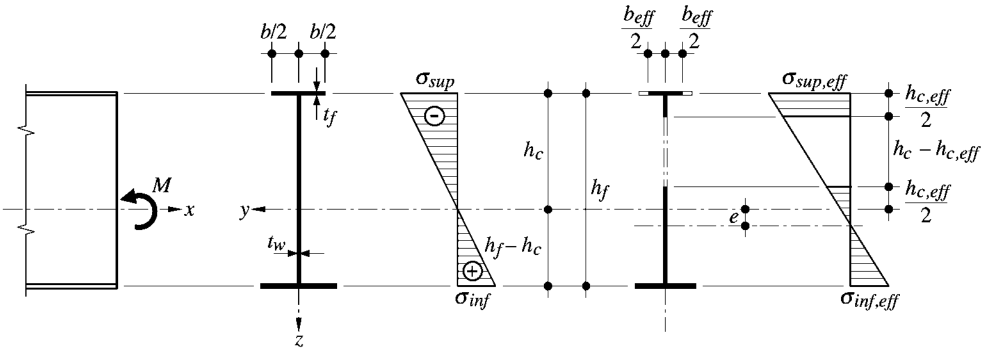
h_c	:	depth of the web in compression
h_f	:	depth measured between the mid-thickness of the flanges
t_w	:	thickness of the web

For a web with linear, pinned supports (provided by the two flanges), the buckling coefficient k is defined as follows:

$$k = \frac{16}{1 + \psi + \sqrt{(1+\psi)^2 + 0.112(1-\psi)^2}} \tag{12.29}$$

ψ	:	ratio (including its sign) between the minimum and maximum stresses, $\psi = \sigma_{inf}/\sigma_{sup}$, according to figure 12.5(a).

The effective depth $h_{c,eff}$, which contributes to the section bending resistance up to collapse, is placed in the parts of the web that do not buckle. These are the regions adjacent to the neutral axis and the



(a) Cross section (b) Effective section

Fig. 12.5 Effective section and distribution of stresses for a singly-symmetric plate girder in bending.

compression flange. The distribution of $h_{c,eff}$ is approximately 0.6 $h_{c,eff}$ next to the neutral axis and 0.4 $h_{c,eff}$ next to the flange. As a simplification, however, it can be assumed, as is suggested in the SIA standard, that the effective depth distributes equally between these two regions.

The effective section, therefore, looks like that shown in Figure 12.5(b). This reduced section, with a "hole" in the web, should be used when calculating the bending moment resistance of the section. This figure also shows the reduced area of the compression flange. Unless the flange is fully effective, this reduced area should be considered when calculating the elastic section modulus W_{eff}.

The reduced section, with a "hole" in the web, implies that a revised neutral axis position should be determined, rather than using that for the gross section. The neutral axis moves towards the tension flange; therefore, determining the bending moment resistance of the effective section requires calculation of a revised neutral axis position. A revised value of the second moment of area I_{eff} should also be calculated. The distance e between the neutral axis of the gross section and that of the effective section may be determined using the following equation:

$$e = \frac{h_c}{2} \cdot \frac{t_w(h_c - h_{c,\,eff})}{A_a - t_w(h_c - h_{c,\,eff})} \tag{12.30}$$

A_a : area of the gross cross section of the steel beam

The second moment of area I_{eff} of the effective section may be determined using the following equation:

$$I_{eff} = I_a - \frac{h_c^2}{4} \cdot \frac{A_a t_w(h_c - h_{c,\,eff})}{A_a - t_w(h_c - h_{c,\,eff})} - t_w \frac{(h_c - h_{c,\,eff})^3}{12} \tag{12.31}$$

I_a : second moment of area of the gross cross section of the steel beam

Equations (12.30) and (12.31) are only valid provided the compression flange is totally effective. If that is not the case, then the characteristics of the effective section can still be obtained by adding to the gross section those areas that are ineffective, as negative areas.

The ultimate bending moment resistance M_R about the major axis of such a plate girder, with a reduced area to allow for local buckling of the compression elements, and with the stress in the compression flange limited to the lateral torsional buckling stress, is given by:

$$M_R = M_D = \sigma_D \cdot W_{c,\,eff} \tag{12.32}$$

M_D : lateral torsional buckling resistance
σ_D : lateral torsional buckling stress from (12.14)
$W_{c,eff}$: elastic section modulus calculated with respect to the mid-thickness of the compression flange of the effective section

with

$$W_{c,\,eff} = \frac{I_{eff}}{h_c + e} \tag{12.33}$$

Equation (12.32) is only valid for calculating the ultimate bending resistance if $h_c + e \geq h_f/2$. Otherwise, this resistance should be calculated using the elastic section modulus $W_{t,eff}$ with respect to the mid-thickness of the tension flange of the effective section. When lateral torsional buckling is not critical ($\bar{\lambda}_D \leq 0.4$), f_y replaces σ_D in Equation (12.32).

When the resistance of a plate girder is limited by lateral torsional buckling ($\sigma_D < f_y$), the yield strength f_y may be replaced by $\sqrt{\sigma_D \cdot f_y}$, when calculating the effective widths b_{eff} (12.9) and $h_{c,eff}$ (12.28). This approach takes into account the fact that when lateral torsional buckling occurs, the average bending stress in the compression flange is less than the yield strength of the steel f_y. It leads to less conservative values of the effective widths, but does require iteration.

Influence of Longitudinal Stiffeners

As a general rule it is not economical to weld longitudinal stiffeners to a web in compression in order to ensure that its total area contributes to the section bending resistance. The difference between the bending moment resistance calculated using W_c and that with $W_{c,eff}$ is often only a few percent, which does not justify the additional cost. On the other hand, longitudinal stiffeners welded to the compression flange of a box section (Sect. 12.8) are justified because the flange makes a major contribution to bending resistance, so it is important that the total flange area is effective.

If, however, a plate girder requires one or more longitudinal stiffeners anyway to reinforce the web so that slenderness limits are respected (web breathing, § 12.7.3), or such stiffeners are needed during erection of the steelwork, they may also be taken into account when determining the ultimate bending resistance of

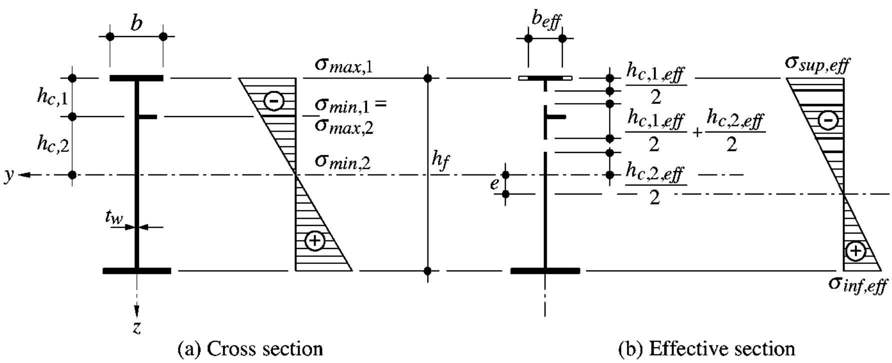


Fig. 12.6 Effective section and distribution of stresses for a plate girder with a longitudinal stiffener.

the section. The calculation of this resistance is based on the same concept of effective widths as discussed elsewhere, with an effective width being calculated for each sub-panel of the web. These sub-panels ($h_{c,1}$ and $h_{c,2}$ in Fig. 12.6(a)) are bounded by the compression flange and the longitudinal stiffener (or even two such stiffeners). A nominally pinned support is assumed at each edge of the panel. If the longitudinal stiffeners possess sufficient torsional stiffness (box section stiffener formed from a channel section), the panels may be considered to be built-in at these supports.

One condition to note, which is essential for this approach to be valid, is that the longitudinal stiffeners must be able to provide adequate support to the web as it buckles, in both the pre-buckling and post-buckling states. This means that the stiffeners must not only be sufficiently resistant, they must also be sufficiently stiff in bending to provide a true lateral support for the web panels without displacing laterally themselves. They must be able to do this up to failure of the beam. The conditions that must be fulfilled by these stiffeners with regard to resistance and stability are given in Paragraph 12.6.4.

The effective depth $h_{c,i,eff}$ of each web sub-panel i (Fig. 12.6(b)) may be calculated using Equation (12.28):

$$h_{c,i,eff} = 0.86\sqrt{k_i}\sqrt{\frac{E}{f_y}} \cdot \frac{h_{c,i}}{h_i} t_w \leq h_{c,i} \tag{12.34}$$

The yield strength f_y may be replaced in this equation by the maximum stress acting on the edges of the panel being considered. This stress $\sigma_{max,i}$ is calculated for the total area of the web (no loss of effective area), although any reduction in the compression flange area is taken into consideration. The buckling coefficient k is defined by Equation (12.29) for a web sub-panel that is simply supported at its edges. The factor ψ_i represents the ratio between the stresses $\sigma_{min,i}$ and $\sigma_{max,i}$ that act at the edges of the panel. Half the effective width $h_{c,i,eff}$ is assumed to be distributed to each edge of the panel i, as shown in Figure 12.6(b).

It is beneficial to use a table when calculating the effective modulus W_{eff} of such a section. The area of the longitudinal stiffeners may be taken into account, although their contribution to bending resistance is negligible.

12.2.6 Verification of Structural Safety (ULS)

In this paragraph we will summarise the different steps that are needed in order to check the structural safety of a plate girder subject to bending.

1. Check the web slenderness h_f/t_w (12.6) to ensure that the compression flange will not buckle vertically into the web.
2. Check the slenderness of the part of the web in compression h_c/t_w (12.88), to avoid any fatigue effects.

If these two checks are not satisfied, then either the web thickness should be increased, or an appropriately placed longitudinal stiffener should be used. The option to increase the web thickness is often the more economical of the two.

3. Check the slenderness of the compression flange to avoid buckling by rotation, or local buckling, $(b/2)/t_f$ according to Equation (12.8).

If this condition, which ensures that the total area of the compression flange contributes to bending resistance, is not met, then the slenderness $(b/2)/t_f$ must be modified so that the flange is totally effective.

Given its significant contribution to the bending resistance, it is important that the total flange area is effective for a plate girder. This is not always possible for box girders.

4. Check lateral torsional buckling of the beam (or lateral buckling of the compression flange) by determining the lateral torsional buckling stress σ_D in the compression flange (12.14). If this stress is less than f_y, then lateral torsional buckling will occur before the ultimate bending resistance can be achieved. In order to increase the lateral torsional buckling stress, either the second moment of area (in the transverse sense, about the z axis of the section) of the compression flange can be increased, or the distance between the lateral supports to the flange could be reduced.
5. Check the slenderness of the web to establish if a reduced effective depth should be used when calculating the elastic section modulus $W_{c,eff}$ (12.33).

If Checks 3 and 5 do not necessitate either a reduction in the flange width or the web depth, then the bending moment resistance M_R is defined by (12.1). If this is the case, then lateral torsional buckling should be checked as with rolled sections, taking into account the conditions given in Table 6 of the standard SIA 263 for an EE calculation method.

If Checks 3 and 5 do require a reduction of the effective widths, then the resistance M_R is defined by (12.32).

6. Check the structural safety of the section, taking into account the resistance factor γ_a:

$$M_{Ed} \leq M_{Rd} = \frac{M_R}{\gamma_a} \tag{12.35}$$

M_{Ed} : design value of bending moment M
M_{Rd} : design value of bending moment resistance
M_R : bending moment resistance
γ_a : resistance factor for steel

12.2.7 Numerical Example: Bending Resistance

Consider a simple span plate girder destined for use in a bridge. The cross section dimensions are as shown in Figure 12.7(a):

- Upper and lower flanges: width $b = 600$ mm, thickness $t_f = 20$ mm,
- Web: depth $h_w = 2000$ mm, thickness $t_w = 12$ mm,
- Steel: grade S355

Determine the bending moment resistance of the beam assuming that the upper flange is restrained laterally, by rigid supports, every 10 m.

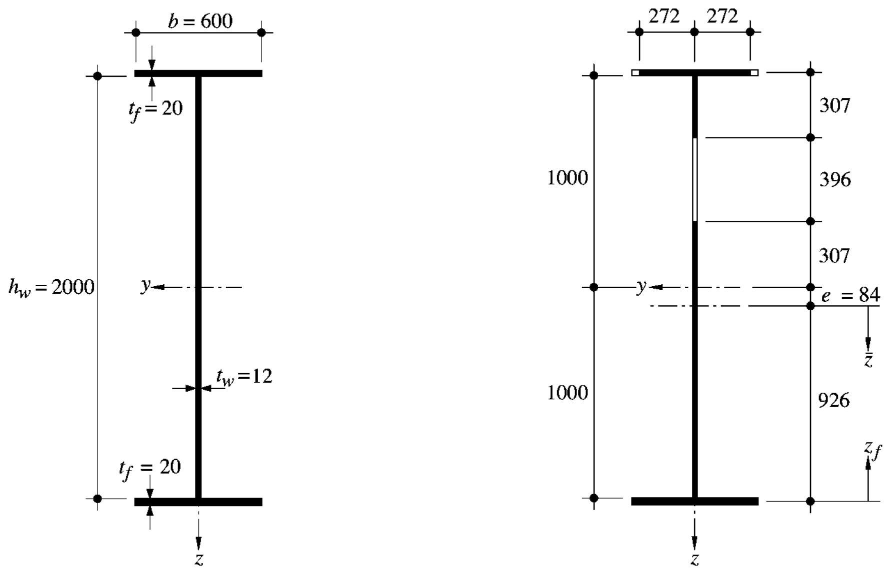
Check Slenderness Values

- *Vertical buckling of the compression flange into the web*

  According to Equation (12.6): $\frac{h_f}{t_w} \leq 240 \rightarrow \frac{2000+20}{12} = 168 \leq 240$ $\Rightarrow$ OK

- *Web breathing*

  According to Equation (12.88) for slender beams supporting variable loads, and with a doubly symmetric cross section, $h_c = h_f/2$



(a) Cross section

(b) Effective section

Fig. 12.7 Cross sections of a plate girder.

$$\frac{h_c}{t_w} \leq 100 \rightarrow \frac{2020/2}{12} = 84 \leq 100 \qquad \Rightarrow \text{OK}$$

- *Compression flange*

  According to Equation (12.8), for a doubly symmetric cross section, check that the total area of the compression flange contributes to the moment resistance of the section:

$$\frac{(b/2)}{t_f} \leq 14 \rightarrow \frac{600/2}{20} = 15 > 14 \qquad \Rightarrow \text{Not OK}$$

  Therefore, one must define an effective width of the compression flange according to (12.9):

$$\frac{b_{eff}}{2} = 0.56\sqrt{\frac{E}{f_y}} \cdot t_f = 0.56\sqrt{\frac{210\,000}{355}}\, 20 = 272 \text{ mm}$$

- *Web in bending*

  According to Equation (12.26), for a doubly symmetric cross section, check that the total area of the web contributes to the moment resistance of the section:

$$\frac{h_f}{t_w} \leq 102 \rightarrow \frac{2020}{12} = 168 > 102 \qquad \Rightarrow \text{Not OK}$$

  One must define, therefore, an effective depth of web, either using (12.27) for a doubly symmetric cross section, or the general Equation (12.28) with $k = 23.9$:

$$h_{c,\,eff} = 0.86\sqrt{k}\sqrt{\frac{E}{f_y}}\cdot\frac{h_c}{h_f}t_w = 0.86\sqrt{23.9}\sqrt{\frac{210\,000}{355}}\cdot\frac{1010}{2020}12 = 614\ \text{mm}$$

$$\frac{h_{c,\,eff}}{2} = 307\ \text{mm}$$

Properties of the Effective Cross Section

The reduced cross section comprises an effective width of compression flange and an effective depth of web, as shown in Figure 12.7(b). The effective section properties may be calculated using Table 12.8, taking as the reference axis the mid-thickness of the lower flange.

Table 12.8 Calculation of the effective cross section properties.

Element of the Cross Section	A [mm^2]	z_f [mm]	$A \cdot z_f$ [$10^6\,mm^3$]	$\bar{z}$ [mm]	$A \cdot \bar{z}^2$ [$10^9\,mm^4$]	I_{propre} [$10^9\,mm^4$]
Upper flange	10 880	2020	22.0	–1094	13.0	0.0
Upper web	3564	1862	6.6	--936	3.1	0.0
Lower web	15 684	664	10.4	262	1.1	2.2
Lower flange	12 000	0.0	0.0	926	10.3	0.0
Total	42 128	–	39.0		27.5	2.2
	Location of the neutral axis: 926 mm			$I_{eff} = 29.7 \cdot 10^9\ mm^4$		

The neutral axis is located at a distance $e = 1010 - 926 = 84$ mm from the y axis, in the direction of the tension flange. The elastic modulus of the effective section is given by:

$$W_{c,\,eff} = \frac{29.7 \cdot 10^9}{1010 + 84} = 27.1 \cdot 10^6 \text{mm}^3$$

This modulus is 15% less than that of the gross section. It would only be 9% less if the total area of the compression flange were able to contribute (which could be achieved by adopting a flange with an identical area but different dimensions, such as 500 × 24).

Bending Moment Resistance

The bending moment resistance of the section is:

$$M_R = 27.1 \cdot 10^6 \text{mm}^3 \cdot 355\,\text{N/mm}^2 = 9.64 \cdot 10^9 \text{Nmm} = \mathbf{9638\ kNm}$$

However, the bending resistance of the beam may be limited by lateral torsional buckling. This corresponds to lateral buckling of the compression flange (about the z-z axis), which may be considered as a strut with a cross sectional area equal to the effective area of the compression flange plus that part of the effective depth of the web located next to this flange. This strut has the following properties:

$$I_D = \frac{20 \cdot 544^3}{12} + \frac{297 \cdot 12^3}{12} = 268 \cdot 10^6 \text{mm}^4$$

$$A_D = 544 \cdot 20 + 12 \cdot 307 = 14.6 \cdot 10^3 \text{mm}^2$$

$$i_D = \sqrt{\frac{I_D}{A_D}} = \sqrt{\frac{268 \cdot 10^6}{14.6 \cdot 10^3}} = 136 \text{ mm}$$

$$\lambda_K = \frac{l_D}{\sqrt{\eta} \cdot i_D} = \frac{10\,000}{1.0 \cdot 136} = 73.5$$ with η taken as 1.0 (for uniform moment, which is a conservative assumption)

- Elastic critical stress for lateral torsional buckling:

$$\sigma_{crD} = \frac{\pi^2 E}{\lambda_k^2} = \frac{\pi^2 \cdot 210\,000}{73.5^2} = 384 \text{ N/mm}^2$$

- Reduction factor (using buckling curve c)

$$\bar{\lambda}_D = \sqrt{\frac{f_y}{\sigma_{crD}}} = \sqrt{\frac{355}{383}} = 0.961$$

$$\Phi_D = 0.5[1 + \alpha_D(\bar{\lambda}_D - 0.2) + \bar{\lambda}_D^2\ ] = 0.5[1 + 0.49(0.961 - 0.2) + 0.961^2] = 1.15$$

$$\chi_D = \frac{1}{\Phi_D + \sqrt{\Phi_D^2 - \bar{\lambda}_D^2}} = \frac{1}{1.15 + \sqrt{1.15^2 - 0.961^2}} = 0.562$$

- Lateral torsional buckling stress:

$$\sigma_D = \chi_D \cdot f_y = 0.562 \cdot 355 = 200 \text{ N/mm}^2$$

- Lateral torsional buckling resistance:

$$M_R = M_D = 27.1 \cdot 10^6 \text{mm}^3 \cdot 200 \text{N/mm}^2 = 5.42 \cdot 10^9 \text{Nmm} = \mathbf{5420\ kNm}$$

12.3 Shear Resistance

12.3.1 Introduction

The shear resistance of a plate girder, like that of a rolled section (TGC 10, Section 4.4), is primarily provided by its web. However, the webs of plate girders are normally more slender, their web plates being thin relative to their depth (§ 5.3.2). This makes them more susceptible to local buckling under the effects of shear.

Unfortunately, the linear elastic theory for local buckling (Chapter 12 of TGC Volume 10) does not allow the real ultimate resistance of a panel in compression to be directly determined. This theory is also inadequate for calculating the true resistance of a web panel, in a plate girder, which is subject to shear. The ultimate shear resistance V_R can only be calculated by considering the post-buckling behaviour of the web, that is the resistance available beyond the attainment of the elastic critical buckling stress τ_{cr}.

The behaviour of a panel in shear therefore comprises two phases (Fig 12.9):

- Prior to local buckling (Fig. 12.9(a)), the state of the in-plane stresses is a combination of tension and compression of equal intensity (there are diagonals in tension and compression at 45° relative to the edges for a square panel). The critical value for shear V_{cr} is determined using linear elastic theory (***pre-buckling behaviour***).
- After the compression stresses have resulted in local buckling of the panel (this can be pictured as overall buckling of a diagonal compression strut as shown in Fig. 12.9(b)), additional panel resistance is provided solely by the diagonal in tension. The tension stresses in this diagonal can continue to increase up to the point of plastification (***post-buckling behaviour***). This additional, post-buckling, shear resistance is called V_σ. It should be noted, however, that it is only possible to achieve the extra resistance that may be provided by the diagonal in tension (which acts as a tie) if the panel that is subject to shear is bordered by rigid elements (permitting the so-called membrane effect). In the case of plate girders, this is achieved when the flanges and transverse stiffeners are effectively rigid. In such cases an effective truss is formed, comprising the rigid elements at the boundaries plus the tension diagonal, superimposed on the pre-buckling shear panel.

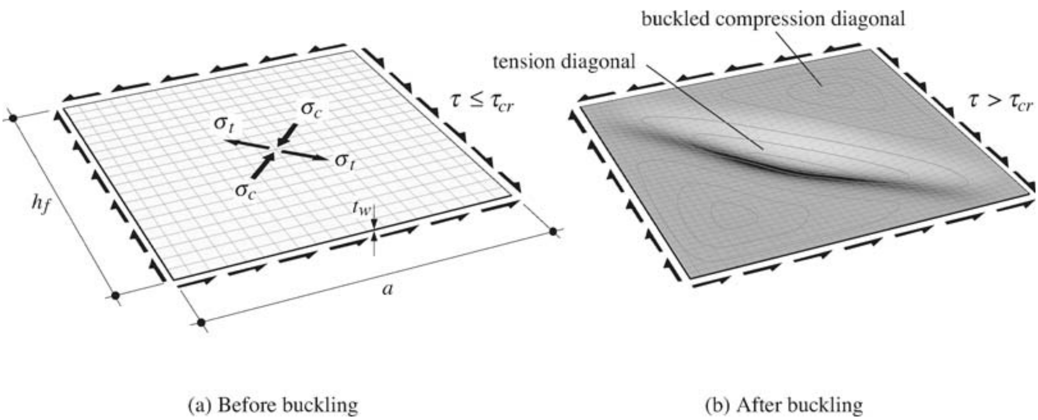


Fig. 12.9 Buckling of a panel in shear.

Therefore, the shear resistance V_R comprises two terms:

$$V_R = V_{cr} + V_\sigma \tag{12.36}$$

V_{cr} : pre-buckling contribution of the web to the shear resistance
V_σ : post-buckling contribution of the web to the shear resistance

12.3.2 Contribution of Elastic (Pre-buckling) Behaviour

The elastic contribution, which is the maximum shear force that the web can resist before buckling, is calculated using linear elastic buckling theory and is given by:

$$V_{cr} = \tau_{cr} h_f t_w \tag{12.37}$$

τ_{cr} : critical shear stress
h_f : depth measured between the mid-thickness of the flanges
t_w : web thickness

with

$$\tau_{cr} = k\frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{h_f}\right)^2 = 0.9kE\left(\frac{t_w}{h_f}\right)^2 \tag{12.38}$$

The buckling coefficient k is a function of the slenderness $\alpha = a/h_f$, where a is the distance between the transverse stiffeners. For a web panel that is assumed to be simply supported along all four edges (the boundary conditions are pinned supports), this coefficient is equal to:

$$k = 4.0 + \frac{5.34}{\alpha^2} \text{ for } \alpha \leq 1 \tag{12.39}$$

$$k = 5.34 + \frac{4.0}{\alpha^2} \text{ for } \alpha \geq 1 \tag{12.40}$$

For small values of slenderness h_f/t_w of the panel in shear, and depending on the value of the coefficient α, the critical shear stress given by Equation (12.38) may be close to the yield strength in shear τ_y. To allow for non-linear behaviour between the limit of elastic behaviour and the yield strength of the steel, the critical stress τ_{cr} is reduced if it exceeds the limit of elastic behaviour at $0.8\tau_y$. On the other hand, if the web is compact, then the critical shear stress may exceed τ_y as there is no instability of the web due to local buckling.

The conditions noted above may be described using a relative slenderness for the panel in shear $\bar{\lambda}_w$, defined as:

$$\bar{\lambda}_w = \sqrt{\frac{\tau_y}{\tau_{cr}}} \tag{12.41}$$

τ_y : yield strength in shear, normally, $\tau_y = f_y/\sqrt{3}$

- If $\bar{\lambda}_w \leq 0.9$ (compact web) the shear resistance is defined as:

$$V_R = \tau_y A_w \tag{12.42}$$

$A_w = h_f t_w$: "conventional" web area

- If $0.9 < \bar{\lambda}_w \leq 1.12$, ($\tau_{cr} > 0.8\tau_y$), the critical shear stress is equal to the reduced critical stress $\tau_{cr,red}$ given by:

$$\tau_{cr,red} = \sqrt{0.8\tau_y\tau_{cr}} \tag{12.43}$$

- If $\bar{\lambda}_w > 1.12$ there is no reduction in the critical stress.

Equation (12.42) is the same as that defining the ultimate shear resistance of a rolled section. A slenderness limit can be defined for the web, beyond which the tension tie is mobilised. In other words, beyond this limit, post-buckling behaviour is effective. Consideration of Equations (12.38) and (12.41), and the first condition noted above, (namely $\bar{\lambda}_w \leq 0.9$), gives:

$$\frac{h_f}{t_w} \leq 1.12\sqrt{k}\sqrt{\frac{E}{f_y}} \tag{12.44}$$

For beams without vertical stiffeners, α tends to infinity and k to 5.34 (eq. 12.40), so this slenderness limit becomes:

- $h_f/t_w \leq 78$ for S235 steel,
- $h_f/t_w \leq 63$ for S355 steel.

If the slenderness of the web panel is less than these limits, then the ultimate shear resistance is defined by Equation (12.42). If it is greater, then the ultimate resistance is defined by Equation (12.36), taking into account any reduction in critical stress according to (12.43).

12.3.3 Contribution of Post-buckling Behaviour

When the elastic critical buckling stress is reached in a web panel subject to shear, the compression strut buckles. However, this does not represent failure of the panel. Additional shear resistance is then mobilised due to the diagonal in tension, with the solid web beam (Fig. 12.10(a)) effectively transforming itself into a *truss* in the post-buckling phase (Fig. 12.10(b)).

The post-buckling contribution V_σ due to the tension diagonal is based on the principles of the model shown in Figure 12.11. In the region of the tension diagonal, the tensile stress σ_t is assumed, as a simplification, to be uniformly distributed over a certain width of the diagonal. Based on this simplification different models have been developed. The essential differences between models are the way in which the tension diagonals are anchored (in the flanges, the stiffeners, and the adjacent panels), and the failure mechanism of the system.

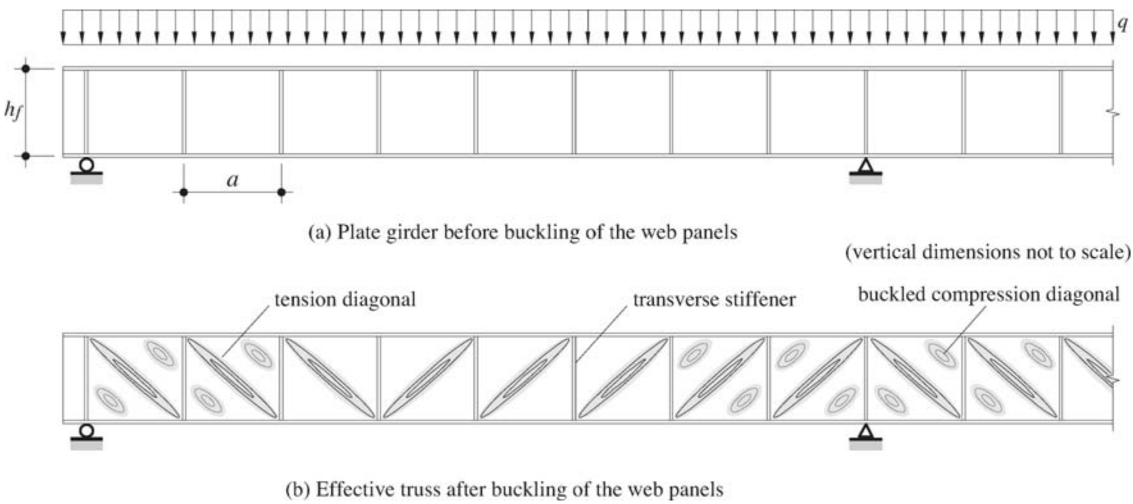


Fig. 12.10 Shear behaviour of a plate girder.

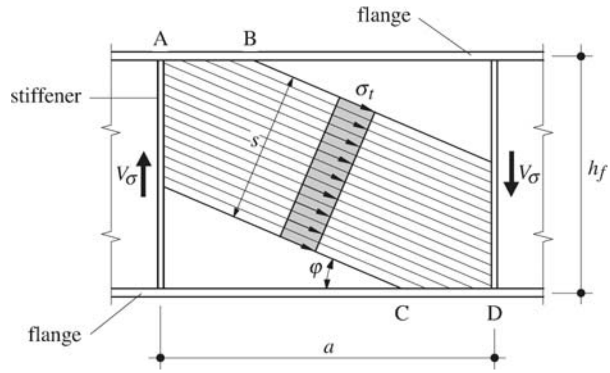


Fig. 12.11 Schematic representation of a diagonal tie in the web panel of a plate girder.

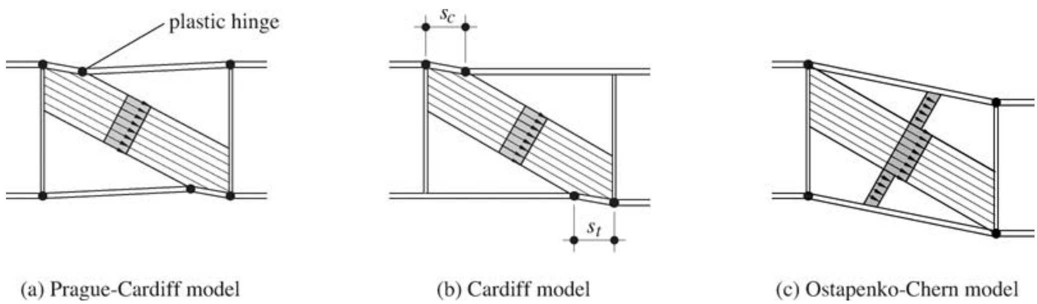


Fig. 12.12 Failure mechanisms for a web panel in a plate girder.

The theory developed by Basler [12.4] is based on the hypothesis that the tension diagonals are solely anchored in the adjacent web panels, as the flanges are considered to be too flexible to provide support. Other theories do assume anchorage is provided by the flanges, and are differentiated by the failure mechanism that is assumed (Fig. 12.12).

The Prague-Cardiff model developed by Rockey and Skaloud [12.5] is worth noting. It is based on numerous tests to establish the influence of the flange stiffness on the angle, width, and position of the diagonals in tension. The mechanism associated with this model is shown in Figure 12.12(a), with a peculiarity that the angle of the tie is the same as that of the web panel diagonal. A more recent development of this model, based on the mechanism shown in Figure 12.12(b) and more elaborate assumptions concerning the angle of the tension diagonals, led to the Cardiff model [12.6]. Ostapenko and Chern [12.7] consider a frame mechanism in their theory (Fig. 12.12 (c)), and extend the width of the tension diagonal. Other models are described, with numerous references, in [12.8]. The model of Basler, which was used as the basis for the standard SIA 263, and the Cardiff model are considered in more detail below.

Basler Model

In his theory Basler makes the hypothesis that tension diagonals can only form if the boundary conditions allow it. These boundaries comprise the transverse stiffeners, which form the uprights of the truss model, and the flanges, which form the chords. Basler assumes that the vertical bending stiffness of the flanges and the horizontal stiffness of the stiffeners are low. As a consequence of these assumptions, the tension diagonals can only anchor themselves in the adjacent web panels, which are assumed to be rigid elements, and the flanges are not mobilised in bending. Figure 12.13 illustrates the principles of the Basler model with a tension diagonal that is of constant width and bounded by two straight lines coming from the corners of the panel. The tension forces T_2 in the diagonals in two adjacent panels are in equilibrium, and the tension T_1 must be anchored in the triangles ABC of the adjacent web panel. The force T_1 is resolved into a horizontal force H and a vertical force F_s, where F_s defines the compression that acts on the transverse stiffener. The hypotheses that form the basis of Basler's model have been confirmed on numerous occasions through observations made during tests [12.9].

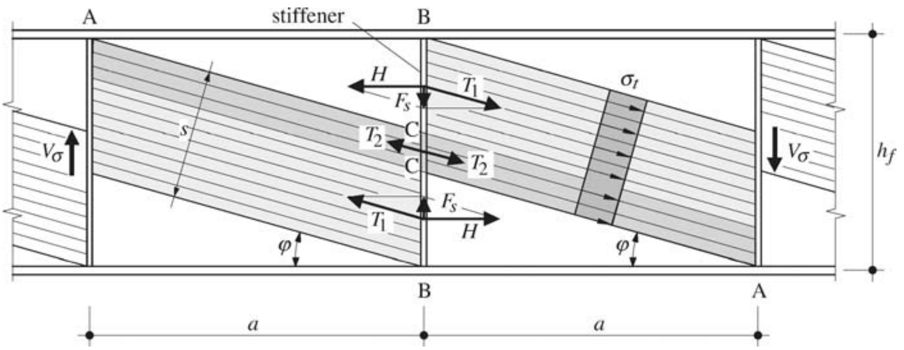


Fig. 12.13 Principles of the Basler model.

The tension diagonal that forms in each web panel allows an additional shear force V_σ to be carried. This additional force can be defined by making a vertical cut I-I through the web of a beam that is subject to constant shear (Fig. 12.14(a)). It comprises the vertical component $V_{\sigma 1}$ of the tensile force in the tension diagonal, plus the contribution $2V_{\sigma 2}$ of the shear force in the two adjacent, rigid web triangles that anchor the oblique band (Fig. 12.14(b)):

$$V_\sigma = V_{\sigma 1} + 2V_{\sigma 2} \tag{12.45}$$

According to the notation in Figure 12.14, the vertical component $V_{\sigma 1}$ of the tensile force T acting in the diagonal is expressed by the following equation:

$$V_{\sigma 1} = \sigma_t \cdot t_w \cdot s \cdot \sin\varphi = \sigma_t \cdot t_w \cdot \sin\varphi\,(h_f \cos\varphi - a \sin\varphi) \tag{12.46}$$

The inclination φ of the oblique band may be determined by assuming that $V_{\sigma 1}$ is a maximum when the ultimate shear resistance is reached. This angle is calculated with $\mathrm{d}V_{\sigma 1}/\mathrm{d}\varphi = 0$, from which one obtains:

$$\tan 2\varphi = \frac{h_f}{a} = \frac{1}{\alpha} = \tan\theta \tag{12.47}$$

from which

$$\varphi = \theta/2 \tag{12.48}$$

Given that the forces T_2 in the oblique bands are in equilibrium from one panel to the next, only the forces T_1 need to be anchored in the rigid triangles. By isolating a panel ABC (Fig. 12.14(b)), the shear force in this rigid triangle can be calculated because it is constant and equal to its support reaction. The horizontal force H, which also represents the build up of the axial force in the beam flange over the length a of the panel, results in the vertical support reaction $V_{\sigma 2}$ at point A (obtained by equilibrium of moments about B):

$$V_{\sigma 2} = \frac{H \tan\varphi}{2} \tag{12.49}$$

with

$$H = \sigma_t \cdot t_w \cdot a \cdot \sin\varphi \cdot \cos\varphi \tag{12.50}$$

gives

$$V_{\sigma 2} = \frac{\sigma_t \cdot t_w \cdot a \cdot (\sin\varphi)^2}{2} \tag{12.51}$$

Substituting Equations (12.46) and (12.51) into (12.45), and taking into account (12.47), the relationship that defines the post-buckling resistance as a function of the web panel dimensions and the tensile stress σ_t acting in the diagonal is obtained:

$$V_\sigma = \frac{\sigma_t \cdot h_f \cdot t_w}{2} \cdot \frac{1}{\sqrt{1+\alpha^2}} \tag{12.52}$$

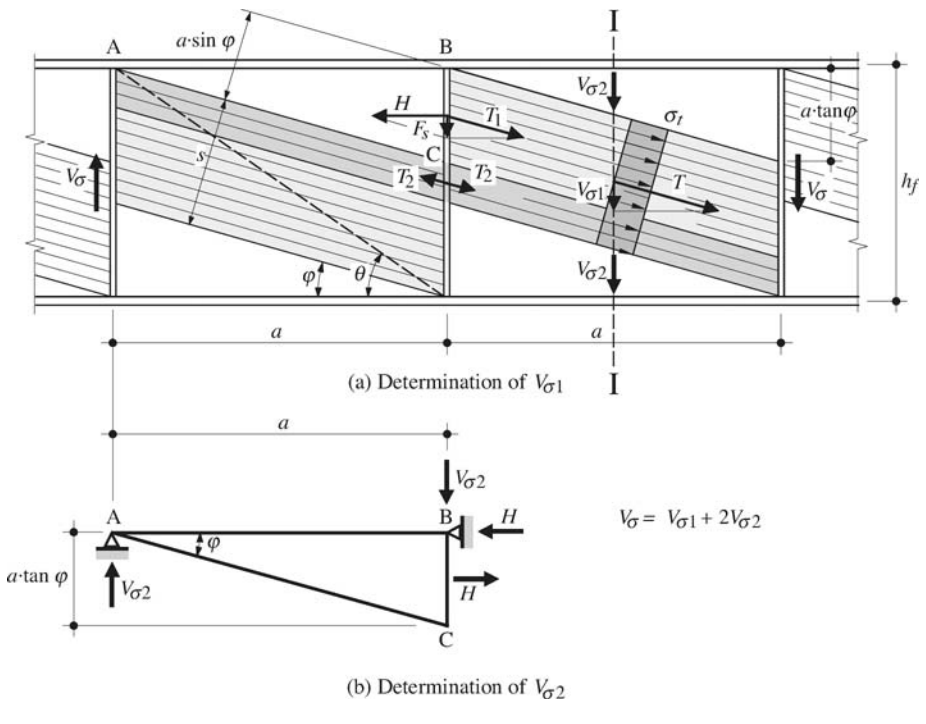


Fig. 12.14 Post-buckling shear resistance according to the Basler model.

The value of σ_t may be calculated taking into account that the sum of the stresses τ_{cr} and σ_t, due to the two resistance modes V_{cr} and V_σ, is limited by the stress state defined by von Mises for plastification, resulting in:

$$\sigma_t = \sqrt{f_y^{\,2} - \tau_{cr}^{\,2}\left(3 - \left(\frac{3}{2}\sin 2\varphi\right)^2\right)} - \frac{3}{2}\tau_{cr}\sin 2\varphi \tag{12.53}$$

Accepting some conservatism [12.9], this equation may be simplified to:

$$\sigma_t = f_y\left(1 - \frac{\tau_{cr}}{\tau_y}\right) = \sqrt{3}(\tau_y - \tau_{cr}) \tag{12.54}$$

Substituting (12.52) and (12.37) into (12.36) and taking into account (12.54), one obtains the *shear resistance* according to Basler's model:

$$V_R = \left(\tau_{cr} + \frac{\sqrt{3}(\tau_y - \tau_{cr})}{2\sqrt{1+\alpha^2}}\right) \cdot A_w = \tau_R \cdot A_w \tag{12.55}$$

τ_R	:	limiting shear stress
$A_w = h_f t_w$	:	"conventional" web area

Experimental work carried out in Sweden [12.10] on plate girders without vertical stiffeners has demonstrated that the angle of the tension diagonal is never less than the value for a virtual panel with an aspect ratio α of around 3. This means that when a plate girder does not have transverse stiffeners, or the spacing between them is such that $\alpha > 3$, a value of $\alpha = 3$ may be used in Equation (12.55), provided that the tension diagonal can be anchored at the beam ends.

However, tests have also shown that, unless specific detailing is adopted at the beam ends, anchorage may be insufficient, meaning that the total post-buckling resistance cannot be mobilised. In such situations the shear resistance of the *end panel* must be reduced, and is limited to the following value (which has been verified experimentally):

$$V_R = 0.9\sqrt{\tau_y \cdot \tau_{cr}} \cdot A_w \le \tau_y A_w \tag{12.56}$$

In this expression the value of τ_{cr} is always defined by (12.38), using the effective value of α, even if α is greater than 3.

The compression force F_s acting in the transverse stiffener (Fig 12.14(a)) is equal to the vertical component of T_1, which is expressed relative to α and taking into account (12.54):

$$F_s = \left(1 - \frac{\tau_{cr}}{\tau_y}\right)\left(\frac{\alpha}{2} - \frac{\alpha^2}{2\sqrt{1+\alpha^2}}\right) f_y A_w \tag{12.57}$$

Cardiff Model

This model builds on the mechanism shown in Figure 12.15. The model makes the hypothesis that the lengths s_c and s_t may be defined assuming that plastic hinges form at A, B, C and D (Fig. 12.15). The principle of virtual work applied, for example, to the upper flange for the work done by the plastic hinges A and B, and the work done by the force in the tension diagonal that is anchored along s_c, results in:

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$$\sigma_t \cdot t_w \cdot s_c \cdot (\sin\varphi)^2 \cdot \frac{s_c}{2} = 2M_{pl,N} \tag{12.58}$$

from which

$$s_c = \frac{2}{\sin\varphi} \cdot \sqrt{\frac{M_{pl,N}}{\sigma_t \cdot t_w}} \le a \tag{12.59}$$

$M_{pl,N}$: plastic moment resistance of the flange, reduced by the presence of axial force (TGC Vol. 10, Equation (4.78))

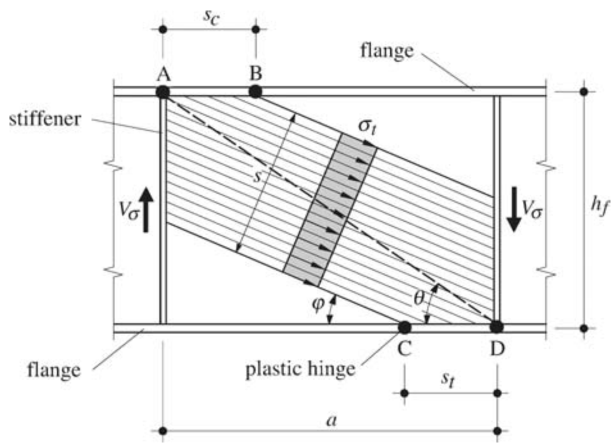

Fig. 12.15 Post-buckling shear resistance according to the Cardiff model.

Taking into account the contributions of the lengths s_c and s_t to the diagonal of width s (12.46), one can define the post-buckling contribution to the shear resistance according to this model:

$$V_\sigma = \sigma_t \cdot h_f \cdot t_w \cdot (\sin\varphi)^2 \cdot \left[\cot\varphi - \cot\theta + \frac{1}{h_f}(s_c + s_t)\right] \tag{12.60}$$

The tensile stress σ_t can be determined from (12.53). The choice of angle φ should be such that V_σ is a maximum. It is, therefore, necessary to adopt an iterative process, which converges rapidly, starting with an initial value of $\varphi = 2\theta/3$. Conservatively, the angle φ may also be taken as 0.6θ.

Substituting (12.60) into (12.45), the shear resistance according to the Cardiff model is obtained:

$$V_R = \left(\tau_{cr} + \sqrt{3}(\tau_y - \tau_{cr})(\sin\varphi)^2 \cdot \left[\cot\varphi - \cot\theta + \frac{1}{h_f}(s_c + s_t)\right]\right) \cdot A_w = \tau_R \cdot A_w \tag{12.61}$$

Influence of Longitudinal Stiffeners

Longitudinal stiffeners also play a role in the shear resistance of a plate girder. Effectively, these stiffeners divide the depth of the web panels into several sub panels, and, in particular, increase the local buckling resistance of the panels (pre-buckling contribution) and, therefore, the ultimate shear resistance. The development of tension diagonals in the post-buckling phase is also influenced by the presence of longitudinal

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13 Composite Beams

Composite trusses used for the road and rail bridges at Hagneck (CH).
Eng. Bureau d'ingénieurs DIC, Aigle.
Photo ICOM.

13.1 Introduction

This chapter describes the analysis and design of composite beams as used in bridges. They may be either plate girders or box girders structurally connected to a concrete slab. In particular, it covers the specifics of structural design of composite beams as used in bridges, making reference to TGC Volumes 10 and 11 for more general principles of composite construction. The analysis and verifications associated with composite steel-concrete elements in general are covered in TGC Volume 10 Sections 4.7 and 5.8, whilst composite beams used in buildings are covered in TGC Volume 11, Section 10.5.

Section 13.2 describes action effects that are specific to composite beams, such as the effects of concrete shrinkage and effects due to temperature. The various methods of structural analysis that may be adopted for composite beams are presented in Section 13.3. In certain instances other possibilities beyond just a purely elastic calculation of the resistance of composite cross sections, for example analysis methods that take into account the elasto-plastic behaviour of composite beams, can be adopted. In particular, as shown in Figure 13.1, in span where the moments are positive, it is possible to consider plastic cross section resistance, while above the intermediate supports, where the concrete in tension is cracked, the resistance of the cross section comprising the steel beam and longitudinal slab reinforcement should be assumed elastic, or reduced elastic. Both the influence of the cracked concrete in tension on the stiffness of continuous composite beams and the response of these beams to the actions on them are treated as well in this section.

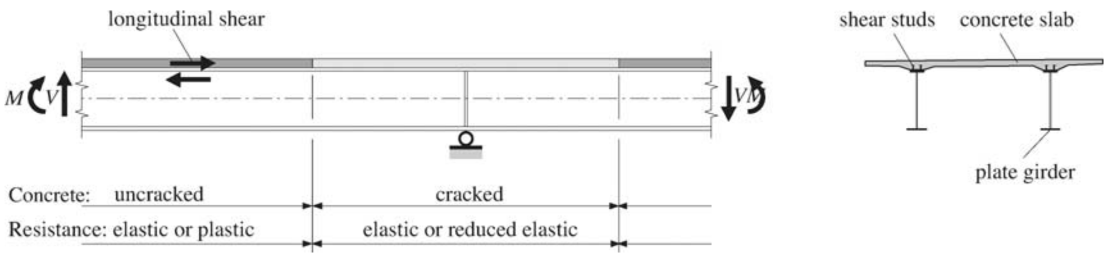


Fig. 13.1 Section resistance and types of analysis for a composite bridge beam.

Models for the resistance in bending and shear of composite beams as used in bridges, as well as the various checks for structural safety, are described in Section 13.4. Calculation of the longitudinal shear that is transferred between the steel and concrete by the shear connection (Fig. 13.1), the resistance of the shear studs that make this connection, and the checks for these studs depending on whether elastic or plastic behaviour is considered are covered in Section 13.5. If the mechanical connection, which assures the tying together of the steel and concrete, is sufficiently strong, then a check is needed to ensure that the concrete slab can resist the longitudinal shear that is transferred into it. Section 13.6 considers this issue in the context of structural safety.

Finally, Section 13.7 covers issues related to serviceability limit states and the checks that must be carried out to ensure good behaviour in service of a composite bridge.

13.2 Action Effects Specific to Composite Beams

13.2.1 Introduction

Those actions that may apply to either a steel or composite steel-concrete bridge were considered in a general manner in Chapter 10. The particular properties of concrete as it hydrates, and subsequently when the concrete slab is connected to the steel structure, have certain consequences which must be known and understood, and, where necessary, taken into account when checking composite members. These properties concern the warming of concrete as it hydrates and the subsequent cooling of the slab, as well as shrinkage and creep of the concrete.

The issue of warming and cooling of the slab while the concrete hydrates was considered in Paragraph 8.5.2 in terms of its influence on cracking of the partially cured concrete. A simplified approach for estimating the magnitude of the tensile stresses that develop during this part of the process of forming the slab was provided. Certain measures were proposed to either reduce the tensile stresses or reduce their effect, including longitudinal prestressing of the slab. The different methods of prestressing that may be adopted, as well as the corresponding prestress losses, were considered in Section 8.6. Whatever the method adopted for prestressing, it is necessary to guarantee that the prestressing forces are introduced into the composite section. This particular point is considered in Paragraph 13.5.4, which covers the introduction of concentrated forces into the steel-concrete connection.

As part of the calculation of elastic cross section resistance, the question of concrete creep and its influence on the design of composite beams is considered in Paragraph 13.4.2. How to take concrete shrinkage into account when designing composite bridges is considered below. A difference in temperature between the concrete and the steel structure is treated in a similar way to shrinkage, and, therefore, is also considered below.

13.2.2 Shrinkage

Concrete shrinkage (due to drying), like creep, is influenced by the humidity of the ambient air, the dimensions of the slab, and the concrete quality. Standard SIA 262 does not distinguish between the different forms of shrinkage, because considerable doubt remains as to how to attribute the different causes to shrinkage strains. According to this standard, for slab concrete grade C 30/37, and a “relative” depth of $h_0 \cong 300$ mm, for a typical road bridge slab, the shrinkage strain ε_{cs} is, after an infinite time:

- $\varepsilon_{cs,\infty} \cong 0.03\%$ for a relative humidity of 60%,
- $\varepsilon_{cs,\infty} \cong 0.02\%$ for a relative humidity of 80%.

In reality, shrinkage varies over time, notably as a function of the relative humidity of the air. Using a logarithmic time scale, Figure 13.2 shows measured values of relative air humidity and the shrinkage of specimens placed in the same environment as is typical for a freeway bridge on the Swiss plane. The specimens had roughly the same relative depth as a bridge slab and contained 0.5% reinforcement. The concrete used was identical to that used in the bridge slab, corresponding to grade C 30/37. After over two years of measurements, the maximum shrinkage was of the order of 0.015% for relative humidity varying between 60 and 90%. It is worth noting that the shrinkage coefficient has a tendency to increase more rapidly when the relative humidity reduces, and to reduce when the humidity increases. By considering such in-situ measurements, one can estimate that a long term shrinkage strain of 0.025% is representative of reinforced concrete bridge slabs executed using normal concrete.

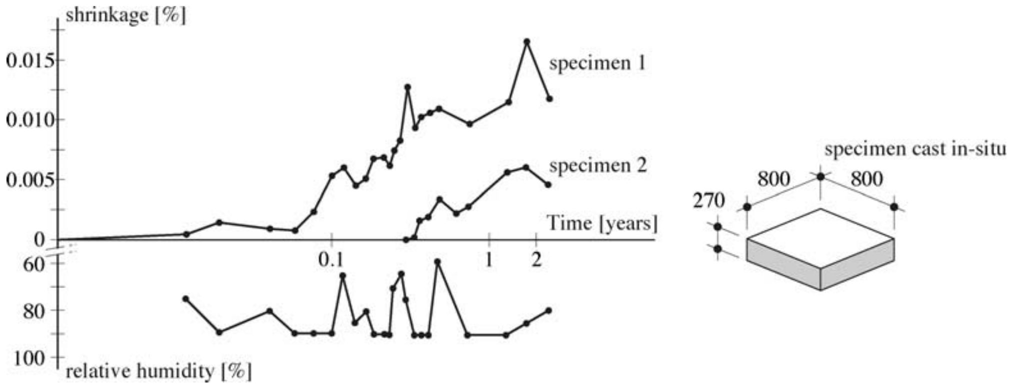


Fig. 13.2 Shrinkage measurements for samples cast in-situ (0.5% reinforcement).

To calculate the shrinkage effects in a composite bridge cross section, reference should be made to the considerations developed in TGC Volume 11, Paragraph 10.5.5. Figure 13.3 shows the normal stresses acting on a composite cross section due to the different forces that are representative of shrinkage for a simple beam.

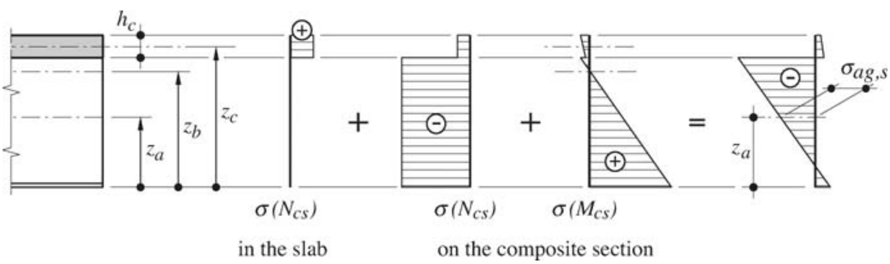


Fig. 13.3 Stresses due to shrinkage (simple beam).

The principle points to consider when calculating the internal forces due to shrinkage are:

- Because shrinkage of the concrete slab is restrained by the steel-concrete connection, the slab is subjected to a *tensile force* N_{cs}.
- To maintain equilibrium the composite cross section is subjected to a *compressive force* N_{cs}, and a *positive bending moment* M_{cs} (known as the primary moment).
- For a *simple beam* the normal stresses due to shrinkage (Fig. 13.3) are constant along the length of the beam, in equilibrium within a given cross section, and result in no additional shear force in the steel-concrete connection. At the beam ends the normal stresses are zero. This means that the normal force F_{vs}, which results from the stresses acting in the slab, must be introduced at the beam ends. This force acts locally on the steel-concrete connection in these regions (§ 13.5.4).
- For a *continuous beam*, to ensure continuity of curvature over the intermediate supports, shrinkage results in redundant negative bending moments. The vertical and longitudinal shear forces corresponding to these moments act on the steel-concrete connection (Fig. 13.5).

The normal force N_{cs} and the moment M_{cs} are defined respectively by:

$$N_{cs} = \varepsilon_{cs}(t) \cdot E_{cs} \cdot A_c = \varepsilon_{cs}(t) \cdot E_a \frac{A_c}{n_s} \quad (13.1)$$

$$M_{cs} = N_{cs} \cdot (z_c - z_b) \quad (13.2)$$

$\varepsilon_{cs}(t)$: shrinkage strain, at time t
E_{cs} : elastic modulus for concrete when considering shrinkage, taking into account creep ($E_{cs} = E_{cm}/2$)
A_c : area of the concrete slab (bh_c)
E_a : elastic modulus of steel
n_s : elastic modular ratio, taking into account shrinkage $n_s = E_a/E_{cs}$
E_{cm} : average value of the elastic modulus of concrete (according to the standard SIA 264)
z_c : location of the centre of gravity of the concrete slab (Fig. 13.3)
z_b : location of the centre of gravity of the composite section (Fig. 13.3)

The normal force in the slab F_{vs}, which must be anchored by the shear connectors at the beam ends, is equal to the resultant of the normal force acting in the steel section, namely the stress $\sigma_{ag,s}$ acting at the centre of gravity z_a of the steel section (Fig. 13.3) multiplied by the area A_a:

$$F_{v,s} = \sigma_{ag,s} \cdot A_a = \left[\frac{N_{cs}}{A_b} + \frac{M_{cs}}{I_b}(z_b - z_a)\right] \cdot A_a \quad (13.3)$$

N_{cs} : normal force acting on the composite section (negative)
M_{cs} : bending moment acting on the composite section (positive)
A_b : area of the resisting composite cross section, determined using n_s
I_b : second moment of area of the composite cross section, determined using n_s

Effect of Shrinkage in the Slab

A simplified relationship taken from [13.1] can be used to estimate the magnitude of the tensile stress σ_{cs} acting in the slab at the intermediate supports and internal spans of a continuous composite beam. This relationship is based on the hypothesis that the concrete is uncracked and that the total bending moment due to shrinkage may be neglected. Effectively, the sum of the primary and redundant moments at the intermediate supports is generally small, and their influence is negligible. This tensile stress is given by:

$$\sigma_{cs} = \frac{\varepsilon_{cs}(t) \cdot n_A \cdot E_{cm} \cdot E_a}{n_A \cdot E_a + n_A \cdot E_a \cdot \chi \cdot \varphi + E_{cm}} \quad (13.4)$$

$\varepsilon_{cs}(t)$: shrinkage strain, at time t
n_A : retention coefficient, $n_A = A_a/A_c$, (§ 8.5.2)
A_a : area of the steel beams
A_c : area of the concrete slab
E_{cm} : elastic modulus of concrete (average value)
E_a : elastic modulus of steel
χ : ageing coefficient
φ : creep coefficient

Information concerning the creep coefficient φ and the ageing coefficient χ may be found in TGC Volume 8. Figure 13.4 shows an application of Equation (13.4) as a function of the creep coefficient for two composite bridges with different retention coefficients, namely $n_A = 0.04$ (span approximately 30 m) and 0.12 (span approximately 80 m). In this application the shrinkage strain is taken as 0.015%, the ageing coefficient as 0.6, and the average elastic modulus of the concrete as 32 kN/mm^2.

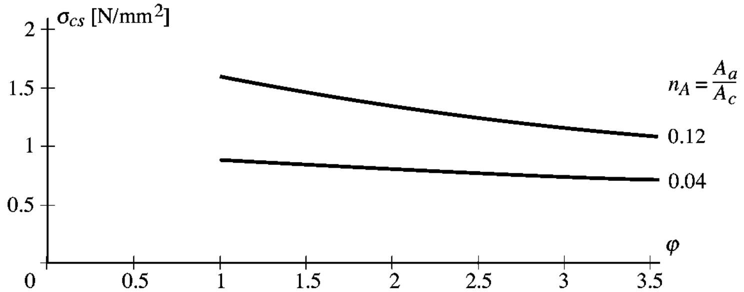


Fig. 13.4 Tensile stress σ_{cs} in the slab due to shrinkage as a function of the creep coefficient φ.

The results of this application example show that for a creep coefficient φ of 2.0, the tensile stresses vary between 0.8 N/mm^2 and 1.4 N/mm^2, as the retention coefficient n_A varies between 0.04 (30 m span) and 0.12 (80 m). These values of tensile stress, therefore, remain below the concrete resistance in tension, and so cannot alone explain the transverse cracking of the concrete across the slab, which may develop in the intermediate support zones of a continuous composite bridge. However, they do, with time, add to the tensile stresses due to other actions. This accumulation of stresses may eventually provoke transverse cracking of the concrete slab.

Effect of Shrinkage in the Steel Beam

In span

In the spans, due to shrinkage of the concrete slab, the steel is normally in compression over most of its depth, whether it is a simple beam (Fig. 13.3) or continuous beam. For the latter the redundant negative bending moments due to shrinkage (Fig. 13.5) make an additional contribution to the compression in the steel section. Therefore, a conservative approach during structural design is to ignore the effects of shrinkage, because the other actions result in tensile stresses in the steel beams in span. In particular:

- When a design uses the *elastic resistance*, the effects of normal force and total moment due to shrinkage generally result in compressive stresses in the lower flange of the steel beam. If these compressive stresses are neglected, then the design is conservative in this region.
- When a design uses the *plastic resistance* for a *simple beam*, the primary effects of shrinkage need not be considered because they disappear when the cross section behaves plastically. For a *continuous beam*, only the redundant moment needs to be considered. However, this moment is negative along the length of the beam. Hence, neglecting it means that the total positive moment in span due to other actions is not reduced. This also means the design will be conservative.

At intermediate supports

At the intermediate supports of a continuous composite beam, the ultimate resistance of the composite cross sections is based on elastic behaviour (§ 13.4.2). This means that all the action effects on the section must be taken into account, including shrinkage effects, which contribute to increased compressive stresses in the lower flange.

The effects of shrinkage on the composite cross section for a beam that is continuous over several spans, with an *uncracked concrete* slab over the intermediate supports, are shown in Figure 13.5(a), see also TGC Vol. 10, § 5.8.5. A simplified estimate of the effects in the steel beam may be made by neglecting the bending moments. The total moments acting at the intermediate supports and in the internal spans are relatively small, particularly when the number of spans increases. If only the normal force acting in the steel beam and the reinforcement is considered, the stress σ_{as} acting in the steel (beam and reinforcement) is given by:

$$\sigma_{as} = \frac{N_{cs}}{A_a + A_s} = \frac{\varepsilon_{cs}(t) \cdot E_a \cdot A_c}{n_s(A_a + A_s)} = \frac{\varepsilon_{cs}(t) \cdot E_a}{n_s(n_A + \rho)} \tag{13.5}$$

N_{cs}	:	normal force due to shrinkage (13.1)
A_a	:	area of the steel beam
A_s	:	area of the reinforcement
n_s	:	elastic modular ratio, taking into account shrinkage, $n_s = E_a/E_{cs}$
n_A	:	retention coefficient, $n_A = A_a/A_c$
r	:	percentage of reinforcement in the slab

Assuming a final value of 0.025% for the shrinkage strain $\varepsilon_{cs,\infty}$, with 1.5% reinforcement above the intermediate supports of the composite beam, and a modular ratio n_s of 12, the compressive stress in the steel beam σ_{as} varies between 80 N/mm^2 and 33 N/mm^2 as the retention coefficient n_A varies between 0.04 and 0.12. This level of compressive stress, which must also be multiplied by the relevant load factor when checking structural safety, is significant. However, before the ultimate resistance of a beam is reached, the concrete in tension at the intermediate supports will crack, and the hypothesis of the calculation (which assumes that the concrete contributes over the whole beam length) is no longer satisfied.

If *cracking of the concrete in tension* at the intermediate supports is taken into account explicitly, then the analysis of the beam subject to shrinkage is more complicated. In a simplified way the model shown in Figure 13.5(b) may be applied, with the forces due to shrinkage applied at the ends of the lengths of beam in span, along the length of which the concrete in homogeneous. Effectively, one assumes that shrinkage becomes negligible, or disappears, when the concrete is cracked [13.1], that is, in the regions of a composite beam above the intermediate supports. As a result of this, the normal force is no longer constant over the length of the beam, and only bending moment is present at the intermediate supports. This moment corresponds directly to the redundant bending moment in these zones, as there is no primary moment.

For this case where the cracked concrete is ignored, based on numerical studies of several composite bridges [13.1], it seems that the compressive stress $\sigma_{cs,inf}$ acting in the lower flange of the steel beam (due to a shrinkage of 0.025%) varies as a function of span and has an average value of 19 N/mm^2. This value, which is more representative of the real behaviour of the beams in a bridge, should be taken into account when checking structural safety. In order to allow for shrinkage at the intermediate support locations, one may assume, therefore, a *design value of 25 N/mm^2* for the compression stress in the lower flange. A corresponding value may be estimated for the tensile stress in the upper flange, taking into account the position of the neutral axis of the cross section.

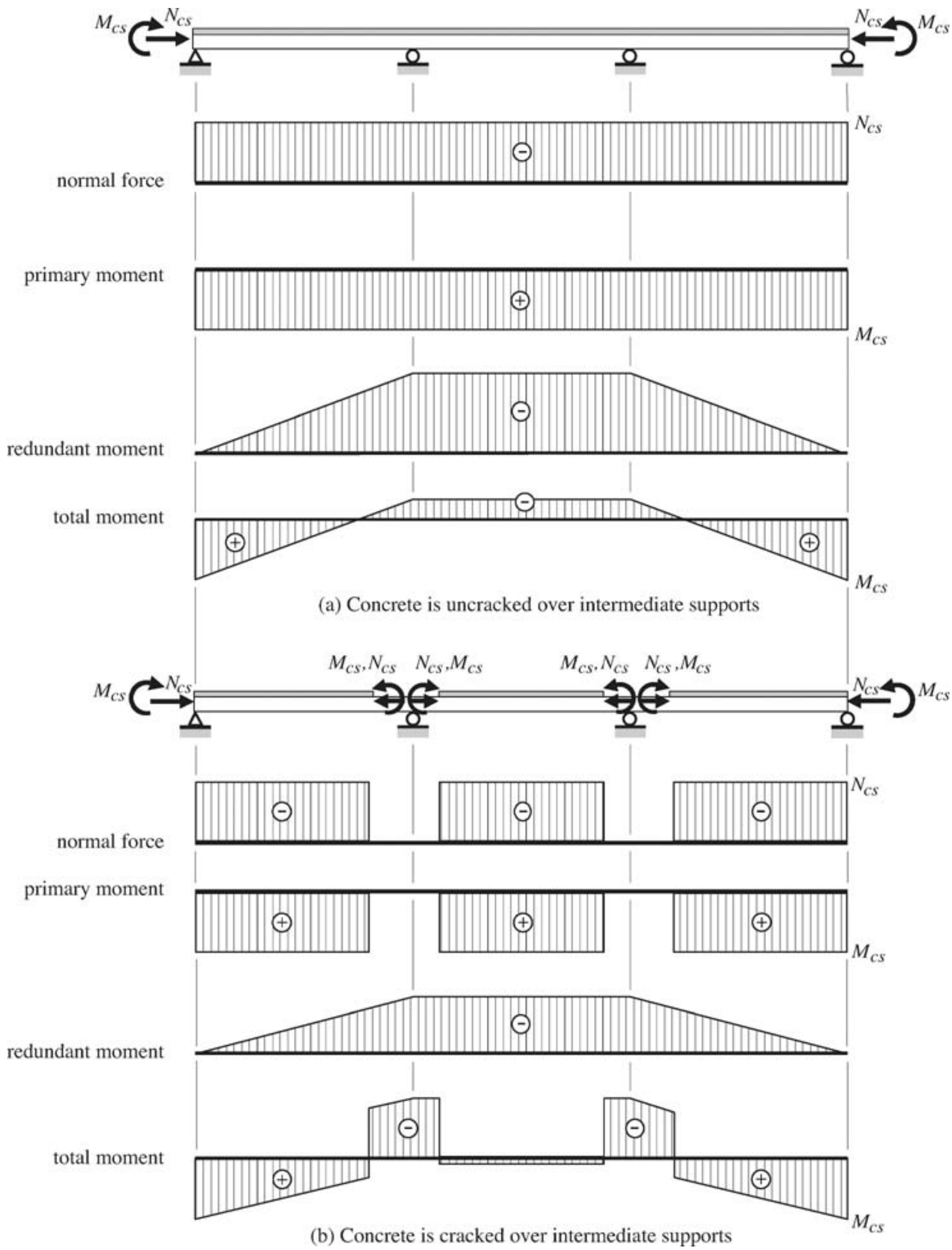


Fig. 13.5 Effect of shrinkage on a continuous composite beam.

13.2.3 Temperature

Actions due to temperature, which should be taken into account when designing a bridge according to the relevant standards, were presented in Paragraph 10.4.2. It was also noted in that paragraph that the load models related to temperature might be very different from reality, notably those that concern temperature gradient.

A *uniform variation in temperature* results in either lengthening or shortening of the beams. These changes in length can normally be accommodated by the range of movement of the expansion joints. Daily *temperature gradients* result in curvature of simple beams and, additionally, redundant moments and forces in continuous beams.

Normal stresses that are in equilibrium across the section or residual stresses may also develop if the temperature gradient only affects part of the depth of the cross section such as, for example, the slab of a composite beam. This is the case even for a simple beam. Generally, the calculation of internal moments and forces due to temperature may be carried out with the same considerations, and in a similar way, as when calculating the effects of shrinkage.

Typically, a *non-uniform distribution of temperature* over the depth of a composite beam cross section is not considered in this analysis because it is not only very difficult to estimate for a specific bridge, but also it is not easy to calculate the effects. This is particularly the case when there is cracking of the concrete in tension.

A temperature gradient results in slow and daily variations in deflections, which normally do not adversely affect either the behaviour or appearance of a bridge. When checking serviceability of composite beams, it can normally be ignored. However, this gradient also results in stresses in the composite cross sections, which should be taken into account when performing the check of structural safety. Within this context, a realistic calculation of the stresses should be based on a realistic temperature distribution, which it is not possible to know in advance.

Concerning this subject, an experimental and analytical study of the behaviour of composite bridges in Switzerland, exposed to measured variations in temperature, allows the following conclusions to be drawn [13.2]:

- Temperature gradients in composite cross sections develop mainly over the slab thickness.
- Over the depth of the steel beam, the temperature variations are more pronounced in the web than in the flanges (the flanges are thicker).
- Temperature varies more rapidly in the steel than in the concrete.
- The thicker the surfacing of the slab, the smaller the temperature gradient in the slab.
- Solar reflection from the ground amplifies the temperature gradient in the steel beams for bridges near to the ground.
- The maximum stresses calculated for the steel beam and concrete slab do not result from the same temperature distribution in a composite section. Also, these distributions do not normally correspond to the greatest temperature gradient.
- The influence of non-uniform temperature distribution on the stresses is sufficiently important that it should not be neglected in calculations.

Figure 13.6 shows a temperature distribution resulting in maximum compressive stresses in the steel beam. This example is calculated for a daily variation of extreme temperatures and a bridge of 42 m span. It is worth repeating that another temperature distribution will induce the maximum tensile stresses in the slab.

Despite the complexity of the phenomenon, the study noted above nevertheless highlights that the stresses in composite sections, which appear due to daily variations in temperature, are relatively independent of the dimensions of the section and, therefore, the span. For the daily extremes of temperature found in Switzerland, the corresponding values of stress are given in Table 13.7. These are for a bridge

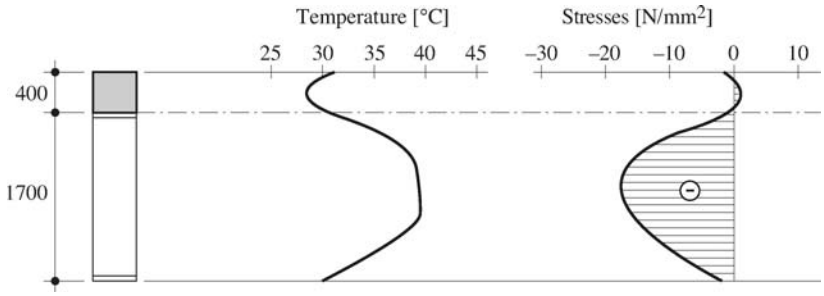


Fig. 13.6 Example of temperature distribution inducing maximum compression stresses in a composite section.

with an open cross section, with 100 mm thick surfacing covering a 400 mm thick slab above the steel beams. The values of stress given in this table are those that are of interest to the design engineer, namely the tensile stresses in the concrete and the compressive stresses in the steel.

Table 13.7 Stresses in a composite cross section due to maximum daily temperature variations.

	Span 40 m			Span 130 m		
	top fibre	inside	bottom fibre	top fibre	inside	bottom fibre
Maximum tension stresses in the slab [N/mm^2]	0.2	1.6	1.6	–0.4	1.4	1.1
Maximum compression stresses in the steel [N/mm^2]	–3.0	–16.0	–4.0	–4.0	–16.0	–2.0

It can be noted from Table 13.7 that the values of maximum stress are not greatly influenced by the span, neither do they necessarily occur in the extreme fibres of the section. Simulations have shown that the maximum values in span and at the supports are similar.

As a conclusion from this study, it may be assumed that, over the whole length of an open section composite bridge, due to the effect of an extreme daily temperature variation as found in Switzerland:

- The slab may be in tension over its lower half, and even up to three quarters of its depth, with a stress that may reach 1.6 N/mm^2.
- The steel beam may be in compression over the whole depth of its web, with a stress that may reach 20 N/mm^2. The lower flange is generally in compression with a stress of around 5 N/mm^2.

These values, coming from extreme variations in temperature, do not normally need to be multiplied by a load factor when checking structural safety. In some cases a smaller load factor that includes only the effects of modelling uncertainty may be appropriate.

Simulations have also shown that, in the absence of surfacing, the tensile stress in the lower part of the slab may reach 2.5 N/mm^2. The surfacing, however, does not have a noticeable effect on the stresses in the steel section. As the slab thickness reduces, the maximum tensile stresses tend to reduce to around 0.7 N/mm^2 for a 250 mm slab.

For a box girder composite bridge, the maximum tensile stresses in the slab are roughly the same as those for an open cross section. The maximum compressive stresses in the steel are also the same, although the maximum is situated in the lower flange because the temperature in this flange is more or less identical to that in the web (for a twin girder bridge the flange temperature is lower).

When checking structural safety it is recommended that the stresses due to the development of a temperature gradient need not be explicitly calculated. These stresses remain small in the steel beams. For a check of a steel beam above an intermediate support, assuming elastic behaviour, the numerical values given above may be used. In span, considering elasto-plastic behaviour, the majority of the stress disappears and, consequently, need not be considered.

13.3 Calculation of Internal Moments and Forces

13.3.1 Principles

For the structural analysis and design of a composite bridge, the calculation of the bending moments and shear forces is normally based on a model that assumes elastic behaviour of the load carrying structure. In this elastic analysis each action must be taken into account considering the structure that resists it. In particular one should distinguish between:

- the *steel beam* alone, which must resist certain actions during the erection of the steel structure and the construction of the concrete slab, and
- the *composite beam*, which must resist those actions that occur after connecting the steel and concrete.

For both of these two situations, the distribution of the second moments of area of the *resisting cross sections* along the length of the beam must be considered. For a composite bridge, *cracking* of the concrete in tension at the intermediate supports results in a reduction in beam stiffness in these regions. This reduced stiffness should be taken into account when calculating the internal moments and forces due to actions affecting the composite beam. Additionally, when calculating the second moment of area of the resisting composite cross sections, it is necessary to consider an *effective width* of the concrete slab. This is defined as the width over which one may assume a uniform distribution of normal stresses.

13.3.2 Resisting Cross Sections

When calculating internal moments and forces, it is necessary to take into account the methods to be used for erection of the steelwork and construction of the slab. It is particularly important to know the moment at which the permanent and variable actions will be applied to the beam. This knowledge of the load history enables the resisting section – steel alone or composite – to be defined at each stage. The distribution of second moments of area, needed to calculate the internal moments and forces, can then be identified.

The influence of the method of erection of the steelwork and the construction of the slab on the internal moments and forces acting in the beams have already been identified in Sections 7.5 and 8.4, respectively. As an example, Figure 13.8 shows the load history and the resisting sections used to determine the bending moments for a slab cast on formwork that is fixed to the steel structure, assuming temporary support of the beams during concreting. For this example, if the concrete is to be cast in several stages, then different resisting sections must be considered, either composite or steel alone, when calculating the moments due to concreting of the slab (§ 8.4.4).

For *calculation of the elastic resistance* of composite cross sections, the duration of load application should be taken into account. This is achieved by introducing different modular ratios *n*, as a function of the duration of load application, when calculating the characteristic properties of the resisting cross sections (§ 13.4.2). This means that the distribution of second moments of area along the length of a composite beam is different when a permanent action is present from that when traffic (a short term action) is present.

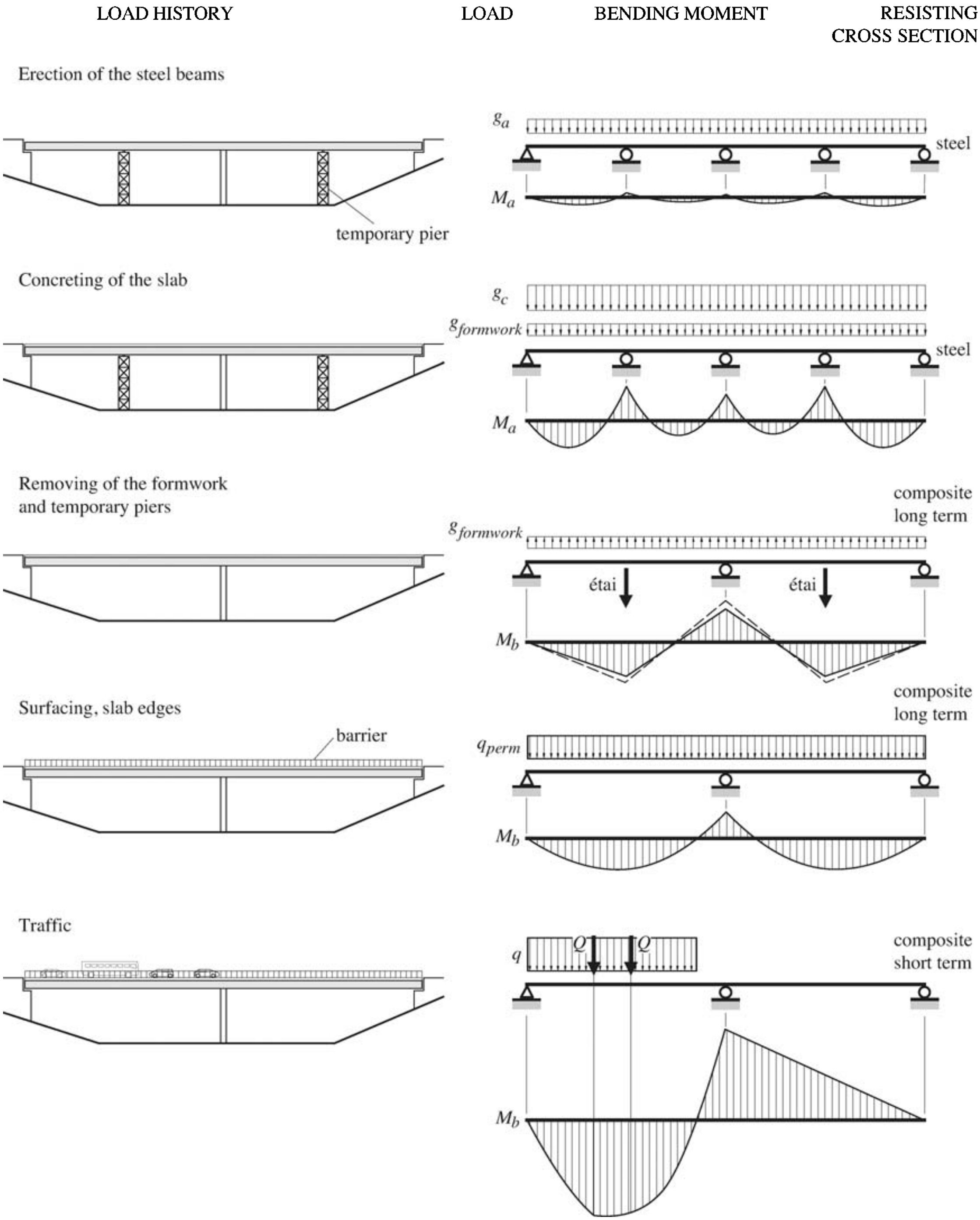


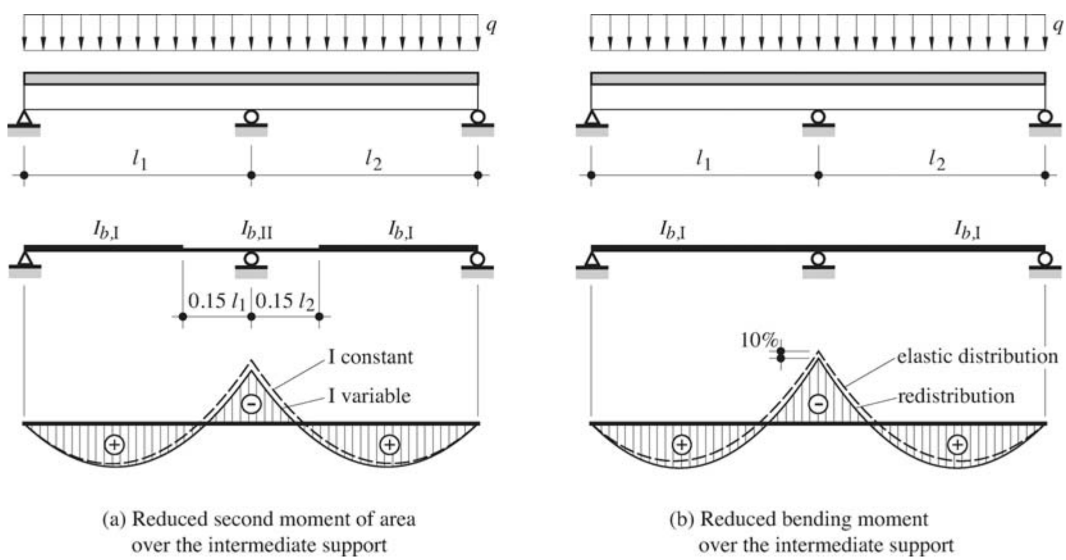
Fig. 13.8 Example of structural forms and resisting cross section types for calculating bending moments as a function of the load history.

Practically, however, it is accepted that for calculating internal moments and forces, this difference has only a minor impact, thus a single distribution may be assumed. Normally, this distribution considers the second moments of area calculated with a modular ratio n for short term actions. The effects of concrete cracking at the intermediate supports must clearly be taken into account when determining the distribution of composite beam second moments of area (§ 13.3.3).

For *calculation of the plastic resistance* of composite cross sections in span (§ 13.4.3), it should be remembered that the load history has no effect. On the other hand, the bending moments should always be calculated with due consideration of the actions that act on the steel structure alone and those that act on the composite structure.

13.3.3 Influence of Cracking

Cracking of the concrete in tension above the intermediate supports should be taken into account when determining the second moments of area of the composite cross sections in these regions. The resisting cross section comprises the steel beams and the slab reinforcement. Strictly speaking, in order to define the length over which cracking occurs, an initial analysis of the continuous beam is needed, taking into account the contribution of the concrete in tension over the total length of the beam. Then, in the negative moment regions over the lengths within which the tensile resistance of the concrete is reached or exceeded, a reduced second moment of area should be defined, ignoring the presence of the concrete for the subsequent analysis. For the initial analysis Eurocode 4 proposes that the characteristic load case for serviceability (rare load case according to § 9.5.2) is used, including long term effects. Then, using the calculated bending moment diagram, the zones where the theoretical stress in the extreme concrete fibre exceeds twice the average tensile resistance f_{ctm} of concrete (f_{ctm} = 2.9 N/mm^2 for grade C30/37) should be assumed to have a cracked slab in the next calculation iteration.



(a) Reduced second moment of area over the intermediate support

(b) Reduced bending moment over the intermediate support

Fig. 13.9 Calculation of elastic bending moments in a continuous composite beam.

A simplified method is also available for taking into account the reduction in second moment of area at the supports, as illustrated in Figure 13.9(a). Over a length $0.15l$ on either side of the intermediate support, a reduced second moment of area $I_{b,II}$ (state II) is used, calculated assuming cracked concrete. Over the rest of the span, a second moment of area $I_{b,I}$ (state I) is used, calculated assuming the concrete is uncracked. The second moment of area $I_{b,II}$ corresponds to a cross section comprising the totality of the steel beam plus the reinforcement present within the effective width of slab. It should be noted that this simplified method should only be used for continuous beams having a ratio between adjacent spans l_{min}/l_{max} greater than 0.6. When the slab comprises precast elements, the cracked zone is shorter, because there is less shrinkage than in the case of concrete poured in-situ.

For initial sizing another approach may be adopted. The bending moments are calculated assuming a second moment of area $I_{b,I}$ throughout, therefore assuming the concrete to be uncracked throughout (even where it is in tension). Then the bending moments at the supports are redistributed by a default value of 10% to take concrete cracking into account (Fig. 13.9(b)). The moments in span must be increased correspondingly. This default reduction only applies to the bending moments due to actions that occur after the steel-concrete connection has been formed, and for continuous beams with similar spans.

13.3.4 Effective Slab Width

The definition of the effective width of the concrete slab is given in TGC Volume 10, Paragraph 5.8.2. This effective slab width, over which the normal stresses are assumed to be uniform, is used to model the real non-uniform distribution of stresses over the total width of the slab. This non-uniform stress distribution is due to the phenomenon known as *shear lag*. The effective width depends on the structural form, the type of loading (concentrated or distributed), and the position of the loads relative to the cross section being considered. A number of numerical simulations of composite beams used in bridges have allowed different models for calculating the effective width to be defined, including those proposed in the SIA standards and the Eurocodes. Figure 13.10 shows a definition of *effective slab width*, which is similar to that proposed in Eurocode 4 and used, as a simplification, for all load types.

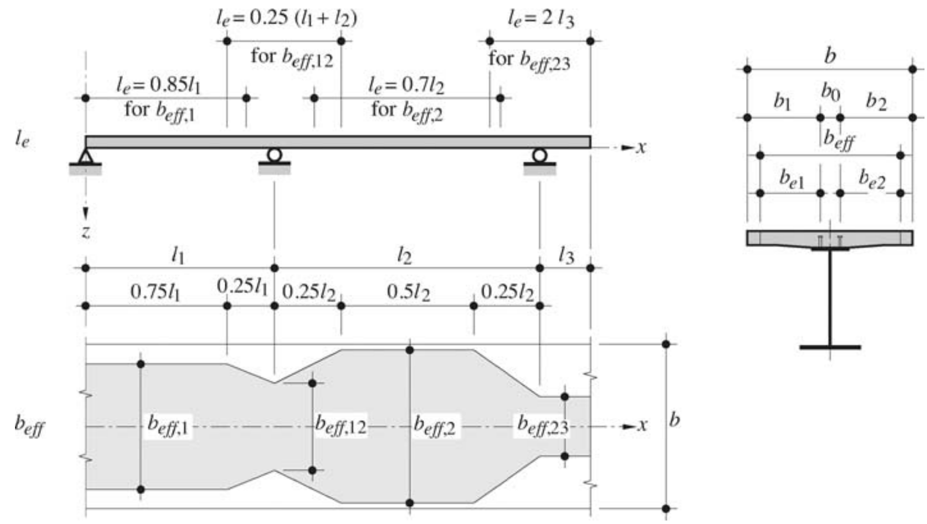


Fig. 13.10 Effective width of the concrete slab.

In this model the effective slab width b_{eff} varies depending on whether it is calculated at a support or in span, as shown in Figure 13.10. The effective width is defined by the following equation:

$$b_{eff} = b_0 + \sum b_{ei} \tag{13.6}$$

b_0 : distance between the external rows of shear studs
b_{ei} : effective slab width on each side of b_0

The value of b_{ei} is defined as:

$$b_{ei} = \frac{l_e}{8} \le b_i \tag{13.7}$$

l_e : approximate distance between the points of zero moment (Fig. 13.10)
b_i : distance between the edge row of studs and the edge of the slab, or the distance between these studs and half the distance to the adjacent beam

The effective slab width, along with its depth, allows calculation of the area of concrete slab to be considered as part of the resisting composite cross section. This then enables calculation of the second moment of area I_b and elastic section modulus $W_{el,b}$ to be used in calculation of the composite cross section elastic resistance. As a conservative approach the same effective width may be used to define the contribution of the slab to the plastic resistance of the composite cross section, and to calculate the plastic section modulus $W_{pl,b}$.

13.4 Cross Section Resistance and Verification of Structural Safety (ULS)

13.4.1 Section Classification and Resistance Models

The resistance of a composite cross section may be determined using either a *plastic resistance* model or an *elastic resistance* model. The cross section resistance is a function of the behaviour of those parts of the plate girder that are in compression. The more slender the plate elements, the more rapidly local instability phenomena (local buckling) develop, thereby limiting the resistance of the cross section. Classes have been defined for steel sections (TGC Vol. 10, § 12.3.2) based on limiting slenderness values for their individual plate elements in compression. As for steel plate girders (§ 12.2.1), the choice of resistance model to be applied depends on the class to which the composite cross section belongs. The choice of a resistance model (elastic, plastic) is therefore a function of:

- the classification of the steel section in the region being considered,
- the classification of the other sections that make up the beam (§ 13.4.3),
- the type of check being carried out: serviceability, fatigue, or structural safety.

The section classification is based on slenderness limits for the different plate elements, which are defined in the various codes and standards. For plate girders, when the non-dimensional slenderness $\overline{\lambda}_p$ for buckling of plates in compression is not less than 0.9, the section is class 4, and calculating its resistance involves determining effective widths. For such a section an *EER calculation* should be used: calculation of the elastic moments and forces, and calculation of the reduced elastic resistance (as the total area of the section is not effective). This type of calculation is typical for composite sections in negative moment regions, where the concrete slab is in tension and assumed cracked when determining the properties of the resisting cross sections. Effectively, the concrete in tension makes no contribution to the section resistance, and the webs of the plate girders are in compression over a significant part of their depth and so do not normally satisfy the criterion of $\overline{\lambda}_p < 0.9$ for the non-dimensional slenderness.

When the non-dimensional slenderness $\overline{\lambda}_p$ is less than 0.9, an *EE calculation* is possible, which is calculation of the elastic moments and forces and calculation of the elastic resistance of the composite cross sections without any loss of resisting steel area. This type of calculation is used in positive moment regions. It may also be used in negative moment regions, for example for short span bridges having cross sections that are not class 4.

When the cross sections are class 1 or 2, an *EP calculation* may be used, which is a calculation of the elastic moments and forces with a calculation of the plastic resistance of the composite cross sections. Because the neutral axis position of composite cross sections in span is close to the upper flange, almost the whole of the steel section is in tension, so an EP calculation is possible, as long as certain conditions discussed in Paragraph 13.4.3 are satisfied.

As far as serviceability and fatigue checks are concerned, as well as checks on the different phases of erection of the steel structure, the behaviour of the beams must remain elastic; hence, only an elastic resistance model can be used.

Table 13.11 summarises the different calculation models that are possible when analysing and designing a composite plate girder.

Table 13.11 Possible calculation models for cross section resistance.

Limit States	Support Regions	Span Regions
Serviceability, fatigue, erection phases	EER, EE	EE, EER*
Structural safety	EER, EE	EE, EP**
* eventually necessary during erection ** see restrictions at Paragraph 13.4.3		

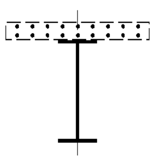
13.4.2 Elastic Resistance

The ultimate resistance of a composite cross section based on an elastic resistance model is still widely used for composite bridges. This type of calculation requires all the actions to be taken into account, including those due to different effects of concrete behaviour, and it must recognise the load history. A simplified model that is typically used for this approach comprises replacing the concrete part of the composite cross section by an equivalent steel part, which is defined considering the modular ratio n_{el} (TGC Vol. 10, § 4.7.2). This is the ratio E_a/E_{cm} between the elastic moduli of steel and concrete. The modular ratio reflects the characteristics of concrete in that it varies depending on whether the actions on the beam are associated with short term or long term application, or are related to shrinkage. This means that a calculation of elastic resistance requires the definition of several resisting cross sections as a function of the duration of the action.

Since the resisting cross sections vary, it is not possible to define a unique value of the elastic moment resistance $M_{el} = f_y \cdot W_{el}$. This means that an elastic analysis model comprises: calculating the stresses acting on different resisting cross sections and adding these stresses together to determine the design values of stress acting on different parts of the section.

Table 13.12 summarises the resisting cross sections to consider when calculating the elastic resistance in span and at intermediate supports, as well as the corresponding actions. To determine the properties of a composite cross section, the modular ratio n_{el} may be calculated in a simplified way, assuming that the elastic modulus of concrete is two or three times smaller than the average modulus E_{cm} for actions associated with shrinkage and long term load application, respectively.

Table 13.12 Elastic cross section resistance and corresponding actions.

	Cross Section Resistance	Actions
Span	 steel	*Loads during erection* examples: self weight of the steel beam, of the concrete slab (if no temporary supports) construction loads.
	 composite $n_{el} = \frac{E_a}{E_{cm}} = n_0$	*Short term* examples: traffic, temperature.
	 composite $n_{el} = 3\frac{E_a}{E_{cm}} = n_\varphi$	*Long term* examples: surfacing, mechanical components, long term actions after steel-concrete connection, temporary support reactions.
	 composite $n_{el} = 2\frac{E_a}{E_{cm}} = n_s$	*Shrinkage or similar action effects* examples: support settlement and loss of prestress.
Intermediate support	 steel	*Loads during erection* examples: self weight of the steel beam, of the concrete slab (if no temporary supports) construction loads.
	 steel and reinforcement	*All loads* acting after steel-concrete connection.

For composite cross sections at intermediate supports, the concrete in tension is assumed to make zero contribution to the resistance. Therefore, the resisting cross section comprises the steel beam and the reinforcement within the effective width of slab.

When, during erection of the steelwork, the steel beam alone constitutes the resisting cross section, or when the resisting composite cross section at a support comprises the steel beam plus reinforcement, the elastic neutral axis is close to the mid-depth of the steel web and the section is often class 4. To determine the stresses acting on such sections, calculation of the elastic section modulus $W_{c,eff}$ (§ 12.2.5) must take into account an effective depth of web in compression $h_{c,eff}$. This effective depth is determined considering, by use of the stress ratio ψ, the initial distribution of stresses calculated for the whole steel section. If done rigorously, at the intermediate supports this process will involve consideration of each of the resisting cross sections with corresponding stresses. However, simplifications to this procedure, as shown in Figure 13.13, result in values that are very close to those obtained from rigorous calculations.

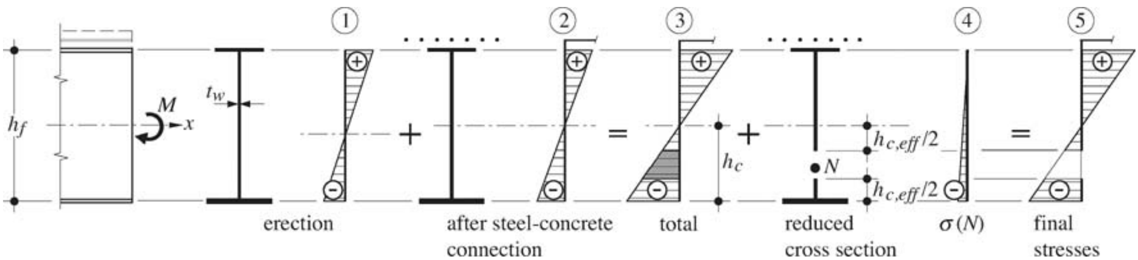


Fig. 13.13 Distribution of stresses in a composite cross section situated in a negative moment region.

In *negative moment* regions the effective depth of web $h_{c,eff}$ may be calculated based on the distribution of total stresses (distribution ③), rather than calculating two different effective depths based on the stress distributions ① and ②. To find the final stresses (distribution ⑤) acting on the reduced cross section, the stresses due to the notional force N, which is eccentric to the neutral axis (distribution ④), should be added to the stress distribution ③. This force N corresponds to the stresses that can not be resisted by the depth of web between the two effective parts $h_{c,eff}/2$. One may also recalculate new distributions ① and ② acting on the effective section, and add them together to obtain the final distribution ⑤.

In *span*, during erection before the steel structure is connected to the concrete slab, the effective depth of web $h_{c,eff}$ may be calculated, recognising that the design value of stress acting in the compression flange is limited to the lateral torsional buckling stress (§ 12.2.4). For initial sizing, because the lateral torsional buckling stress is not yet known, it can be assumed that $h_{c,eff}/2 = 25 \cdot t_w$. This approximation comes from application of Equation (12.25), and is sort of an average appropriate for plate girders in span as used for composite bridges.

13.4.3 Plastic Resistance

For a continuous plate girder, it is possible to calculate the ultimate composite cross section resistance using a plastic resistance model in the regions of positive moment (in span), because almost the totality of the steel section is in tension in such regions. Calculation of the design value of the plastic moment resistance $M_{pl,Rd}$ is carried out according to the method described in TGC Volume 10, Paragraph 4.7.4.

The advantage of a calculation method that takes into account plastic cross section behaviour when determining its ultimate resistance is that a number of effects can be ignored, and so the analysis is greatly simplified. These are the load history (loads acting on the steel section alone during erection, or subsequently permanent and imposed loads acting on the composite structure) and the effects of internal residual stresses or of imposed deformations (shrinkage, temperature, support settlement). On the other hand, the following points must be considered when determining the plastic resistance in span of a composite bridge:

- ductility of the composite cross section,
- redistribution of bending moments, and
- consequences for serviceability.

Each of these points is considered below.

Ductility

The plastic resistance of a section in bending is reached when the total cross section is plastified (TGC Vol. 10, sect. 4.3). Taking into account the history of loads that act on the different resisting sections (§ 13.3.2), and the fact that the shape factor of a composite section is greater than that of a steel I section (TGC Vol. 10, § 4.7.4), the deformation capacity, or ductility, of the section must be high. This deformation capacity in bending is defined by the moment-curvature relationship. Figure 13.14 shows a schematic moment-curvature relationship for a composite cross section subject to positive bending moment. The first case (Fig. 13.14(a)) represents that of a beam, which is completely propped during construction. The second case (Fig. 13.14(b)) is for an unpropped beam, so the steel section alone is initially subject to a bending moment M_g due to self weight, and subsequent loads act on the composite section. The plastic moment resistance is identical for these two cases, but for the unpropped beam, the total curvature ϕ_{tot} needed to attain M_{pl} is greater. On the other hand, for the unpropped beam, plastification of the section begins at a lower level of load, represented by M_{el}, than for the propped beam.

In reality, for slender beams as used in composite bridges, to attain the full plastic moment resistance M_{pl} of the section in span, there must be a large amount of curvature, and it is difficult to guarantee that

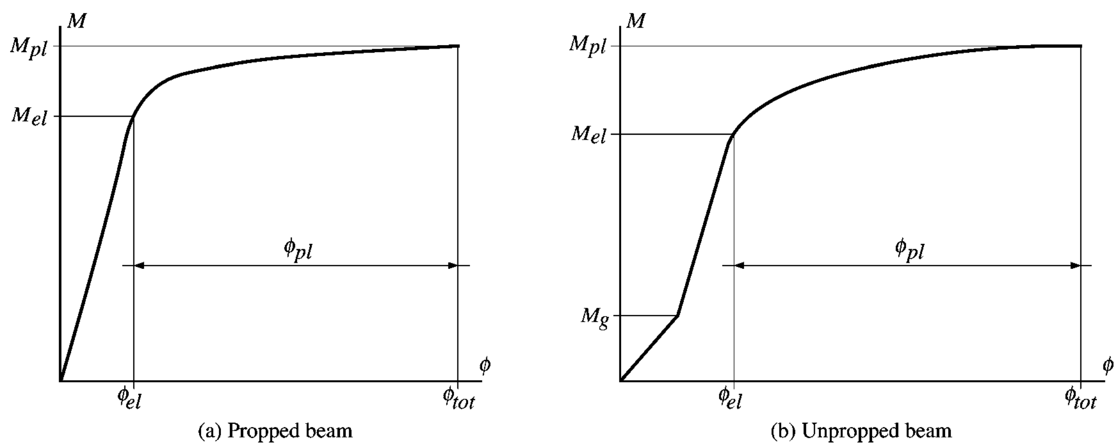


Fig. 13.14 Schematic moment-curvature relationship for a composite cross section subject to positive bending moment.

the full resistance can be achieved without other consequences for the general behaviour of the composite beam. This is particularly the case when the beam is continuous.

Numerical simulations [13.1] have shown that for typical composite bridge cross sections, for a plastic curvature of $\phi_{pl} = 5\ \phi_{el}$, the lower flange and over half of the upper flange are plastified. In this situation the moment attained is approximately 95% of the plastic moment for a propped beam, and 90% for an unpropped beam. In recognition of this difficulty in obtaining full plastification of the cross section, it is accepted that the design value of the ultimate resistance of a composite section is span is:

$$M_{Rd} = 0.95 M_{pl,\,Rd} \text{ for beams that are propped during erection and,} \tag{13.8}$$

$$M_{Rd} = 0.90 M_{pl,\,Rd} \text{ for beams that are unpropped} \tag{13.9}$$

These limitations are appropriate for *continuous beams* because they are linked to the calculation hypotheses concerning redistribution of bending moments from the spans to the intermediate supports and limitations concerning the ratio between adjacent spans (13.10) that justify use of a plastic calculation. For *simple beams* there is obviously no restriction concerning adjacent spans and the full plastic moment resistance can normally be attained, namely $M_{Rd} = M_{pl,Rd}$.

Finally, given the different ways in which a composite beam may be designed, it is possible that the plastic neutral axis will descend to low down in the web of the steel beam. This can reduce the ductility of the section, notably by leading to concrete crushing. Studies [13.3] have shown that if the depth of web in compression h_c is less than 35% of the total web depth, a plastic resistance calculation is acceptable.

Use of high strength steel in a composite section also results in a lower neutral axis position, and therefore reduced ductility. To take into account this phenomenon, when S420 or S460 steel is to be used, the relevant codes and standards recommend that the design value of plastic moment resistance $M_{pl,Rd}$ is reduced in Equations (13.8) and (13.9) by using a coefficient β, shown in Figure 13.15.

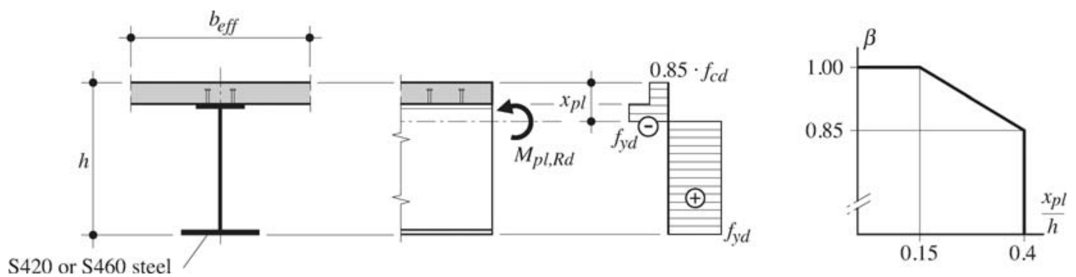


Fig. 13.15 Reduction of the plastic resistance moment when S420 or S460 steel is used.

Redistribution of Bending Moments

For continuous beams, when the composite cross sections in span begin to plastify, they lose bending stiffness and, as a consequence, there is a redistribution of bending moments towards stiffer regions, namely the intermediate supports (Fig. 13.16(a)). In other words, the moments in span increase less quickly than the moments at the supports, so the form of the bending moment diagram no longer corresponds to that determined assuming elastic behaviour.

Typically for a bridge, the cross sections at the intermediate supports are class 3 or 4, in which case they can only accept this redistribution when the magnitude of the redistributed moment is less than the elastic resistance of the section at a support. However, because the sections at the supports and in span are determined for two different load positions (Fig. 13.16), the cross sections at the supports defined for load position 2 ($M^-_{2,Ed} \leq M^-_{el,Rd}$) are normally able to resist the additional (redistributed) moments $M^-_{1,Ed,red}$ due to load position 1.

Because a calculation of applied moments and forces assuming elastic behaviour provides no indications concerning the magnitude of the redistributed moments, it may be that, depending on the ratios between adjacent spans, the redistributed moments due to load position 1 are greater than the elastic resistance at the supports ($M^-_{1,Ed,red} > M^-_{el,Rd}$). Such cases clearly fail the structural safety checks at the supports and, hence, are not acceptable. Numerical simulations (performed for the condition $\phi_{pl} = 5\ \phi_{el}$) have allowed a limit to be defined for the ratio between adjacent spans, within which plastic resistance may be assumed in the spans:

$$\frac{l_{min}}{l_{max}} \geq 0.6 \qquad (13.10)$$

l_{min} : adjacent short span
l_{max} : adjacent long span

Based on the results of numerous studies carried out concerning real bridges, it seems that for load position 1, the increase in moments at intermediate supports, due to redistribution, is around 20% for a two span bridge and 15% for a bridge with three spans. It is important to check that the intermediate cross sections located between the supports and mid-span regions can resist the envelope of redistributed moments.

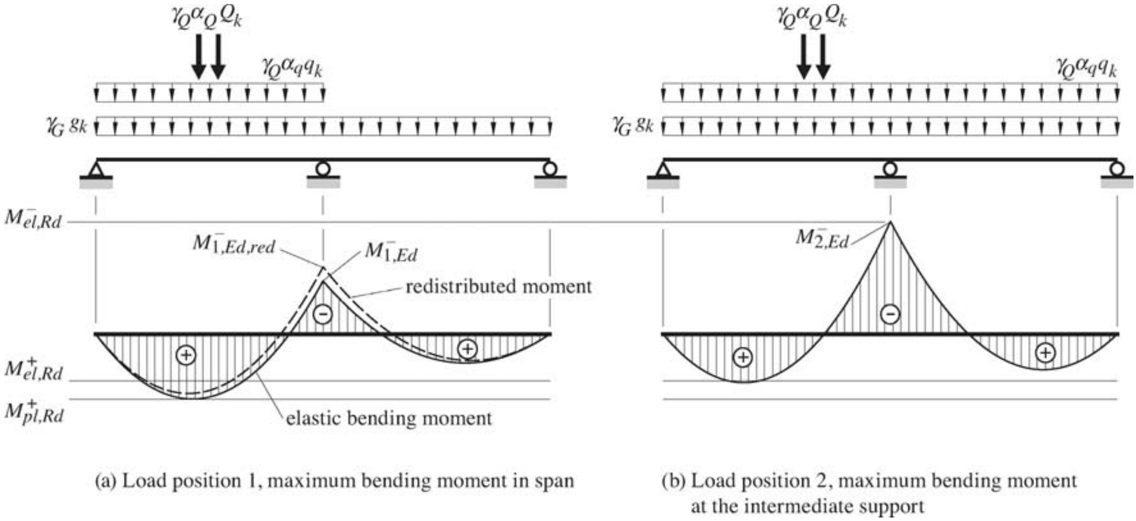


Fig. 13.16 Moment redistribution due to plastification in the span.

Consequences for Serviceability

A structural safety check for a composite beam based on calculation of the plastic resistance in span normally implies a different distribution of material from that resulting from a calculation of elastic resistance. In particular, this distribution could lead to a reduction of the material in span to such an extent that it became possible to enter into the plastic domain under serviceability loading. Plastification in service would result in permanent deformation of the beam, which generally is undesirable for a bridge. Therefore, when the ultimate resistance is calculated assuming plastic behaviour, it is necessary to calculate the stresses in service under the rare load case, taking into account load history, and to ensure that these stresses remain below the yield strength of the steel.

13.4.4 Verification of Structural Safety in Bending (ULS)

Verification of structural safety in bending, as for shear, should be carried out over the whole length of a beam, with particular attention paid to the regions where the envelope of applied moments is a maximum and to the regions where there is a change in the dimensions of the steel section. Structural safety must also be checked for each erection phase, as well as the final state, for which there is imposed loading. The actions to consider in these checks are determined in the various load cases and are defined by the hazard scenarios as part of the design process.

Elastic Calculation

In span

Checking structural safety comprises comparing design values of applied stresses with design values of resistance, taking into account the loading history and various resisting cross sections (Tab. 13.12). Figure 13.17 shows the process for calculating the stresses in the case of an unpropped bridge (Fig. 13.17(a)) during concreting, and for the same bridge when propped (Fig. 13.17(b)).

With a rigorous approach this calculation should be carried out for time $t = 0$, that is the moment at which the structure starts to be used, and at time $t = \infty$ when the different effects of concrete behaviour have occurred (in particular shrinkage and creep). Creep results in a redistribution of stresses from the slab to the steel section. After an infinite time the stresses due to shrinkage, according to the principles shown in Figure 13.3, should be added to those shown in Figure 13.17. It can be seen that after an infinite time, the lower flange is normally more highly stressed than at time zero, whereas the slab is more stressed at the start. The slab is also more highly stressed when the beam is propped during construction, while the lower flange is subject to greater tension when the bridge is unpropped.

In order to check structural safety, one must demonstrate that in the extreme fibres of the section, those furthest from the neutral axis, the material strength is not exceeded. This is done using the following general expression:

$$\sum \sigma_{Ed} \leq f_d \tag{13.11}$$

$\sum \sigma_{Ed}$: sum of the design values of the stresses
f_d : design value of material resistance: $f_d = f_{yd} = f_y/\gamma_a$ for steel and $f_d = f_{cd} = f_{ck}/\gamma_c$ for concrete
f_y : yield strength of the steel in the region being considered
f_{ck} : characteristic concrete cylinder strength
γ_a, γ_c : resistance factor for the material: $\gamma_a = 1.05$ for steel and $\gamma_c = 1.5$ for concrete (recommended values that may be altered nationally)

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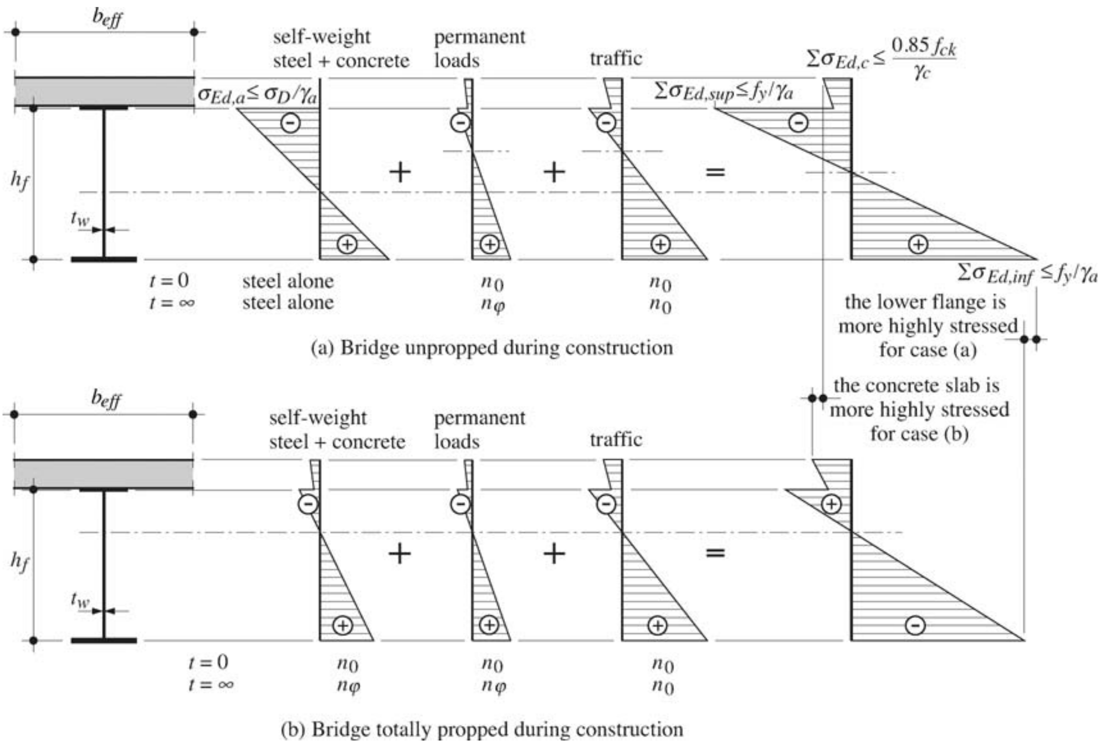

Fig. 13.17 Distribution of stresses for a cross section in the span, and limiting values.

For the situation that is often found in practice, namely an unpropped beam as shown in Figure 13.17(a), an important structural safety check during erection concerns the upper flange of the steel beam. This flange is in compression, and the beam must possess sufficient resistance to lateral torsional buckling. Because during erection the upper flange is not yet connected to the concrete slab, it is not restrained laterally, other than at cross bracing positions, so is susceptible to buckling. The relevant design check is given below:

$$\sigma_{Ed,a} \leq \frac{\sigma_D}{\gamma_a} \tag{13.12}$$

$\sigma_{Ed,a}$: design value of the compressive stress in the flange due to actions prior to connecting the steel to the concrete (self weight of the steel and concrete, construction loads)

σ_D : lateral torsional buckling stress according to Paragraph 12.2.4.

At intermediate supports

In the regions of the intermediate supports, the concrete is in tension and assumed to be cracked, therefore it makes no contribution to the cross section resistance. In this case the stresses are calculated in accordance with the principles shown in Figure 13.13. Checking the stresses in the reinforcement and upper steel flange is carried out according to (13.11), according to (13.12) for the compression stresses in the lower steel flange, which must be checked for lateral torsional buckling considering all the actions that are present.

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14 Cross Bracing and Plan Bracing

Cross bracing above a support. Access viaduct to the freeway junction at Aigle (CH).
Eng. Piguet & Associés, Lausanne.
Photo ICOM.

14.1 Introduction

This chapter considers the forces that act on the cross bracing and plan bracing of both steel and composite bridges. It also presents analytical methods that are typically used to calculate the internal moments and forces to consider when designing the structural elements that make up the different types of cross and plan bracing.

Section 5.6 described different conceptual design options for cross bracing, and studied the functions of this bracing. Construction details for cross bracing, and how it is joined to the main steel beams, were presented in Section 6.4. Then, Sections 5.7 and 6.5, respectively, described different types of plan bracing, and explained their conceptual design.

Section 14.2 describes the loads and other actions to consider for the structural design of cross and plan bracing of beam bridges, taking into account the different functions of the structural elements. The forces acting on the cross bracing are then considered, together with the resulting internal moments and forces that vary according to the conceptual design of the bracing, and the type of transverse cross section (open or closed) of the bridge (Sect. 14.3). Section 14.4 deals with these same issues for plan bracing. Finally, in Section 14.5, structural safety and the stiffness of cross and plan bracing are examined.

14.2 Loads and Actions

The different actions to consider for the conceptual and structural design of the cross and plan bracing are a result of the functions that these bracings fulfil in a bridge, or, indeed, the functions that the designer wants them to fulfil. Therefore, it is useful to be reminded of these functions in order to better understand the loads and actions that need to be taken into account when calculating the internal moments and forces and when designing the bracing.

14.2.2 Functions of Cross and Plan Bracing

The main beams, the cross bracing, and the plan bracing together form the planes of the spatial structure of a bridge which allow the vertical actions and horizontal forces to be transferred to the foundations (Sect. 5.2). The cross bracing and plan bracing have the principal function (§ 5.6.1 and § 5.7.1) of resisting and transferring the horizontal forces due to:

- wind (§ 14.2.2),
- restraining the main beams against lateral torsional buckling (§ 14.2.3).

In addition to these aspects of load transfer, the cross bracing also fulfils the following significant function:

- maintain the shape of the transverse cross section of the bridge, and introduce torsion (§ 11.2.4) into the beam (§ 14.2.4).

Depending on the conceptual design, the cross bracing may also fulfil the following functions:

- act as supports for jacks that may be used to lift the bridge during repairs, or when replacing bearings at the piers or abutments (§ 14.2.5),
- act as supports for any ducts or inspection walkways that are attached to the bridge,
- act as supports for the slab formwork, depending on how the slab is constructed.

The moments and forces acting on the cross bracing due to the last two functions should be defined for each specific case, depending on the particular conceptual design of a bridge and its cross bracing. They are not considered below.

It is particularly important to consider the horizontal forces due to *wind* and *restraint against lateral torsional buckling* of the main beams when designing the cross and plan bracing of a bridge with an open cross section. In span, each set of cross bracing transfers these forces to the plan bracing. At the supports, the cross bracing transfers the sum of these forces from the plan bracing to the piers or abutments.

As a result of their closed cross section, box girder bridges do not possess, strictly speaking, plan bracing. They resist wind by horizontal bending of the box. Neither are box girders subject to lateral torsional buckling, again as a result of their closed cross section. The sets of cross bracing, or diaphragms, in the span of box girder bridges, therefore, are not directly stressed by the wind actions (unlike twin girder bridges), although they may be indirectly stressed by introducing a torque due to wind into the box. On the other hand, the cross bracing at the piers and abutments of box girder bridges must be able to transfer the wind loads, which act horizontally on the girder, to the supports.

The other essential function of the cross bracing is to *maintain the shape of the transverse cross section* of a bridge. In the case of a bridge that is subject to torsion due to the imposed loads, the wind, or due to curvature of the bridge, the cross bracing resists distortion of the cross section shape and maintains its geometry. For *twin girder* bridges the cross bracing is only lightly stressed when fulfilling this function, because the open cross section resists torsion primarily in antisymmetric bending of the beams (§ 11.5.2). On the other hand, the cross bracing for *straight box girder* bridges is much more highly stressed when the box has to resist primarily in uniform torsion (§ 11.4.1). For *curved bridges* (Sect. 11.7), be they of open or closed cross section, the cross bracing is particularly important when fulfilling this function of maintaining the shape. In span, the sets of cross bracing introduce the torque due to curvature into the beams, and at the supports (those supports able to resist torsion), they transform the torsional moments into vertical reactions.

The reinforced concrete slab fulfils the functions of the plan bracing for composite steel-concrete bridges (once the slab has been structurally connected to the steel beams). When fulfilling this function, the concrete slab, which acts as a horizontal beam, is normally sufficiently stiff against transverse bending and only experiences negligible normal stresses. However, it is often necessary to use temporary steel plan bracing to resist the horizontal loads during erection, in other words, prior to completion of the slab.

14.2.2 Wind

As presented in Paragraph 5.4.1, the horizontal forces due to wind are different for the cross bracing in span and at supports. In span the cross bracing transfers the wind forces into the plan bracing, while at the supports the bracing transfers the resultant of these forces R_H from the plan bracing into the supports (Fig. 14.1).

To determine the moments and forces acting on the cross bracing in span due to lateral wind, it is assumed that the lower flange of the main beams supports the transverse wind pressure acting on the lower half of the web, by acting as a continuous beam supported by the sets of cross bracing. Wind pressure on the upper half of the web passes directly into the slab, or temporary erection bracing, of a composite bridge. Figure 14.1 shows the forces due to wind acting on frame cross bracing, as well as the structural form of the cross bracing in span and at the supports. In span the cross bracing is stressed by the horizontal force $Q_w = 1/2qhe$ (e = the spacing of the cross bracing), and it transfers this force into the slab that supports it transversally (Fig. 14.1(a)).

At the piers and abutments, the cross bracing must transfer the horizontal reaction R_H from the beam, which is horizontally loaded by the total wind force (for a bridge of open cross section), to the supports. The cross bracing is supported and restrained horizontally by the piers or abutments (Fig. 14.1(b)). The horizontal reaction R_H at a pier or abutment is resisted either by both bearings or by a single bearing, if one of them is free to move transversally (Fig. 5.18). Finally, the support provided by the piers depends on their stiffness in transverse bending (Fig. 5.19).

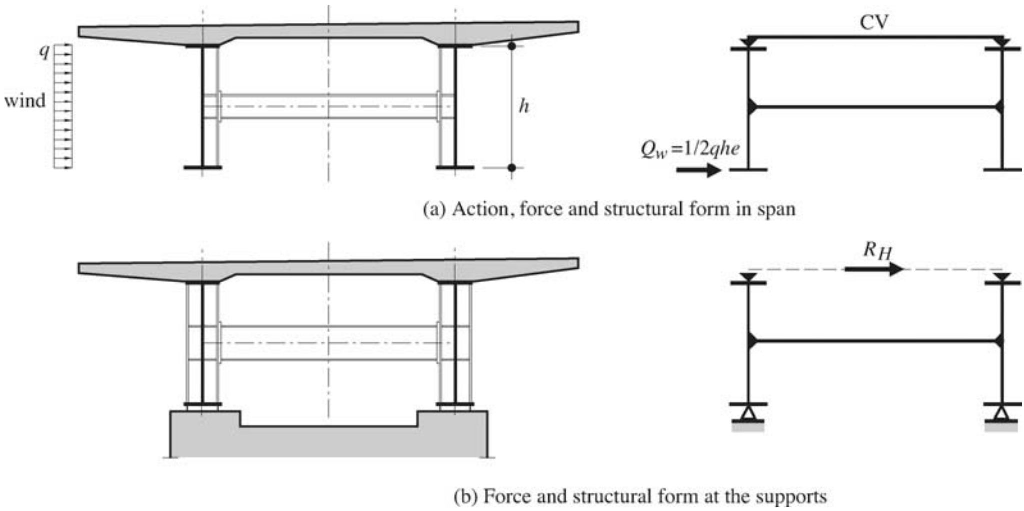


Fig. 14.1 Action due to wind, and structural form for the cross bracing of a twin girder bridge.

14.2.3 Lateral Torsional Buckling Restraint

The resistance to lateral torsional buckling of plate girders and the relevant design checks were discussed in Paragraph 12.2.4. This resistance is a function, amongst other things, of the buckling length. When the compression flange is not restrained by the plan bracing, the buckling length is given by the spacing between adjacent cross bracings. The cross bracing restrains the lower flange adjacent to the intermediate supports and the upper flange in span (before the connection between the steel beams and concrete slab becomes effective).

The horizontal force H_D (Fig. 14.2) needed to restrain the compression flange against lateral torsional buckling and acting on the cross bracing is taken as 1% of the normal force N in the compression member. The area of the compression member A_D is taken as the area of the flange plus part of the web, namely half the effective web area but not more than one third of the area of web in compression (Equations (12.12) and (12.13)). Figure 14.2 shows the forces H_D acting on cross bracing located adjacent to an intermediate pier of a bridge. The lateral torsional buckling may be either symmetric or antisymmetric depending on the flexural stiffness provided by the elements that make up the cross bracing. In this representation of the deformation of the cross bracing, the slab is taken as an element that has infinite lateral stiffness (it is fully rigid). The force H_D may also act on only one side of the cross bracing, when the actions due to traffic are positioned eccentrically (in the transverse sense).

As is the case when considering wind loading, the horizontal forces H_D are transferred by the cross bracing into either the temporary plan bracing during erection of the steelwork or construction of the slab or into the concrete slab once the composite bridge is in its final state. These forces are then transferred by the plan bracing or slab to the cross bracing at the supports. During construction of the slab, whatever method of construction is used, the self-weight of the slab is normally the load case to consider when checking structural safety against lateral torsional buckling of the upper flanges of the beams in span. Once the steel beams and slab have been structurally connected, lateral torsional buckling of this upper flange is no longer possible.

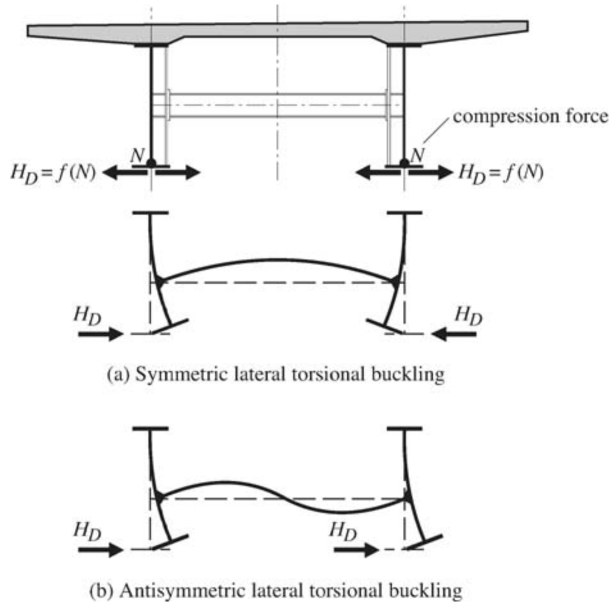


Fig. 14.2 Flanges in compression near to the piers and possible deformations of the cross girders.

14.2.4 Effect of Curvature

The need for the cross bracing to maintain the shape of the transverse cross section against deformation is particularly important for curved bridges. Deformation of the cross section at mid-span of a twin-girder curved bridge with uniform loading is shown in Figure 14.3. In the absence of cross bracing (Fig. 14.3(a)), each beam is subject to a vertical displacement and a rotation, which together lead to a deformation of the cross section (which, therefore, does not maintain its shape). The presence of cross bracing (Fig. 14.3(b)) linking the two beams together ensures, at least locally, that the two beams do deform together, and thereby maintains the shape of the cross section. As was discussed in Paragraph 11.7.1, the cross bracing fulfils an essential function for curved bridges and is necessary to guarantee equilibrium of the beams. That way the cross bracing effectively introduces the torque, due to curvature, into the curved beams.

The moments and forces acting on the cross bracing as a result of the curvature of the beams may be defined by considering the equilibrium of a small segment of curved beam (Fig. 14.4(a)). Over the length

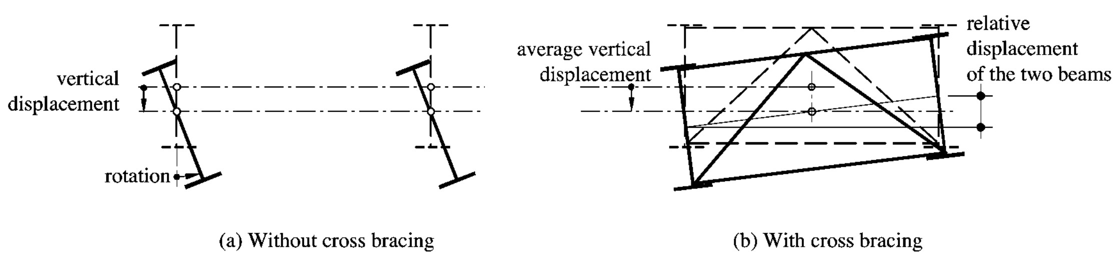


Fig. 14.3 Deformation of curved beams at mid-span.

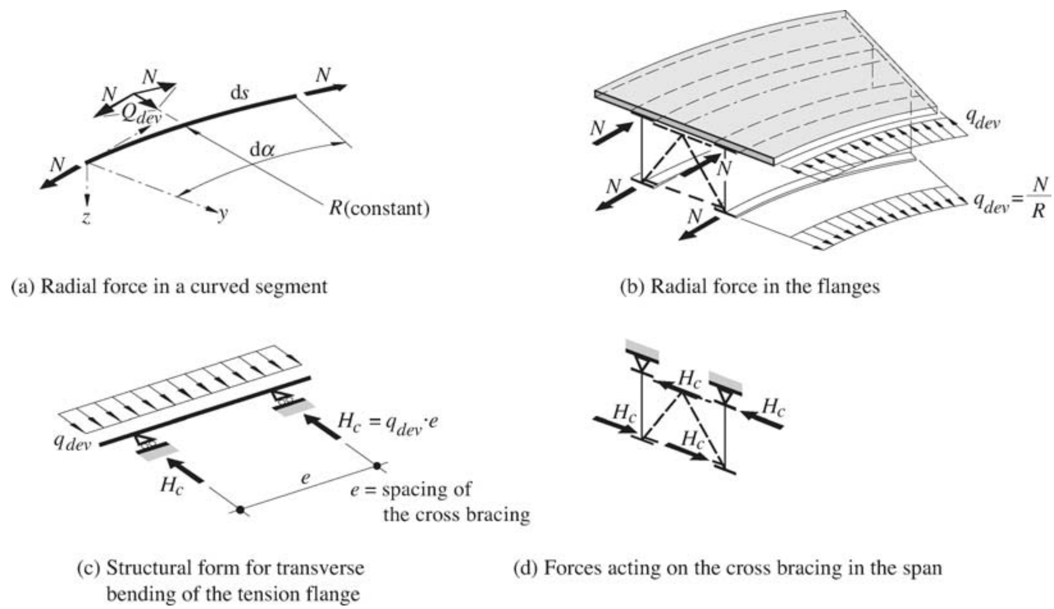


Fig. 14.4 Effect of the radial forces due to curvature on the cross bracing in the span.

d*s* of this segment, which is subject to an axial force N that is assumed to be constant, the curvature results in a radial force q_{dev} (deviation force) that is uniformly distributed over the length d*s*:

$$q_{dev} = \frac{Q_{dev}}{\mathrm{d}s} = \frac{N \cdot \mathrm{d}\alpha}{\mathrm{d}s} = \frac{N}{R} \tag{14.1}$$

For a curved bridge the radial forces act horizontally on each of the beam members (Fig. 14.4(b)). *Member* is here defined as the part of the section in either tension or compression, namely a flange plus part of the web. The radial forces act towards the "inside" for tension members and the "outside" for compression members, and thereby provoke changes to the shape of the cross section. Cross bracing resists changes to the shape. Between cross bracings, as a simplification, the uniformly distributed radial forces are assumed to act in the planes of the flanges. If the spacing e between cross bracings is big, the radial forces can be relatively large compared to those due to vertical bending of the beams. This is why for a curved bridge, the sets of cross bracing are often more closely spaced than those found in a straight bridge.

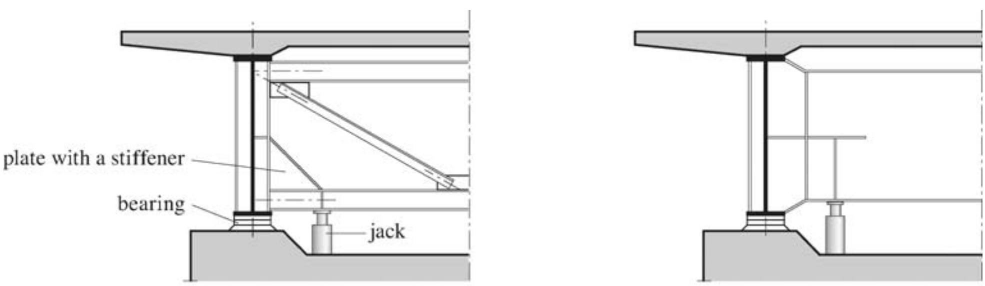
The radial forces affecting the cross bracing may be obtained by subjecting the flanges of an equivalent straight beam to a transverse horizontal load q_{dev}. Figure 14.4(c) shows schematically the radial forces acting on the lower flange of a bridge beam. The transverse bending that results from these forces may be determined by considering the structural form shown in the same figure. The sets of cross bracing that provide the horizontal supports to the beam must resist the continuous beam reactions, which are taken as $H_C = q_{dev} \cdot e$. When the flange is structurally connected to the concrete slab, the corresponding radial forces are resisted directly by the latter.

Figure 14.4(d) shows the structural form of truss cross bracing, and the loading on it that results from curvature. Because the cross bracing provides horizontal support to both tension and compression members, it is loaded by radial forces effectively acting on a closed system, or, in other words, the horizontal radial forces are not transferred from the cross bracing into the plan bracing and from there down to the supports. The cross bracing is stressed as a panel in shear, by which means it can introduce the effects of curvature (torsion) into the bridge cross section.

In the case of a box girder bridge, the cross bracing must also introduce the torque due to curvature into the box. At each cross bracing location, the effect of the curvature is represented as a couple of horizontal forces H_C, which are assumed to act in the planes of the flanges and perpendicularly to the box. The horizontal force H_C is calculated by considering the total force N acting in either the compression or tension member of the box.

14.2.5 Temporary Supports for Bearing Replacement

When it is necessary to lift the bridge at the pier or abutment locations, and the designer wishes the cross bracing to be used as transfer elements between the jacks and main beams, they must clearly be designed for this function. When it is necessary to repair or replace a bearing, the bridge is lifted, and the support reaction from the beam passes through the cross bracing to the jack (Fig. 14.5). Even though the support reaction is normally due only to the self-weight of the structure plus, perhaps, a reduced traffic load (use of the bridge may be partially or, indeed, totally prohibited during such operations), the cross bracing must transfer the support reaction and, therefore, is subject to vertical shear. This load case results in a need for particular construction details and may require the size of the cross bracing elements to be modified. In certain cases it may necessitate a change of the type of cross bracing, for example a truss cross bracing (Fig. 14.5(a)) may need to be replaced by a frame cross bracing or even a diaphragm (Fig. 14.5(b)).



(a) Truss cross bracing (b) Diaphragm cross bracing

Fig. 14.5 Examples of detailing of the cross bracing to allow jacking of the bridge.

14.3 Moments and Forces on the Cross Bracing

Having now identified the different actions on the cross bracing, this section considers the moments and forces within the structural elements that form the cross bracing. Distinctions are made between bridges having an open cross section and those with a closed cross section, as well as between the different types of cross bracing.

In *span* the sets of cross bracing are subject to horizontal forces Q_w due to the wind (Fig. 14.1), and in some cases the forces H_D due to their function as supports against lateral torsional buckling of the compression members (Fig. 14.2). The sets of cross bracing are also subject to the moments and forces that transfer the torque, which results from maintaining the cross section shape (closed sections), and the forces

H_C due to curvature (Fig. 14.4). These various horizontal forces do not all act at the same time with their maximum values. Which forces act simultaneously, and their corresponding magnitudes, depends on the distribution of the imposed loads, the form of the bridge, and the hazard scenarios that are to be considered.

At the *piers* and *abutments*, the cross bracing transfers the horizontal reactions, due to the wind loading and lateral torsional buckling restraint, from the plan bracing into the supports (assuming that the plan bracing is not located in the same plane as the bearings). For box girder bridges the cross bracing also transfers the torsional moments into the supports, or, more specifically, into those supports that are designed to resist these moments.

In the paragraphs below the horizontal force that acts on the cross bracing is defined by the variable H, without identifying the origin of the force (wind, lateral torsional buckling restraint, curvature).

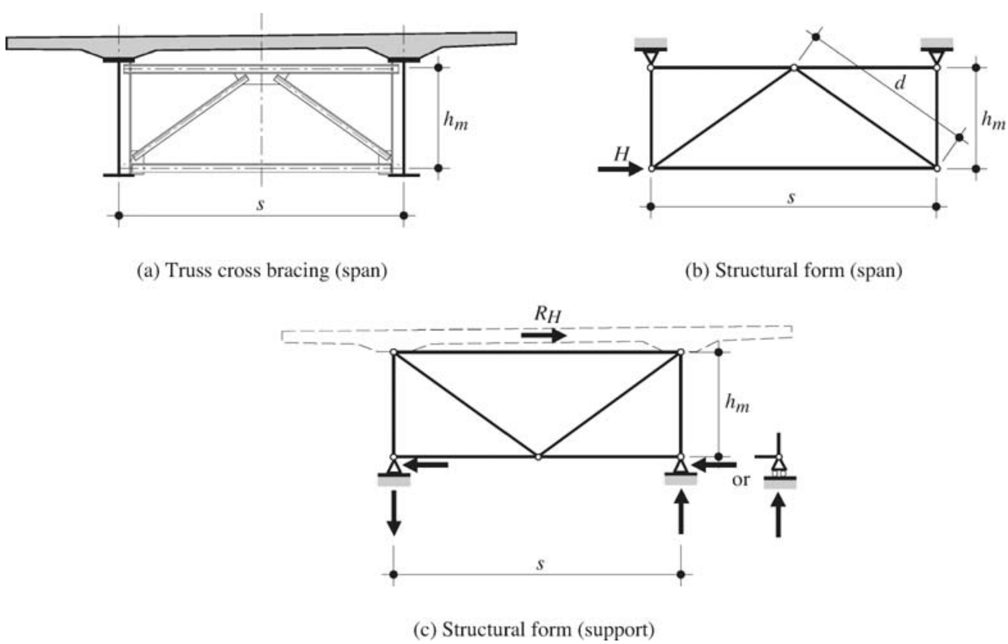
14.3.1 Bridge with an Open Cross Section

The models presented below are intended to help explain the behaviour of the cross bracing, and to enable the designer to rapidly determine the moments and forces acting in the cross bracing. The slab is considered to be infinitely stiff against transverse bending, and thereby to provide support to the cross bracing.

Truss Cross Bracing

Figure 14.6(a) shows a bridge with an open cross section and truss cross bracing, of principal dimensions h_m and s. The corresponding structural form is presented in Figure 14.6(b). The normal force D that acts in the truss diagonals of length d, due to the force H, is obtained by considering equilibrium at the nodes and is given by:

$$D = \frac{H \cdot d}{s} \tag{14.2}$$



(a) Truss cross bracing (span)

(b) Structural form (span)

(c) Structural form (support)

Fig. 14.6 Structural form of truss cross bracing.

Figure 14.6(a) illustrates an example of in-span cross bracing. For cross bracing at a pier or abutment, which must transfer the plan bracing reactions to the supports, the structural form is shown in Figure 14.6(c). The fixed supports represent the beam supports comprising the piers or abutments. As a simplification, the horizontal slab reaction R_H is considered to be applied as two equal forces, one above each main beam upper flange. When identifying the structural form of the cross bracing, it is clearly important to take into account the conceptual design of the supports, one of which may allow transverse movement (Fig. 5.18).

Frame Cross Bracing

Figure 14.7(a) shows an open cross section with frame cross bracing, including the principal dimensions. The simplified structural form of this type of cross bracing is shown in Figure 14.7(b).

For cross bracing at a pier or abutment, which transfers the plan bracing reactions into the supports, the simplified structural form is shown in Figure 14.7(c). The fixed supports shown in this simplified form represent the support of the beams, namely the piers or abutments. As a simplification the horizontal slab reaction R_H is considered to be applied as two equal forces, one above each main beam upper flange. The relationships given in Table 14.8 do not apply if one of the supports allows transverse movement. In such situations the structural form is determinate, so the moments and forces are easily calculated.

The spring constants K, relative to bending and normal force [14.1], as well as the constant K_V to take into account shear when calculating deformations, are needed in order to determine the moments, forces,

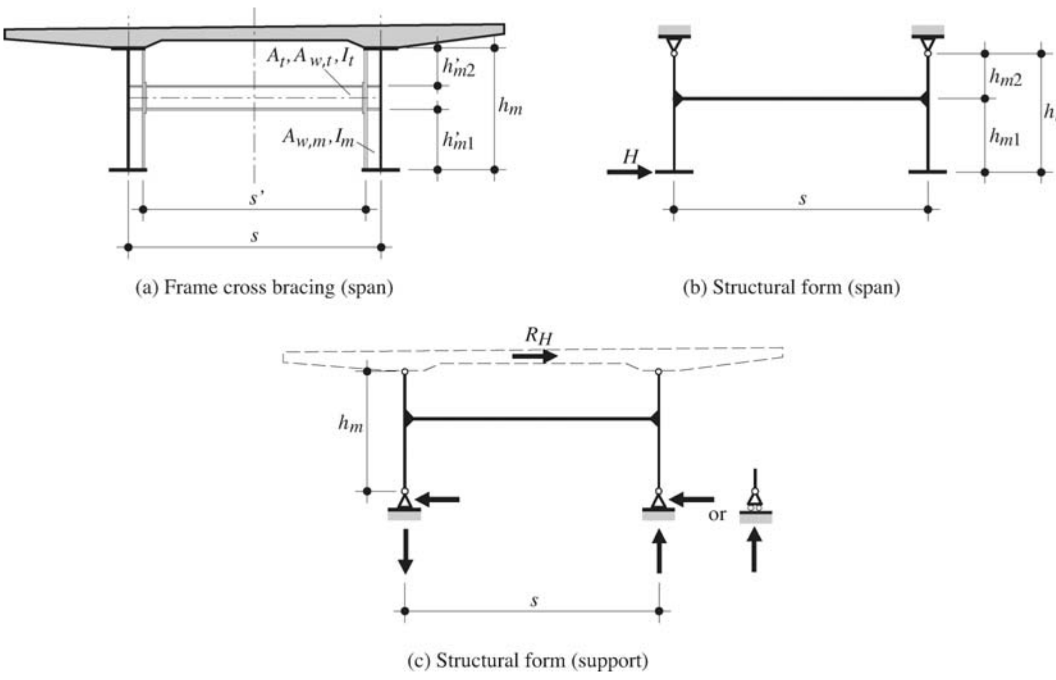


Fig. 14.7 Structural form of frame cross bracing.

and deformations of the frame. These constants are defined by the following equations, using notation given in figure 14.7(a):

- *Lower part of the upright* $K_{m1} = \dfrac{h'^3_{m1}}{3EI_m}, \quad K_{V,m1} = \dfrac{h'_{m1}}{GA_{w,m}}$ (14.3)

- *Upper part of the upright* $K_{m2} = \dfrac{h'^3_{m2}}{3EI_m}, \quad K_{V,m2} = \dfrac{h'_{m2}}{GA_{w,m}}$ (14.4)

$$K_{t1} = \frac{s' \cdot h^2_{m1}}{2EI_t}, \quad K_{V,t} = \frac{2s'}{GA_{w,t}}$$

- *Cross girder* $K_t = \dfrac{s' \cdot h^2_m}{2EI_t}$ (14.5)

$$K_{t2} = \frac{s' \cdot h^2_{m2}}{2EI_t}$$

$$K_{tN} = \frac{s'}{2EA_t}$$

In order to calculate the second moment of area I_m of the frame uprights, one may assume that a width of web corresponding to 25 times the web thickness participates in the effective cross section. The areas of the upright webs and cross girder are defined as $A_{w,m}$ and $A_{w,t}$, respectively.

Table 14.8 shows the forces that act on either symmetrically or antisymmetrically loaded cross bracing (Fig. 14.2), and defines their magnitudes. Values of the displacement v of the cross bracing, when subject to horizontal forces H, are also given in this table.

As the relationships given in Table 14.8 illustrate, the moments and forces in the cross girder are relatively unaffected by its location within the depth of the cross section. The choice of location is more driven by the conceptual design and intended use (for example, to support ductwork or formwork), than by the moments and forces to be carried.

The displacement v of the cross bracing, calculated using the equations given in Table 14.8 and measured at the point where the horizontal forces are applied, includes deformations of the frame in shear. This shear deformation may be of the order of 20% of the displacement due to bending. The displacements v are calculated based on the conservative assumption that there is no moment transfer from the uprights into the slab, because the joints at these locations are assumed to be pinned.

Diaphragm

The only force acting on cross bracing in the form of a diaphragm, comprising a stiffened plate that reaches down to the beam supports, is pure shear resulting from the horizontal loading. The use of this type of cross bracing is rare for bridges of open cross section, although for some long span bridges, the use of a diaphragm over the abutments may be justified.

14.3.2 Bridge with a Closed Cross Section

As noted in Paragraph 14.2.1, box girder bridges resist wind by acting as a horizontal beam. Unless the wind loading results in torsion of the box, the cross bracing in span is unstressed. If torsion is present the

Table 14.8 Internal moments and forces in frame cross bracing subject to forces H on the lower flanges.

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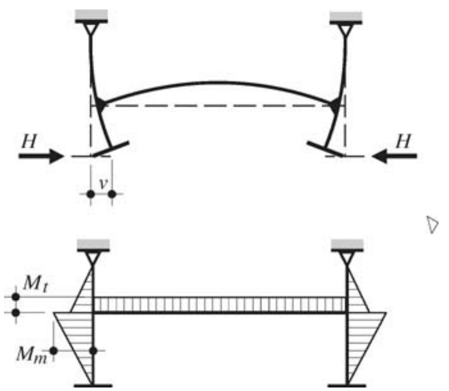
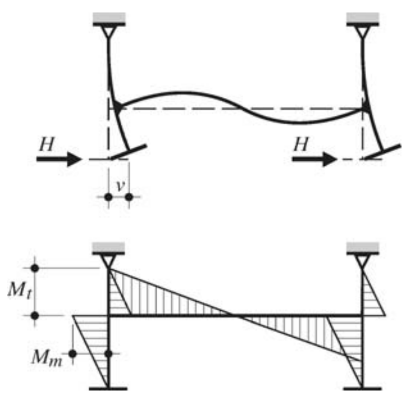
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">Internal moments and forces in the cross girder</td></tr>
<tr><td>$$M_t = Hh_m \cdot \frac{K_{tN} + K_{m2} \cdot \frac{h_{m1}}{h_m}}{K_{t2} + K_{tN} + K_{m2}}$$ $$N_t = H \cdot \frac{K_{m2} + K_{t2} \cdot \frac{h_m}{h_{m2}}}{K_{t2} + K_{tN} + K_{m2}}$$ $$V_t = 0$$</td><td>$$M_t = H \cdot h_m$$ $$N_t = 0$$ $$V_t = 2H \cdot \frac{h_m}{s}$$</td></tr>
<tr><td colspan="2">Internal moments and forces in the lower part of the upright</td></tr>
<tr><td colspan="2">$M_m = Hh_{m1}$ $N_m = 0$ $V_m = H$</td></tr>
<tr><td colspan="2">Displacement of the cross bracing</td></tr>
<tr><td>$$v = H \cdot \left[K_{m1} + K_{V,\,m1} + K_{t1} + K_{tN} - \frac{\left(K_{tN} - K_{t2} \cdot \frac{h_{m1}}{h_{m2}}\right)^2}{K_{t2} + K_{tN} + K_{m2} + K_{V,\,m2}} \right]$$</td><td>$$v = H \cdot \left[K_{m1} + K_{V,\,m1} + K_{m2} + K_{V,\,m2} + \frac{K_t}{3} + K_{V,\,t} \cdot \left(\frac{h_m}{s}\right)^2 \right]$$</td></tr>
</table>

cross bracing must introduce this torque into the beam, as considered below. On the other hand, the cross bracing at the supports must transfer to the supports that part of the wind loading that acts transversally on the upper member of the box. This is similar to the function fulfilled by the slab of an open section composite bridge.

The function they fulfil in *maintaining the shape* of a bridge cross section results in the greatest stresses in the cross bracing of a box girder. When cross bracing is present, the box resists the torque applied to the bridge in uniform torsion, by means of a shear flow. Torque may be due to:

- imposed loads positioned eccentrically on the deck,
- horizontal forces with a resultant that does not act through the shear centre of the cross section,
- curvature of the bridge.

The combination of actions that results in torque is, therefore, a function of the form of the bridge and the load case being considered. As will be highlighted below, this combination must take into account the signs of the moments and forces that act on the cross bracing, as they may oppose one another, even when the torque act in the same sense.

The cross bracing *in span* introduces torques into the box. These loads can only be introduced into the box at cross bracing locations in the span because the box itself is too flexible to permit a direct transformation of the torques into a shear flow in the box walls. The cross bracing forms a stiff plane, thereby providing a kind of torsional support for the flexible box. The torques are introduced, therefore, as concentrated

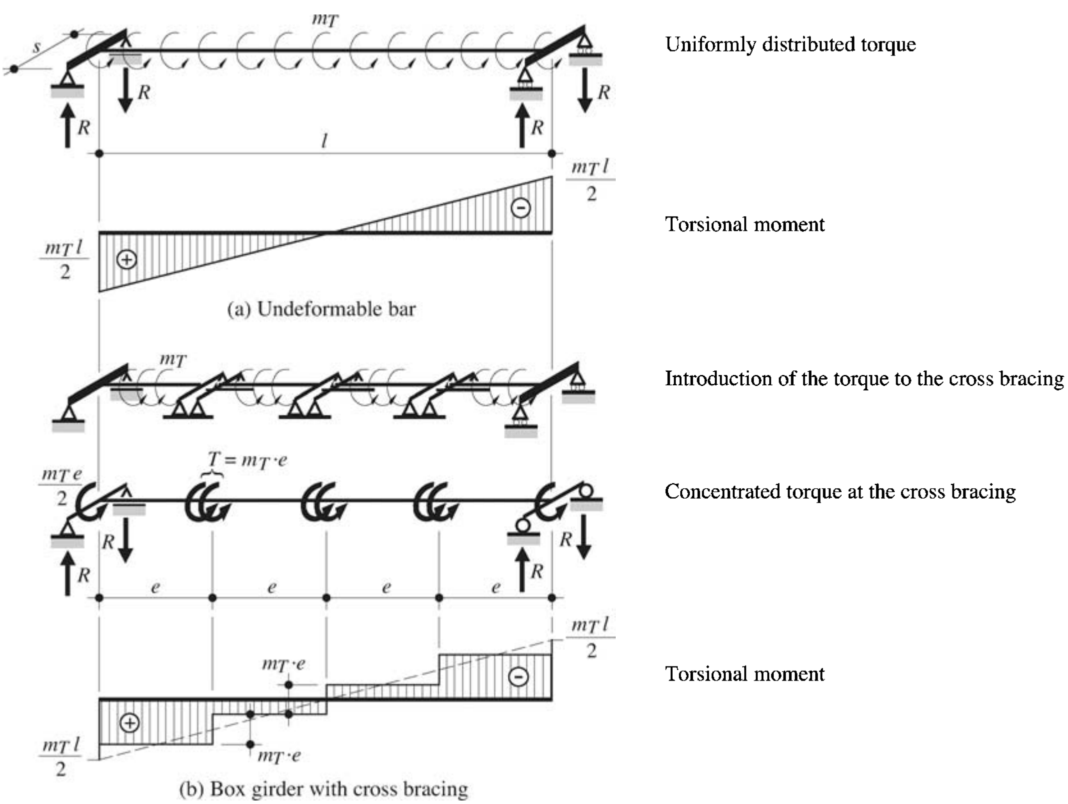


Fig. 14.9 Behaviour of a box girder bridge subject to a uniformly distributed torque.

torsional moments, at the location of each cross bracing in span. This means that to determine the internal moments and forces in the box due to torsion, the simplified structural form that represents the beam as an undeformable bar (Fig. 14.9(a)) is replaced by a structural form representing the box with cross bracing, as shown in Figure 14.9(b). The real behaviour of the bridge is somewhere between these two simple representations. However, if a sufficient number of stiff cross bracings is present (§ 14.5.2), the behaviour approaches that where the torque is transformed into shear flow, and the distortion of the box is small.

The closed section transfers torsion to the supports by resisting in uniform torsion. The cross bracing at the *piers and abutments* transforms the shear flow into a couple of forces R to the two box supports.

Loading of the Cross Bracing in Span

Each cross bracing in the span is influenced by torques to the left and right of the beam length that it bounds. Effectively, the cross bracings represent rigid supports against torsion for each length of box that is bounded by two cross bracing positions (Fig. 14.9(b)). As noted above, this means that each cross bracing must resist and transform into a shear flow within the box the torques acting to the left and right of each length of box.

Because the sets of cross bracing are normally uniformly spaced within a given span, their design may, conservatively, be based on the moments shown in Figure 14.9(b). When there are n cross bracings in a span there are $(n + 1)$ lengths of beam, and the torsional moment at the end of each length (for a uniformly distributed torque m_T) is given by $m_T l/(2 \cdot (n + 1))$. The concentrated torque T acting on the cross bracings, which are a distance e apart, is equal to twice this value:

$$T = \frac{m_T\, l}{n+1} = m_T\, e \tag{14.6}$$

Loading of the Cross Bracing at the Abutments and Piers

At piers and abutments that have been conceived to act as torsional supports for the box (meaning there are two bearings per support), the internal torsional moment is transformed into a couple of forces which act

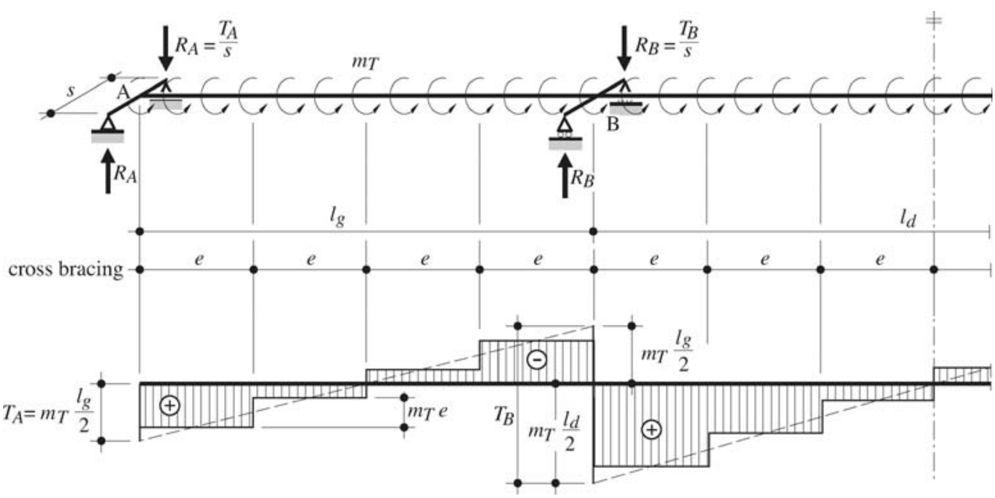


Fig. 14.10 Torional moment diagram, also showing the support reactions.

on the support. In other words, the shear flow in the box at this point is transformed into these forces by the cross bracing. This means that the cross bracing at pier and abutment locations is subject to a concentrated torque that is significantly greater than that applied to the cross bracing in span. In the case of a *simple beam*, as shown in Figure 14.9, the torque acting on the cross bracing and transferred to the supports as a couple of forces is equal to $m_T l/2$.

For a *continuous box* girder, the cross bracing at an intermediate support must transfer the sum of the torsional moments to the left and right of the support (assuming, of course, that the support has been conceived to resist torsion). The cross bracing at the intermediate pier B, therefore, must resist the following concentrated torque (Fig. 14.10):

$$T_B = m_T \frac{l_g}{2} + m_T \frac{l_d}{2} \tag{14.7}$$

For this case of a continuous beam, the cross bracing at the left hand abutment, as in the example of a simple beam, is subject to a concentrated torque $T_A = m_T l_g/2$.

Variation in Shear Flow Due to Cross Bracing

As explained above, the torque acting on the closed section is concentrated at the cross bracings as a series of torques T, then transformed into a shear flow in the box walls (Fig. 14.9(b)). At each cross bracing location, therefore, there is a variation in the shear flow Δv that is proportional to the concentrated torque T, and which may be expressed as (according to Equation (11.5)):

$$\Delta v = \frac{T}{2\Omega} \tag{14.8}$$

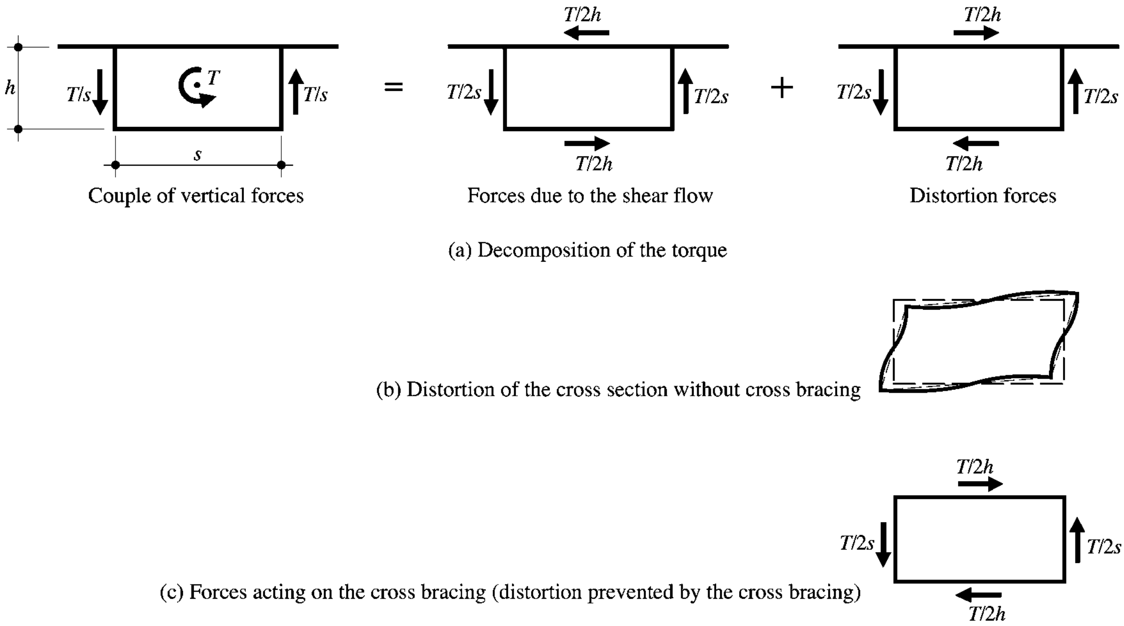
T : concentrated torque acting on the cross bracing (in span or at a support)
Ω : area defined by the median line of the closed section

The model shown in Figure 14.11 may be used to determine the forces in the cross bracing itself. The concentrated torque T which results, for example, from the *eccentric positioning of the traffic loads* acting on the bridge, may be resolved into a couple of vertical forces T/s which act at the positions of the box webs, and correspond to the antisymmetric loading of the cross section. In the box walls, the variation of shear flow is given by $\Delta v = T/(2sh)$ and introduces the following forces:

- *Webs* $$\Delta v \cdot h = \frac{T}{2sh} \cdot h = \frac{T}{2s} \tag{14.9}$$

- *Flanges* $$\Delta v \cdot s = \frac{T}{2sh} \cdot s = \frac{T}{2h} \tag{14.10}$$

These forces, which result from the closed shear flow in the cross section, correspond to the resistance in uniform torsion of the section. Even though this system of forces is in global equilibrium with the torque T, it is not in equilibrium with the antisymmetric loading of the section and the forces in each horizontal plane (Fig. 14.11(a)). It is, therefore, necessary to add a second system of forces in order to assure equilibrium of the section. This set of forces, which are known as distortion forces, results in a distortion of the



(a) Decomposition of the torque

(b) Distortion of the cross section without cross bracing

(c) Forces acting on the cross bracing (distortion prevented by the cross bracing)

Fig. 14.11 Determining the forces in the cross bracing.

cross section (Fig. 14.11(b)) that is associated with normal stresses σ_w and shear stresses τ_w due to warping of the cross section. If distortion of the cross section is prevented by cross bracing that is sufficiently stiff, then this second set of forces stresses the cross bracing (Fig. 14.11(c)).

Because the cross bracings maintain the shape of the cross section only at discrete locations, there will always be, along the length of a beam, a combination of normal stresses σ_x due to bending and normal stresses σ_w due to warping of the cross section. The ratio between these stresses, σ_w/σ_x, which is greatest at the corners of the box, depends on the number and stiffness of the cross bracings. It is normally accepted that the stresses σ_w may be ignored in the structural design of a box girder if this ratio is less than 0.05. This is normally the case if there are at least five cross bracings, sufficiently stiff, in each span [14.2].

Truss Cross Bracing

The truss cross bracing of the box girder shown schematically in Figure 14.12 should be designed to resist the forces shown in Figure 14.11(c). These forces stress the uprights and chords of the truss formed from the transverse stiffeners that are welded to the webs and flanges (Fig. 6.18). The force D in the diagonals is determined by considering equilibrium of one of the truss nodes. For example, consider equilibrium of node A that links the diagonals to the lower flange of the box (Fig. 14.12):

$$2D\cos\alpha = \frac{T}{2h} \rightarrow 2D\left(\frac{s/2}{d}\right) = \frac{T}{2h} \tag{14.11}$$

The force D in each diagonal is given by:

$$D = \frac{T}{2hs} \cdot d = \Delta v \cdot d \tag{14.12}$$

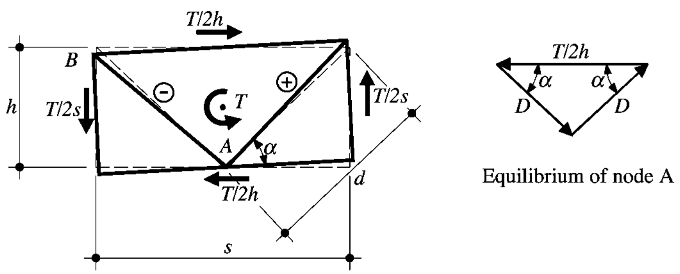


Fig. 14.12 Forces in the diagonals of the truss cross bracing of a closed section bridge.

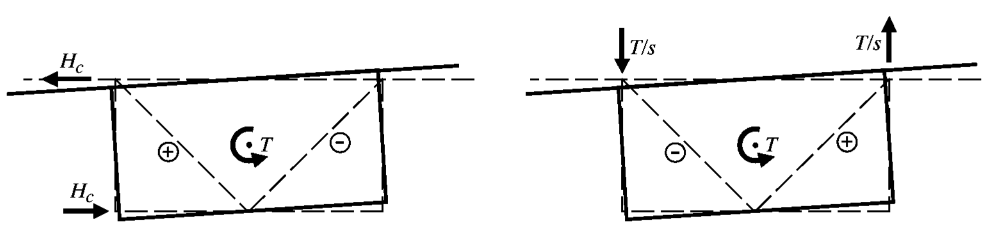
It is enlightening to consider the analogy with the forces acting in the walls of a box girder due to the variation in shear flow, as defined in (14.9) and (14.10). The force in the truss diagonals is directly proportional to the variation in shear flow Δv, introduced by the cross bracing, multiplied by the diagonal length d. By considering node B it can also be demonstrated that the vertical component of the force D is equivalent to $T/2s$. For the example shown in Figure 14.12, if the box is deformed in the positive sense of the torque T, the diagonal to the left is subject to compression, while the diagonal to the right is in tension.

When the torque on a beam is due to its *curvature*, the cross bracing that introduces this torsion into the box is subject to a concentrated torque resulting from the couple of horizontal forces H_C (§ 14.2.4) acting on the box flanges (Fig. 14.13(a)). The corresponding forces in the truss cross bracing diagonals are determined in the same way as when considering the introduction of a couple of vertical forces (Fig. 14.13(b)). Careful attention should be paid to the sign of the normal force acting in the diagonals. As shown in Figure 14.13, for a concentrated torque T in a given sense, depending on whether the torque corresponds to a couple of horizontal or vertical forces, the forces in the diagonals have a different sign!

For a curved bridge the normal force D in the diagonals of truss cross bracing, expressed as a function of the radial force H_C, with $\Delta v = H_C/2s$, is given by:

$$D = \frac{H_C \cdot d}{2s} \tag{14.13}$$

Comparing Equation (14.13) with Equation (14.2), which gives the normal force in the diagonals of truss cross bracing for an open section bridge, and with all else being equal, if a closed section is used, the force in the diagonals in span due to curvature is halved. This means that as far as the loading on truss cross bracing is concerned, it becomes more and more beneficial to consider a closed cross section solution as the bridge curvature becomes more pronounced.



(a) Couple of horizontal forces (curvature) (b) Couple of vertical forces (traffic)

Fig. 14.13 Cross section deformation and sign convention for the forces in the diagonals of the truss cross bracing.

Diaphragm

Consider a diaphragm comprising a solid plate of thickness t, loaded in shear by the system of forces shown in Figure 14.11(c). The shear stresses τ in the diaphragm are determined by dividing the forces by the (cross section) area of the diaphragm. For example, with an area $h \cdot t$ for the vertical contact surface between the diaphragm and web of a box:

$$\tau = \frac{T}{2s} \cdot \frac{1}{h \cdot t} = \frac{\Delta v}{t} \tag{14.14}$$

It can be confirmed that the same result is obtained if the horizontal face of the diaphragm is considered. The shear stress corresponds to the variation in shear flow introduced by the cross bracing divided by the thickness of the plate forming the diaphragm.

14.4 Forces on the Plan Bracing

14.4.1 Horizontal Forces

The forces acting in the plan bracing of a bridge are determined in the same way as for a single storey industrial building (TGC Vol. 11, Sect. 14.3). Equilibrium of the horizontal forces, mainly wind, acting on the structure must be assured. That means they must have a load path down to the supports, so that the stability of the structure is guaranteed. The horizontal forces due to wind are transferred into the plan bracing in part directly by the main beams, and partly via the cross bracing (Fig. 5.17). It was noted earlier that truss plan bracing comprises the main beams, which form the chords, and cross girders, or indeed the chords of the cross bracing, which form the uprights (Fig. 5.26). The magnitudes of the forces in the plan bracing are a function of the horizontal structural form of the bridge (§ 5.4.2). These forces are inversely proportional to the effective length of the cross girders, which is equal to the spacing between the main beams.

14.4.2 Influence of the Form of Bracing

As discussed in Paragraph 5.7.2, the plan bracing is affected by the longitudinal deformations of the main beams to which it is attached, as well as to any horizontal forces. The magnitudes of the forces that are applied to the members making up the truss plan bracing depend on its geometric form and its position within the depth of the cross section.

Figure 14.14 provides a schematic representation of the deformation that is imposed in span for three different types of plan bracing, which are in all cases fixed to the main beams in the lower part of the cross section. For plan bracing in the form of a *Saint Andrews Cross* (X), the bracing is not free to elongate in order to follow the deformations of its chords under vertical load. Therefore, it is subject to "parasitic" forces resulting from this restraint. For plan bracing in the form of a *diamond*, the diagonals are subject to smaller deformations because the main beams are relatively flexible transversally and so offer less restraint than is the case with X bracing. For plan bracing in the form of a K, the joints between the diagonals and the "uprights" of the plan bracing are flexible and, therefore, lead to only very small forces in the plan bracing diagonals.

In the case of *temporary plan bracing* used during erection of the steelwork, the parasitic forces that develop, mainly in the diagonals, may be ignored, given that the longitudinal deformation of the main beams is normally less during erection than in the final state. If, on the other hand, the plan bracing is to

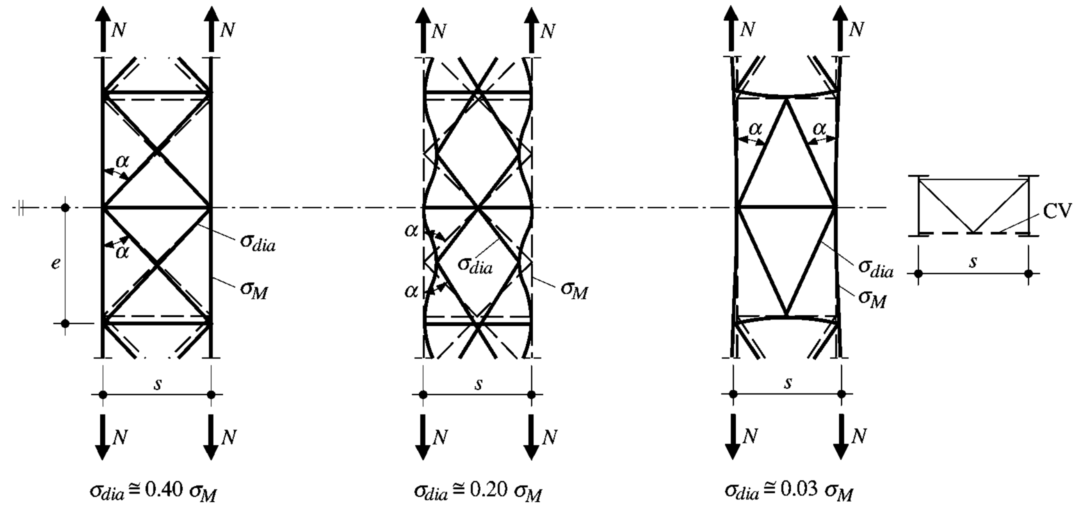


Fig. 14.14 Deformations imposed on the plan bracing (CV) by the main beams as a function of the geometrical form of bracing.

remain active throughout the life of the bridge, then either it should be joined near to the level of the neutral axes of the beams, or its form should be such that the parasitic forces will be small, or these forces should be taken into account in the structural design of the bracing.

The equations given below allow the stresses σ_{dia} in the diagonals to be estimated as a function of the stresses σ_M present in the chords to which the bracing is attached. These equations are given for a geometry that is symmetrical about the bridge axis, for uniform spacing of the cross bracings, and for a constant stress in the chords [14.3].

X plan bracing:
$$\sigma_{dia} = \sigma_M \cdot \frac{\cos^2\alpha}{1 + \dfrac{A_{dia}}{A_M}\cos^3\alpha + 2\dfrac{A_{dia}}{A_T}\sin^3\alpha} \tag{14.15}$$

Diamond plan bracing
$$\sigma_{dia} = \sigma_M \cdot \frac{\cos^2\alpha}{1 + \dfrac{A_{dia}}{A_M}\cos^3\alpha + 2\dfrac{A_{dia}}{A_T}\sin^3\alpha + \dfrac{A_{dia}\cdot e^2}{48 I_M}\sin^2\alpha\cdot\cos\alpha} \tag{14.16}$$

K plan bracing
$$\sigma_{dia} = \sigma_M \cdot \frac{\cos^2\alpha}{1 + \dfrac{A_{dia}}{A_M}\cos^3\alpha + \dfrac{A_{dia}}{A_T}\sin^3\alpha + \dfrac{A_{dia}\cdot e^2}{3 I_T}\sin^3\alpha} \tag{14.17}$$

A_{dia} : area of a bracing diagonal
A_M : area of a bracing chord (equal to the area of the beam flange)
A_T : area of a bracing upright (equal to the area of a cross bracing girder)

I_M : second moment of area of the chord in the plane of the plan bracing
I_T : second moment of area of an upright in the plane of the plan bracing
e : spacing of the plan bracing uprights (cross bracing)

As an example, for two main beams 6.0 m apart, with a 500 × 50 mm upper flange, an angle $\alpha = 45°$, 101.6 × 10 mm hollow section diagonals, and an HEA 300 cross girder, the diagonals of X plan bracing are subject to a stress σ_{dia} equal to about 40% of the stress σ_M present in the beam flange. For diamond plan bracing σ_{dia} is about 20% of σ_M and for K plan bracing it is around 3% of σ_M. It can be concluded that bracing in the shape of an X is not a very good solution for permanent plan bracing.

14.4.3 Bracing of an Open Box Section

Plan bracing may be used to close a box that would otherwise be "open", either in the temporary situation when the steelwork is being erected by launching, or as permanent bracing. A composite twin girder cross section may be closed in this way, so that it behaves like a box girder, by placing plan bracing at the level of the beam lower flanges. In addition to the wind, this bracing is stressed by the shear flow that corresponds to the resistance in uniform torsion of the closed section. Figure 14.15 shows the forces in the diagonals and uprights of a truss that is used to close an open box [14.4]. The shear flow v corresponds to the shear flow resulting from the internal torsional moment T in the cross section under consideration:

$$v = \frac{T}{2\Omega} \tag{14.18}$$

v : shear flow
T : internal torsional moment in the section under consideration
Ω : area defined by the median line of the closed section

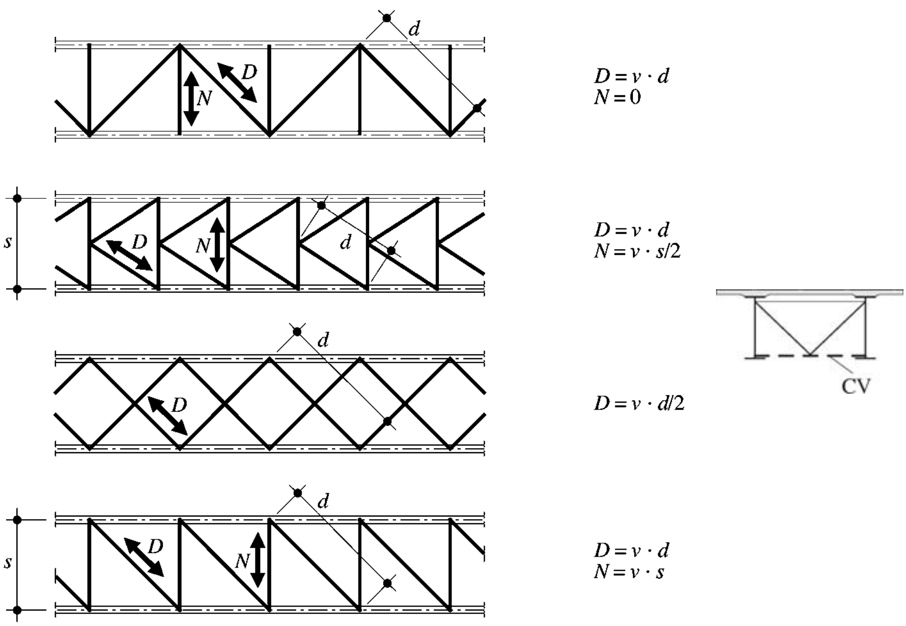


Fig. 14.15 Forces due to shear flow v in lower plan bracing, denoted CV, used to close an open section.

14.5 Structural Design

The cross bracing and plan bracing must satisfy numerous conditions related to *structural safety*, taking into account the assessment of the various hazard scenarios and load cases to which they are subjected. Except for some particular cases, perhaps during certain phases of the erection, there are no serviceability requirements as such to be satisfied for these structural elements.

However, in order to limit the vertical deformations in transverse bending of plan bracing under self-weight, and the potential for resonance, it is important that the members forming the cross and plan bracing are *not too slender*. Also, even if the members forming the cross bracing are sufficient to meet structural safety needs, the bracing must also be *sufficiently stiff* in plane. This stiffness is needed to ensure that the real behaviour of the bridge structure corresponds to that modelled for structural analysis.

14.5.1 Structural Safety (ULS)

The tension and compression members that make up the cross and plan bracing in the form of trusses must satisfy the requirements relative to structural safety (TGC Vol. 11, § 12.3.3). The cross girders of frame cross bracing must satisfy conditions related to resistance and stability (lateral torsional buckling) of elements in compression and bending (TGC Vol. 11, § 12.2.2). Planar cross bracing (a diaphragm) is primarily loaded in shear and, therefore must satisfy the conditions related to panels in shear (Sect. 12.3).

Some of the members making up the cross bracing and plan bracing may fulfil several functions simultaneously in order to assure the structural safety of the whole bridge structure. For example, the *chords of the plan bracing* are also part of the main beams, notably the upper or lower flange, depending on whether

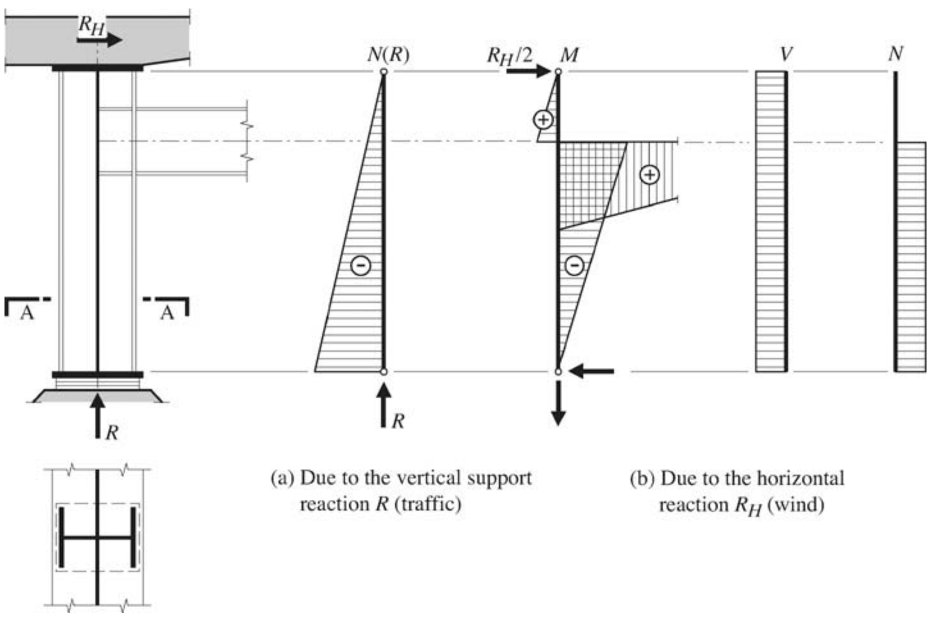


Fig. 14.16 Forces in the upright of a frame cross bracing at a support (bearing stiffener).

the plan bracing is placed high or low within the depth of the cross section. For *cross bracing* the uprights of a truss or frame also fulfil the function of vertical stiffeners to the webs of the main beams and, for cross bracing at a support, the uprights transfer the support reactions into the beams. However, the maximum forces associated with each of these functions need not be simply added together for structural verifications, unless they relate to the same load case. Also, the maximum forces are not necessarily applied to the same cross section of the member.

For example, Figure 14.16 shows the forces acting on an upright of a frame cross bracing at a support. When it is transferring the support reaction R (Fig. 14.16(a)), the upright is subject to an axial force that is greatest at its base. This force reduces as it transfers to the web, and is zero at the top of the upright. To fulfil this function the upright is designed for the greatest support reaction R, as a column in compression would be designed, according to Section 12.6. When it is fulfilling its function as part of the frame cross bracing, the upright is also subject to a bending moment, an axial force, and a shear force (Fig. 14.16(b)) due to the horizontal reaction R_H of the plan bracing (Fig. 14.7), which comprises the slab for a composite bridge.

For the design of the frame, it is necessary to take into account the maximum value of the reaction R_H. This value comes from the load case resulting in the greatest horizontal force (wind). This is not the same load case as that which results in the greatest vertical reaction R (traffic). Therefore, it is not necessary to consider these two forces acting together, with their maximum values, in the design of the upright. Also, the cross section of the upright that is most highly loaded is different: for the load case associated with wind, the most highly stressed cross section is at the corner of the frame, whereas for traffic it is at the base of the upright. If the cross girder is located close to the main beam upper flange, it is acceptable to completely separate the effects of the two forces.

It is also acceptable, in certain situations, to separate load cases based on plastic design principles and to assume that certain parts of the cross section fulfil certain functions, and other parts fulfil other functions. For example, for an upright of a frame cross bracing comprising a Tee section, to which the cross girder is attached:

- the web of the Tee may be given the function of stiffening the web panel and checked accordingly (§ 12.6.1, Equation (12.70)),
- the flange of the Tee, combined with part of the beam web, may be assumed to resist the bending moment (note that for uprights the normal and corresponding shear forces are normally small and may be ignored).

14.5.2 Minimum Dimensions

Truss Bracing

A truss, be it used for either cross bracing or plan bracing, is conceived to be symmetric about the bridge axis because these structural elements primarily resist wind loading, which clearly may act in either transverse direction. This means that whichever way the wind is blowing, some of the diagonals are in compression. The self-weight of the members forming the truss (if it is horizontal) results in transverse bending that cannot always be ignored (second order effects for elements in compression). It is also necessary to avoid using diagonals that are too slender because traffic or wind loading may introduce resonant vibration, which could lead to fatigue cracking. The standard SIA 263 recommends that the slenderness of members satisfies the following limiting values:

- primary structural member in compression $\lambda_K \leq 200$
- structural member in compression and subject to fatigue $\lambda_K \leq 160$

λ_K	:	slenderness of the element ($\lambda_K = l_K/i$)
l_K	:	buckling length of the member (TGC Vol. 10, Table 5.32)
i	:	radius of gyration of the element

For cross bracing or plan bracing that must remain functional throughout the life of a bridge, it is advisable to not exceed a slenderness of 160. For temporary plan bracing a slenderness of 200 (or even more) may be acceptable if the period during which it must fulfil its functions is short.

Frame Cross Bracing for Open Cross Sections

It is possible that the moments and forces acting on frame cross bracing in span are small, and that the design of the elements to satisfy these forces, particularly the cross girder, results in cross sections of small depth. To create sufficient bending stiffness for the cross bracing, one must ensure that the displacement at the base of the frame subject to characteristic wind loading is less than 1/500 times the depth of the steel beams. This deformation criterion should mean that the cross girders in span are at least IPE 300 sections. If other sections are used, the designer should ensure that the cross girders in span have at least an equivalent second moment of area. At the supports, the wind reaction coming from the plan bracing is relatively large, and normally results in cross girders that are stiffer.

The in-plane stiffness of frame cross bracing also affects the effective length for lateral torsional buckling of the compression flanges of the main beams (§ 12.2.4). As far as this stability phenomenon is concerned, the cross bracing fulfils the function of lateral support to the flange, and its flexibility must be taken into account. The stiffer the cross bracing, the closer the buckling length is to the actual spacing of the cross bracings, and the greater the lateral torsional buckling resistance of the beam.

Cross Bracing for Closed Cross Sections

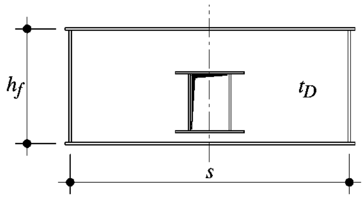
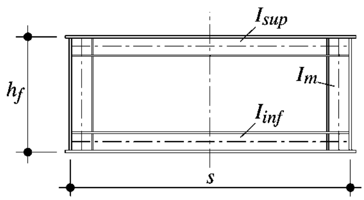
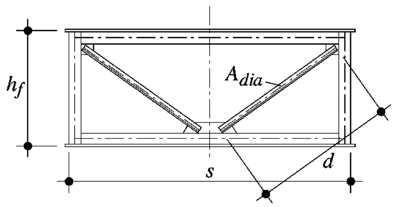
The cross bracing maintains the shape of a closed section and introduces torque into the cross section (box). The cross bracing must act as an indeformable plane in order to fulfil this function. Although a diaphragm formed from a stiffened plate may be reasonably assumed to be a plane that is sufficiently stiff and indeformable, other types of cross bracing used in box girders (truss or closed frame bracings, Fig. 5.21) may not be. When there is a lack of stiffness, the box may distort, and the normal stresses associated with warping of the cross section may no longer be negligible (Fig. 11.16).

Numerical studies undertaken to investigate this issue [14.2] have allowed a value for *minimum stiffness against distortion of the cross bracing* to be defined. This minimum stiffness guarantees that the distortion of the closed section remains small and that, locally, the normal stresses in the section associated with this distortion are less than 5% of the normal stresses, due to overall bending of the box girder. This study also revealed that for continuous beams, five cross bracings in span, each of sufficient stiffness, can guarantee that this 5% limit is not exceeded. With a slenderness for the beam of h/l = 1/20 to 1/25, the spacing of the cross bracings will be $3.3h$ to $4h$.

Numerical studies have shown that the stiffness of frame or truss cross bracing must be at least equal to 20% of the distortional stiffness offered by a diaphragm in order to be considered sufficiently stiff. The distortional stiffness, given by the spring constant K_D, of the cross bracing may be determined using the information given in Table 14.17, derived from [14.5]. In order to calculate the diaphragm stiffness, which serves as a reference value against which the stiffness of the other options are compared, one can assume it is formed from a plate with a thickness t_D of 20 mm.

In this table, the second moment of area I_m, and I_{sup} or I_{inf}, as appropriate for frame cross bracing, should be determined relative to the plane of the web or flanges, and it may allow for some contribution from the web and flanges of the box. E and G are the elastic modulus and the shear modulus, respectively.

Table 14.17 Distortional stiffness of cross bracing in a box section.

	Diaphragm cross bracing $K_D = G t_D s h_f$
	Frame cross bracing $K_D = \dfrac{24EI_m}{\alpha_0 h_f}$ $\alpha_0 = 1 + \dfrac{(2s)/h_f + 3(I_{inf} + I_{sup})/I_m}{(I_{inf} + I_{sup})/I_m + (6h_f/s) \cdot (I_{inf} I_{sup}/I_m^2)}$
	V Truss cross bracing $K_D = \dfrac{EA_{dia} s^2 h_f^2}{2d^3}$

Allowing for these requirements for minimum stiffness of the cross bracing may result in frame bracing with large members in order to satisfy the 20% rule, meaning that it is often more economical to use truss cross bracing in span. At the supports the forces to transfer are such that, normally, a diaphragm is needed to satisfy structural safety requirements.

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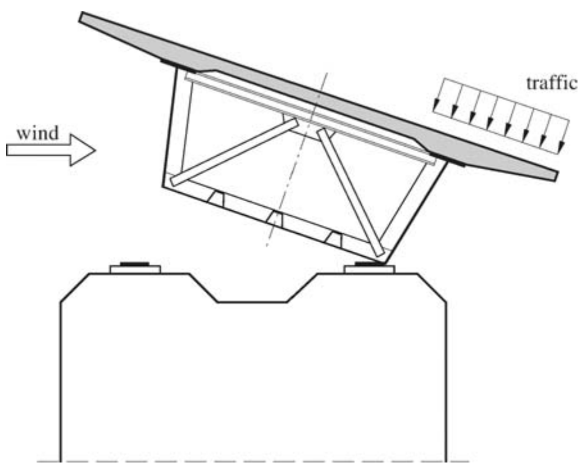
15 Overall Stability



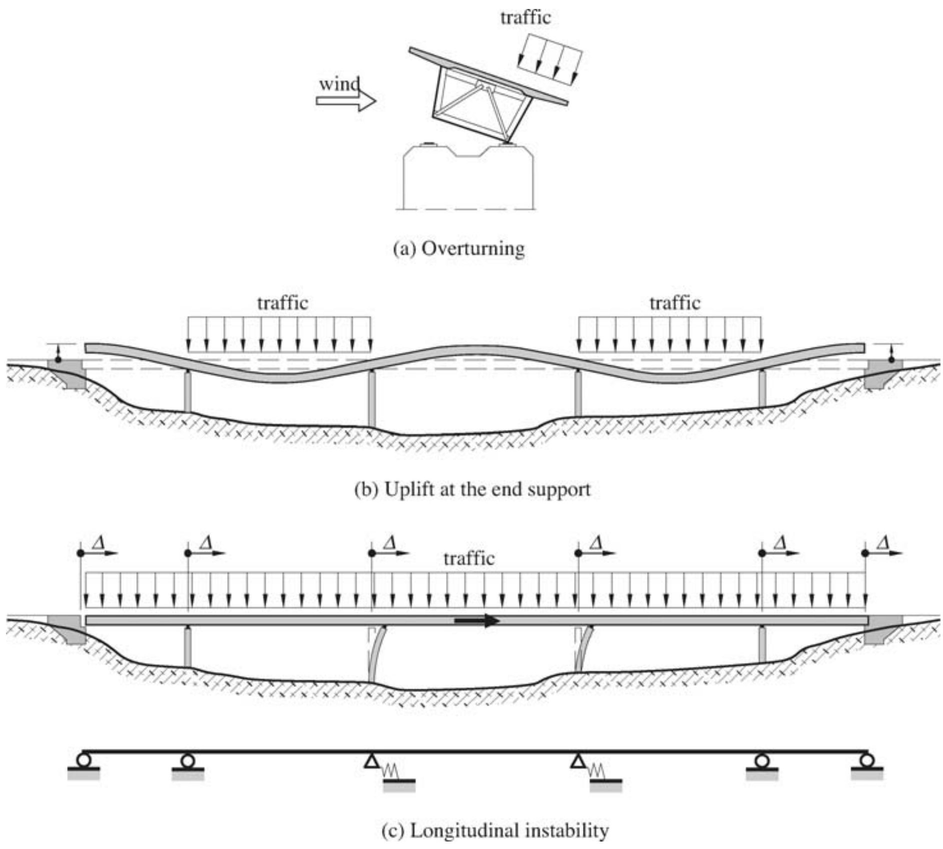
15.1 Introduction

This chapter considers overall stability of a bridge, and the design options to be taken to guarantee this stability. This has nothing to do with the cross sectional resistance of the individual structural members, but rather equilibrium of the structure as a whole. The designer must ensure that no overall movement would result in general instability of the bridge.

The structure must neither *overturn* nor *uplift* at the supports. For bridges on flexible piers, it is important that the piers are not excessively flexible, as this could lead to *bridge instability* as a result of longitudinal movement of the superstructure. Therefore, the designer must verify that under those actions that have an unfavourable effect, the structure resists any movements with a sufficient margin of safety against instability. This must be the case both during construction and in service.

To summarise, this chapter considers the three types of instability for beam bridges illustrated in Figure 15.1, namely:

- overturning by rotation about a longitudinal axis of the bridge (Fig. 15.1(a)),
- uplift at the supports (Fig. 15.1(b)),
- longitudinal instability of a bridge on flexible piers (Fig. 15.1(c)).



(c) Longitudinal instability

Fig. 15.1 Overall stability of beam bridges.

The first two of these types of movement concern the static equilibrium of the structure as a whole. They may arise when the bearings are not able to resist the negative support reactions that develop. The third type, longitudinal instability, should be thought of as a problem of a multi-bay sway frame that is not restrained in its plane, which is a typical structural form for bridges on flexible piers (§ 5.3.4).

After considering those actions that must be taken into account when checking overall stability (Sect. 15.2), the three types of instability noted above are considered in detail in Sections 15.3 (stability against overturning), 15.4 (uplift at supports), and 15.5 (longitudinal stability of bridges on flexible piers).

15.2 Actions to Consider

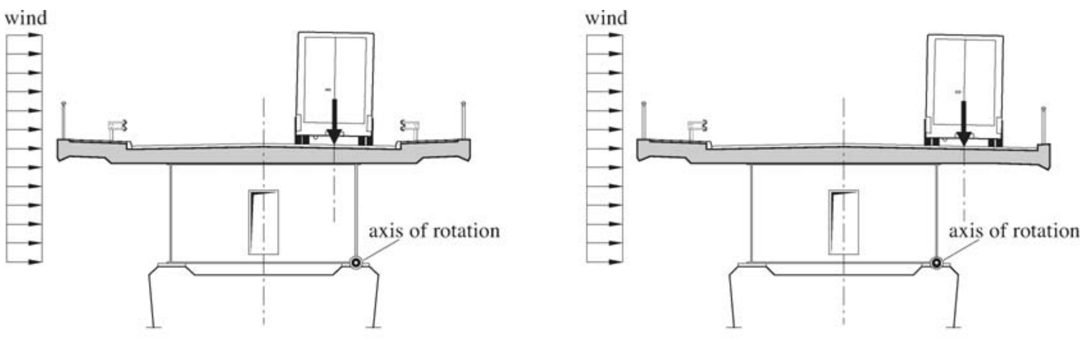
When checking overall stability of a bridge, one must always try to identify the most unfavourable hazard scenario by applying the destabilising actions with their maximum values and the stabilising actions with their minimum values. It is not unusual for the same force to act in a manner that is both stabilising and destabilising, as far as the overall stability of a structure is concerned, according to the position of its line of action. It is also possible that some of the effects of a given action are favourable while others are, simultaneously, unfavourable. Therefore, the *global* effect of an action must be considered when deciding whether or not it should be taken into account in a given hazard scenario. Particularly for the permanent loads, the designer must decide if a load factor greater or less than 1.0 should be used.

Figure 15.2 illustrates this concept for a check against overturning. Wind blowing on a group of trucks can have a destabilising effect, while the self-weight of the trucks may be either stabilising (Fig. 15.2(a)) or destabilising (Fig. 15.2(b)). The global effect of traffic in the "wind" hazard scenario, therefore, may be either stabilising or destabilising, depending on the positioning of the trucks on the bridge and the relative effects of the wind and the self-weight of the trucks.

It is unusual for all the unfavourable actions to act simultaneously. It is essential, therefore, to consider at least two hazard scenarios when checking overall stability: that which maximises the destabilising actions and that which minimises the stabilising actions. The most critical case may even be somewhere between these two extremes.

When checking stability against overturning or uplift at the supports by calculation, as presented in Paragraph 9.6.1, it is necessary to ensure that the following condition is satisfied. This corresponds to a type 1 limit state:

$$E_{d,\,dst} \leq E_{d,\,stb} \tag{15.1}$$



(a) Stabilising traffic load (b) Destabilising traffic load

Fig. 15.2 Influence of traffic position on overturning of a bridge subject to wind.

$E_{d,dst}$: design value for the effects of the destabilising actions
$E_{d,stb}$: design value for the effects of the stabilising actions

Both Table 9.5 and the standard SIA 260, as well as the Eurocodes, give the load factors to consider according to whether the action is favourable (stabilising) or unfavourable (destabilising).

The paragraphs below indicate how the different actions should be considered when carrying out the checks described in Sections 15.3 to 15.5. Chapter 10 contains more general information concerning the loads and actions on a bridge.

15.2.1 Permanent Actions

The permanent actions may have an effect that is either favourable or unfavourable according to different aspects of the overall stability of the bridge being considered. For example, the self-weight of the structure generally has a favourable effect when considering overturning, but is unfavourable as far as buckling of the piers is concerned (which is relevant to longitudinal stability of a bridge on flexible piers).

The partial load factor γ_G should be different depending on whether the permanent actions have a *globally* favourable or unfavourable effect for the case under consideration. For a given hazard scenario, a single load factor is applied to all the permanent actions, even if they are favourable for some parts of the structure and unfavourable for others (or vice versa); the global effect dictates this factor. For example, considering uplift at the supports of a continuous beam, the self-weight of certain spans will be stabilising, and that of others will be destabilising. Nevertheless, the load factor applied to self-weight should not differ from one span to another for the same load case.

15.2.2 Traffic Loads

The traffic loads should be placed in unfavourable positions, determined by considering the transverse and longitudinal influence lines, for the effect being considered. For example, when checking against overturning of a bridge superstructure, the most unfavourable hazard scenario normally only includes a single loaded notional traffic lane, located on one of the deck cantilevers (Fig. 15.2(b)).

15.2.3 Wind

Wind acts on both the deck (superstructure) and bridge supports (substructure). Transverse wind is relevant to stability against overturning of the superstructure, and longitudinal wind on the piers creates unfavourable movements as far as the longitudinal stability of the bridge is concerned. This hazard scenario is considered in Section 15.5.

The resultant of the transverse wind forces on the superstructure acts at approximately the mid-depth of the deck when there is no traffic present on a bridge. When there is dense traffic (for example, a stationary queue of trucks), the effective depth subject to wind increases, and the resultant of the transverse wind acts somewhere around the mid distance between the top of the trucks and the underside of the main beams. This increase in lever arm results in a significant increase in the overturning moment that must be taken into account (Sect. 15.3). The magnitude of the wind forces may be determined using the guidance given in Paragraph 10.4.1, as well as that given in the relevant standards.

15.2.4 Seismic Actions

Seismic actions are the result of ground movements which lead to both vertical and horizontal accelerations of a bridge. The vertical accelerations have the effect of varying the apparent self-weight, which may result in uplift of the deck when the weight reduces, or buckling of the piers, if it increases substantially. Horizontal accelerations, transferred by the piers and abutments to the superstructure, may result in either overturning of the bridge, when they act transversally, or overall instability, when they act longitudinally. The magnitude of the vertical and horizontal accelerations to be considered may be calculated using guidance given in the standard SIA 261, which also provides guidance on geometrical dimensions to adopt for moving bearing supports (§ 10.6.1).

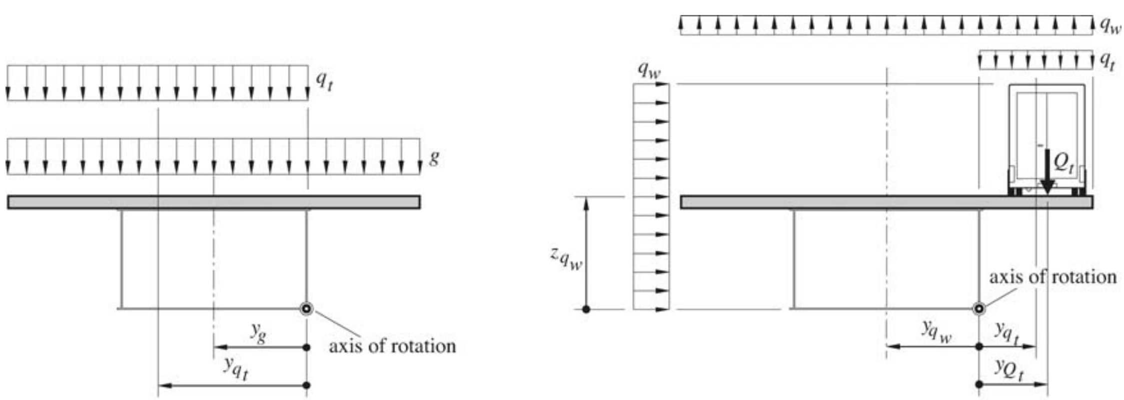
15.3 Overturning

15.3.1 The Phenomenon

Overturning happens when the superstructure rotates about a longitudinal axis as a result of an overturning moment. Figure 15.1(a) shows an example where the rotation is about a line of bearings. If the conceptual design of the supports is such that they can resist negative reactions, then overturning cannot happen, although clearly the designer must ensure that their resistance is adequate. Usual bearings cannot resist negative reactions. This means that the support reactions must remain positive for all the planned hazard scenarios, both during construction and in service.

15.3.2 Verification of Structural Safety (ULS)

Structural safety against overturning is checked using Equation (15.1), by comparing the effects (moments) of the stabilising actions with those of the destabilising actions. Moments are determined relative to the axis of rotation being considered, which will be one of the lines of supports (Fig. 15.2). Figure 15.3(a) shows stabilising actions, and Figure 15.3(b) shows destabilising actions. Figure 15.3 also shows the lever arms, y and z, of the resultants of the distributed actions and the concentrated traffic load. The list below gives, for this example, the two types of action that should be considered, depending on the particular hazard scenario (wind, traffic or earthquake), which include the corresponding load factors.



(a) Stabilising actions (b) Destabilising actions

Fig. 15.3 Examples of actions to consider when checking overturning of a bridge.

Stabilising actions are:

- the permanent loads g (self-weight of the structure and the non-structural elements),
- the traffic q_t located on the "stabilising" side of the supports (considering the axis of rotation), if this traffic is also used in the hazard scenario when it has a destabilising effect (wind on the trucks).

Destabilising actions are:

- the traffic q_t and Q_t located on the "destabilising" side of the supports (considering the axis of rotation), in particular on one of the slab cantilevers,
- the wind q_w that acts transversally and, potentially, vertically upwards, although the latter case is often ignored in calculations,
- the seismic action q_{acc} that acts transversally or vertically upwards (not shown in Figure 15.3).

15.3.3 Design Options

If stability against overturning is not guaranteed, then there are a number of ways to remedy this situation. These include:

- increase the width of a box girder (Fig. 15.4(a)),
- increase the spacing of the supports on the piers and/or abutments (Fig. 15.4(b)),
- specify bearings that can resist negative reactions: not forgetting the increased cost and reduced durability associated with this option,
- in the case of a curved bridge, which is more sensitive to overturning, reduce the span (§ 11.7.1),
- in the case of a rail bridge, use a conceptual design with two lines on one deck rather than two decks, each carrying a single line, as this facilitates wider spacing of the supports (§ 16.2.2).

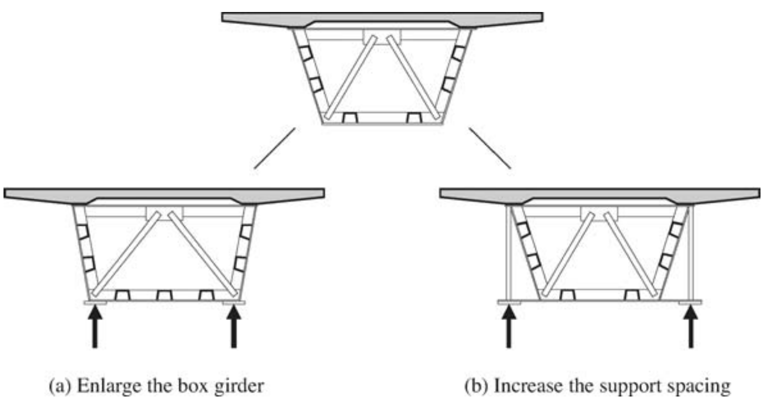


Fig. 15.4 Design options to improve resistance to overturning.

15.4 Uplift at Supports

15.4.1 The Phenomenon

The term *uplift at supports* is here used to describe uplift in the longitudinal sense of the bridge, as opposed to uplift in a transverse sense during overturning (Sect. 15.3). This type of uplift may arise, primarily, in the following situations:

- Bridges that are continuous over several spans (Fig. 15.1(b)): when one span is loaded and the adjacent spans are not, then the unloaded spans have a tendency to lift. Depending on the specifics, the beams may lose contact then with the supports at a pier or abutment. This phenomenon is accentuated if the unloaded neighbouring span is shorter than the loaded span, as is often the case with the end spans.
- During construction of a composite bridge, when the concrete slab is being placed on the steel beams, uplift may occur because the self-weight of the steelwork is small compared to that of the concrete. This is shown in Figure 15.5 for uplift at an abutment.
- For a skew bridge, if the torsional stiffness and the ratio between the skew e and span l are large, then the "external" supports (A′ and B″) have a tendency to lift, as is shown in Figure 11.28.

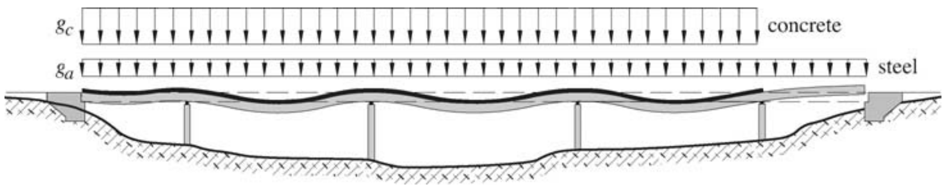


Fig. 15.5 Uplift of the end support during concreting of the slab.

A negative support reaction at an intermediate pier may be very damaging for the bearings if they have not been designed to cope with it. That said, the most detrimental case is when there is uplift at an abutment, because this results in a discontinuity at the joint between the deck and the abutment. Such a discontinuity can become a serious threat to traffic safety and, due to the resulting traffic impacts, it can seriously damage the bearings, the road joints, and, indeed, the bridge structure.

15.4.2 Verification of Structural Safety (ULS)

Structural safety against uplift should be considered as a type 1 limit state (Equation (15.1)) if the purpose of the checks is to ensure that there is no uplift at the supports. It should be considered as a type 2 limit state if the purpose of the checks is to guarantee the resistance of the structure following uplift at a support. The structural form of the bridge will change once the superstructure loses contact with the support where uplift has occurred.

15.4.3 Design Options

The most efficient means of preventing uplift at the abutment supports is to limit the ratio between the length of the first internal span and that of the end span. If this ratio exceeds 1.5, then uplift problems may occur. The adoption of bearings that can resist uplift should also be considered, although they are not commonly used. Another option is to provide ballast in the end span, for example by using a heavy concrete cross girder at the abutment.

15.5 Longitudinal Stability of Bridges on Flexible Piers

15.5.1 The Phenomenon

When a bridge has bearings that are fixed to one of the abutments (Fig. 15.6(a)), then longitudinal movement of the entire superstructure is prevented. The forces that act on the bridge in a longitudinal sense, such

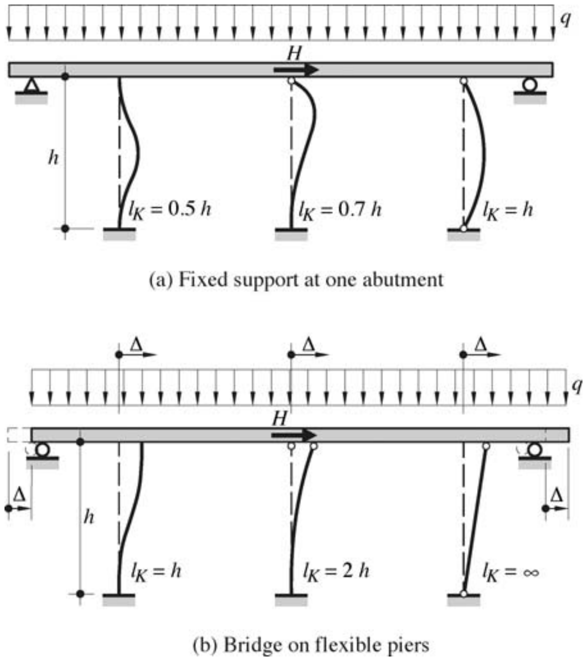


Fig. 15.6 Deflection and buckling lengths of the piers.

as those due to traffic braking or accelerating and seismic actions, are transferred to the abutment by the fixed supports. The buckling length of the piers is less than or equal to their height h, because the fixity at the abutment means they act like columns in a laterally restrained non-sway frame.

For a bridge on flexible piers, which has moving bearings at both abutments, the superstructure is only restrained longitudinally by the piers that are fixed to it, as shown in Figure 15.6(b). If these piers are too slender, and, therefore, too flexible in bending in the longitudinal sense of the bridge, they may be subject to large displacements under the horizontal forces. In such cases the superstructure, even when fixed bearings are used on the piers, does not represent a fixed point for the tops of the piers. The buckling length of the piers is then equivalent to that of a column in a sway frame that is not restrained laterally. This means that the buckling length of the piers is greater than the pier height h, which of course means this length is greater than the buckling length when there is a fixed bearing at one of the abutments.

The structural safety of the piers of a bridge on flexible piers is assured when the ultimate resistance of each of them has been shown to be satisfactory by checks that take into account second order effects in both the longitudinal and transverse directions. However, even when their resistance is sufficient, the piers may be very flexible, so the designer must also check that the complete set of piers (that are fixed to the superstructure) possesses sufficient stiffness in the longitudinal direction. This stiffness can be estimated by calculating α, an amplification factor that is applied to the total design vertical load (this is analogous to the situation of a multi-bay frame with nodes that are not fixed). Compared with a building sway frame (for which α_{cr} is used as an indicator of lateral stiffness, TGC Vol. 11, § 11.2.5), the multi-bay frame formed by the deck and piers of a bridge possesses the following peculiarities:

- the piers are not normally rigidly joined to the superstructure; rather, they are pinned,
- the bending stiffness of the superstructure (deck) is normally significantly greater than that of the piers.

The latter of these two peculiarities means that the calculations of *longitudinal stability* for a bridge on flexible piers can be substantially simplified, because bending deformations of the superstructure can be ignored.

Several methods exist for determining the elastic critical buckling load of such a system (for example, energy methods or stiffness methods including axial forces). A simplified method is presented below which has the advantage that, despite being simple, it is sufficiently accurate.

Such calculation methods do not allow the distribution of an external longitudinal force between the piers of a bridge on flexible piers to be identified. Rather, they simply allow overall stability of the bridge to be checked, including the effect of compression in the given piers. This is a question of elastic buckling by bifurcation. Checking the *structural safety* of each pier, considered independently, is carried out using the methods presented in TGC Volume 8, Chapter 9 (concrete piers) and Volume 10, Chapter 10 (steel piers). Checking the *resistance* of the piers is not covered in the present volume. Reference [15.1] gives detailed guidance on checking concrete piers in terms of both resistance and stability.

15.5.2 Verification of Structural Safety (ULS)

Hypotheses

The elastic critical buckling load of a frame with nodes that are not fixed is determined by considering its displaced form and with the following hypotheses:

- the material is linear elastic; in other words, the elastic modulus and second moment of area of the piers do not change as the loads and displacements increase,
- the bending stiffness of the superstructure is much greater than that of the piers,
- the piers are without imperfections and perfectly straight,
- the bearings are perfectly centred on the axes of the piers,
- the piers are loaded by the superstructure alone,
- buckling is by bifurcation.

The condition for stability is then reduced to a condition of equilibrium of the sum of the horizontal forces H_i due to a longitudinal displacement Δ_0 at the top of the pier. These forces and displacements are shown in Figure 15.7. Second order effects are taken into account by introducing an amplification factor, which is applied to the displacements (Equation 15.2). This factor is a function of the ratio between the design value of the load on a given pier and its elastic critical buckling load (Euler). The state of equilibrium is characterised by the condition that, for a given displacement at the top of the piers, the frame remains in its displaced position. In other words, the sum of the horizontal forces transferred between the piers and superstructure must be zero. If the sum of these forces is positive, then the system is stable; if it is negative, the system is unstable.

The designer, therefore, must identify the value α by which the vertical loads are multiplied. If it is greater than 1.0, then the system is stable, although it is commonly accepted [15.1] that the value of α must be greater than at least 2.0 for the longitudinal stiffness of the system to be considered sufficient.

The longitudinal displacement Δ takes second order effects into account using the following relationship between this displacement and the applied normal force αN_{Ed}:

$$\Delta = \Delta_0 \frac{1}{1 - \frac{\alpha N_{Ed}}{N_{cr}}} \tag{15.2}$$

Δ : longitudinal displacement of the top of the pier, which is equal to the longitudinal displacement of the superstructure

Δ_0 : initial displacement imposed on the structure

N_{Ed} : design value of the normal force acting on each pier (here compression is taken as positive)

N_{cr} : elastic critical buckling load of the pier, considering the buckling mode with displaced nodes

α : amplification factor for vertical loads

The elastic critical buckling load of a pier, for buckling by bifurcation, is given by the following equation (TGC 2, Sect. 20.3):

$$N_{cr} = \frac{\pi^2 EI}{l_K^2} \tag{15.3}$$

where l_K represents the buckling length of the pier, as shown in Figure 15.6(b).

Horizontal Forces Developed at the Tops of the Piers

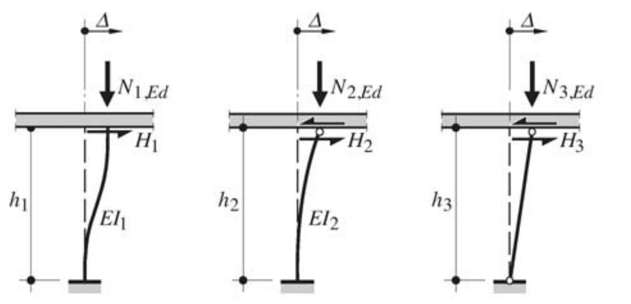
Due to horizontal displacement of the superstructure, in order to calculate the horizontal force H_i developed at the top of each pier, which is fixed to the superstructure, the following three types of pier must be distinguished:

- a pier with fixed supports at both ends (Fig. 15.7(a)),
- a pier which has a fixed support at its base but is pinned at its top, where it joins the superstructure (Fig. 15.7(b)),
- a pier that has pinned supports at both ends (Fig. 15.7(c)).

A pier that is pinned at both ends is only stable when it is held by the superstructure. In the absence of any vertical loading, there is no horizontal force at the top of such a pier, as a result of a displacement Δ_0. When vertical loading is present, then a horizontal force develops as the pier inclines. This force, which is the horizontal component of the normal force in the pinned-pinned pier, acts on the superstructure in a destabilising way; hence, it increases the displacement of the superstructure. Conversely, the restraining forces that develop in piers that are fixed at their base act to oppose the displacement Δ_0.

Piers with *fixed supports at both top and base* are rarely used with steel and composite bridges; they are normally considered to be more appropriate for concrete bridges. The displacement Δ_0 associated with a force H_1 at the top of a pier (Fig. 15.7(a)) is given by Equation (15.4):

$$\Delta_0 = \frac{H_1 h_1^3}{12 EI_1} \tag{15.4}$$



(a) Fixed at both ends (b) Fixed and pinned (c) Pinned at both ends

Fig. 15.7 Types of pier connection to the superstructure.

Combining this equation with Equations (15.2) and (15.3) results in Equation (15.5), which describes the relationship between the restraining force at the top of the pier H_1 and the total displacement Δ. The buckling length to use for this calculation of N_{cr} is $l_K = h_1$.

$$H_1 = \frac{12EI_1}{h_1^3}\left(1 - \frac{\alpha N_{1,Ed}}{N_{cr}}\right)\Delta = \frac{12\Delta}{\pi^2 h_1}(N_{cr} - \alpha N_{1,Ed}) \tag{15.5}$$

An analogous relationship can be derived for the case of a pier that is *fixed at its base but pinned at the top*. The relationship (15.4) between the initial displacement Δ_0 and the restraining force at the top of the pier (Fig. 15.7(b)) is modified as follows:

$$\Delta_0 = \frac{H_2 h_2^3}{3EI_2} \tag{15.6}$$

This time using a buckling length $l_K = 2h_2$, the restraining force H_2 is given by:

$$H_2 = \frac{3EI_2}{h_2^3}\left(1 - \frac{\alpha N_{2,Ed}}{N_{cr}}\right)\Delta = \frac{12\Delta}{\pi^2 h_2}(N_{cr} - \alpha N_{2,Ed}) \tag{15.7}$$

Despite its appearance, Equation (15.7) is not the same as Equation (15.5) because the elastic critical buckling load N_{cr} is not calculated using the same buckling length.

For a pier that is *pinned at its base and its top*, there is no increase in deformation due to second order effects. Equation (15.2) is not, therefore, appropriate. It is sufficient to simply consider equilibrium of the pier, which can only be subject to pure compression, given its structural form, in order to calculate the magnitude of the horizontal component (Fig. 15.7(c)):

$$H_3 = -\alpha N_{3,Ed} \cdot \frac{\Delta}{h_3} \tag{15.8}$$

The negative sign in this equation indicates that the force H_3 acting at the top of the pinned-pinned pier has a destabilising effect on the system.

Amplification Factor

The stability condition is expressed as equilibrium of the horizontal forces developed at the tops of the piers:

$$\Sigma H = \frac{12\Delta}{\pi^2}\left[\left(\sum_i \frac{N_{cr,i}}{h_i} - \alpha \sum_i \frac{N_{i,Ed}}{h_i}\right) - \alpha \frac{\pi^2}{12}\sum_j \frac{N_{j,Ed}}{h_j}\right] = 0 \tag{15.9}$$

i : index for the piers that are either fixed-fixed or fixed-pinned
j : index for the piers that are pinned-pinned

The amplification factor α for the vertical loads is then given by:

$$\alpha = \frac{\sum_i \frac{N_{cr,i}}{h_i}}{\sum_i \frac{N_{i,Ed}}{h_i} + \frac{\pi^2}{12}\sum_j \frac{N_{j,Ed}}{h_j}} \tag{15.10}$$

The load amplification factor is not affected by the magnitude of the displacement Δ. This is because the horizontal force developed at the tops of the piers is proportional to Δ, and the displacement is the same for all piers (given that they are fixed longitudinally to the superstructure). Therefore, all the horizontal forces, be they stabilising or destabilising, are proportional to the single value of longitudinal displacement Δ. This is expressed by Equation (15.9).

Consideration of the Hypotheses

The hypotheses used to calculate the critical buckling load assume buckling to occur by bifurcation. However, collapse of a multi-bay sway frame always occurs by divergence (second order resistance) at a lower load than that associated with bifurcation (elastic instability). This means that the vertical load amplification factor α does not reflect the real margin of safety of a system, because the hypotheses considered above are never all satisfied for a real bridge.

The effects of wind on the piers, eccentricity of the supports, or imperfections in the pier geometry can all result in collapse of the bridge at a load that is significantly lower than the critical load. Also, the stiffness of the piers in reality is affected by cracking of the concrete and by variable values of the elastic modulus, which changes as a function of the load level.

Cracking of the concrete piers depends on the levels of axial load and bending moment, and the elastic modulus depends on the duration of loading (recognising creep effects). One should adopt prudent values to allow for these two effects when considering the longitudinal stability of a bridge on flexible piers. Additionally, the hypotheses considered above assume that the piers have perfectly fixed supports at their bases; this is not always guaranteed, and account should be taken of any rotation that may occur at foundation level by increasing the buckling length.

In the absence of a more exact study of overall stability, to take into account these various influences, a factor α of at least 2.0 would seem reasonable in order to guarantee sufficient stiffness of a sway frame.

Numerical Example

The bridge on flexible piers shown in Figure 15.8 comprises piers that are pinned at both ends, piers that are fixed at their base and pinned at the top, and piers that are fixed at both ends. The normal force N_{Ed} acting on each pier is due to the self-weight of the structure and road traffic present over the total road surface. Table 15.9 gives input data for each pier, as well as the elastic critical buckling loads N_{cr}. It is assumed that all the piers have an identical bending stiffness, namely $EI = 840 \cdot 10^{12}$ Nmm^2.

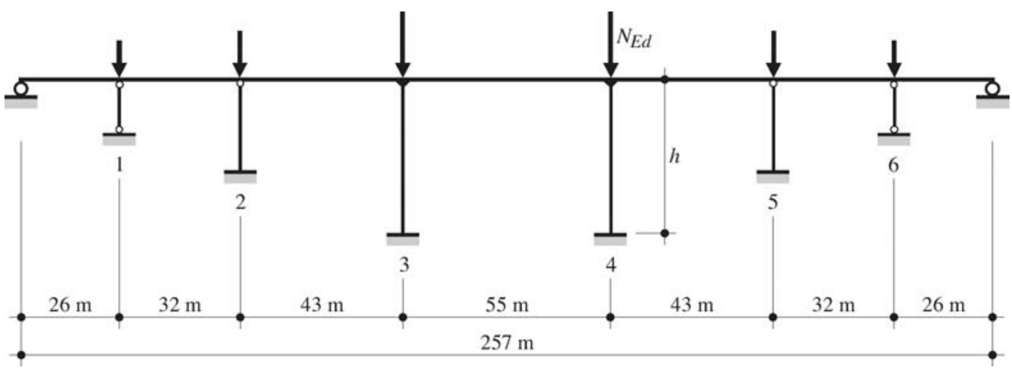


Fig. 15.8 Example of a bridge on flexible piers.

Table 15.9 Input data and critical buckling loads of the piers

Pier	1	2	3	4	5	6
Type	pinned at both ends	base fixed and top pinned	fixed at both ends	fixed at both ends	base fixed and top pinned	pinned at both ends
N_{Ed} [kN]	2800	3500	4900	4900	3500	2800
h [m]	7	12	20	20	12	7
l_K		$2h$	h	h	$2h$	
N_{cr} [10^3 kN]		14.4	20.7	20.7	14.4	

- For piers 2 to 5: $$\sum \frac{N_{cr}}{l} = \frac{2 \cdot 14.4 \cdot 10^3}{12 \cdot 10^3} + \frac{2 \cdot 20.7 \cdot 10^3}{20 \cdot 10^3} = 4.47\,\text{kN/mm}$$

$$\sum \frac{N_{Ed}}{l} = \frac{2 \cdot 3500}{12 \cdot 10^3} + \frac{2 \cdot 4900}{20 \cdot 10^3} = 1.07\,\text{kN/mm}$$

- For piers 1 and 6: $$\sum \frac{N_{Ed}}{l} = \frac{2 \cdot 2800}{7 \cdot 10^3} = 0.80\,\text{kN/mm}$$

The amplification factor for the vertical loads, according to Equation (15.10), is:

$$\alpha = \frac{4.47}{1.07 + \frac{\pi^2}{12}0.80} = 2.59$$

Because the amplification factor is greater than 2.0, the frame system may be considered to be sufficiently stiff to guarantee the longitudinal stability of the bridge on flexible piers.

15.5.3 Design Options

The provision of adequate longitudinal stiffness, to assure the longitudinal stability of a bridge on flexible piers, primarily concerns the piers. One may, in particular:

- increase the second moment of area of the stabilising piers,
- increase the number of piers that are fixed at their base and pinned at the top,
- reduce the number of piers that are pinned at both ends.

It should also be remembered that changing the structural form – by giving one of the abutments a fixed bearing – is a way of assuring longitudinal stability. Alongside overall stability considerations, the designer must check that each individual pier has sufficient resistance, considering: horizontal displacement at the top of the pier, horizontal forces and vertical loads, eccentricity of the actions, and real behaviour of the materials. These must be checked for the construction phase as well as for the bridge in service.

15.6 Reference

[15.1] BRÜHWILER, E., MENN, CH., *Stahlbetonbrücken*, Springer-Verlag, Vienna, New York, 2003.

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16 Railway bridges

Truss bridge over the Rhône River at Massongex (1904) (CH).
Eng. CFF
Photo Yannick Parvex (CFF)

16.1 Introduction

This chapter serves as an introduction to the conceptual and structural design of railway bridges. The basic principles and some of the input data concerning the actions for road bridges are equally valid for railway bridges. The purpose of this chapter, therefore, is to highlight and detail some specifics related to railway bridges.

The primary differences between road and railway bridges are a result of the nature and magnitude of the traffic loads to consider, as well as the added importance of fatigue, which often affect the design of the structure for a railway bridge. These differences result in typical transverse cross sections and construction details that are specific to this type of bridge. Figure 16.1 shows a typical cross section for a composite railway bridge with twin tracks. Serviceability requirements also differ significantly from those relevant to a road bridge, in particular those related to functionality of the bridge and to user comfort. Finally, there are certain requirements associated with the interaction between the railway and the bridge structure that warrant particular attention, such as those concerning the influence of the location of the joints in the rails and the joints in the structure on the structural behaviour and durability of the bridge.

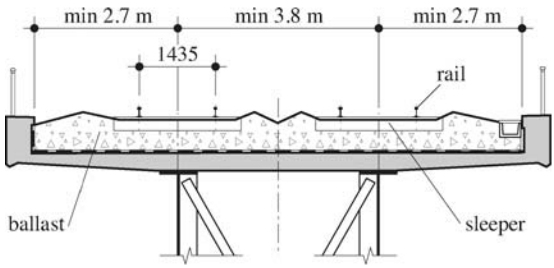


Fig. 16.1 Typical cross section for a composite railway bridge.

Some rail networks comprise two types of track: normal and narrow gauge. The vast majority of the tracks are normal gauge, and the narrow tracks are essentially used for private railway lines, often found in mountainous regions. This chapter only deals with railway bridges for normal-gauge tracks. For narrow-gauge tracks reference should be made both to guidance given by the private companies concerned and to the relevant standards.

In Switzerland the relevant authority, as far as railway bridges is concerned, is the Federal Office of Transport (FOT). The International Union of Railways (UIC) assures worldwide coordination of railway matters. Between them these organisations produce numerous regulations and guidance documents, some of which are mandatory. There are too many documents to list them all in this chapter, but reference is given to some where it is necessary or of particular interest. These are extremely useful and important documents to consider during the conceptual design of a railway bridge as they contain, amongst other things: typical construction details, examples of transverse cross sections, dimensions of standard components such as the rails, sleepers and troughs for ballast, or even headroom requirements relative to railways.

Section 16.2 describes the main aspects relevant to the conceptual design of a railway bridge, namely the choice of structural form, the cross section, and the construction details. Section 16.3 defines the loads to consider during analysis and design. Section 16.4 presents the specific checks that are associated with serviceability limit state and fatigue safety.

16.2 Conceptual Design

16.2.1 Longitudinal Structure

Types of Bridge

Practically all the types of longitudinal structures described in Section 5.3 could be considered for a railway bridge. However, the need to respect requirements for short-term deformations, which are more stringent than those for road bridges, mean that these requirements often govern the design of a railway bridge. A consequence of this is that it is wise to conceive a structure that is inherently stiff in bending, thus favouring beam bridges using trusses or composite sections rather than cable stayed or suspension bridges. Tied arch bridges (§ 18.2.2) are also well suited to the requirements of carrying a railway.

Traditional Structural Concept for a Railway Bridge

Steel railway bridges must be able to support very heavy loads, and there is a long tradition of the design and construction of such bridges going back to the 19th century, a time well before concrete started to be used and composite bridges could be envisaged. The traditional concept for railway bridges tries to minimise the self weight of material used while, at the same time, creating a stiff structure. Trusses are a typical result of this concept, and they are used for the primary structure as well as for both secondary members and the plan bracing.

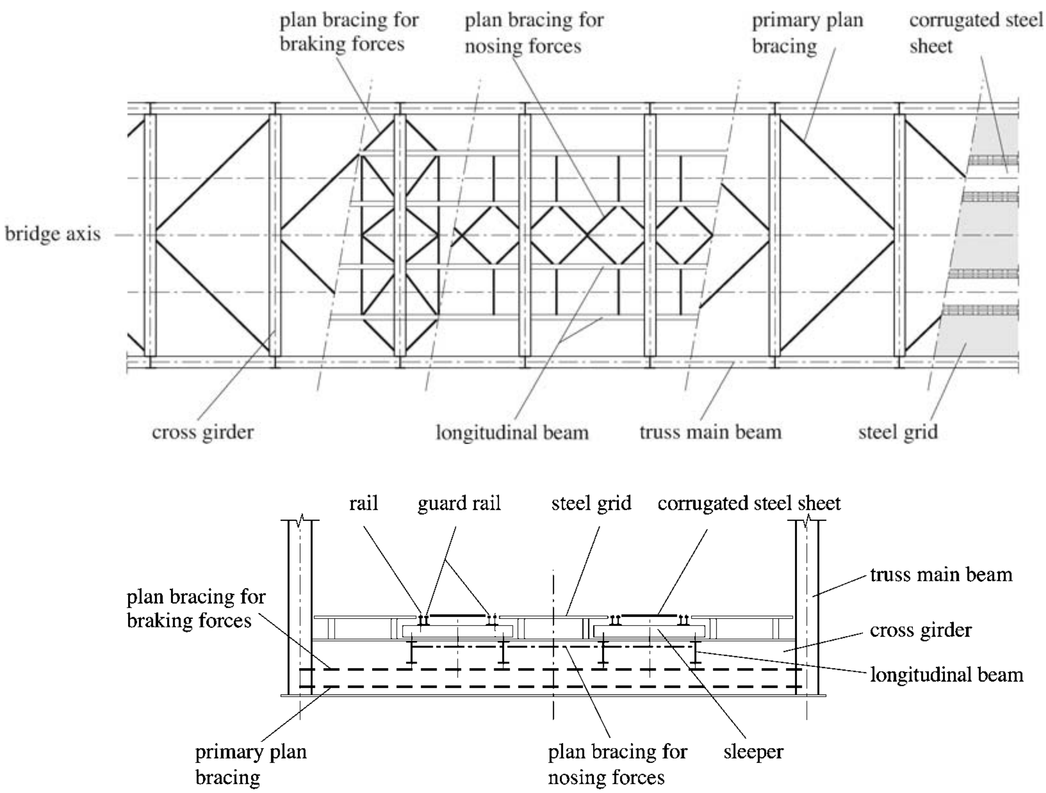


Fig. 16.2 Traditional configuration for a railway bridge.

Figure 16.2 shows a traditional configuration for a railway bridge. It is characterised by the presence of numerous structural members, which fulfil different functions within a certain hierarchy. The vertical loads are transferred from their point of application (the rails) to the supports via the sleepers and longitudinal beams (tertiary structural members), then the cross bracing (secondary structural members), before being transferred to the primary structural members, namely the main beams. Self-weight follows a similar load path from its point of application to the bridge supports. Traditional railway bridges are not normally ballasted, which results in some noise pollution during the passage of a train. This is the main reason why railway bridges of this traditional configuration are no longer built.

The various horizontal forces are transferred to the transverse supports by a system of several sets of plan bracing. Three different plan bracings may be distinguished for a bridge of traditional design:

- *Primary plan bracing*: positioned between the sets of cross bracing, this bracing ensures the resistance of the structure to horizontal forces acting perpendicularly to the bridge axis (such as wind or centrifugal forces). It comprises a truss, the chords of which are the main beams and the uprights are the cross bracing.
- *Plan bracing for nosing forces*: positioned between the longitudinal beams, this bracing transfers the horizontal forces that act laterally on the rails (wind blowing on the train, nosing force, or centrifugal force) to the cross bracing, which itself forms the uprights of the primary plan bracing. The longitudinal beams form the chords of this truss. For the design of the secondary plan bracing, the structural form is assumed to comprise a series of simple beams supported by the cross girders.
- *Plan bracing for braking forces*: related to the cross bracing locations, perpendicular to the bridge axis, this bracing ensures that the longitudinal forces associated with braking and acceleration are transferred to the main beams. These forces, which are applied to the rails, are transferred to the plan bracing for braking forces via the longitudinal beams. The main beams then transfer these forces longitudinally, in compression or tension, to the fixed supports of the bridge.

Modern Structural Concept for a Railway Bridge

The current concept for a railway bridge is, in a way, very similar to that for a road bridge. For example, a twin girder composite railway bridge comprises a reinforced concrete slab that may be prestressed and is structurally connected to the main beams, which are normally plate girders. Sets of cross bracing complete the structural steelwork. With such a structural concept, the concrete slab replaces the three sets of plan bracing found in a traditional railway bridge. Modern railway bridges are normally ballasted (§ 16.2.2).

Simple Beams and Continuous Beams

There are two possible options for the longitudinal structural form of a multi-span beam bridge: a continuous beam (or beams) or a succession of simple beams. While the solution using a continuous beam has hardly any disadvantages for road bridges, with regards to railway bridges, there are advantages and disadvantages associated with both options. The principal advantages of each option are:

Succession of simple beams

- insensitive to settlement of the supports,
- does not require expansion joints for the rails,
- better distribution of braking forces into the bridge substructure,
- reduced fatigue effects (beams are stiffer),
- if needed, it is easier to replace part of the bridge.

Continuous beam

- fewer supports, bearings, and joints in the bridge,
- reduced deformations (deflections and support rotations),
- shallower deck,
- more slender piers (no need for double bearings at the tops of the piers).

The increasing emphasis placed on the serviceability of bridges, and, in particular, their durability, makes the corresponding design requirements more severe than those associated with structural safety. This means that today the continuous beam option is used almost universally, in order to reduce the number of joints and bearings and to limit the deformations. Reducing deformations has become even more important with the introduction of railway lines, on which high-speed trains may exceed 200 km/h.

Spacing of Bridge Expansion Joints

The spacing of the expansion joints in the structure is important for a railway bridge because the movement of the joints is limited by the ability of the rail, sleeper, and ballast combination to withstand differential movements with the deck. In the absence of expansion joints in the rails – which, for reasons of economy and passenger comfort, is almost always the case today – the structure cannot move freely without creating forces in the rails and in the deck. In other words, there is an interaction between the rails and the bridge.

To take this behaviour into account, railway regulations in Switzerland and throughout Europe require the designer to consider this interaction when determining the distances between the joints in a bridge. More detail is given in Paragraph 16.4.2.

The designer does not need to carry out any explicit calculations concerning the interaction between the railway and bridge structures if the expansion length l_T (Fig. 16.3) is less than 90 m for a composite bridge, or 60 m for a steel bridge, assuming the three following conditions are satisfied:

- the rails are type UIC 60 with an ultimate tensile strength of 900 N/mm^2,
- the ballast is well compacted beneath the sleepers,
- the bridge has a radius of curvature, in the horizontal plane, greater than 1500 m.

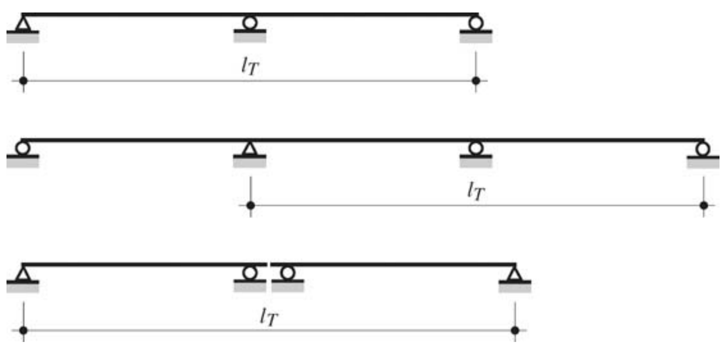


Fig. 16.3 Definition of expansion length l_T.

16.2.2 Transverse Cross Section

The choice of geometry for the transverse cross section and the dimensions of the structural members is a function of a number of parameters including the position of the slab, the number of tracks, and the presence (or otherwise) of ballast under the rails. These individual characteristics of the transverse cross section are discussed separately in this paragraph, even though, in reality, they are not independent of each other. The modern configuration that was discussed in the previous paragraph results in railway bridges with a transverse cross section similar to that found in road bridges, namely either twin girder or box girder steel beams acting compositely with a concrete slab.

Slab Position

Although numerous railway bridges have a lower slab (Fig. 16.4(a)), this solution is normally only adopted when the distance between the rails and the lower face of the deck does not allow an upper slab to be used (Fig. 16.4(b)). An upper slab configuration is generally preferred because it possesses the following advantages:

- The structural steel members are protected from the weather by the slab, which is itself protected by waterproofing. This results in greater durability with a reasonable amount of maintenance.
- The alignment of the rails can be adjusted and the bridge deck widened without having to move the main beams.
- For bridges carrying two tracks, a lower slab position requires wide spacing of the main beams ($\geq$ 10 m), which results in a significant transverse span for the slab cross girders.
- A derailed train may damage the structure when a lower slab configuration is used.

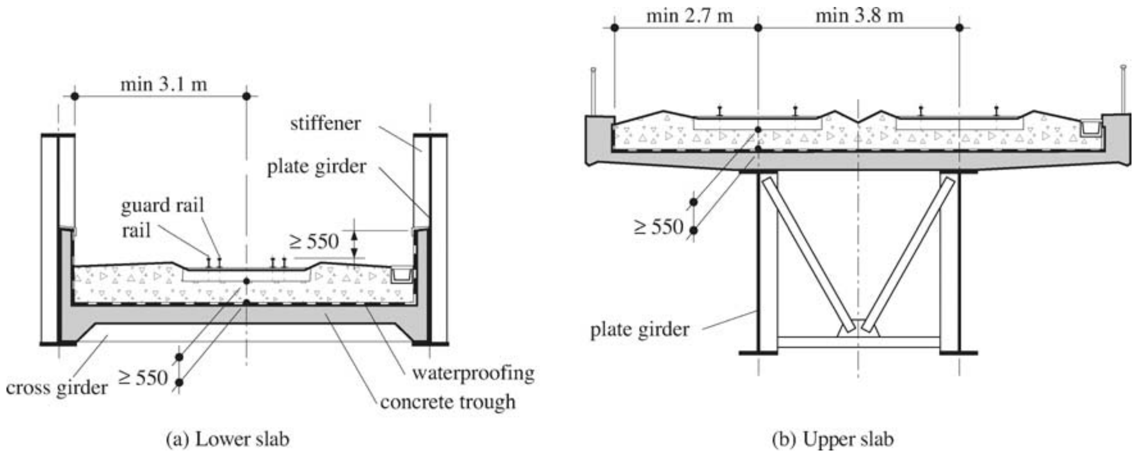


Fig. 16.4 Examples of composite bridges with lower or upper slab.

However, railway bridges with a lower slab are advantageous when the headroom beneath the bridge is limited. This configuration may also make noise barriers unnecessary, or they may be integrated with the main beams without significant increase in the apparent depth of the deck. This configuration, therefore, may be particularly beneficial in an urban environment.

Number of Tracks

If the railway line comprises two tracks, then the following two options may be considered:

- two separate bridges, each carrying one track, or
- a single bridge carrying both tracks.

The appropriate option is determined by the conditions associated with a given project, such as the spacing between the tracks, the position of the slab, or the span of the bridge:

- *Two bridges each with one track*: this option makes erection easier because the bridge elements, which are lighter and less cumbersome, may be fabricated off site then transported to site, resulting in fewer site-welded connections. Using two separate bridges also facilitates maintenance and replacement, as one of the lines can remain open to traffic while work is carried out on the other.
  Using two bridges is, however, only an option if there is sufficient space between the two tracks, namely at least 6.5 m between their centrelines. Typically this distance is only 3.8 m, so the alignment of the tracks must be adjusted in the regions approaching and exiting the bridge.
  For bridges with a *lower slab*, it is advantageous to consider two separate bridges wherever possible. It will normally be the most economical solution, particularly due to savings in the weight of the cross bracing.
- *Single bridge with two tracks*: for a bridge with an *upper slab*, the choice of a single bridge is often the most economical.

Ballasting of the Railway Track

Current regulations often require railway tracks on bridges to be ballasted. This configuration has the advantage of providing continuous support for the rails, which facilitates maintenance (machines to rearrange the ballast) and guarantees optimum comfort for the passengers. The ballast also reduces noise pollution and fatigue effects because:

- the dynamic amplification of the loads is reduced when the rails are ballasted, rather than fixed directly to the structure,
- the ballast results in good distribution of the loads from the axles to the rails and structure, which reduces the stress range $\Delta\sigma$. This is particularly the case for structural members located immediately below the rails (slab, cross girders).

Exceptions are sometimes justified, for example for bridges in an urban environment where the entire line is without ballast, or for moveable bridges where the use of ballast is incompatible with the deck movements that are required and the discontinuities at the joints. It should also be noted that the use of ballast increases the weight of a bridge, which can be significant particularly for long span bridges. It also increases the distance between the rails and the lower face of the deck, which can be a problem if headroom beneath the structure is limited.

When the rails are fixed directly to the structure without ballast, the whole structural system should be designed to provide flexibility. For railway bridges with a traditional configuration, the sleepers are directly placed on the longitudinal beams, and the flexibility is achieved by positioning the beams eccentrically to the rails. For railway bridges with a modern configuration, the flexibility between the rails and the slab is achieved by using a special fastening system for the rails, using mastic and polyurethane resins.

Main Structural Members

The main beams, be they composite or not, tend to be less slender than for a road bridge. This is because of the heavy vertical loads that act on them (traffic loads and ballast) as well as the more stringent deflection limits (Sect. 16.4). As a first approximation a slenderness for the steel beams of $l/15$ should be considered for a composite twin girder bridge with an upper slab, where l is the distance between the points of zero moment (under self-weight). For composite box girder bridges, or those with twin box section beams, the slenderness may be up to $l/20$.

For the concrete slab it is necessary to distinguish between upper and lower positions:

- *Lower slab* (Fig. 16.4(a)): the slab does not sit directly on the main beams because of its considerable span. Instead, it is supported on the cross girders. It does not necessarily contribute to the longitudinal bending resistance of the bridge, but depending on the way it is fixed to the cross girders, composite action may be achieved in the transverse direction. The concrete slab effectively replaces the plan bracing of a traditional configuration at the level of the rail.
  It is normally difficult to inspect the joints between the cross girders and main beams because these joints are partially, or totally, hidden by the slab. Therefore, this configuration should only be chosen in certain circumstances, when headroom beneath the bridge is limited.
- *Upper slab* (Fig. 16.4(b)): the slab sits directly on the main beams, to which it transfers the vertical loads in transverse bending. It effectively replaces the longitudinal beams found in a traditional railway bridge. Normally, the slab is structurally connected to the steel beams and, therefore, contributes to the longitudinal bending resistance as part of a composite cross section. The slab also replaces the upper plan bracing.
  The minimum thickness for a slab between the main beams, or on a box girder, is around 300 mm for bridges with ballast and a single track. A slightly thinner slab may be used when the rails are located on an elastic fastening system (no ballast). For a bridge with ballast and two tracks, a thickness of 400 mm is typical.

The slab (or trough) may also be made of steel (orthotropic deck, Sect. 6.7). It then comprises a steel plate that is normally stiffened in both directions, although it is beneficial to only stiffen the plate transversally to reduce the number of stiffener intersections and, thereby, improve fatigue performance [16.1]. Because it is integral with the main beams, an orthotropic deck contributes to the longitudinal bending resistance, as well as fulfills the function of plan bracing. Using an orthotropic deck allows the self-weight of the structure to be reduced, which is of particular interest for long span bridges. This solution does result in increased fabrication costs, and requires particular attention to be paid to issues of fatigue, but when circumstances dictate a light-weight solution, or a particularly shallow deck, the use of an orthotropic deck can be beneficial.

Options to Avoid Derailment

The derailment of a train crossing, or even when approaching a bridge, must be considered and appropriate measures taken to avoid it. Derailment presents the following dangers:

- impact of the train against the bridge structure, particularly when a lower slab configuration is used,
- train falling off the bridge, particularly when an upper slab is used.

Therefore, some means of keeping the train more-or-less on the rails should be considered, for example by using guard rails (Fig. 16.4(a)), or parapets with a height above rail level of at least 550 mm. There is no need for guard rails on bridges that are less than 10 m in total length.

When guard rails are not used, the designer must ensure that the derailed train (§ 16.3.5) will not overturn, and that neither the resistance of the bridge nor its overall stability would be compromised.

To reduce the likelihood of an impact following derailment, a certain space must be kept free to the sides of a track. For example, for a bridge crossing a railway, the location of the piers should be such that the distance from the rails to the piers is sufficient to avoid an impact that could result in pier collapse (Fig. 16.5). The minimum distance needed is a function of a number of parameters, such as the importance of

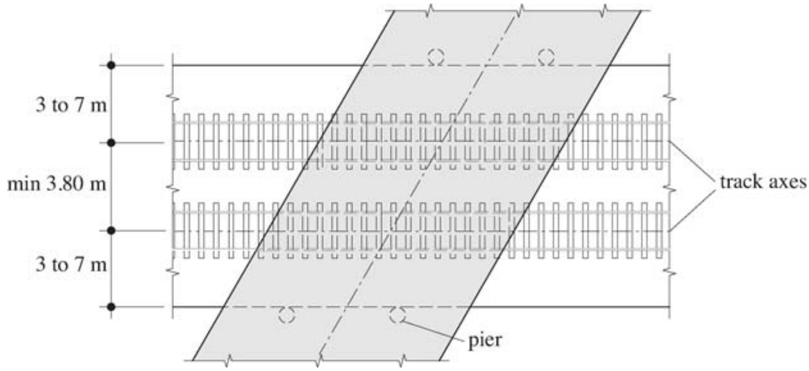


Fig. 16.5 Required minimum distance to the piers for a bridge crossing over a railway line.

the railway line, the speed that trains may reach, or, indeed, the radius of curvature in plan of the line. It also reflects the potential damage, including collapse of the bridge, which could be caused by an impact. More information is given in [16.2].

Collection and Evacuation of Rainwater

The longitudinal and transverse falls that are needed to collect and evacuate water are defined by the client's representative. According to regulations issued, for example, by the Swiss railways authority [16.3], the longitudinal fall must be at least 1.5%, with a maximum value of 5%. Transversally, the guidance shown in Figure 16.6 must be respected.

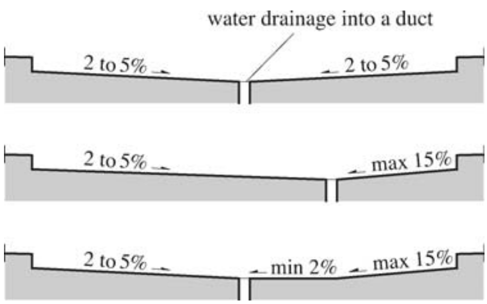


Fig. 16.6 Cross falls for evacuation of water from the slab surface.

Because railway bridges are not exposed to de-icing salts, and trains have only a small probability of dropping pollutants, collecting water that lands on a railway bridge is not as important as for a road bridge. The water may be allowed to drain directly into a waterway, although in all cases reference should be made to regulations produced by the relevant rail authority, requirements of the client's representative, and any particular requirements concerning the environment (zoning plans, laws concerning water protection).

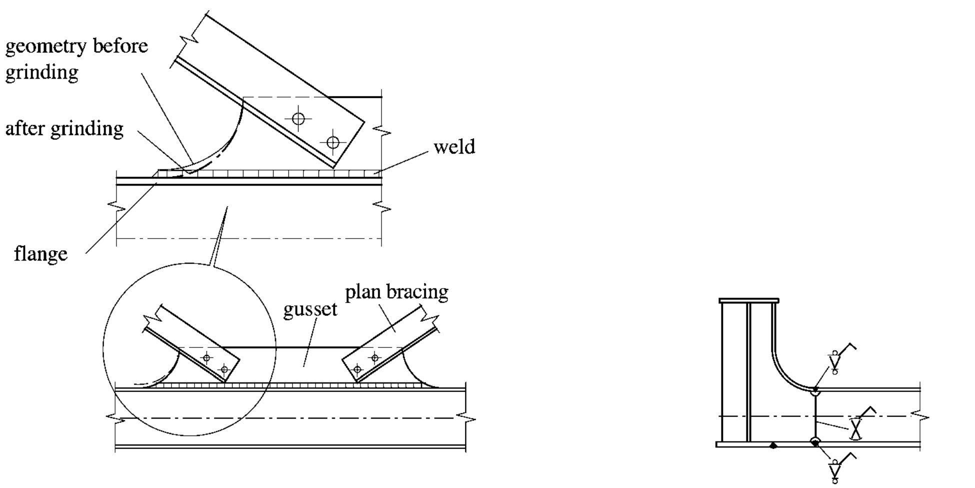
16.2.3 Fatigue and Joints

The planned service life for railway bridges is normally 100 years. During this period the bridge should be able to support, with an adequate level of safety, all rail traffic and the stress variations that result from it. This means that particular attention must be paid to issues of fatigue. The designer must:

- conceptually design and size the joints so that they are fatigue resistant,
- ensure that the steel structure and joints can be inspected, so that any fatigue cracks that develop can be detected.

The main way of ensuring good fatigue resistance of a joint is to design and detail it such that the flow of stresses is as continuous as possible. One must also pay attention to "parasitic" moments and forces – those that result from real behaviour, which may differ from that assumed during design.

Some examples of typical construction details are given in Figure 16.7. Figure 16.7(a) shows a joint between a plan bracing gusset plate and the flange of a main beam. Rounding and grinding the plate and weld reduces the stress concentrations at the end of the weld. Figure 16.7(b) shows a joint between the cross girder of a bridge with a lower slab and a main beam. For such a joint it is important to create a continuous flow of stresses, and, thereby, reduce the stresses in the web of the main beam that result from its rigid support of the cross girder. Examples of a node in a Warren truss are shown in Figures 6.21(c) and 6.23(b). The gusset plates are cut with a rounded profile to avoid stress concentrations.



(a) Plan bracing to flange joint (b) Cross girder to main beam joint

Fig. 16.7 Examples of construction details with good fatigue resistance.

The propagation of a fatigue crack may result in brittle failure. Using high quality steel can reduce the probability of this happening. For railway bridges, the minimum steel quality should be J2 for the primary structural members. Paragraph 4.5.4 provides detailed information on choosing appropriate steel, as a function of temperature and plate thickness.

It is possible to improve the fatigue behaviour of some types of joint by treating the welds with an adequate post-welding treatment. Details of relevant methods are given in TGC Volume 10, Paragraph 13.6.5, and in references [16.1] and [16.4].

16.2.4 Specific Construction Details

Numerous construction details that are specific to railway bridges are provided in the various regulations and other guidance published, for example, in reference [16.3]. From amongst these details, Figure 16.8 shows two examples for an expansion joint in the structure. When the expansion length l_T (Fig. 16.3) is less than or equal to 90 m, the expansion joints only need limited movement. In such cases elastomer joints can be used; these have the advantage of being able to support continuous ballast (Fig. 16.8(a)). For greater expansion lengths the joints must be able to accommodate greater movements, and the ballast must be discontinuous at the joint locations (Fig. 16.8(b)). The ballast is retained on either side of the joint using an appropriate, bespoke, solution.

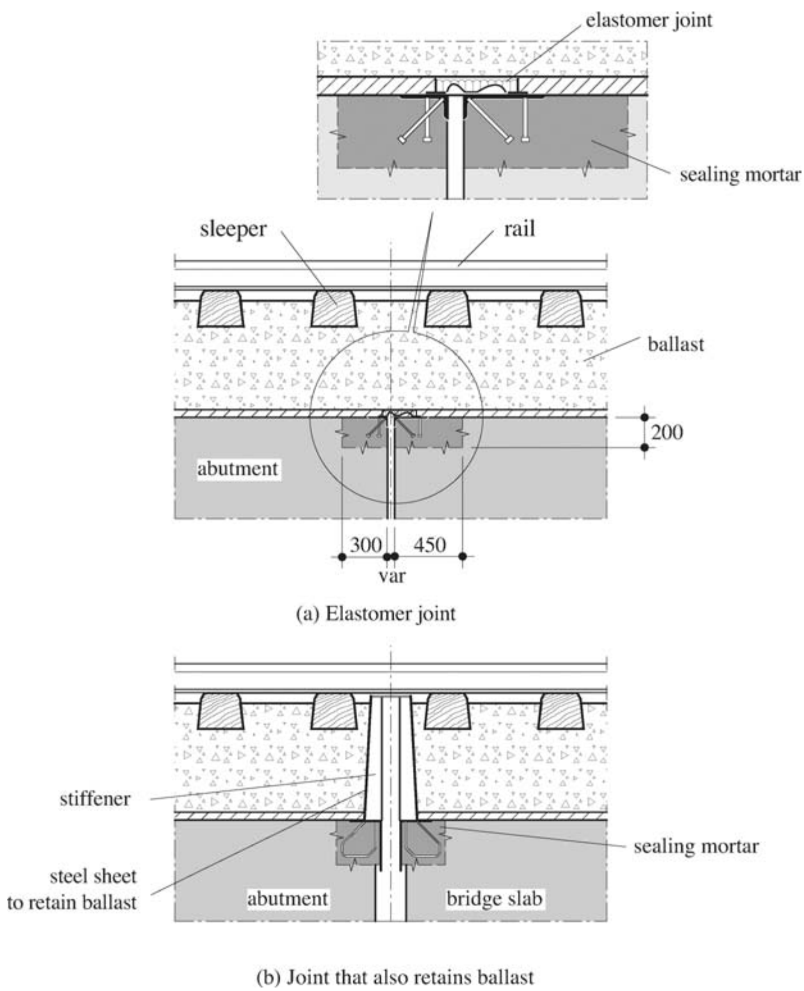


Fig. 16.8 Examples of expansion joints.

16.2.5 Appearance

As mentioned in Chapter 4, when considering the qualities required of a bridge, appearance is a complex and subjective issue. In addition to the references cited in Chapters 4 and 17, it may be helpful to consider the guidance issued by the Swiss railways authority on this subject [16.5].

16.3 Loads and Actions

16.3.1 Self-weight

Self-weight is calculated as the product of the average density of each material and its corresponding volume. Densities for various materials are given in the relevant standards. Additional information on this subject is given in the paragraphs below.

Structure

The self-weight of the structural members is estimated, first of all, in an empirical way, then calculated and modified as necessary as the design progresses. For a composite structure the weight of the slab is defined by its width, which is fixed as a function of the number of tracks, and its average thickness (minimum 300 mm for a single track and 400 mm for two tracks). To estimate the self-weight of the steelwork, one may, as a first approximation, consider Equation (10.1) assuming a slab width of $2b = 10$ m.

Ballast, Rails, and Sleepers

The density of in-situ ballast is 17 kN/m^3. The minimum thickness of ballast is dictated by the relevant authorities as a function of the requirements for maintenance of the rails (machines for placing and compacting the ballast). In the absence of more precise information, one may assume a thickness of ballast of 550 mm below the lower face of the sleepers.

A range of rails and sleepers is used to construct railways. The rails are distinguished by their cross-sectional form and area, and, therefore, also by their weight per metre length. Sleepers are distinguished by the material used (wood, concrete, steel) and their thickness. The choice of a rail and sleeper combination depends on:

- the importance of the line (speed, weight, number of trains),
- the width of the track (narrow or normal gauge),
- the country.

In the absence of other guidance, the information given in Table 16.9 may be used for initial sizing. The relevant authority (national or international, for example UIC) may provide additional information on a case by case basis. The type of track is normally specified by the client's representative.

Table 16.9 Indicative weights for rails and sleepers used in Switzerland (taken from [16.6]).

Elements	Characteristic	Weight		Remarks
Rail	Normal gauge	0.54 to 0.60	kN/m	Weight per rail according to the axle load allowed on the line
	Narrow gauge	0.35 to 0.55	kN/m	
Sleeper	Timber	0.90	kN/sleeper	Standard spacing: 60 cm
	Concrete	2.0 to 2.8	kN/sleeper	
	Steel	0.70	kN/sleeper	

16.3.2 Traffic Loads

The imposed loads to be considered for railway bridges are described in the standard SIA 261, based on the load models for trains produced by the International Union of Railways (UIC). These models are also given in the European standards [16.7], which additionally provide detailed information on the loads to consider for high-speed lines (speeds greater than 200 km/h), and how to carry out an explicit dynamic analysis of the structure.

Vertical Loads

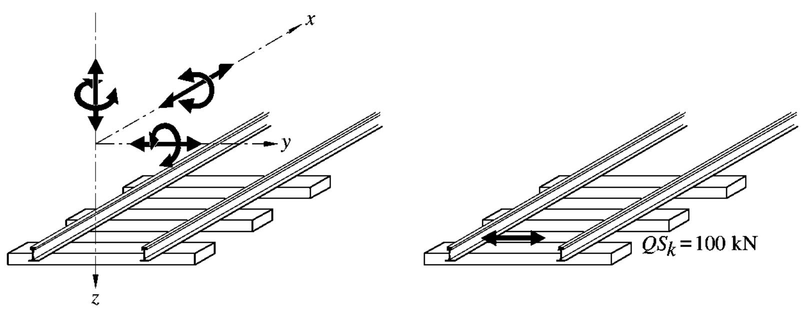
The actions resulting from railway traffic depend on the type of track (normal or narrow gauge) and the type of traffic circulating on the line (normal or heavy traffic, high speed). The standard SIA 261 gives different load models for these different types of traffic. For special railway lines, such as cog-wheel or funicular railways or lines allowing the passage of special trains, data concerning the loads are available from the relevant authority.

Three load models are defined in the standard SIA 261. The authority identifies lengths of railway line over which model 3 need not be applied. The possibility of simultaneous presence of trains on different tracks must be allowed for on bridges that support a number of tracks. The vertical loads are multiplied by a dynamic amplification factor that depends on the influence length of the structural member under consideration (§ 16.3.3). They must also be multiplied by the coefficient α (coefficient for the classification of standardised load models according to § 16.3.4). The loads should always be placed in their most unfavourable position. The axle and distributed loads that are favourable, according to the relevant influence line, are not included in the calculation of internal moments and forces.

Transverse Horizontal Forces

Unlike road bridges, a *centrifugal force* should be taken into account when designing railway bridges. The centrifugal force may be determined using guidance given, for example, in the standard SIA 261. It is assumed to act 1.80 m above the top of the rails. This force should be multiplied by the coefficient α but not by the dynamic amplification factor.

Additionally, those forces associated with so-called nosing should be considered. The *nosing force* is dynamic, and results from the train movements due to irregularities in the rails and suspension components. As shown in Figure 16.10(a) these movements, which vary with time, are three-dimensional and



(a) Train movements (b) Load model

Fig. 16.10 Nosing force.

have rotational components. An exact determination of the force that results from these movements is complex, which is why the relevant standards propose the use of an equivalent static force. This static force is shown in Figure 16.10(b). Its characteristic value is 100 kN in SIA 261, and it acts at the top of the rails, perpendicularly to their axis. The nosing force should be positioned in its most unfavourable location. It should be multiplied by the coefficient α but not by the dynamic amplification factor.

Longitudinal Horizontal Forces

The effects of *braking* and *acceleration* of trains are represented as longitudinal horizontal forces. These forces act parallel to the rail axis and are applied at the top of the rails (the rolling surface). Their magnitude is defined in the standard SIA 261. They should be multiplied by the coefficient α but not by the dynamic amplification factor.

16.3.3 Dynamic Amplification Factor

The static stresses and deformations of a railway bridge are amplified by several dynamic phenomena:

- the speed of loading, which is linked to the train speed and the influence length of the member under consideration,
- the frequency of load application, which is a function of the spacing of the axles and the speed of the train,
- the imperfections and irregularities of the rails and the train wheels, which cause variations in the forces associated with wheel loading.

To take these various phenomena into account, the vertical traffic loads must be increased using a dynamic amplification factor Φ. This is used when checking structural safety, including fatigue considerations, as well as serviceability. The standard SIA 261 gives a relationship that allows this factor to be calculated as a function of the influence length of the member under consideration. The standard includes a table that may be used to determine the influence length of different members in a bridge structure.

In addition to the factor Φ (which is defined in the same way as given in SIA 261), European standards [16.7] define three additional dynamic amplification factors to take into account the dynamic phenomena associated with high-speed lines ($v > 200$ km/h), the particular needs of assessing existing bridges, and for fatigue calculations.

16.3.4 Coefficient for the Classification of Normalised Load Model

The coefficient α takes into account the difference between the load model (§ 16.3.2) and the real traffic that uses a given railway line. It is primarily a function of the types of trains that run on the line, and varies between 0.75 and 1.46. All the loads and forces associated with a particular group of actions should be multiplied by α, be they vertical actions, or transverse or longitudinal horizontal forces.

In Switzerland the most common value for α, according to the standard SIA 261, is 1.33 for checking structural safety. This value is recommended by the UIC for main lines and lines used for international goods traffic. When considering fatigue and serviceability, a value of $\alpha = 1.0$ is normally used.

16.3.5 Derailment and Impact Loads

When derailment occurs, the impact due to the considerable weight of a train hitting one of a bridge's structural members can be enormous. As a first option one should therefore take measures to ensure that derailment, resulting in impact, does not occur. Such measures were discussed in Paragraph 16.2.2. If avoidance is impossible, then the relevant codes and standards provide values for equivalent static forces, which allow the effect of an *impact* against the structure to be determined.

The magnitude of the equivalent static force, which is taken as an accidental action, varies between 1500 kN and 4000 kN depending on the distance between the structural members that could be hit and the axis of the railway track. The standard SIA 261 gives more information concerning how this force acts on the structure.

Two load models are defined in SIA 261 to take into account *derailment*. These allow the stability of the structure against overturning, and structural safety, to be checked. Derailment is taken as an accidental action. As noted above, it may be possible to take measures such as those presented in Paragraph 16.2.2 to ensure that a train does not deviate too far from its intended path, so does not strike any structural members.

16.3.6 Aerodynamic Effects on Noise Barriers

When a train passes between or alongside noise barriers, the mass of air that is displaced by the train causes pressure increases and decreases at the front and rear of the train (Fig. 16.11). These actions may be modelled using equivalent static forces. The characteristic value q_k of the pressure that acts on the noise barriers may be determined using the diagram shown in Figure 16.11, which also shows how this pressure is distributed. The speed v should be taken as the design speed defined for the line. Additionally, to allow for specific dynamic effects, the first and last five metres of a noise barrier, and any zones where there are variations to the dimensions of the barrier in either elevation or plan, should be loaded by a pressure equal to $2q_k$.

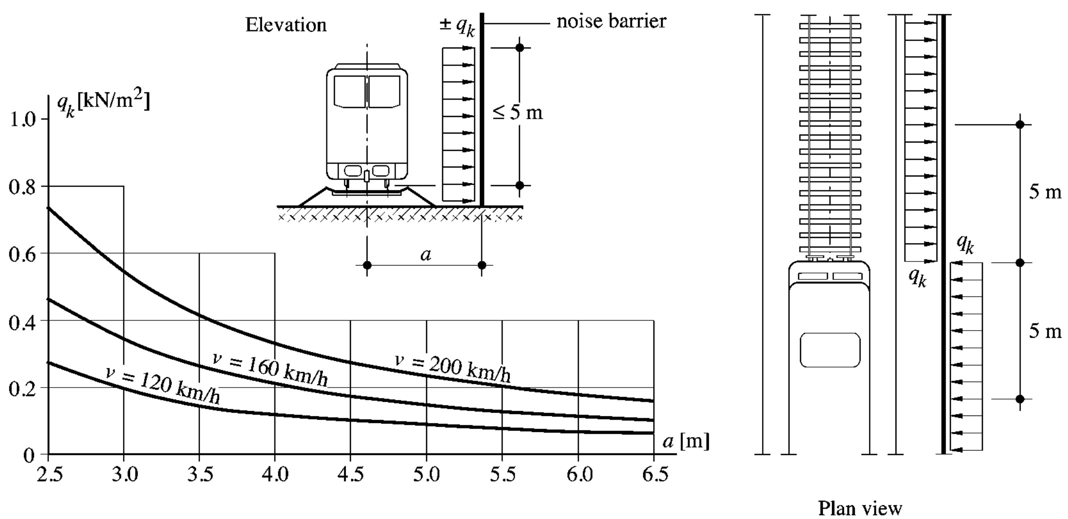


Fig. 16.11 Equivalent static pressure q_k for aerodynamic effects on noise barriers.

Diagrams that are similar to that shown in Figure 16.11, but valid for other types of structure, such as canopies on station platforms and walkways over the rails, are given in EN1991-2 [16.7].

16.3.7 Temperature

The effect that temperature has on a railway bridge is determined in the same way as for a road bridge. That said, for bridges with ballast, the characteristic values of vertical temperature gradients (§ 10.4.2) may be reduced by 50%.

16.4 Verifications

As with other types of bridge, one must ensure that both structural safety and serviceability are guaranteed. For railway bridges two particular actions should be taken into account, namely derailment of a train and the vibrations caused by the passage of a train.

Derailment must not result in bridge collapse, although it may clearly cause service disruption. Design options taken to prevent derailment are associated with the serviceability of the bridge and may be checked using appropriate serviceability load and resistance factors. However, possible consequences of derailment affecting the resistance or stability of a bridge should be checked as an ultimate limit state.

Vibrations can result in two types of problem. Certain vibrations can disrupt the ballast, which could lead to derailment. Vibrations caused by the regular passage of bogeys can result in significant dynamic amplification, similar to resonance of the structure. This may affect the structural safety of the bridge.

16.4.1 Verification of Structural Safety (ULS)

The checking of structural safety, including fatigue, is carried out in the same way for both railway and road bridges. Chapters 9 to 14 contain more information on this subject.

For checking fatigue safety, the type of railway line must be known (heavy traffic, national or regional) in order to estimate the likely volume of traffic and define certain details of the checks (correction factor λ_1 according to the standard SIA 261).

As noted above, when design options associated with limiting bridge deformations are used as a means of preventing derailment, they may be considered as serviceability criteria. On the other hand, some of the consequences of derailment must be considered as ultimate limit states resulting from an accidental action. In particular, such consequences are:

- *impact* of a train against the structure, which may be limited or prevented by using appropriate construction details (§ 16.2.2), or taken into account in the design calculations (§ 16.3.5),
- *overturning* of the bridge due to a derailed train can be considered by applying a load, which is eccentric to the axis of the track, according to specific load models (standard SIA 261).

The *loading frequency* is calculated considering the speed of the train and the spacing of the bogeys. If this is close to one of the natural frequencies of the bridge, then *resonance*, resulting in significant dynamic amplification, can occur. Resonance is an issue for structural safety that particularly affects high speed lines ($v > 200$ km/h). For bridges carrying lines limited to speeds less than 200 km/h, the likelihood of resonance is small and, according to Swiss standards, does not warrant explicit checking because, supposedly, limiting deflections implicitly covers it (§ 16.4.2). European standards [16.7] contain more detailed guidance for situations when a dynamic analysis is required, particularly for bridges carrying high speed lines.

16.4.2 Verification of Serviceability (SLS)

The serviceability of railway bridges concerns the comfort of the passengers and those in the neighbouring areas, the functionality of the bridge, and its appearance. These different serviceability limit states, as well as the criteria associated with each of them, are described below. They are taken from the standard SIA 260 and Annex 2 of Eurocode EN1990 [16.8]. Clearly when checking the serviceability of a railway bridge, the specifics of this type of structure must be taken into account. Limits for railway bridges are more stringent than those associated with road bridges, mainly to prevent train derailment. However, it should be noted that, in the first instance, adequate maintenance of the rails is a good way of reducing the probability of derailment.

Comfort

Vertical accelerations should be limited in order to assure the comfort of passengers. This check is implicitly covered by the checks made on vertical deflections (see below).

Noise nuisance caused by the passage of a train over a bridge may be particularly unpleasant. Currently the most common design option taken to reduce this noise is to support the rails on ballast. The ballast absorbs the sound waves generated by the wheels on the rails, as well as the mechanical vibrations that, if they were transferred to the steel structure, could lead to annoying resonant sounds. When it is not possible to use ballast, special rail fastening may be used between the rails and the steel structure, or the concrete slab. These fastenings include a layer of material that can absorb the vibrations coming from the vehicle and rails. Noise barriers may also be used to combat this problem. However, such barriers often affect the appearance of a bridge and may create problems of durability (fixings or materials used). Vibrations resulting from wind or the passage of trains may result in fatigue cracking. Therefore, the best option is to try to reduce the noise at its source, for example using modern rolling stock and ballasted rails, before thinking about the use of noise barriers.

Functionality

The functionality of a railway line is ensured if the probability of derailment is sufficiently low. This objective may be achieved by limiting:

- vertical acceleration, to ensure the stability of a ballasted track (when the basic speed for the line is less than 200 km/h, this is achieved indirectly by limiting vertical deflections),
- twisting of the railway track, to ensure compatibility between the rails and vehicles,
- rotation of the beams at their end supports, and the vertical deflection of the deck relative to the abutments, to ensure that the rails are continuous at the bridge ends,
- the compressive stresses in the rails resulting from the interaction between the structure and the rails (to prevent buckling of the rails).

Swiss standards limit the *vertical deflection* of a railway bridge as follows:

- $w < w_{lim} = l/800$ for $v < 80$ km/h,
- $w < w_{lim} = l/(15v - 400)$ for 80 km/h $\leq v \leq$ 200 km/h.

Deflection checks should be carried out considering the frequent service loads, increased by the dynamic amplification factor (§ 16.3.3) and with $\alpha = 1.00$ (§ 16.3.4). The deflection limits w are severe but allow less frequent maintenance work on the line. For example, smaller bridge deformations considerably reduce the amount by which the ballast becomes disturbed and, therefore, increase the interval between maintenance interventions to rearrange it.

Twisting of the track is defined by the ratio θ/L, where θ is the angle of rotation of the plane A-B-C-D when it twists. This angle is shown in Figure 16.12, and is measured between the straight line A-D and the line A-D′ defined by the plane after twisting. The length L is normally taken as 3 m. According to the standard SIA 260 twisting should be limited to 0.7 mrad/m under the frequent service loads, increased by the dynamic amplification factor (§ 16.3.3) and with $\alpha = 1.33$ (§ 16.3.4), for lines with a basic speed in excess of 120 km/h. When the speed is less than 120 km/h, the twisting is limited to 1.0 mrad/m. European standards propose similar limiting values. In practice this check is only carried out for bridges that are relatively flexible in torsion, for example temporary bridges, or for high speed lines ($v > 200$ km/h), in which case more severe limits are applied and can govern the conceptual design.

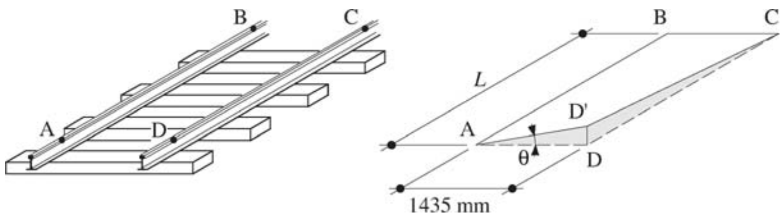


Fig. 16.12 Definition of twisting for a railway track.

The ends of the slab should be perpendicular to the axes of the railway lines in order to limit twisting. As far as possible, skew bridges should be avoided, because the skew results in significant twisting when a train passes over the joints between the deck and abutments. If the alignment of the railway line is oblique to the axis of the obstacle to be crossed by the bridge, transition zones should be designed so that the ends of the slab remain perpendicular to the axis of the tracks, as shown in Figure 16.13. However, the free

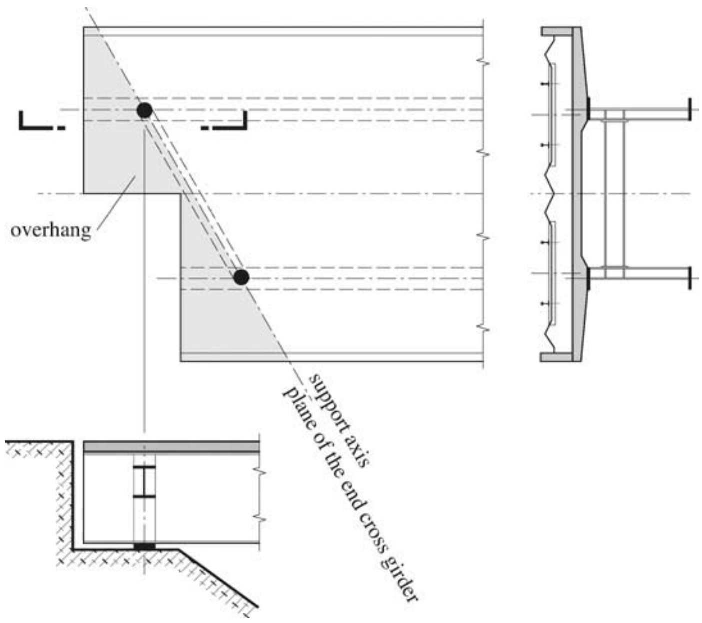


Fig. 16.13 Recommended layout for the abutments and supports of a skew bridge.

length of slab behind the supports must be limited to avoid excessive vertical displacements of the slab relative to the backfill. Filling behind the abutments of skew bridges with lean mix concrete may be another way of limiting the twisting of the track at the bridge ends [16.3].

Vertical displacements of the bridge are also reduced by limiting the *rotations φ at the end supports* (Fig. 16.14(a)). The discontinuity caused by the transition from ground (backfill, abutment) to structure can result in an increased likelihood of derailment, as well as causing passenger discomfort. The client's representative will define a rotation limit as a function of the basic speed of the railway line. Switzerland adopts values that are more severe than those given in the relevant European standards. In practice, however, explicit calculation of end rotations may not be necessary, because vertical deflection limits, and limits on the stresses in the rails, implicitly result in acceptable rotations. For a bridge comprising a succession of beams (Fig. 16.14(b)), the vertical deflection limit w_{lim} should be halved, so that the total rotation φ of the bridge deck above intermediate supports is acceptable.

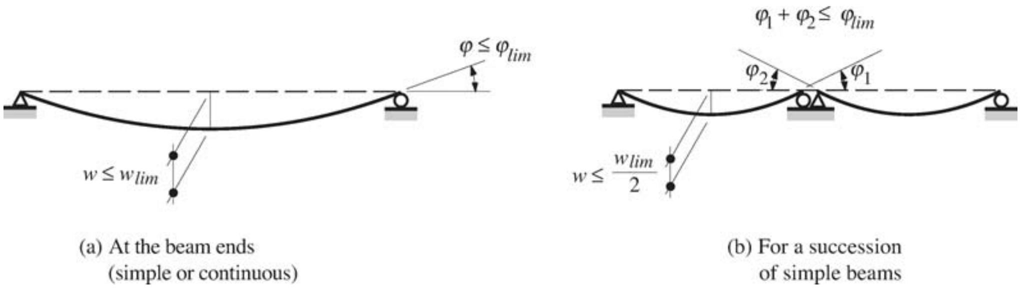


Fig. 16.14 Limits for deck rotation at the supports.

Limiting the *vertical deflection* of the slab relative to the abutments is also needed to ensure the continuity of the rails, by avoiding any abrupt changes. Such a relative deflection may occur, in particular, when the slab cantilevers out beyond the supports. The standard SIA 261 limits this deflection to 2 mm ($v \leq 160$ km/h) or 3 mm ($v > 160$ km/h). This deflection should be calculated with the frequent serviceability loads, increased by the dynamic amplification factor (§ 16.3.3) and with $\alpha = 1.00$ (§ 16.3.4).

The *stresses in the rails* must be limited in order to avoid them buckling laterally in summer and cracking in winter. When the rails are jointed (joined with fish plates), then the restraint forces are very small. Only the transfer of vertical and horizontal axle loads into the ballast must be guaranteed.

For the more usual case of continuous rails (welded), they must be able to resist higher levels of stress. The connection between rail and sleepers means that the longitudinal movements of the rails are directly transferred to the sleepers. However, the resistance offered by the ballast prevents free movement of the rails and, thus, they are subject to longitudinal forces. Additionally, the bridge represents a flexible foundation for the railway track and this leads to supplementary forces in the rails. Effectively, the interactions between the railway track and the bridge structure mean that deformations are partially transferred, respectively, to the bridge and the track.

In addition to the stresses due to the direct effects of the train wheels, stresses in the rails mainly come from three sources:

- *Temperature variation*: stresses develop over the whole length of the track, not just at the bridges. They are the result of relative displacement between the rails and the bridge structure when the rails are subject to a temperature increase. Lengthening of the structure is allowed by its expansion joints, but partially restrained by the rails. The latter act as a continuous elastic support against lengthening of the structure.

- *Braking or acceleration of trains:* stresses develop over the whole length of the track, not just at the bridges. They are the result of transferring the braking or acceleration forces from the rails to the support, which may be either the ground or the bridge slab. Depending on the stiffness of the ballast and the stiffness of the piers, these forces may propagate over great lengths of rail, causing significant relative displacements between the rails and the bridge structure.
- *Bending of the bridge structure* under vertical loads: stresses are due to the eccentricity of the rails relative to the neutral axis of the bridge cross section. The rails follow the vertical deflections of the bridge, and are, therefore, subject to a force as a result of the bending. The magnitude of the stresses in the rails increases as the stiffness of the connections between rails and slab increases, and/or the eccentricity of the rails with respect to the neutral axis of the bridge increases.

Swiss regulations [16.2] and EN 1991-2 [16.7] both propose a stress limit in service of 72 kN/mm^2 for compression and 92 kN/mm^2 for tension. These limits are valid if all the following conditions are satisfied:

- type UIC 60 rail with a tensile strength of 900 N/mm^2,
- ballast that is well compacted beneath the sleepers, and
- a bridge that is either straight or has a radius of curvature greater than 1500 m.

In order to calculate the stresses in the rails, the interaction between the structure and the track must be taken into account using an appropriate model (Fig. 16.15). The model must include the axial stiffness of the rails (EA), the axial and bending stiffnesses of the structure (EA and EI), the stiffness (k_1) of the connection between the railway track and the deck (sleepers and ballast), the stiffness (k_2) of the piers that are connected to the beam, as well as the presence of expansion joints in the structure and track. Frequent variable actions should be considered, which means primarily those due to traffic and temperature. The traffic loads need not be increased, either by the dynamic amplification factor or the coefficient α. Practically, this interaction calculation is not needed if the distance between the structure's expansion joints is less than a given value, for example that proposed in Paragraph 16.2.1.

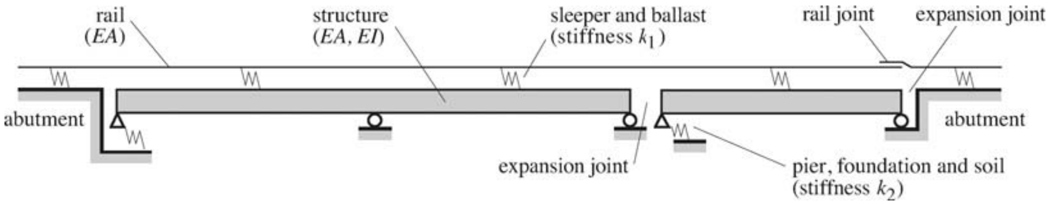


Fig. 16.15 Example of how the interaction between the structure, ballast, sleepers, and rails may be modelled when calculating the stresses in the rails.

EN 1991-2 [16.7] gives more detailed information on how interaction between the structure and the railway track should be taken into account, explaining the principles and giving guidance on models to use and parameters to take into consideration.

Appearance

The appearance of a bridge is directly influenced by deflections of the structure under quasi-permanent loads, including those deflections that result from the behaviour of the concrete slab over time, and any precamber. According to the standard SIA 260, this deflection should be less than $l/700$.

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17 Bridges for Pedestrians and Cyclists

Moveable footbridge rotating about a horizontal axis: "Gateshead Millennium Bridge" (UK).
Eng. Gifford and Partners, Southampton, UK.
Photo ICOM.

17.1 Introduction

This chapter considers the conceptual and structural design of bridges for pedestrians and cyclists, which are often simply called *footbridges*. Such bridges exist in a wide variety of structural forms, normally being characterised by the lightness of the structure and by architectural qualities that reflect the environment into which they are placed.

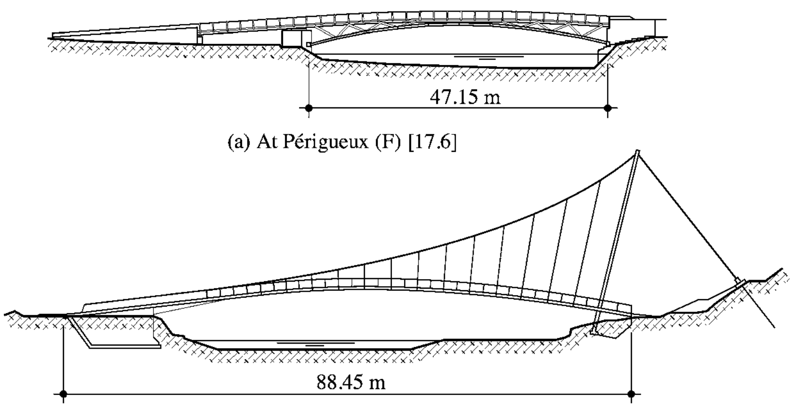
The wide variety of structural forms that may be used is one of the most differentiating aspects of footbridges. This diversity is possible because the imposed loads on such bridges are lower than those on road or railway bridges. Therefore, the self-weight of the structure is also lower and, consequently, dynamic phenomena have a big influence on the bridge conceptual design. Sensitivity to vibrations is becoming an even greater issue as the development of higher strength materials permits the use of smaller cross sections, or longer spans, in more slender structures. A reduction in the bending (vertical and horizontal) and torsional stiffnesses of the bridge, and, to a lesser extent, a reduction in mass, increase the probability of resonance occurring under the effects of pedestrians.

The structural design and the construction of footbridges follow the same general rules as those for road and railway bridges described elsewhere in this book. The purpose of this chapter is, therefore, to highlight and detail some specifics, and provide complementary guidance, related to the conceptual design (Sect. 17.2) and loads and actions to consider (Sect. 17.3) for footbridges. Dynamic phenomena are described in detail in Section 17.4, in particular the behaviour of footbridges and their verification under actions due to pedestrians.

17.2 Conceptual Design

17.2.1 Structure

Compared to road and railway bridges, the concept adopted for footbridges must satisfy different constraints in terms of planned use. Also, because the imposed loads are low compared with those associated with motorised traffic, the range of structural options for a given span is significantly greater. Figure 17.1 shows two examples of footbridges that are light and transparent, namely a very flat arch that is pinned at both ends (Fig. 17.1(a)), and a cable supported structure (Fig. 17.1(b)).



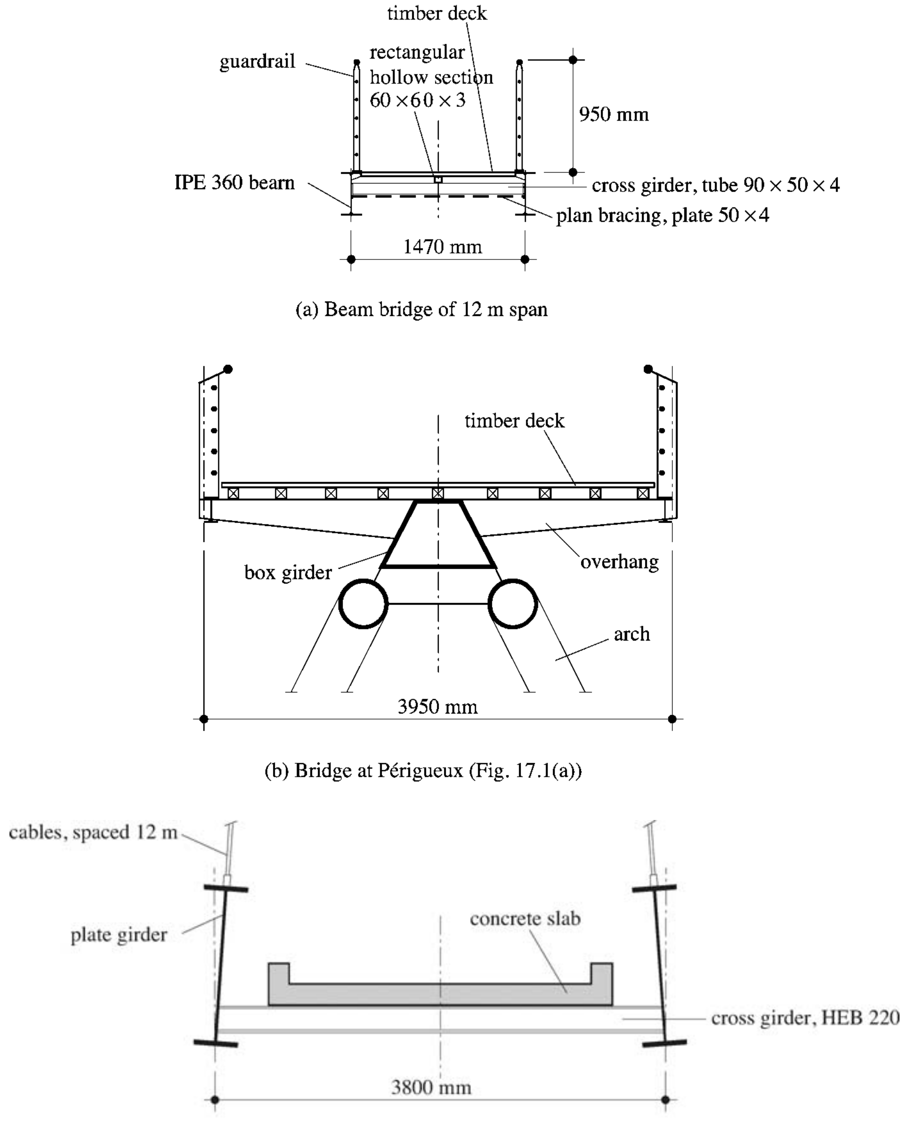
(a) At Périgueux (F) [17.6]

(b) Over the Rhône at Sierre (CH)

Fig. 17.1 Examples of footbridges.

The range of options for the load carrying structure, and, indeed, for the structural form, is vast. One of the main criteria that governs the final choice is integration of the bridge into the chosen site. Several engineers have published works on this subject, references [17.1] to [17.5] being of particular interest.

There are also numerous possibilities for the structural form of cross section, meaning that the designer can create an innovative solution. Some example cross sections are shown in Figure 17.2. Achieving a good appearance sometimes takes precedent over considerations of economy, for example by using



(a) Beam bridge of 12 m span

(b) Bridge at Périgueux (Fig. 17.1(a))

(c) Cable stayed bridge

Fig. 17.2 Typical cross sections of footbridges.

variable depth structural members, or forms that are elegant yet complicated. For short to medium spans, it is possible to use rolled sections for the main structural members (Fig. 17.2(a)), and they are more economical than fabricated girders. Rolled sections may be considered because of the low levels of loading imposed on footbridges. Fabricated beams may be used when spans are greater, for example a central box girder (17.2(b)). Plate girders may be used with a lower slab, so that the girders also act as guard rails, as shown in Figure 17.2(c). These have a dual purpose role, to act as hand rails for pedestrians to steady themselves, and as guards to stop the pedestrians or cyclists falling off the bridge.

17.2.2 Decking

The decking can be formed from a number of materials such as timber, concrete, composite materials or steel. The steel options include grids, anti-slip plates, and, indeed, orthotropic plates. Some types of decking can contribute to the overall resistance of the bridge, while others merely serve as the surface for foot and cycle traffic, and must not be connected to the rest of the structure. Materials with a smooth surface, such as timber, may become slippery when the weather is frosty. The choice of material depends on a number of criteria, of which the most important are:

- the planned use (pedestrians, cyclists, light service vehicles),
- the planned service life,
- aesthetic and architectural considerations,
- the presence of water (covered bridge or one with the deck exposed to the weather),
- whether the decking is structural or not,
- the degree of local resistance needed against imposed loads (punching and bending),
- the roughness and adherence needed,
- the mass of the structure,
- the need to dissipate and damp vibrations,
- the cost.

Timber

Timber has been used as decking material (planks) for as long as bridges have existed. It is particularly suited to covered footbridges. If a bridge is not covered, then efficient water evacuation must be assured by leaving a gap of several millimetres between planks so that water can drain through the deck and drying is rapid. Some tropical species of hardwood are sufficiently durable to be exposed to the weather, for example Ipe, Iroko, and Doussie.

If light service vehicles are to be used for bridge maintenance, then resistance against punching (high concentrated loads) must be carefully considered.

Concrete

Decking formed from reinforced concrete is distinguished mainly by its relatively high self-weight, high mechanical resistance, and good durability. To improve user comfort and durability, concrete slabs are sometimes covered with a bituminous layer. If the concrete slab is structurally connected to the steelwork, it can help to increase the bending resistance of a footbridge, may serve as plan bracing, and, in some cases, improve the dynamic behaviour of the bridge.

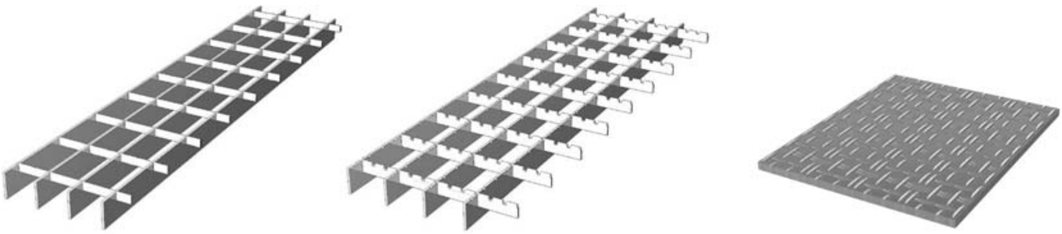
The use of prefabricated concrete elements to form the decking of a footbridge is particularly interesting, for the following reasons:

- the small width of footbridges often allows prefabricated elements to be transported without difficulty, due to their reduced dimensions,
- because of the low self-weight per linear metre of decking, prefabricated elements of considerable length can be placed using commonly available cranes,
- prefabrication allows the duration of the construction works to be shortened.

Steel Grid and Anti-slip Plates

The use of these steel solutions, which are characterised by their low self weight, is particularly appropriate for the decking of service footbridges for which aesthetic considerations are of little relevance. Steel grids (Fig. 17.3(a)) may act as structural elements to support concentrated loads for spans of up to around 1.5 m, while anti-slip plates (Fig. 17.3(b)) can span up to around 1.0 m.

The transparency of a grid may not provide an acceptable feeling of safety for users, particularly when there is a deep drop beneath the bridge! Therefore, such a solution is most commonly used for service footbridges. On the other hand, the use of a grid does allow snow to pass readily through the openings, thereby avoiding the need for snow removal.



(a) Steel grids (b) Anti-slip plate

Fig. 17.3 Elements of a steel deck.

Orthotropic Decks

Steel orthotropic decks (Sect 6.7) are used for long span footbridges that require a deck, which is both lightweight and can contribute to the overall resistance of the bridge. Unfortunately, orthotropic decks are also expensive compared to the alternatives. They must be covered by a bituminous (or composite) surfacing layer in order to provide users with a surface that is sufficiently rough.

Composite Materials

Composite materials have recently provided yet another possibility for the decking of footbridges. They possess good qualities in terms of adherence and resistance to abrasion. Forming the joints associated with a composite deck is currently one of the main issues with the use of this material.

Construction Details and Joints

Joints between the decking and the load carrying structure must be configured in a way to avoid standing water, as this could lead to rapid deterioration, particularly of timber and steel decks. The joints should also be configured so that if and when deterioration does occur, they facilitate replacement of the deck.

Unless the decking has been conceived and designed to contribute to the overall resistance of the bridge, it is important that the joints allow relative slip between the structure and decking. If this is not possible, the decking will experience some of the forces and moments present in the structure, such as those due to bending or temperature effects.

17.3 Loads and Actions

The loads and actions on footbridges are in nature similar to those that act on road bridges. Chapter 10 provides a description of the different types of load and action, such as temperature, wind, seismic actions, or creep and prestressing forces. This section only considers self-weight and service loads.

17.3.1 Self-weight

The self-weight of a footbridge is normally low due to the relatively small loads that act on it. In comparison with a road bridge, the ratio between traffic loading and self-weight of a footbridge is higher. For a footbridge this ratio may exceed 1.5, while it would be more like 0.5 for a road bridge of similar span.

Unlike road and railway bridges, it is difficult to establish an empirical formula for estimating the self-weight as a function of the service load or span, due to the variety of types of structure, and structural forms, used for footbridges. Therefore, self-weight should be estimated based on sketches, or experience of similar bridges already in use, and then checked and modified accordingly following preliminary sizing and prior to structural design.

17.3.2 Service Loads

Bridges for pedestrians and cyclists are designed, for example according to the standard SIA 261, for a distributed service load with a characteristic value of 4.0 kN/m^2. The load factor γ_q is taken as 1.5, and the reduction coefficients are taken as $\psi_0 = \psi_1 = 0.4$ and $\psi_2 = 0.0$. There is no default dynamic amplification factor to apply to the loads, as an explicit check of dynamic behaviour is required for footbridges (Sect. 17.4).

According to the standard SIA 261, the local effects of a light service vehicle are allowed for by considering a concentrated load Q_k of 10 kN that is, in principle, not combined with the distributed load q_k. The reduction coefficients are taken as $\psi_i = 0.0$ ($i = 0$ to 2). Clearly this nominal load should be increased if it is possible for heavier vehicles to access the footbridge. Design options should be taken to avoid accidental access of road vehicles, such as creating a bank or some other form of adequately strong obstacle.

17.4 Dynamic Behaviour

17.4.1 Introduction

Unlike road or railway bridges, dynamic effects often govern conceptual and structural design of footbridges. For such structures which are light and normally slender, serviceability requirements must be given careful consideration. This is particularly the case as far as vibrations, and their effect on user comfort, are concerned. Pedestrians and wind forces both act on the structure in a dynamic way, and these lead to vibrations of the structure. The amplitude and frequency of these vibrations depend on a number of parameters, in particular:

- the stiffness of the structure, including secondary and non-structural members, about vertical and horizontal (bending) and longitudinal (torsion) axes,

- the mass of the structure,
- the damping of the structure, which is strongly influenced by the behaviour of the materials used and the joints (notably the joints between the deck and structure), and
- the frequency of the actions due to pedestrians or wind.

Vibrations of the structure may:

- Adversely affect the *comfort of users*: vibrations (which result in movements and accelerations) create a negative feeling in users. A footbridge that vibrates may be perceived as unacceptable, or even dangerous (without really being so), by users. This is a question of serviceability.
- Affect the *fatigue resistance*: when the service loads (light wind, normal use by pedestrians) result in constant vibrations, then fatigue phenomena may occur and lead to damage of the structure.
- Cause *resonance*: in exceptional circumstances, if the frequency of the actions is very close to some of the natural frequencies of the bridge, if the excitation is continuous, and if the bridge has weak damping, then resonance can occur. This is a question of structural safety.

Vibrations of the structure that have a negative effect on users, creating a sense of insecurity, are the main cause of temporary closures to allow structural modifications of footbridges. Such feelings of insecurity normally occur well before the vibrations are such that they could cause structural damage or lead to resonance.

Therefore, it is essential to carefully check the dynamic behaviour of footbridges in order to assure both their serviceability and structural safety. Serviceability is normally guaranteed by limiting the amplitude and frequency of the vibrations that are felt by the users.

17.4.2 Conceptual Design and Corrective Measures

It is important to include consideration of the dynamic behaviour of a footbridge as part of the project starting with the conceptual design, so that retrospective – and often complex and expensive – modifications are not needed to improve a bridge that would be unsatisfactory. The principal parameters that influence dynamic behaviour are considered below. How these should be taken into account in design calculations is considered in Paragraph 17.4.3.

Bending Stiffness and Torsional Stiffness

Stiffness is here defined as the ratio between the force F applied to a structure and the deformation w that takes place due to the force F. Stiffness has a direct impact on the natural frequencies of a bridge: the stiffer the structure, the higher the natural frequencies. The size of the structural members, the materials used, the span, and the presence of bracing all affect the stiffness of a bridge. Also, the stiffness can be increased by the presence of non-structural elements such as the decking or guardrails. A calculation of natural frequencies that ignores the influence of the non-structural elements will result in an underestimation of the real values.

Mass

Mass also has a direct effect on natural frequency: the higher the mass of a bridge, the lower its natural frequencies. The choice of material used for the decking of a footbridge (a lightweight deck formed from composite material or timber, or a heavyweight deck formed from concrete) affects the overall mass, and, therefore, the natural frequencies.

It is possible to adjust the mass after construction if the dynamic behaviour of a footbridge is found to be unsatisfactory, for example by adding mass to the deck. However, such a solution is not normally satisfactory because the mass must be increased four fold in order to halve the natural frequencies. Increasing the mass also requires strengthening of the structure, which normally increases its stiffness and so has exactly the opposite effect on the natural frequencies to what is needed.

Damping

Damping diminishes the vibrations of a structure. The greater the damping the more rapidly the structure returns to a state of rest. A high level of damping allows unacceptable vibrations to be avoided, in other words, to limit the magnitude of movements and to avoid resonance when a structure is subjected to a cyclic action.

Damping depends in particular on:

- the choice of materials: a concrete slab damps the structure better than a steel one, even more so when the concrete is cracked,
- the configuration of the joints: bolted joints dissipate more energy than welded ones,
- the types of bearings and foundations: bearings that allow movement dissipate energy, as do movements at the structure to soil interface.

Therefore, a distinction is made between the damping due to:

- the materials: internal dissipation of energy,
- the structure: dissipation of energy for example by friction in the joints and the bearings, and due to the non-structural elements,
- the bridge as a whole: dissipation of energy, for example, due to the soil structure interaction.

Dissipating or Mass Dampers

The use of dampers can improve the dynamic behaviour of a footbridge. Dissipating dampers primarily affect the amplitude of vibrations. The mechanical energy of the bridge in motion is transformed into heat, which reduces the frequency and amplitude of the vibrations. In practice, such dampers may be viscous dampers, of the type used in road vehicles. How to reflect the presence of these dampers in the bridge design is considered in [17.7].

Mass dampers, which are also known as *tuned mass dampers*, operate in a completely different way. By coupling the bridge structure to a mass-spring-damper system (for example, placed under the deck at mid-span), the bridge vibrations are disturbed by the response of the suspended mass, if it is moving in the opposite direction to the bridge. If the mass and the spring stiffness are chosen carefully, then the response can be such that the amplitude of the bridge vibrations is significantly reduced, to the extent that they no longer trouble the bridge users. In terms of physics, the suspended mass changes the natural frequency of the bridge-mass system so that it becomes sufficiently far from the excitation frequency.

It is worth noting that a mass damper can only be tuned to attenuate one vibration frequency. If a bridge has several natural frequencies within the range that may be excited by users, then several dampers, possibly acting in different vibration directions (vertical, horizontal, torsional) will be needed in order to ensure satisfactory dynamic behaviour. Mass dampers are all the more effective when the damping ratio ζ of the structure is low ($< 5\%$). This means they are more effective for steel bridges than those made from concrete. They are the most common type of damper used to reduce the vibrations of footbridges.

It is beneficial to consider, early on in the design process, where dampers could be placed for footbridges that are particularly light and slender and with natural frequencies that are likely to cause vibration problems during service.

17.4.3 Dynamic Analysis

Undertaking a dynamic analysis of a footbridge normally comprises determining its natural frequencies and, in some cases, calculating the deflections and accelerations that will be caused by the passage of pedestrians. Various calculation methods are available for determining the natural frequencies. The most basic, which are based on a simple analytical model, are generally satisfactory for preliminary sizing or for so-called sanity checking of software output. If a more refined approach is required, then numerical modelling is an option, for example using finite elements. Despite the ease of use of such software, the designer must always pay careful attention to the results that are generated to ensure they are reasonable.

Figure 17.4 shows a single degree of freedom system, comprising a mass, a spring, and a damper. Such a model allows the response (first vibration mode) of a structure that exhibits linear elastic behaviour to be calculated. The structure is modelled with a mass M, has a stiffness k and damping c, and is subject to a force that varies with time $F(t)$. The stiffness k, which is also known as the spring constant, is defined as the ratio between the static force F and the deflection w due to this force. The response of the structure may be a deflection $w(t)$ or one of its derivatives (speed, acceleration).

Described below is how to calculate the stiffness k, the damping ratio ζ (which depends on the value of damping c), the natural frequencies f, and finally the deflections $w(t)$ and accelerations $a(t)$ of a footbridge subject to dynamic loads $F(t)$.

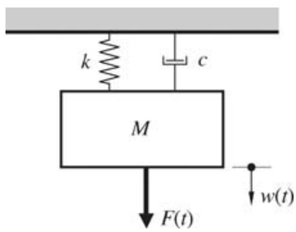


Fig. 17.4 Mass, spring and damper model – single degree of freedom.

Stiffness

The stiffness k of a bridge depends on several parameters, as explained in Paragraph 17.4.2. To model a footbridge, it is particularly important to take into account that:

- concrete deck elements may crack, and this may be taken into account by using a reduced elastic modulus for the concrete,
- the duration of variable loads (pedestrians, wind) is normally short, so there is no need to take into account creep of the concrete elements,
- the influence of the non-structural elements may need to be taken into account, as they can significantly increase the stiffness of the structure.

These effects, in particular the contribution of the non-structural elements, cannot be estimated with any great precision. As a consequence the calculated stiffness will only ever be an approximation of the real stiffness. This should be remembered when natural frequencies are being checked (§ 17.4.5), as some engineering judgement may be needed.

Damping

Damping is often defined as the logarithmic decrease δ of the amplitude of vibration (Eq. (17.1)). In this equation i refers to a given cycle of vibration of amplitude w_i, of a single degree of freedom system that vibrates freely: $F(t) = 0$, $w(t = 0) = w_0$

$$\delta = \ln\left(\frac{w_i}{w_{i+1}}\right) \tag{17.1}$$

The damping depends on the amplitude of the vibration. It can increase up to a value that is four times the damping value for small amplitudes. The lower bound of damping corresponds to the damping of the superstructure, while the upper bound takes into account that part of the damping that is due to the infrastructure, which can be significant.

Equation (17.2) defines the *damping ratio* ζ, which is a value that is often used to characterise the damping of a structure:

$$\zeta = \frac{\delta}{2\pi} \tag{17.2}$$

The damping ratio ζ is often expressed as a percentage of the *critical damping* (the damping for which return to the position of equilibrium is the most rapid, but without oscillation). For the single degree of freedom system shown in Figure 17.4, the damping ratio is expressed as:

$$\zeta = \frac{c}{2M\omega} \tag{17.3}$$

c : damping constant [Ns/m]
M : mass in [kg]
ω : natural circular frequency $\omega = \sqrt{k/M}$
k : stiffness or spring constant [N/m]

Table 17.5, which is taken from [17.7], gives several typical values for the damping ratio of footbridges as a function of the materials used. More precise values for damping can be found in [17.8]. The values given in Table 17.5 correspond to minimum values that may be used in design calculations for footbridges.

Table 17.5 Typical minimum damping ratios of footbridges [17.7].

Type of Footbridge	ζ [%]
Steel	0.3
Steel-concrete composite	0.5
Concrete	0.7
Timber	1.5

Vibration Frequencies

A system possesses as many natural frequencies, and, therefore, modes of vibration, as its masses possess degrees of freedom. A structure that has a distributed mass possesses an infinite number of degrees of freedom and, therefore, an infinite number of natural frequencies. The first mode, which is associated with the lowest frequency, is called the *fundamental mode*, and the corresponding frequency is called the *fundamental frequency*. The modes that follow are called the *second mode*, the *third mode*, and so on. This is also the case for naming of the natural frequencies. Normally, only the first vibration frequencies of a structure are of any practical importance.

Figure 17.6 shows values of the first, or fundamental, frequency for 67 footbridges as a function of their span. A curve drawn through these values relates the frequency f to the span l. It can be seen that for footbridges having a span in excess of 30 m, there is a significant probability of the structure being set in motion by pedestrians with an average step frequency of 2.0 Hz.

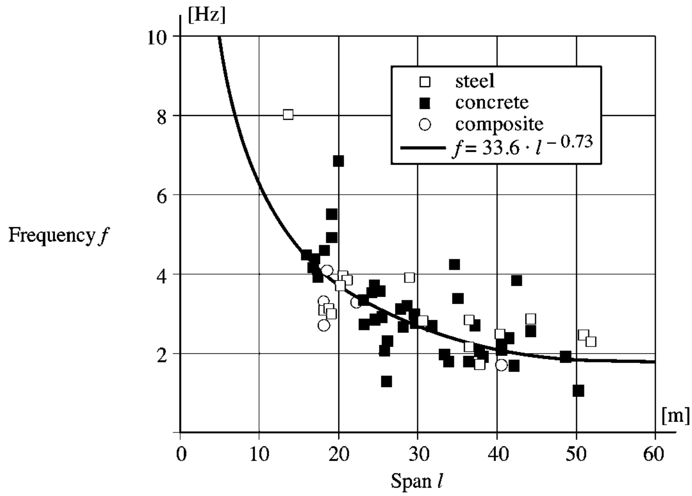


Fig. 17.6 First natural frequency of footbridges as a function of span [17.9].

The fundamental frequency f_0 [Hz] and the period T_0 [s] of a *single degree of freedom* system (Fig. 17.4) are calculated as follows:

$$f_0 = \frac{\omega_0}{2\pi} \tag{17.4}$$

$$T_0 = \frac{1}{f_0} \tag{17.5}$$

with ω_0, the circular frequency that relates the mass M [kg] to the stiffness k [N/m], given by the following equation:

$$\omega_0 = \sqrt{\frac{k}{M}} \tag{17.6}$$

For systems with *several degrees of freedom*, calculating the natural frequencies and vibration modes requires the use of software. With that said, simple methods do exist that provide a reasonable estimate of the natural frequencies of the type of continuous beam found in bridges. By transforming a continuous structural form into an equivalent form with only one degree of freedom, it is possible to use a simple calculation to determine the fundamental frequency. When the bridge comprises simple beams with differing end supports, integrating the differential equations that represent the movement of the structure [17.7] allows the generalised mass M^* [kg] to be calculated, as well as the generalised stiffness k^* [N/m] that is used to calculate the circular frequency using equation (17.6).

In the case of a simple beam of span l with a stiffness EI and distributed mass m, the generalised mass M^* and the generalised stiffness k^* may be calculated using Equations (17.7) and (17.8):

$$M^* = \phi_m \cdot ml \tag{17.7}$$

$$k^* = \phi_k \cdot \frac{EI}{l^3} \tag{17.8}$$

ϕ_m : mass coefficient, see Table 17.7
ϕ_k : stiffness coefficient, see Table 17.7

Table 17.7 presents values for the coefficients ϕ_m and ϕ_k for different structural forms and types of loading. These coefficients are used to represent a system with distributed mass and stiffness as an equivalent single degree of freedom system (Fig. 17.4). For cases that are not covered by Table 17.7, it is necessary to resolve the equations presented in [17.9]. A particular example where this approach is needed is for beams that are not of uniform cross section.

Table 17.7 Equivalence coefficients to allow a system with distributed mass and rigidity to be represented by a model with a single degree of freedom [17.9].

Structural Form	Load	Mass Coefficient ϕ_m		Stiffness Coefficient ϕ_k
		Concentrated mass at mid-span	Uniformly distributed mass	
Simple beam	Distributed	–	0.5	48.7
	Concentrated at mid-span	1.0	0.5	48.7
Beam with pinned and fixed supports	Distributed	–	0.479	113.9
	Concentrated	1.0	0.479	113.9
Beam fixed at both ends	Distributed	–	0.396	198.5
	Concentrated	1.0	0.396	198.5

For a continuous beam on three or four supports (spans ηl, l and ξl), Figure 17.8 allows its fundamental frequency f to be derived from the fundamental frequency f_0 of a simple beam of span l.

The vibration frequencies are influenced by the damping of a structure. For steel footbridges this influence may be neglected, given the low damping ratio (Table 17.5). However, the fundamental vibration frequency may be significantly modified by the presence of pedestrians, if their mass is big relative to the self weight of the structure. When this is the case, the mass of the pedestrians should be allowed for in the value of M^*, or in the equivalent value of distributed mass m.

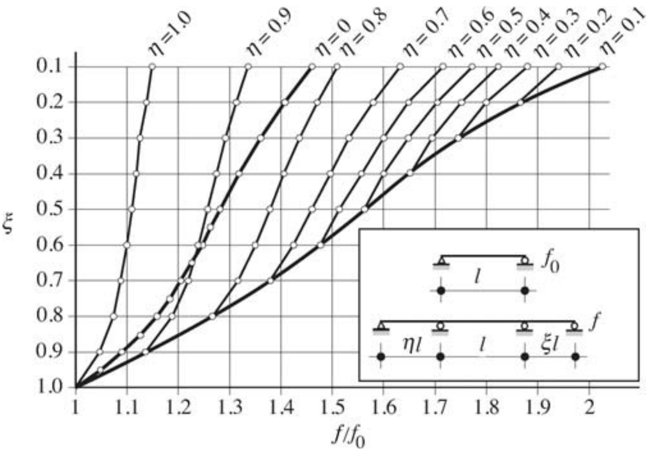


Fig. 17.8 Fundamental frequency for two and three span beams [17.10].

Acceleration and Amplitude of Vibration

The accelerations to which pedestrians are subjected depend on a number of parameters. In particular, the acceleration is very much dependent on the excitation force that acts on the bridge. This means that acceleration is a characteristic of the structure and the loading together, unlike the natural frequency, which is a characteristic of the structure alone. The same is true for the amplitude of vibrations that are caused by variable imposed loads.

Based on an analytical model of a system with distributed mass and constant stiffness, the amplitude of the vibrations w_{max} and the accelerations a_{max} may be determined using Equations (17.9) and (17.10). These equations are taken from [17.7] and are applicable when the exciting force is sinusoidal of the form $F_0 \cdot \sin(\omega t)$. These equations describe a stationary phenomenon, and represent an upper bound for a transient phenomenon.

$$w_{\max} = \frac{1}{\sqrt{(1-(\omega/\omega_0)^2)^2 + 4\zeta^2(\omega/\omega_0)^2}} \cdot \frac{F_0}{k^*} = \phi_{dyn} \cdot \frac{F_0}{k^*} \tag{17.9}$$

$$|a_{\max}| = \omega^2 \cdot w_{\max} = \omega^2 \phi_{dyn} \cdot \frac{F_0}{k^*} \tag{17.10}$$

ϕ_{dyn}	:	dynamic amplification factor
ω	:	circular frequency of the exciting force
ω_0	:	circular frequency, or self pulsation, of the structure
ζ	:	damping ratio

The ratio F_0/k^* represents the static displacement (at zero frequency) of the mass of the system due to the load F_0. The dynamic amplification factor ϕ_{dyn} is a function of the ratio ω/ω_0 between the circular frequencies (or pulses) of the sinusoidal force and the structure, as well as the damping ratio ζ. For weak damping, as is the case with steel footbridges, the dynamic amplification is very large if the frequency of the exciting force is very close to the natural frequency of the structure. The structure resonates when $\omega \cong \omega_0$, and, in this case, the dynamic amplification factor reaches its maximum value:

$$\phi_{dyn,\,max} = \frac{1}{2\zeta} \tag{17.11}$$

Actions Due to a Single Pedestrian

Vertical and horizontal exciting forces act on a footbridge as a result of the passage of a pedestrian. Typical vertical frequencies for a walking pedestrian vary between 1.6 Hz and 2.4 Hz, with an average value of about 2.0 Hz. When a pedestrian runs, the frequencies vary between 2.0 Hz and 3.5 Hz. Horizontally, the transverse exciting frequency due to a pedestrian is half the vertical value.

The force $F(t)$ that is applied to a footbridge by a moving pedestrian comprises a static component (his weight) and a dynamic component that is the sum of harmonic functions with frequencies that are multiples of the frequency of the action. The vertical force is expressed as (broken down into a Fourier series):

$$F(t) = G_0 + G_1 \sin(2\pi f_p t) + \sum_{i=2}^{n} G_i \sin(2i\pi f_p t - \varphi_i) \tag{17.12}$$

G_0	:	static force corresponding to the weight of the pedestrian, normally taken as 700 N
G_1	:	amplitude (force) of the first harmonic. For a frequency f_p close to 2 Hz, $G_1 = 0.4$ and $G_0 = 280$ N (with 0.4 = coefficient of the Fourier series for $F(t)$ limited to the first three terms)
G_i	:	amplitude (force) of the i^{th} harmonic
f_p	:	fundamental frequency of the force applied by the pedestrian (walking or running)
φ_i	:	phase difference between the i^{th} harmonic and the first one
n	:	number of harmonics considered

Normally only the first three harmonics are considered. In practice, footbridges are excited vertically by the first harmonic: $f_p = f_0$. However, they may also vibrate significantly due to higher harmonics, such as the second, $2f_p = f_0$.

The horizontal force due to a pedestrian, which acts transversally on the structure, has no static component. It is expressed by the following equation, considering only the first harmonic:

$$F(t) = 0.05 G_0 \sin\left(2\pi\left(\frac{f_p}{2}\right)t\right) = G_1 \sin\left(2\pi\left(\frac{f_p}{2}\right)t\right) \tag{17.13}$$

The horizontal force due to a pedestrian, which acts longitudinally on the structure, is expressed by:

$$F(t) = 0.2 G_0 \sin(2\pi f_p t) = G_1 \sin(2\pi f_p t) \tag{17.14}$$

Until recently the dynamic analysis and design of footbridges was based on consideration of the passage of a lone pedestrian. However, this approach has its limitations, in particular it does not cover bridges in urban locations that are subject to groups of pedestrians, which may be of varying density.

Actions Due to Several Pedestrians

When a bridge is subject to several pedestrians simultaneously, the intensity of the loading, and, therefore, the response of the structure, increases relative to those due to a single pedestrian. The actions due to several pedestrians must take into account the random nature, and lack of synchronicity, of movements. The walking frequencies and weights of the pedestrians may be described by a probability distribution, while the phase shift, which corresponds to the time at which individual pedestrians walk onto the bridge, is an uncertainty. Also, a pedestrian has a maximum effect when at mid-span. When the pedestrian is near to a support, the excitation of the structure is less.

These different variables mean that the exciting forces due to several pedestrians are cumulative in some cases, and offset each other in other cases. Because the forces due to pedestrians are not of a sinusoidal form, and act at different places along the length of a bridge, it is very difficult to obtain a solution by analytical integration. It is necessary, therefore, to consider either numerical methods, or simplified models. Different models have been developed to describe the actions of a group of pedestrians on a footbridge. By way of example, the results of a model contained in guidance from SETRA [17.11] are considered below.

Based on numerical simulations, the model allows an equivalent number of pedestrians N_{eq} to be defined. This group is assumed to be perfectly synchronised in phase and frequency. Table 17.9 defines N_{eq} as a function of the density of pedestrian traffic d and the damping ratio ζ. The total number of pedestrians N is equal to the density multiplied by the deck surface area of the footbridge.

Table 17.9 Equivalent number of synchronised pedestrians N_{eq} [17.11].

Pedestrian Traffic	Footbridge Class	Density d [pedestrian/m^2]	N_{eq}
Small density	III	0.5	$10.8\sqrt{N \cdot \zeta}$
Dense	II	0.8	
Very dense	I	1.0	$1.85\sqrt{N}$

The equivalent number of pedestrians should be distributed over the bridge and, to calculate the maximum acceleration, a force should be applied that has the same sign as that of the modal deformation being considered, and the same frequency as the corresponding natural frequency of the structure (normally the first mode shape and first natural frequency). For the model that has been developed [17.11], the load to be taken into account may be multiplied by a reduction coefficient ψ, which allows for the fact that resonance of the structure becomes less likely as the range of natural frequencies of the pedestrians becomes remote from the natural frequency f of the structure. Figure 17.10 defines this reduction coefficient ψ for walking pedestrians, and both vertical and horizontal vibrations.

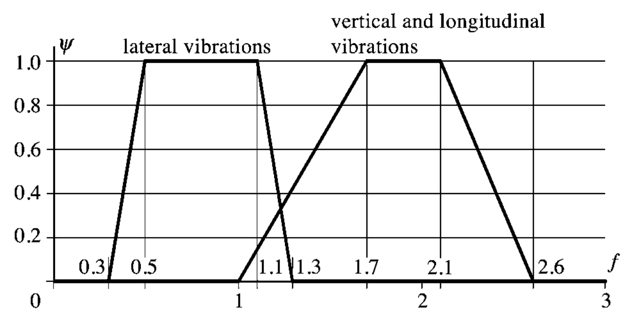


Fig. 17.10 Reduction factor ψ for the dynamic action of pedestrians [17.11].

For example, according to [17.11] for a footbridge of length l supporting pedestrian traffic that is assumed to be dense, the line load [N/m] from the pedestrians to consider in the dynamic analysis for vertical vibration is as follows:

$$F(t) = \psi \cdot 280 \cdot \frac{N_{eq}}{l} \cdot \cos(2\pi f_0 t) \tag{17.15}$$

This force should be applied along the length of the beam in either a positive or negative sense depending on the deflection (same sense as the deflection) for the mode being analysed.

It is worth adding that when a crowd of pedestrians moves across a bridge, it is possible that they gradually synchronise their walking frequency, and get into phase with the movement of the structure. This *forced synchronisation* mainly affects the lateral vibration of footbridges. A pedestrian who is sensitive to lateral movements (even when the amplitude is as low as 2 or 3 mm) instinctively tries to compensate for his perceived loss of equilibrium by adjusting the frequency of his movements to that of the bridge movements. By doing this he contributes directly to making the bridge resonate. This phenomenon is more noticeable when more pedestrians are present. Two recent examples, where this has been a problem, are the Millennium Bridge in London and the Solferino Bridge in Paris, both of which had to be temporarily closed.

For this forced synchronisation, studies [17.12] have shown that one can estimate that the synchronisation of a dense crowd of walking pedestrians at the frequency of the transverse mode corresponds to the effect of $0.2N$ pedestrians walking in a synchronised way, where N is the number of pedestrians present on the bridge.

17.4.4 Design of a Mass Damper

When a footbridge has unsatisfactory dynamic behaviour, despite having been designed taking dynamics into account, adding dampers to the structure is a possible solution. Mass dampers are often used. Figure 17.11 shows a schematic model of a footbridge (mass M) with an added mass damper (mass M_a). This model of two masses is a damped system with two degrees of freedom. If the mass M_a and the spring stiffness k_a are chosen carefully, then the response of the combined bridge and damper may be such that the amplitude of vibrations tends to zero. Physically, the suspended mass modifies the natural frequency of the combination so that it moves sufficiently far from the excitation frequency. This has the effect of reducing the dynamic amplification factor ϕ_{dyn} in Equations (17.9) and (17.10).

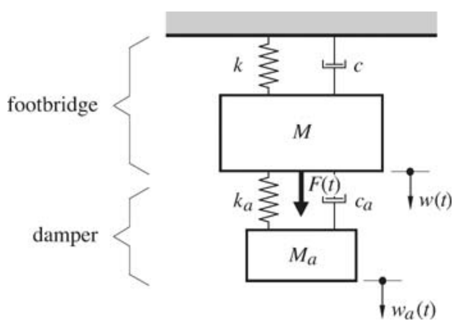
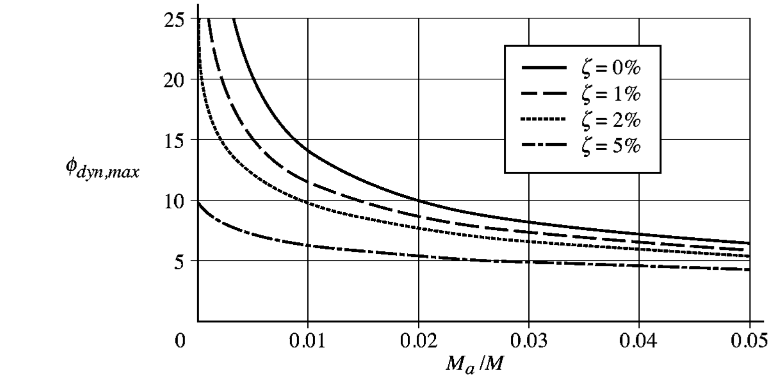
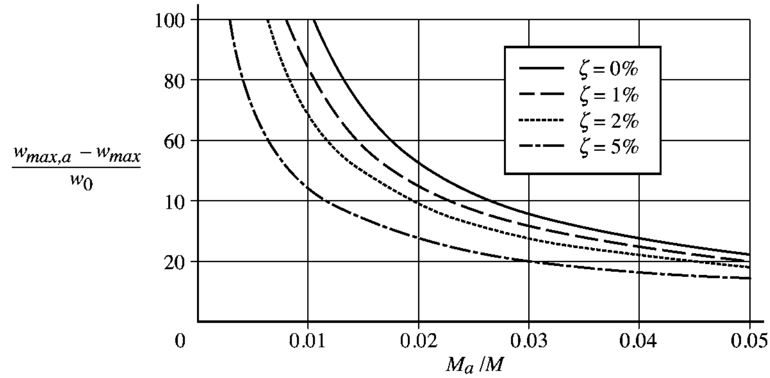


Fig. 17.11 Mass, spring, and damper model – two degrees of freedom.



(a) Dynamic amplification of the damped structure



(b) Maximum relative displacement between the damper and the structure

Fig. 17.12 Displacements of a damped footbridge and the damper itself.

When designing a mass damper, the most important choice concerns the ratio between the masses M_a/M. This ratio has a direct influence on the response of the structure and the movement of the damper relative to the footbridge. The ratio may be chosen using the curves shown in Figure 17.12, which are taken from [17.13].

Figure 17.12(a) shows the maximum dynamic amplification factor $\phi_{dyn,max}$ of a structure fitted with a damper, as a function of the mass ratio M_a/M and the damping ratio ζ of the structure alone. It can be seen that for structures with a damping ratio greater than 5%, the addition of a mass damper has hardly any effect. It can also be seen that with a mass ratio M_a/M greater than around 0.02, the reduction in maximum amplification factor is small when compared with the increase in mass of the damper, even for small values of damping ratio.

Figure 17.12(b) shows the relative displacement between the damper $w_{max,a}$ and the damped structure w_{max}, normalised with respect to the static displacement w_0 (due to G_1, Eq. 17.12), as a function of the mass ratio M_a/M and damping ratio ζ of the structure alone. When space below the deck is limited, the magnitude of relative displacement can be a determining parameter for the ratio M_a/M.

Once the mass ratio is chosen, Figure 17.13 allows the optimum frequency f_{opt} of the damper to be chosen, in order to dampen the vibration mode with frequency f, as a function of the damping ratio ζ of the structure alone.

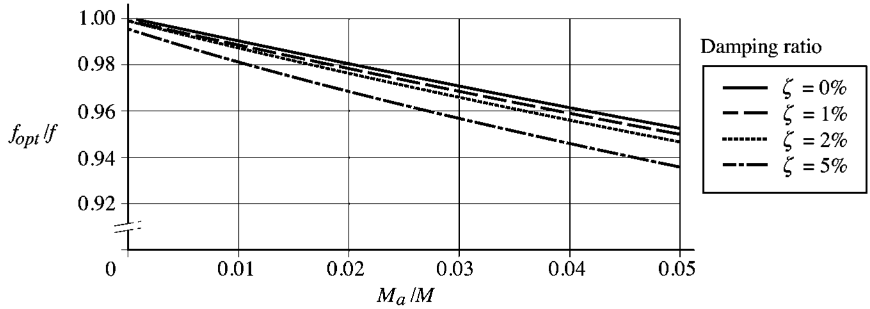


Fig. 17.13 Optimum frequency for a mass damper.

For *footbridges without damping* ($\zeta = 0$) the value of optimum frequency f_{opt} and optimum damping ratio ζ_{opt} of a damper may be calculated using Equations (17.16) and (17.17). These optimal values can also be used to design dampers for steel footbridges that have a very low damping ratio:

$$f_{opt} = \frac{f}{1 + M_a/M} \tag{17.16}$$

$$\zeta_{opt} = \sqrt{\frac{3(M_a/M)}{8(1 + M_a/M)^3}} \tag{17.17}$$

Mass dampers are effective only when they are exactly tuned to the real natural frequency of the footbridge for the mode of vibration that is to be damped. On the other hand, a difference between the damping ratio ζ_a of the damper, and the optimum damping ζ_{opt} that it should have, is less important.

As a **numerical application** of the calculation of the mass of a mass damper, consider the footbridge described at the end of this chapter (§ 17.4.6). The mass m of the bridge is 1500 kg/m, the span l is 40 m, and the mass ratio M_a/M is chosen to be 0.02. Taking into account the generalised mass M^* (Table 17.7), one finds:

- $M_a = 0.02M^* = 0.02\,(0.5ml) = 0.02 \cdot 0.5 \cdot 1500 \cdot 40 = 600$ kg.
- The optimum frequency of the damper should be $f_{opt} = \frac{f}{1 + 0.02} = 0.98f$ where f is the frequency of the mode to be damped.
- So the optimum damping ratio should be $\zeta_{opt} = \sqrt{\frac{3 \cdot 0.02}{8(1 + 0.02)^3}} = 0.084$.

17.4.5 Verifications and Limiting Values

Methodology

Comparing the natural frequencies of a footbridge with the frequencies of the exciting forces from pedestrians allows a rapid estimation of the sensitivity of the bridge to dynamic actions. If comparison of frequencies has not satisfied the designer, checking dynamic behaviour, by considering accelerations, is carried out in a second step.

- *Calculation of natural frequencies*. Normally, only the first modes of vibration are of practical importance. If these frequencies are outside the range to be avoided, then the dynamic analysis need go no further, as user comfort may be assumed to be satisfactory.

- *Calculation of accelerations and deflections*. When the natural frequencies are within the range of frequencies associated with pedestrian actions, the dynamic analysis must be taken further by determining the accelerations or deflections of the structure. If these accelerations (or deflections) are below the limiting values, then the structure may be assumed to have satisfactory dynamic behaviour. If this is not the case, then either the mass or stiffness of the structure must be modified, or dampers added (§ 17.4.2).

Normally, dynamic analysis is carried out using software that can take into account the dynamic actions due to pedestrians (for example, as specified by Equation (17.15)). As an alternative to checking accelerations, it is possible to compare the deflections of the structure with deflections that are acceptable to the users in order to guarantee their comfort. This approach is also discussed in more detail below.

Natural Frequencies

The relevant codes and standards identify a range of frequencies that the natural frequencies of a structure should avoid. This range corresponds to the frequencies associated with the exciting forces from pedestrians. When the natural frequency of a structure is within this range, its vibrations will be amplified. User comfort may be reduced, and the structural safety of the footbridge put at risk.

For vertical vibrations the average frequency for a walking pedestrian is 2.0 Hz, while that of somebody running varies between 2.0 and 4.0 Hz. This is why the standard SIA 260 recommends avoiding all natural frequencies for *vertical vibrations* between 1.6 and 4.5 Hz. If the natural frequencies of a bridge fall within this range, then SIA 260 says that a dynamic analysis must be carried out.

It is also important to check the frequencies associated with the transverse *horizontal vibration* modes and the torsional vibrations of the structure. Alternate left and right foot-falls can result in such forms of vibration. Natural frequencies to be avoided are from 0.7 Hz to 1.3 Hz for vibrations that are either transverse or torsional. Frequencies between 1.6 and 2.5 Hz should also be avoided if they correspond to a longitudinal horizontal vibration mode.

Guidance from SETRA [17.11] proposes different ranges of vibration frequency depending on the importance of the structure and the density of pedestrian traffic, as well as the type of calculation to carry out and the actions to consider (Table 17.9).

It is generally accepted that pedestrians cannot cause vibrations of a footbridge having a lowest natural frequency above 5.0 Hz for vertical vibrations and 2.5 Hz for horizontal and torsional vibrations.

Accelerations

When high strength materials are used, or with cable supported structures, or simply when spans are long, it is often not possible to avoid certain natural frequencies that fall within the range of excitation frequencies due to pedestrians. In such cases user comfort may be impaired, and the serviceability of the footbridge compromised. Comfort is not measured in terms of frequency, but rather in terms of accelerations, or deflections.

The user tolerance threshold to a certain acceleration or deflection is a function of a number of parameters, such as the frequency, the duration of exposure (is it a transient effect lasting a few seconds or permanent, associated with the time it takes to cross the bridge), the density of traffic (single pedestrian or crowd), and the perception of the individual (which depends on age and sex). In reality, thresholds are very subjective, and they should not be considered as absolute limits but rather as transitions between tolerable and intolerable accelerations.

Several standards and references propose tolerance thresholds for acceleration as a function of the bridge frequency. Figure 17.14 [17.14] shows different limits. The limit shown in bold in this figure is that adopted by EN 1991 Eurocode 1. This limit is defined by Expression (17.18):

$$a_{lim,v} = 0.5\sqrt{f_v} \leq 0.70 \text{ [m/s}^2\text{]} \tag{17.18}$$

f_v : the natural frequency for vertical vibration of the bridge [Hz] that is the most likely to be excited by pedestrians, which is normally the first vertical frequency.

Equation (17.19) gives a similar definition for the limiting value of horizontal acceleration:

$$a_{lim,h} = 0.14\sqrt{f_h} \leq 0.20 \text{ [m/s}^2\text{]} \tag{17.19}$$

f_h : the natural frequency for horizontal vibration of the bridge [Hz] that is the most likely to be excited by pedestrians, which is normally the first horizontal frequency.

Reference [17.8] provides more detail on the limiting values proposed in different codes and standards.

The guidance from SETRA [17.11] defines levels of comfort. For example, for vertical accelerations a_v of a bridge, the following levels are defined:

- *maximum comfort* when a_v lies below 0.5 m/s^2,
- *average comfort* when a_v lies between 0.5 m/s^2 and 1.0 m/s^2,
- *minimum comfort* when a_v lies between 1.0 m/s^2 and 2.5 m/s^2,
- accelerations in excess of 2.5 m/s^2 are considered to be unacceptable.

Horizontal accelerations a_h are limited to 0.1 m/s^2 in order to avoid forced synchronisation. According to the guidance from SETRA, depending on the class of footbridge (Table 17.9), the client's representative can choose a level of comfort that is to be provided for the users.

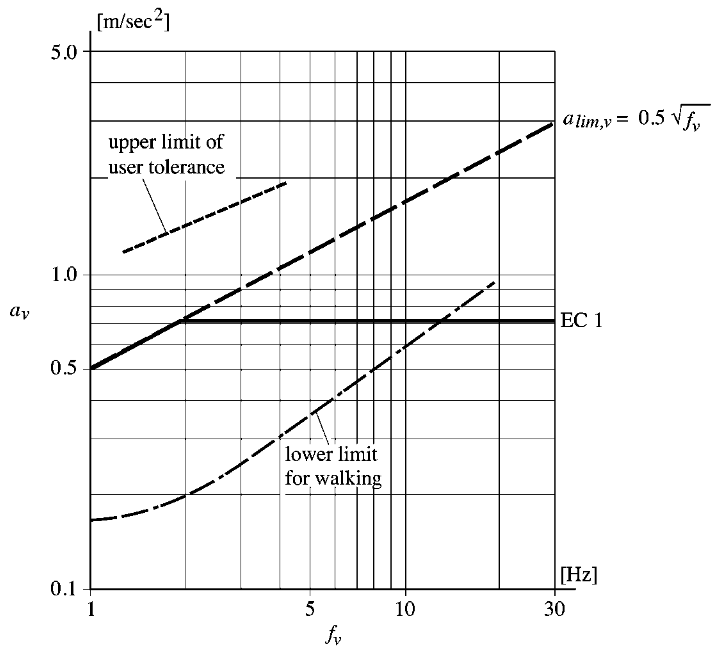


Fig. 17.14 Tolerance thresholds for vertical acceleration a_v as a function of natural frequency f_v [17.14].

For the simple case of a footbridge of uniform stiffness on two supports, the maximum acceleration at mid-span may be calculated using the following equation for the first harmonic:

$$a_{max} = \omega^2 \cdot \frac{w_0}{2\zeta} = 4\pi^2 f^2 \cdot \frac{w_0}{2\zeta} \qquad (17.20)$$

f	:	natural frequency of the bridge and frequency of the harmonic of amplitude G_1 – from Equation (17.12) for a single pedestrian and vertical acceleration – from Equation (17.13) for a single pedestrian and transverse horizontal acceleration – from Equation (17.14) for a single pedestrian and longitudinal horizontal acceleration
w_0	:	static deflection corresponding to G_1
ζ	:	damping ratio

To determine the acceleration of a bridge supporting several pedestrians, the amplitude G_1 may be multiplied by $\psi \cdot N_{eq}/l$ (Table 17.9, [17.11] and Fig. 17.10) to obtain the line load to consider when calculating the acceleration associated with the first mode of vibration.

Deflections

The authors of reference [17.15] propose two thresholds for the limiting value of deflections w_{lim}: one above which pedestrians are likely to feel that the structure is vibrating, and one above which the feeling of discomfort is such that they refuse to cross the bridge. These thresholds are shown in Figure 17.15 as a function of the natural frequency of the bridge, with a distinction between pedestrians that are stationary and those that are moving.

For the simple case of a footbridge of uniform stiffness on two supports, the maximum deflection due to a single pedestrian can be calculated using Equation (17.10):

$$w_{max} = \frac{a_{max}}{\omega^2} = \frac{w_0}{2\zeta} \qquad (17.21)$$

Table 17.16 gives some examples of deflections calculated using Equation (17.21), for two values of acceleration and as a function of the natural frequency of the structure.

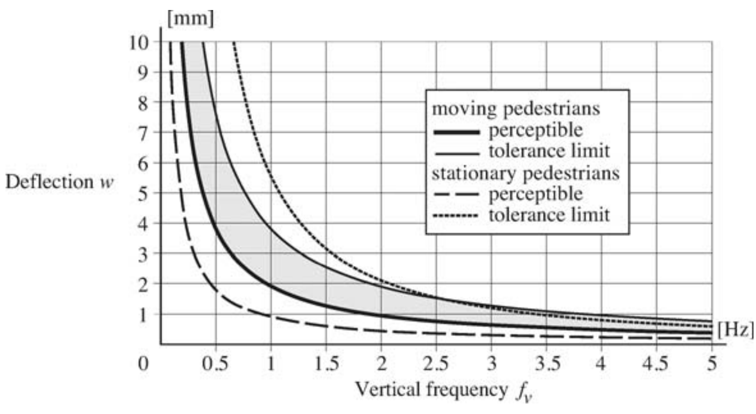


Fig. 17.15 Limits of perception and tolerance for vertical vibrations [17.15].

Table 17.16 Deflection w_{max} as a function of natural frequency and for two values of acceleration.

Natural Frequency of the Footbridge [Hz]	Deflection w_{max} [mm] for Acceleration	
	0.5 m/s^2	1.0 m/s^2
1.0	12.7	25.3
2.0	3.2	6.3
3.0	1.4	2.8

17.4.6 Numerical Application

Check the dynamic behaviour of a footbridge with a 40 m simple span. The bridge is class III according to Table 17.9. It comprises two plate girders supporting a 100 mm thick composite slab. The usable width for pedestrians is 2.50 m. Other characteristics of the footbridge are:

- second moment of area of the composite deck: $I = 0.030$ m^4
- mass per unit length: $m = 1500$ kg/m,
- damping ratio: $\zeta = 0.6\%$.

Determination of the First Natural Frequency for Vertical Vibrations
The fundamental frequency is determined from the circular frequency $f = \omega/2\pi$, with $\omega = \sqrt{k/M}$. Because it is a structure with uniform mass, Equations (17.7) and (17.8) are used to define the generalised characteristics M^* and k^* for calculating ω.

With a low density crowd ($d = 0.5$ pedestrians/m^2), the number of pedestrians on this class III footbridge is given by $N = 0.5 \cdot 2.5\text{ m} \cdot 40\text{ m} = 50$ pedestrians.

The total mass of the pedestrians is 50 x 70 kg = 3500 kg, or, per linear metre, this is 3500/40 = 87.5 kg/m. Such a mass will have an influence on the frequency and should be taken into account. The frequency calculations below are for both an empty bridge, and the bridge with pedestrians, using ϕ_m and ϕ_k according to Table 17.7.

$$M^* = \phi_m \cdot ml = 0.5 \cdot 1500 \cdot 40 = 30\,000 \text{ kg empty (31 750 kg with pedestrians)}$$

$$k^* = \phi_k \cdot \frac{EI}{l^3} = 48.7 \cdot \frac{210 \cdot 10^9 \cdot 0.030}{40^3} = 4.79 \cdot 10^6 \text{ N/m}$$

$$f = \frac{1}{2\pi} \cdot \sqrt{\frac{k^*}{M^*}} = \frac{1}{2\pi} \cdot \sqrt{\frac{4.79 \cdot 10^6}{30\,000}} = 2.01 \text{ Hz empty (1.95 Hz with pedestrians)}$$

These frequencies are within the range of frequencies associated with walking pedestrians. According to the standard SIA 260, they are, therefore, within the range of frequencies to be avoided. It is necessary, therefore, to move to the next step of checking, namely to undertake a dynamic analysis of the structure.

Calculation of Maximum Acceleration
According to Equation (17.20): $a_{max} = \omega^2 \cdot \dfrac{w_0}{2\zeta}$

The static deflection w_0 is calculated considering the distributed load G_1, taking into account an equivalent number of pedestrians N_{eq} moving in a synchronised way across the footbridge, so according to Table

17.9: $N_{eq} = 10.8\sqrt{N \cdot \zeta}$ and taking into account the coefficient ψ. This coefficient is here taken as 1.0 (Fig. 17.10) because the natural frequency of the structure is close to the frequency of walking pedestrians, which means there is a strong probability that resonance could occur.

$$w_0 = \frac{5G_1 l^4}{384 \cdot EI} = \frac{5(\psi \cdot 280 \cdot 10.8\sqrt{N \cdot \zeta}/l)l^4}{384 \cdot EI} =$$

$$\frac{5(1.0 \cdot 280 \cdot 10.8\sqrt{50 \cdot 0.006}/40)40^4}{384 \cdot 210 \cdot 10^9 \cdot 0.030} = 219 \cdot 10^{-6}\ \mathrm{m}$$

For the bridge with pedestrians:

$$a_{max} = \omega^2 \cdot \frac{w_0}{2\zeta} = \frac{k}{M} \cdot \frac{w_0}{2\zeta} = \frac{4.79 \cdot 10^6}{31\,750} \cdot \frac{219 \cdot 10^{-6}}{2 \cdot 0.006} = 2.75\ \mathrm{m/s^2}$$

Because the maximum acceleration exceeds 2.5 m/s^2, it is in the region that is deemed unacceptable for user comfort (Fig. 17.14). Therefore, either the characteristics of the bridge must be modified, for example by increasing its stiffness EI, or the use of a damper must be considered (see numerical application at the end of Paragraph 17.4.4).

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18 Arch Bridges

Kirchenfeld arch bridge at Berne (1883) (CH).
Eng. M. Probst, Ins and J. Röthlisberger, Neuchâtel.
Photo Kentaro Yamada.

TGV Mediterranean, Viaduc de la Garde Adhémar (2000) (F).
Eng. Bureau d'études Greisch, Belgium.
Photo ICOM.

18.1 Introduction

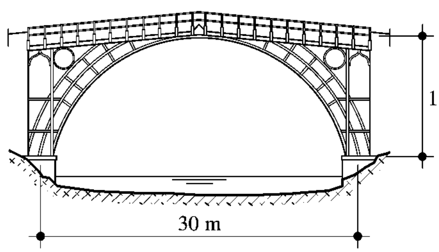
The structural form of the oldest bridges, which were constructed from stones and then masonry, was an arch. Its form means that loads are transferred to the supports in compression. There are no tensile stresses in the materials. The structure of the bridge at Coalbrookdale (Fig. 18.1(a)), which was the first metallic bridge in the world, is also an arch. But arch bridges are not just a thing of the past. The undeniable aesthetic qualities of arch bridges, and the way they integrate into a landscape, are additional attributes that make arch bridges a common choice even today. Also, given that displacements of the deck tend to be small, arch bridges are very often used on high-speed railway lines [18.1].

A large number of arch bridges have been constructed in recent years, many in the form of a tied arch, also called a "bowstring" arch. For these structures the arch is placed above the plane of the deck (Fig. 18.1(b)), while for other forms, the deck may sit on the arch, or be located at an intermediate position (Fig. 18.1(c)).

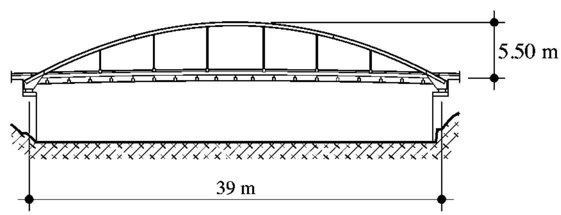
For medium to long spans, say 80 to 200 metres, arches are very competitive compared to other structural forms, while cable supported structures become of interest for longer spans. Within this span range, one also often finds inclined leg bridges, which have the simple form of a polygonal arch. References [18.2], [18.3], and [18.4] show examples of arch bridges, while Figures 3.14 and 3.15 show the two arch bridges with currently the longest spans.

It is interesting to note that many arches are built of steel, even though steel elements may be subject to instability when loaded in compression, which is a characteristic of the arch form. Therefore, concrete would appear to be a more appropriate choice for arches. However, steel possesses other advantages; in particular its low self-weight, weldability, and speed of erection are all advantageous during construction. The low self-weight allows the cranes that are needed on site to be relatively small, low cost, and manoeuvrable.

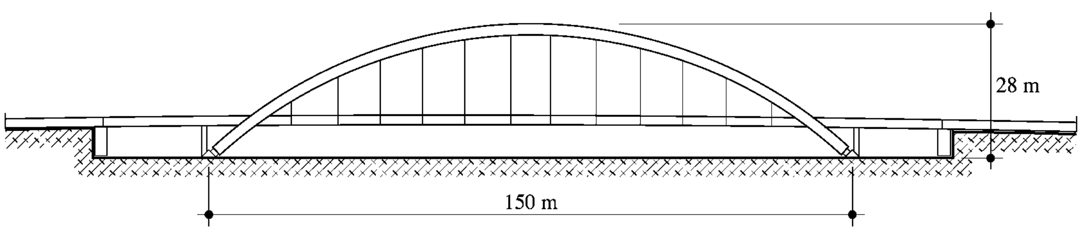
Section 18.2 describes arch bridges in terms of their form and geometry, identifying the influence of these on the structural behaviour of an arch. Considerations of form and geometry allow a number of ideas



(a) Upper deck (Coalbrookdale (GB), see also Fig. 3.2)



(c) Lower deck (passage du Châtelard (CH))



(b) Intermediate deck (Praça Dalla Coutinho, Brazil)

Fig. 18.1 Classification of arch bridges as a function of the deck position.

to be introduced, which are then treated in more detail later in the chapter from the point of view of conceptual design (Sect. 18.3), erection (Sect. 18.4), structural analysis (Sect. 18.5), and important points to consider for verifications (Sect. 18.6).

18.2 Form and Function

18.2.1 Position of the Deck

Traditionally, arch bridges were constructed from stones or masonry with the deck in an upper position supported on the arches. At the time, no suitable materials were available to make cable hangers that could take loads in tension, making it impossible to build arches with the deck at an intermediate or lower position. Today, concrete arch bridges still tend to have an upper deck, while steel arch bridges are conceived most often with a lower deck.

The choice of deck position is largely dictated by the local environment into which the bridge will be placed. An upper deck is often advantageous when crossing a deep gorge, against the sides of which the arch can be supported, while for crossing a river in relatively flat terrain, an arch with a lower deck is often more appropriate.

The rest of this chapter focuses mainly on arches with a lower deck, as this is the most common arrangement for steel arches.

18.2.2 Structural Form

Figure 18.2 presents the four most common structural forms for arch bridges. Arches that are fixed with rigid joints at both ends (Fig. 18.2(a)) are typical when masonry, or nowadays more commonly reinforced concrete, is used. This is because of the continuity between the foundations and the structure itself. Arches with two or three pinned joints (Fig. 18.2(b) and (c)) are more appropriate for steel structures, for which pinned joints are easily achieved (unlike when concrete is used).

An arch that is fixed at both ends has three degrees of redundancy; an arch that is pinned at both ends has one degree, and a three-pin arch has no redundancy. The magnitude of vertical displacements decreases as the degree of redundancy increases, while the moments and forces in bending, particularly at the ends of the arch and the crest, increase with the degree of redundancy.

The construction method may influence the choice of structural form (Sect. 18.4). For all forms of arch other than those that are tied, a large horizontal support reaction must be provided by the foundation

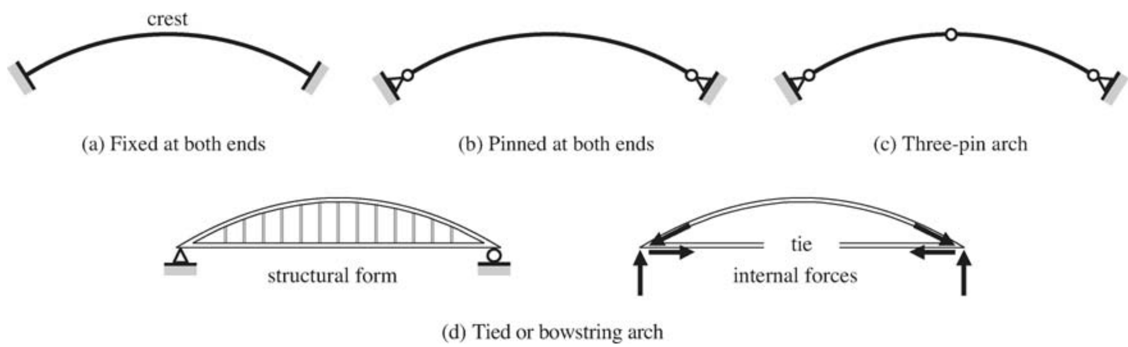


Fig. 18.2 Structural forms for arches.

ground because of the horizontal component of the compression force at the ends of the arch. This is the reason why arch bridges are most suitable when the ground is of good quality, particularly when the arch can be founded on rock.

Figure 18.2(d) shows a tied arch, which is also known as a "bowstring" arch. In Anglo-Saxon countries, this term is reserved for bridges formed from simple span trusses of varying depth (fish belly) [18.2]. The idea is to join the two ends of the arch, using a tie, in order to internally balance the horizontal support reactions. Having achieved this, only the vertical component of the axial force in the arch is transferred to the abutments, making this form much less sensitive to the quality of the foundation ground. Another advantage of this structural form is that it is insensitive to support settlements and uniform variations in temperature. Because of the redundancy of the form, the magnitude of internal moments and forces and vertical displacements are a function of the flexural stiffness of the arch relative to the tie (Sect. 18.5).

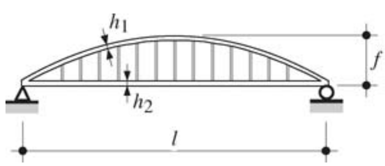
18.2.3 Number of Arches

One or more arches may be used. For single arch bridges the traffic lanes are positioned to either side of the arch, which is normally centred on the bridge axis. When a number of arches are used, the lanes are placed between the arches.

18.2.4 Slenderness

Figure 18.3 illustrates the definition of slenderness for an arch bridge. The slenderness λ_1 is given by the ratio between the total rise of the bridge f (to the crest of the arch) and the span l. The lower the slenderness, the greater the flexibility of a bridge, although too much rise spoils the appearance of an arch bridge. A slenderness λ_1 between 1/5 and 1/6 is often chosen, for both road and railway bridges.

The choice of section sizes for the deck and the arch is important. The distribution of material between the deck and the arch is decided as part of the conceptual design: a slim arch but a massive deck, or vice-versa. As explained in Paragraph 18.5.2, this choice has a significant influence on the behaviour of the bridge under asymmetric loading. If h_1 is the depth of the cross section of the arch members, and h_2 that of the deck, then the slenderness $\lambda_2 = (h_1+h_2)/l$ generally lies between 1/30 and 1/45, depending on whether there are one or two arches.



$$\lambda_1 = \frac{f}{l} \qquad \lambda_2 = \frac{h_1 + h_2}{l}$$

Fig. 18.3 Notation and definitions for slenderness.

18.3 Conceptual Design and Construction Elements

18.3.1 Load Paths

Transfer of the vertical loads acting on an arch bridge is illustrated in Figure 18.4. The deck acts in longitudinal and transversal bending to transfer the loads acting on it into the lower ends of the hangers, via the cross girders and, in some cases, lateral beams. The forces are then transferred from the hangers into the

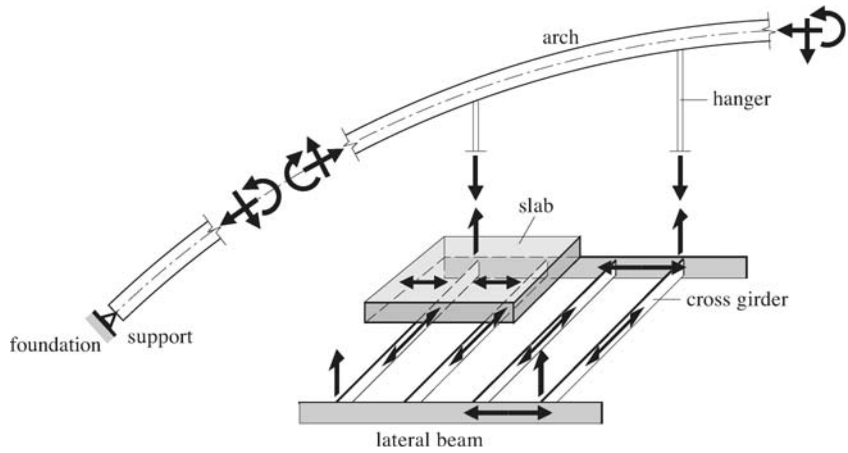


Fig. 18.4 Paths for vertical loads in an arch bridge.

arches. The arches act in compression, often accompanied by bending, to transfer the forces to the supports and foundations and, when present, into the tie member.

18.3.2 Arches

Arches must be designed to resist compression and bending. Both the buckling and bending resistances are highly dependent on the second moment of area of the arch cross section. This means that a cross section that has a sufficiently large second moment of area about both its major and minor axes is desirable, for example a rolled hollow section or fabricated box section. It may be that the axis with the greater second moment of area should be in the vertical plane to resist out of plane buckling of the arch. This is normally the case when there is no, or little, bracing between the arches.

It is possible to use rolled open sections for the arch members when the span is short; they have the benefit of considerably simplifying the joints with the hangers. They are limited to use with relatively short span arches because of the range of section sizes available.

18.3.3 Deck

The deck is a fundamental element for transferring the loads from their point of application to the supports. For its function as a rolling surface for traffic, the deck must satisfy several requirements concerning bending stiffness and resistance. For a tied arch bridge, the conceptual design of the deck must take into account its contribution as part of the tie.

An example of the *traditional deck configuration* for a composite arch bridge is shown in Figure 18.5, comprising a reinforced concrete slab, cross girders, and lateral beams. The slab works primarily in bending to transfer the local traffic loads to the cross girders. The cross girders, which are normally at 5.0 m centres, are supported on the lateral beams that are generally positioned level with the base of the arches. The lateral beams transfer the forces from the cross girders into the hangers. The slab is connected to the upper flanges of the cross girders to produce a composite cross section in the central part of the cross girder.

If the spacing between the cross girders is increased, then additional beams, generally called stringers, must be added. These run longitudinally to support the slab. The lateral beams are then relieved of some

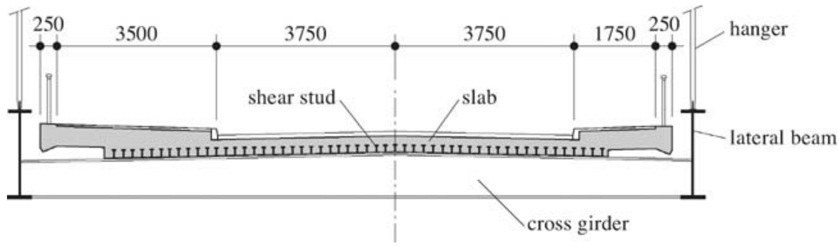


Fig. 18.5 Example of a traditional deck of a composite arch bridge.

load and may even be eliminated. In such cases the hangers are joined to the cross girders and will have a greater cross section.

Tied Arches

Tied arches differ in design and detailing from those that have no tie. For tied arches, the horizontal component of the compression in the arch must be able to pass into the tie.

The configuration of the *deck* may be exactly the same as that of a deck that does not resist tension (Fig. 18.5). The lateral beams fulfil the function of ties in resisting the tension forces. The connections between the concrete slab and steelwork must be designed in a way that avoids the slab being affected by the tension in the lateral beams, so that concrete cracking is limited. Therefore, the slab is not connected directly to the lateral beams, but only to the cross girders. For the same reason the slab is not connected to the cross girders at the ends of the bridge, which connect together the two arches at their bases.

Figure 18.6 shows a *modern deck configuration* of an arch bridge, in which the concrete slab forms an integral part of the tie, working with the lateral beams to resist tension. Compared with a traditional configuration, this option has the advantage of allowing shorter cross girders for a given deck width. It is also beneficial in terms of durability, because the joints between the cross girders and the lateral beams are protected from the weather by the slab. On the other hand, it requires the detailing of the slab at the ends of the bridge to be appropriate for the introduction of tension.

Figure 18.7 shows two examples of how the ends of a slab can be designed so that the concrete slab carries some tension. In Figure 18.7(a) a system that is rigid in its plane, comprising members that form a truss and can be integrated into the plan bracing, is fixed to the ends of the lateral beams. The truss members are formed from I sections and connected to the slab by shear studs. For this case the spacing between cross girders is small, and the hangers are connected to the lateral beams.

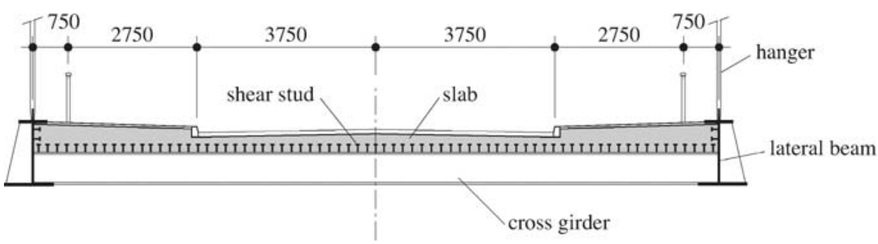
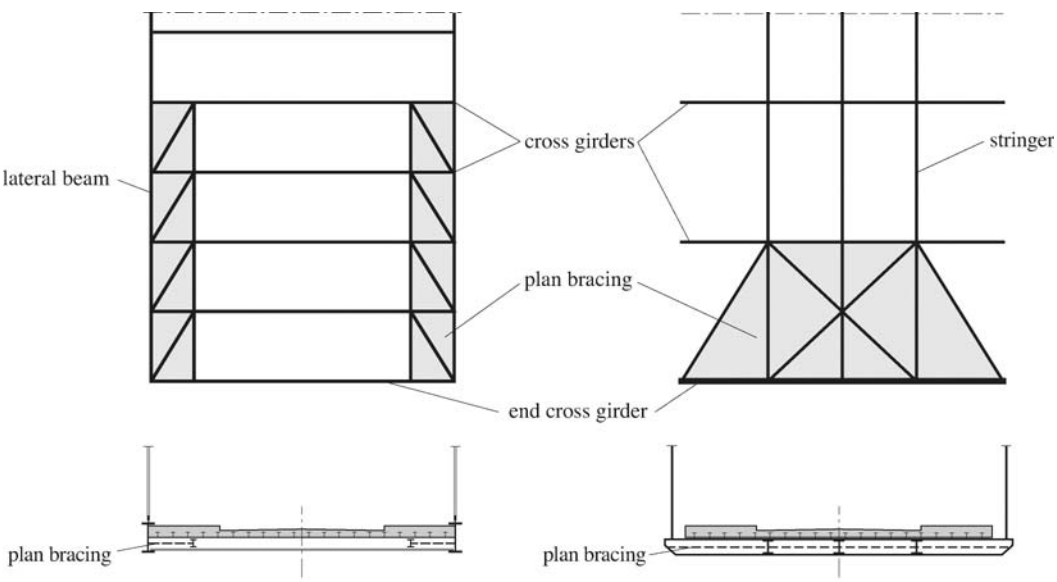


Fig. 18.6 Example of a deck that contributes to the tension resistance of the tie.

Figure 18.7(b) shows the horizontal plan bracing at the ends of the deck, which links the first two cross girders and the stringers supporting the slab. In this case, the cross girders are more widely spaced and directly attached to the hangers, as there are no lateral beams.

For both of these examples, the cross girders and the plan bracing are connected to the slab via shear connectors. Normally, the cross girder at the bridge ends is stiffer in both bending and torsion than the other cross girders, to assure good transfer of tension into the slab. The designer must also pay attention to the fact that using the concrete slab as part of the tie may result in transverse cracking. Sufficient longitudinal reinforcement must be used to limit the opening of these cracks.



(a) Tie comprising the lateral beams and the slab (b) Tie comprising the stringers and the slab

Fig. 18.7 Examples of bridge end details to introduce tie tension into the slab.

Construction Details

Figure 18.8 shows an example of a joint between a cross girder and a lateral beam. It shows the reinforced concrete slab, the cross girder, and shear studs as well as the vertical stiffener that is welded to the web of the beam. This joint provides a degree of end fixity for the cross girder and, due to the resulting frame action, enhances the resistance to lateral torsional buckling of the lateral beam. It also allows the wind forces that act on the beam to be transferred into the slab, which fulfils the function of plan bracing.

For tied arches the joint between the arch and the lateral beams is a region that is particularly highly loaded by the forces in the arch and tie, as well as the support reactions. Stiffening must be sufficient to allow these forces to be properly resisted. Figure 18.9 shows an example of such a detail, taken from the bridge over the Marne Canal in Strasbourg. The plate that sits on the support is heavily stiffened. There is a smooth transition at the support from the box girder of the arch into the I section of the lateral beam.

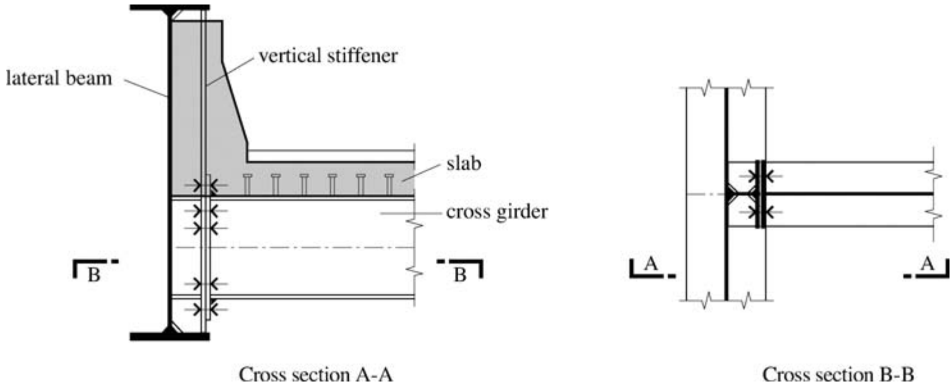


Fig. 18.8 Example of the joint between a cross girder and a lateral beam.

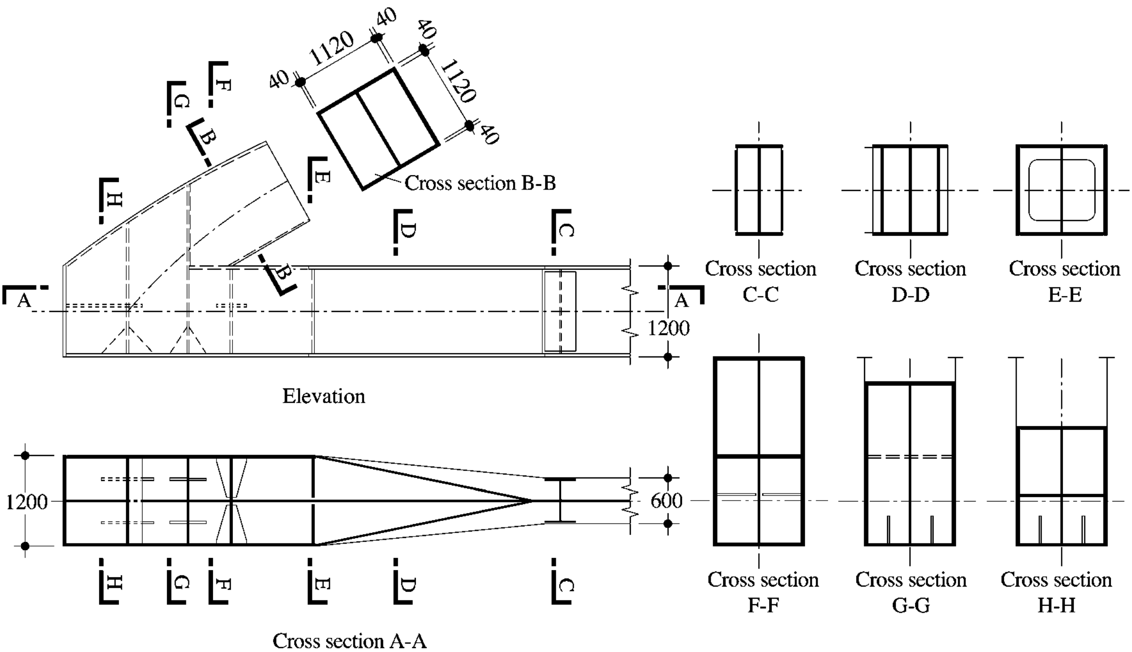


Fig. 18.9 Stiffening of an arch to lateral beam joint for a tied arch. Bridge over Marne Canal at Strasbourg (span 103 m, deck width 11 m) [18.5].

18.3.4 Hangers

Types of Hanger

The hangers between the deck and the arch may be cables, bars, or even rolled sections. Figure 18.10 shows two types of hanger, as well as the joints between these hangers and the lateral beams and arches.

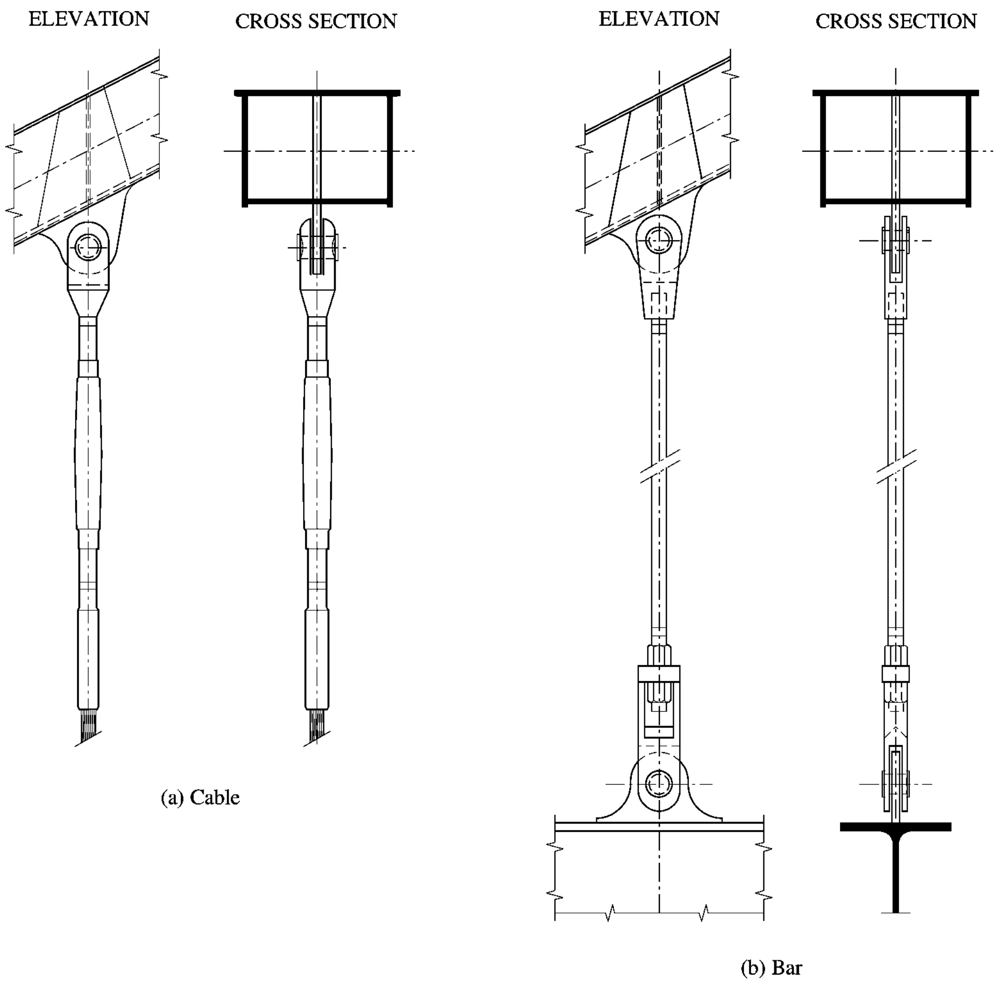


Fig. 18.10 Examples of hangers and their joints.

Bars, which are threaded at their ends, are often used because they offer several benefits when compared with cables. In particular:

- joints are simple,
- they are easily prestressed without the need for special equipment,
- their elastic modulus is approximately 15% higher than that of cables.

To form a long hanger, if it is necessary to join several bars end to end, this can be achieved by welding. However, this must be carefully carried out, giving due consideration to fatigue and brittle failure of the connections. To avoid welded connections it may be desirable to use cables when the hangers are long.

Rolled sections are nowadays only rarely used for hangers. Their main advantage over bars and cables is their greater bending stiffness. This stiffness may be needed when a bridge only has a single arch, because the lack of bracing (between two arches) reduces the out of plane stiffness of a bridge.

Joints

Ideally a hanger should be pinned at both ends, as any fixity will result in bending stresses that could result in fatigue cracking at the joints. For this reason pinned joints are normally used for railway bridges. For small diameter bars or cables, standard pinned joints are available. These are formed from cast components (Fig. 18.10(a) and (b)). However, when the hangers are formed from solid bars, it is sometimes easier to design a joint that does not really act as a pin, for example by welding the bar to a gusset plate, as shown in Figure 18.11.

The fatigue resistance of such a welded joint must be carefully considered. For this particular example, the gusset plate is welded directly to the web of the lateral beam through an opening in its upper flange. This detailing avoids the danger of lamellar tearing in the upper flange, which could occur if the gusset were welded directly to it (TGC Vol. 10, Fig. 7.10).

If it is envisaged that it may be necessary to replace a hanger following vehicle impact, or because of a fatigue problem, then this should be allowed for in the joint configuration. Pinned joints normally lend themselves to replacement, because they often include a pin that may be temporarily withdrawn to allow the hanger to be replaced.

Depending on the importance of a particular bridge and the erection method adopted, the conceptual design of the hangers and their end joints should take into account how they will be loaded. Ways of introducing tension into the hangers, such as with hydraulic jacks, will require some free space around the joints.

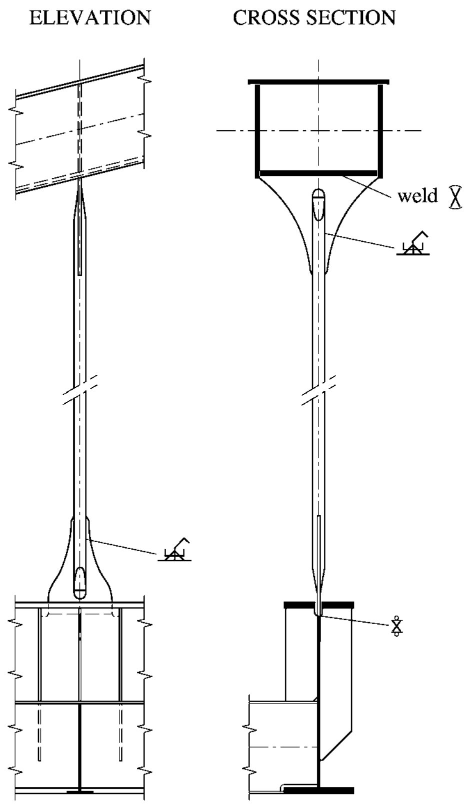


Fig. 18.11 Example of the joints of a welded hanger.

The designer must also check the local resistance of the arches where the hangers are attached. Stiffening of the arch in these regions is necessary to facilitate good distribution of the forces introduced by the hangers. An example of stiffening of a box section arch is shown in Figure 18.12.

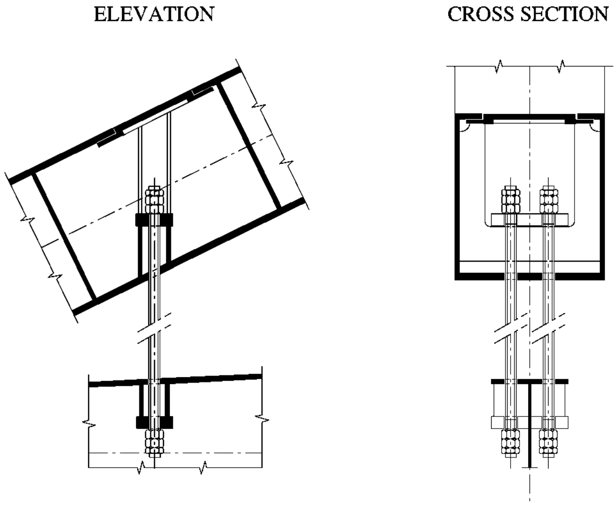


Fig. 18.12 Example of stiffening the joining of a hanger to a box arch. Bridge over the Marne Canal at Strasbourg [18.5].

Arrangement of the Hangers

The spacing, and any inclination, of the hangers are important parameters to consider during the conceptual design of an arch bridge. In addition to affecting the appearance of a bridge, these parameters influence the dimensions of the deck, the stiffness of the bridge, and the resistance of the arches to overall buckling. Each of these points is considered in Section 18.5.

Figure 18.13 shows schematic elevations of two arch bridges: the first (Fig. 18.13(a)) has vertical hangers, while the second (Fig. 18.13(b)) has inclined hangers. Inclined hangers result in a better distribution, into the arch, of the vertical traffic loads that act on the deck (Sect. 18.5). In particular, for asymmetric loading, the bending moments in the arches and deck are smaller when inclined hangers are used.

Consideration of these points can result in original and, indeed, extreme solutions. For example, arch bridges have been designed with a network of inclined hangers that cross at three or even four points to better distribute the loads that lead to bending of the bridge [18.6]. In this way a slenderness, as defined in Paragraph 18.2.4, of around $\lambda_2 = (h_1 + h_2)/l = 1/100$ can be achieved for road bridges supporting two-way traffic. Significant economies of material can be achieved, and erection can be easier due to the low self-weight of the structure. However, it would be difficult for such slender forms to support the loads associated with freeway traffic. The decks of bridges using a network of hangers, and with a slenderness of 1/100, have a maximum width of around 7 m.

The appearance of inclined hangers can be attractive when a bridge is viewed from a position that is perpendicular to its axis. However, from any other angle, this pleasing appearance may transform into an apparently disorganised criss-cross of hangers, which also reduce the transparency of the bridge. This problem is exacerbated if the two arches are inclined towards each other to meet at their crests.

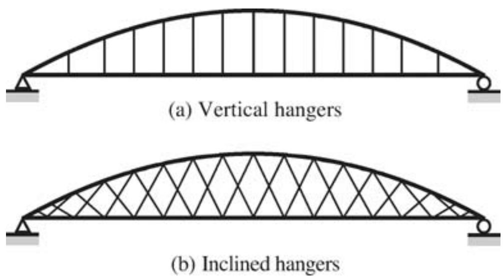


Fig. 18.13 Examples of hanger layout.

Vibrations and Damping

Hangers may be set in motion, or vibrate, by traffic, wind, or even rain. Such vibrations may result in fatigue damage to rigid joints at the ends of the hangers, due to the local bending at these points. While numerous studies have been made of the aerodynamic behaviour of cables used with cable stayed and suspension bridges, less information exists on hangers for arch bridges. Reference [18.7] makes an interesting contribution to the study of wind-rain interaction. The vibration frequency of a cable in tension is studied in [18.8]. The tensile force in a hanger increases its natural frequency.

A number of options may be considered for avoiding excessive vibration of hangers. Dampers may be added, or a rough surface may be adopted for the cables or bars.

18.3.5 Bracing Between the Arches

As for a composite beam bridge, the slab fulfils the function of plan bracing for the deck in the final state. For arch bridges, bracing between the arches makes an important contribution to out of plane stability by tying together the two arches and increasing the stiffness of the complete structure. If the arches are

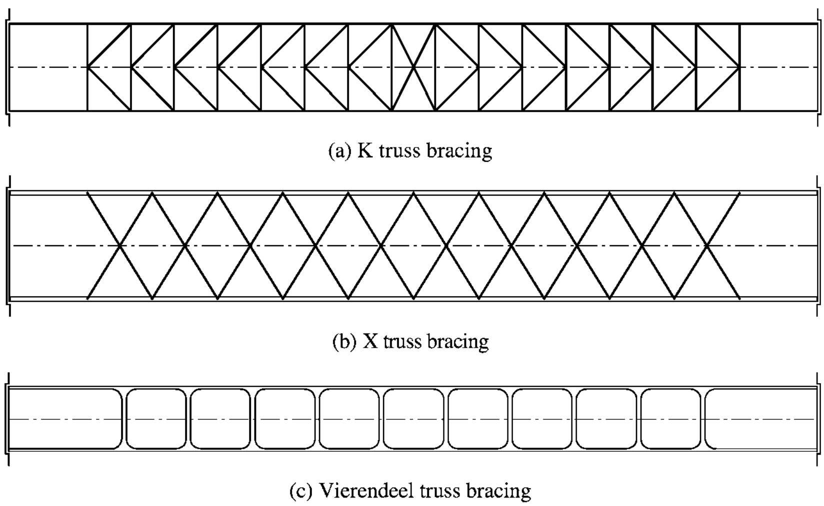


Fig. 18.14 Examples of bracing between the arches (viewed from above).

nominally pinned at their bases, about their longitudinal axis, then the bracing that ties them together is absolutely indispensible to avoid the arches being unstable.

Different examples of bracing between the arches are shown in Figure 18.14. The solution without triangulated braces (Fig. 18.14(c)) acts as a Vierendeel truss to resist transverse forces. This solution is often adopted for its aesthetic qualities even though the out of plane stiffness of the complete structure, therefore its stability, is less than can be achieved when triangulated bracing is used. For an arch bridge with the deck in a lower position, the layout of the bracing must be such that head-room requirements for traffic passing over the bridge are satisfied.

It should be noted that it is not always necessary to include bracing between the arches. When such bracing is not used, the arches must be fixed rigidly to the cross girders at their ends (Fig. 18.26(a)).

18.4 Erection Methods

Different erection methods for steel bridges are described in detail in Chapter 7. Certain methods that are specific to arch bridges are considered below.

18.4.1 Cantilever Erection with Cable Supports

This method comprises erecting a half arch, which is cantilevered from the bank with temporary support from cable stays, as shown schematically in Figure 18.15(a). This method is only of interest for arches

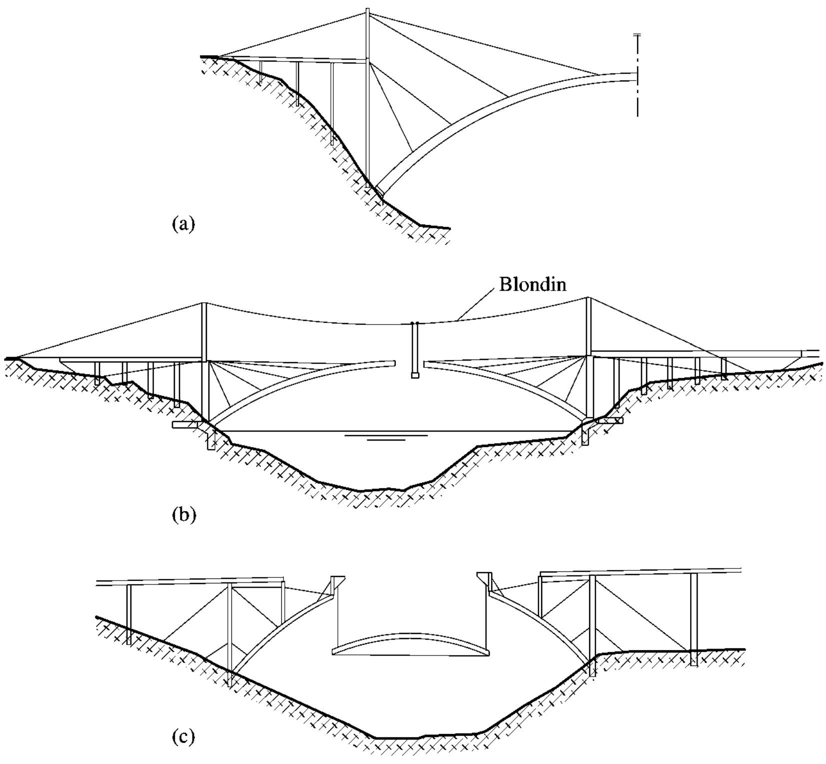


Fig. 18.15 Examples of erection of an arch.

with an upper deck, because the cable stays can then be supported from temporary pylons that need only be of limited height.

When conditions permit, it is also possible to lift elements of the arch into place from the ground, using a winch or a crane. For bridges of large span, or when access is very difficult, an aerial ropeway (sometimes called a *Blondin* after Charles Blondin) may be used (Fig. 18.15(b)). Also for long span bridges, in order to reduce the time needed on site for erection, the central part of the span may be assembled on the ground then lifted into place using large jacks. This method, which is shown in Figure 18.15(c), often requires the use of a temporary tie across the central part of the arch to limit bending during the lifting operation.

18.4.2 Lifting or Lowering of a Half Arch

When the local environment permits, each half of an arch can be built on the bank starting from an abutment and assembling the different arch elements horizontally or vertically (Fig. 18.16). Once a half arch is completed, it is either lifted or lowered into its final position using jacks and cable stays. This method is of interest for arches with two or three pinned joints because such joints are needed to allow the half arch to be tilted into position.

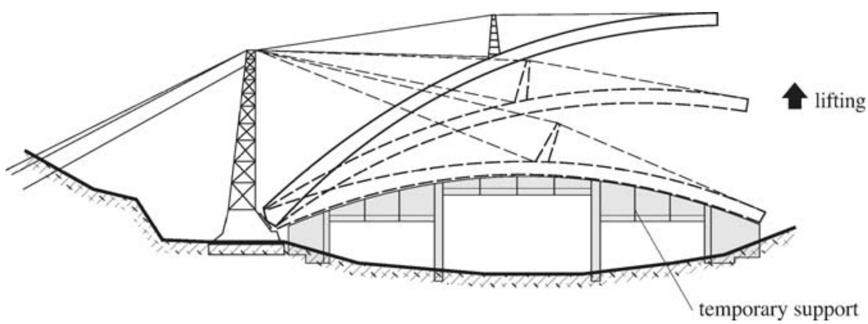


Fig. 18.16 Erection sequence for raising a half arch.

18.4.3 Preassembly of the Complete Bridge

Tied arch bridges lend themselves to placement of the complete bridge because their support reactions are uniquely vertical. A temporary tie must be used during erection to stabilise arches that will not be tied in their final state. Preassembly is only possible when there is sufficient space to construct the arches on the banks or along the axis of the bridge. In urban environments, therefore, preassembly of the complete bridge is often difficult. Three ways of placing a complete arch bridge may be envisaged: by launching, by rotation, or by sliding.

The complete bridge may be *launched* when it crosses a navigable waterway and there is sufficient space for preassembly on one of the banks in line with the bridge axis. The leading edge of the bridge is landed on a barge, which can then cross the waterway (Fig. 18.17). Because the supports and structural form of an arch bridge vary according to the launching phase and are different from those associated with the bridge in its final position, it is normally necessary to provide the arch with temporary supports as the hangers are designed for tension forces only. This is also shown in Figure 18.17.

The complete bridge may be manoeuvred into its final position by *rotation*. The bridge is erected on one of the banks, parallel with the bank edge. Once the deck and arches are finished, the bridge is rotated about the abutment into its final position. When this method can be used, it is economical because the work associated with erection of the arch is carried out on the ground, without the need for any special equipment.

Finally, placing by *sliding* sideways can be particularly advantageous when a bridge in service is to be replaced. The new bridge may be constructed parallel with the existing bridge, on temporary supports, and then slid sideways after the existing bridge has been removed.

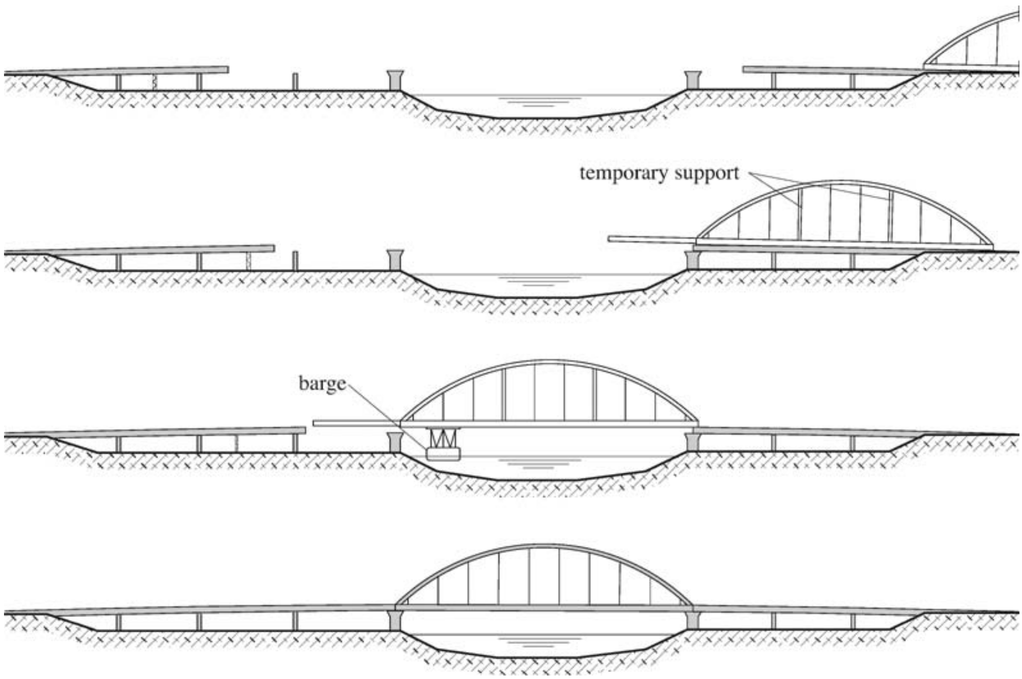


Fig. 18.17 Launching a preassembled structure using a barge.

18.4.4 Erection of the Arches Supported on the Deck

An alternative method may be used if the deck of the arch bridge is sufficiently strong and stiff in bending, or perhaps it can be supported on temporary supports. To begin with, the deck is constructed either from the ground or by launching. Once the deck is in place, it serves as a work platform for erection of the arches. Falsework towers are supported on the deck and allow relatively easy erection of the arches. Such an erection procedure is shown in Figure 18.18. Clearly this method may only be used for arch bridges with a lower deck.

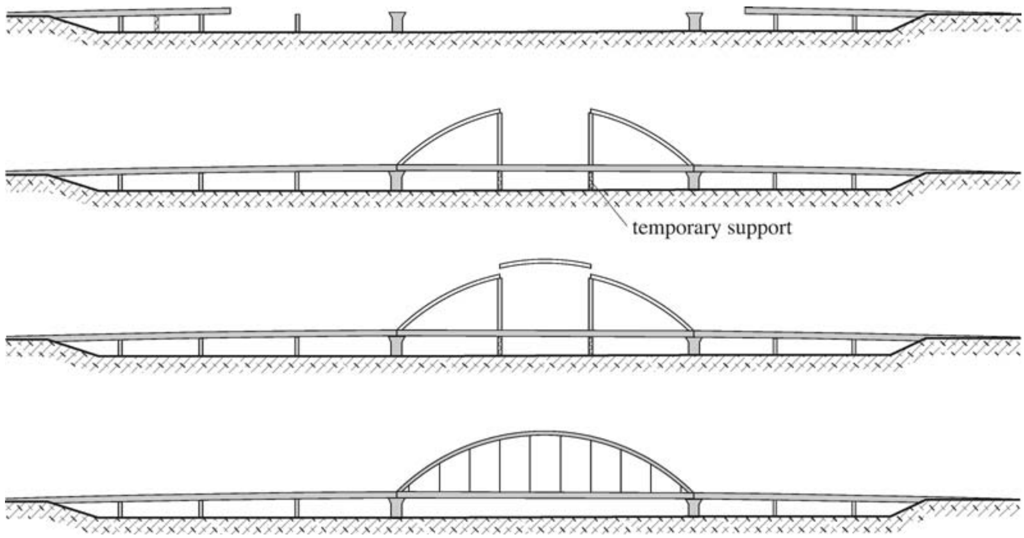


Fig. 18.18 Example of launching the deck, followed by erection of the arches.

18.5 Structural Analysis

18.5.1 Internal Moments and Forces

Compression Line

Ideally the form of an arch should coincide with the compression line defined by the internal forces due to the loads (TGC Vol 1, § 3.9.3). In such a case, the loads would only result in normal compression forces in the arch. Figure 18.19 shows the ideal forms of an arch for both uniformly distributed and concentrated

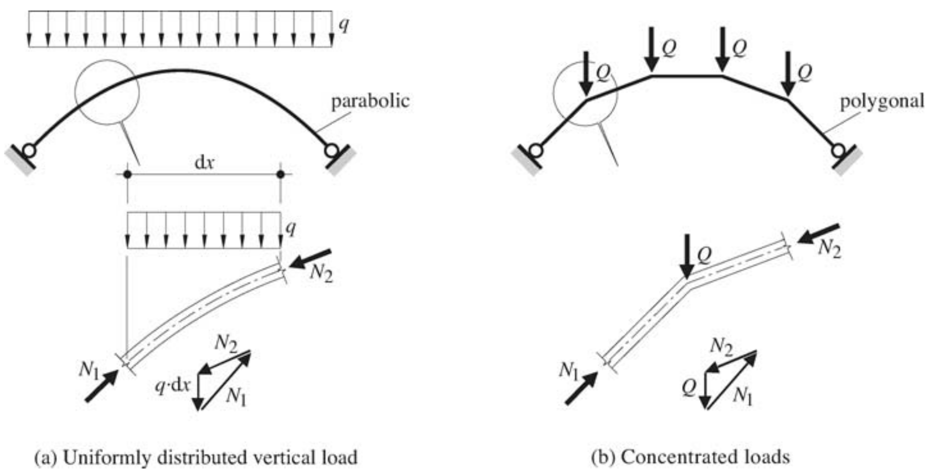


Fig. 18.19 Ideal form for an arch, depending on the loads.

loads. In the theoretical case of a vertical load that is uniformly distributed over a horizontal surface, the arch would be parabolic. For concentrated loads at the hanger positions, the ideal form of the arch would be polygonal. However, because the imposed load is generally not uniformly distributed over the arch, and arches are rarely polygonal, they are also subject to bending moments. The bending moment at a particular point on the arch may be defined as the product of the normal force acting in the cross section and its eccentricity relative to the axis of the arch.

Calculation of the Internal Moments and Forces

Calculating the internal moments and forces in an arch requires resolving the equations of equilibrium and compatibility for a redundant system. Analytical expressions may be used for preliminary design when geometry and load cases are simple.

A parabolic arch that is pinned at both ends, of span l and rise f, is not subject to any bending moment as a result of a load q that is uniformly distributed relative to the horizontal (such as the self weight of the deck). It is subject to a normal compressive force, which has a horizontal component H given by:

$$H = \frac{ql^2}{8f} \tag{18.1}$$

The normal force in the arch is given by Equation (18.2), where α represents the angle between the tangent to the axis of the arch and the horizontal. Figure 18.20 shows the distribution of the normal force N for this situation.

$$N = \frac{H}{\cos\alpha} \tag{18.2}$$

It should be noted that the self weight of an arch acts along its axis, not relative to the horizontal. This means that the ideal form of an arch, to resist its own self weight, is not parabolic but rather an inverted catenary (TGC Vol. 1, § 10.4.4).

When a bridge is subject to loads that are not uniformly distributed, or *concentrated*, there exists bending of the arch. Calculating the bending moments is complicated because of the large number of parameters that affect the result. However, a designer may resort to the use of numerical methods. Paragraph 18.5.2 provides some qualitative information on the behaviour of arch bridges subject to asymmetric or concentrated loads.

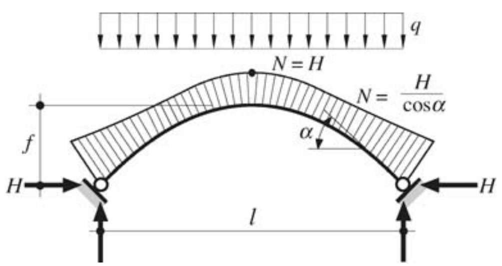


Fig. 18.20 Normal compression force in a parabolic arch that is pinned at both ends and subject to uniform vertical load.

18.5.2 Asymmetric and Concentrated Loads

As discussed above, it is only possible for an arch to be subject to pure compression if the line of action of the internal forces, due to self-weight and traffic loads, coincides with the axis of the arch. This is never the case. Particular load cases that do not satisfy this condition are traffic loads that act either asymmetrically or in a concentrated manner on the deck. Such loads are considered separately below.

Asymmetric Loads

Figure 18.21 shows the effects of both symmetric and asymmetric loads on the deformation, axial force, and bending moment in an arch that is either pinned at both ends or tied. If the form of the arch is such that the compression line coincides with the axis of the arch (Fig. 18.21(a)), there is no bending moment, and the normal force, for a parabolic arch, may be calculated using Equation (18.2). An asymmetric load over half the span (Fig. 18.21(b)) normally results in the greatest bending moments in the arch. On the other hand, for this situation, the normal force will be less than for the symmetric case.

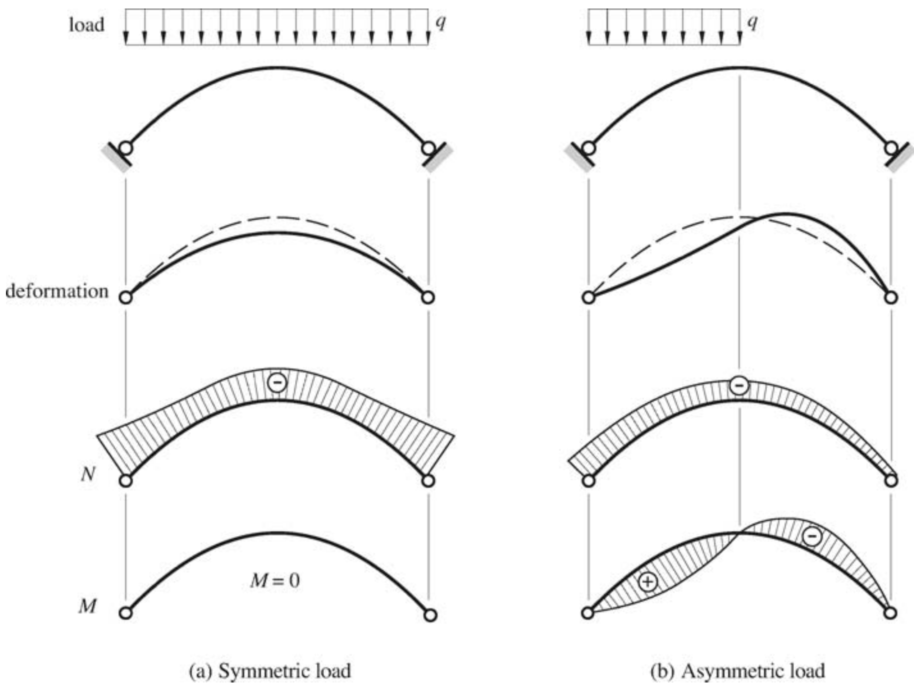


Fig. 18.21 Effect of symmetric and asymmetric uniform loads on deformation, normal force, and bending moment.

In drawing the conclusions stated above, the bending stiffness of the deck was neglected, assuming that the loads were directly applied to the arch. However, the bending stiffness of the arch relative to the deck may have a significant influence on the distribution of internal moments and forces in the structure. The axial stiffness of the hangers also has an influence, but one that is much smaller. Figure 18.22 shows the influence of the bending stiffness of the arch relative to the deck for two extreme cases – flexible arch and stiff deck and vice versa – by illustrating the deformation of, and the moments in, the deck.

For a stiff deck (Fig. 18.22(a)), only a small portion q^* of the load q is transferred to the arch. The remainder of the load is carried in bending of the deck. For a flexible deck, almost the entire load is transferred to the arch. Flexibility in bending of a deck means that it follows the deformed shape of the arch without supporting the load. The influence of the relative stiffness is the same for a tied arch.

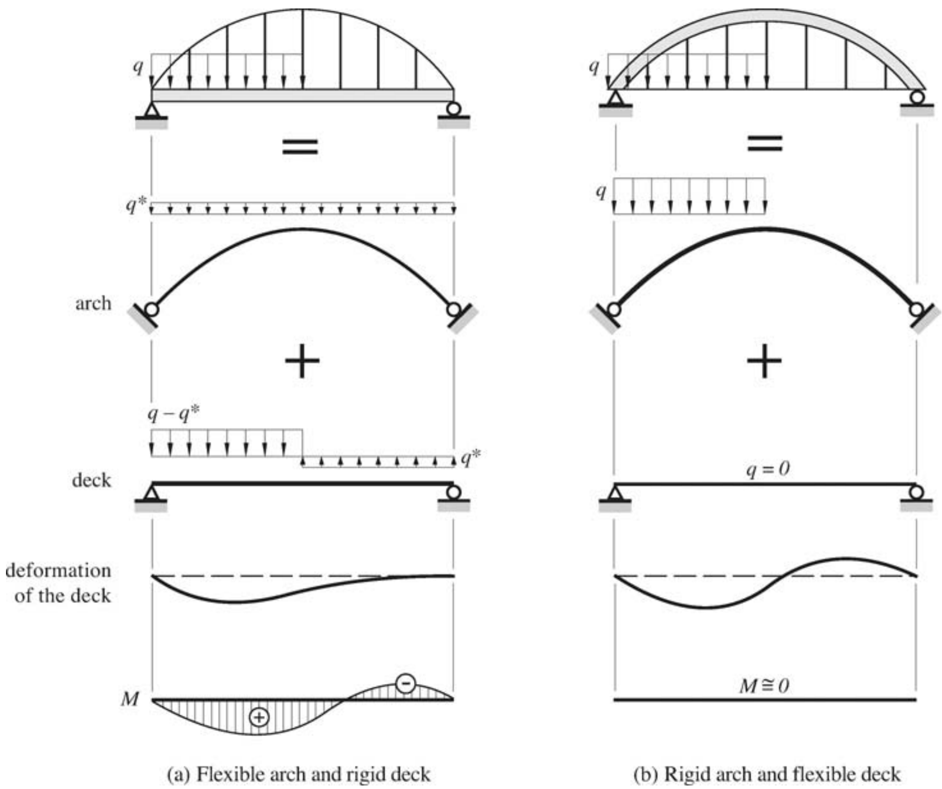


Fig. 18.22 Influence of the stiffness of the arch relative to that of the deck, for asymmetric uniform load.

To summarise, the stiffness of the deck relative to the arch will dictate the proportion of the total load that is carried by each of them. It is, therefore, up to the designer to decide which structural member will transfer the asymmetric part of the loading to the supports, by providing it with the necessary stiffness. On the other hand, the normal forces in the arch and hangers are less influenced by this relative stiffness.

Concentrated Loads

Heavy concentrated loads, such as those from the axles of road or railway convoys, may result in significant bending moments in an arch, as shown in Figure 18.23(a). To reduce these bending moments, the loads should be transferred to the arch through several hangers, which effectively diffuse the load between its point of application and its transfer into the arch. When the deck is relatively stiff, this diffusion is achieved automatically as the deck bends (Fig. 18.23(b)). Deformation of the deck means that the neighbouring hangers are loaded, in addition to the hanger immediately adjacent to the point of load application. Therefore, the load is transferred to the arch with a greater distribution, producing less bending in it.

Figure 18.24 shows the effects of inclined hangers (V shape) on the distribution of a concentrated load into an arch. The length over which a given deck load is introduced into the arch is significantly greater when the hangers are inclined rather than vertical. This effect is pronounced when the deck is flexible. Also, a deck supported by a number of small diameter hangers will exhibit better behaviour, in terms of

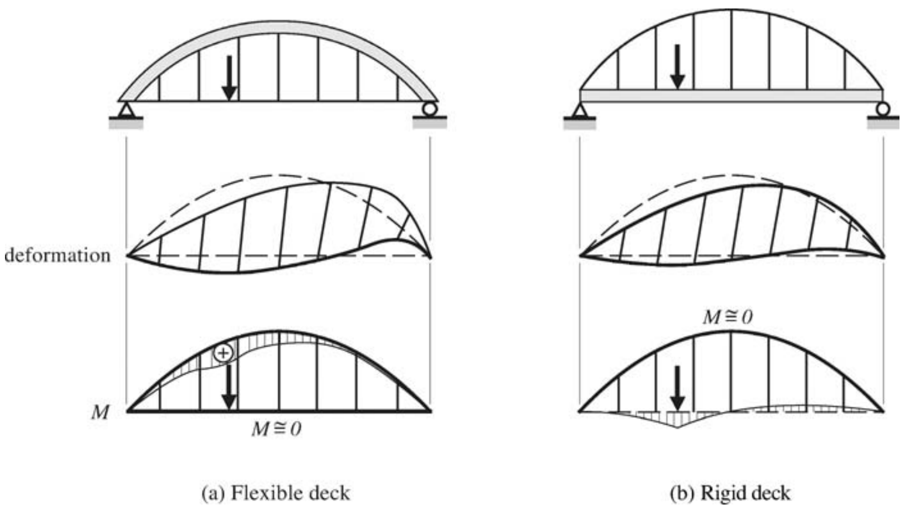


Fig. 18.23 Influence of the stiffness of the deck for a concentrated load.

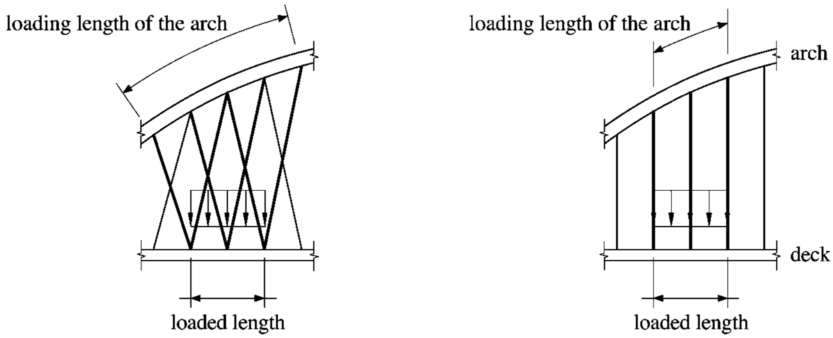


Fig. 18.24 Use of inclined hangers to introduce load into the arch.

carrying concentrated loads, than a deck of comparable stiffness supported by a smaller number of larger hangers.

The overall behaviour in bending of an arch bridge, therefore, is strongly influenced by numerous factors, notably the relative stiffness of the deck and arch, the spacing between and orientation of the hangers, and, to a lesser extent, the stiffness of the hangers. All these factors should be taken into account during the conceptual design and structural analysis of the bridge to obtain the best overall solution.

18.5.3 Stability of the Arches

Buckling Modes

Questions concerning the stability of arches in compression and bending are amongst the most delicate and difficult that the designer of such a structure faces. In effect the buckling length must be identified for a curved bar that is subject to compression and bending, restrained by hangers (elastic supports), and normally stabilised out of plane by bracing linking two arches together. Stability against buckling, therefore, must be verified for a number of cases, such as those shown schematically in Figure 18.25:

- in plane buckling of the complete arch,
- out of plane buckling of the complete arch, or two arches tied together by bracing,
- buckling, either in plane or out of plane, of part of the arch between hanger positions, or between cross bracing or bracing between the arches.

When the buckling length is known, the buckling stress may be calculated, and checking is carried out in the same way as for any other beam subject to compression and bending. Consideration should be given to a possible reduction in the effective cross section of the arch to allow for local buckling of the plates forming its cross section. The principles of checking stability and resistance are given in Paragraph 18.6.2.

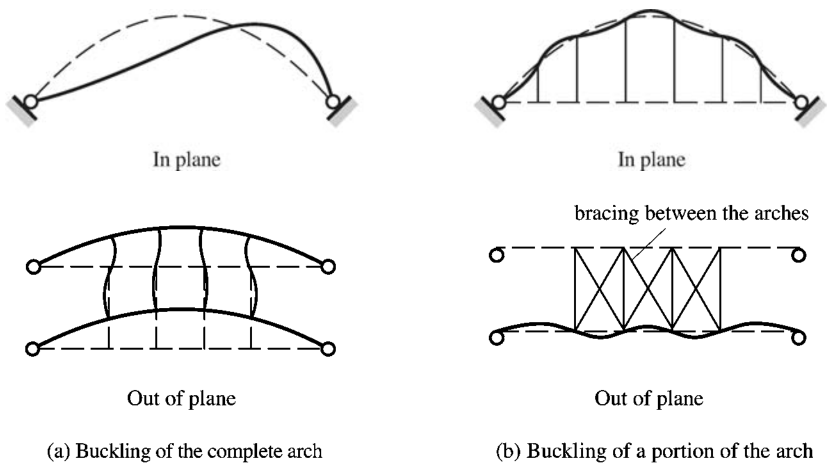


Fig. 18.25 Buckling modes to consider.

Determinant Parameters

The main parameters that influence the stability of an arch, and their importance, are listed below:

- The structural form of an arch has a governing influence on the buckling length. Distinction must be made between buckling in the plane of the arch and out of plane. As for columns, an arch that is fixed at both ends has a shorter buckling length than an arch that is pinned at both ends. A pinned arch has the greatest in plane buckling length. Out of plane, such an arch is unstable, as is an arch that is pinned at both ends, unless it is supported by bracing between the arches. Figure 18.26(a) shows how an arch may be built-in to the end cross girder and the lateral beam in order to reduce its buckling lengths. Because this cross girder is effectively an elastic support, its torsional and bending stiffnesses must be taken into account when determining the buckling lengths, in and out of plane, of the arch. The modelling of this detail is shown in Figure 18.26(b).

- For a given span and type of loading, the magnitude of load that an arch is able to support increases as the rise f increases. This is because the horizontal force H trying to spread the arch reduces as the rise increases (18.1), an effect that compensates for the greater length of the arch (and, therefore, its greater buckling length). This is valid when the ratio f/l is less than around 0.3, which is always the case for modern steel arch bridges (§ 18.2.4).
- The second moments of area I_x and I_y of the arch cross section have a determining effect on its stability. The greater these second moments of area, the greater will be the buckling resistance of the arch.
- The in-plane stiffness of the bracing between the arches (which may be a triangulated truss, or a Vierendeel truss, Fig. 18.14) influences the out of plane buckling length of the arch.
- Inclining the arches inwards helps with the stability of the complete structure by creating, where they meet, a lateral support against buckling.
- The axial stiffness and number of hangers influence the in plane stability of an arch. The more hangers are present, and the greater their diameter, the greater their stabilising effect.
- The bending stiffness of the deck, to which the hangers are fixed, also influences the stability of the arches because it acts to stiffen the support provided by the hangers.

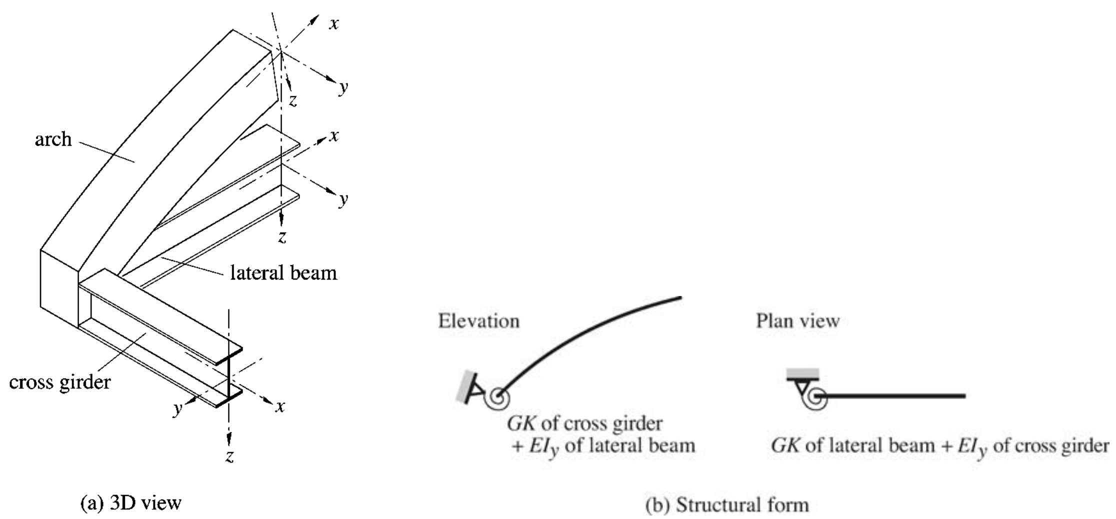


Fig. 18.26 Semi-rigid restraint of an arch end by a cross girder and a lateral beam.

These last two points, concerning the stabilising effect the hangers and deck have on the arch in plane, are illustrated in Figure 18.27. Figure 18.27(a) shows the structural form of a pin ended arch bridge with lower deck. Figure 18.27(b) shows the first buckling mode of the same arch, illustrating the way in which the arch is held by the hangers and deck, which apply restoring forces to the arch. Modelling of the arch (Fig. 18.27(c)) shows how this effect can be considered in a simple manner, for calculation purposes, by modelling the arch as a straight bar in compression, of equivalent length to the arch, and supported elastically along its length.

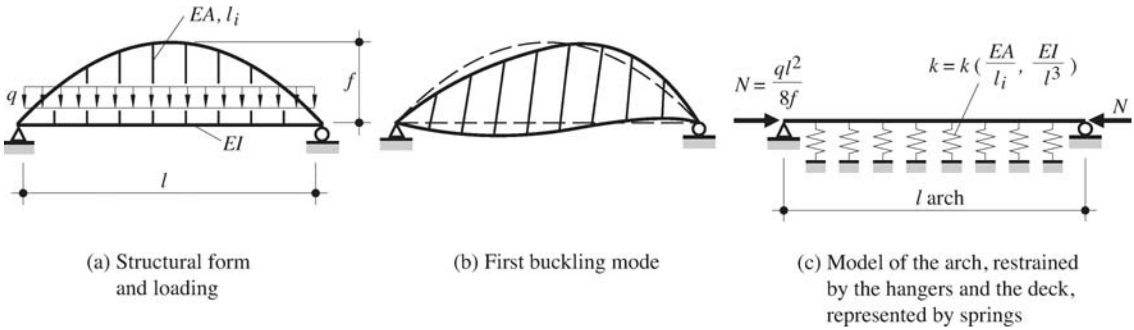


Fig. 18.27 Modelling the in-plane buckling of an arch.

Determining the Buckling Length

Understanding the behaviour of a curved bar subject to an interaction of moments and forces is complicated. The buckling length l_K may be determined by first calculating the elastic critical buckling load, then using this load to calculate the buckling length using Euler's Equation (18.3):

$$N_{cr} = \frac{\pi^2 EI}{l_K^2} \tag{18.3}$$

Detailed analytical calculations are beyond the scope of this chapter. Information on this subject may be found, for example in [18.9], but practicing engineers will use software to determine the critical load or to carry out a second order analysis. The paragraphs that follow provide general information to allow the designer to verify if the results of a numerical analysis are reasonable.

In-plane buckling

Figure 18.28 shows some example design curves, taken from [18.9] (Table 16.15), that allow the elastic critical buckling load of an arch to be determined, conservatively (three structural forms: three-pin arch, pinned at both ends, fixed at both ends). The curves are valid for an arch that is of constant second moment of area, and subject to a uniformly distributed load. Annex D.3 of EN 1993-2 [18.10] gives additional curves that may be used to define the buckling length.

Figure 18.29 shows an example of design curves, taken from [18.9] (Figure 6.163), that allow the favourable effect that the hangers have on the critical load $N_{cr,sus}$ to be taken into account, compared with the critical load N_{cr} for the arch without hangers. These curves do not reflect the favourable effect that the bending stiffness of the deck has on the critical load.

In the case of a tied arch, due to the tensile force in the deck, the favourable effect that the deck has on the stability of the arch is enhanced by the second order effects. This force tends to increase the bending stiffness of the deck. The design curves shown in Figure 18.30, which are taken from [18.9] (Table 6.20), allow the elastic critical buckling load to be determined for an arch, taking into account the relative stiffness of the arch, hangers, and deck. It can be seen that the stabilising effect of these elements can be very significant, particularly when the deck is relatively stiff.

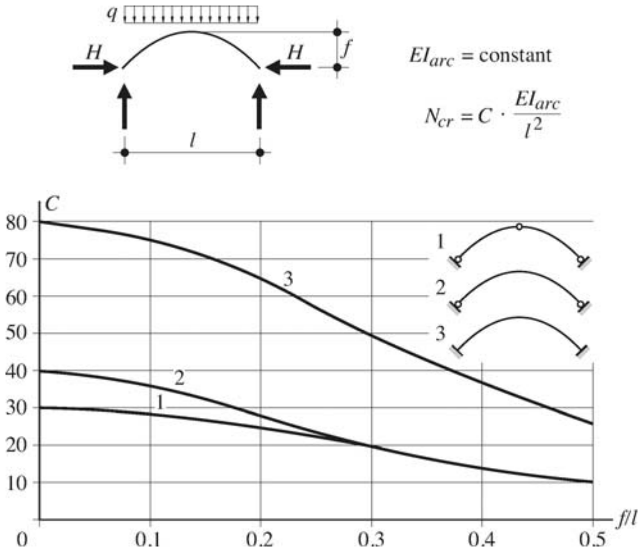


Fig. 18.28 Determining the elastic buckling load for a parabolic arch of constant bending stiffness and for three structural forms, subject to a uniformly distributed load [18.9].

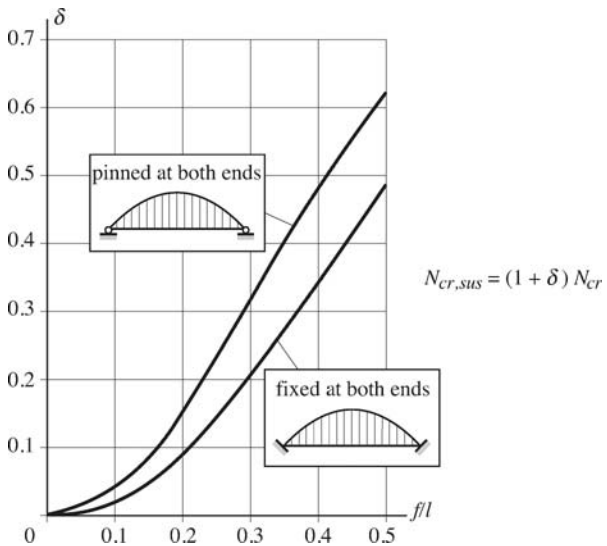
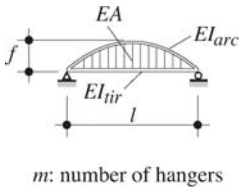


Fig. 18.29 Increase in the elastic buckling load of an arch (pinned or fixed at both ends) due to the restoring forces provided by the hangers [18.9].

Buckling out of plane

The design curves given in the Figures 18.28 to 18.30 only consider buckling in the plane of the arch. To determine the buckling length out of plane, one must take into account the elastic supports provided by the bracing that ties the two arches together structurally (Figs. 18.14 and 18.25).



$$\rho_V = \frac{EI_{tir}}{EI_{arc}}$$

$$\rho_H = \frac{(m+1)\,l^2}{8} \cdot \frac{EA}{EI_{arc}}$$

$$N_{cr} = C \cdot \frac{EI_{arc}}{l^2}$$

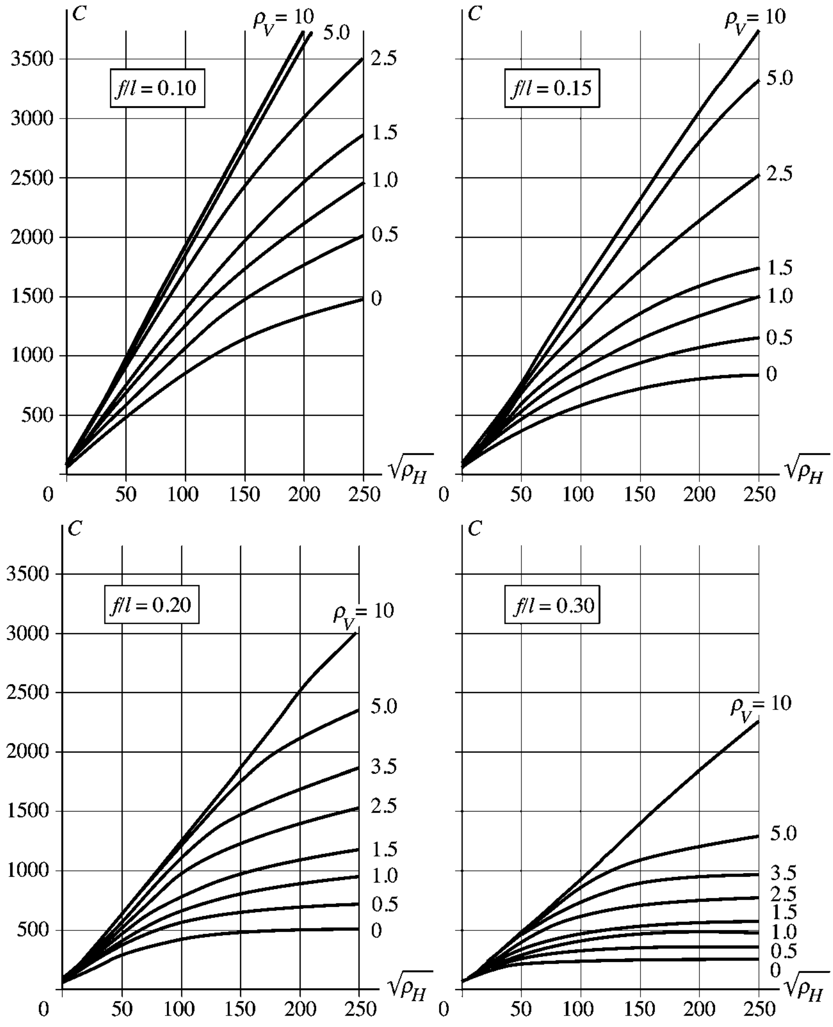


Fig. 18.30 Elastic buckling load for a parabolic tied arch subject to a uniformly distributed deck load, and as a function of the member stiffnesses [18.9].

18.6 Verifications (ULS)

18.6.1 Position of Traffic Loads

Figure 18.31 shows the load positions that should be considered to obtain either the maximum normal force (Fig. 18.31(a)), or the maximum bending moment (Fig. 18.31(b)), in the arch. In the first case, the uniformly distributed load is placed over the complete span of the arch. For the second, two options must be evaluated:

- asymmetric load, over half the span of the deck,
- symmetric load, over a certain length l^*, which depends on the influence line for bending moment in the middle of the arch.

The forces in the hangers and the elements of bracing may be determined by placing the loads according to the influence lines of the elements being considered. To determine the influence lines for the deck, the bending stiffness of the arch must be taken into account as well as the axial stiffness of the hangers. The deck must not be modelled simply as a continuous beam either on fixed supports or on springs representing the hangers.

In addition to the hazard scenarios associated with the use of the bridge, certain specific accidental situations must be considered, such as the rupture of a hanger due to traffic impact for a bridge with a lower deck.

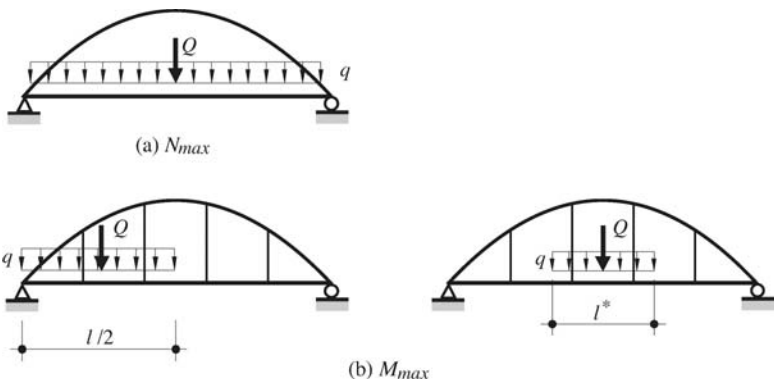


Fig. 18.31 Traffic load positions resulting in (a) maximum normal force and (b) maximum bending moment, respectively.

18.6.2 Verifications of the Arch

Once the buckling length of the arch is known (§ 18.5.3), checking the arch for stability may be carried out, both in plane and out of plane, by modelling the arch as a straight bar of the same buckling length using the various formulae given, for example, in SIA 263 or Eurocode 3.

For example, checking the resistance of the (arch) cross section may be undertaken using the interaction formulae proposed in the standards for a bar in compression and bending (TGC Vol. 11, § 12.4.1). Reducing the effective cross section of the arch, to allow for local buckling, should be considered if the conditions noted in Chapter 12 are not satisfied (see also Table 5a of the standard SIA 263).

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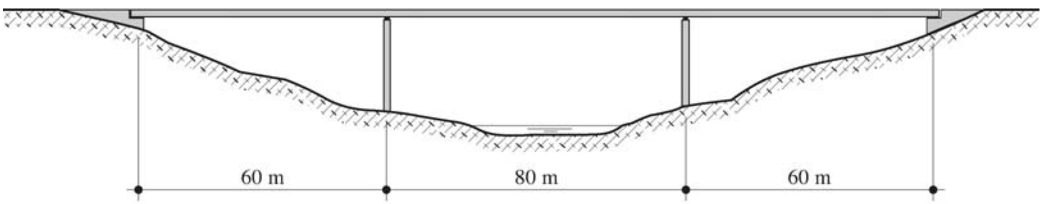
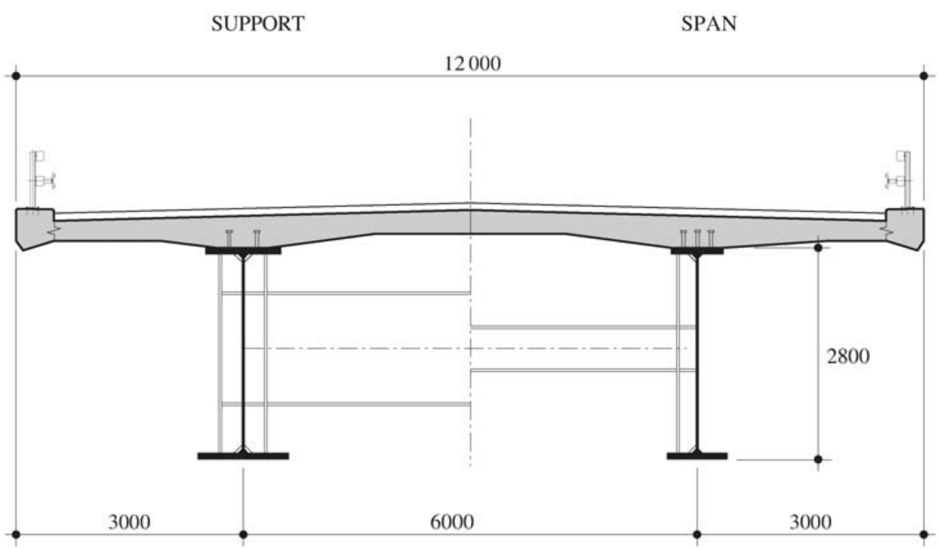
19 Example of a Composite Bridge





19.1 Introduction

This numerical example presents the design of a twin-girder composite bridge. The aim of the example is to illustrate the different calculation steps that are needed to check structural safety and serviceability of a composite bridge. These steps are carried out as described in the earlier chapters of this book that deal with structural analysis and design.

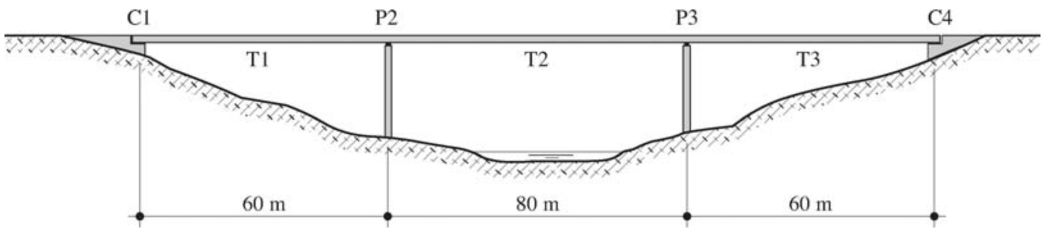


Fig. 19.1 Elevation of the example bridge.

The structure considered in this example is a twin-girder composite bridge, with a standard configuration for this type of structure. It is taken from guidance produced by SETRA [19.1], adapted to the principles of design covered in this book. The example bridge has a simple geometry (the bridge is straight and without a skew), which limits the complexity of the calculations. Following a description of the bridge, its use and the materials adopted, one way to carry out the preliminary design will be shown in Section 19.3.

Section 19.4 shows the hazard scenarios and actions taken into consideration. Section 19.5 describes both the structural analysis and the effects due to concrete cracking in tension. It also highlights the consequences of choosing a particular method for constructing the slab on the distribution of internal moments and forces. Section 19.6 covers the structural safety checks for the most highly stressed cross sections in span and over an intermediate support. Design of the steel-to-concrete shear connection, which is carried out taking into account both elastic and plastic distributions of longitudinal shear, is presented in Section 19.7. Sections 19.8 and 19.9 are dedicated to checking fatigue safety and serviceability, respectively.

Iterative aspects of design are not covered by this example, with checks being carried out only for the final choice of beam size. Structural analysis is limited to calculations for the superstructure, and only covers hazard scenarios where traffic is the leading action. The structure is checked both during construction and in its final state.

The emphasis of this example is on the overall process of design, with its various phases, and the numerical calculations associated with structural analysis and the various detailed checks. Numerous references are given to explanations and equations presented in previous chapters.

19.2 Description

19.2.1 Type of Use

The bridge is to carry two lanes of road traffic, travelling in opposite directions, as well as two footpaths for pedestrians and cyclists. It is not open to exceptional traffic. The design life for the load carrying structure is 70 years.

19.2.2 Elevation and Plan

An elevation of the example bridge is shown in Figure 19.1. Its total length is 200 m, split into three spans of 60 m, 80 m and 60 m. The deck is horizontal with a precamber of 120 mm in the central span and 100 mm in the end spans. It is straight in plan, with no skew.

19.2.3 Typical Cross Section including Concrete Slab

Figure 19.2 shows typical cross sections for the composite bridge at a support and in span. The rolling surface comprises two traffic lanes, each 3.5 m wide, and a 2.0 m wide footpath on each side of the traffic lanes. A 0.5 m wide edge beam at each side completes the slab, giving a total width of 12.0 m. For this example the edges, which support the crash barriers, are assumed to be non-structural. The profile of the slab rises to a central peak, with falls of 2.5% to facilitate water run-off.

The slab cantilevers out by 3.0 m, with a spacing between the main beams of 6.0 m. The slab is 425 mm thick above the main beams, 300 mm thick at its centre, and 240 mm thick at the cantilever ends. This means the average thickness h_c is 350 mm. There is 0.75% longitudinal reinforcement in span and 1.5% at the supports.

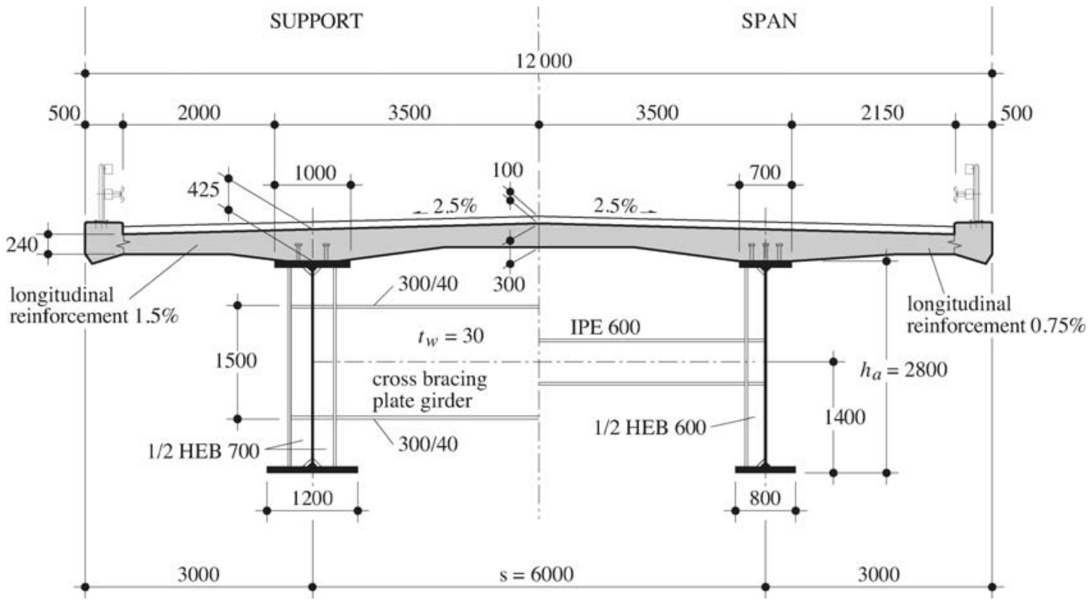


Fig. 19.2 Typical cross sections at a support and in span.

19.2.4 Main Beams

Typical cross sectional dimensions for the main beams are shown in Figure 19.2. The steel beam depth h_a is constant over the whole length of the structure, at 2.8 m. The width and thickness of the flanges, as well as the web thickness, vary along the length of the beams, as shown in Figure 19.3.

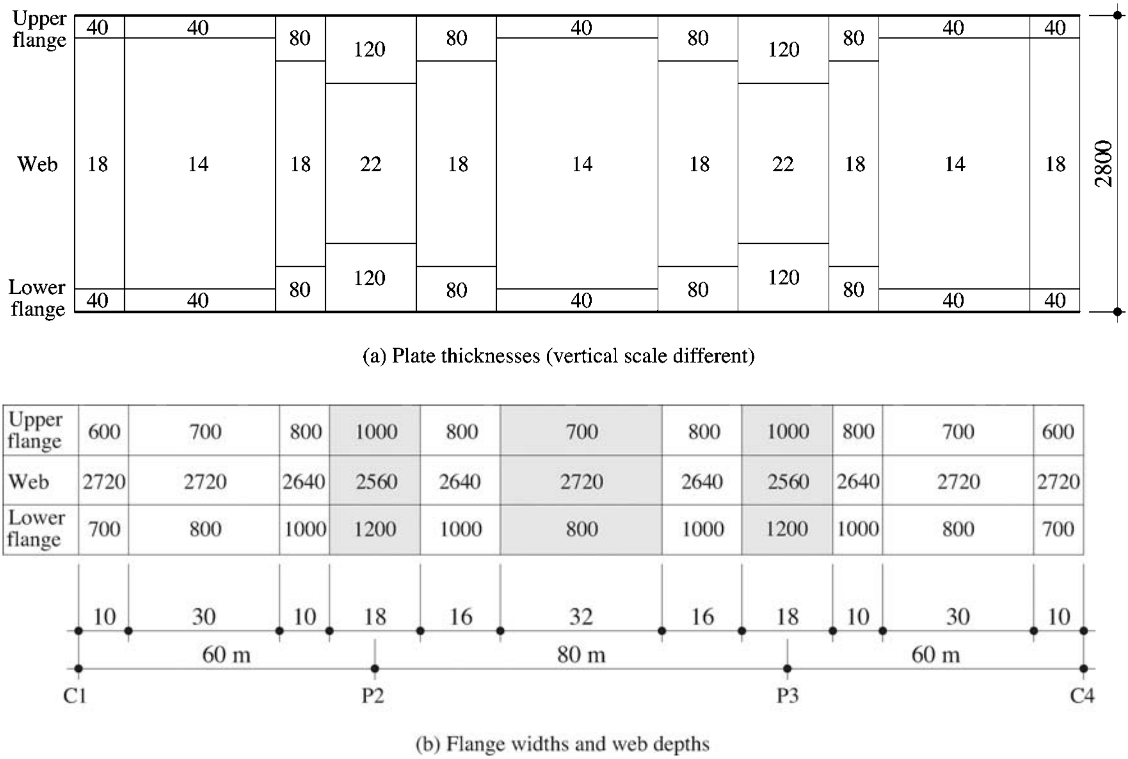


Fig. 19.3 Final dimensions for the elements of the plate girders.

Table 19.4 summarises the second moments of area I_a (steel) and I_b (composite) and elastic moduli W_{inf} for the lower flange, giving values both above the pier P2 and in the middle of the central span at position T2. Values are given for the bare steel (total section) and for different values of modular ratio n_{el}. The average second moment of area for the composite cross section (an average that is weighted according to the lengths of particular sections, for short term loading) along the axis of the beam is $I_b = 477 \cdot 10^9$ mm^4.

Table 19.4 Key properties for the steel and composite cross sections.

Cross Section	Modular Ratio n_{el}	Cross Section over Pier P2 I [10^9 mm^4]	Cross Section over Pier P2 W_{inf} [10^6 mm^3]	Cross Section at Mid Span T2 I [10^9 mm^4]	Cross Section at Mid Span T2 W_{inf} [10^6 mm^3]
Steel alone	–	502	405	137	104
Steel + reinforcement	–	583	421	–	–
Composite, short term	6.0	997	470	359	137
Composite, long term	18.0	749	446	288	130

19.2.5 Cross Bracing and Stiffeners

The steel beams are cross braced at the abutments, the intermediate supports, and in span using frame type cross bracing. The sets of **cross bracing** are at **7.5 m** centres in the end spans, and **8.0 m** centres in the central span. Figure 19.2 shows the cross bracing at the supports and in span. At the supports the cross girders comprise a plate girder with a depth of 1.5 m, whilst in span a rolled section IPE 600 is used. The girders are located at mid-depth of the main beams. At the supports, the uprights of the cross bracing comprise two half HEB 700 sections, which are welded to either side of the webs of the main beams. The uprights in span comprise a half HEB 600, which is welded to the inside face of the main beam web. The intermediate vertical **stiffeners** are flat plates, FLB 30 × 350, welded to the inside of the beams at **4.0 m** centres.

19.2.6 Plan Bracing

Temporary plan bracing is used during erection. This comprises a truss formed from the main beams (chords), the cross girders (uprights), and diagonals (LNP 200 × 16) located in the plane of the cross girders. In the final state the concrete slab fulfils the function of plan bracing.

19.2.7 Steel-to-Concrete Connection

At the supports, two rows of **22 mm diameter shear studs** are welded to the upper flange of each beam. The studs are at 275 mm centres, giving 7.3 studs per metre length of beam. In span, in the regions where plastic deformations may occur, three rows of 22 mm diameter studs are welded to the upper flanges. These studs are at 170 mm centres, giving 18.0 studs per metre length.

19.2.8 Fabrication and Erection

The structural steelwork members are fabricated in a workshop, transported to site then lifted into place. The slab is cast in situ using mobile formwork that moves along the steel beams. In this example two methods for concreting the slab are studied: concreting sequentially (Fig. 19.5(a)), and concreting the mid-span sections before the pier sections (Fig. 19.5(b)). In both cases the individually concreted sections have a length of 25 m.

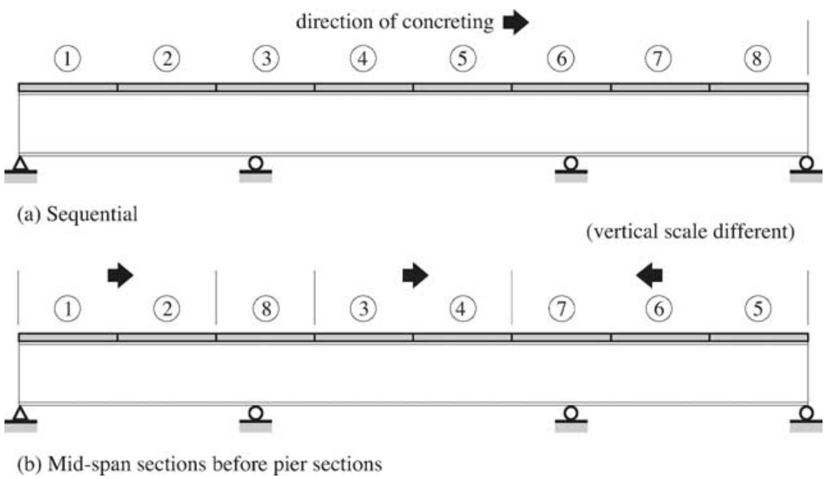


Fig. 19.5 Placing the concrete (in 25m-long pours).

19.2.9 Materials

The main beams, stiffeners and cross bracing are formed from S355J2W+N grade steel (weathering steel). The yield strength of the individual steel plates depends on the thickness (Table 4.7). The steel used for the shear studs is cold drawn grade S235J2, which has an ultimate strength of $f_{u,D}$ = 450 N/mm^2 (§ 13.5.2).

The slab comprises grade C35/45 concrete, in accordance with the standard SIA 262. Its characteristic resistance in compression is f_{ck} = 35 N/mm^2, its average elastic modulus E_{cm} = 35 kN/mm^2 (k_E = 10 000), and its average shear modulus $G_c = E_{cm}/(2(1+\nu)) = 35/(2\ (1 + 0.2)) = 14.6$ kN/mm^2. Creep is taken into account by reducing the value of the elastic modulus (§ 13.4.2): $E_{c\phi} = E_{cm}/3 = 11.7$ kN/mm^2. This means that the elastic modular ratio $n_{el} = E_a/E_c$ varies depending on the nature of the loads acting on the composite section (TGC Vol. 10, § 4.7.2):

- for short term loading : $\boldsymbol{n_0} = E_a/E_{cm} = 210/35 = \mathbf{6}$
- for shrinkage : $\boldsymbol{n_s} = E_a/E_{cs} = 210/(35/2) = \mathbf{12}$
- for long term loading : $\boldsymbol{n_\phi} = E_a/E_{c\phi} = 210/(35/3) = \mathbf{18}$

The plastic modular ratio n_{pl} (TGC Vol. 10, § 4.7.2), for design values of f_{yd} = 355/1.05 = 338 N/mm^2 (for a 40 mm thick flange in span) and $f_{cd} = 0.85 \cdot 35/1.5 = 19.8$ N/mm^2, is given by:

- $\boldsymbol{n_{pl}} = f_{yd}/f_{cd} = 338/19.8 = \mathbf{17.05}$

Reinforcing steel is grade B500B. This has a yield strength of f_{sk} = 500 N/mm^2, with a design value of f_{sd} = 500/1.15 = 435 N/mm^2. The plastic modular ratio relative to the reinforcement is given by:

- $\boldsymbol{n_{pls}} = f_{yd}/f_{sd} = 338/435 = \mathbf{0.777}$.

19.3 Preliminary Design

Preliminary design comprises defining – in a way that is simple, quick and informed by experience – the dimensions of the most highly stressed cross sections of the steel beams, in order to give an initial choice of the characteristics of these sections. This choice is needed in order to progress to a structural analysis that will determine the internal moments and forces corresponding to the static behaviour of the bridge. For the preliminary design of the main beams it is normally sufficient to analyse the load case that has traffic as a leading action, combined with the permanent loads.

The results of this initial calculation of internal moments and forces allow verification of the preliminary dimensions chosen for the beams (and for these to be modified if necessary); the dimensions of other elements are then determined, with the internal moments and forces subsequently recalculated, and so on as the design progresses. In this iterative way it is possible to converge towards the optimum dimensions of the structural members, from a structural safety point of view. The dimensions must subsequently be checked to ensure they also satisfy all requirements relative to fatigue safety and serviceability. For this example, the results of such a procedure correspond to the distribution of materials as shown in Figure 19.3 for the main beams.

Within the context of preliminary design and this example, the ways in which to define the dimensions of a cross section in the middle of the central span, as well as at an intermediate support, are illustrated.

19.3.1 Depth of the Steel Beams

The depth of the steel beams may be defined using Equation (5.1) to calculate the slenderness l/h as a function of the span l and slab width $2b$. For the central span this gives:

$$\frac{l}{h} = 20 + \frac{l-30}{5} - \frac{2b-12}{2.5} = 20 + \frac{80-30}{5} - \frac{12-12}{2.5} = 30$$

This slenderness of 30 corresponds to a beam depth of 2666 mm. For this example, we have chosen a slightly deeper section of 2800 mm, corresponding to a slenderness of 28.5 for the central span. The depth is constant over the total length of the bridge.

The choice of beam depth affects the dimensions of the flanges; the greater the depth the smaller the area of the flanges will be, and vice versa. In order to define an optimal beam depth, with respect to beam weight, it would be necessary to consider several designs with different depths. However, in practice, the beam depth initially chosen is normally retained throughout the design process, and the dimensions of the flanges, as well as the web thickness, are adapted to suit the magnitude of the internal moments and forces.

19.3.2 Actions

Self-weight

For preliminary design it is justified to use a simplified approximation of the vertical actions on the steel beams in order to estimate the internal moments and forces. Because the dimensions of the sections and the distribution of material along the length of the beams are not yet known, the initial analysis of the structure can only take place by making certain assumptions, such as constant inertia over the length of the beams.

- The self-weight of the steelwork may be estimated using, for example, Equation (10.1), which in our case gives:

$$g_a = 0.1 + \frac{0.02 l_m}{0.6 + 0.035(2b)} = 0.1 + \frac{0.02 \cdot 68}{0.6 + 0.035 \cdot 12} = 1.43 \text{ kN/m}^2$$

- For a single beam, the self weight of the steelwork is therefore: $\boldsymbol{g_a}$ = 6.0 m · 1.43 kN/m^2 **= 8.6 kN/m**.
- The self weight of the slab may be calculated assuming an average thickness of 350 mm, which for a single beam results in: $\boldsymbol{g_c}$ = 0.35 m · 6.0 m · 25 kN/m^3 = **52.5 kN/m**.
- The self weight of the non-structural elements comprises that of the road surfacing plus that of the crash barriers. For surfacing that is 100 mm thick, over a width of 11.0 m, this gives a value for a single beam of: $\boldsymbol{g}$ **= 13.2 kN/m**.
- The self weight of the crash barriers is assumed to be **1.0 kN/m** per beam.

Traffic Loads

Traffic load model 1 (§ 10.3.1) is placed on the rolling surface in a position that is most unfavourable, both longitudinally and transversally, for the moments or forces under consideration. Transversally the traffic load acting on a beam is determined using a transverse influence line (§ 11.5.2). It should be remembered that for this type of bridge, with an open cross section, the beams resist torsional loads (torque), coming from eccentrically placed traffic, in non-uniform torsion. The influence line allows an equivalent load to be defined, one that only loads the beam in bending and results in the same normal stresses in the cross section as those resulting from the non-uniform torsional resistance plus bending. For preliminary design, as a simplification, one may adopt a transverse influence line with ordinates of 0.9 and 0.1 above the beams (Fig. 11.23).

Load model 1 placed in a transversally unfavourable position (Fig. 11.26) defines the distributed and concentrated loads that act on a beam. Using a calculation procedure similar to that presented for final design (§ 19.5.1) one finds that: $\boldsymbol{q_k}$ **= 30.9 kN/m** and $\boldsymbol{Q_k}$ **= 786 kN** (the loads from the two axles, at 1.2 m spacing longitudinally, are added for the preliminary design).

19.3.3 Cross Section Dimensions

The design values of the bending moments at the supports and in span are used to obtain a first estimate of the flange dimensions of the steel beams during preliminary design of a composite bridge. First, the sum of the bending moments due to self-weight and traffic is divided by the depth of the beam. The resulting normal force, when divided by the design strength of the steel, then allows an initial value of flange area to be determined. During this preliminary design phase the experience of the engineer is a useful guide to adjusting the calculated bending moments or steel strength to suite the peculiarities of a given situation.

For example, the engineer might already allow for the fact that, under loads acting on the steelwork alone, the bending moments at the supports will in reality be greater than those calculated assuming constant beam stiffness, because the definitive steel beams will inevitably have larger dimensions at the

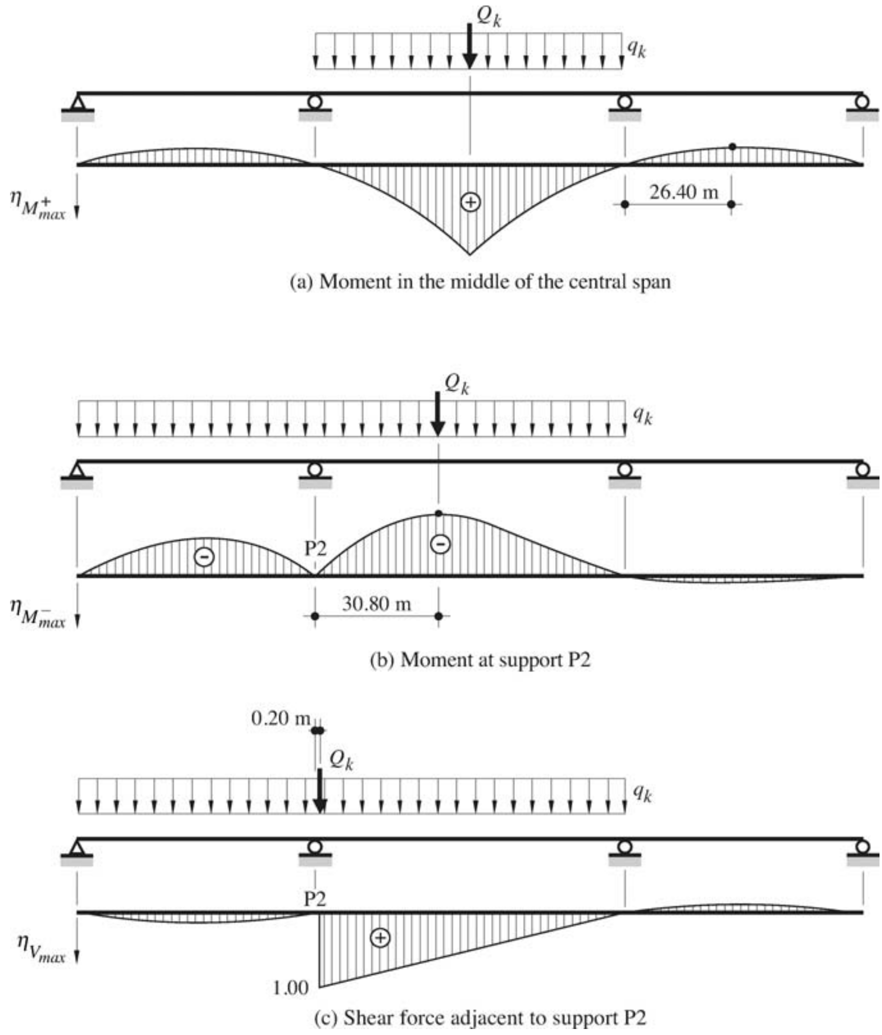


Fig. 19.6 Longitudinal influence lines and positioning of the traffic loads.

supports than in span (Table 5.10). He may reduce the steel strength by anticipating that the flange thickness will be greater than 40 mm. When determining the size of compression flanges he may assume a steel resistance less than yield to allow for lateral torsional buckling. He may also take into account some contribution from the web to the bending resistance in span if he is planning on using a plastic resistance model.

To calculate the web thickness in span, a first estimate may be made by considering vertical buckling of the compression flange into the web (§ 12.2.2), respecting a minimum web thickness of 10 mm. For a first estimate of web area at the supports the maximum shear force may be divided by a shear strength τ_R of around 100 to 130 N/mm^2.

For this example, with values for the actions as given in Paragraph 19.3.2, with load factors of 1.35 for self weight and 1.50 for traffic, and for traffic loads placed in an unfavourable position longitudinally (see Fig. 19.6), the following values are obtained.

Moments in the Middle of the Central Span

- due to loads on the composite beam: $\boldsymbol{M_{b,Ed}}$ **= 35 996 kNm**[1],
- due to loads on the steel beam: $\boldsymbol{M_{a,Ed}}$ **= 29 920 kNm**,
- due to loads on the steel beam at the construction stage: $\boldsymbol{M_{a,Ed}}$ **= 32 941 kNm**.

Moments at an Intermediate Support

- due to loads on the composite beam: $\boldsymbol{M_{b,Ed}}$ **= – 34 688 kNm,**
- due to loads on the steel beam: $\boldsymbol{M_{a,Ed}}$ **= – 51 384 kNm**,

Shear Force

- Immediately to the right of the first intermediate support: $\boldsymbol{V_{Ed}}$ **= 7256 kN**.

Cross Section in the Middle of the Central Span

As an initial estimate of the web thickness, $\boldsymbol{t_w}$ **= 12 mm** is obtained using Equation (12.6) with S355 steel and h_f taken as the total beam depth:

$$t_w \geq \frac{h_f}{240} = \frac{2800}{240} = 11.7\text{mm}$$

The cross-sectional area of the *lower flange* is calculated starting from the assumption that the calculated bending moment acting on the steel beam is a little too big because, during final design, the steel section at the supports will be larger than that in span. The initial value of moment that has been calculated could, for example, be reduced by at least 15%. Also, the design value of steel yield strength f_{yd} is equal to 335/1.05 = 319 N/mm^2 for thicknesses in excess of 40 mm. The area of the lower flange is therefore calculated as follows:

$$A_{f,\,inf} = \frac{M_{b,\,Ed} + 0.85 M_{a,\,Ed}}{h \cdot f_{yd}} = \frac{(35\,996 + 0.85 \cdot 29\,920)10^6}{2800 \cdot 319} = 68\,773 \approx 68\,800 \text{ mm}^2$$

[1] For this example, results are given with excessive precision so that the reader can more easily spot values elsewhere in the text. In practice one would round the values to 3 or 4 significant numbers, systematcially expressing them as values times 10^3 or 10^6.

Because plastic resistance of the section is going to be considered, it is possible to deduct the contribution of the part of the web in tension from the area calculated above. Although this contributing area is not yet known one may, for example, deduct an area of 16 800 mm^2, corresponding to half the web depth. This leaves a required area for the lower flange of 52 000 mm^2. A flange of **800** × **65 mm** provides this area.

For preliminary design purposes the cross-sectional area of the *upper flange* is based on consideration of the construction stage, with loads on the steel beam alone and recognising that during construction of the slab the upper flange in compression may buckle laterally (lateral torsional buckling of the beam). It is possible to reduce the initial estimate of moment acting on the steel beam in span by 15%, and one may take into account a lower steel strength due to lateral torsional buckling of, for example, $\sigma_D = 300$ N/mm^2.

$$A_{f,sup} = \frac{0.85 M_{a,Ed}}{h \cdot \sigma_D / \gamma_a} = \frac{0.85 \cdot 32941 \cdot 10^6}{2800 \cdot 300/1.05} = 35\,000 \text{ mm}^2$$

A flange of **700** × **50 mm** corresponds to this area.

Cross Section at an Intermediate Support

As an initial estimate of the web thickness $\boldsymbol{t_w}$ **= 22 mm** is obtained by dividing the shear force by the depth of beam and assuming a resistance in shear of, for example, $\tau_{Rd} = 120$ N/mm^2.

$$t_w = \frac{V_{Ed}}{h \cdot \tau_{Rd}} = \frac{7256 \cdot 10^3}{2800 \cdot 120} = 21.6 \text{ mm}$$

The area of the *lower flange* is calculated taking into account the considerations made for the flange in span. This means that the initial estimate of bending moment acting on the steel beam alone is increased by 15%. The compression flange is also susceptible to lateral buckling (lateral torsional buckling of the beam):

$$A_{f,inf} = \frac{M_{b,Ed} + 1.15 M_{a,Ed}}{h \cdot \sigma_D / \gamma_a} = \frac{(34\,688 + 1.15 \cdot 51\,384) \cdot 10^6}{2800 \cdot 300/1.05} = 117\,225 \text{ mm}^2$$

A flange of **1000** × **120 mm** provides this area.

The area of the *upper flange* corresponds to that of the lower flange, and the slab reinforcement (1.5% of the area of the slab = 31 500 mm^2) can potentially also be taken into account for some design checks, namely those associated with the composite cross section. However, during this phase of the preliminary design it is difficult to determine the force that will be carried by the reinforcement, so its contribution is conservatively ignored. The area of the steel flange can therefore be calculated using the equation given above, but with a design value of steel yield strength $f_{yd} = 319$ N/mm^2. The calculated area is approximately 10 500 mm^2, which corresponds to a flange of **1000** × **105 mm**.

Summary of the Preliminary Design

Table 19.7 summarises the dimensions of the steel cross sections obtained from the preliminary design. Comparing these values with those given by the detailed design (Fig. 19.3) it can be seen that the section at the supports predicted by the preliminary design is insufficient, whilst the opposite is true for the section in span. These differences are largely due to the initial calculation of the internal moments and forces assuming constant beam stiffness, whereas during detailed design the stiffness is varied as a function of those moments and forces. The bending moments given by the structural analysis (§ 19.5.3) are greater at the supports and smaller in span than those considered during preliminary design. That said, the dimensions given by the preliminary design, using simple assumptions, already give a good idea of the final sizes of the steel beams.

Table 19.7 Cross section dimensions obtained from the preliminary design.

	Mid-span	Support
Upper flange	700 × 50	1000 × 105
Web	12 × 2800	22 × 2800
Lower flange	800 × 65	1000 × 120

19.4 Hazard Scenarios and Actions

For this example we illustrate the design of a composite beam covering the various checks that are associated with it. The hazard scenarios and limit states for checking of structural safety and serviceability are first of all described, with their corresponding load cases. The calculation of values for the various actions that need to be considered for the structural analysis of the composite beam is then illustrated.

19.4.1 Hazard Scenarios and Limit States

Structural Safety

Tables 19.8 and 19.9 present the different hazard scenarios to be considered when checking structural safety of the bridge superstructure. The ultimate limit states associated with the detailed design of the superstructure are of type 2 (cross section resistance, collapse mechanism of the load carrying structure) and type 4 (fatigue resistance of the load carrying structure).

Table 19.8 Hazard scenarios for structural safety checks during erection.

Hazard Scenario	N°	Permanent Actions		Leading variable Action		Accompanying variable Action		Design Options
		G_k	γ_G	Q_{k1}	γ_Q	Q_{ki}	y_{0i}	
Failure of steel beams	1	Steelwork Concrete (wet and hardened)	1.35	Construction load	1.50	Temperature	0.60	Structural design

Table 19.9 Hazard scenarios for structural safety checks in the final state.

Hazard Scenario	N°	Permanent Actions		Leading variable Action		Accompanying variable Action		Design Options
		G_k	γ_G	Q_{k1}	γ_Q	Q_{ki}	ψ_{0i}	
Failure of composite beams	2	Steelwork and concrete Shrinkage (if unfavourable)	1.35 1.00	Traffic model 1	1.50	Temperature	0.60	Structural design
Fatigue failure of the steelwork's elements	3	–		Traffic model 1, only concentrated loads	1.00	–		Good fatigue design of construction details

Serviceability

The serviceability checks illustrated in this example only concern the composite beam. The serviceability limit states SLS that are considered are summarised in Table 19.10.

Table 19.10 Limit states for serviceability checks in the final state.

Serviceability Limit State	N°	Permanent Actions		Leading variable Action		Accompanying variable Action		Design Options
		G_k	γ_G	Q_{k1}	ψ_{11}	Q_{ki}	ψ_{0i}	
Comfort of users	4	–	–	Traffic model 1	0.75	–		Deflection limitation
Appearence of the structure	5	Steelwork and concrete Shrinkage (if unfavourable)	1.00 1.00	–		–		Deflection limitation, precamber, control of cracking
Elastic behaviour of the load carrying structure	6	Steelwork and concrete Shrinkage (if unfavourable)	1.00 1.00	Traffic model 1	0.75	Temperature	0.60	Structural design

19.4.2 Actions

Self-weight of the Steel Beams

The cross section of the steel beams varies along the length of the bridge according to the final distribution of material, as illustrated in Figure 19.3. The self-weight of the structure is calculated using these dimensions and the density of the steel, which gives a weight per unit volume of 78.5 kN/m^3. The weight of the cross bracing and stiffeners is allowed for by simply multiplying the weight of the main beams by 1.10. The weight of temporary plan bracing used during erection is ignored.

Self-weight of the Concrete Slab

The cross-sectional dimensions of the concrete slab are shown in Figure 19.2. These dimensions are constant along the length of the bridge and result in a slab cross-sectional area, including the edge beams, of $A_c = 4.26$ m^2. The weight per unit volume of reinforced concrete is taken as 25 kN/m^3 (note that this value is not the same in all codes and standards, and varies depending on whether the concrete is wet or dry), so the self-weight of the slab is 106.5 kN/m, or g_c **= 53.2 kN/m** per beam.

Self-weight of the Non-structural Elements

Non-structural elements comprise the surfacing, crash barriers and ductwork (the self-weight of the non-structural concrete edge beams is included within that of the slab). Characteristic values are given in Paragraph 10.2.2. For this example the surfacing has a thickness of 100 mm and a weight of 26.4 kN/m, given the width of 11 m and weight per unit volume of 24 kN/m^3. The self weight of each crash barrier is estimated as 1.0 kN/m. The weight of the ducts and the water flowing through them is assumed to be negligible. The self-weight of the non-structural elements per beam is therefore given by:

g = 26.4 kN/m/2 + 1.0 kN/m = **14.2 kN/m**

Shrinkage

For sections that are designed assuming elastic behaviour, the compressive stress in the lower flange of the cross section at intermediate supports is taken as 25 N/mm^2 (§ 13.2.2). This level of stress corresponds to a shrinkage of ε_{cs} = **0.025%**. The normal force resulting from this shrinkage must be anchored, using shear connectors, at the ends of the composite beam. The value of this force can be determined using Equation (13.3). Checking of the shear connectors at the bridge ends is not included in this numerical example.

Traffic

Actions due to road traffic are determined in accordance with Paragraph 10.3.1. For this example, the width of the slab surface available to traffic is 11.0 m. The number n of notional lanes is 11 m/3 m, given to the nearest whole number = 3.

The route on the bridge is not open to exceptional traffic, meaning that load model 3 does not have to be considered. Horizontal forces due to acceleration and braking are ignored when checking the superstructure. This means that only load model 1 need be considered. Individual loads take the following values (Fig. 10.2):

- q_{k1} **= 9.0 kN/m^2**
- $q_{k2} = q_{k3} = q_{kr}$ **= 2.5 kN/m^2**
- Q_{k1} **= 300 kN**
- Q_{k2} **= 200 kN**

The coefficients α_i are equal to: $\alpha_{qi} = \alpha_{Qi}$ **= 0.9**.

Wind

The wind forces are calculated using the standard SIA 261, in particular its annexes C and E. This standard allows the horizontal q_1 and vertical q_3 forces on the deck to be calculated. It can be shown that for the bridge geometry considered in this example the vertical forces q_3 act upwards (coefficient $c_{f3} < 0$), so would unload the structure. Vertical wind forces are therefore not considered in this example. The horizontal wind forces are used to design the steel plan bracing for the erection phase, although this is not covered by the example.

Snow

Maximum snow loads cannot act simultaneously with traffic loads. Because traffic is heavier than snow, the latter will not govern design and so is not considered in this example.

Temperature

Temperature effects are taken into account according to Paragraph 13.2.3. A compressive stress of **–16 N/mm^2** acts in the webs of the main beams, a compressive stress of **–4 N/mm^2** in the lower flanges, and a tensile stress of **1.6 N/mm^2** in the concrete slab.

Construction Loads During Concreting of the Slab

These loads result from the self-weight of the moving formwork and any build up of wet concrete. They must be defined as a function of the type of formwork and the methods and controls put in place during concreting to prevent excessive build up of materials. For this example these actions are covered by considering a uniformly distributed load of 2 kN/m^2, which is a load of **11 kN/m** per beam, applied over the length of each concreting phase, namely a length of 25 m for this example.

Impact

The effect of a lateral vehicle impact against a crash barrier is covered by considering an equivalent static force acting at 1.15 m above the road surface. The characteristic value of this force is $\boldsymbol{Q_{0,y}}$ **= 600 kN** (Table 10.10). It is used to check the adequacy of the transverse reinforcement in the slab. This check is not covered by the example.

Earthquake

Seismic forces rarely govern the design of a bridge superstructure. This is not the case for the piers and supports where seismic forces may become predominant. Seismic forces are not covered by this example.

19.5 Structural Analysis

The structural analysis determines the internal moments and forces, using an analytical model that is representative of the real behaviour of the structure. For a composite beam bridge the internal moments and forces in the beams are determined using a first order elastic analysis. The traffic loads are placed on the bridge in positions that are unfavourable, both transversally and longitudinally. Transversally, for example, for the analysis of a twin-girder bridge one uses the transverse influence line to identify the most unfavourable position. Longitudinally the loads are placed unfavourably according to the influence lines corresponding to the different internal moments and forces being considered.

In this example of a composite bridge design, we will study in detail several approaches for taking into account concrete cracking above the intermediate supports when calculating bending moments. We will also study the effects of two different methods for in-situ casting of the concrete slab, namely concreting sequentially and concreting the mid-span sections before the pier sections.

19.5.1 Transverse Positioning of Traffic Loads

The transverse influence line for the loads is determined according to Paragraph 11.5.2, and particularly Equations (11.47) and (11.55). The effective width of deck is given by $2b_{eff}$ = 11.0 m from Equation (13.6), which corresponds to the width of the concrete slab (remembering that the edge beams are assumed to be non-structural).

G_c = G_{c0} = 14.6 kN/mm^2 (§ 19.2.9)
K_c = $1/3 \cdot 2b_{\text{eff}} \cdot h_c^3 = 1/3 \cdot 11\,000 \cdot 350^3 = 157 \cdot 10^9$ mm^4
E = E_a = 210 kN/mm^2
I_y = $I_b = 477 \cdot 10^9$ mm^4 (average value according to § 19.2.4)
l = $0.70 \cdot 80$ m = 56 m (distance between points of zero moment for a continuous beam)
s = 6.0 m (spacing of the main beams)

Use of Equation (11.47) gives $\alpha^2 = 1.0$ and therefore the following values for the ordinates of the transverse influence line for loads above the main beams: η_1 **= 0.85**, η_2 **= 0.15**. Strictly speaking one should determine a different transverse influence line for each span and each cross section (at supports, in span). However, in practice this distinction is rarely made and values for the main span are assumed to apply throughout the length of a bridge.

Using the transverse influence line one can determine the load per metre of beam q_k due to the distributed traffic loads, in accordance with Figure 19.11 and Equation (11.62):

$$q_k = \sum \eta_i \alpha_{qi} q_{ki} b_i = 0.97 \cdot 0.9 \cdot 9.0 \cdot 3.0 + 0.62 \cdot 0.9 \cdot 2.5 \cdot 3.0 + 0.27 \cdot 0.9 \cdot 2.5 \cdot 3.0 + 0.04 \cdot 0.9 \cdot 2.5 \cdot 0.79 = 29.6 \text{ kN/m}$$

The concentrated loads are considered in the same way, in accordance with Equation (11.61):

$$Q_k = \sum \eta_i \alpha_{Qi} Q_{ki} = (1.08 + 0.85) \cdot 0.9 \cdot 300 + (0.73 + 0.50) \cdot 0.9 \cdot 200 = 743 \text{ kN}$$

This concentrated load produces an effect on the beam, which represents that of the two groups of double axles as shown in Figure 10.2. The simplification to ignore the longitudinal distance of 1.20 m between the axles is conservative.

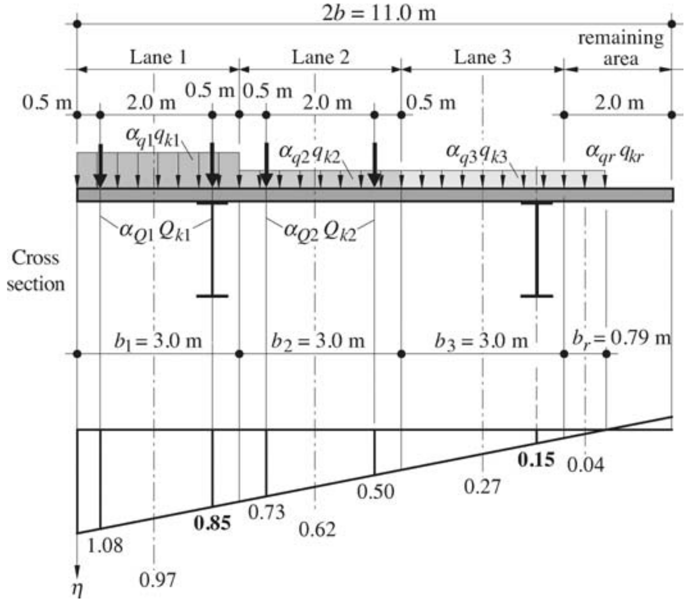


Fig. 19.11 Transverse influence line and positioning of the traffic loads.

19.5.2 Longitudinal Positioning of Traffic Loads

The most unfavourable positions of the traffic loads in the longitudinal sense are also determined using influence lines. These are shown in Figure 19.6 for different sections and different internal moments and forces. The unfavourable positions for the traffic loads for the internal moments and forces under consideration are also shown.

19.5.3 Internal Moments and Forces

Values of the internal moments and forces along the axis of the bridge depend on the stiffness of the different cross sections (at support, in span) and, for composite beams, the transverse cracking of the slab at intermediate supports. This cracking is a function of the concreting method. To take this cracking into account, diagrams of the forces acting in the composite beam are determined using a two-step calculation, such as that described at the beginning of Paragraph 13.3.3. Simplified methods for taking into account cracking, including the effects of concreting methods, are discussed in Paragraph 19.5.4.

The internal moments and forces shown in the various Figures are calculated using characteristic values of the actions described in Paragraph 19.4.2. The characteristic values q_k and Q_k of the traffic loads are those calculated in Paragraph 19.5.1.

Erection of the Steelwork

The internal moments and forces that result after erection of the steelwork are shown in Figure 19.12. The self-weight of the steel beam is calculated and positioned according to the distribution of material given in Figure 19.3. The value of bending moment shown at 8 m from mid-span will be used for the summation of moments and forces during concreting of the slab.

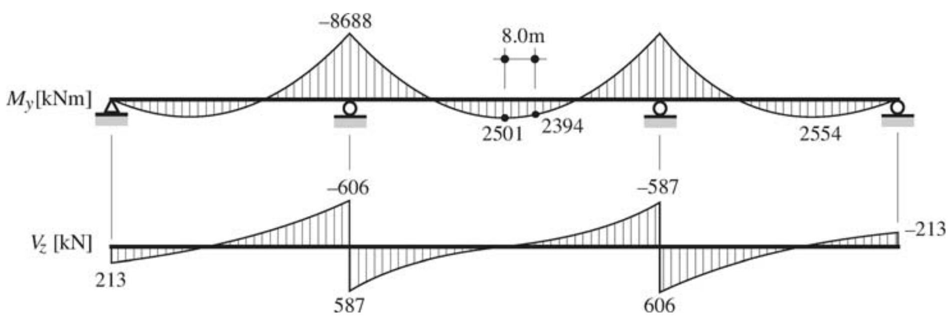


Fig. 19.12 Internal moments and forces due to the self-weight of the steel structure.

Concreting of the Slab

Figure 19.13 gives the internal moments and forces for concreting the mid-span sections before the pier sections. Figure 19.13(a) shows the maximum moment in the middle of the central span (stage 4 according to Figure 19.5(b)). The internal moments and forces due to the construction loads during the fourth stage are shown in Figure 19.13(b). The internal moments and forces after completing concreting of the whole slab are shown in Figure 19.13(c). The values shown in Figures 19.13(a) and (c) do not include the effects of the construction loads.

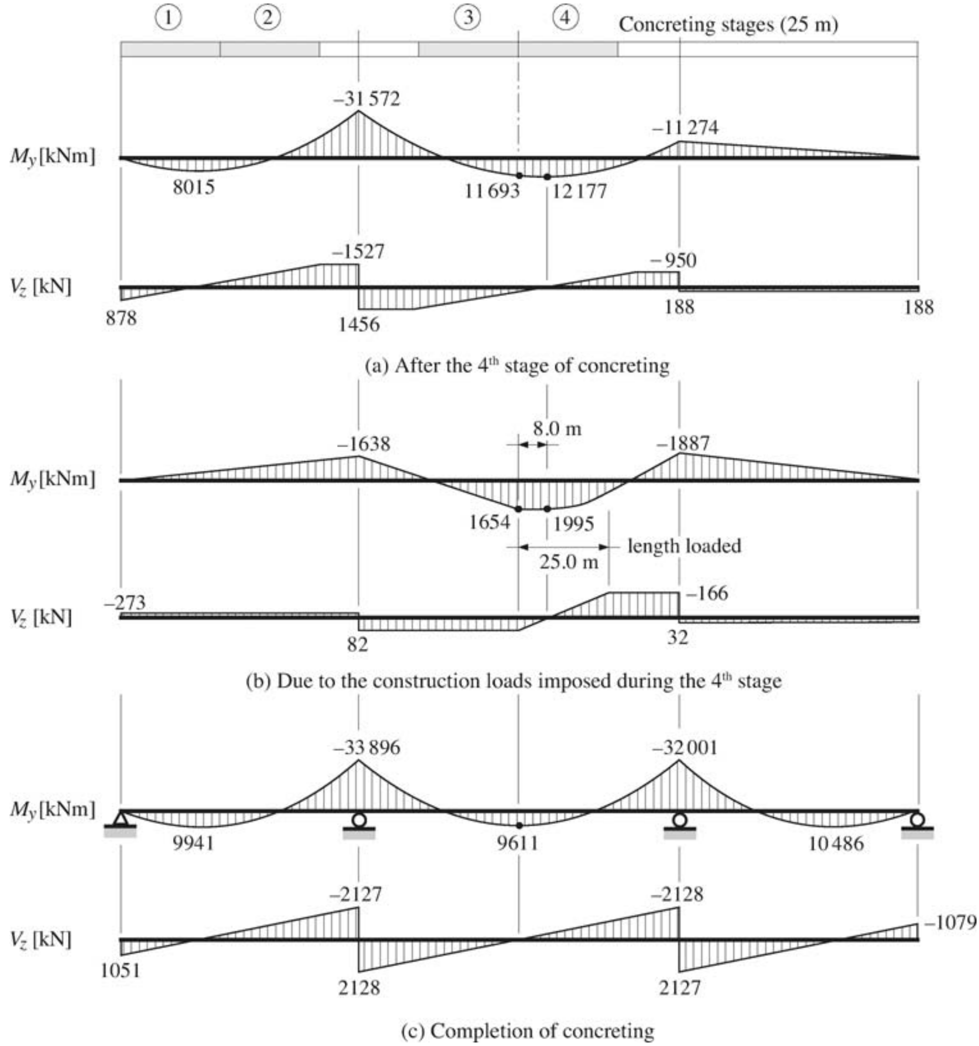


Fig. 19.13 Internal moments and forces due to the self-weight of the slab (mid-span sections concreted before pier sections).

Internal Moments and Forces in the Final State

Figure 19.14 shows the internal moments and forces due to the different actions on the bridge in its final state (in addition to its self-weight), namely the self-weight of the non-structural elements (Fig. 19.14(a)) and the traffic loads (Fig. 19.14(b)). The internal moments and forces due to traffic placed in an unfavourable position in the longitudinal sense correspond to the longitudinal load positions defined in Figure 19.6.

The effects of shrinkage and temperature are introduced directly into the design checks in the form of imposed stresses (§ 19.4.2). It is therefore not necessary to calculate distributions of internal moments and forces corresponding to these actions.

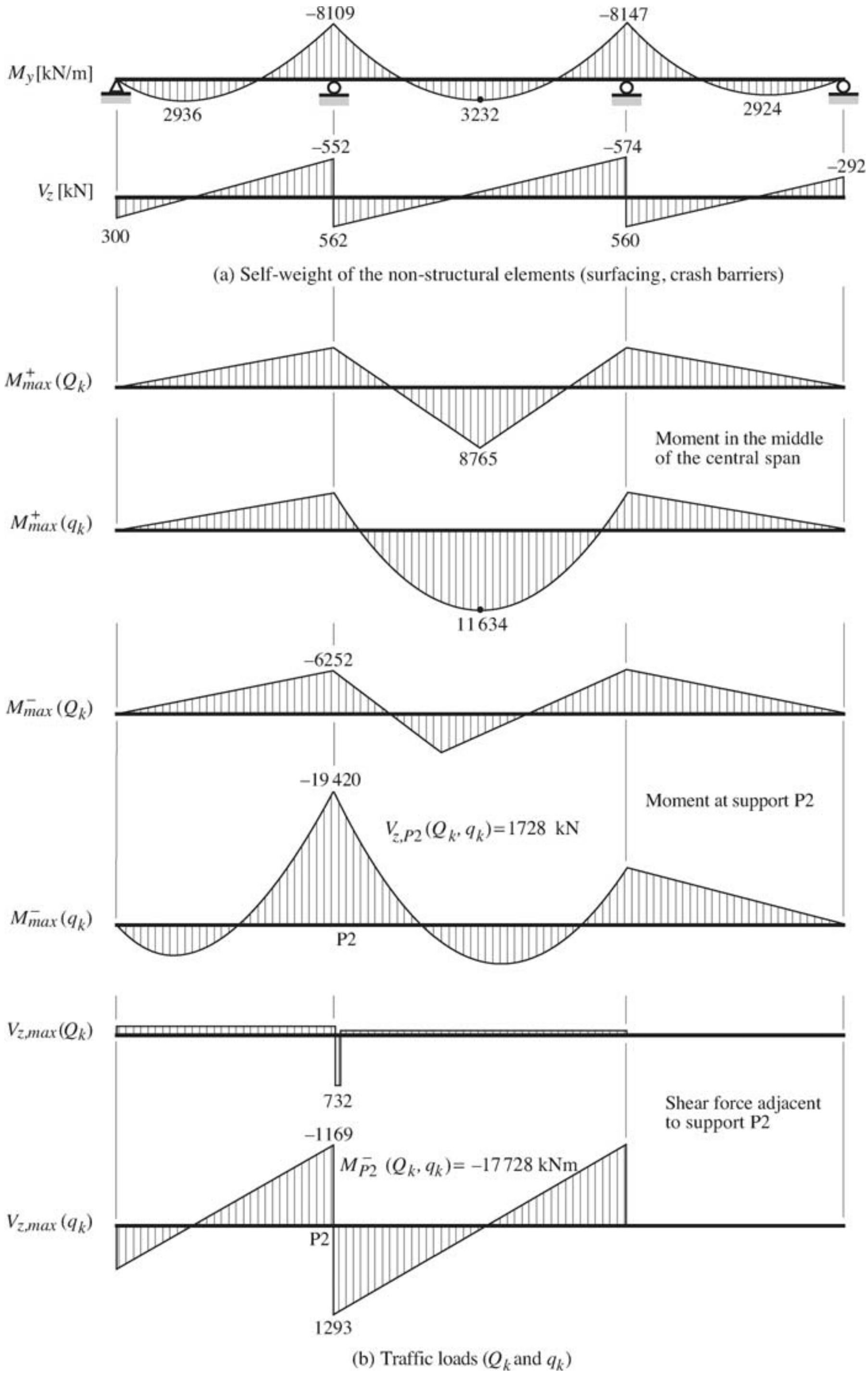


Fig. 19.14 Internal forces due to the self-weight of the non-structural elements and due to the traffic loads.

19.5.4 Cracking and Method of Placement of the Concrete

Effect of Concrete Cracking Above Supports

The influence of the calculation method (described in §13.3.3) used to take into account transverse cracking of the concrete slab above the intermediate supports (P2 and P3) is estimated by comparing the distributions of internal moments and forces. These comparisons are for loading due to the self-weight of the non-structural elements, because this loading is applied after concreting of the final stage. The conclusions drawn for this action are qualitatively equally valid for traffic loads. Three methods for taking into account cracking of the slab above the intermediate supports are compared:

- method 1: calculation necessitating two steps that is described as "exact", for concreting either sequentially (1a) or mid-span sections before pier sections (1b),
- method 2: a direct method of calculation with an assumed cracked slab length of $0.15l$ on either side of the intermediate supports (Fig. 13.9(a)),
- method 3: calculation that assumes an uncracked section and a default redistribution of 10% of the support moments into the spans (Fig. 13.9(b)).

The bending moments in the central span and at support P2 are summarised in Table 19.15 for each of these calculation methods. It can be seen that methods 2 and 3 give results that are almost identical and are good approximations for concreting sequentially. However, when the mid-span sections are concreted before the pier sections, the reduced cracking that results also reduces the redistribution of moments from the supports into the spans. For this method of concreting the simplified calculation methods (methods 2 and 3) do not give accurate results. Only the two-step calculation method (method 1) can predict the distribution of moments with accuracy.

Table 19.15 Influence of the calculation method on the bending moments.

		Method 1: "exact" calculation		Method 2	Method 3
Cross Section		Sequential concreting	Concreting mid-span sections before pier sections	Cracked slab length of $0.15l$	Redistribution of 10% of M
Central mid-span	[kNm]	3561	3252	3955	3943
	[%]	100	91	111	111
Support P2	[kNm]	7545	8147	7427	7417
	[%]	100	108	98	98

Effect of the Method of Placement of the Concrete

To study the influence of the method of placement of the concrete (Fig. 19.5) the distribution of stresses at different points in the cross section can be compared. This comparison is carried out under permanent loads, namely the self-weight of the steelwork, the concrete and the non-structural elements.

The stresses calculated for the two methods of concreting sequence considered in this example are presented in Figure 19.16. These stresses were derived using a complete two-step calculation to allow for concrete cracking at the supports.

It can be seen that concreting the mid-span sections before the pier sections reduces significantly the tensile stresses in both the reinforcement, and the concrete slab, above the intermediate supports. Consequently it is likely that the crack widths will be smaller than when sequential concreting is adopted, which

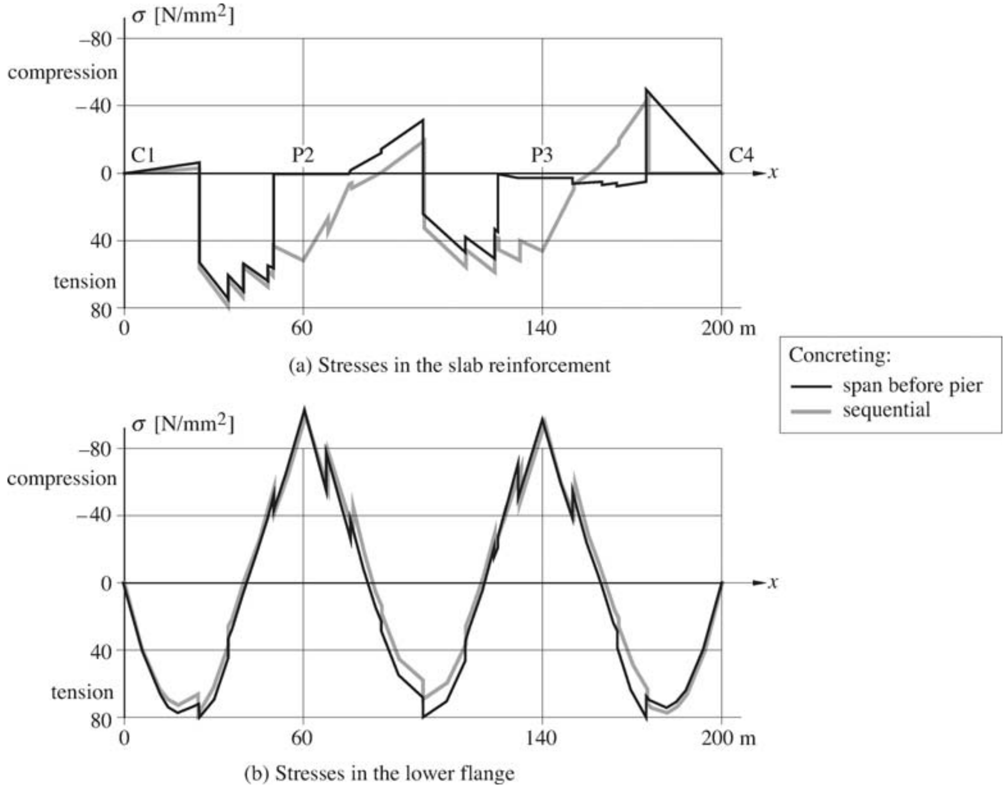


Fig. 19.16 Influence of the sequence of slab concreting on the stresses in the bridge.

is beneficial both for the appearance of the bridge and the durability of the slab when its surface may be in contact with water containing de-icing salts. The stresses in the lower flange are little influenced by the method used to place the concrete. The remainder of this example will only consider the method where concrete is placed at mid-spans before over the piers.

19.6 Verification of Structural Safety (ULS)

Verification of structural safety comprises ensuring that the dimensions of all beam cross sections are sufficient to resist the internal moments and forces that act on them. According to Section 9.6, structural safety is confirmed when the design criterion $E_d \leq R_d$ is satisfied. E_d represents the design value of the action effects, determined using the load combination for the hazard scenario being considered. R_d represents the design value of resistance, such as resistance to bending, to lateral torsional buckling, or to shear.

It should be remembered that structural safety must be checked for the different construction phases and for the bridge in its final state. For the purposes of this example, structural safety checks are illustrated for the following cases and cross sections:

- construction stage: most heavily stressed cross section in the central span,
- final state: cross section in the middle of the central span,
- final state: cross section above the intermediate support P2.

Checks of resistance to bending and shear make reference to information contained in Chapters 12 and 13. Checks associated with the structural safety of the steel-to-concrete shear connection, and fatigue safety, are covered in Sections 19.7 and 19.8 respectively. Before considering the checks themselves, Paragraph 19.6.1 defines the various resisting cross sections and their properties.

19.6.1 Resisting Cross Sections

Steel Sections

The steel sections considered in this example are those presented in Figure 19.3, with the properties given in Table 19.4.

Effective Slab Width

The effective slab width is calculated using Equations (13.6) and (13.7). The edge beam of 0.5 m (Fig. 19.2) is not taken into consideration because it could be modified or even removed in future. The distance between the rows of shear studs is $b_0 = 0.40$ m. The distance between points of zero moment is taken as $0.7l$ for the central span, where l is the span length. For the cross section at mid-span this results in the following effective width:

$$b_{e1} = \frac{0.7 \cdot 80\,\mathrm{m}}{8} = 7.0\,\mathrm{m} > b_1 = (2.5 - 0.2) = 2.3\,\mathrm{m} \text{ therefore } b_{e1} = 2.3\,\mathrm{m}$$

$$b_{e2} = \frac{0.7 \cdot 80\,\mathrm{m}}{8} = 7.0\,\mathrm{m} > b_2 = (3.0 - 0.2) = 2.8\,\mathrm{m} \text{ therefore } b_{e2} = 2.8\,\mathrm{m}$$

$$b_{eff} = b_0 + \sum b_{ei} = 0.4 + 2.3 + 2.8 = 5.5\,\mathrm{m} = b$$

The slab is therefore totally effective in contributing to the resistance of the cross section. One finds the same result for other cross sections in span and at the supports using Figure 13.10. The area of slab that participates in the resistance of each beam is therefore:

- $A_c = b_{eff} \cdot h_c = 5500 \cdot 350 = \mathbf{1925 \cdot 10^3\ mm^2}$,

And the area of reinforcement (§ 19.2.3) that participates is:

- $A_s = 1.5 \cdot A_c/100 = \mathbf{28.9 \cdot 10^3\ mm^2}$ at the intermediate support,
- $A_s = 0.75 \cdot A_c/100 = \mathbf{14.4 \cdot 10^3\ mm^2}$ in span.

For calculations it is recognised that the slab includes a 75 mm deep haunch above each of the steel beams, which results in a real slab depth of 425 mm in these regions (Fig. 19.2), when combined with the average slab depth of 350 mm (§ 19.2.3). The area of the haunch is not included when determining cross-sectional resistance, but its depth is allowed for when determining the position of the centre of gravity of the slab and the reinforcement above the upper flange: $h_{Gc} = 75 + 350/2 = 250$ mm.

19.6.2 Cross Section in the Span During Construction

The cross section considered is that located 8 m to the right of the middle of the central span (Fig. 19.13), which is where the bending moment is greatest at the end of the fourth concreting stage. During this phase the upper flanges of the main beams are not held laterally, to prevent lateral torsional buckling, until the concrete has hardened. Nevertheless, some restraint is provided by temporary plan bracing, located at mid-depth of the main beams, in the plane of the cross girders. The main beams must remain elastic during erection; checking of this cross section is therefore based on an elastic resistance model taking into account lateral torsional buckling. The notation that is used to define the steel section is illustrated in Figure 19.17.

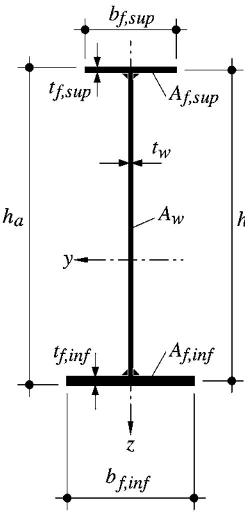


Fig. 19.17 Notation for the plate girder cross sections.

Design Value of the Bending Moment

The bending moment M_{Ed} for hazard scenario no. 1 (Table 19.8) and with values taken from the bending moment diagrams shown in Figures 19.12 and 19.13 is given by:

$$M_{Ed} = 1.35\,(2394 + 12\,177) + 1.50 \cdot 1995 = \mathbf{22\,663\,kNm}$$

For this temporary construction stage the concrete slab protects the steel beams from direct sunlight so there is no temperature gradient over the depth of the steel beams; this means that temperature effects may be ignored for this hazard scenario.

Verification of the Cross Section

Buckling of the compression flange into the web (12.6):

$$h_f/t_w = 2760/14 = 197 \le 0.40 \cdot E_a/f_y = 0.40 \cdot 210\,000/355 = 240 \qquad \Rightarrow \text{OK}$$

Buckling by rotation of the compression flange (12.8)

$$(b_{f,\,sup}/2)/t_f = (700/2)/40 = 8.75 \le 0.56\sqrt{E/f_y} = 0.56\sqrt{210\,000/355} = 14 \qquad \Rightarrow \text{OK}$$

Resistance to Lateral Torsional Buckling

Cross-sectional area as shown in Figure 19.3:

$$A_a = A_{f,inf} + A_w + A_{f,sup} = 800 \cdot 40 + 14 \cdot 2720 + 700 \cdot 40 = 98\,080\ \text{mm}^2$$

Position of the centre of gravity of the gross cross section relative to the mid-thickness of the lower flange:

$$z_G = \frac{A_{f,inf} z_{f,inf} + A_w z_w + A_{f,sup} z_{f,sup}}{A_a} = \frac{800 \cdot 40 \cdot 0 + 14 \cdot 2720 \cdot 1380 + 700 \cdot 40 \cdot 2760}{98\,080} =$$

$$z_G = 1324\ \text{mm}$$

Second moment of area of the steel cross section (Table 19.4):

$$I_a = 137 \cdot 10^9\ \text{mm}^4$$

Depth in compression:

$$h_c = h_f - z_G = 2760 - 1324 = 1436\ \text{mm}$$

Ratio between the extreme stresses:

$$\psi = \sigma_{\min}/\sigma_{\max} = -z_G/h_c = -1324/1436 = -0.922$$

Effective depth in compression (12.28), using a buckling coefficient $k = 22.03$ (12.29)

$$h_{c,eff} = 0.86 \cdot \frac{1436}{2760}\sqrt{22.03\frac{210\,000}{355}} \cdot 14 = 715\text{mm} < h_c = 1436\ \text{mm}$$

Distance between the neutral axis of the gross cross section and the neutral axis of the effective cross section (12.30):

$$e = \frac{1436}{2} \cdot \frac{14(1436 - 715)}{98\,080 - 14(1436 - 715)} = 82\ \text{mm}$$

Second moment of area of the effective cross section (12.31):

$$I_{eff} = 137 \cdot 10^9 - \frac{1436^2}{4} \cdot \frac{98\,080 \cdot 14(1436 - 715)}{98\,080 - 14(1436 - 715)} - 14\frac{(1436 - 715)^3}{12} = 131 \cdot 10^9\ \text{mm}^4$$

Modulus of the elastic cross section calculated relative to the compression fibre (12.33):

$$W_{c,eff} = \frac{131 \cdot 10^9}{1436 + 82} = 86.4 \cdot 10^6\ \text{mm}^3$$

To calculate the resistance to lateral torsional buckling M_D, one must determine the lateral torsional buckling stress σ_D, which is a function of the buckling length l_D of the compression flange. This length is taken as the distance between lateral restraints to the flange, therefore the distance between cross bracings,

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and depends on the stiffness of these restraints. This stiffness may be calculated using Equation (12.23) to determine the lateral displacement v of the restraint under a unit transverse force $H = 1$ N. At this phase of the construction the cross bracing rests on the plan bracing, which may be taken as a lateral support for the cross bracing. Figure 19.18 shows the structural form of the frame cross bracing assumed for calculations, as well as the cross sections of the elements forming the frame cross bracing. The displacement is calculated as $v = 45 \cdot 10^{-6}$ mm/N.

Equation (12.21) allows the buckling length l_D to be calculated. In this equation the second moment of area I_D is that about the z axis of the area A_D of the member in compression, for which the area is calculated according to Equation (12.12). The spacing of cross bracing e in the central span is 8000 mm.

$$A_D = b_{f,sup} \cdot t_{f,sup} + \frac{h_{c,eff}}{2} \cdot t_w = 700 \cdot 40 + \frac{715}{2} \cdot 14 = 33\,005 \text{ mm}^2$$

$$I_D = \frac{t_{f,sup} \cdot b_{f,sup}^3}{12} + \frac{h_{c,eff}}{2} \cdot t_w^3/12 = \frac{40 \cdot 700^3}{12} + \frac{715}{2} \cdot 14^3/12 = 1.14 \cdot 10^9 \text{mm}^4$$

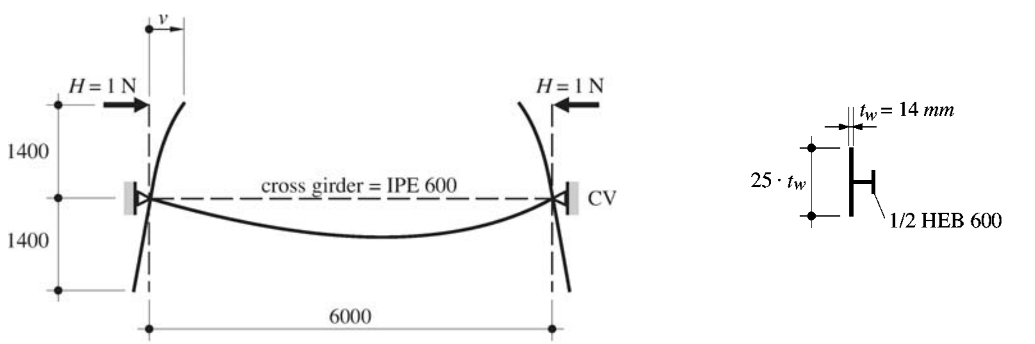
$$l_D = \sqrt[4]{\frac{\pi^4}{4} 210\,000 \cdot 1.14 \cdot 10^9 \cdot 8000 \cdot 45 \cdot 10^{-6}} = 6768 \text{ mm} < 8000 \text{ mm}$$

Because l_D is smaller than e, l_D is taken to be $e = 8000$ mm. The stress σ_D can then be calculated as follows. In the zone at mid-span the bending moment is almost constant. Therefore it may be assumed that $\eta = 1.0$ from which $l_K = l_D = 8000$ mm (12.10). The radius of gyration of the member in compression is then:

$$i_D = \sqrt{I_D/A_D} = \sqrt{1.14 \cdot 10^9/33005} = 186 \text{ mm}$$

Using (12.11) and $\lambda_K = l_K/i_D$ gives $\sigma_{cr,D} = 1120$ N/mm^2, and therefore a slenderness $\overline{\lambda_D}$ (12.18) of:

$$\overline{\lambda_D} = \sqrt{f_y/\sigma_{cr,D}} = \sqrt{355/1120} = 0.563$$



(a) Structural form of the cross bracing

(b) Cross section of the upright

Fig. 19.18 Calculation of the displacement v of the frame cross bracing.