DESIGNERS' GUIDES TO THE EUROCODES

DESIGNERS' GUIDE TO EUROCODE 1: ACTIONS ON BRIDGES

EN 1991-2, EN 1991-1-1, -1-3 TO -1-7 AND EN 1990 ANNEX A2

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DESIGNERS' GUIDE TO EUROCODE 1: ACTIONS ON BRIDGES

EN 1991-2, EN 1991-1-1, -1-3 TO -1-7 AND EN 1990 ANNEX A2

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Preface

EN 1991, *Eurocode 1: Actions on Structures* includes ten parts which provide comprehensive information and guidance on all actions that it is normally necessary to consider in the design of bridges, building and civil engineering structures. All Parts have now been published by the European Committee for Standardisation (CEN) as European Standards (ENs).

EN 1990, Eurocode 0: Annex A2 to EN 1990: Basis of structural design, application for bridges, which has been published as 'Amendment A1' (EN1990:2002/A1, December 2005). In the following text of the book, this part of Eurocode is referred to in its shortened title 'EN 1990 Annex A2' or 'EN 1990:2002/A1' when used to define a reference. This Eurocode defines combination of actions and some serivceability state criteria.

Aims and objectives of this guide

The principal aim of this guide is to help users understand, in terms of application to actions on bridges, the following parts of EN 1991 *Actions on Structures*.

EN 1991-1-1 *Densities, self-weight and imposed loads*
EN 1991-1-3 *Snow loads*
EN 1991-1-4 *Wind actions*
EN 1991-1-5 *Thermal actions*
EN 1991-1-6 *Actions during execution*
EN 1991-1-7 *Accidental actions*
EN 1991-2 *Traffic actions*
and EN 1990 Annex A2

This guide should be read in conjunction with the sister book to this volume, namely the TTL *Designers' Guide to Eurocode 1: Actions on Buildings*, where guidance is given on basic clauses on classification of actions, design situations etc. which apply to both bridges and buildings.

In producing this guide the authors have endeavoured to provide explanations and commentary to the clauses in EN 1991 and EN 1990 Annex A2 for all the categories of users identified in the foreword of each Eurocode part. Although the Eurocodes are primarily intended for the design of buildings and civil engineering works, EN 1991 is intended for the consideration of a wider category of users which includes:

- designers and contractors
- clients
- product manufacturers
- public authorities and other bodies who produce regulations.

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Layout of this guide

EN 1991 *Eurocode 1: Actions on Structures* has ten parts which are described in the Introduction to this Designers' Guide. This publication gives guidance on the parts mentioned above. The guide is divided into eight chapters and covers information for the design of bridges in EN 1991 through the following chapters:

- Chapter 1 provides an introduction and gives guidance on general aspects of the design of bridges using the Eurocodes.
- Chapter 2 covers non-traffic actions for persistent design situations (i.e. densities, self-weight, imposed loads and climatic actions).
- Chapter 3 covers actions during execution.
- Chapter 4 covers traffic loads on road bridges.
- Chapter 5 covers traffic loads on footbridges.
- Chapter 6 covers traffic loads on railway bridges.
- Chapter 7 covers accidental actions.
- Chapter 8 covers combinations of actions for road bridges, footbridges and railway bridges.

The authors would like to remind readers that this designers' guide cannot be used in place of the Eurocodes but rather should be used alongside these standards.

Acknowledgements

This guide would not have been possible without the successful completion of EN 1991 as well as EN 1990 Annex A2 and the authors would like to thank all those who contributed to its preparation. Those involved included the members of the Project Teams and the National Delegations. The following individuals are especially thanked: Mr H. Mathieu, Professor Luca Sanpaolesi, Professor Gerhard Sedlacek, Dr Paul Luchinger, Mr Paolo Formichi, Mr Lars Albrektson, Mr Malcolm Greenley, Mr Ray Campion, Mr Peter Wigley and Mr Ian Bucknall.

The authors would especially like to thank Professor Pierre Spehl of Seco who provided an example of wind actions on bridges.

This book is dedicated to the following:

- The authors' employers and supporters and the General Council for Environment and Sustainable Ministry of Ecology, Energy, Sustainable Development and Town and Country Planning, Paris; the UIC (International Union of Railways, headquarters in Paris), which provided the platform for problems in railway bridge design to be studied. The UIC was also especially helpful in providing substantial financial help for studies and measurements to be undertaken into the aerodynamic effects of passing trains, the dynamic analysis of railway bridges for high-speed trains and helped advance the treatment of the interaction effects between bridge and track. Without this help, the high standard of the structural Eurocodes would not have been achieved; and BRE Garston, the Department of Communities and Local Government, London and the Highways Agency in the UK.
- The authors wives, Elisabeth Calgaro, Jacqueline Tschumi and Vera Gulvanessian, for their support and patience over the years.

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CHAPTER 1

Introduction and general aspects of the design of bridges with Eurocodes

This Designers' Guide is intended to help engineers in using the Eurocodes for the design of new bridges (road bridges, footbridges and railway bridges). It deals with the determination of actions applicable to bridges during execution and normal use, and their combination for the verification of the appropriate ultimate and serviceability limit states. Actions due to earthquakes, defined in Eurocode 8, are outside the scope of this Designers' Guide.

1.1. The Eurocodes

The first European Directive on public procurement was published in 1971 but its practical application concerning the calculation of civil engineering works proved to be very difficult. This was mainly due to a clause forbidding, for a public tender, the rejection of a tender on the grounds that this tender was based on design standards in force in a country different from the country where the construction work was to be undertaken. For that reason, it was decided in 1976 to develop a set of European structural design codes, mainly based on studies carried out by international scientific associations, that could be widely recognized for the judgement of tenders.

In the early 1980s, the first documents, called Eurocodes, were published as provisional standards under the responsibility of the Commission of European Communities. After lengthy international inquiries and after the adoption of the Unique Act (1986), it was decided to transfer the development of the Eurocodes to CEN (the European Committee for Standardisation) and to link them to the Construction Product Directive (CPD). The transfer took place in 1990 and CEN decided to publish the Eurocodes first as provisional European standards (ENVs) and then as European standards (ENs).

In the Foreword of each Eurocode, it is noted that the member states of the European Union (EU) and the European Free Trade Association (EFTA) recognise that Eurocodes serve as reference documents for the following purposes:

- As a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement No. 1 – Mechanical resistance and stability – and Essential Requirement No. 2 – Safety in case of fire.
- As a basis for specifying contracts for construction works and related engineering services.

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Table 1.1. The Eurocodes Programme

EN 1990	Eurocode:	Basis of structural design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

- As a framework for drawing up harmonized technical specifications for construction products (ENs and ETAs).

In fact, the Eurocodes have also been developed to improve the functioning of the single market for products and engineering services by removing obstacles arising from different nationally codified practices for the assessment of structural reliability, and to improve the competitiveness of the European construction industry and the professionals and industries connected to it, in countries outside the European Union.

The Structural Eurocode programme comprises the following standards, as shown in Table 1.1, generally consisting of a number of parts.

The Eurocodes are intended for the design of new construction works using the most traditional materials (reinforced and prestressed concrete, steel, steel and concrete composite construction, timber, masonry and aluminium). It should be appreciated that the principles of the main Eurocode EN 1990 *Eurocode – Basis of structural design*[1] are applicable when the design involves other materials and/or other actions outside the scope of the Eurocodes. Moreover, EN 1990 is applicable for the structural appraisal of existing construction, in developing the design for repairs and alterations or in assessing changes of use. This applies, in particular, to the strengthening of existing bridges. Of course, additional or amended provisions may have to be adopted for the individual project.

1.2. General design principles and requirements for construction works

The general principles for the design of civil engineering works are defined in EN 1990 *Basis of structural design*. Their application to the design of bridges is briefly discussed below.

1.2.1. General – fundamental requirements

The verification rules in all Eurocodes are based on the limit state design using the partial factors method.

In the case of bridges, most accidental scenarios leading to catastrophic failure are due to gross errors during execution, impacts during normal use or uncontrolled scour effects. Such risks may be avoided, or their consequences mitigated, by adopting appropriate design and execution measures (e.g. stabilising devices) and by appropriate control of quality procedures. During its working life, the collapse of a bridge may be the consequence of the following:

- A possible accidental situation (e.g. exceptional scour near foundations). See Fig. 1.1.
- Impact (e.g. due to lorry, ship or train collision on a bridge pier or deck, or even an impact due to a natural phenomenon). See Fig. 1.2.
- Development of fatigue cracks in a structure with low redundancy (e.g. cracks in a welded joint in one of the two girders of a composite steel–concrete bridge deck) or failure of cables due to fatigue. Concerning this question, the design Eurocodes establish a distinction between damage-tolerant and non-tolerant structures. See Fig.1.3.

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Fig. 1.1. Example of effects of scour around bridge piers (Pont des Tours, France, 1998)

- Brittle behaviour of some construction materials, e.g. brittle steel at low temperatures. (This type of risk is very limited in the case of recent or new bridges but it may be very real in the case of old bridges.)
- Deterioration of materials (corrosion of reinforcement and cables, deterioration of concrete, etc.). See Fig. 1.4.

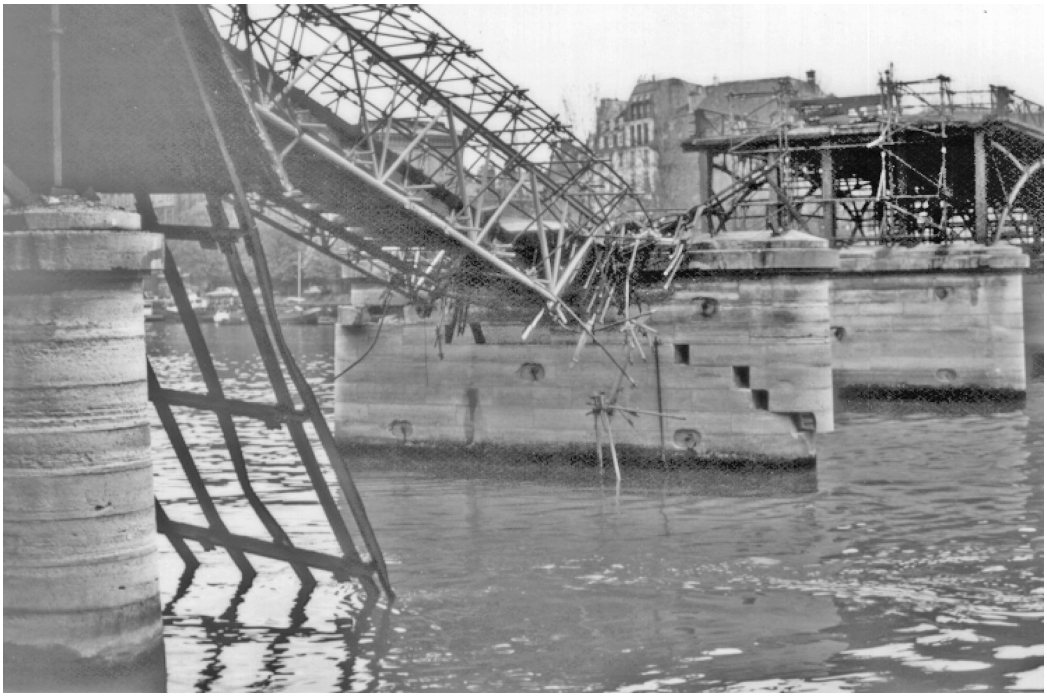


Fig. 1.2. Ship impact on a bridge pier (Pont des Arts, Paris, 2001)

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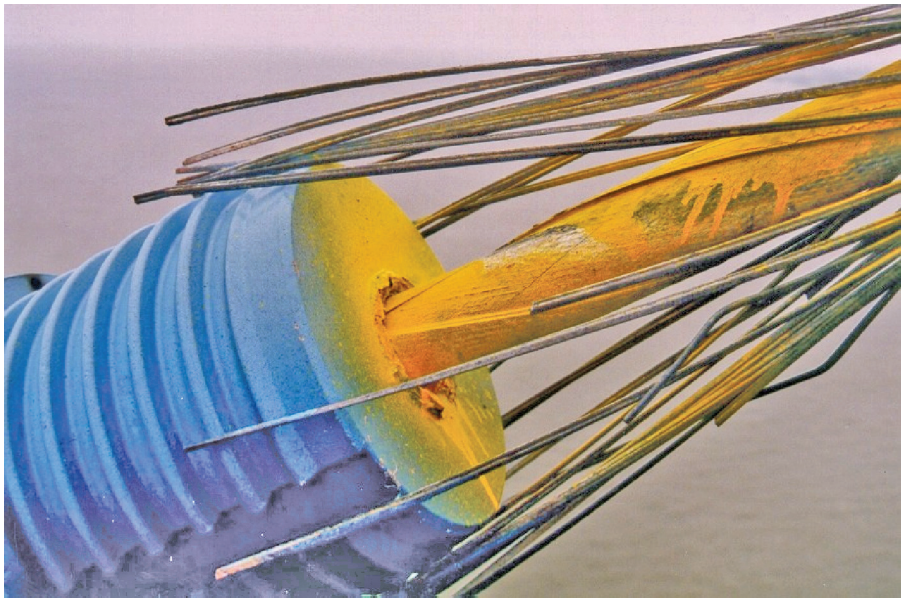


Fig. 1.3. Example of fatigue effects on cables

1.2.2. Design working life and durability

Bridges are public works, for which public authorities may have responsibilities as owner and also for the issue of national regulations on authorised traffic (especially on vehicle loads) and for delivery and control dispensations when relevant, e.g. for abnormally heavy vehicles.

One major requirement is the design working life. Table 1.2, which reproduces parts of *Table 2.1* in EN 1990, gives indicative values for the design working life of several types of construction works.

Thus, a design working life of 100 years is commonly agreed for bridges by experts and relevant authorities, but the meaning of this value needs some clarification.



Fig. 1.4. Examples of deterioration of materials

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Table 1.2. Indicative design working life (See EN 1990, Table 2.1 for all values)

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures*
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3		Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

* Structures or parts of structures that can be dismantled with a view to being reused should not be considered as temporary.

First, all parts of a bridge cannot be designed for the same design working life, for obvious economical reasons. In particular, structural bearings, expansion joints, coatings, or any industrial product cannot be designed or executed for such a long working life. And, in the case of road restraint systems, the concept of design working life is not really relevant.

Table 2.1 of EN 1990 makes a distinction between replaceable and non-replaceable structural members. The design working life intended for non-replaceable members, or in other words for load-bearing structural members, is given in Categories 4 and 5. Regarding load-bearing structural members, EN 1990 specifies the following: *cl. 2.1(1)P: EN 1990*

> *'A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way*
>
> – *sustain all actions and influences likely to occur during execution and use, and*
> – *meet the specified serviceability requirements for a structure or a structural element.'*

EN 1990 *Clause 2.4(1)P* states: *cl. 2.4(1)P: EN 1990*

> *'The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance. . . .*
>
> *The environmental conditions shall be identified at the design stage so that their significance can be assessed in relation to durability and adequate provisions can be made for protection of the materials used in the structure.'*

This means that, by the end of the design working life, generally irreversible serviceability limit states should not be exceeded, considering a reasonable programme of maintenance and limited repair. Of course, the design working life may be used directly in some fatigue verifications for steel members, but more and more frequently, requirements concerning, for example, the penetration of chlorides into concrete or the rate of carbonation after x years are specified in the project specification of bridges.

Finally, the design of a bridge is not only a matter of architecture or of calculation: it has to be considered as a living form which needs care.

1.2.3. Reliability differentiation

For the purpose of reliability differentiation the informative *Annex B* of EN 1990 defines three consequence classes (CC1 to CC3) in *Table B1* of EN 1990. Although the classification into consequence classes is the responsibility of the relevant authority, many bridges can be considered as belonging to the medium class (CC2) described by '***Medium*** *consequence for loss of human life, economic, social or environmental consequences* ***considerable***', which means that the general rules given in the design Eurocodes may be used without additional *cl. 2.2(1)P: EN 1990*

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severe requirements. Nevertheless, in the case of very important road and railway bridges (e.g. large spans on skews or bridges in seismic zones), they should be appropriately classified in the higher consequence class CC3 (***High*** *consequence for loss of human life, or economic, social or environmental consequences* ***very great***). Therefore, some design assumptions or requirements, in the project specification, may be more severe than those adopted in the Eurocodes, or some partial factors (for actions or resistances) may be more conservative than the recommended values. The decision concerning the classification of a bridge is taken by the client or the relevant authority. Various differentiation measures may be adopted depending on the quality of design, design supervision and execution inspection. One of these measures consists of applying a factor K_{FI}, given in *Table B3* of EN 1990, to unfavourable actions. However, it is mentioned in *Annex B* of EN 1990 that other measures (e.g. quality control in the design and execution phases) are normally more effective in ensuring safety.

It is also mentioned that reliability differentiation may also be applied through the partial factors on resistance γ_M. However, this is not normally used except in special cases such as fatigue verification (see EN 1993).

Special attention should be made to some bridges in seismic zones (see EN 1998 and its TTL (Thomas Telford Ltd) Designers' Guide.[2] From a practical point of view, serviceability requirements should be taken from Parts 2 of Eurocodes 2, 3, 4, 5 and 8, and, for ultimate limit states, preference should be given to combinations of actions based on Expression 6.10 of EN 1990.

cl. 6.4.3.2: EN 1990

1.3. The design of bridges with Eurocodes

The use of the Eurocodes for the design of bridges is already widely adopted. This is due mainly to the fact that since the introduction of the Eurocodes many countries have ceased to update their national codes, causing them to become obsolete and unusable. In addition the globalisation of engineering activities, which is the case for major bridges, implies the establishment of contracts based on an internationally recognised technical basis.

Currently, very few important (see for example Fig. 1.5) or monumental bridge or civil engineering structures in Europe are designed and executed without a reference (for the whole or part of the structure, for normal use or during execution) to the Eurocodes. This



Fig. 1.5. The Millau Viaduct – an example of the use of Eurocodes for the launching phase

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Table 1.3. Design of bridges with Eurocodes

Eurocode	Part of Eurocode	Title and/or scope
EN 1990 – Eurocode: Basis of structural design	Main text	Structural safety, serviceability and durability Principles of partial factor design
	Annex A2	Application for bridges (combinations of actions)
EN 1991: Eurocode 1 – Actions on structures	Part 1-1	Densities, self-weight and imposed loads
	Part 1-3	Snow loads
	Part 1-4	Wind actions
	Part 1-5	Thermal actions
	Part 1-6	Actions during execution
	Part 1-7	Accidental actions due to impact and explosions
	Part 2	Traffic loads on bridges (road bridges, footbridges, railway bridges)
EN 1992: Eurocode 2 – Design of concrete structures	Part 1-1	General rules and rules for buildings
	Part 2	Reinforced and prestressed concrete bridges
EN 1993: Eurocode 3 – Design of steel structures	Part 1	General rules and rules for buildings, including: – *Part 1-1* – General rules and rules for buildings – *Part 1-4* – Stainless steels – *Part 1-5* – Plated structural elements – *Part 1-7* – Strength and stability of planar plated structures transversely loaded – *Part 1-8* – Design of joints – *Part 1-9* – Fatigue strength of steel structures – *Part 1-10* – Selection of steel fracture toughness and through-thickness properties – *Part 1-11* – Design of structures with tension components made of steel – *Part 1-12* – Supplementary rules for high strength steel
	Part 2	Steel bridges
EN 1994: Eurocode 4 – Design of composite steel and concrete structures	Part 1-1	General rules and rules for buildings
	Part 2	Composite bridges
EN 1995: Eurocode 5 – Design of timber structures	Part 1-1	General rules and rules for buildings
	Part 2	Timber bridges
EN 1997: Eurocode 7 – Geotechnical design	Part 1	Geotechnical design
EN 1998: Eurocode 8 – Design of structures for earthquake resistance	Part 1	General rules, seismic actions and rules for buildings
	Part 2	Bridges

demonstrates that the Eurocodes do not limit creativity but in fact allow architects and engineers to achieve their designs with more boldness and more responsibility.

The Eurocode parts that need to be (partly or totally) used for the design of a bridge are given in Table 1.3.

The structural fire design of bridges is not dealt with in this Designers' Guide. This type of design situation is normally not covered by the Eurocodes, even though the consequences

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of accidental exposure of bridges to fire actions (e.g. lorries burning over or below a bridge deck) are increasingly taken into account for the design of important and monumental bridges. However, the fire Parts of Eurocodes may be used as guidance for the type of problem under consideration.

The scope of this Designers' Guide is to explain how to calculate the most common actions applicable to bridges and how to establish the combinations of actions for the various ultimate and serviceability limit states. The rules concerning specifically the verification of concrete, steel, steel–concrete composite or timber bridges are explained in the respective TTL publications.[3–6]

The design of bridges located in seismic zones is evoked in this Designers' Guide but actions due to earthquakes are beyond its scope. See instead the TTL Designers' Guide for EN 1998.[2]

The principles and requirements for safety, serviceability and durability of structures are defined in *EN 1990: Eurocode: Basis of structural design*[1] which is the head document in the Eurocode suite. In particular, it provides the basis and general principles for the structural design of bridges, including geotechnical aspects and situations involving earthquakes, execution and temporary structures.

1.4. Evolution of traffic loads

1.4.1. Road traffic loads

The volume of road traffic is continually increasing. The average gross weight of heavy lorries is also increasing because, for obvious economical reasons, these lorries travel with full load. Furthermore, many of them do not comply with legal limits (maximum weight and, sometimes, maximum dimensions). With this in mind, it is useful to refer to Council Directive 96/53/EC,[7] laying down, for certain road vehicles circulating within the Community, the maximum authorized dimensions in national and international traffic and the maximum authorized weights in international traffic, amended by Council Directive 2002/7/EC[8] of the European Parliament and of the Council laying down the maximum authorized dimensions in national and international traffic and the maximum authorized weights in international traffic.

The vehicles are classified by Council Directive 70/156/EC.[9] The Directive defines four vehicle categories, namely M, N, O and G. G corresponds to off-road vehicles. For 'normal' road vehicles, the classification M, N, O is described in Table 1.4.

Table 1.4. Vehicle categories

Category	Description
M	Motor vehicles with at least four wheels designed and constructed for the carriage of passengers. This category includes three sub-categories, M1, M2 and M3, depending on the number of seats and the maximum mass
N	Motor vehicles with at least four wheels designed and constructed for the carriage of goods. This category includes three sub-categories, N1, N2 and N3, depending on the maximum mass. Category N3 vehicles have a maximum mass exceeding 12 tonnes
O	Trailers (including semi-trailers). Four sub-categories are defined, O1, O2, O3 and O4, depending on the maximum mass. Category O4 corresponds to trailers with a maximum mass exceeding 10 tonnes

The maximum dimensions and related characteristics of vehicles are defined in Council Directive 96/53/EC,[7] amended by Council Directive 2002/7/EC.[8] They are summarized in Table 1.5.

The maximum weights of vehicles are defined in Council Directive 96/53/EC,[7] and the most usual weights are summarized in Table 1.6.

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Table 1.5. Standardized dimensions of vehicles

Characteristics	Dimensions (m)
Maximum length	– motor vehicle other than a bus: 12.00 – trailer: 12.00 – articulated vehicle: 16.50 – road train: 18.75 – articulated bus: 18.75 – bus with two axles: 13.50 – bus with more than two axles: 15.00 – bus + trailer: 18.75
Maximum width	– all vehicles: 2.55 – superstructures of conditioned vehicles: 2.60
Maximum height	4.00 (any vehicle)

From Table 1.6 it can be seen that the maximum weight for a road vehicle is 40 tonnes or 44 t, depending on its type. These values are 'static' values (dynamic effects may be important – see the Annex to Chapter 4) and, in reality, a significant proportion of lorries have a higher weight than authorized. For these reasons, and because higher limits may be defined in the future, the road traffic load models are calibrated with appropriate safety margins.

Concerning the maximum authorised axle weight of vehicles, the limits are:

- 10 t for a single non-driving axle
- 11 t, 16 t, 18 t and 20 t, for tandem axles of trailers and semi-trailers, depending on the distance between the axles (less than 1 m, between 1.0 m and less than 1.3 m, between 1.3 m and less than 1.8 m, 1.8 m or more respectively).
- 21 or 24 t for tri-axle trailers and semi-trailers, depending on the distance between axles (1.3 m or less, over 1.3 m and up to 1.4 m respectively)
- 11.5 t, 16 t, 18 t or 19 t for tandem axles of motor vehicles depending on the distance between axles (less than 1 m, 1.0 m or greater but less than 1.3 m, 1.3 m or greater but less than 1.8 m respectively).

Table 1.6. Most usual weights of road vehicles

Vehicles	Maximum weight (t)
Vehicles forming part of a vehicle combination:	
– Two-axle trailer	18
– Three-axle trailer	24
Vehicle combinations:	
– Road trains with five or six axles:	
(a) two-axle motor vehicle with three-axle trailer	(a) 40
(b) three-axle motor vehicle with two- or three-axle trailer	(b) 40
– Articulated vehicles with five or six axles:	
(a) two-axle motor vehicle with three-axle semi-trailer	(a) 40
(b) three-axle motor vehicle with two- or three-axle semi-trailer	(b) 40
(c) three-axle motor vehicle with two- or three-axle semi-trailer carrying a 40-foot ISO container as a combined transport operation	(c) 44
Motor vehicles:	
– two-axle motor vehicles	18
– three-axle motor vehicles	25 or 26
– four-axle motor vehicles with two steering axles	32
Three-axle articulated buses	28

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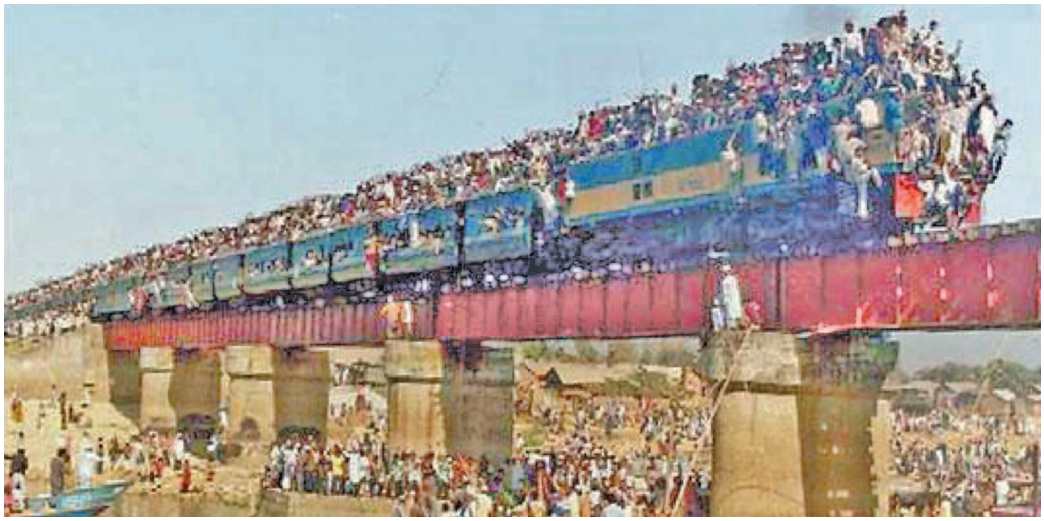


Fig. 1.6. Overloaded train

As for the maximum vehicle weight, the maximum values of axle weights are 'static' values. Real dynamic values (i.e. values including dynamic effects) may be very much higher depending on the quality of the carriageway.

1.4.2. Rail traffic loads

Overloading can be a risk, as is clearly evident in Fig. 1.6 and Fig. 1.7.

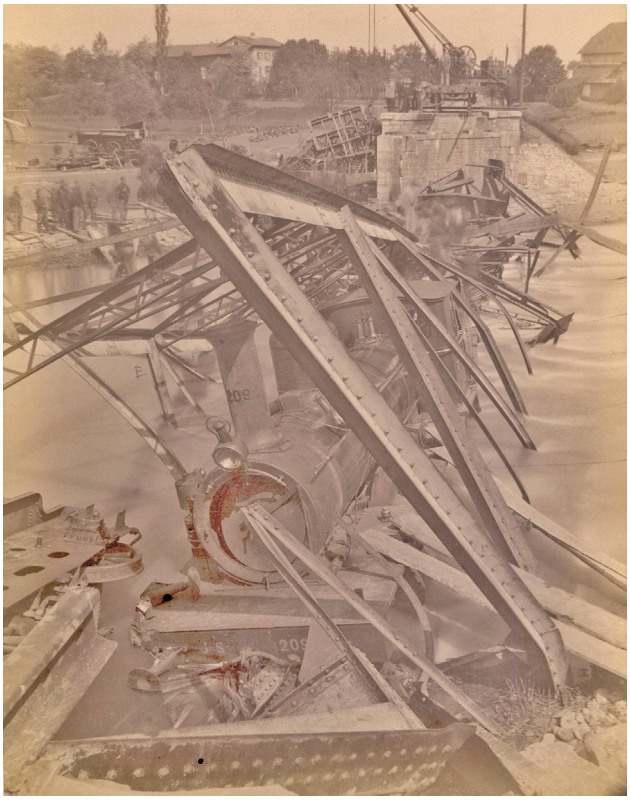


Fig. 1.7. Bridge in Münchenstein (Switzerland). The bridge collapsed on 14 June 1891 under a fully occupied train by buckling of the upper flange; 73 people died

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Table 1.7. Types of train

Type of train	Speeds (km/h)	Axle loads (kN)	Average weight (kN/m)
Passenger trains:			
– suburban multiple units	100–160	130–196	20–30
– locomotive-hauled trains	140–225	150–215	15–25
– high-speed trains	250–350	170–195*	19–20
Freight trains:			
– heavy abnormal loads	50–80	200–225	100–150
– heavy freight	80–120	225–250†	45–80
– trains for track maintenance	50–100	200–225	30–70
– fast, light freight	100–160	180–225	30–80

* Future high-speed trains due to European Directive TSI (Technical System Interoperability):
Axle loads:
180 kN for 200 km/h $< V \leq$ 250 km/h
170 kN for 250 km/h $< V \leq$ 300 km/h
160 kN for 300 km/h $V >$ 300 km/h

† Important note: the latest studies concerning freight traffic evolution undertaken by European railways lead to the conclusion that axle loads of 300 kN should be enabled in say 100 years on the European network.

Rail bridges are built to carry a mixture of traffic which is likely to change during their 200-year lifetime. The traffic can be categorized as either passenger or freight trains, the latter being locomotive hauled. Table 1.7 shows their actual speeds, axle loads and average weights per metre length, all as ranges of values commonly encountered or planned.

In relation to Table 1.7 it should be noted that:

- the average weight of locomotives ranges from 50 to 70 kN/m
- the length of the vehicles classed as very heavy loads ranges from 15 to 60 m; they mainly affect the support moments of continuously supported bridges and simply supported medium-span bridges.

Particular train lines may have physical restriction on the line (curves, gradients, weak existing bridges) and additionally commercial and operating requirements. All these factors are known and planned for at any given time, but may, and probably will, change in the course of time. At present, for example, very heavy freight traffic is not allowed on a number of lines, including most suburban and high-speed passenger lines.

High-speed passenger lines, however, can sometimes also carry all kinds of freight on their track. It is therefore reasonable to build new bridges that are capable of carrying any of the present and anticipated traffic.

UIC produced a load model which covers the greatest static actions of all known and planned trains, as well as a load model for very heavy loads. The above-mentioned load models are the basis for the load models (Load Model 71, SW/0 and SW/2) presented in EN 1991-2 and Chapter 6 of this Designers' Guide.

Unfortunately, for political reasons, the Eurocodes are unable to recommend which factor α together with Load Model 71 to enable the 300 kN axle load traffic in the long-term future. The reason for the long-term is because authorities require about 100 years to change or upgrade all weak bridges on certain lines, due to practical and commercial reasons.

Note: It is recommended to apply a factor of $\alpha = 1.33$ to Load Model 71 (see Chapter 6) from now on for all constructions which are being designed to carry international rail freight traffic in Europe. Important background for the recommended value is given in Section 6.7.2 of this Designers' Guide. The relevant authorities should seek to reach agreement on this value of the alpha factor to be adopted everywhere.

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CHAPTER 2

Determination of non-traffic actions for persistent design situations

This chapter is concerned with the determination of non-traffic actions applicable to bridges during the persistent (see EN 1990) design situations. The material in this chapter is covered in the following parts of EN 1991 *Actions on structures*:

EN 1991-1-1 *General actions – Densities, self-weight, imposed loads for buildings*
EN 1991-1-3 *General actions – Snow loads*
EN 1991-1-4 *General actions – Wind actions*
EN 1991-1-5 *General actions – Thermal actions*

Some aspects of EN 1990 Annex A2 (this is covered fully in Chapter 8).

Reference may be made to the TTL *Designers' Guide to Eurocode 1: Actions on Buildings*[1] which gives a comprehensive discussion on EN 1991-1-1 and EN 1991-1-3 to EN 1991-1-5.

2.1. Self-weight of the structure and other permanent actions (EN 1991-1-1)

In accordance with EN 1991-1-1 (***Clause 5.1(2)***), the self-weight of a bridge includes the structure, structural elements and products, and non-structural elements (fixed services and bridge furniture) as well as the weight of earth and ballast. Examples of fixed services are cables, pipes and service ducts (generally located within footways, sometimes within the deck structure). Examples of bridge furniture are waterproofing, surfacing and other coatings, traffic restraint systems (safety barriers, vehicle and pedestrian parapets), acoustic and anti-wind screens, ballast on railway bridges.

cl. 5.1(2): EN 1991-1-1

The weight of earth may be considered as included in the self-weight of the construction works, or as a permanent action. In fact, this classification is of minor importance for the combinations of actions. The important point is the determination of representative values. Independently of geotechnical actions such as earth pressure on retaining walls, vertical earth loading is met, for example, in the case of spread foundations, pile caps, culverts, etc.

2.1.1. Self-weight of the structure

In accordance with EN 1990 *Eurocode: Basis of Structural Design*, the total self-weight of structural and non-structural members is taken, in terms of combinations of actions, as a

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Table 2.1. Examples of nominal density of some construction materials (Data taken from EN 1991-1-1, Tables A.1, A.3 and A.4)

Materials	Density, γ (kN/m^3)
Concrete (see EN 206)	
Lightweight:	
– density class LC 1.0	9.0 to 10.0[(1),(2)]
– density class LC 2.0	18.0 to 20.0[(1),(2)]
Normal weight:	24.0[(1),(2)]
[(1)] Increase by 1 kN/m^3 for normal percentage of reinforcing and prestressing steel	
[(2)] Increase by 1 kN/m^3 for unhardened concrete	
Mortar	
Cement mortar	19.0 to 23.0
Wood (see EN 338 for timber strength classes)	
Timber strength class C14	3.5
Timber strength class C30	4.6
Timber strength class D50	7.8
Timber strength class D70	10.8
Glued laminated timber (see EN 1194 for timber strength classes)	
Homogeneous glulam GL24h	3.7
Homogeneous glulam GL36h	4.4
Combined glulam GL24c	3.5
Combined glulam GL36c	4.2
Metals	
Aluminium	27.0
Iron, cast	71.0 to 72.5
Iron, wrought	76.0
Steel	77.0 to 78.5

Table A2.2(B) Note 3: EN 1990: 2002 A1 cl. 3.2(1) cl. 4.1.2(3): EN 1990 EN 1991-1-1 cl. 4.1.2(5): EN 1990

single action. Then, '*the variability of G may be neglected if G does not vary significantly during the design working life of the structure and its coefficient of variation is small. G_k should then be taken equal to the mean value*.

The self-weight of the structure may be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses.

For example, effects of actions due to self-weight of reinforced or prestressed concrete structures (and non-structural parts made of the same material, such as concrete safety barriers) are normally determined from their nominal dimensions (taken from the drawings – ***Clause 5.2.1(2)***) and a nominal value of 25 kN/m^3 for density of traditional hardened reinforced or prestressed concrete.

cl. 5.2.1(2)

Table A4: EN 1991-1-1

Similarly, effects of actions due to self-weight of steel structures are determined from their nominal dimensions and an appropriate value of density. According to Table 2.1, the density of construction steel may be selected within the range 77–78.5 kN/m^3. In fact, 77 kN/m^3 = 7.85 (t/m^3) × 9.81 (m/s^2) represents the correct value and should be adopted in all cases.

If the density of materials is significantly different from their nominal values, upper and lower characteristic values need to be be taken in account.

Table 2.1 gives examples of the nominal density for some common construction materials.

Where ranges of values are given for some densities, the value to be taken into account for an individual project should be defined in the project specification. In cases where it is not defined, the best solution is to adopt the mean value.

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Table 2.2. Examples of nominal density of some bridge materials (Data taken from EN 1991-1-1, Table A.6. See EN 1991-1-1 for missing values)

Bridge materials	Density, γ (kN/m^3)
Pavement of road bridges:	
Gussasphalt and asphaltic concrete	
Mastic asphalt	
Hot-rolled asphalt	23.0
Infills for bridges:	
Sand (dry)	15.0 to 16.0[(1)]
Ballast, gravel (loose)	15.0 to 16.0[(1)]
Hardcore	18.5 to 19.5
Crushed slag	13.5 to 14.5[(1)]
Packed stone rubble	
Puddle clay	
[(1)] Given in other tables as stored materials	
Pavement of rail bridges:	
Concrete protective layer	25.0
Normal ballast (e.g. granite, gneiss)	20.0
Basaltic ballast	26
	Weight per unit bed length,[(2),(3)] g_k (kN/m)
Structures with ballasted bed:	
Two rails UIC 60	1.2
Prestressed concrete sleeper with track fastenings	4.8
Concrete sleepers with metal angle braces	–
Timber sleepers with track fastenings	1.9
Structures without ballasted bed:	
Two rails UIC 60 with track fastenings	1.7
Two rails UIC 60 with track fastenings, bridge beam and guard rails	4.9
[(2)] Excludes an allowance for ballast	
[(3)] Assumes a spacing of 600 mm	

2.1.2. Weight of bridge furniture

Concerning effects of actions due to the weight of bridge furniture, the characteristic values of densities of materials and nominal weight of products should be defined in the project specification. Table 2.2 gives the nominal density of some bridge materials. *Table A6: EN 1991-1-1*

As explained for the case of densities for Table 2.1, where a range of values is given for a bridge material, the mean value should be adopted if the value to be taken into account is not defined in the project specification.

For the determination of characteristic values, the recommended deviations from nominal values are given in Table 2.3. *cl. 5.2.3: EN 1991-1-1*

2.1.3. Weight of earth

Concerning fill above buried structures, EN 1991-1-1 highlights the fact that upper and lower characteristic values should be taken into account if the material is expected to consolidate, become saturated or otherwise change its properties during use. In fact, in the case of culverts (especially in urban areas), various design situations may have to be taken into account during the design working life of the structure (in particular, variations of the fill thickness). *cl. 5.2.3: EN 1991-1-1*

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Table 2.3. Determination of characteristic values for bridge furniture

Bridge furniture	Deviation from nominal value
Depth of ballast on railway bridges	± 30%
Waterproofing, surfacing and other coatings	± 20% if post-execution coating included, + 40% to −20% if post-execution coating not included
Cables, pipes and service ducts	± 20%
Parapets, kerbs, joints, fasteners, acoustic screens	0% (nominal values)

For the design, in the absence of any information for the individual project, it may be recommended to adopt a nominal density for gravity actions due to earth equal to $2\,kN/m^3$.

2.2. Snow loads (EN 1991-1-3)

The field of application of EN 1991-1-3 *Snow loads* does not include special aspects of snow loading, for example snow loads on bridges. Hence, EN 1991-1-3 is normally not applicable to bridge design for the persistent design situations. During execution, rules are defined where snow loading may have significant effects (see Chapter 3). However, there is no reason to exclude snow loads on bridges, in particular in the case of roofed bridges (see Fig. 2.1 for the persistent design situations).

For road and railway bridges in normal climatic zones:

- significant snow loads and traffic loads cannot generally act simultaneously (see Chapter 8)
- the effects of the characteristic value of snow loads on a bridge deck are far less important than those of the characteristic value of traffic loads.



Fig. 2.1. Example of roofed bridge

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In the case of footbridges, in particular in Nordic countries, snow loads may be the leading action in combinations of actions.

Concerning snow loads on the roof of a roofed bridge, the characteristic value is determined exactly in the same way as for a building roof (see Chapter 5 of TTL *Designers' Guide for EN 1991: Actions on Buildings*).[1] The combination of snow loads and traffic loads may be defined at the national level or directly for the individual project. Guidance is given in Chapter 8.

The basic design parameter is the characteristic value of snow load on the ground, represented by a uniformly distributed load s_k (kN/m^2), which is determined from an annual probability of exceedence of 0.02 (i.e. a return period of 50 years (***Clause 1.6.1: EN 1991-1-3***)) in accordance with EN 1990. For an individual project, this characteristic value is given by the national map. In certain areas, the meteorological data give some isolated extreme values as outliers from the rest of the values, which cannot be taken into account for the statistical treatment leading to s_k. In these areas, the Eurocode gives an additional value of snow load on the ground, called s_A, which is taken into account as an accidental action. If not defined in the National Annex, this accidental snow load on the ground may be determined from the following recommended formula:

cl. 1.6.1: EN 1991-1-3

cl. 4.3: EN 1991-1-3

$$s_{Ad} = 2s_k$$

Moreover, Annex A to EN 1991-1-3 gives, for each country, the corrective factors for taking into account the altitude or a return period different from 50 years (see Chapter 3).

The load exerted by snow on a roof depends on several parameters: thermal properties of the roof; roughness of its surface; closeness of other construction works; heating; velocity of wind, rain and other kinds of fall. In the case of roofed bridges, there is generally no heat flux in the vertical direction through the roof (some footbridges, for example between two buildings, may be designed with an air-conditioned envelope).

The characteristic snow load on the roof for persistent and transient design situations is determined from the following formula:

cl. 5.2: EN 1991-1-3

$$s = \mu_i C_e C_t s_k$$

where

μ_i is the shape factor, and its value is given by the Eurocode for most roof shapes
C_e is the exposure factor
C_t is the thermal factor, equal to 1.00 except if otherwise specified.

The coefficient C_e may be differentiated as follows for different topographies (data taken from ***Table 5.1, EN 1991-1-3***).

Table 5.1: EN 1991-1-3

Topography	C_e
Windswept topography: flat unobstructed areas exposed on all sides without, or with little, shelter afforded by terrain, higher construction works or trees.	0.8
Normal topography: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.	1.0
Sheltered topography: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.	1.2

Figure 2.2 gives examples of μ factors for three cases (pitched, duo-pitched and cylindrical roof) which may be applicable for roofed bridges.

Along the edge of a roof, the snow can accumulate and remain suspended. The corresponding design load is knife-edged (Fig. 2.3) and applied to the roof edge. Its

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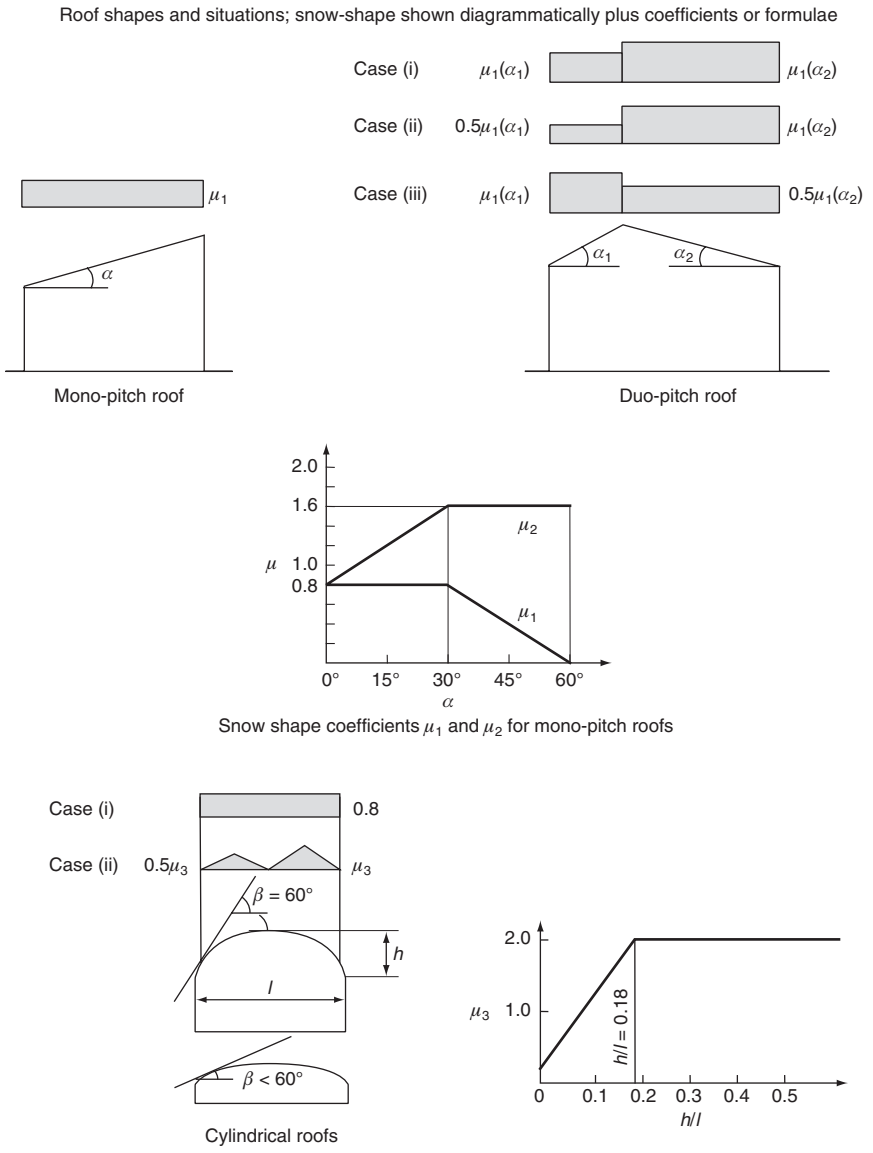


Fig. 2.2. Determination of shape coefficient μ (Data taken from EN 1991-1-3, 5.3)

cl. 6.3: EN 1991-1-3

characteristic value may be calculated from the formula:

$$s_e = \frac{ks^2}{\gamma}$$

where k is a factor, varying between 0 and 2.5 depending on the climate and the constituent material of the roof. The equation allows the irregularity of the snow layer shape to be taken

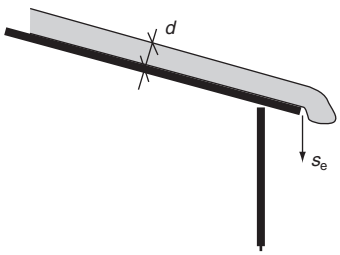


Fig. 2.3. Snow load applicable to the edge of a roof

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into account and may be determined from the formula:

$$k = \frac{3}{d\,(\text{metres})} \leq d\gamma$$

where γ is the snow density which may be taken equal to $3\,\text{kN/m}^3$ (recommended value) in the absence of more precise data.

2.3. Wind actions on bridges (EN 1991-1-4)

2.3.1. General

Section 8 of EN 1991-1-4 gives rules for the determination of quasi-static effects of natural wind actions (aerodynamic effects due to trains along the rail track are defined in EN 1991-2, see Chapter 6 of this Designers' Guide) for the structural design of bridges (decks and piers). These rules are applicable to bridges having no span greater than 200 m, the height of the deck above ground being less than 200 m, and not subject to aerodynamic phenomena (see Section 2.3.6 below). EN 1991-1-4 indicates that for normal road and railway bridge decks of less 40 m span, a dynamic response procedure is generally not needed. *cl. 1.1(2): EN 1991-1-4*

EN 1991-1-4 is applicable to single bridge decks with one or more spans of classical cross-section (slab bridges, girder bridges, box-girders, truss bridges, etc.) and constant depth. Examples are given in Fig. 2.4. *cl. 8.1: EN 1991-1-4*

Aerodynamic effects of passing vehicles are outside the scope of this part. Aerodynamic effects induced by passing trains are described in EN 1991-2, 6.6 (and see Chapter 6 of this Designers' Guide). *cl. 8.3.1(7): EN 1991-1-4*

Specific provisions may have to be defined for unusual cross-sections. Arch, suspension or cable-stayed, roofed, moving bridges and bridges including multiple or significantly curved decks are normally excluded from the field of application of the Eurocode, but the general procedure is applicable with some additional rules which may be defined in the National Annex or for the individual project. *cl. 8.1(1): EN 1991-1-4*

For skew bridges the rules given in *Section 8* of the Eurocode may be considered as approximations whose acceptability depends on the skew angle.

For the design of bridges during execution, see Chapter 3 of this Designers' Guide.

Where two similar decks are located at the same level (e.g. two decks bearing the two carriageways of a motorway) and separated transversally by a gap not significantly exceeding 1 m, the wind force on the windward structure may be calculated as if it were a single structure. On the leeward deck the wind force may be taken as the difference between the wind forces calculated for the combined decks and those for the windward deck alone. Where the decks are dissimilar or the air gap significantly exceeds 1 m, each deck may be considered separately without any allowance for shielding. *Note 3 to cl. 8.3.1(1): EN 1991-1-4*

2.3.2. Notation

In *Section 8* of EN 1991-1-4, whose scope is devoted to wind actions, the symbols defined in the Eurocode are used; to aid understanding, these are supplemented here by a few extra symbols.

Wind actions on bridges produce forces in the x, y and z directions as shown in Fig. 2.5, where:

- x is the direction parallel to the deck width, perpendicular to the span
- y is the direction along the span
- z is the direction perpendicular to the deck.

The significant dimensions of the bridge deck are:

- L length in y-direction
- b width in x-direction
- d depth in z-direction.

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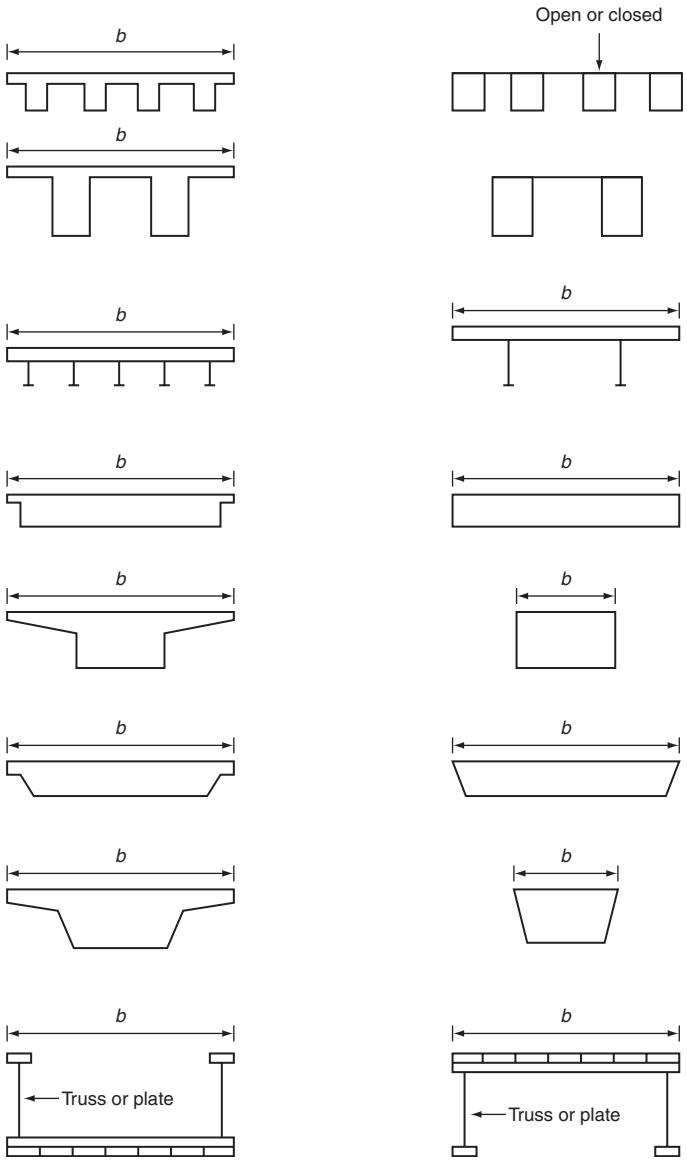


Fig. 2.4. Examples of bridge deck cross-sections

2.3.3. Reference areas for bridge decks

Design wind forces are due to the application of wind pressures to reference areas. In the case of bridges, pressures act on: the deck; its piers; its equipment, such as road restraint systems (parapets and barriers), acoustic screens, etc.; and on traffic vehicles (road vehicles or trains). Wind actions on bridge piers are examined in Section 2.3.6 below.

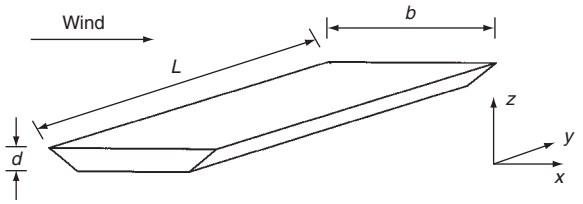


Fig. 2.5. Directions of wind actions

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Reference area in the x-direction

In the x-direction, the total effective reference area $A_{\mathrm{ref},x}$, for combinations of actions, is different depending on the presence or not of traffic on the bridge deck. If traffic loads are the leading action in the combination of actions, an additional height is taken into account for the determination of wind forces. In this Designers' Guide, this additional height is denoted d^* for road bridges and d^{**} for railway bridges. *cl. 8.3.1(4): EN 1991-1-4*

In the absence of traffic loads, the method for the determination of $A_{\mathrm{ref},x}$ is described:

(a) for decks with plain (web) beams, the sum of ***(see Figure 8.5 and Table 8.1 of EN 1991-1-4)****:* *Fig. 8.5 and Table 8.1: EN 1991-1-4*
- *(1) the face area of the front main girder*
- *(2) the face area of those parts of the other main girders projecting under (underlooking) this first one*
- *(3) the face area of the part of one cornice or footway or ballasted track projecting over the front main girder*
- *(4) the face area of solid restraints or noise barriers, where relevant, over the area described in (3) or, in the absence of such equipment, 0.3 m for each open parapet or barrier.*

(b) for decks with trussed girders, the sum of:
- *(1) the face area of one cornice or footway or ballasted track*
- *(2) those solid parts of all main truss girders in normal projected elevation situated above or underneath the area as described in (1)*
- *(3) the face area of solid restraints or noise barriers, if relevant, over the area described in (1) or, in the absence of such equipment, 0.3 m for each open parapet or barrier.*

However, the total reference area should not exceed that obtained from considering an equivalent plain (web) beam of the same overall depth, including all projecting parts.

(c) for decks with several main girders during construction, prior to the placement of the carriageway slab: the face area of two main girders.

If the effects of traffic loads are taken into account for the bridge deck, the additional depths, see Fig. 2.6, are: *cl. 8.3.1(5): EN 1991-1-4*

- $d^* = 2$ m, from the level of the carriageway, on the most unfavourable length, independently of the location of the vertical traffic loads
- $d^{**} = 4$ m from the top of the rails, on the total length of the bridge.

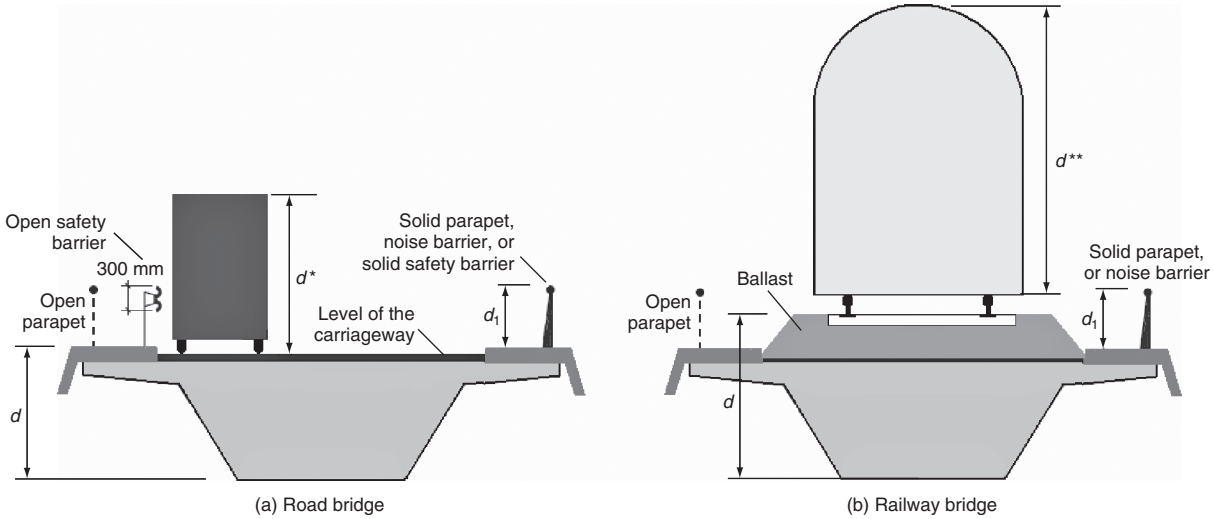


Fig. 2.6. Parameters and dimensions for the determination of wind forces

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Table 2.4. Additional depth to be used for the assessment of $A_{\text{ref},x,\text{l}}$

Road restraint system	On one side	On both sides
Open parapet or open safety barrier	$d' = 300$ mm	$2d' = 600$ mm
Solid parapet or solid safety barrier	d_1	$2d_1$
Open parapet and open safety barrier	$d' = 600$ mm	$2d' = 1200$ mm

Table 8.1: EN 1991-1-4

The additional area due to the presence of parapets or barriers is assessed from an additional depth d' or d_1 as given in Table 2.4, where d_1 is the nominal height of a solid parapet or a solid safety barrier.

Figure 2.6 also illustrates the various depths or parameters to be taken into account for the calculation of wind forces in the case of decks with plain (web) beams.

cl. 8.3.3(2): EN 1991-1-4

Reference area in the z-direction

The reference area $A_{\text{ref},z} = L \times b$ is equal to the plan area.

2.3.4. Height of the bridge deck

cl. 8.3.1(6): EN 1991-1-4

The height of the bridge deck is a parameter for assessment of the wind action on it. The reference height, z_e, is taken as the distance from the lowest ground level to the centre line of the bridge deck structure, disregarding other parts of the reference areas (Fig. 2.7).

2.3.5. Procedure for the determination of quasi-static wind forces on bridge decks

Two procedures are defined in the Eurocode for the determination of quasi-static wind forces: a 'developed' procedure and a 'simplified' procedure. The developed procedure is presented hereafter as a sequence of steps, but no details are given on the determination of the various coefficients. The simplified procedure is explained in 'Simplified method for assessment of wind force in x-direction' below.

Step 1: Fundamental value of basic wind velocity

cl. 4.2: EN 1991-1-4

In the absence of any traffic on the bridge, the fundamental value of basic wind velocity, $v_{\text{b},0}$, is the fundamental parameter for all civil engineering structures. It is taken from the national wind map or from national tables for the individual project.

Step 2: Basic wind velocity

cl. 4.2(2)P: EN 1991-1-4

For the determination of the characteristic value of wind forces, the basic wind velocity is calculated from the formula:

$$v_{\text{b}} = c_{\text{dir}} c_{\text{season}} v_{\text{b},0}$$

where c_{dir} is the directional factor and c_{season} is the season factor.

In general, the global factor $c_{\text{dir}} c_{\text{season}}$ may be taken equal to 1, so that $v_{\text{b}} = v_{\text{b},0}$. For the execution phase, see Chapter 3 of this Designers' Guide.

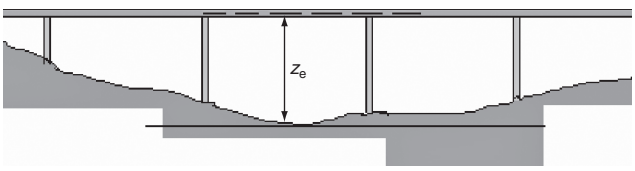


Fig. 2.7. Reference height above ground for a bridge deck

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Step 3: Determination of the mean wind velocity depending on height

In accordance with the definition, the mean wind velocity at height z above ground is determined from the following formula: *cl. 4.3.1: EN 1991-1-4*

$$v_m(z) = c_r(z)c_0(z)v_b$$

where

cl. 4.3.3: EN 1991-1-4

$c_r(z)$ is the roughness factor

$c_0(z)$ is the orography factor (taking account of the presence of hills, cliffs, etc.). In general, it may be taken equal to 1, so that $v_m(z) = c_r(z)v_b$. *cl. 4.3.2: EN 1991-1-4*

Step 4: Determination of the mean velocity pressure at height z

$$q_b(z) = \tfrac{1}{2}\rho v_m^2(z)$$

with $\rho =$ air density $= 1.25\,\text{kg/m}^3$.

Step 5: Determination of peak velocity pressure

$$q_p(z) = c_e(z)q_b(z) \qquad \text{EN 1991-1-4, 4.5}$$

where $c_e(z)$ is the exposure coefficient. The developed recommended expression of this coefficient is: *cl. 4.4 and 4.5: EN 1991-1-4*

$$c_e(z) = 1 + 7I_v(z)$$

where $I_v(z)$ is turbulence intensity at height z and is equal to:

$$I_v(z) = \frac{k_I}{c_0(z)\ln(z/z_0)} \text{ for } z_{min} \le z \le z_{max}$$

$$I_v(z) = \frac{k_I}{c_0(z_{min})\ln(z_{min}/z_0)} \text{ for } z \le z_{min}$$

where

k_I is the turbulence factor, generally equal to 1.0

z_0 is the roughness length, depending on the terrain category.

It is assumed that the methodology for the determination of the peak velocity pressure is applicable to the wind pressures accompanying road and railway traffic.

Step 6: Determination of the wind force on the bridge deck in the x-direction

Basic expression

The basic expression of the wind force on the bridge deck in the x-direction is given as $F_{Wk,x}$ (characteristic value in the absence of traffic on the bridge deck):

$$F_{Wk,x} = c_s c_d \times c_f \times q_p(z_e) \times A_{ref,x}$$

where

$c_s c_d$ is a structural factor which can be interpreted as the product of two other factors: a size factor c_s (which takes into account the reduction effect on the wind action due to the non-simultaneity of occurrence of the peak wind pressures on the whole surface) and a dynamic factor c_d (which takes into account the increasing effect from vibrations due to the turbulence in resonance with the structure). In the quasi-static procedure, $c_s c_d$ may be taken equal to 1.0 for bridges (the two factors compensate each other)

c_f is the drag (or force) coefficient, noted $c_{f,x}$ for the wind force in the x-direction. *cl. 8.3.1(1): EN 1991-1-4*

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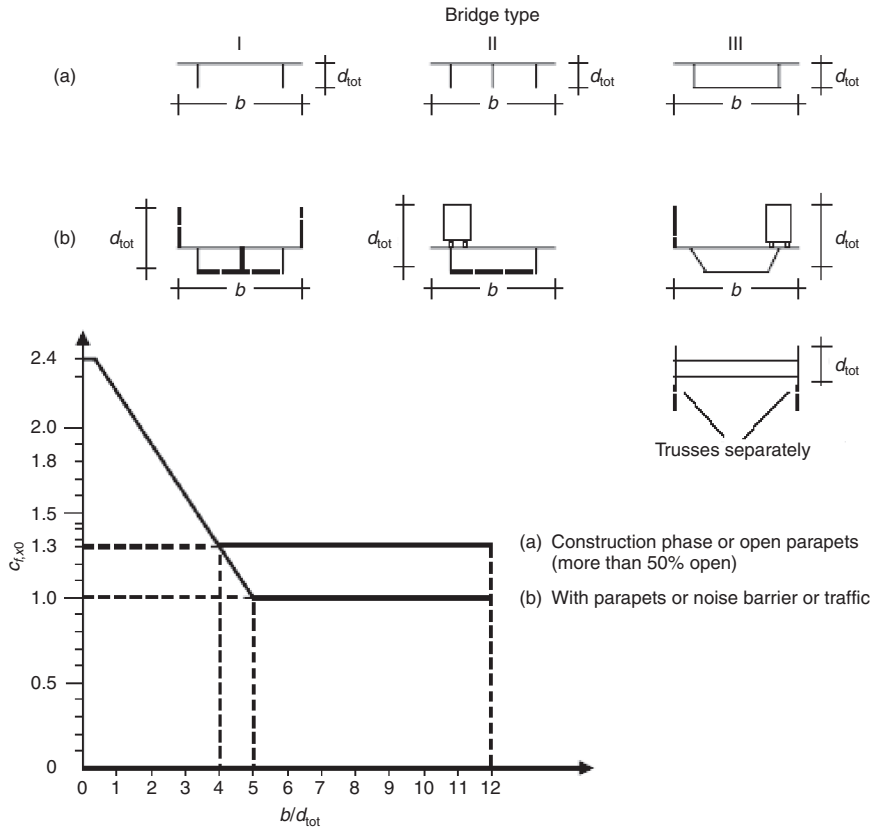


Fig. 2.8. Force coefficient for bridges, c_{f,x_0} (see EN 1991-1-4, Figure 8.3)

Determination of the drag coefficient $c_{f,x}$

In general, the drag coefficient for wind action on bridge decks in the x-direction may be taken from the formula:

$$c_{f,x} = c_{f,x_0}$$

where

c_{f,x_0} is the force coefficient without free-end flow. Indeed, in the case of a common bridge, the wind flow is deviated only along two sides (over and under the bridge deck), which explains why it usually has no free-end flow.

Note 2 to cl. 8.3.1(1): EN 1991-1-4

For bridges for which the Eurocode is applicable, the recommended value of c_{f,x_0} is equal to 1.30; however, it may also be taken from Fig. 2.8. It should be noted that the wind direction may be inclined compared to the deck surface due to the slope of the terrain in the oncoming wind direction. The field of validity of the value 1.30 or of Fig. 2.8 corresponds to an angle of inclination within the range of values (-10° to $+10^\circ$). Where the angle of inclination of the wind exceeds 10°, special studies are recommended for the determination of the drag coefficient.

cl. 8.3.1(2): EN 1991-1-4

Where the windward face is inclined to the vertical (Fig. 2.9), the drag coefficient c_{f,x_0} may be reduced by 0.5% per degree of inclination, α_1, from the vertical, limited to a maximum reduction of 30%.

cl. 8.3.1(3): EN 1991-1-4

Where a bridge deck is sloped transversally, c_{f,x_0} should be increased by 3% per degree of inclination, but not more than 25%.

Important note

EN 1991-1-4 defines two basic wind speeds to be taken into account when traffic loads are applied to the bridge deck: $v^*_{b,0}$ for road bridges (23 m/s) and $v^{**}_{b,0}$ for railway bridges (25 m/s). When the leading action of the combination of actions (see Chapter 8) is the

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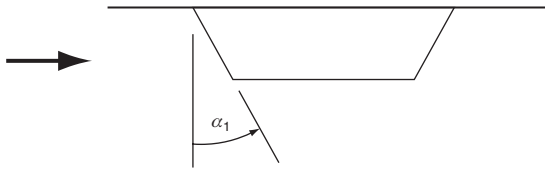


Fig. 2.9. Bridge with inclined windward face

traffic action, wind actions may be taken into account as accompanying actions – they are normally represented by the symbol $\psi_0 F_{Wk}$ where F_{Wk} is the characteristic value calculated on the depth of the deck, including the additional depths d^* and d^{**} where relevant, and ψ_0 is the combination factor.

EN 1991-1-4 recommends limiting the value of $\psi_0 F_{Wk}$ to the values F_W^* or F_W^{**} calculated from the basic wind speeds $v_{b,0}^*$ and $v_{b,0}^{**}$. In fact, these wind speed values should be considered as basic values, with the same definition as $v_{b,0}$, which is meaningless. At the ENV stage, the intention was to define a maximum uniform wind speed compatible with real traffic; but it appears that this unform wind speed is meaningless because wind actions always fluctuate with time and the procedure defined in EN 1991-1-4 is intended to calculate peak values.

Therefore, it is recommended by this Designers' Guide to ignore the concept corresponding to forces F_W^* or F_W^{**} and to adopt the following position.

If the wind action is the unique variable action of the combination of actions (see Chapter 8 of this Designers' Guide), its magnitude (characteristic value) is calculated with the depth of the deck as defined in Section 2.3.3 above. If the leading action of the combination of actions is due to traffic loads, the wind action is an accompanying action and is calculated with a reference area including the additional depths d^* or d^{**} according to the relevant rules previously explained. This method is illustrated in Fig. 2.10 for road bridges.

Simplified method for assessment of wind force in x-direction

The characteristic value of the wind force in the x-direction may be obtained using the following expression: *cl. 8.3.2: EN 1991-1-4*

$$F_{Wx} = \tfrac{1}{2}\rho v_b^2 C A_{ref,x}$$

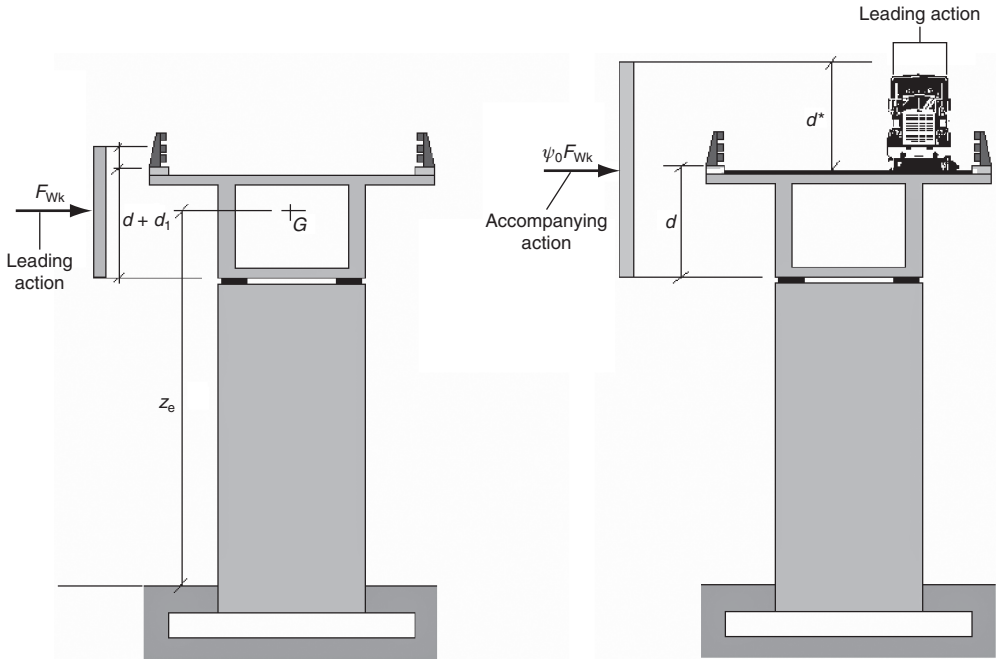


Fig. 2.10. Determination of wind actions (leading or accompanying actions) in the case of road bridges

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Table 2.5. Wind load factor C for bridges (Data taken from EN 1991-1-4, Table 8.2)

b/d_{tot}	$z_e \leq 20$ m	$z_e = 50$ m
≤ 0.5	6.7	8.3
≥ 4.0	3.6	4.5

where C is a 'global' wind load factor ($C = c_e \times c_{f,x}$) as given in Table 2.5, the values being based on the following assumptions:

- terrain category II according to Table 4.1 of EN 1991-1-4
- force coefficient $c_{f,x}$ according to ***Clause 8.3.1(1)*** *cl. 8.3.1(1): EN 1991-1-4*
- the orography factor $c_o = 1.0$
- the turbulence factor $k_I = 1.0$.

Table 2.5 has been established as follows:

$$c_e(z) = [1 + 7I_V(z)]c_r^2(z)$$

$$c_r(z) = k_r \ln\left(\frac{z}{z_0}\right)$$

$$k_r = 0.19\left(\frac{z_0}{z_{0,II}}\right)^{0.07} = 0.19 z_0 = z_{0,II} = 0.05 \text{ metres}$$

$$I_v(z) = \frac{1}{\ln(z/z_0)}$$

Therefore:

$$c_e(z) = \left[1 + \frac{7}{\ln(z/z_0)}\right](0.19)^2 \ln^2(z/z_0) = 0.0361 \ln^2(z/z_0) + 0.2527 \ln(z/z_0)$$

For $z_e \leq 20$ m, the values correspond to $z_e = 20$ m

$$c_e(z) = 0.0361 \ln^2(400) + 0.2527 \ln(400) = 2.809$$

- for $b/d_{tot} \leq 0.5$, $c_{f,x} = 2.4 \Rightarrow C = 2.809 \times 2.4 = 6.74$
- for $b/d_{tot} \geq 4.0$, $c_{f,x} = 1.3 \Rightarrow C = 2.809 \times 1.3 = 3.65$

For $z_e = 50$ m

$$c_e(z) = 0.0361 \ln^2(1000) + 0.2527 \ln(1000) = 3.468$$

- for $b/d_{tot} \leq 0.5$, $c_{f,x} = 2.4 \Rightarrow C = 3.468 \times 2.4 = 8.32$
- for $b/d_{tot} \geq 4.0$, $c_{f,x} = 1.3 \Rightarrow C = 3.468 \times 1.3 = 4.50$

The global wind force is applied to the whole reference area.

For intermediate values of b/d_{tot} linear interpolation may be used.

The reduction for an inclined windward face is not applicable with this simplified method.

Determination of wind forces in y- and z-directions

cl. 8.3.3 and 8.3.4: EN 1991-1-4

In general, the longitudinal wind forces in the y-direction need not be taken into account. Nevertheless, if considered necessary, the Eurocode gives the following simplified rules:

- for plated bridges, 25% of the wind forces in the x-direction
- for truss bridges, 50% of the wind forces in the x-direction.

For the assessment of wind forces in the z-direction (lift forces), the same procedure as for wind forces in the x-direction is to be adopted as in EN 1991-1-4. The relevant expression is:

$$F_{Wk,z} = c_{f,z} \times q_p(z_e) \times A_{ref,z}$$

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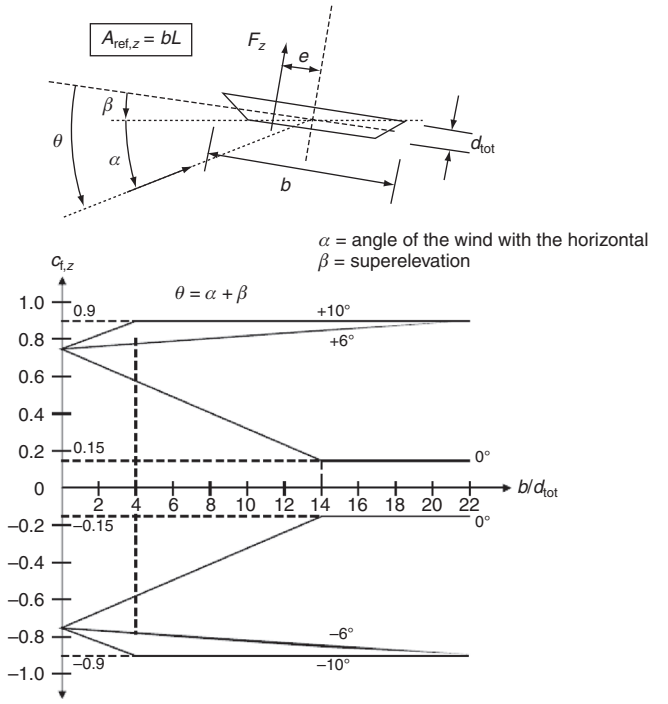


Fig. 2.11. Force coefficient $c_{f,z}$ for bridges with transversal slope and wind inclination

The force coefficient, $c_{f,z}$, which should be defined for the particular project, may be taken from Fig. 2.11. In using it:

- the depth d may be limited to the depth of the deck structure, disregarding the traffic and any bridge equipment
- the onflow angle α may be taken as $\pm 5^\circ$ due to turbulence.

As a simplification, $c_{f,z}$ may be taken equal to 0.9. The eccentricity of the force in the x-direction may be set to $e = b/4$.

2.3.6. Wind effects on bridge piers

Wind actions on piers and pylons may be calculated by using the general format defined in *Section 8 of EN 1991-1-4*, as consistently as possible with construction elements having like shapes and dimensions or, failing that for some factors or coefficients, with the assistance of test results. The determination of wind actions on piers is important, in particular, for the design of foundations.

Piers and pylons often have a variety of shapes and dimensions, and factors and coefficients need to be commonly specified for particular projects or directly determined from wind tests. In common cases:

- the value of the $c_s c_d$ factor, for moderately slender piers with a height less than 15 m, may be taken equal to 1 in persistent design situations, and 1.2 in transient design situations. In other cases values calculated in accordance with *Section 6 of EN 1991-1-4* are generally acceptable
- for the values of the force coefficients, reference may be made to *Clauses 7.2.2, 7.4, 7.6, 7.7, 7.8 and 7.9 of EN 1991-1-4*.

Specifically for tall bridge piers or pylons, it is possible to use EN 1991-1-4 for a first approach of wind effects. Hereafter, the main steps of the calculation process are identified from EN 1991-1-4.

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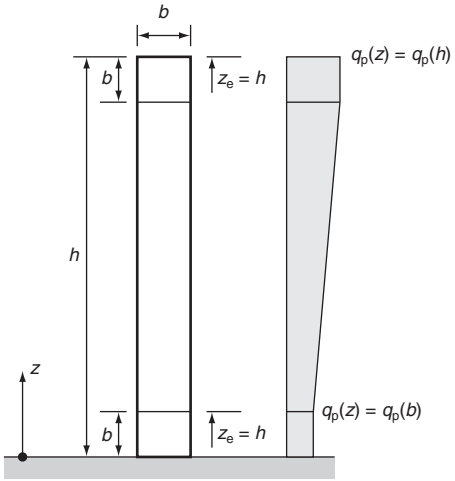


Fig. 2.12. Reference height depending on *h* and *b*, and corresponding velocity pressure profile

Expression (5.3): EN 1991-1-4

The general expression of the wind force as reproduced from ***Expression (5.3) of EN 1991-1-4*** is as follows:

$$F_W = c_s c_d \times c_f \times q_p(z_e) \times A_{ref}$$

cl. 5.3.2: EN 1991-1-4

and the wind force acting on the structure may be determined by vectorial summation over the individual structural elements by using the following expression:

$$F_W = c_s c_d \times \sum_{\text{elements}} c_f \times q_p(z_e) \times A_{ref}$$

A procedure is given in *EN 1991-1-4 Clause 7.2.2* for buildings, but it may be applied to bridge piers higher than 15 m. Figure 2.12 shows an adaptation of the rules given for vertical walls or buildings rectangular in plan.

2.3.7. Specific combination rules for wind actions

The forces exerted on various parts of a bridge by a wind blowing in the same direction (e.g. piers) should be considered as simultaneous if they are unfavourable, in particular for the design of foundations.

The forces produced in the *x*- and *y*-directions are due to wind blowing in different directions and normally are not simultaneous. The forces produced in the *z*-direction can result from the wind blowing in a wide range of directions; if they are unfavourable and significant, they should be taken into account as simultaneous with the forces produced in any other direction.

The wind actions on bridge decks and their supporting piers should be calculated by identifying the most unfavourable direction of the wind on the whole structure for the effect under consideration. However, if a bridge has a small angle of skew, it is sufficient to calculate separately the wind actions on deck and piers and then to cumulate them.

cl. 8.4.1(1): EN 1991-1-4

2.4. Thermal actions (EN 1991-1-5)

Eurocode 1 Part 1-5 (EN 1991-1-5) defines the thermal actions to be taken into account for bridges. For the calculation of these actions, the thermal expansion coefficient of materials is needed. For example, for traditional steel and concrete, it is $\alpha_T = 12 \cdot 10^{-6}/^{\circ}\text{K}$ but values for other materials are given by the EN 1991-1-5.

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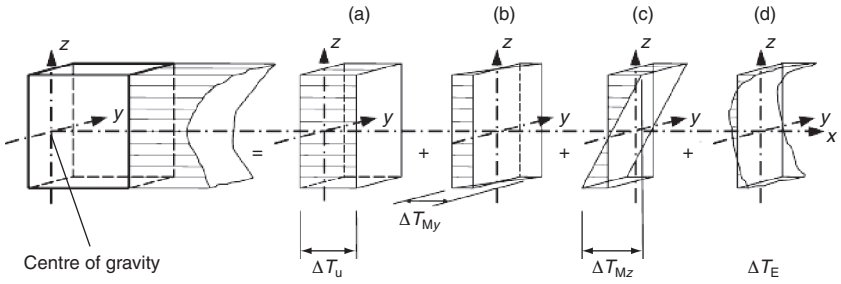


Fig. 2.13. Diagrammatic representation of constituent components of a temperature profile

2.4.1. Actions of temperature in bridge decks

cl. 6.1.1: EN 1991-1-5

EN 1991-1-5 distinguishes three types of bridge decks:

Type 1	Steel deck	Steel box girder Steel truss or plate girder
Type 2	Composite deck	
Type 3	Concrete deck	Concrete slab Concrete beam Concrete box-girder

The thermal effects in bridge decks are represented by the distribution of the temperature resulting from the sum of the four terms (Fig. 2.13): (a) component of the uniform temperature, (b) and (c) components of the temperature linearly variable according to two axes contained in the plan of the section, and (d) a residual component.

Section 4: EN 1991-1-5

Uniform component

The extreme characteristic values of the uniform temperature component are given in the national temperature map. These values are based on a return period of 50 years, but formulae are given in Annex A, derived from a Gumbel law (law of extreme values of type I) for the assessment of extreme temperatures based on a different return period. For the sake of user-friendliness, the application of these formulae is represented diagrammatically (Fig. 2.14) as ratios between the maximum (minimum) for a probability of exceedence p and the maximum (minimum) for a return period of 50 years (probability of exceedence = 0.02).

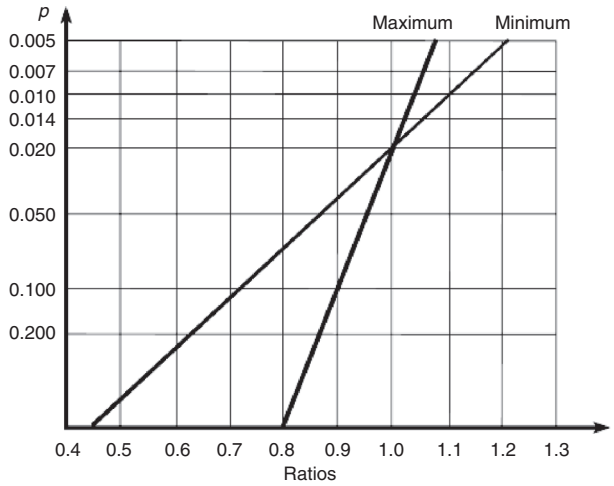


Fig. 2.14. Ratios $T_{max,p}/T_{max}$ and $T_{min,p}/T_{min}$

Figure A.1: EN 1991-1-5

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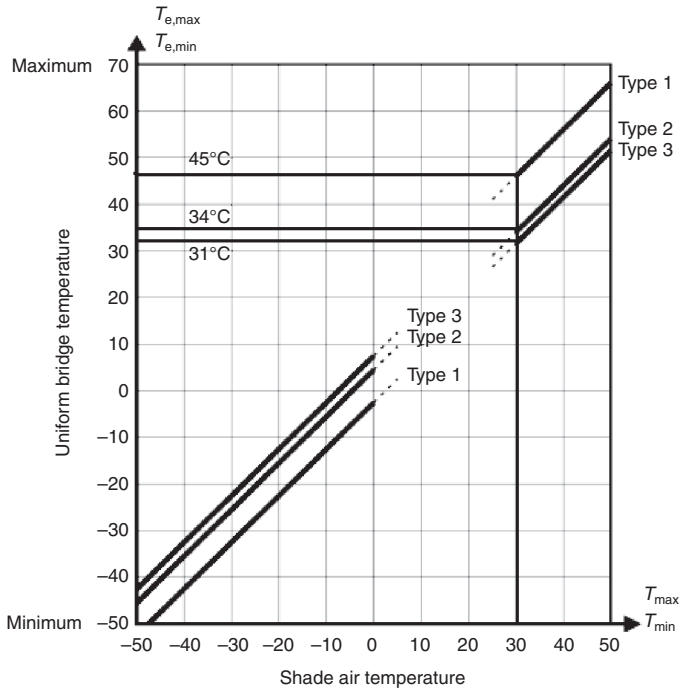


Fig. 2.15. Correlation between the min/max shade air temperature (T_{min}/T_{max}) and min/max uniform bridge temperature component ($T_{e,min}/T_{e,max}$)

Figure 6.1: EN 1991-1-5

The maximum and minimum characteristic values of effective temperatures in bridges, denoted $T_{e,max}$ and $T_{e,min}$, are determined from the maximum and minimum shade air temperature, noted T_{max} and T_{min}, which are given in the National Annex. Figure 2.15 shows the correlation between the shade air temperature and the effective temperature of the bridge. For example, for a characteristic value of 30°C for shade air temperature, the characteristic effective uniform temperature is approximately equal to 31°C for a bridge of type 3, 34°C for a bridge of type 2 and 45°C for a bridge of type 1.

For the design of expansion joints and bearings, the characteristic range ($T_{e,min}/T_{e,max}$) of the variation of temperature is considered around an average (or probable) effective value, denoted T_0. In the absence of any specification for the individual project, the following extreme range of values of temperatures may be used for the design of expansion joints (total opening) and bearings (Fig. 2.16):

$$T_{e,max} - T_{e,min} + 2S$$

cl. 6.1.3.3(3): EN 1991-1-5

The recommended value for S is given in EN 1991-1-5; if temperature T_0 is normally foreseeable at the time of installation of the bearings or expansion joints, S may be taken equal to 10°C. If the temperature T_0 is unknown, S may be taken equal to 20°C. In the National Annexes, these values may be adjusted and slightly differentiated between joint opening and bearing movement.

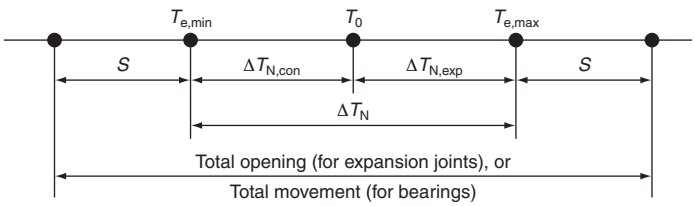


Fig. 2.16. Temperature variations for the design of expansion joints and bearings

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Table 2.6. Recommended values of linear temperature difference component for different types of bridge decks for road, foot and railway bridges (Data taken from EN 1991-1-5, Table 6.1; see EN 1991-1-5 for missing values)

Type of deck	Top warmer than bottom $\Delta T_{M,heat}$ (°C)	Bottom warmer than top $\Delta T_{M,cool}$ (°C)
Type 1:		
Steel deck	18	
Type 2:		
Composite deck	15	
Type 3:		
Concrete deck		
– concrete box girder	10	
– concrete beam	15	
– concrete slab	15	

Other components

In most cases, only the component of uniform temperature and the linear component in the vertical direction are taken into account for the design of bridge decks. However, in certain cases it may be necessary to take in account the horizontal linear component. In the absence of precise requirements, a value of 5°C is recommended as the characteristic value of the linear difference of temperature between the outer edges of the deck.

cl. 6.1.4.3: EN 1991-1-5

Concerning the linear temperature variation in the vertical direction, EN 1991-1-5 defines positive and negative temperature differences between the top and the bottom of bridge decks. The variation of temperature is assumed to be linear. The characteristic values of these linear temperature differences are given in Table 2.6. The proposed values are applicable to road bridges, footbridges and railway bridges without any differentiation.

The values given in Table 2.6 represent upper bound values of the linearly varying temperature difference component for a representative sample of bridge geometries. They are based on a depth of surfacing of 50 mm for road and railway bridges. For other depths of surfacing a 'correction' factor k_{sur} is applicable to these values. Recommended values for this factor k_{sur} are given in Table 2.7.

A more refined method is based on the consideration of non-linear gradients between the bottom and the top of the deck. Diagrams of non-uniform temperature in the vertical direction for the three types of bridge decks are given in Figs 2.17, 2.18 and 2.19.

Table 2.7. Recommended values of k_{sur} to account for different surfacing thickness bridges (Data taken from EN 1991-1-5 Table 6.2; see EN 1991-1-5 for missing values)

Road, foot and railway bridges						
Surface thickness (mm)	Type 1		Type 2		Type 3	
	Top warmer than bottom k_{sur}	Bottom warmer than top k_{sur}	Top warmer than bottom k_{sur}	Bottom warmer than top k_{sur}	Top warmer than bottom k_{sur}	Bottom warmer than top k_{sur}
Unsurfaced	0.7	0.9	0.9	1.0	0.8	1.1
Water-proofed[(1)]						
50	1.0	1.0	1.0	1.0	1.0	1.0
100	0.7	1.2	1.0	1.0	0.7	1.0
150						
Ballast (750 mm)	0.6	1.4	0.8	1.2	0.6	1.0

[(1)] These values represent upper bound values for dark colour

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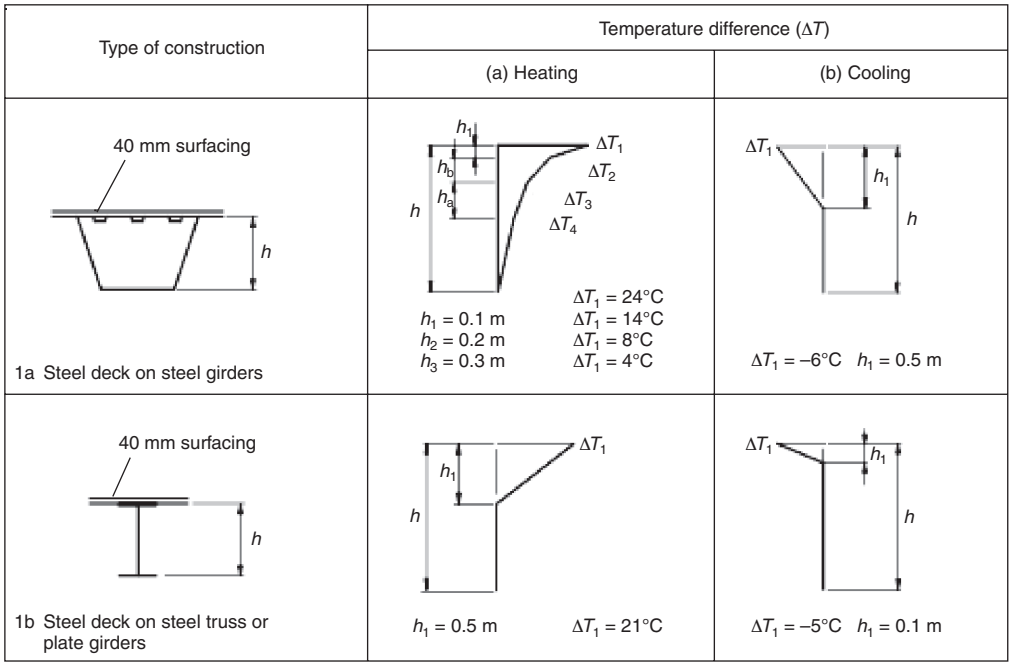


Fig. 2.17. Temperature differences for bridge decks: Type 2 – Composite decks bridges (Reproduced from EN 1991-1-5, with permission from BSI)

For composite steel and concrete decks, the temperature profiles defined in Figure 2.18 may be considered as the most suitable profiles.

2.4.2. Complementary rules

EN 1991-1-5 gives rules concerning the simultaneity of uniform and temperature difference components, and rules concerning differences in the uniform temperature component between structural elements.

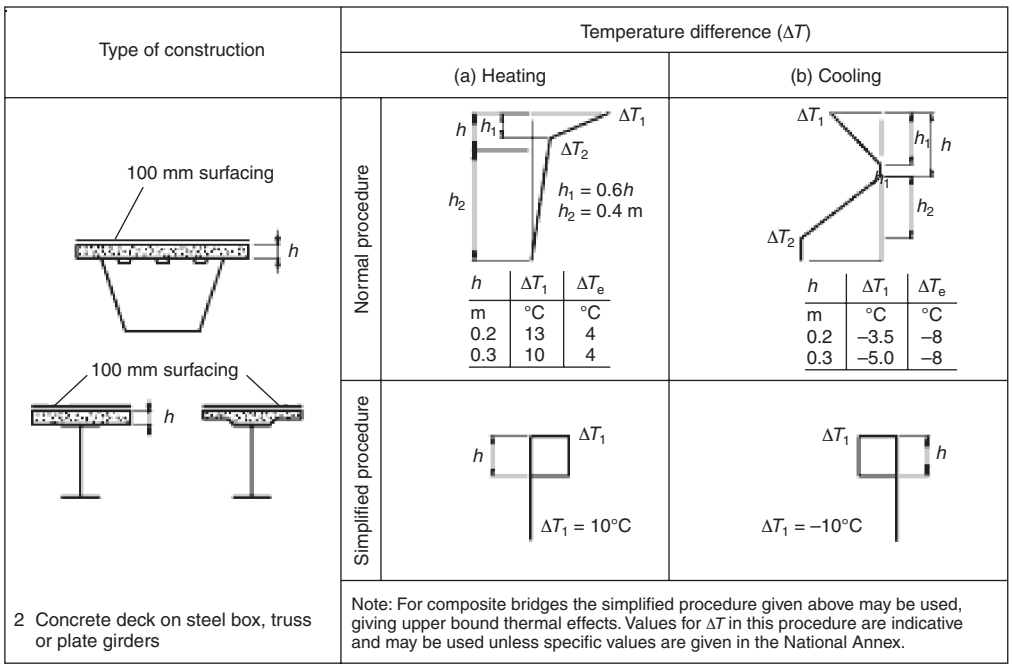


Fig. 2.18. Temperature differences for bridge decks: Type 3 – Concrete decks bridges (Reproduced from EN 1991-1-5, with permission from BSI)

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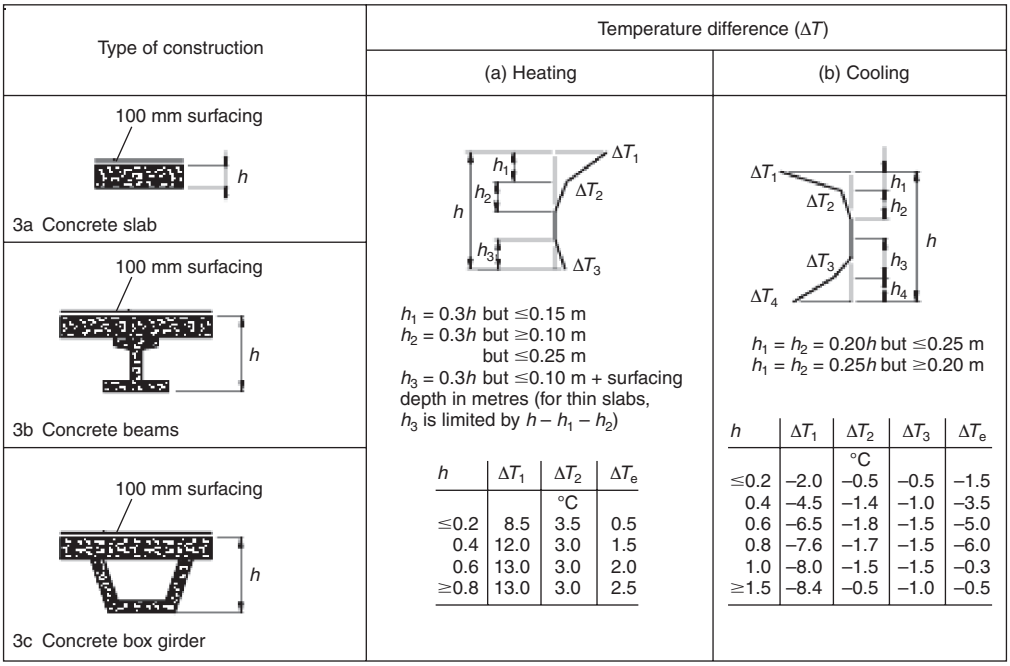


Fig. 2.19. Temperature differences for bridge decks: Type 3 – concrete decks bridges (see EN 1991-1-5, Figure 6.2c)

Simultaneity of the uniform and temperature difference components

The uniform temperature component gives rise to action effects in framed bridges such as portal bridges or arch bridges when they are statically undetermined. Physically, the two components (uniform and temperature difference) exist and they have to be taken into account simultaneously. Of course, they cannot be both represented by their characteristic value. For that reason, EN 1991-1-5 recommends two expressions that can be termed 'sub-combinations': *cl. 6.1.5: EN 1991-1-5*

$$\Delta T_{\mathrm{M,heat}}(\text{or } \Delta T_{\mathrm{M,cool}}) + \omega_{\mathrm{N}} \Delta T_{\mathrm{N,exp}}(\text{or } \Delta T_{\mathrm{N,con}})$$

or

$$\omega_{\mathrm{M}} \Delta T_{\mathrm{M,heat}}(\text{or } \Delta T_{\mathrm{M,cool}}) + \Delta T_{\mathrm{N,exp}}(\text{or } \Delta T_{\mathrm{N,con}})$$

the 'sub-combination' giving the most adverse effect being chosen. The recommended values of ω_{N} and ω_{M} are:

$$\omega_{\mathrm{N}} = 0.35 \text{ and } \omega_{\mathrm{M}} = 0.75$$

which gives:

$$\Delta T_{\mathrm{M,heat}}(\text{or } \Delta T_{\mathrm{M,cool}}) + 0.35 \Delta T_{\mathrm{N,exp}}(\text{or } \Delta T_{\mathrm{N,con}})$$

or

$$0.75 \Delta T_{\mathrm{M,heat}}(\text{or } \Delta T_{\mathrm{M,cool}}) + \Delta T_{\mathrm{N,exp}}(\text{or } \Delta T_{\mathrm{N,con}})$$

Where both linear and non-linear vertical temperature differences are used ΔT_{M} should be replaced by ΔT which includes ΔT_{M} and ΔT_{E}. *cl. 6.1.4.2: EN 1991-1-5*

Differences in the uniform temperature component between different structural elements

In some cases, differences in the uniform temperature component between different types of structural elements may cause unfavourable action effects. Such circumstance are encountered, for example, in suspension or cable-stayed bridges where temperature differences may develop between the deck and the supporting cables.

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cl. 6.1.6: EN 1991-1-5

In the absence of specification for the individual projet, EN 1991-1-5 recommends the following temperature differences:

- 15°C between main structural elements (e.g. tie and arch)
- 10°C and 20°C for light and dark colour respectively between suspension/stay cables and deck (or tower).

cl. 6.2: EN 1991-1-5

2.4.3. Actions of temperature in the piers of bridges

EN 1991-1-5 prescribes to take in account the effects of a linear gradient of temperature between opposite surfaces of piers. If not specified for the individual project, it seems appropriate to consider a characteristic value for the linear gradient equal to 5°C in the case of concrete piers, hollowed or full.

Moreover, it is necessary to consider, a difference of temperature between internal and external faces of a wall (in the case of hollowed piers) for which, in the absence of particular indications, the recommended characteristic value is 15°C. For steel piers, expert advice may be needed.

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Annex A to Chapter 2: Aerodynamic excitation and aeroelastic instabilities

A2.1. General – aerodynamic excitation mechanisms

For the design of flexible bridges, the most appropriate analysis has to be selected between a quasi-static or a dynamic response procedure. In most cases, normal road and railway bridge decks with spans less than 40 m do not need any dynamic analysis under wind actions. Some flexible bridges may be susceptible to various forms of aerodynamic excitation which are briefly described in this annex. In fact, the need of a dynamic response procedure for the design of a flexible bridge is a matter of engineering judgement. Informative Annexes E and F of EN 1991-1-4 give guidance to recognise where a dynamic response procedure may be appropriate.

Note 3 to cl. 8.2(1): EN 1991-1-4

A2.1.1. Limited amplitude response

This phenomenon includes both vortex-induced oscillations and turbulence response induced by the forces and moments developed by wind gusts on bridge decks. The fluctuations of aerodynamic forces and moments are due to:

- fluctuations of the wind velocity itself (turbulence in the wind direction)
- the wind inclination to the horizontal (vertical turbulence, which generates fluctuations of the angle between the wind direction and the deck plane).

The forces and moments can fluctuate over a wide range of frequencies and if sufficient energy is present in frequency bands encompassing one or more natural frequencies of the structure then vibration may occur.

Proximity effects such as wake buffeting may also cause large turbulence response.

Limited amplitude response can cause unacceptable stresses or fatigue damage.

A2.1.2. Divergent amplitude response

Divergent amplitude response can cause amplitudes which rapidly increase to large values, and may lead to structural damage. Identifiable aerodynamic mechanisms leading to oscillations of this type include the following:

- Galloping and stall flutter. Galloping instabilities arise on certain shapes of deck cross-section because of the characteristics of the variation of the wind drag, lift and pitching moments with angle of incidence or time.
- Classical flutter. This involves coupling (i.e. interaction) between the vertical bending and torsional oscillations.

A2.1.3. Non-oscillatory divergence

Non-oscillatory divergence is a form of aerodynamic torsional instability which can occur if the aerodynamic torsional stiffness is negative. At a critical wind speed the negative aerodynamic stiffness becomes numerically equal to the structural torsional stiffness resulting in zero total stiffness, which may lead to structural damage and therefore should be avoided.

A2.2. Dynamic characteristics of bridges

Important note: Section A2.2 is restricted to giving guidance on the clauses relating to bridges in EN 1991-1-4 Annex F. It gives the basic information for the application of EN 1991-1-4 Annex E and the determination of some important parameters. For that reason, this section is placed before the section devoted to vortex shedding and aeroelastic instabilities.

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F.1: EN 1991-1-4

In EN 1991-1-4 Annex F (Informative), calculation methods assume that structures have a linear elastic behaviour and classical normal modes. Dynamic structural properties are therefore characterised by:

- natural frequencies
- modal shapes
- equivalent masses
- logarithmic decrements of damping.

The fundamental vertical bending frequency $n_{1,B}$ of a plate or box girder bridge may be approximately derived from the following expression:

$$n_{1,B} = \frac{K^2}{2\pi L^2}\sqrt{\frac{EI_b}{m}} \qquad \text{EN 1991-1-4, (F.6)}$$

where

L is the length of the main span in metres
E is Young's modulus in N/mm^2
I_b is the second moment of area of cross-section for vertical bending at mid-span in m^4
m is the mass per unit length of the full cross-section at midspan (for permanent loads) in kg/m
K is a dimensionless factor depending on span arrangement defined hereafter.

(a) For single-span bridges
$K = \pi$ if simply supported, or
$K = 3.9$ if propped cantilevered, or
$K = 4.7$ if fixed end supports.

(b) For two-span continuous bridges
K is obtained from Fig. A2.1, using the curve for two-span bridges, where L_1 is the length of the side span and $L > L_1$.

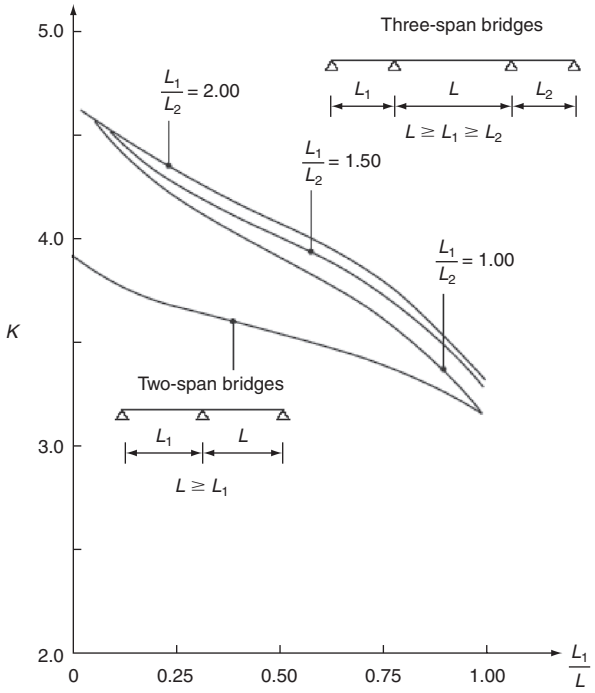


Figure F.2: EN 1991-1-4

Fig. A2.1. Factor K used for the derivation of fundamental bending frequency

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(c) For three-span continuous bridges
K is obtained from Fig. A2.1, using the appropriate curve for three-span bridges, where L_1 is the length of the longest side span and L_2 is the length of the other side span and $L > L_1 > L_2$.

This also applies to three-span bridges with a cantilevered/suspended main span.

If $L_1 > L$ then K may be obtained from the curve for two-span bridges, neglecting the shortest side span and treating the largest side span as the main span of an equivalent two-span bridge.

(d) For symmetrical four-span continuous bridges (i.e. bridges symmetrical about the central support)
K may be obtained from the curve for two-span bridges in Fig. A2.1, treating each half of the bridge as an equivalent two-span bridge.

(e) For unsymmetrical four-span continuous bridges and continuous bridges with more than four spans
K may be obtained from Fig. A2.1 using the appropriate curve for three-span bridges, choosing the main span as the greatest internal span.

Expression F.6: EN 1991-1-4

The Eurocode mentions that if the value of $\sqrt{EI_b/m}$ at the support exceeds twice the value at mid-span, or is less than 80% of the midspan value, then the ***Expression (F.6) of EN 1991-1-4*** (see above) should not be used unless very approximate values are sufficient.

The fundamental torsional frequency of plate girder bridges is equal to the fundamental bending frequency calculated from *Expression (F.6) of EN 1991-1-4* (see above), provided the average longitudinal bending inertia per unit width is not less than 100 times the average transverse bending inertia per unit length.

The fundamental torsional frequency of a box girder bridge may be approximately derived from the following expression:

$$n_{1,T} = n_{1,B}\sqrt{P_1(P_2 + P_3)} \qquad \text{EN 1991-1-4, (F.7)}$$

with:

$$P_1 = \frac{mb^2}{I_p} \qquad \text{EN 1991-1-4, (F.8)}$$

$$P_2 = \frac{\sum r_j^2 I_j}{b^2 I_p} \qquad \text{EN 1991-1-4, (F.9)}$$

$$P_3 = \frac{L^2 \sum J_j}{2K_2 b^2 I_p (1+\nu)} \qquad \text{EN 1991-1-4, (F.10)}$$

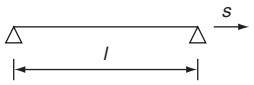
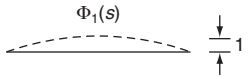
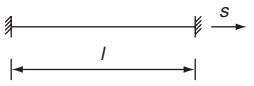
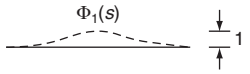
where

- $n_{1,B}$ is the fundamental bending frequency in Hz
- b is the total width of the bridge deck
- m is the mass per unit length defined above (for Expression (F.6))
- ν is Poisson's ratio of girder material
- r_j is the distance of individual box centre-line from centre-line of bridge
- I_j is the second moment of mass per unit length of individual box for vertical bending at mid-span, including an associated effective width of deck
- I_p is the second moment of mass per unit length of cross-section at midspan. It is described by the following expression:

$$I_p = \frac{m_d b^2}{12} + \sum (I_{pj} + m_j r_j^2) \qquad \text{EN 1991-1-4, (F.11)}$$

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Table A2.1. Fundamental flexural vertical mode shape for simple supported and clamped structures and structural elements (Data taken from EN 1991-1-4, Table F.1)

Scheme	Mode shape	$\Phi_1(s)$
		$\sin\left(\pi\frac{s}{\ell}\right)$
		$\frac{1}{2}\left[1-\cos\left(2\pi\frac{s}{\ell}\right)\right]$

where

m_d is the mass per unit length of the deck only, at midspan
I_{pj} is the mass moment of inertia of individual box at midspan
m_j is the mass per unit length of individual box only, at midspan, without associated portion of deck
J_j is the torsion constant of individual box at midspan. It is described by the following expression:

$$J_j = \frac{4A_j^2}{\oint \frac{ds}{t}} \qquad \text{EN 1991-1-4, (F.12)}$$

where

A_j is the enclosed cell area at midspan
$\oint ds/t$ is the integral around box perimeter of the length/thickness ratio for each portion of box wall at midspan.

Note to cl. F.2(7): EN 1991-1-4

EN 1991-1-4 mentions in a note that a slight loss of accuracy may occur if the proposed *Expression (F.12)* is applied to multibox bridges whose plan aspect ratio (i.e. span/width) exceeds 6.

The fundamental flexural vertical mode $\Phi_1(s)$ of bridges may be estimated as shown in Table A2.1.

The equivalent mass per unit length m_e of the fundamental mode is given by the following expression:

$$m_e = \frac{\int_0^\ell m(s)\Phi_1^2(s)\,ds}{\int_0^\ell \Phi_1^2(s)\,ds} \qquad \text{EN 1991-1-4, (F.14)}$$

where

m is the mass per unit length
ℓ is the height or span of the structure or the structural element
$i = 1$ is the mode number.

For structures supported at both ends of span ℓ with a varying distribution of the mass per unit length, m_e may be approximated by the average value of m over a length of $\ell/3$ centred at the point in the structure in which $\Phi_1(s)$ is maximum (see Table A2.1).

A2.2.1. Logarithmic decrement of damping

The logarithmic decrement of damping δ for fundamental bending mode may be estimated by the following expression:

$$\delta = \delta_s + \delta_a + \delta_d \qquad \text{EN 1991-1-4, (F.15)}$$

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Table A2.2. Approximate values of logarithmic decrement of structural damping in the fundamental mode, δ_s, for bridges (Data taken from EN 1991-1-4, Table F.2; see EN 1991-1-4 for missing values)

Structural type		Structural damping, δ_s
Steel bridges and lattice steel towers	Welded High-resistance bolts Ordinary bolts	0.03
Composite bridges		0.04
Concrete bridges	Prestressed without cracks With cracks	0.10
Timber bridges		
Bridges, aluminium alloys		0.02
Bridges, glass- or fibre-reinforced plastic		
Cables	Parallel cables Spiral cables	0.006

where

δ_s is the logarithmic decrement of structural damping
δ_a is the logarithmic decrement of aerodynamic damping for the fundamental mode
δ_d is the logarithmic decrement of damping due to special devices (tuned mass dampers, sloshing tanks, etc.).

Approximate values of logarithmic decrement of structural damping, δ_s, are given in Table A2.2.

The logarithmic decrement of aerodynamic damping, δ_a, for the fundamental bending mode of along-wind vibrations may be estimated by the following expression:

$$\delta_a = \frac{c_f \rho v_m(z_s)}{2 n_1 \mu_e} \qquad \text{EN 1991-1-4, (F.16)}$$

where

c_f is the force coefficient for wind action in the wind direction stated in Section 7 of EN 1991-1-4
μ_e is the equivalent mass per unit area of the structure, which for rectangular areas is given by the following expression:

$$\mu_e = \frac{\int_0^h \int_0^b \mu(y,z)\Phi_1^2(y,z)\,\mathrm{d}y\,\mathrm{d}z}{\int_0^h \int_0^b \Phi_1^2(y,z)\,\mathrm{d}y\,\mathrm{d}z} \qquad \text{EN 1991-1-4, (F.17)}$$

where

$\mu(y,z)$ is the mass per unit area of the structure
$\Phi_1(y,z)$ is the mode shape.

The mass per unit area of the structure at the point of the largest amplitude of the mode shape is normally a good approximation to μ_e.

In most cases the modal deflections $\Phi(y,z)$ are constant for each height z and instead of *Expression (F.16)* the logarithmic decrement of aerodynamic damping δ_a, for along-wind vibrations may be estimated by the following expression:

$$\delta_a = \frac{c_f \rho b v_m(z_s)}{2 n_1 m_e} \qquad \text{EN 1991-1-4, (F.18)}$$

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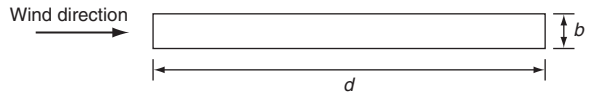


Fig. A2.2. Notation for EN 1991-1-4, Annex E

If special dissipative devices are added to the structure, δ_d should be calculated by suitable theoretical or experimental techniques.

For cable-stayed bridges, it is recommended to factor the values given in this Table by 0.75.

A2.3. Vortex shedding and aeroelastic instabilities

Important Note 1: As for Section A2.2 above, this Section A2.3 is restricted to giving guidance to the clauses on bridges in EN 1991-1-4 Annex E.

Important Note 2: In Annex E of EN 1991-1-4, the notation concerning the width and depth of a bridge deck is different from the notation defined in *Section 8*. In all formulae, the notation is as represented in Fig. A2.2. The depth of the deck is, in general, called the width (or the reference width) because this is the dominant parameter for wind effects.

A2.3.1. Vortex shedding

Vortex shedding occurs when vortices are shed alternately from opposite sides of the structure. This gives rise to a fluctuating load perpendicular to the wind direction. Structural vibrations may occur if the frequency of vortex shedding is the same as a natural frequency of the structure. This condition occurs when the wind velocity is equal to a critical wind velocity as defined below. Typically, the critical wind velocity is a frequent wind velocity indicating that fatigue, and thereby the number of load cycles, may become relevant.

The response induced by vortex shedding is composed of broad-banded response that occurs whether or not the structure is in motion, and narrow-banded response originating from motion-induced wind load.

Note 1: Broad-banded response is normally most important for reinforced concrete structures and heavy steel structures.

E.1.1: EN 1991-1-4 Note 2: Narrow-banded response is normally most important for light steel structures.

A2.3.2. Basic parameters for vortex shedding and other types of instability

Four fundamental parameters are involved in the description of the main aeroelastic phenomena: the Strouhal number, the Scruton number, the critical wind velocity and the Reynolds number.

(1) Strouhal number

The Eurocode gives a value of the Strouhal number for different cross-sections (Table E.1), but for bridge decks the most useful information is given in Fig. A2.3 below.

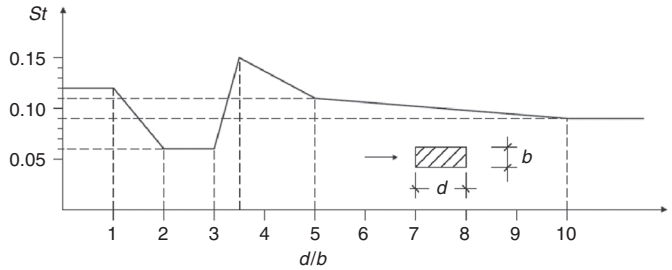


Fig. A2.3. Strouhal number (St) for rectangular cross-sections with sharp corners (EN 1991-1-4, Figure E.1)

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It should be noted that for piers with a circular cross-section, the Strouhal number is 0.18.

(2) Scruton number
The susceptibility of vibrations depends on the structural damping and the ratio of structural mass to fluid mass. This is expressed by the Scruton number Sc, which is given by the following expression:

$$Sc = \frac{2\delta_s m_{i,e}}{\rho b^2} \qquad \text{EN 1991-1-4, (E.4)}$$

where

- δ_s is the structural damping expressed by the logarithmic decrement
- ρ is the air density under vortex-shedding conditions, with a recommended value equal to $1.25\,kg/m^3$
- $m_{i,e}$ is the equivalent mass m_e per unit length for mode i as defined in Section A1.2 of this Designers' Guide
- b is the **reference width of the cross-section** at which resonant vortex shedding occurs.

(3) Critical wind velocity
The critical wind velocity for bending vibration mode i is defined as the wind velocity at which the frequency of vortex shedding equals a natural frequency of the structure or a structural element and is given by the following expression:

$$v_{crit,i} = \frac{b n_{i,y}}{St} \qquad \text{EN 1991-1-4, (E.2)}$$

where

- b is the **reference width of the cross-section** at which resonant vortex shedding occurs and where the modal deflection is maximum for the structure or structural part considered; for circular cylinders the reference width is the outer diameter
- $n_{i,y}$ is the natural frequency of the considered flexural mode i of cross-wind vibration; for approximations of $n_{1,y}$ see Section A1.2 of this Designers' Guide
- St is the Strouhal number.

(4) The Reynolds number
The vortex-shedding action on a circular cylinder depends on the Reynolds number Re at the critical wind velocity $v_{crit,i}$. The Reynolds number is given by the following expression:

$$Re(v_{crit,i}) = \frac{b v_{crit,i}}{\nu} \qquad \text{EN 1991-1-4, (E.5)}$$

where

- b is the **outer diameter of the circular cylinder**
- ν is the kinematic viscosity of the air ($\nu \approx 15 \times 10^{-6}\,m^2/s$)
- $v_{crit,i}$ is the critical wind velocity.

A2.3.3. Criteria for vortex shedding

EN 1991-1-4 recommends to investigate the effect of vortex shedding when the ratio of the largest to the smallest crosswind dimension of the structure, both taken in the plane perpendicular to the wind, exceeds 6. The effect of vortex shedding need not be investigated when

E.1.2(1): EN 1991-1-4

$$v_{crit,i} > 1.25 v_m \qquad \text{EN 1991-1-4, (E.1)}$$

where

- $v_{crit,i}$ is the critical wind velocity for mode i
- v_m is the mean wind velocity at the cross-section where vortex shedding occurs.

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A2.3.4. Vortex shedding action

The effect of vibrations induced by vortex shedding should be calculated from the effect of the inertia force per unit length $F_w(s)$, acting perpendicular to the wind direction at location s on the structure and given in the following expression:

$$F_w(s) = m(s) \times (2\pi n_{i,y})^2 \Phi_{i,y}(s) \times y_{F,max} \qquad \text{EN 1991-1-4, (E.6)}$$

where

$m(s)$ is the vibrating mass of the structure per unit length (kg/m)
$n_{i,y}$ is the natural frequency of the structure
$\Phi_{i,y}(s)$ is the mode shape of the structure normalized to 1 at the point of maximum displacement
$y_{F,max}$ is the maximum displacement over time of the point with $\Phi_{i,y}(s)$ equal to 1.

A2.3.5. Calculation of the crosswind amplitude

E.1.5.2 and E.1.5.3: EN 1991-1-4

Two different approaches for calculating the vortex-excited crosswind amplitudes are defined in EN 1991-1-4. The second approach covers more specifically structures such as chimneys or masts. Therefore, only the first approach is mentioned hereafter for an application to bridges.

The largest displacement $y_{F,max}$ can be calculated using the following expression:

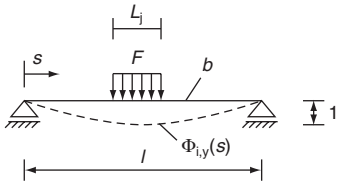
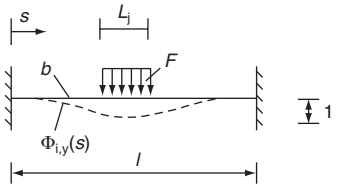
$$\frac{y_{F,max}}{b} = \frac{1}{St^2}\frac{1}{Sc} K K_W c_{lat} \qquad \text{EN 1991-1-4, (E.7)}$$

where

St is the Strouhal number
Sc is the Scruton number
K_W is the effective correlation length factor by which the aeroelastic forces are taken into account
K is the mode shape factor
c_{lat} is the lateral force coefficient.

In the case of bridges, K_W and K may be assessed by the formulae given in Table A2.3 (theoretical expressions may be found in the Eurocode).

Table A2.3. Correlation length factor K_W and mode shape factor K usable for bridges (Data taken from EN 1991-1-4 Table E.5)

Structure	Mode shape, $\Phi_{i,y}(s)$	K_W	K
	see Table A2.1 $n = 1$; $m = 1$	$\cos\left[\frac{\pi}{2}\left(1 - \frac{L_j/b}{\lambda}\right)\right]$	0.10
	see Table A2.1 $n = 1$; $m = 1$	$\frac{L_j/b}{\lambda} + \frac{1}{\pi}\sin\left[\pi\left(1 - \frac{L_j/b}{\lambda}\right)\right]$	0.11

Note 1: The mode shape, $\Phi_{i,y}(s)$, is taken from Table A2.1.
n is the number of regions where vortex excitation occurs at the same time
m is the number of antinodes of the vibrating structure in the considered mode shape $\Phi_{i,y}$
Note 2: $\lambda = \ell/b$

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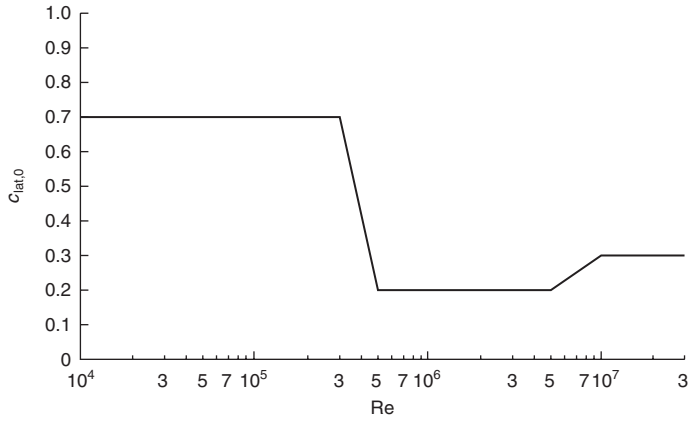


Fig. A2.4. Basic value of the lateral force coefficient $c_{lat,0}$ versus Reynolds number $\mathrm{Re}(v_{crit,i})$ for circular cylinders (EN 1991-1-4 Figure E.2)

Table E.2: EN 1991-1-4

The lateral force coefficient c_{lat} is determined from a basic value, $c_{lat,0}$, – for bridges decks, it may be taken equal to 1.1.

For piers with a circular cross-section, the basic value $c_{lat,0}$ may be determined by using Fig. A2.4.

The lateral force coefficient, c_{lat}, is given in Table A2.4.

In general, for common cases, $c_{lat} = c_{lat,0}$

A2.3.6. Galloping

Galloping is a self-induced vibration of a flexible structure in crosswind bending mode. Non-circular cross-sections are prone to galloping. Ice may cause a stable cross-section to become unstable. Galloping oscillation starts at a special onset wind velocity v_{CG} and normally the amplitudes increase rapidly with increasing wind velocity.

The onset wind velocity of galloping, v_{CG}, is given in the following expression:

$$v_{CG} = \frac{2Sc}{a_G} n_{1,y} b \qquad \text{EN 1991-1-4, (E.18)}$$

where

Sc is the Scruton number
$n_{1,y}$ is the crosswind fundamental frequency of the structure (see Section A1.2 of this Designers' Guide)
b is the width as defined in Table A2.5

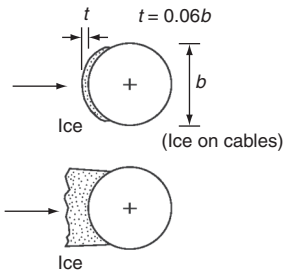
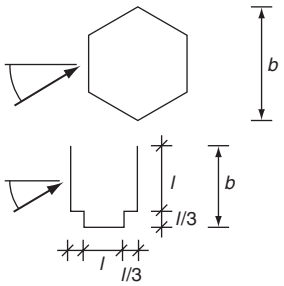
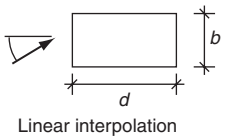
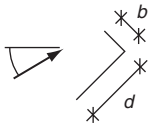
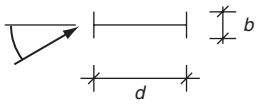
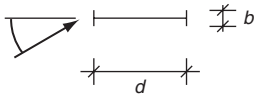
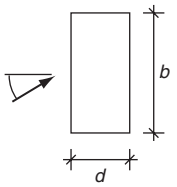
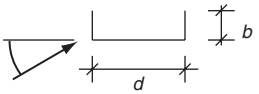
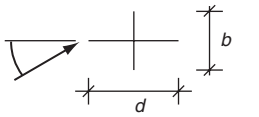
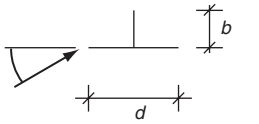
Table A2.4. Lateral force coefficient c_{lat} versus critical wind velocity ratio, $v_{crit,i}/v_{m,Lj}$ (Data taken from EN 1991-1-4, Table E.3)

Critical wind velocity ratio	c_{lat}
$\frac{v_{crit,i}}{v_{m,Lj}} \leq 0.83$	$c_{lat} = c_{lat,0}$
$0.83 \leq \frac{v_{crit,i}}{v_{m,Lj}} < 1.25$	$c_{lat} = \left(3 - 2.4\frac{v_{crit,i}}{v_{m,Lj}}\right) c_{lat,0}$
$1.25 \leq \frac{v_{crit,i}}{v_{m,Lj}}$	$c_{lat} = 0$

where
$v_{crit,i}$ is the critical wind velocity (see expression (E.1))
$v_{m,Lj}$ is the mean wind velocity in the centre of the effective correlation length

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Table A2.5. Factor of galloping instability a_G (Data taken from EN 1991-1-4, Table E.7; see EN 1991-1-4 for missing values)

Cross-section		Factor of galloping instability, a_G	Cross-section		Factor of galloping instability, a_G
		1.0			
 Linear interpolation	$d/b = 2$			$d/b = 2$	0.7
	$d/b = 1.5$	1.7		$d/b = 2.7$	
	$d/b = 1$			$d/b = 5$	7
 Linear interpolation	$d/b = 2/3$	1		$d/b = 3$	
	$d/b = 1/2$			$d/b = 3/4$	3.2
	$d/b = 1/3$	0.4		$d/b = 2$	

Note: Extrapolations for the factor a_G as function of d/b are not allowed.

a_G is the factor of galloping instability (Table A2.5); if no factor of galloping instability is known then $a_G = 10$ may be used.

It should be ensured that:

$$v_{CG} > 1.25 v_m \qquad \text{EN 1991-1-4, (E.19)}$$

where v_m is the mean wind velocity at the height at which the galloping process is expected; this is likely to be the point of maximum amplitude of oscillation.

If the critical vortex-shedding velocity v_{crit} is close to the onset wind velocity of galloping v_{CG}

$$0.7 < \frac{v_{CG}}{v_{crit}} < 1.5 \qquad \text{EN 1991-1-4, (E.20)}$$

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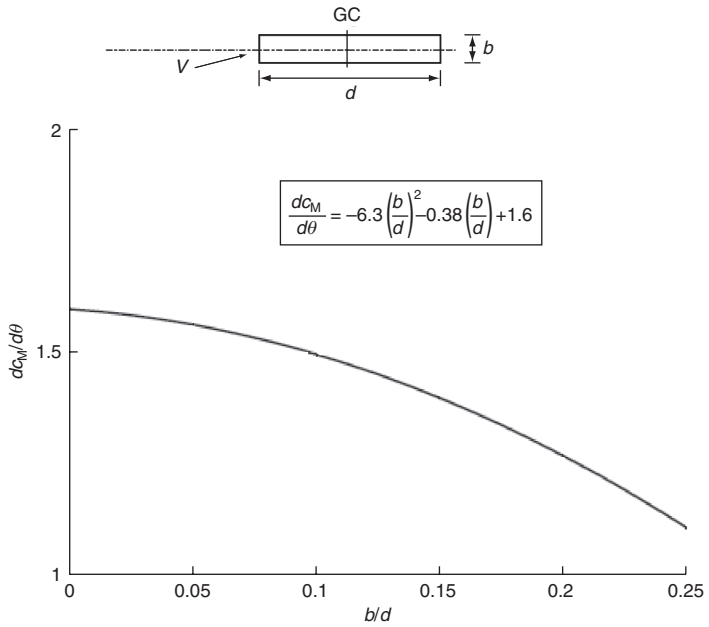


Fig. A2.5. Rate of change of aerodynamic moment coefficient, $dc_M/d\Theta$, with respect to geometric centre GC for a rectangular section (Reproduced from EN 1991-1-4, with permission from BSI)

then interaction effects between vortex shedding and galloping are likely to occur. In this case specialist advice is recommended.

A2.3.7. Divergence and flutter

E.4: EN 1991-1-4

Divergence and flutter are instabilities that occur for flexible plate-like structures, such as signboards or suspension-bridge decks, above a certain threshold or critical wind velocity. The instability is caused by the deflection of the structure modifying the aerodynamics to alter the loading. Divergence and flutter should be avoided.

The procedures given by the EN 1991-1-4 provide a means of assessing the susceptibility of a structure in terms of simple structural criteria. If these criteria are not satisfied, specialist advice is recommended. In fact, the criteria are only developed for plate-like structures, i.e. structures such that:

- have an elongated cross-section (like a flat plate) with b/d (depth/width) less than 0.25
- the torsional axis is parallel to the plane of the plate and normal to the wind direction, and the centre of torsion is at least $d/4$ downwind of the windward edge of the plate, where d is the inwind depth of the plate measured normal to the torsional axis. This includes the common cases of torsional centre at geometrical centre, i.e. centrally supported signboard or canopy, and torsional centre at downwind edge, i.e. cantilevered canopy
- the lowest natural frequency corresponds to a torsional mode, or else the lowest torsional natural frequency is less than 2 times the lowest translational natural frequency.

For this type of structure, the critical wind velocity for divergence is given in the following expression:

$$v_{\text{div}} = \left(\frac{2k_\Theta}{\rho d^2 \dfrac{dc_M}{d\Theta}} \right)^{1/2} \qquad \text{EN 1991-1-4, (E.24)}$$

where

k_Θ is the torsional stiffness

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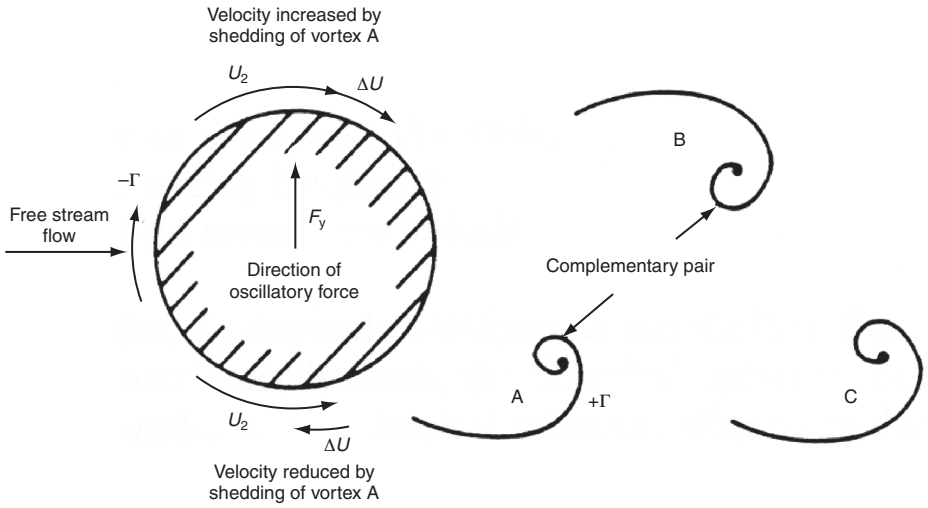


Fig. A2.6. Principle of vortex shedding at a circular cylinder

c_M is the aerodynamic moment coefficient, given in the following expression:

$$c_M = \frac{M}{\frac{1}{2}\rho v^2 d^2} \qquad \text{EN 1991-1-4, (E.25)}$$

$\frac{dc_M}{d\Theta}$ is the rate of change of aerodynamic moment coefficient with respect to rotation about the torsional centre, where Θ is expressed in radians

M is the aerodynamic moment of a unit length of the structure

ρ is the density of air

d is the inwind depth (chord) of the structure (see Fig. A2.5)

b is the width.

Values of $dc_M/d\Theta$ measured about the geometric centre of various rectangular sections are given in Fig. A2.5.

The stability criteria are:

$$v_{div} > 2v_m(z_s) \qquad \text{EN 1991-1-4, (E.26)}$$

where $v_m(z_s)$ is the mean wind velocity at height z_s.

A2.4. Aerodynamic excitation of cables

Very limited guidance is given in EN 1991-1-4 concerning aerodynamic excitation of cables, in particular cable stays. When exposed to periodic excitation, cable stays can, under certain conditions, accumulate energy and oscillate with substantial amplitudes. This vibration rarely endangers the structural integrity of the structure, but it is disturbing for users and may cause fatigue damage to the cable stays if not controlled.

Cable vibration has two origins:

- displacement of anchorages, under the effect of traffic or wind loading on the bridge deck, called 'parametric excitation'
- various effects of wind acting directly on the cables, called wind-induced vibrations.

Two types of vibration mechanisms may be distinguished:

- resonance of the stay to external excitation, resulting in rather small amplitudes – up to two cable diameters
- aeroelastic instability, characterized by very high amplitudes – up to several metres.

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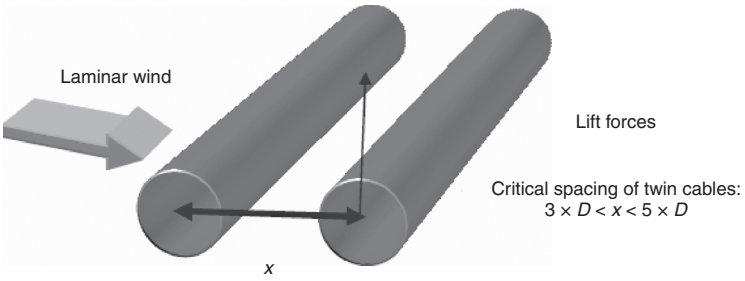


Fig. A2.7. Wake galloping excitation mechanism

The main wind-induced vibrations are:

- rain-and-wind-induced vibrations
- vortex shedding
- cable galloping
- parametric excitation.

Rain-and-wind-induced vibrations are relatively large vibrations of bridge cables during moderate winds combined with rain; it is an instability phenomenon. The interaction of moderate wind with moderate to heavy rain tends to form two water rivulets running down the cable at the top and bottom of the cable section. The top rivulet is in unstable equilibrium while running down the cable, and will therefore form a sinusoidal path. This oscillatory movement periodically affects the drag coefficient of the cable along the cable length and thus transfers energy from the wind flow to the cable. A simple countermeasure consists in appropriate surface treatments such as double helical ribs or longitudinal grooves.

Vortex shedding is a classical phenomenon that not only applies to stay cables but as a general rule to all circular cylindrical structures that are an obstacle in a fluid flow. The wake of the obstacle consists of the Von Karman vortex street (Fig. A2.6).

The vortices are shed alternately on one side and then on the other of the obstacle. Once the vortices have grown to a certain size, they detach from the cylinder and apply a periodic force transversal to the direction of the flow. Most of the stay cables have eigenfrequencies below 2 Hz for the first modes. The critical wind speeds for stay-cable vibration disorders due to vortex shedding are very low, and such winds are unable to transfer a considerable amount of energy to the stay. Consequently, vortex shedding is not a governing problem for stay-cable vibration.

Cable galloping is a form of aeroelastic instability that can occur on certain poorly profiled elastic structures in laminar flow. Three different forms of galloping have been observed on various bridges: ice galloping (the aerodynamic cross-section of the stay becomes similar to the wing of an airplane, due to ice – see Table A2.5 of this Designers' Guide); wake galloping (a cable is excited by the wake of a Von Karman vortex street caused by an obstacle further upstream, e.g. another cable or a tower – see Fig. A2.7); buffeting (dynamic action of the turbulent wind), parametric excitation.

Parametric excitation may appear under the action of wind on the deck or pylons, or by the action of traffic: the whole bridge structure may vibrate to a greater or lesser degree. Cable-stay vibration may also be caused by the periodic displacement of the anchorages, induced by the vibrations of the bridge structure. In-plane resonance occurs when global in-plane bridge modes excite the cables at $\frac{1}{2}$, 1 or 2 times one of their eigenfrequencies. This phenomenon is called '$\frac{1}{2}$, 1, 2 resonance'. The global in-plane movement of either the bridge deck or the bridge towers generates a longitudinal displacement of the stay anchorages, which induces additional strain into the cable.

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Annex B to Chapter 2: Example calculations for wind actions on bridges

In all the examples, all references to expressions and figures are to EN 1991-1-4.

B2.1. Example 1: Slab bridge (road bridge)

Height of the bridge: 6 m above ground.

Terrain category II: $v_{b0} = 24\,\text{m/s}$ (from a National map)

Table 4.1 $z_0 = 0.04\,\text{m} \qquad z_{min} = 2\,\text{m}$

Orography factor:

$c_o = 1$

Assumptions:

$c_{dir} = 1 \qquad c_{season} = 1 \Rightarrow v_b = v_{b,0} = 24\,\text{m/s}$

Terrain factor:

$$k_r = 0.19\left(\frac{z_0}{z_{0,II}}\right)^{0.07} = 0.19 \tag{4.5}$$

$$c_r(z) = k_r \ln\left(\frac{z}{z_0}\right) \Rightarrow c_r(6) = 0.19 \ln\left(\frac{6}{0.04}\right) = 0.952 \tag{4.4}$$

$$v_m(z) = c_r(z)c_0(z)v_b \Rightarrow v_m(6) = 0.952 \times 24 = 22.85\,\text{m/s} \tag{4.3}$$

Basic velocity pressure:

$$q_b(z) = \frac{1}{2}\rho v_m^2(z) \Rightarrow q_b(6) = \frac{1}{2} \times 1.25 \times 22.85^2 = 326.3\,\text{N/m}^2$$

Determination of $c_e(6m)$ (*Fig. 4.2*)

$c_e(6m) = 2.0$ – see Fig. B2.2.

Peak velocity pressure:

$$q_p(z) = c_e(z)q_b(z) \Rightarrow q_p(6) = 2 \times 326.3 = 653\,\text{Pa} = 0.653\,\text{kN/m}^2 \tag{4.9}$$

(a) In the absence of traffic on the bridge deck, the total depth is $1.00 + 0.60 = 1.60\,\text{m}$

$b/d_{tot} = 10/1.6 = 6.25$

$$F_{Wk,x} = c_s c_d \times c_f \times q_p(z_e) \times A_{ref,x} \tag{5.3}$$

$c_s c_d = 1$ (8.2(1) Note 2)

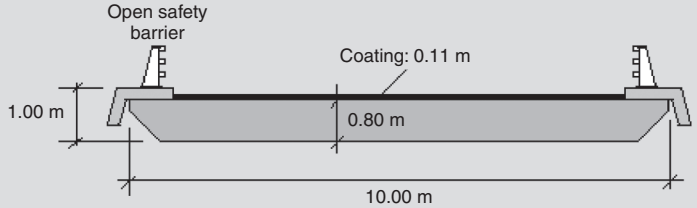


Fig. B2.1. Cross-section of the bridge deck

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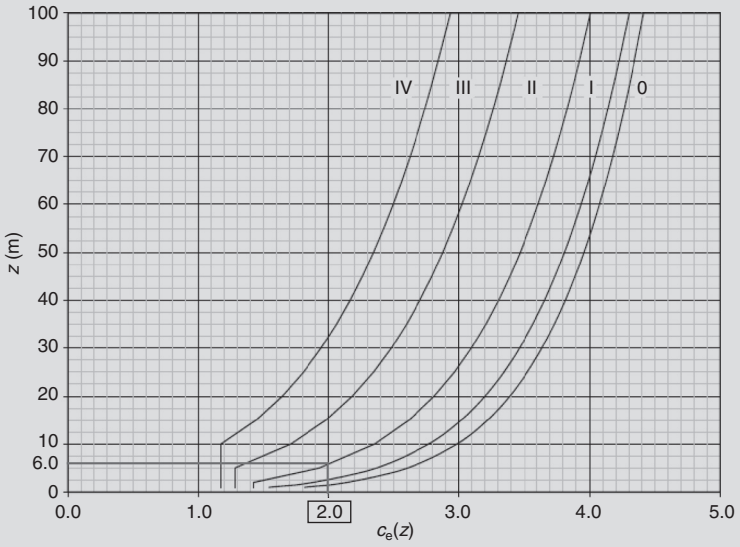


Fig. B2.2. Determination of the exposure factor at 6 m

$c_f = c_{fx,0} = 1.3$ (see Fig. B2.3)

$F_{Wk,x} = 1 \times 1.3 \times 0.653 \times 1.6 = 1.358\,\text{kN/m}$

(b) With road traffic on the bridge deck, the total depth is $0.80 + 0.11 + 2.00 = 2.91\,\text{m}$

$b/d_{tot} = 10/2.91 = 3.44$

$F_{Wk,x} = c_s c_d \times c_f \times q_p(z_e) \times A_{ref,x}$

$c_s c_d = 1$

$c_f = c_{fx,0} = 1.45$ (see Fig. B2.4)

$F_{Wk,x} = 1 \times 1.45 \times 0.653 \times 3 = 2.84\,\text{kN/m}$

This characteristic value is multiplied by the combination factor ψ_0 because the wind action is an accompanying action when road traffic loads are applied to the bridge deck. With the

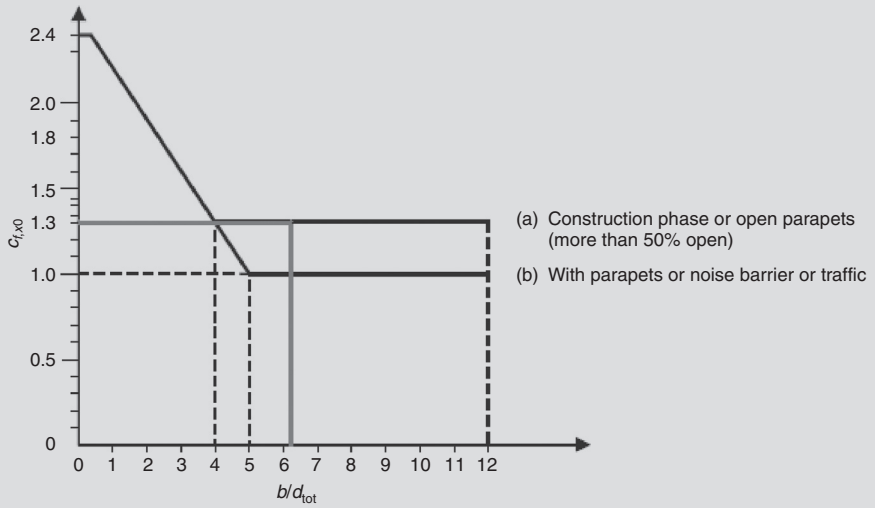


Fig. B2.3. Determination of the force coefficient without traffic

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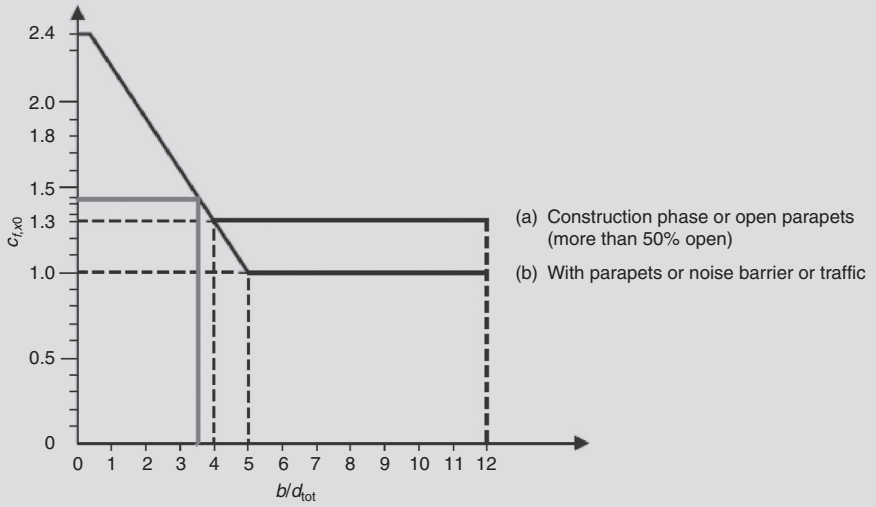


Fig. B2.4. Determination of the force coefficient with traffic

recommended value $\psi_0 = 0.6$ (see Chapter 8 of this Designers' Guide), the representative value of wind action is $\psi_0 F_{\mathrm{Wk},x} = 0.6 \times 2.84 = 1.70\,\mathrm{kN/m}$. This value is higher than the value of the wind force in the absence of road traffic loads.

B2.2. Example 2: Prestressed concrete bridge (road bridge)

The geometrical data of the bridge under consideration are given in Fig. B2.5.

At midspan, the reference height of the deck above the water level is $z = 15\,\mathrm{m}$

Assumptions:
Terrain category 0 (coastal area):

$v_{\mathrm{b0}} = 26\,\mathrm{m/s}$ (from a National map)

$z_0 = 0.003\,\mathrm{m}$ $\quad z_{\min} = 1\,\mathrm{m}$ (Table 4.1)

Orography factor:

$c_{\mathrm{o}} = 1$ (flat zone)

$c_{\mathrm{dir}} = 1$ $\quad c_{\mathrm{season}} = 1 \Rightarrow v_{\mathrm{b}} = v_{\mathrm{b,0}} = 26\,\mathrm{m/s}$

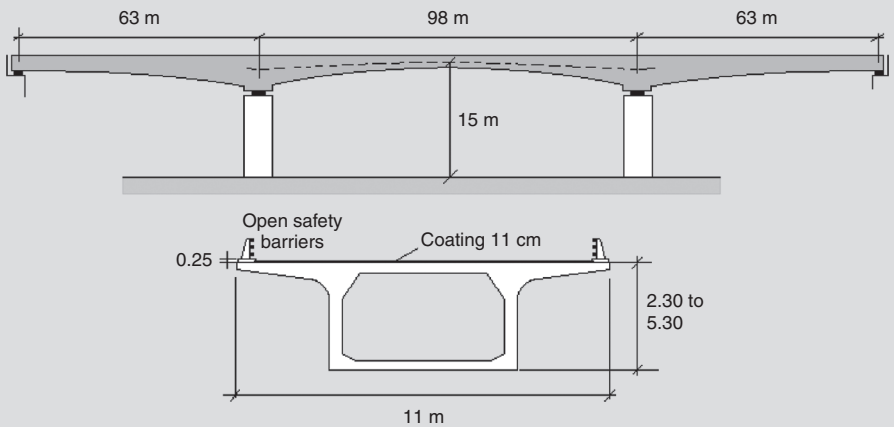


Fig. B2.5. Description of the bridge

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Terrain factor:

$$k_r = 0.19\left(\frac{z_0}{z_{0,II}}\right)^{0.07} = 0.19\left(\frac{0.003}{0.05}\right)^{0.07} = 0.156 \quad (4.5)$$

$$c_r(z) = k_r \ln\left(\frac{z}{z_0}\right) \Rightarrow c_r(15) = 0.156 \ln\left(\frac{15}{0.003}\right) = 1.329 \quad (4.4)$$

$$v_m(z) = c_r(z)c_0(z)v_b \Rightarrow v_m(15) = 1.329 \times 26 = 34.55\,\text{m/s} \quad (4.3)$$

$$q_b(z) = \frac{1}{2}\rho v_m^2(z) \Rightarrow q_b(15) = \frac{1}{2} \times 1.25 \times 34.55^2 = 746.06\,\text{N/m}^2$$

Determination of the peak velocity pressure from the formulae given in EN 1991-1-4:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_o(z)\ln(z/z_0)} = \frac{1.0}{1.0 \times \ln(15/0.003)} = 0.117 \quad (4.7)$$

where k_I is the turbulence factor, taken with the recommended value which is 1.0.

$$q_p(z) = q_b(z)[1 + 7I_v(z)] \Rightarrow q_p(15) = 746.06 \times (1 + 7 \times 0.117)$$
$$= 1357\,\text{N/m}^2 = 1.357\,\text{kN/m}^2$$

Calculation of the wind force in the x-direction

(a) In the absence of traffic on the bridge deck

At midspan, the total depth is $2.30 + 0.25 + 0.60 = 3.15$ m (see Fig. B2.5)

At piers, the total depth is $5.30 + 0.25 + 0.60 = 6.15$ m

$b/d_{tot} = 11/3.15 = 3.50$ in the first case; and $= 11/6.15 = 1.79$ in the second case.

$$F_{Wk,x} = c_s c_d \times c_f \times q_p(z_e) \times A_{ref,x} \quad (5.3)$$

$c_s c_d = 1$ (this assumption is conservative)

$c_f = c_{fx,0} \cong 1.5$ or 2

See Fig. B2.6.

At midspan:

$$F_{Wk,x} = 1 \times 1.5 \times 1.357 \times 3.15 = 6.412\,\text{kN/m}$$

At piers:

$$F_{Wk,x} = 1 \times 2.0 \times 1.357 \times 6.15 = 16.69\,\text{kN/m}$$

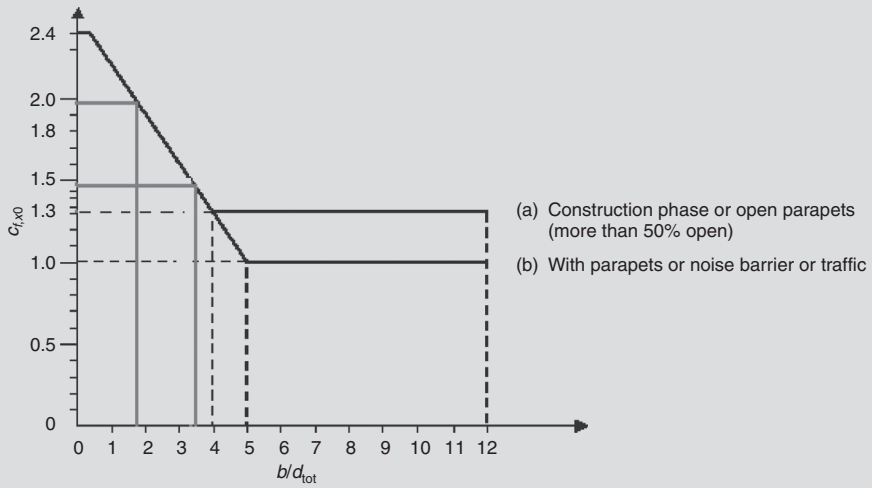


Fig. B2.6. Determination of the force coefficient at midspan and at pier without traffic

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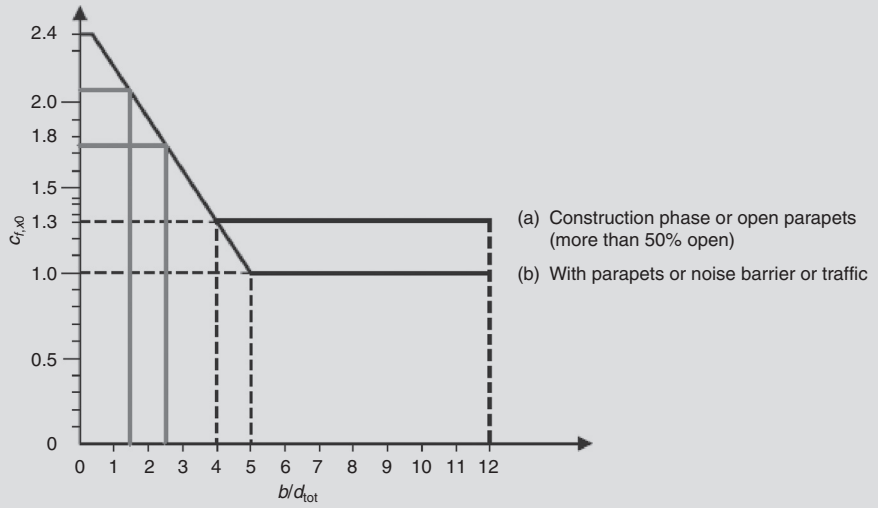


Fig. B2.7. Determination of the force coefficient at midspan and at pier with traffic

(b) With road traffic on the bridge deck, the total depth is:

At midspan:

$$d_{tot} = 2.30 + 0.11 + 2.00 = 4.41 \text{ (metres)} \Rightarrow b/d_{tot} = 11/4.41 = 2.49$$

At piers:

$$d_{tot} = 5.30 + 0.11 + 2.00 = 7.41 \Rightarrow b/d_{tot} = 11/7.41 = 1.48$$

$$F_{Wk,x} = c_s c_d \times c_f \times q_p(z_e) \times A_{ref,x}$$

$$c_s c_d = 1 \text{ (Fig. B2.7)}$$

$$c_f = c_{fx,0} = 1.77 \text{ or } 2.1 \text{ at midspan or at piers.}$$

At midspan:

$$F_{Wk,x} = 1 \times 1.77 \times 1.357 \times 4.41 = 10.6\,\text{kN/m}$$

At piers:

$$F_{Wk,x} = 1 \times 2.1 \times 1.357 \times 7.41 = 21.12\,\text{kN/m}$$

As for the example in B2.1, these characteristic values are multiplied by the combination factor ψ_0 because the wind action is an accompanying action when road traffic loads are applied to the bridge deck. With the recommended value $\psi_0 = 0.6$ (see Chapter 8 of this Designers' Guide), the representative value of wind action is:

At midspan:

$$\psi_0 F_{Wk,x} = 0.6 \times 10.6 = 6.36\,\text{kN/m}$$

At piers:

$$\psi_0 F_{Wk,x} = 0.6 \times 21.12 = 12.67\,\text{kN/m}$$

B2.3. Example 3: Bridge with high piers

Consider a multi-span bridge deck with span lengths of 120 m, for example a composite steel–concrete bridge. The terrain category is II, the orography factor is $c_0 = 1$ ($k_r = 0.19$, $z_0 = 0.05$ m), the basic wind velocity is $v_b = 24$ m/s. The highest piers are 140 m. For such a structure, several problems need to be investigated:

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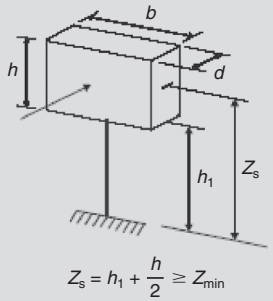


Fig. B2.8. Representation of a pointlike structure

- the verification of stability during execution (see Chapter 3 of this Designers' Guide)
- the determination of wind actions during persistent design situations, the assessment of the factor $c_s c_d$ being difficult
- possibly the aerodynamic behaviour of the whole structure (superstructure and infrastructure).

The wind force is calculated from the formula:

$$F_W = c_s c_d \times c_f \times q_p(z_e) \times A_{ref} \tag{5.3}$$

where $c_s c_d$ is the structural factor. Where $h_{pier} > 60$–70 m, it is appropriate to calculate the structural factor in accordance with EN 1991-1-4 Annex B (procedure 1)

(a) Structural factor:

$$B^2 = \frac{1}{1 + 0.9\left(\frac{b+h}{L(z_s)}\right)^{0.63}} \tag{B.3}$$

where

b, h is the width and height of the structure respectively
$L(z_s)$ is the turbulent length scale given in B.1(1) at reference height z_s defined in Figure 6.1 of EN 1991-1-4 (represented below as Fig. B2.8). It is on the safe side to use $B^2 = 1$.

Hence:

$h_1 = 120\,\text{m}, h = 4\,\text{m}, z_s = 140 + 2 = 142\,\text{m}$

For the application, we adopt $b = 120$ m, which represents a span length. $L(z_e)$, turbulent length scale:

For $z_s = 142$ m:

$$L(z_s) = 300\left(\frac{z_s}{200}\right)^{0.67 + 0.05\ln(z_0)} = 300\left(\frac{142}{200}\right)^{0.52} = 251\,\text{m} \tag{B.1}$$

Hence:

$$B^2 = \frac{1}{1 + 0.9\left(\frac{b+h}{L(z_s)}\right)^{0.63}} = \frac{1}{1 + 0.9\left(\frac{124}{251}\right)^{0.63}} = 0.63$$

and

$$I_v(z_s) = \frac{1}{c_0(z_s)\ln(z_s/z_0)} = \frac{1}{\ln(142/0.05)} = 0.126 \tag{4.7}$$

$$c_s = \frac{1 + 7I_v(z_s)\sqrt{B^2}}{1 + 7I_v(z_s)} = \frac{1 + 7 \times 0.126 \times 0.794}{1 + 7 \times 0.126} = 0.90 \tag{6.2}$$

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This shows a reduction effect on the wind action due to the non-simultaneity of occurrence of the peak wind pressures on the surfaces of about 10%.

(b) Dynamic factor

$$c_d = \frac{1 + 2k_p I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7I_v(z_s)\sqrt{B^2}} \tag{6.3}$$

where

z_s is the reference height for determining the structural factor, see Fig. B2.8
k_p is the peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation
I_v is the turbulence intensity previously calculated
B^2 is the background factor, allowing for the lack of full correlation of the pressure on the structure surface, previously calculated.
R^2 is the resonance response factor, allowing for turbulence in resonance with the vibration mode

$$k_p = \sqrt{2\ln(\nu T)} + \frac{0.6}{\sqrt{2\ln(\nu T)}} \tag{B.4}$$

ν is the up-crossing frequency given in the expression

$$\nu = n_{1,x}\sqrt{\frac{R^2}{B^2 + R^2}} \quad \nu \geq 0.08\,\text{Hz} \tag{B.5}$$

where

$n_{1,x}$ is the natural frequency of the structure; the limit of $\nu \geq 0.08$ Hz corresponds to a peak factor of 3.0
T is the averaging time for the mean wind velocity, $T = 600$ s.

The resonance response factor R^2 allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using the following expression:

$$R^2 = \frac{\pi^2}{2\delta} S_L(z_s, n_{1,x}) R_h(\eta_h) R_b(\eta_b) \tag{B.6}$$

where

δ is the total logarithmic decrement of damping
S_L is the non-dimensional power spectral density function
R_h, R_b are the aerodynamic admittance functions.

All these quantities are calculated by the following process:

$$v_m(z_s) = k_r \ln\left(\frac{z_s}{z_0}\right) v_b$$

$$\delta = \delta_s + \delta_a + \delta_d \qquad \text{EN 1991-1-4, (F.15)}$$

$\delta_s = 0.04$ for composite bridges (Table A2.1)

$$\delta_a = \frac{c_f \rho v_m(z_s)}{2 n_1 \mu_e} \qquad \text{EN 1991-1-4, (F.16)}$$

$\delta_d = 0$ for the bridge under consideration.

$$f_L(z_s, n_{1,x}) = \frac{n_{1,x} L(z_s)}{v_m(z_s)}$$

$$S_L(z, n) = \frac{n S_v(z, n)}{\sigma_v^2} = \frac{6.8 f_L(z, n)}{[1 + 10.2 f_L(z, n)]^{5/3}}$$

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with $z = z_s \quad n = n_{1,x}$ (B.2)

$$\eta_h = \frac{4.6h}{L(z_s)} f_L(z_s, n_{1,x}) \quad \eta_b = \frac{4.6b}{L(z_s)} f_L(z_s, n_{1,x})$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}), \quad R_h = 1 \text{ for } \eta_h = 0 \tag{B.7}$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b}), \quad R_b = 1 \text{ for } \eta_b = 0 \tag{B.8}$$

In our case, $n_{1,x} = 0.30\,\text{Hz}$ has been directly calculated.

$$v_m(z_s) = k_r \ln\left(\frac{z_s}{z_0}\right) v_b = 0.19 \ln\left(\frac{142}{0.05}\right) \times 24 = 36.26\,\text{m/s}$$

$$\delta_a = \frac{c_f \rho v_m(z_s)}{2 n_1 \mu_e} = \frac{1.3 \times 1.25 \times 36.26}{2 \times 0.3 \times 900} = 0.11$$

(the equivalent mass per unit area of the structure is taken equal to $900\,\text{kg/m}^2$)

$$\delta = 0.11 + 0.04 = 0.15$$

$$f_L(z_s, n_{1,x}) = \frac{n_{1,x} L(z_s)}{v_m(z_s)} = \frac{0.30 \times 251}{36.26} = 2.08$$

$$S_L(z, n) = \frac{n S_v(z, n)}{\sigma_v^2} = \frac{6.8 f_L(z, n)}{(1 + 10.2 f_L(z, n))^{5/3}} = \frac{6.8 \times 2.08}{(1 + 10.2 \times 2.08)^{5/3}} = 0.0806$$

$$\eta_h = \frac{4.6h}{L(z_s)} f_L(z_s, n_{1,x}) = \frac{4.6 \times 4}{251} \times 2.08 = 0.152$$

$$\eta_b = \frac{4.6b}{L(z_s)} f_L(z_s, n_{1,x}) = \frac{4.6 \times 120}{251} \times 2.08 = 4.574$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) = 0.906$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b}) = 0.195$$

$$R^2 = \frac{\pi^2}{2\delta} S_L(z_s, n_{1,x}) R_h(\eta_h) R_b(\eta_b) = \frac{\pi^2}{2 \times 0.15} \times 0.0806 \times 0.906 \times 0.195 = 0.47$$

$$\nu = n_{1,x} \sqrt{\frac{R^2}{B^2 + R^2}} = 0.30 \sqrt{\frac{0.47}{0.47 + 0.63}} = 0.196 \geq 0.08\,\text{Hz}$$

$$k_p = \sqrt{2 \ln(\nu T)} + \frac{0.6}{\sqrt{2 \ln(\nu T)}} = \sqrt{2 \ln(600 \times 0.196)} + \frac{0.6}{\sqrt{2 \ln(600 \times 0.196)}} = 3.28$$

And finally:

$$c_d = \frac{1 + 2 k_p I_v(z_s) \sqrt{B^2 + R^2}}{1 + 7 I_v(z_s) \sqrt{B^2}} = \frac{1 + 2 \times 3.28 \times 0.126 \sqrt{0.63 + 0.47}}{1 + 7 \times 0.126 \sqrt{0.63}} = 1.098$$

$$c_s c_d = 0.90 \times 1.098 = 0.98$$

This example shows that the coefficient $c_s c_d$ is, in most cases, very close to 1.

B2.4. Example 4: Bow string bridge

This example was primarily developed by Professor Pierre Spehl, chief engineer at SECO and member of the project team for EN 1991-1-4. The bridge is a road bridge and its type is

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a bow-string with two steel arches. The terrain category is II: $z_0 = 0.05\,\text{m}$, $z_{\min} = 2\,\text{m}$ (*Table 4.1*).

$v_b = 26.2\,\text{m/s}$ (from a National Annex)

Span length: $L = 135\,\text{m}$

The deck is a composite steel and concrete structure composed of two steel beams of I-shaped cross-section and a concrete slab. The deck dimensions are: width $d = 10\,\text{m}$; depth $b = 1.8\,\text{m}$ (**notation of Annex E**).

The reference deck height over the reference water level is $z_e = 10\,\text{m}$.

The mass per metre is $m = 8200\,\text{kg/m}$

The mass moment of inertia per metre is $I_p = 105\,000\,\text{kgm}^2/\text{m}$

The calculated natural frequencies are:

- Mode 1 (bending, 2nd mode): 0.498 Hz
- Mode 2 (torsion, 1st mode): 0.675 Hz
- Mode 3 (bending, 3rd mode): 0.937 Hz
- Mode 4 (torsion, 3rd mode): 1.034 Hz
- Mode 5 (torsion, 2nd mode): 1.263 Hz

Criteria for vortex shedding:

$$\frac{d}{b} = \frac{10}{1.8} = 5.55 \Rightarrow St \cong 0.11$$

$$v_{\text{crit},1} = b\frac{n_{1,z}}{St} \tag{E.2}$$

For mode 1:

$$\frac{1.8 \times 0.498}{0.11} = 8.15\,\text{m/s}$$

For mode 5:

$$\frac{1.8 \times 1.293}{0.11} = 21.2\,\text{m/s}$$

$$k_r = 0.19\left(\frac{z_0}{0.05}\right)^{0.07} = 0.19 \tag{4.5}$$

$$c_r = k_r \ln\left(\frac{z_e}{z_0}\right) = 0.19\ln\left(\frac{10}{0.05}\right) = 1 \tag{4.4}$$

$$v_m(z_e) = c_r v_b = 26.2\,\text{m/s} \tag{4.3}$$

$$1.25 v_m = 32.75\,\text{m/s} \tag{E.1}$$

The vortex-shedding effects need to be examined for every mode corresponding to a natural frequency less than:

$$\frac{32.75 \times 0.11}{1.8} = 2\,\text{Hz} \tag{E.1}$$

Maximum vertical deflection:

$$z_{F,\max} = \frac{bKK_W c_{\text{lat}}}{St^2 Sc} \tag{E.7}$$

Scruton number:

$$Sc = \frac{2\delta_s m_{i,e}}{\rho b^2} \tag{E.4}$$

$$\delta_s = 0.03 \quad m_{i,e} = 8200\,\text{kg/m} \Rightarrow Sc = \frac{2 \times 0.03 \times 8200}{1.25 \times 1.8^2} = 121.5 \tag{Table F.2}$$

$$c_{\text{lat}} = 1.1 \tag{Table E.2}$$

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$$K = 0.10 \quad \text{(Table E.5)}$$

$$K_W = \cos\left[\frac{\pi}{2}\left(1 - \frac{6}{(135/1.8)}\right)\right] = 0.125 \quad \text{(Table E.5)}$$

Vertical deflection:

$$z_{F,max} = \frac{1.8 \times 0.1 \times 0.125 \times 1.1}{0.11^2 \times 121.5} = 0.0168 \text{ metres}$$

Verification of the correlation length:

$$\frac{z_{F,max}}{b} = \frac{0.0168}{1.8} = 0.009 < 0.10 \quad \text{(Table E.4)}$$

The criteria are met.
Vertical acceleration:

$$j_z = (2\pi n_{1,z})^2 z_{F,max} = (2\pi \times 0.675)^2 \times 0.0168 = 0.302 \text{ m/s}^2$$

This acceleration is not significant for pedestrian comfort.

Aeroelastic instability

Factor of galloping instability

$$d/b \approx 5, a_G = 7 \quad \text{(Table E.7)}$$

$$v_{CG} = \frac{2Sc}{a_G} n_{1,z} b = \frac{2 \times 121.5}{7} 0.498 \times 1.8 = 31.11 \text{ m/s} < 32.75 \text{ m/s} \quad \text{(E.18)}$$

There is a risk of galloping instability:

$$\left(\text{limit: } a_G < \frac{7 \times 32.75}{31.11} = 7.37\right)$$

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CHAPTER 3

Actions during execution

3.1. General

The material in this chapter is mainly covered in Part 1-6 of EN 1991 *General Actions – Actions during execution*[1] which provides principles and general rules for the determination of actions to be considered for the verification of buildings and civil engineering works during their execution, and also auxiliary construction works which, in accordance with the definition given in the Eurocode, are '*works associated with the construction processes that are not required after use when the related execution activities are completed and they can be removed. Such works could include, for example, falsework, scaffolding, propping (systems), cofferdam, bracing, launching nose*'.

cl. 1.5.2.1: EN 1991-1-6

The following actions that will occur during the execution process are in the scope of EN 1991-1-6 which describes to varying levels of detail:

- actions on structural and non-structural members during handling
- geotechnical actions
- actions due to prestressing effects
- pre-deformations
- temperature, shrinkage, hydration effects
- wind actions
- snow loads
- actions caused by water
- actions due to atmospheric icing
- construction loads
- accidental actions
- seismic actions.

Two categories of actions need to be distinguished:

- actions caused by water, which are completely defined in this part of Eurocode 1, and construction loads (note however that actions caused by water are not specific to construction phases; the rules may also be used for permanent design situations)
- actions other than construction loads and actions caused by water, which are already defined in other parts of Eurocode 1 (self-weight, temperature, wind, accidental actions, snow loads), other Eurocodes (soil movement, earth pressure, prestressing, concrete shrinkage/hydration effects, seismic actions) or other international standards (atmospheric ice loads).

Combinations of actions need to be established in accordance with EN 1990/Annex A2[2] (see Chapter 8 of this Designers' Guide), and the design of the structures follows the rules given in the relevant design Eurocodes.

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Table 3.1. Classification of actions (other than construction loads) during execution stages (Data taken from EN 1991-1-6, Table 2.1)

Action	Classification				Remarks	Source
	Variation in time	Classification/ origin	Spatial variation	Nature (static/dynamic)		
Self-weight	Permanent	Direct	Fixed with tolerance/ free	Static	Free during transportation/ storage. Dynamic if dropped	EN 1991-1-1
Soil movement	Permanent	Indirect	Free	Static		EN 1997
Earth pressure	Permanent/variable	Direct	Free	Static		EN 1997
Prestressing	Permanent/variable	Direct	Fixed	Static	Variable for local design (anchorage)	EN 1990, EN 1992 to EN 1999
Pre-deformations	Permanent/variable	Indirect	Free	Static		EN 1990
Temperature	Variable	Indirect	Free	Static		EN 1991-1-5
Shrinkage/ hydration effects	Permanent/variable	Indirect	Free	Static		EN 1992, EN 1993, EN 1994
Wind actions	Variable/accidental	Direct	Fixed/free	Static/dynamic	(*)	EN 1991-1-4
Snow loads	Variable/accidental	Direct	Fixed/free	Static/dynamic	(*)	EN 1991-1-3
Actions due to water	Permanent/variable/ accidental	Direct	Fixed/free	Static/dynamic	Permanent/variable according to project specifications. Dynamic for water currents if relevant	EN 1990
Atmospheric ice loads	Variable	Direct	Free	Static/dynamic	(*)	ISO 12494
Accidental	Accidental	Direct/ indirect	Free	Static/dynamic	(*)	EN 1990, EN 1991-1-7
Seismic	Variable/accidental	Direct	Free	Dynamic	(*)	EN 1990 (4.1), EN 1998

(*)The source documents need to be examined with the National Annexes in which additional relevant information may be provided.

3.2. Classifications of actions

Actions other than construction loads may be classified as permanent or variable, direct or indirect, fixed or free, static or dynamic in accordance with the rules defined in EN 1990. A breakdown is given in Table 3.1 which reproduces Table 2.1 of EN 1991-1-6.

cl. 2.2.1: EN 1991-1-6

Construction loads are represented by a unique symbol Q_c and are classified as direct variable actions. Depending on their nature, they are generally free, but may be fixed in some circumstances; they may have a static or a dynamic character. Table 3.2 gives a general overview of the classification of construction loads.

3.3. Design situations and limit states

The execution of a bridge is a transient situation, or a suite of transient situations if the bridge is built in steps. However, accidental actions or accidental situations may occur, for example the loss of static equilibrium due to the fall of a member, failure of a stabilizing

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Table 3.2. Classification of construction loads (Data taken from EN 1991-1-6 Table 2.2; for missing values, see EN 1991-1-6)

Action (short description)	Classification				Remarks	Source
	Variation in time	Classification/ origin	Spatial variation	Nature (static/dynamic)		
Personnel and hand tools		Direct	Free	Static	–	–
Storage movable items	Variable		Free	Static/dynamic	Dynamic in case of dropped loads	EN 1991-1-1
Non-permanent equipment		Direct	Fixed/free	Static/dynamic	–	EN 1991-3
Movable heavy machinery and equipment	Variable		Free	Static/dynamic	–	EN 1991-2, EN 1991-3
Accumulation of waste materials		Direct	Free	Static/dynamic	Can impose loads on, for example, vertical surfaces also	EN 1991-1-1
Loads from parts of structure in temporary states	Variable		Free	Static	Dynamic effects are excluded	EN 1991-1-1

device, earthquake, storm conditions, etc. Therefore, the appropriate transient, accidental and, where relevant, seismic design situations need to be selected, defined and taken into account for the design of the bridge.

cl. 3.1(1)P: EN 1991-1-6

3.3.1. Background concerning the determination of the characteristic value of variable actions

The major problem concerning the choice of characteristic values of variable actions, especially climatic actions, for transient design situations is the danger of defining these characteristic values on the basis of return periods shorter than those agreed for persistent design situations. In other words, is it acceptable or not, and by how much, to reduce the characteristic values of variable actions during execution and, more generally, during transient design situations?

This question is motivated by the common idea that rather high values of these actions are unlikely to be reached for short periods (which is often the case for design situations during execution), and taking these values into account may in some cases be very expensive.

In this background information, the following notation and definitions are used (they are not used in the Eurocode itself).

$Q_{k,pers}$ characteristic value of a variable action for persistent design situations
$Q_{k,trans}$ characteristic value of a variable action for transient design situations
T_{dwl} design working life of the structure
$T_{Q,pers}$ return period of the characteristic value of a variable action for persistent design situations
$T_{Q,trans}$ return period of the characteristic value of a variable action for transient design situations
$T_{Q,real}$ real (or physical) return period of the characteristic value of a variable action
T_{trans} duration of a transient design situation

To determine the appropriate characteristic values for transient design situations by referring to characteristic values for persistent design situations, the following points are taken into account:

- the foreseeable duration of the various transient design situations

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- the additional information that may be collected concerning the magnitude of the actions, depending on the duration and dates of the transient design situations
- the identified risks, including possibilities of intervention.

Although the design working lives do not intervene directly in the choice of $Q_{k,pers}$, the comparison of the characteristic values is based on a comparison of the respective durations T_{trans} and T_{dwl}. For any high value Q^* of Q the probability of exceeding this value is approximately proportional to the following ratio as far as the random process representing the action can be considered as stationary:

$$\frac{\text{Prob}(Q > Q^*) \text{ during } T_{trans}}{\text{Prob}(Q > Q^*) \text{ during } T_{dwl}} \cong \frac{T_{trans}}{T_{dwl}}$$

For climatic actions the additional information is generally linked to:

- the seasonal aspect, for periods that can be measured on a month scale; when it can be taken into account, 3 months may generally be considered as the nominal value of T_{trans}
- and/or the possibility of obtaining reliable meteorological information, for periods that are measured in merely a few days or hours; when appropriate, 1 day may generally be considered as the nominal value of T_{trans}.

For man-made actions, the additional information may generally be linked to the control of the actions and of their effects; the duration is then not a major parameter for the comparison.

In general, 1 year may be accepted as the nominal value of T_{trans}; at this timescale, the action process may be considered as stationary and the same as for persistent situations.

The basic principles of risk assessment are generally applicable, but data are in most cases very specific; in particular it is often possible to prevent or to reduce the consequences of an initially unexpected event, which may justify accepting a higher probability for such unfavourable events.

Some other differences between transient and persistent design situations may have to be taken into account; for example:

- for a variable action whose maxima follow a Gumbel's law, the coefficient of variation is higher for a shorter period than for T_{dwl} (the standard deviation does not depend on the period, but the mean value is lower); as a consequence the values of the partial factors applicable to variable actions γ_F should be slightly increased
- in terms of resistance, during execution the concrete strength has not yet reached its final value (unfavourable effect), but the deterioration of materials, especially of steel, has not yet occurred (favourable effect).

The numerical determination of characteristic values for a 1-year transient design situation may be based on the consideration of return periods, which is valid for stationary processes.

In line with EN 1990, the characteristic value of climatic actions in persistent design situations is based on an annual probability of exceedance equal to 0.02, which means a return period $T_{Q,pers} = 50$ years.

The probability of a failure during transient situations is not fully independent of the probability of failure during persistent design situations in spite of the involvement of some specific basic variables. However, it has been recognised that in common cases, the mutual dependency has very significant consequences on the reliability level only when the influence of permanent actions G is dominant by comparison with the influence of variable actions Q. Assuming roughly a full independence of failure probability during transient and persistent design situations, it appears that, by reducing for transient situations the return periods proportionally to the duration of the situations (i.e. multiplying them by T_{trans}/T_{dwl}), the same probability of failure is approximately obtained during transient and persistent design situations.

However, if an equal probability of failure is accepted for transient and persistent design situations, it immediately appears that, in spite of the mutual dependency of annual failure probabilities, taking into account a persistent situation consisting of, for example, 50

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transient situations would considerably increase the cumulative failure probability. Conversely, if $Q_{k,trans}$ were taken equal to $Q_{k,pers}$, the number of failures during transient situations would obviously be very low compared to what is accepted for persistent situations.

Thus, the characteristic value for a 1-year transient design situation may be taken equal to the combination value for persistent design situations. The format of the combinations is justified by Turkstra's rule: the effects of $Q_{1k} + \psi_{0.2}Q_{2k}$ and of Q_{1k} acting alone should correspond approximately to the same return period. We have indeed, for two actions, two combinations, and therefore for the joint effect a return period divided by 2, but in practice acting ψ_0 factors are chosen so that all possible influence ratios of Q_1 and Q_2 are taken into account (see Designers' Guide to EN 1990[3]); further, the difference in failure probabilities is not significant for the reliability format.

The choice of ψ_0 factors may be influenced by some liability considerations: for lawyers, a value of an action smaller than its codified characteristic value may be considered as normally foreseeable, the codified values being considered, in a general manner, as a boundary between reprehensible and non-reprehensible liabilities. As a consequence the product $\gamma_F\psi_0$ cannot be less than 1 in ultimate limit state (ULS) verifications. The same rule is assumed for the characteristic values during transient situations.

Numerically, for climatic actions, if as given in EN 1990 *Basis of Structural Design* for buildings, the value $\psi_0 = 0.7$ is accepted, it can be easily calculated that:

- for an action with a coefficient of variation equal to 0.2 of its maximum values in 50 years (which is commonly accepted for wind and snow), and distributed in accordance with a Gumbel's law, the nominal return period of $\psi_{01}Q_{1k}$ is approximately equal to 5 years, i.e. $0.1T_{Q,nom}$
- the product $\gamma_Q\psi_0$ is 1.05 when $\psi_0 = 0.7$, which is conservative and therefore acceptable.

For a 1-year transient design situation, mainly for climatic actions, a 5-year return period (instead of 50 years) is acceptable. For shorter transient situations (e.g. 3 months or 3 days) characteristic values may be reduced further on the basis of additional information from various origins. In some cases any reduced characteristic value may have to be reconsidered for optimization of the reliability level.

3.3.2. The design rules given in EN 1991-1-6

The design rules given in EN 1991-1-6 are simplified rules in order to remain usable by designers, but the numerical values derive from the previous background developments and are normally conservative.

The first step is the analysis of the various construction phases, which need individual consideration. The second step consists of assigning a nominal duration to each selected phase, the nominal duration being higher or equal to the real duration. The Eurocode takes into account four nominal durations: less than 3 days, between 3 days and 3 months, between 3 months and 1 year, and more than 1 year. Table 3.3 gives recommended return periods associated with each of these nominal durations for the determination of characteristic values.

The choice of a nominal duration of 3 days may be retained for a slightly longer execution phase if appropriate organizational measures are taken, for example the launching of a rather light structure such as a steel girder.

Nevertheless, concerning wind actions, a minimum wind velocity is recommended for durations up to 3 months (20 m/s), in accordance with EN 1991-1-4, even for a nominal duration of 3 days. This minimum wind velocity is intended to ensure safety for lifting and moving operations or other construction phases that are of short duration.

Such information can be obtained from weather forecasts of the nearest meteorological station and local wind measurements.

The relationships between characteristic values and return periods for climatic actions are given in the appropriate Parts of Eurocode 1:

- Snow loads

Annex D: EN 1991-1-3

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Table 3.3. Recommended return periods for determination of the characteristic values of climatic actions (Data taken from EN 1991-1-6, Table 3.1)

Duration	Return period (years)	Annual probability
$\leq$3 days	2[a]	0.5
$\leq$3 months (but $>$3 days)	5[b]	0.2
$\leq$1 year (but $>$3 months)	10	0.1
$>$1 year	50	0.02

[a] A nominal duration of 3 days, to be chosen for short execution phases, corresponds to the extent in time of reliable meteorological predictions for the location of the site. This choice may be kept for a slightly longer execution phase if appropriate organizational measures are taken. The concept of mean return period is generally not appropriate for short-term duration.

[b] For a nominal duration of up to 3 months, actions may be determined taking into account appropriate seasonal and shorter-term meteorological climatic variations. For example, the flood magnitude of a river depends on the period of the year under consideration.

If the available data show that the annual maximum snow load can be assumed to follow a Gumbel probability distribution, then the relationship between the characteristic value of the snow load on the ground and the snow load on the ground for a mean recurrence interval of n years is given by the formula:

$$s_n = s_k \left(\frac{1 - V\frac{\sqrt{6}}{\pi}\{\ln[-\ln(1-P_n)] + 0.57722\}}{(1 + 2.5923V)} \right)$$

where

s_k is the characteristic snow load on the ground (return period of 50 years)
s_n is the ground snow load with a return period of n years
P_n is the annual probability of exceedance (equivalent to approximately $1/n$, where n is the corresponding recurrence interval in years)
V is the coefficient of variation of annual maximum snow load.

Example: for $P_n = 0.2$ (which corresponds to a return period of 5 years) and $V = 0.4$:

$$s_{5\,\text{years}} = 0.632 s_k$$

cl. 4.2(2)P: EN 1991-1-4

- Wind actions

The 10-minute mean wind velocity having the probability p for an annual exceedance is determined by multiplying the basic wind velocity v_b by the probability factor, c_{prob}, given by the following expression:

$$c_{prob} = \left\{ \frac{1 - K\ln[-\ln(1-p)]}{1 - K\ln[-\ln(0.98)]} \right\}^n$$

where

K is the shape parameter depending on the coefficient of variation of the extreme-value distribution.
n is the exponent.

The recommended values for K and n are $K = 0.2$ and $n = 0.5$.

Example: for $p = 0.2$ (which corresponds to a return period of 5 years):

$$c_{prob} = \left\{ \frac{1 - 0.2\ln[-\ln(1-0.2)]}{1 - 0.2\ln[-\ln(0.98)]} \right\}^{0.5} = 0.85$$

This means that the wind velocity is multiplied by 0.85, and the dynamic pressure by $0.85^2 = 0.72$.

- Thermal actions (see Chapter 2 of this Designers' Guide and EN 1991-1-5)

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See also the Introduction and Part 6 of the TTL *Designers' Guide to Eurocode 1: Actions on buildings*.[4]

3.3.3. Ultimate limit states

No specific rules are given in EN 1991-1-6 concerning ultimate limit state (ULS) verifications, but it is the responsibility of the designer to select all appropriate design situations during execution in accordance with EN 1990. These design situations can either include accidental actions explicitly or refer to situations after an accidental event. In seismic zones, the seismic design situation to be taken into account during execution needs to be defined with the most basic information being the return period of the design earthquake. *cl. 3.2(1)P: EN 1991-1-6*

Obviously, the verifications of the structure are performed with the appropriate geometry and resistance of the partially completed structure corresponding to the selected design situations. *cl. 3.2(2)P: EN 1991-1-6*

3.3.4. Serviceability limit states

The serviceability limit states to be checked during execution are defined in the material-dependent Eurocodes (i.e. EN 1992 to EN 1995). In general, the objective of these verifications is mitigation of cracking and/or early deflections, and which may adversely affect the durability, fitness for purpose and/or aesthetic appearance in the final stage. As a consequence, load effects due to shrinkage and temperature should be taken into account in the design and should be minimized by appropriate detailing. *cl. 3.3: EN 1991-1-6*

Concerning combinations of actions, the frequent combination of actions is generally not relevant for execution phases of bridges. Therefore, the majority of verifications are based only on the characteristic and/or the quasi-permanent combinations of actions (e.g. for the calculation of shrinkage and creep effects in concrete bridge decks). *cl. 3.3(5): EN 1991-1-6*

Where relevant, serviceability requirements for auxiliary construction works are defined in order to avoid any unintentional deformations and displacements which affect the appearance or effective use of the structure or cause damage to finishes or non-structural members. *cl. 3.3(6): EN 1991-1-6*

3.4. Representation of actions

The determination of representative values of many actions, during execution, follows the same principles and methods as for persistent design situations. Special attention should be given to wind actions, actions due to water and construction loads. The determination of these actions is detailed in Sections 3.4.1 to 3.4.3 below. The other actions are covered in Section 3.4.4 below.

3.4.1, Wind actions (Q_W)

Wind may be the dominant action during the execution of many bridge types. In fact, it may have dynamic effects and can act dangerously during launching phases or where there are risks of:

- loss of static equilibrium
- loss of stability when the structure is on provisional bearings
- instability due to wind-induced vibrations such as vortex-induced crosswind vibrations, galloping flutter and rain-and-wind-induced vibrations possibly leading to fatigue phenomena (slender elements).The Eurocode recommends to examine when a dynamic response design procedure for wind actions is necessary for the execution stages, taking into account the degree of completeness and stability of the structure and its various elements. *cl. 4.7(1): EN 1991-1-6*

The treatment of unbalanced wind actions is not defined in EN 1991-1-6 or in EN 1991-1-4. This type of load is extremely important for segmental prestressed concrete bridges built by

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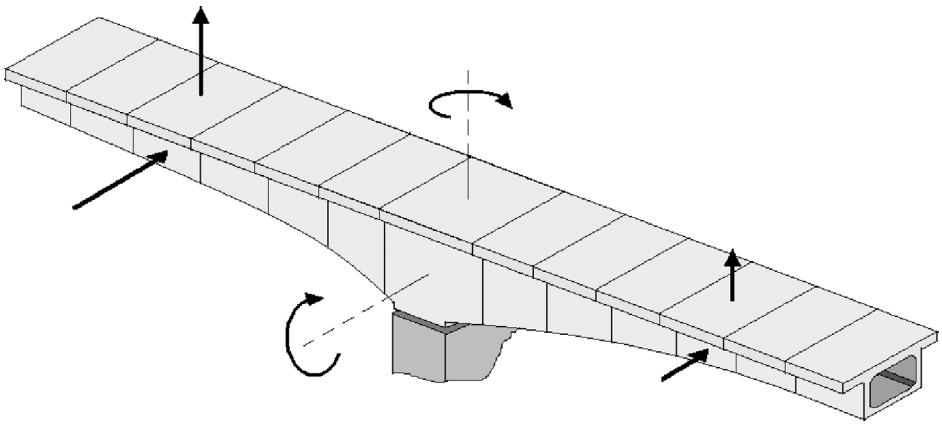


Fig. 3.1. Representation of unbalanced wind effects (drag and lift)

the balanced cantilever method. Indeed, balanced cantilever concrete bridges may be designed with very long spans with high piers across windy valleys or other windy zones. In such cases, structures are more or less flexible and sensitive to gusty wind during construction phases. In the case of very long cantilever arms, wind turbulence, and therefore the wind pressure, is not uniform. Unbalanced drag and unbalanced lift between the two parts of the arm can develop (Fig. 3.1 shows these effects schematically). In some cases, a wind action in the direction of the bridge axis may have to be taken into account.

cl. 4.7: EN 1991-1-6

EN 1991-1-6 states:

> *(2) Where a dynamic response procedure is not needed, the characteristic values of static wind forces Q_W should be determined according to EN 1991-1-4 for the appropriate return period.*
> *(3) For lifting and moving operations or other construction phases that are of short duration, the maximum acceptable wind speed for the operations should be specified.*
> *(4) The effects of wind induced vibrations such as vortex induced cross wind vibrations, galloping flutter and rain-wind should be taken into account, including the potential for fatigue of, for example, slender elements.*
> . . . . . .
> *(6) When determining wind forces, the areas of equipment, falsework and other auxiliary construction works that are loaded should be taken into account.*

According to the authors' experience of bridge design, a dynamic response procedure may be needed if the sum of the pier height and of the half-length of the longest arm is more than 200 m. For a quasi-static approach, it is possible to adopt a simplified approach based on the simplified method defined in *EN 1991-1-4 (Clause 8.3.2)*.

First, in most cases, a return period of 5 years may be selected. The basic wind speed is:

$$v_b = c_{dir} c_{season} v_{b,0.5}$$

and, in general,

$$v_b = v_{b,0.5}$$

where $v_{b,0.5}$ is the fundamental value corresponding to a return period of 5 years.

cl. 8.3.2: EN 1991-1-4

The simplified method (see Chapter 2 of this Designers' Guide and ***Clause 8.3.2: EN 1991-1-4***) gives the following formula:

$$F_W = \tfrac{1}{2}\rho v_b^2 C A_{ref,x} \text{ with } C = c_e \times c_{f,x}$$

and it is possible to introduce the two peak velocity pressures:

$$q_{p,x} = \tfrac{1}{2}\rho v_b^2 c_e \times c_{f,x} \text{ and } q_{p,z} = \tfrac{1}{2}\rho v_b^2 c_e \times c_{f,z}$$

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in the x and z directions, and to calculate them with the same assumptions:

- terrain category II
- $c_0 = 1$
- $k_1 = 1$
- $\rho = 1.25\,\text{kg/m}^{3.}$

Taking into account the expression for c_e:

$$q_{p,x} = v_b^2 \times c_{f,x} \times \left[0.02256 \ln^2(20z) + 0.158 \ln(20z)\right]$$

$$q_{p,z} = v_b^2 \times c_{f,z} \times \left[0.02256 \ln^2(20z) + 0.158 \ln(20z)\right]$$

It is proposed to apply these pressures (characteristic values) horizontally and vertically to half an arm length in order to get the most unfavourable unbalanced wind effects.

Example 3.1
For a box girder prestressed concrete bridge of variable depth, b/d_{tot} may be in the range 1 to 3. The basic wind velocity of a 5-year return period is $0.85 \times 26 = 22.1$ m/s. Let us adopt two pessimistic values: $c_{f,x} = 2$ and $c_{f,z} = 0.9$. If the reference height of the bridge is 80 m, the formulae give:

$$q_{p,x} = 22.1^2 \times 2 \times \left[0.02256 \ln^2(1600) + 0.158 \ln(1600)\right] = 2.338\,\text{kN/m}^2$$

$$q_{p,z} = 22.1^2 \times 0.9 \times \left[0.02256 \ln^2(1600) + 0.158 \ln(1600)\right] = 1.052\,\text{kN/m}^2$$

These values are probably conservative, but in line with real studies performed for the design of bridges on very high piers. Of course, these values are characteristic values.

In Section 113 of EN 1992-2 (*Concrete bridges – Design and detailing rules* – Clause 113.2) a recommended value of an uplift or horizontal pressure acting on one of the cantilevers for the verification of ultimate limit state of structural equilibrium is given. The recommended characteristic value is $0.2\,\text{kN/m}^2$ for the verification of static equilibrium. This value is rather low, but it can be considered that the wind action, with this value, is an accompanying action when the dominant action is an unbalanced effect of self-weight (see Chapter 8 of this Designers' Guide).

3.4.2. Actions caused by water (Q_{wa})

Groundwater is considered as belonging to the family of geotechnical actions (see Eurocode 7 and the TTL Designers' Guide for EN 1997[5]). EN 1991-1-6 gives rules for the determination of:

- (quasi-static) actions exerted by currents on immersed structures
- (quasi-static) actions due to accumulation of debris against immersed structures.

These actions are not specific for transient design situations, but they may have dominant effects on auxiliary structures during execution. Forces due to wave actions are addressed in ISO/DIS 21650.[6] Water and wave actions due to earthquakes (tsunamis) are not covered in the Eurocodes suite.

Actions exerted by currents on immersed structures

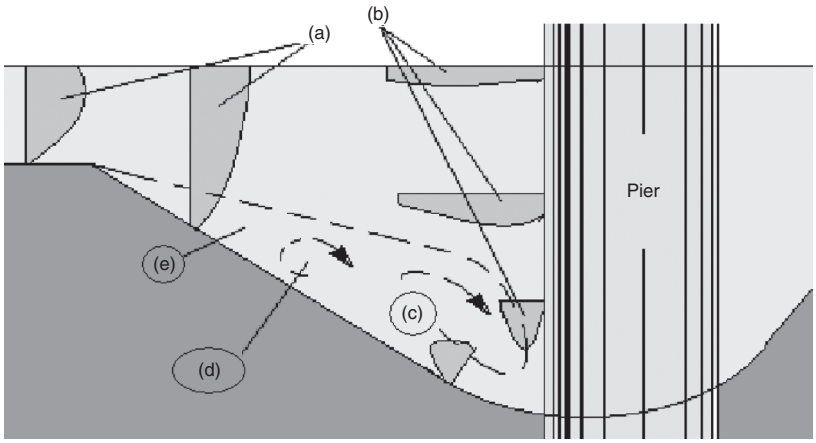
First, the determination of the water depth of a river should take into account an appropriate scour depth. Usually, a distinction is made between the general and the local scour depths. The general scour depth is the scour depth due to river flow, independently of the presence of an obstacle (scour depth depends on the flood magnitude – ***Clause 1.5.2.3: EN 1991-1-6***) and the local scour depth is the scour depth due to water vortices in the vicinity of an obstacle such as a bridge pier (see Fig. 3.2).

cl: 1.5.2.3: EN 1991-1-6

cl. 1.5.2.4: EN 1991-1-6

Actions caused by water, including dynamic effects where relevant, exerted by currents on immersed structures are represented by a force to be applied perpendicularly to the contact

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(a) Representation of horizontal water velocities
(b) Representation of vertical water velocities
(c) Vortex
(d) Small secondary vortex
(e) Dead water

Fig. 3.2. Local scour near a bridge pier

Expression 4.1: EN 1991-1-6

areas (Fig. 3.3). The magnitude of the total horizontal force F_{wa} (N) exerted by currents on the vertical surface is given by the following formula:

$$F_{wa} = \tfrac{1}{2} k \rho_{wa} h b v_{wa}^2$$

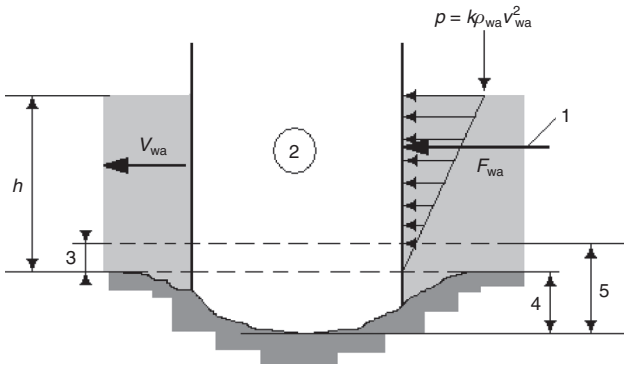
where

v_{wa} is the mean speed of the water averaged over the depth, in m/s
ρ_{wa} is the density of water, in kg/m^3
h is the water depth, but not including local scour depth, in m
b is the width of the object, in m
k is the shape factor:
$k = 1.44$ for an object of square or rectangular horizontal cross-section
$k = 0.70$ for an object of circular horizontal cross-section.

In general, the force due to water current is not critical as regards the stability of bridge piers. However, it may be significant for the stability of cofferdams.

Actions due to accumulation of debris against immersed structures

In some rivers, an accumulation of debris against immersed structures is possible, and the phenomenon may occur regularly. EN 1991-1-6 recommends representing the effects of



1 – Current pressure (p)
2 – Object
3 – General scour depth
4 – Local scour depth
5 – Total scour depth

Fig. 3.3. Pressure and force due to currents currents (Reproduced from EN 1991-1-6, with permission from BSI)

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such accumulation by a force F_{deb} (N), calculated for a rectangular object (e.g. a cofferdam), for example, from the following expression:

$$F_{deb} = k_{deb} A_{deb} v_{wa}^2 \qquad \text{EN 1991-1-6, (4.2)}$$

where

k_{deb} is a debris density parameter; the recommended value is $k_{deb} = 666\,kg/m^3$
v_{wa} is the mean speed of the water averaged over the depth, in m/s
A_{deb} is the area of obstruction presented by the trapped debris and falsework, in m^2.

3.4.3. Construction loads (Q_c)

cl. 1.5.2.2: EN 1991-1-6

As defined in ***Clause 1.5.2.2: EN 1991-1-6***, a construction load is a load that can be present due to execution activities, but is not present when the execution activities are completed. For consistency with this definition, it has been considered that construction loads would be classified as variable actions (see Table 3.2). A construction load may have vertical as well as horizontal components, and static as well as dynamic effects.

In general, construction loads are very varied. To take them easily into account, six sets have been defined in EN 1991-1-6 and models are proposed for some of them. These sets are described in Table 3.4 which reproduces Table 4.1 of EN 1991-1-6. The designer has to identify the construction loads for the design of an individual bridge; however, some heavy loads will only be known after the contractor, who will design the construction loads for the individual project, is selected.

After the identification of the construction loads for the individual project, these loads may be represented in the appropriate design situations, either, as one single variable action, or, where appropriate, different types of construction loads may be grouped and applied as a single variable action. Single and/or a grouping of construction loads should be considered to act simultaneously with non-construction loads as appropriate. Generally, construction loads are represented by the symbol Q_c.

The first set Q_{ca} corresponds to working personnel, staff and visitors, possibly with hand tools or other small site equipment (Fig. 3.4).

EN 1991-1-6 recommends that this loading be modelled as a uniformly distributed load $q_{ca} = 1\,kN/m^2$ (characteristic value) to be applied in order to obtain the most unfavourable effects. The recommended value is rather high, but it includes possible limited dynamic effects. Further, the load of the same origin for the design of scaffoldings is $0.75\,kN/m^2$.

The second set Q_{cb} corresponds to storage of movable items. In general, these loads are unknown in detail, and may have a random magnitude. Figure 3.5 shows a prestressing tendon, stored on a bridge deck during execution, and correctly protected by a plastic membrane. However, in case of rain, the membrane may be filled with water, which considerably increases the total weight.

These actions are modelled as free actions and represented as appropriate by:

- a uniformly distributed load q_{cb} with a recommended characteristic value equal to $0.2\,kN/m^2$
- a concentrated load F_{cb}, to be applied to obtain the most unfavourable effect. The recommended characteristic value of its magnitude is equal to 100 kN.

The third set Q_{cc} corresponds to non-permanent equipment in position for use during execution, either:

- static (e.g. formwork panels, scaffolding, falsework, machinery, containers), or
- during movement (e.g. travelling forms, launching girders and nose, counterweights).

Figure 3.6 shows a travelling form used for the construction of the Rion-Antirion cable-stayed bridge in Greece. Q_{cc} describes loads which are known only when the construction process commences. At the preliminary design stage, such loads may be difficult to estimate; however, for the most common bridge types, some ratios are well known. For example, in the

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Table 3.4. Representation of construction loads (Q_c) (Data taken from EN 1991-1-6, Table 4.1)

Construction loads (Q_c)				
Actions			Representation	Notes and remarks
Type	Symbol	Description		
Personnel and hand tools	Q_{ca}	Working personnel, staff and visitors, possibly with hand tools or other small site equipment	Modelled as a uniformly distributed load q_{ca} and applied to obtain the most unfavourable effects	Note 1: The characteristic value $q_{ca,k}$ of the uniformly distributed load may be defined in the National Annex or for the individual project. Note 2: The recommended value is 1.0 kN/m^2. See also 4.11.2.
Storage of movable items	Q_{cb}	Storage of movable items, e.g.: – building and construction materials, precast elements, and – equipment	Modelled as free actions and should be represented as appropriate by: – a uniformly distributed load q_{cb} – a concentrated load F_{cb}	Note 3: The characteristic values of the uniformly distributed load and the concentrated load may be defined in the National Annex or for the individual project. For bridges, the following values are recommended minimum values: – $q_{cb,k} = 0.2$ kN/m^2 – $F_{cb,k} = 100$ kN where $F_{cb,k}$ may be applied over a nominal area for detailed design. For densities of construction materials, see EN 1991-1-1.
Non-permanent equipment	Q_{cc}	Non-permanent equipment in position for use during execution, either: – static (e.g. formwork panels, scaffolding, falsework, machinery, containers), or – during movement (e.g. travelling forms, launching girders and nose, counterweights)	Modelled as free actions and should be represented as appropriate by: – a uniformly distributed load q_{cc}	Note 4: These loads may be defined for the individual project using information given by the supplier. Unless more accurate information is available, the loads may be modelled by a uniformly distributed load with a recommended minimum characteristic value of $q_{cc,k} = 0.5$ kN/m^2. A range of CEN design codes is available, e.g. see EN 12811 and for formwork and falsework design see EN 12812.
Movable heavy machinery and equipment	Q_{cd}	Movable heavy machinery and equipment, usually wheeled or tracked, (e.g. cranes, lifts, vehicles, lift trucks, power installations, jacks, heavy lifting devices)	Unless specified should be modelled on information given in the relevant parts of EN 1991	Information for the determination of actions due to vehicles when not defined in the project specification, may be found in EN 1991-2. Information for the determination of actions due to cranes is given in EN 1991-3.
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)	Taken into account by considering possible mass effects on horizontal, inclined and vertical elements (such as walls)	Note 5: These loads may vary significantly, and over short time periods, depending on types of materials, climatic conditions, build-up rates and clearance rates, for example.
Loads from parts of a structure in a temporary state	Q_{cf}	Loads from parts of a structure in a temporary state (under execution) before the final design actions take effect (e.g. loads from lifting operations)	Taken into account and modelled according to the planned execution sequences, including the consequences of those sequences (e.g. loads and reverse load effects due to particular processes of construction, such as assemblage)	See also 4.11.2 for additional loads due to fresh concrete.

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Fig. 3.4. Example of construction load Q_{ca}

case of cast-in-place segmental bridges built by the cantilever method, the weight of the travelling form is about 50% of the weight of the heaviest segment.

If the designer has absolutely no idea about the construction systems that will be used, the Eurocode proposes to cover the Q_{cc} load with a free uniformly distributed load with a minimum recommended characteristic value $q_{cc,k} = 0.5\,\text{kN/m}^2$. However, it has to be clearly understood that this uniformly distributed load has no physical meaning.

The fourth family Q_{cd} corresponds to movable heavy machinery and equipment, usually wheeled or tracked (e.g. cranes, lifts, vehicles, lift trucks, power installations, jacks, heavy lifting devices). Figure 3.7 gives examples of this family. These loads need to be known in

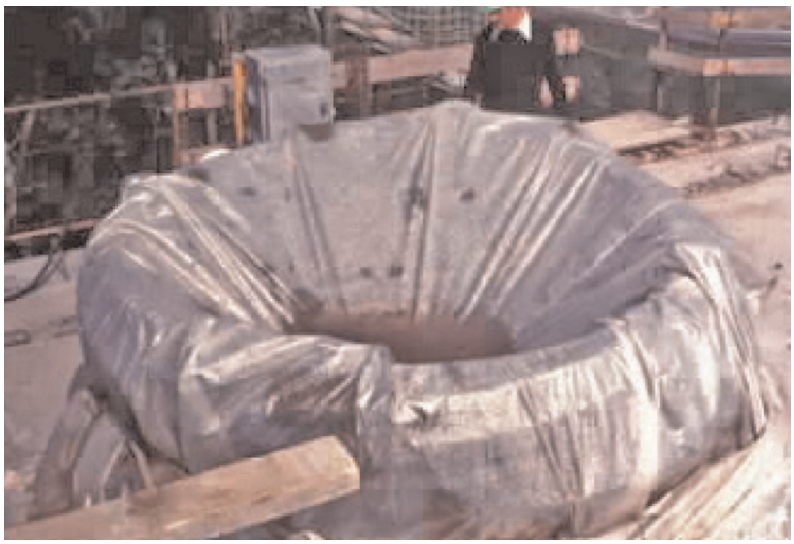


Fig. 3.5. Example of construction load Q_{cb}

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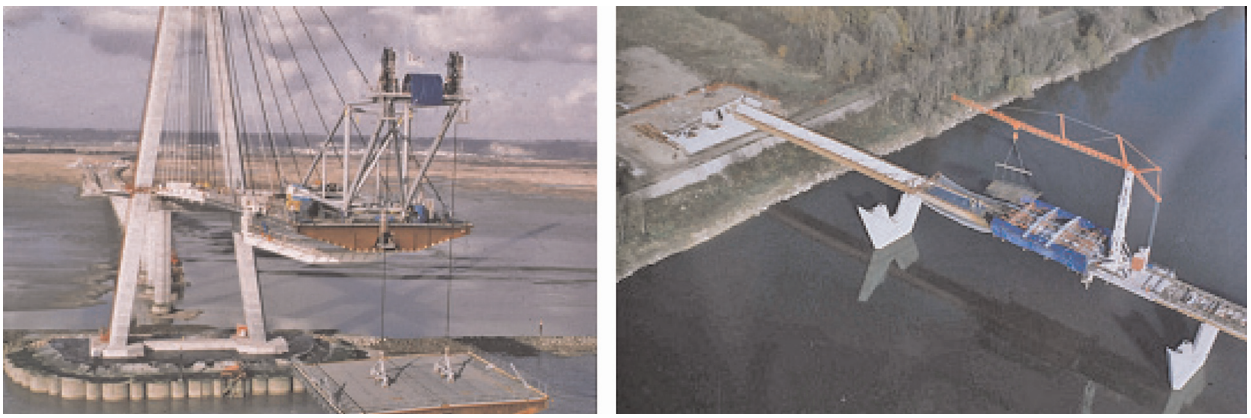


Fig. 3.6. Example of construction load Q_{cc} (Rion-Antirion bridge)

order to perform the appropriate verifications during execution. They can be estimated at the design stage if the construction process is known. No load model is defined by the Eurocode.

The fifth set Q_{ce} corresponds to accumulation of waste materials: it normally does not apply to bridges but it may be envisaged in very special cases (bridges in urban areas) and for certain types of bridges (e.g. robust slab bridges). No load model is defined by the Eurocode.

Finally, the sixth set Q_{cf} corresponds to loads from parts of a structure in a temporary state. A good, and very common, example to illustrate this type of construction load is the concreting of an element. Figure 3.8 shows the casting of concrete for the execution of



(a) (b)

Fig. 3.7. Examples of construction load Q_{cd}: (a) Lifting system (Pont de Normandie); (b) Crane on a composite steel–concrete bridge deck during execution

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Fig. 3.8. Execution of a concrete bridge segment – example of association of $Q_{ca} + Q_{cc} + Q_{cf}$

a bridge segment. In this figure, there are simultaneously Q_{ca} loads (working personnel), Q_{cc} loads (travelling form) and Q_{cf} loads (weight of fresh concrete).

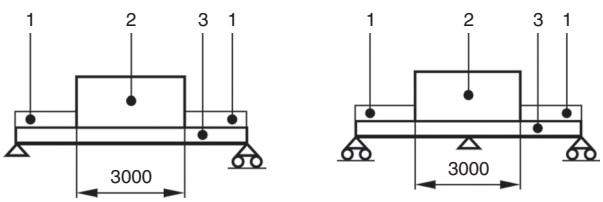
For this type of loading, EN 1991-1-6 recommends a detailed procedure which is summarized in Table 3.5 (reproduced from Table 4.2 of the Eurocode). The load in the working area corresponds to the possibility of a local accumulation of fresh concrete on the slab. In accordance with EN 1991-1-1, the density of fresh normal concrete is 26 kN/m^3. However, other values may have to be taken into account, for example when using self-levelling concrete or precast products for some structural elements of bridges.

3.4.4. Representation of other actions

EN 1991-1-6 highlights some aspects concerning the following actions, which are already defined in other parts of EN 1991, due to the construction phase:

Table 3.5. Recommended characteristic values of actions due to construction loads during casting of concrete (Data taken from EN 1991-1-6, Table 4.2)

Action	Loaded area	Load in kN/m^2
(1)	Outside the working area	0.75 covering Q_{ca}
(2)	Inside the working area 3 m × 3 m (or the span length if less)	10% of the self-weight of the concrete but not less than 0.75 and not more than 1.5 – includes Q_{ca} and Q_{cf}
(3)	Actual area	Self-weight of the formwork, load-bearing element (Q_{cc}) and the weight of the fresh concrete for the design thickness (Q_{cf})



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Fig. 3.9. Snow loads on a bridge deck in winter, during execution

cl. 4.2: EN 1991-1-6
- Actions on structural and non-structural members during handling.
- Geotechnical actions (see EN 1997 and the TTL *Designers' Guide to Eurocode 7*,[5] concerning settlements).

cl. 4.3: EN 1991-1-6
- Actions due to prestressing. If prestressing forces during the execution stage should be taken into account as permanent actions, the loads on the structure from stressing jacks during the prestressing activities should be classified as variable actions for the design of the anchor region. This rule is innovative, and means that the maximum pre-stressing force should be multiplied by a partial factor (probably 1.35) for a verification of the reinforcement at the ultimate limit state of the anchor region.

cl. 4.4: EN 1991-1-6

cl. 4.5: EN 1991-1-6
- Pre-deformations.
- Temperature, shrinkage and hydration effects. In the case of bridges, attention is drawn to the time lag between casting one concrete element to another element that has already hardened. In general, the limit state to be checked is the prevention of unacceptable cracks or crack widths, especially in the case of steel–concrete composite structures. Attention is also drawn to possible restraints from the effects of friction of bearings.

cl. 4.6: EN 1991-1-6
- Snow loads. As shown in Fig. 3.9, snow loads may become a dominant action for bridges during execution, when located on mountain routes: indeed, they may remain for several months (in winter) without any human intervention and accumulation of snow may lead to problems of static equilibrium.

cl. 4.8: EN 1991-1-6

Annex A2 to EN 1991-1-6 gives the following rules. Snow loads on bridges during execution are based on values specified in EN 1991-1-3 taking account of the relevant return period. When daily removal of snow (also during weekends and bank holidays) is required for the project and safety measures for removal are provided, the characteristic snow load should be reduced compared to the value specified in EN 1991-1-3 for the final

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Fig. 3.10. Fall of a travelling form

stage: the recommended characteristic value during execution is 30% of the characteristic value for permanent design situations. However, for the verification of static equilibrium (EQU) in accordance with EN 1990, and where justified by climatic conditions and the anticipated duration of the construction phase, the characteristic snow load should be assumed to be uniformly distributed in the areas giving unfavourable action effects with a recommended characteristic value equal to 75% of the characteristic value for permanent design situations resulting from EN 1991-1-3.

- Actions due to atmospheric icing include mainly loads by ice on water (floating ice), or icing of cables or other structural parts of masts and towers. EN 1991-1-6 refers mainly to ISO 12494 standard.[7] *cl. 4.10: EN 1991-1-6*
- Accidental actions. In accordance with EN 1991-1-6, '*Accidental actions such as impact from construction vehicles, cranes, building equipment or materials in transit (e.g. skip of fresh concrete), and/or local failure of final or temporary supports, including dynamic effects, that may result in collapse of load-bearing structural members, shall be taken into account, where relevant*'. *cl. 4.12: EN 1991-1-6*
- It is the responsibility of the designer to select the accidental design situations and the design values of accidental actions during execution, depending on the type of bridge under construction. The most critical accidental actions are:
  - the loss of stability of a bridge deck during launching due to an exit from temporary bearings
  - the fall of equipment (e.g. a travelling form during its displacement – Fig. 3.10), including the dynamic effects
  - the fall of structural elements (e.g. the fall of a precast segment before the final pre-stressing is active), including dynamic effects (Fig. 3.11)
  - the fall of a crane.

  In general, the dynamic effects may be taken into account by a dynamic amplification factor for which the recommended value is equal to 2. This implies that the action effect of the fall (e.g. of the travelling form) is equivalent to a force equal and opposite to its self-weight. Of course, a linear elastic behaviour of the structure and of its members is assumed. In specific cases a dynamic analysis is needed. Finally, attention *Note 2 to cl. 4.12(1)P: EN 1991-1-6*

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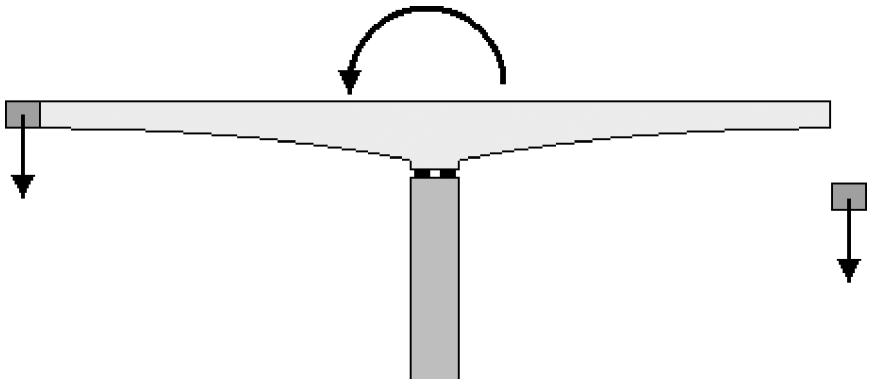


Fig. 3.11. Fall of a precast segment

is drawn to the fact that many of the actions mentioned above may induce movement in the structure: the magnitude of movements and the possibility of progressive collapse may have to be assessed.

- Seismic actions. EN 1991-1-6 mentions that the design values of ground acceleration and the importance factor γ_I need to be defined for the individual project, if it is not defined at the national level through a National Annex. Nevertheless, a project specification for very short-term phases or local effects is generally irrelevant.

3.5. Specific rules

Annex A2 to EN 1991-1-6 provides supplementary rules for bridges. The application of snow loads during execution has already been detailed in Section 3.4.4 of this Designers' Guide.

No specific rules are defined for prestressed concrete bridges built by the cantilevered method during execution. The most important verifications are based on serviceability requirements to avoid excessive cracking and deformations where there is also guidance in EN 1992-2 and the corresponding TTL Designers' Guide.[8]

One of the most important design situations is the loss of static equilibrium. The EQU limit state may have to be checked with the fundamental and/or the accidental design situations. In the most common cases, the accidental design situation may be due to the fall of a travelling form during its displacement or of a precast segment before the final prestressing force applies. In both cases, the dynamic effects need to be taken into account.

Figure 3.12 shows an example of loads which are commonly to be taken into account for prestressed cantilever bridges during execution. A worked example is given in Chapter 8.

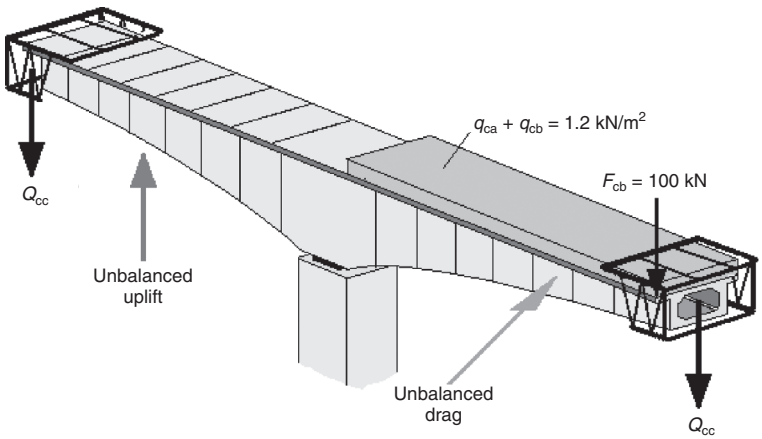


Fig. 3.12. Representation of various actions to be taken into account during execution

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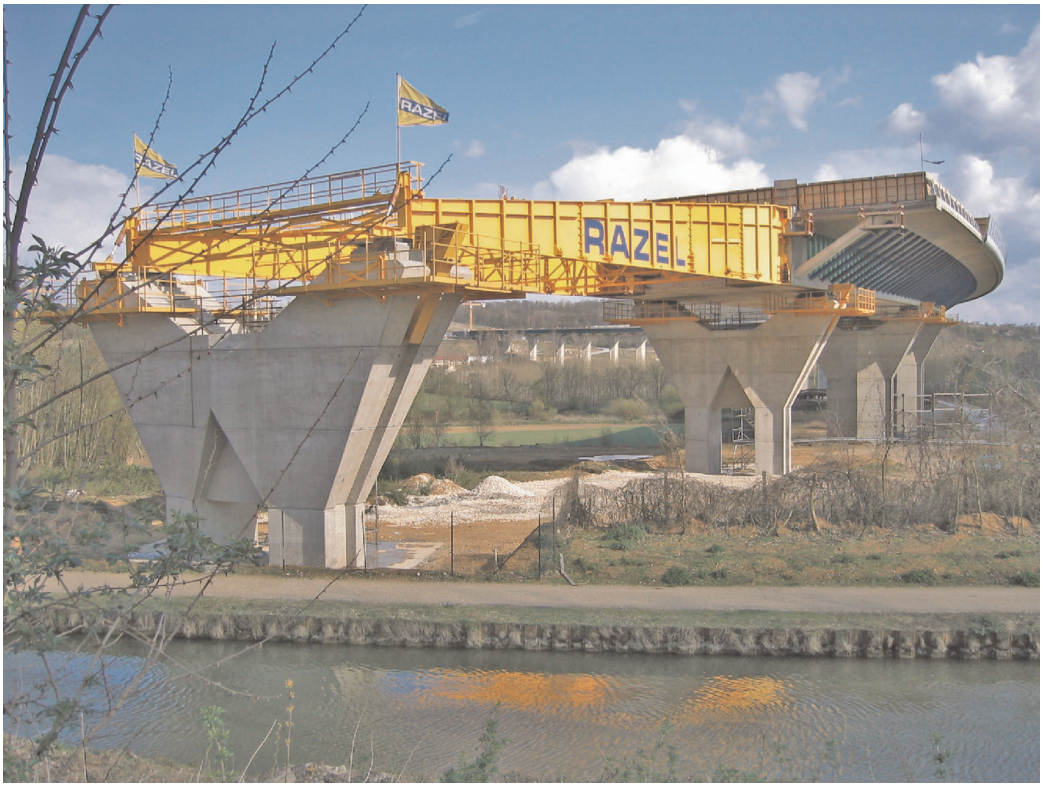


Fig. 3.13. Example of launching of a bridge with a launching nose

As explained in Table 3.2, for some local verifications, the impact area of $F_{cb,k}$ should be defined in the project specification.

Sometimes, in the case of bridges built with precast segments, the project specification defines geometrical uncertainties concerning the precasting form. One way to define these uncertainties is to determine the effects of an angular difference between two precast segments, for example equal to 0.5×10^{-3} rad.

In the case of prestressed concrete or composite bridges built by the incremental launching method, Annex A2 to EN 1991-1-6 gives several complementary rules concerning:

- deflections
- friction effects.

Several methods may be used to launch a prestressed concrete bridge (see the example given in Fig. 3.13). For the launching process, several systems exist, but in any case, the bridge deck slides on steel plates on the beams of the casting area and on provisional bearings on piers.

Prestressed concrete bridges built by the incremental launching method are designed in such a way that consideration of loss of static equilibrium is generally irrelevant. The design situations to be taken into account are mainly related to typical serviceability limit states, with temporary prestressing tendons. For the verification of these limit states, deflections need to be taken into account to cover effects of the possible unevenness of temporary bearings. Recommended characteristic values of deflections in the longitudinal and transverse directions are given as follows: *A2.3: EN 1991-1-6*

- ± 10 mm longitudinally for a single bearing line (all other pads are assumed to be at their theoretical level)
- ± 0.25 cm in the transverse direction for a single bearing line (all other pads are assumed to be at their theoretical level).

Figure 3.14 shows some of the actions and deformations to be taken into account in the design.

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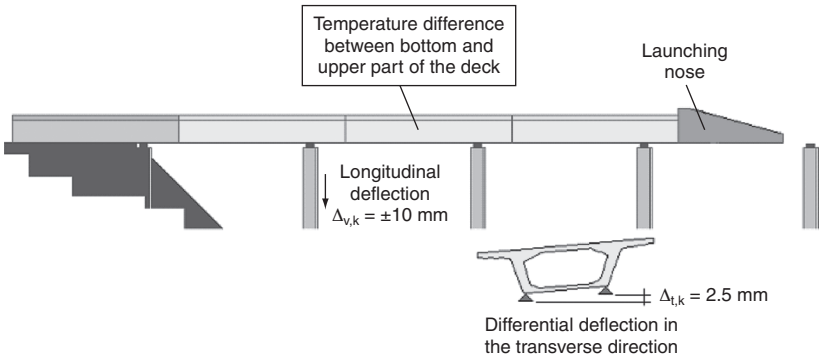


Fig. 3.14. Specific actions during launching of prestressed concrete bridges

Normally, the launching of a bridge is not a continuous process, and the verification of imposed deflections should be made at each launching step. However, this may be very complex for long bridges, and it is acceptable to determine the global effects (maximum and minimum) for the bridge deck in its final position. Such a 'simplified' method is conservative compared to the rule defined in EN 1991-1-6 Annex A2.

The characteristic values of deflections may be adjusted if specific control measures are taken during execution. Attention is drawn to the fact that box-girder bridge decks are very sensitive to a transverse deflection at their ends (e.g. on abutments). In any case, the deflections in the longitudinal and transverse directions are taken into account separately. In some circumstances, settlements of foundations may have to be taken into account.

In some cases, the question of static equilibrium may be crucial (Fig. 3.15).

The launching method for steel girders commonly uses a counterweight because the structure is rather light (Fig. 3.16). The way to check static equilibrium is detailed in Chapter 8 of this Designers' Guide.

Friction effects between the deck and the substructure depend on the nature of the contact: elastomeric bearings with Teflon sliding on stainless steel, steel plates sliding on lubricated steel, etc.



Fig. 3.15. Example of launching of steel girders of a composite bridge over railway tracks

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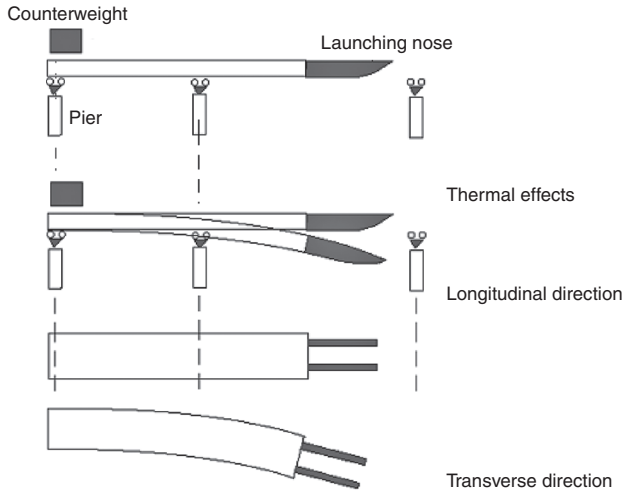


Fig. 3.16. Launching of a bridge deck with a counterweight

Annex A2 to EN 1991-1-6 gives the following recommended values for the determination of friction forces: *A2.5: EN 1991-1-6*

- 10% of the vertical loads for the total longitudinal forces
- at every pier, the longitudinal friction forces are determined by using a lower value and an upper value of friction coefficients, $\mu_{\min}$ and $\mu_{\max}$. The recommended values are $\mu_{\min} = 0$ and $\mu_{\max} = 0.04$.

These recommended values seem to be inconsistent. However, with modern systems, the friction forces at piers are rather low, even when a launching phase starts. However, the friction effects are higher on the beams of the construction area (Fig. 3.17).



Fig. 3.17. The friction effect may be important on the beams of the construction area

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In conclusion, the design value of the total horizontal friction forces should be used for the design of members in the construction area.

In all cases, thermal actions to be taken into account during execution should be defined in the project specification. Indeed, thermal actions may give rise to structural effects where the structure is statically undetermined. As an example, where temporary stays are used, specific rules concerning thermal effects need to be defined for these stays.

The Eurocodes do not define the characteristic values of thermal actions to be taken into account during execution. They have to be defined in the project specification with reference to good practice. For example, in the case of traditional prestressed concrete bridges, a difference of temperature of 6°C between the top slab and the bottom slab is acceptable as a characteristic value.

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CHAPTER 4

Traffic loads on road bridges

4.1. General

This chapter is concerned with the description of traffic load models applicable to road bridges during permanent and transient design situations. The material in this chapter is covered in the relevant sections and Annexes of Part 2 of EN 1991 *Actions on structures – Traffic loads on bridges*. The ψ and γ factors applicable to the components of road traffic for establishing the combinations of actions are given in Chapter 8 of this Designers' Guide, the material of which is covered in EN 1990 Annex A2.

Chapter 4 of EN 1991-2 defines:

- four models of vertical load (denoted LM1 to LM4) for serviceability and ultimate limit state verification except fatigue verification
- models of horizontal forces (braking, acceleration and centrifugal forces)
- five models of vertical load for fatigue verification (denoted FLM1 to FLM5)
- actions for accidental design situations (accidental location of heavy vehicles on various parts of decks, collision forces from vehicles under or on the bridge)
- actions on pedestrian parapets
- load models for abutments and walls adjacent to bridges.

The collision forces from vehicles under the bridge are covered in EN 1991-1-7 and described in Chapter 7 of this Designers' Guide.

From a general viewpoint, all load models defined in Section 4 of EN 1991-2 are applicable for the design of new road bridges including piers, abutments, upstand walls, wing walls and flank walls etc. and their foundations. However, specific rules need to be defined in some cases, for example for bridges receiving simultaneously road and rail traffic, for masonry arch bridges, buried structures, retaining walls and tunnels. *Foreword: EN 1991-2*

Traffic actions for road bridges, as well as for footbridges and railway bridges, consist of variable and accidental actions (or actions related to accidental design situations). However, for normal conditions of use, they have obviously to be treated as free (within some limits) variable actions. Moreover, traffic actions are multi-component actions, which means that a well-identified type of traffic gives rise to vertical and horizontal, static and dynamic forces. In order to facilitate the combinations of actions, EN 1991-2 has introduced the concept of 'group of loads' for road bridges as well as for footbridges and railway bridges. *cl. 2.1(3): EN 1991-2* *cl. 2.1(4): EN 1991-2*

4.2. Field of application

The load models defined in Section 4 of EN 1991-2 are applicable for loaded lengths less than 200 m. This limitation is not really a technical limitation: the calibration of the two main *cl. 4.1(1): EN 1991-2*

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models of vertical loads for limit states other than fatigue (i.e. LM1 and LM2) has been based on effects of actions for influence lines and areas corresponding to loaded lengths less than 200 m (see the annex to this chapter), and this loaded length has been adopted to define the field of application of all models (including fatigue models) in this chapter. In fact, the load models may be used for loaded lengths more than 200 m, but LM1, with α-factors equal to 1 (see Section 4.3.5 below), may give pessimistic results beyond 300 m for a two- or three-lane carriageway. For this reason, the Eurocode mentions that the load models may be defined in the National Annex or for the individual project outside the field of application. In the UK National Annex for EN 1991-2, load model 1 (LM1) is applicable to lengths up to 1200 m.

Note 2 to cl. 4.1(1): EN 1991-2

The Eurocode is deemed to cover road traffic effects corresponding to normally foreseeable situations, but the effects of loads on road construction sites are not automatically covered. Specific verifications need to be performed for the individual project.

4.3. Models of vertical loads to be used for all limit states except fatigue

4.3.1. General

cl. 4.3.1: EN 1991-2

The four models of vertical loads are:

- a main load model (LM1), including concentrated loads (tandem systems, called TS) and uniformly distributed loads (called UDL) and applicable to all bridges
- a model consisting of a single axle with two wheels (LM2), in addition to the previous one (LM1) for the verification of short structural members (3–7 m)
- a model made up by a set of special vehicles intended to take into account the effects of exceptional convoys (LM3)
- a model corresponding to the loading of the surface of the bridge with a uniformly distributed load of $5\,\mathrm{kN/m^2}$, corresponding to the effects (dynamic amplification included) of a crowd (LM4).

LM3 and LM4 are normally used as specified for an individual project, and only when required by the client.

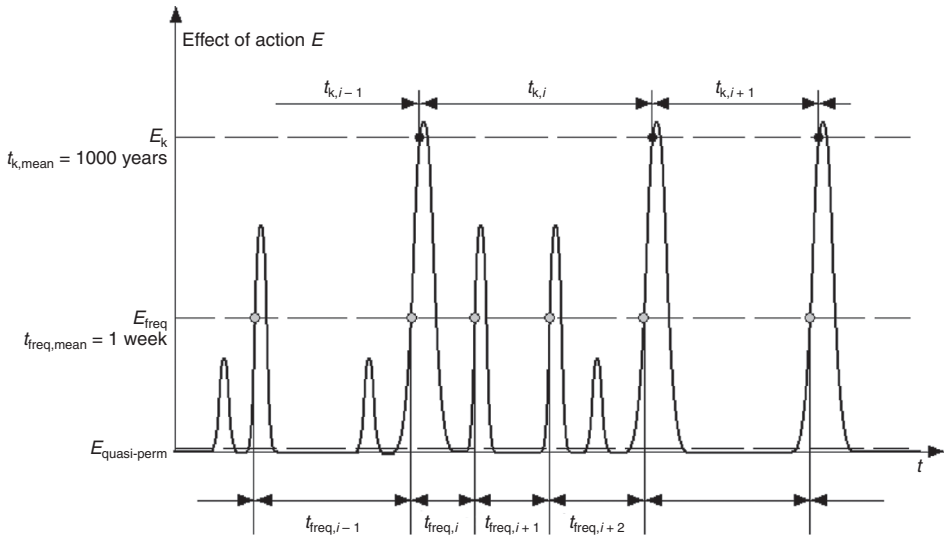
4.3.2. Levels of magnitude for load models LM1 and LM2

Several levels of magnitude are provided for load models LM1 and LM2, corresponding to different return periods, for their use in various combinations of actions:

- The characteristic level corresponds to a return period of 1000 years, which means a probability of being exceeded of 5% in 50 years or 10% in 100 years – see the TTL *Designers' Guide to EN 1990 – Eurocode: Basis of structural design*.[1] (Note that at the ENV stage, an additional level was requested by experts drafting Part 2 of Eurocode 2: Concrete bridges; this level was denoted 'infrequent' and corresponded to a return period of 1 year. The infrequent values of traffic actions are still evoked in EN 1991-2 and in EN 1990 Annex A2; at present it seems that these values are used in some countries.)
- The frequent level corresponds to a return period of one week.
- The quasi-permanent values are generally equal to zero for traffic loads. It should be remembered that, in accordance with EN 1990 – *Eurocode: Basis of Structural Design*, the quasi-permanent value of a variable action is defined as follows: '*value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a determined part of the characteristic value by using a factor* $\psi_2 \leq 1$'. Obviously, for the large majority of road bridges, the quasi-permanent value of traffic loads is close to 0. Nevertheless, for road bridges that support heavy and continuous traffic, a quasi-permanent value different from zero may be appropriate. For bridges with intense traffic and located in seismic areas (***Clause 4.1.2: EN 1998-2***) recommends adopting the value $\psi_2 = 0.2$.

cl. 4.1.2: EN 1998-2

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$t_{k,i}$ is the time between two successive exceedances of the characteristic value
$t_{k,mean}$ is the mean value of $t_{k,i}$, i.e. the return period of the characteristic value
$t_{freq,mean}$ is the mean value of the time $t_{freq,i}$ between two exceedances of the frequent value, i.e. the return period of the frequent value.

Fig. 4.1. Definition of the various levels for effects of traffic loads

The concepts of characteristic, frequent and quasi-permanent levels are represented diagrammatically in Fig. 4.1. See also Chapter 4 of the TTL Designers' Guide to EN 1990.[1]

Background information
Generally, characteristic values of climatic actions for the design of construction works are based on a return period of 50 years (i.e. a probability of exceedence of 2% per year). In the case of road traffic loads, the experts charged with the development of EN 1991-2 adopted a definition of characteristic values based on a probability of exceedence of 5% in 50 years (or 10% in 100 years), which corresponds to a return period of 1000 years. This choice was mainly motivated by a strong will to limit the probability of several occurrences of irreversible serviceability limit states during the reference period (50 years). This was justified by the fact that the approach adopted for road traffic loads started from the assessment of load effects and not, as for climatic loads, from the assessment of a parameter partially representing the action (e.g. wind velocity). Taking into account the hidden safety margins in the models of some variable actions, the order of magnitude of the return period of a climatic action is in the range 200–300 years. Moreover, the tail of the distribution of traffic effects is very narrow (the scatter of the maximum weight of heavy vehicles is limited); as a consequence, there is no significant difference between the characteristic values of actions effects for 1000 and 100 years (see the annex to this chapter). Briefly, the value of the return period has been selected in order to limit the probability for any irreversible limit state to be exceeded during the period of reference and it is rational to think that the loads will increase in the future (see also Chapter 1 of this Designers' Guide).

4.3.3. Division of the carriageway

For application of the various load models, the basic concept is the division of the carriageway into notional lanes. *cl. 4.2.3: EN 1991-2*

First, the width w of the carriageway is measured between inner limits of vehicle restraint systems or between kerbs (see Fig. 4.2) where these kerbs have a minimum height which is defined at the national level, with a recommended value equal to 100 mm. The carriageway *cl. 4.2.3(1): EN 1991-2*

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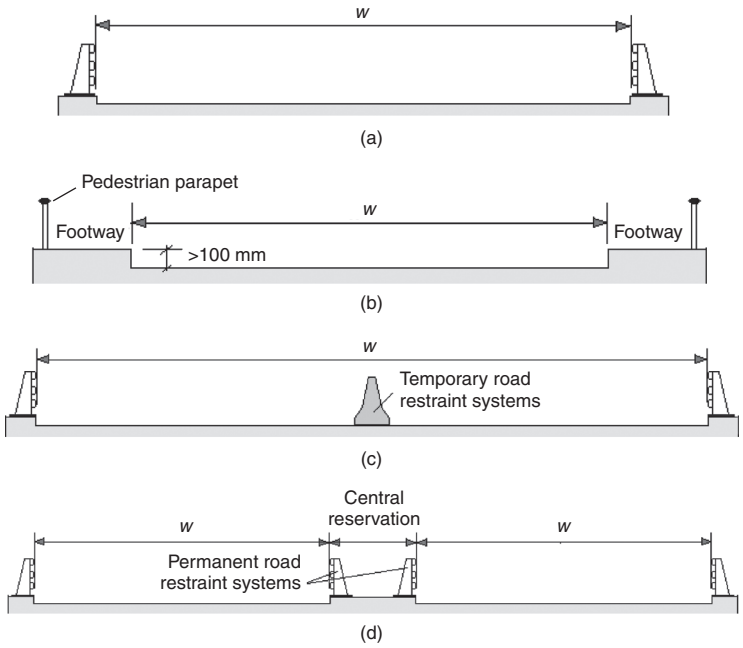


Fig. 4.2. Examples of carriageway widths: (a) Carriageway between safety barriers; (b) Carriageway between footways (or service paths protected by kerbs); (c) Carriageway consisting of two separate parts with a central temporary road restraint system; (d) Carriageway consisting of two separate parts with a central permanent road restraint system: the central reservation is not included in the carriageway width

cl. 4.2.3(2): EN 1991-2

width w is divided into the greatest possible integer number n_l of notional lanes: the normal width of a notional lane is $w_l = 3\,\text{m}$, except for a carriageway width such that $5.4\,\text{m} \leq w < 6\,\text{m}$, as shown in Table 4.1 which reproduces Table 4.1 of the Eurocode.

The difference between the carriageway width and the width of all notional lanes is the width of the remaining area.

cl. 4.2.3(3): EN 1991-2

Where the carriageway width is variable, the division into lanes follows the same principles.

Where the carriageway on a bridge deck is physically divided into two parts separated by a central reservation, then:

- each part, including all hard shoulders or strips, should be separately divided into notional lanes if the parts are separated by a permanent road restraint system;
- *cl. 4.2.3(4): EN 1991-2* the whole carriageway, central reservation included, should be divided into notional lanes if the parts are separated by a temporary road restraint system.

Figure 4.2 gives examples of carriageway widths for their division into notional lanes.

Table 4.1. Number and width of notional lanes (Data taken from EN 1991-2, Table 4.1)

Carriageway width, w	Number of notional lanes	Width of a notional lane, w_l	Width of the remaining area
$w < 5.4\,\text{m}$	$n_l = 1$	3 m	$w - 3\,\text{m}$
$5.4\,\text{m} \leq w < 6\,\text{m}$	$n_l = 2$	$\frac{w}{2}$	0
$6\,\text{m} \leq w$	$n_l = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$

Note: For example, for a carriageway width equal to 11 m, $n_l = \text{Int}\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is $11 - 3 \times 3 = 2\,\text{m}$.

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4.3.4. Location and numbering of lanes and principles for application of load models on individual lanes

cl. 4.2.4 and 4.2.5: EN 1991-2

Load models LM1 and LM2 have been defined and calibrated in order to give effects as close as possible to 'extrapolated target effects' (adjusted to the selected return periods) determined from effects due to measured real traffic. Therefore, it has to be clearly understood that the load models are to be applied on notional lanes which are not physical lanes, and that the numbering of the notional lanes depends on the conditions of application of the load model with the purpose of getting, in all cases, the most adverse effect. In other words, there is no 'physical' numbering of the notional lanes. Nevertheless, the location and numbering of notional lanes is in accordance with the following principles:

- For the application of Load Models LM1 and LM2 for limit states other than fatigue limit states, the lane giving the most unfavourable effect is numbered Lane No. 1, the lane giving the second most unfavourable effect is numbered Lane No. 2, and so on. *cl. 4.2.4(4): EN 1991-2*
- For fatigue verifications, the location and numbering of the lanes is selected depending on the traffic to be expected in normal conditions. Nevertheless, a possible evolution of the carriageway (widening of a bridge deck) may have to be taken into account at the design stage. *cl. 4.2.4(3): EN 1991-2*
- Where the carriageway consists of two parts on the same deck separated by a central reservation, each part, including all hard shoulders or strips, is separately divided into notional lanes for the case of a permanent road restraint system, and the whole carriageway, central reservation included, is divided into notional lanes in the case of a temporary road restraint system. *cl. 4.2.4(5): EN 1991-2*
- However, in any case, where the carriageway consists of two separate parts on the same deck, only one numbering is to be used for the whole carriageway, which means that there is only one lane No. 1 (this lane can, of course, be alternatively on the two parts).
- Where two different decks are supported by the same piers or abutments, only one numbering of the lanes is to be taken into account for the design of the piers or abutments, independently of the fact that there is a specific numbering of the lanes for the design of each bridge deck. For example, if carriageways in Fig. 4.2(c) and (d) are supported by the same deck, there is only one numbering of the whole carriageway. *cl. 4.2.4(6): EN 1991-2*

Even if it is not mentioned in the Eurocode, it is understood that the numbering of the lanes for limit states other than fatigue is determined from the characteristic values of the models of vertical loads. This numbering is retained for verifications where the load models are taken into account with other representative values, for example the frequent values. Figure 4.3 gives an example of division of a carriageway.

Example 4.1. Division of a carriageway

- Unique deck and temporary central road restraint system:

  $w = 24.50\,\text{m}, n_l = 8$ lanes + remaining area 0.50 m

- Unique deck and permanent central road restraint system:

  $w = 2 \times 11.00\,\text{m}, n_l = 3$ lanes + remaining area 2 m on each side

Total: 6 lanes + remaining area 4 m (but only one slow lane – Lane No. 1)

- Two independent decks supported by the same piers:

  $w = 2 \times 11.00\,\text{m}, n_l = 3$ lanes + remaining area 2 m on each side

Two separate lane numberings for their calculation (2 lanes No. 1)
A unique lane numbering for the design of the substructure (1 lane No. 1)

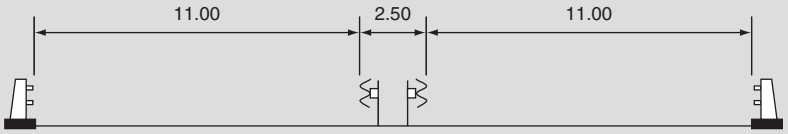


Fig. 4.3. Example of division of a carriageway

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cl. 4.3.2: EN 1991-2

4.3.5. Load model No. 1 (main characteristic model)

Description

The main characteristic model (LM1) is represented in Fig. 4.4. It has been selected and calibrated to cover the most common traffic effects with an appropriate reliability margin. Scientific studies have been performed, based on real traffic data, and on various theoretical developments. After identification of the notional lanes on the carriageway, these lanes are loaded by:

cl. 4.3.2(1): EN 1991-2

- a uniformly distributed load (UDL)
- a tandem system including two axles (TS).

A maximum of three notional lanes are loaded with a single tandem system per lane, which means that, for an individual project or in the National Annex, it can be decided to use only one (not recommended) or two tandem systems.

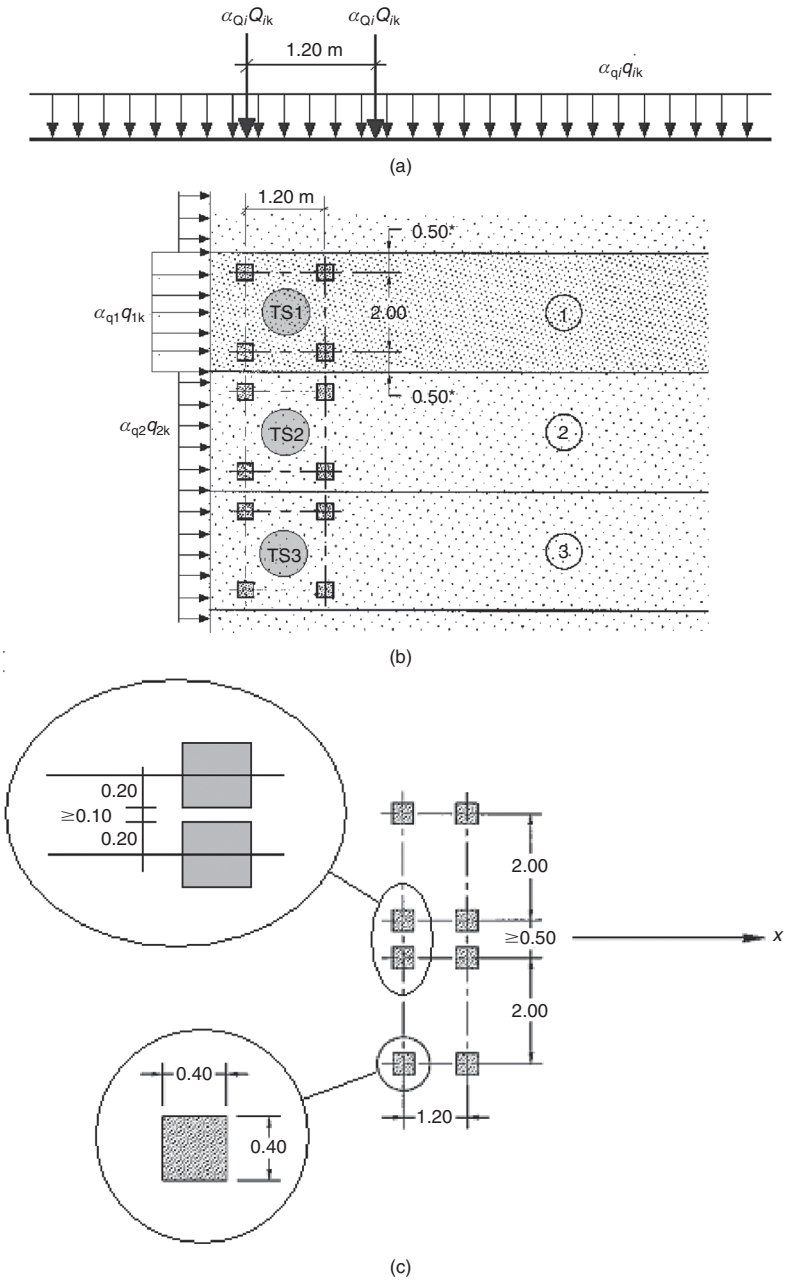


Fig. 4.4. Load Model No. 1: (a) Application of TS and UDL along the longitudinal axis; (b) Application of LM1 on the notional lanes; (c) Location of tandem systems for the verification of short structural members

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Example 4.2. Rules for application of CMA

Figure 4.5 gives an example of application of LM1 to a three-span bridge deck for the calculation of the general bending moment.

The lanes are numbered 1, 2, 3, etc. in such a way that the lane giving the most unfavourable effect is Lane No. 1, the lane giving the second most unfavourable effect is Lane No. 2, etc. In effect, the lane numbering increases as the total loading is less aggressive. This is represented diagrammatically in Fig. 4.6.

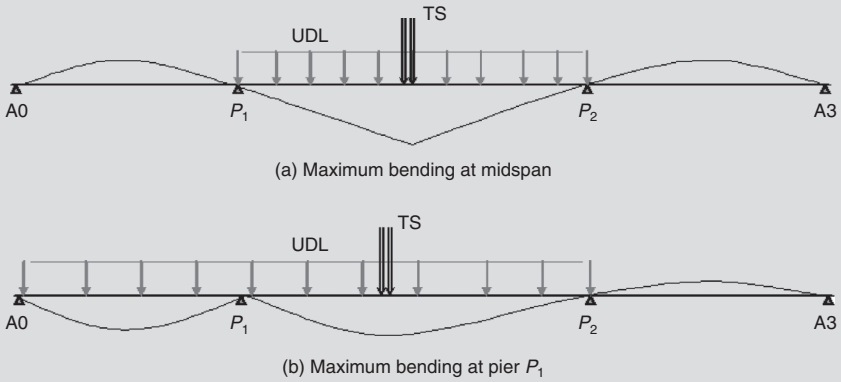


Fig. 4.5. LM1 arrangement to obtain the maximum bending moment in a three-span continuous bridge deck

Only complete tandem systems are taken into account, which means that it is not permitted to apply only one axle or only one wheel line: a tandem system is taken into account if its effects are globally unfavourable, and is not taken into account if its effects are globally favourable. *cl. 4.3.2(1)a: EN 1991-2*

For the assessment of general effects, the tandem systems are assumed to travel centrally along the axes of the relevant notional lanes.

The characteristic value of each axle load of a tandem system located in lane No. i is denoted $\alpha_{Q,i}Q_{ik}$, and the two wheels forming the axle transmit the same load $\alpha_{Q,i}Q_{ik}/2$. The characteristic value of the uniformly distributed load is noted $\alpha_{q,i}q_{ik}$ on lane No. i and $\alpha_{q,r}q_{rk}$ on the remaining area.

$\alpha_{Q,i}, \alpha_{q,i}, \alpha_{q,r}$ are adjustment factors intended to take into account the various types of traffic on bridges.

The uniformly distributed loads are to be applied only in the unfavourable parts of the influence surface, longitudinally and transversally. This means, for example in the transverse direction, that the uniformly distributed load may be applied on a width less than the normal width of a notional lane. *cl. 4.3.2(1)b: EN 1991-2*

For the application of LM1, the effective number of lanes to be loaded depends on the effect under consideration for which the most unfavourable value shall be determined, and

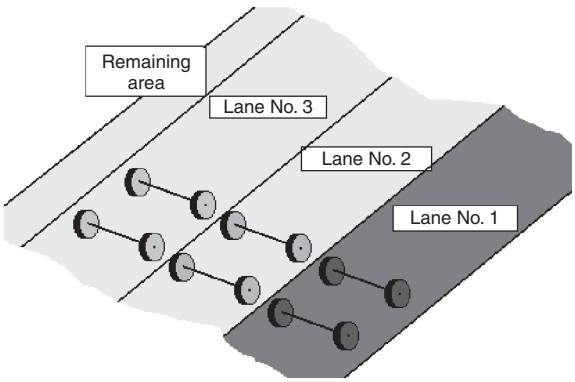


Fig. 4.6. Representation of LM1

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Table 4.2. Load Model 1: 'basic' characteristic values (Data taken from EN 1991-2, Table 4.2)

Location	Tandem system (TS) Axle loads, Q_{ik} (kN)	UDL system q_{ik} (or q_{rk}) (kN/m^2)
Lane No. 1	300	9
Lane No. 2	200	2.5
Lane No. 3	100	2.5
Other lanes	0	2.5
Remaining area (q_{rk})	0	2.5

therefore depends on the appropriate influence area. The lanes are not necessarily adjacent, even if in most cases they are.

LM1 was defined and calibrated in order to be usable for both general and local verifications. For general verifications, as mentioned earlier, the tandem systems travel centrally along the lanes, but for some local effects, two tandems belonging to two different lanes can be closer with a minimum distance of 0.50 m between the lines of two neighbouring wheels (see Fig. 4.4(c)).

The characteristic values of the loads (basic values) are given in Table 4.2, which reproduces *Table 4.2 of EN 1991-2*. They correspond to heavy long-distance international traffic and the dynamic effects are included.

The contact surface of wheels is a square of $0.40\,\text{m} \times 0.40\,\text{m}$. This requires some explanation. The UK National Annex to EN 1991-2, although using the recommended axle loads for the tandem system, does however change to UDL values.

Background information on the dimensions of contact surfaces of wheels

The basic value of the contact pressure of a wheel for the tandem system located on Lane No. 1 is $150/0.16 = 937.5\,\text{kN/m}^2$, which corresponds approximately to the dynamic pressure of a tyre on the road pavement (equal to the inflation pressure plus the structural reaction of the tyre). A detailed study of the local loads transmitted to the carriageway by heavy vehicle wheels was performed in 1989. The lorry tyres are mainly of radial framed type; their specificity is that their deformation is only longitudinal when crushing. The heavy load tyre is approximately square or rectangular with a constant transverse dimension, as shown in Fig. 4.7.

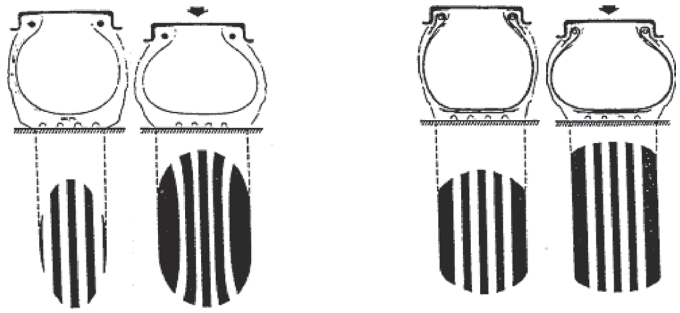


Fig. 4.7. Types of tyre

Physically, the contact area of wide tyres on the upper deck slab is calculated from a transverse dimension of 400 mm on average and for a dynamic situation from a longitudinal length slightly longer than the transverse dimension. The following formula gives a relationship between the wheel load Q (kN) and the average dynamic tyre pressure p (MN/m^2): it is assumed that the vehicle speed (60–80 kph) is such that the contact surface is slightly larger than $400 \times 400\,\text{mm}^2$.

$$p = \frac{Q}{220} - 0.07 \quad Q \leq 140\,\text{kN}$$

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The contact pressure is not always uniformly distributed over the contact area. For some specific scenarios such as hard braking, slipping, partial loss of contact of a wheel, or the beginning of a hydroplaning phenomenon, concentrations of pressure appear under some particular areas of the tyre and transmit in a more aggressive way the load to the deck slab (concrete or steel). For all these reasons, the wheel load of LM1 is rather pessimistic, but not unrealistic.

Adjustment of the characteristic values of LM1: background and recommendations

The selection of values for the various α_Q (for axle loads) and α_q (for distributed loads) adjustment factors by national authorities corresponds to the definition of route classes (traffic classes) or loading classes. Hereafter, some kind of guidance is proposed for the definition of such classes which are, of course, limited to the traffic whose effects are simulated by the main loading system (LM1) and the single-axle system (LM2).

Moreover, it should only refer to the element of traffic that produces most of the effects, i.e. produces effects akin to those produced by the characteristic loads. The properties of this element of traffic are not *a priori* the same as those that induce the main fatigue effects.

Road traffic is mainly characterized by the following parameters:

- its composition, for instance the percentage of lorries
- its density, for instance the average number of vehicles per year or the annual average of vehicle numbers per day
- its conditions, for instance traffic jam frequency
- the extreme loads of vehicles and of their axles
- and, if relevant, the influence of proposed road signs.

Each of these parameters may be quantified, but with some uncertainty; however, the greatest difficulty is to combine them in order to define the traffic classes.

A distinction is needed between uni- and bi-directional traffic. This distinction may be taken as known for an individual project, if any transient situation is controlled by the relevant authority.

The percentage of lorries (vehicles heavier than 3.5 t), taken as an annual average, varies between 10% and 25% for the majority of roads. Table 4.3 gives some information concerning the traffic scenarios used for the calibration of LM1 and LM2.

On main roads on which the traffic rate is high (for instance more than 2000 vehicles per day), variations in the percentage due to local effects are not anticipated during the working life of the bridge. However, this may not apply for roads with a low traffic rate. It has to be considered that the lorry percentage may vary significantly during the daytime, depending on the time of day.

Table 4.3. Basis for the calibration of load models LM1 and LM2

Road type (number of lanes for the records)	Lorry percentage (%)	Percentage related to the vehicle class (%)*				Average value of the lorry maximum load per day (kN)
		1	2	3	4	
Motorway (1 lane)	32	22.7	1.3	65.2	10.8	630
National road (1 lane)	17	26.7	2.5	59.9	10.9	490
Highway with long-distance traffic (1 lane)	32	14.4	6.4	66.9	12.3	570
Motorway (1 lane)	47	41.4	7.0	29.0	22.6	590
Motorway (1 lane)	43	16.6	1.6	40.2	41.6	650
Motorway (1 lane)	26	52.3	14.5	33.2	0.0	400

* Lorry classes are defined as follows: Class 1: single vehicle with two axles; Class 2: single vehicle with more than two axles; Class 3: articulated vehicle; Class 4: vehicle with a trailer

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Traffic jam frequency may be caused by a traffic rate exceeding the upper values of the ranges given in Table 4.3 (even if these values should not be considered as normal design assumptions) or by local situations that are independent of the bridge, for example traffic-lights or crossroads near the bridge.

Usually, except for specific situations (transient situations, controlled traffic, accidental situations) and in some urban areas, the frequency of simultaneous traffic jams in both directions is significantly smaller than for a single direction (10 to 100 times less). Traffic jam frequency should of course be taken into account for long-span bridges (it is not significant for small bridges or small members).

The expected frequency of traffic jams in one direction may thus be taken into account if some values of the α_q factors are fixed without alteration of the α_Q factors.

For bi-directional bridges, the small frequency of traffic jams in both directions is assumed to be taken into account in LM1 which considers one single notional lane No. 1.

The extreme loads of vehicles and axles cannot be easily identified for individual bridges, except for bridges located in areas where traffic conditions are very bad, for example on roads with a 15% (or more) slope.

Note 1 to cl. 4.3.2.3: EN 1991-2

It is for this reason that EN 1991-2 specifies that the factor α_{Q1} shall not be less than 0.8, and the value 0.9 was considered for small roads. It results from a combination of a low density and of a rather favourable distribution of the individual loads.

Nevertheless it seems legitimate to reconsider some extreme vehicle loads in some countries, on the basis of a comparison between the statistical data used for the calibration of LM1 and LM2 and national statistical data. The α_{Q1} factor (for which the extreme load may be the significant parameter), as well the α_{q1} factor and possibly also the α_{Q2} factor, should probably be revised according to the results of the comparison. The lorry maximum load is not directly related to the other parameters; for example, it is possible to have a low circulation density but with very heavy vehicles.

For the definition of traffic classes, a differentiation of the α_{q1} factor is particularly significant. For simplicity, it may be assumed that the choice of the α factors will lead to proportional effects acting on all the representative and design values, which means that in each country the values of the γ and ψ factors will be the same for all classes.

However, it is rational to assume that a country would prefer to modify only a few values of these factors because they may have a significant influence on the projects in that country. In such a case the content of the bridge parts of structural Eurocodes should be considered together with the traffic data.

Moreover, some groups of vehicles may be accidental in some countries, which means that such a situation will only be covered by the ultimate limit state verifications, with reduced safety factors. This could be an example of a socio-economic decision based on technical data, and not merely a technical decision. On the other hand, and because of the weak scatter of the maximum loads during a given time interval for a given traffic scenario, to retain the same fractiles may induce significant numerical consequences on the ψ factor values.

Example of adjustment of the characteristic values of LM1

In general, it will not be advantageous to define many loading classes. The most reasonable would be to define only two classes (Table 4.4):

- a class for road networks with international heavy traffic
- a class of all roads with a more or less 'normal' heavy traffic (even where the expected lorry traffic is rather light, the adoption of heavier loads than necessary – in the short term – gives a more comfortable safety margin and durability).

Table 4.4. Example of loading classes for road bridges

Classes	α_{Q1}	α_{Qi} $i \geq 2$	α_{q1}	α_{qi} $i \geq 2$	α_{qr}
1st class	1	1	1	1	1
2nd class	0.9	0.8	0.7	1	1

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The choice of a class of traffic implies that the expected traffic effects due to corresponding loads will not be exceeded at any time during the design working life of the bridge, considering the development of real traffic and its dynamic effects. For example, this choice may depend on the likelihood of one of the following scenarios occurring once during the design working life:

- 1st class: build-up of very heavy vehicles on the first lane of the bridge, depending on the composition of the expected traffic. This class should remain rather exceptional. It corresponds mainly to roads which have a very high proportion of heavy commercial vehicles (industrial, farm produce or forestry), and especially when international traffic represents a significant part of the total number of heavy vehicles along the itinerary (the number of circulating empty vehicles is therefore small). Attention is drawn to the fact that in the case of bridges with an individual span between 25 and 50 m, the effects of LM1 are very close to real effects, taking into consideration the increase in traffic weight over the last few decades.
- 2nd class: build-up of vehicles similar to those described above, but for common traffic composition on main roads and the highway network. It should be generally adopted for bridges with more than two lanes and at least a 6 m width carriageway, or with access roads to this type of carriageway. It is generally assumed that the uniformly distributed load on the residual area covers the effects of the supplementary traffic.

The UK National Annex to EN 1991-2 does not allow use of the α factors for LM1.

In short, the principles of application of LM1 for a given influence area are as follows:

- Positioning of the lanes, their numbering, and the loading areas, including remaining area, must be undertaken in a manner which gives the most unfavourable effect.
- For the calculation of this effect, the load on the remaining area must be considered totally free, in the longitudinal as well as in the transverse directions.

From a practical point of view (see examples in Section 4.10 below):

- often the tandems should be positioned first so that their total effects (without taking into account the uniform loads) will be most unfavourable
- the first lane can be defined in accordance with the location of the first tandem, and the corresponding uniformly distributed load should be applied on some parts of this lane to get the most unfavourable effects
- the other uniformly distributed loads will be applied on all parts of the deck, outside lane No. 1, where they have the most unfavourable effect; identical values for notional lanes for $i > 1$ and for the remaining area simplify the calculation of this effect.

Simplifications of LM1 — *cl. 4.3.2(6): EN 1991-2*

The following simplified load models may be used, if permitted by the National Annex.

Where general and local effects can be calculated separately, the general effects may be calculated by using the following simplified alternative rules:

- the second and third tandem systems are replaced by a second tandem system with axle weight equal to:

$$(200\alpha_{Q2} + 100\alpha_{Q3})\,\text{kN, or} \qquad \text{EN 1991-2, (4.5)}$$

- for span lengths greater than 10 m, each tandem system is replaced in each lane by a one-axle concentrated load of weight equal to the total weight of the two axles, i.e. $600\alpha_{Q1}$ kN on Lane No. 1, $400\alpha_{Q2}$ kN on Lane No. 2, $200\alpha_{Q3}$ kN on Lane No. 3.

The second simplified alternative rule (unique axles instead of tandems) may be used for preliminary calculation of internal efforts in a bridge deck in the longitudinal direction.

4.3.6. Load Model No. 2 (characteristic model)

cl. 4.3.3: EN 1991-2

The tandem systems of the main model do not cover all the local effects of vehicles of various types. Therefore, for some verifications concerning short structural members (in

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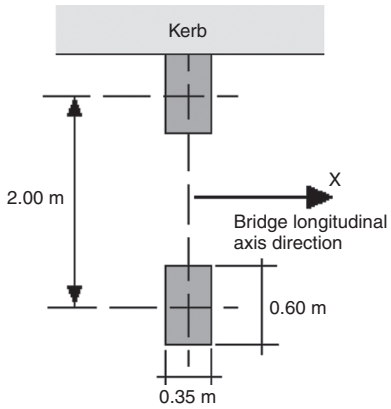


Fig. 4.8. Load Model 2 (LM2)

particular in the case of orthotropic slabs), load model LM1 is completed with an additional complementary load model (LM2) that allows to take into account other contact surfaces than the ones corresponding to wide tyres (in the case of twin wheels) and to correct the effects of LM 1 for short influence lines. It consists of a single axle corresponding to a basic characteristic load of 400 kN to which an adjustment factor β_Q, depending on the class of the expected traffic for an individual project, may be applied (Fig. 4.8). The load is equally distributed between the two wheels (equivalent contact pressure equal to 0.952β in MPa). In general, it is recommended to adopt a β_Q factor equal to α_{Q1} applicable to the heaviest tandem system of LM1; in particular it is equal to 1 for bridges corresponding to a higher class of loading.

cl. 4.3.4: EN 1991-2

4.3.7. Load Model No. 3 (special vehicles)

Annex A: EN 1991-2

Load model No. 3 is, in fact, a set of standardized vehicles intended to cover the effects of special convoys. These standardized vehicles are defined in ***Annex A (informative)*** to EN 1991-2: they are not intended to represent real vehicles, and for a national application it may be necessary to take into account specific heavy loads that cannot be covered by this annex. The standardized vehicles are defined in Tables 4.5 and 4.6, and Fig. 4.9; the vehicle characteristics are the result of a synthesis of permitted arrangements of actual national codes. Load model LM3 is, of course, taken in account only where specified by the client. Normally, the effects of the 600/150 standardized model are covered by the effects of LM1 where applied with α_{Qi} and α_{qi} factors all equal to 1. For convoys of total weight more than 3600 kN, specific rules need to be defined in the project specification or at the national level.

The Eurocode gives innovative rules concerning the simultaneous presence of special vehicles and normal traffic on a carriageway, and the dynamic effects depending on the permitted speed of the vehicles.

Concerning the dynamic effects, a dynamic amplification should be taken into account only where the vehicles are assumed to move at normal speed (about 70 kph). In that case, the dynamic amplification factor may be assessed from the following formula:

$$\varphi = 1.40 - \frac{L}{500}\varphi \geq 1$$

A.3(5): EN 1991-2

where L is the influence length (m).

Concerning the application of special vehicles on notional lanes and the simultaneity of LM1 and special vehicles, the proposed rules are represented in Figs 4.10 and 4.11, which are self-explanatory. As for LM1, the notional lanes should be located as unfavourably as possible in the carriageway. For this case, the carriageway width may be defined as excluding hard shoulders, hard strips and marker strips.

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Table 4.5. Description of special vehicles (Data taken from EN 1991-2, Table A1; see EN 1991-2 for missing values)

Total weight (kN)	Composition	Notation
600	4 axle-lines of 150 kN	600/150
900	6 axle-lines of 150 kN	
1200	8 axle-lines of 150 kN or 6 axle-lines of 200 kN	1200/150 1200/200
1500	10 axle-lines of 150 kN or 7 axle-lines of 200 kN + 1 axle-line of 100 kN	1500/150 1500/200
1800	12 axle-lines of 150 kN or 9 axle-lines of 200 kN	
2400	12 axle-lines of 200 kN or 10 axle-lines of 240 kN or 6 axle-lines of 200 kN (spacing 12 m) + 6 axle-lines of 200 kN	2400/200 2400/240 2400/200/200
3000	15 axle-lines of 200 kN or 12 axle-lines of 240 kN + 1 axle-line of 120 kN or 8 axle-lines of 200 kN (spacing 12 m) + 7 axle-lines of 200 kN	
3600	18 axle-lines of 200 kN or 15 axle-lines of 240 kN or 9 axle-lines of 200 kN (spacing 12 m) + 9 axle-lines of 200 kN	3600/200 3600/240 3600/200/200

4.3.8. Load Model No. 4 (crowd loading) (*EN 1991-2, 4.3.5*)

The load model No. 4 consists of a uniformly distributed load of $5\,kN/m^2$. This load represents the effect of a crowd, including uncorrelated dynamic amplification, and is applicable, where specified by the client, on the whole of the deck including the central reservation. This model is intended to be used for bridges constructed in urban areas

Table 4.6. Description of special vehicles (Data taken from EN 1991-2, Table A2; see EN 1991-2 for missing values)

Weight (kN)	Axle-lines of 150 kN	Axle-lines of 200 kN	Axle-lines of 240 kN
600	$n = 4 \times 150$ $e = 1.50\,m$	–	–
900	$n = 6 \times 150$ $e = 1.50\,m$	–	–
1200			–
1500	$n = 10 \times 150$ $e = 1.50\,m$	$n = 1 \times 100 + 7 \times 200$ $e = 1.50\,m$	–
1800	$n = 12 \times 150$ $e = 1.50\,m$	$n = 9 \times 200$ $e = 1.50\,m$	–
2400	–		
3000	–	$n = 15 \times 200$ $e = 1.50\,m$ $n = 8 \times 200 + 7 \times 200$ $e = 7 \times 1.5 + 12 + 6 \times 1.5$	$n = 1 \times 120 + 12 \times 240$ $e = 1.50\,m$
3600	–		

n: number of axles multiplied by the weight (kN) of each axle in each group
e: axle spacing (m) within and between each group.

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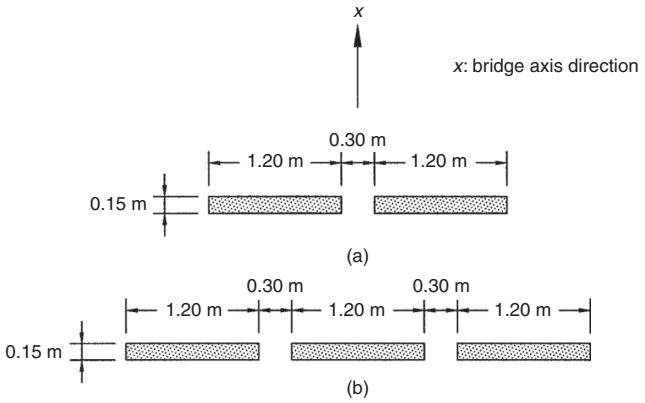


Fig. 4.9. Arrangement of axle-lines and definition of wheel contact areas for LM3: (a) 100–200 kN axle-lines; (b) 240 axle-lines (see EN 1991-2, Figure A.1)

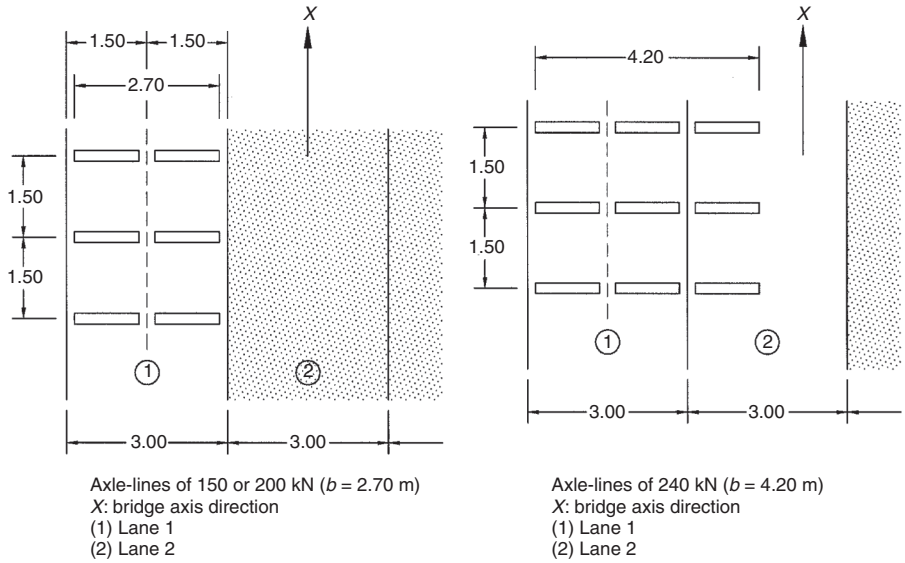


Fig. 4.10. Application of special vehicles on notional lanes for LM3 (see EN 1991-2, Figure A.2)

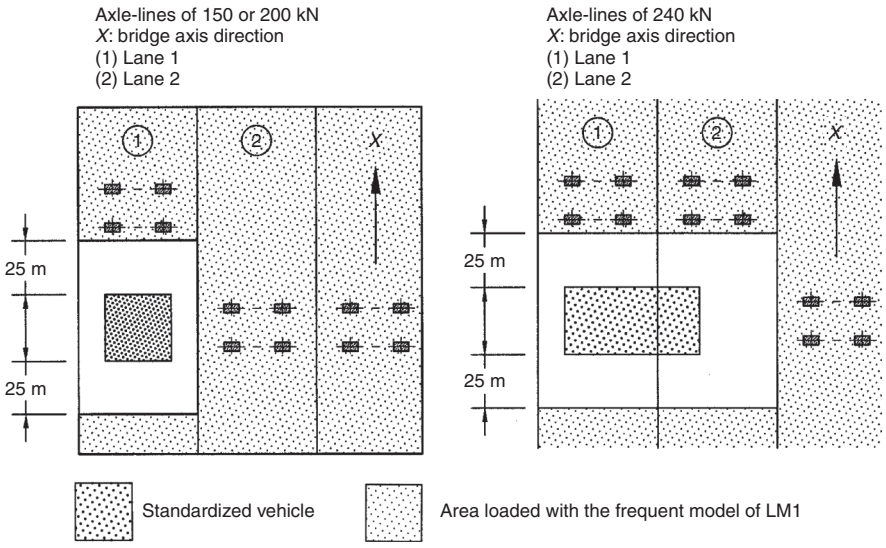


Fig. 4.11. Arrangement of axle-lines and definition of wheel contact areas LM3 (Reproduced from EN 1991-2, with permission from BSI)

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Fig. 4.12. Example of crowd loading on a bridge deck. New York Marathon, Verrazano Bridge (Copyright Martineric, Lille, France. Licensed for reuse under Creative Commons Attribution ShareAlike 2.0 Licence, http://creativecommons.org/licenses/by-sa/2.0/)

where sports or cultural events may take place (Fig. 4.12). The magnitude of $5\,kN/m^2$ has been defined according to existing national codes, but it corresponds to the physical maximum load from human beings (six or seven persons per square metre). See also Part 1 Chapter 6 of the TTL *Designers' Guide for Eurocode 1: Actions on Structures: Actions on buildings* in the part which refers to EN 1991-1-1.[2] This system is dominating only beyond some dimensions of the structure.

4.3.9. Dispersal of concentrated loads

cl. 4.3.6: EN 1991-2

The dispersal of concentrated loads (LM1 and LM2) has been purposely defined as simply as possible: it is taken, through the pavement as well as the concrete slab or the steel top plate, at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the middle plane of the slab or the steel plate. The pressure on the contact area is uniformly distributed. See Fig. 4.13.

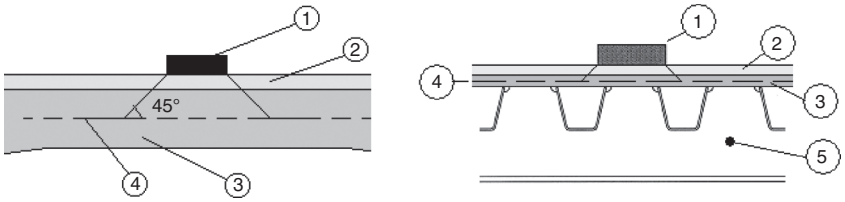


Fig. 4.13. Dispersal of concentrated loads: (a) Pavement and concrete slab; (b) Pavement and orthotropic deck (Reproduced from EN 1991-2, with permission from BSI)

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cl. 4.4: EN 1991-2

4.4. Horizontal forces

cl. 4.4.1: EN 1991-2

4.4.1. Braking or acceleration forces

The breaking or acceleration forces are represented by a longitudinal force, applied at the surfacing level of the carriageway, with a limited characteristic value of 900 kN, and it is calculated as a fraction of the total maximum vertical loads due to LM1 applied to lane No. 1 according to following expression:

$$Q_{1k} = 0.6\alpha_{Q1}(2Q_{1k}) + 0.10\alpha_{q1}q_{1k}w_1L$$

$$180\alpha_{Q1}\,(\text{kN}) \leq Q_{lk} \leq 900\,(\text{kN}) \qquad \text{EN 1991-2, (4.6)}$$

where

- L is the length of the deck or of the part of it under consideration
- $2Q_{1k}$ is the weight of the two axles of tandem system applied to lane No. 1 ($L > 1.2$ m – if not, a single axle weight is taken into account)
- q_{1k} is the density of the uniformly distributed load on lane No. 1
- w_1 is the width (3 m in normal cases) of lane No. 1
- α_{Q1} is the adjustment factor, depending on the loading class.

The magnitude of the braking and acceleration forces is represented diagrammatically in Fig. 4.14 for all adjustment factors equal to 1.

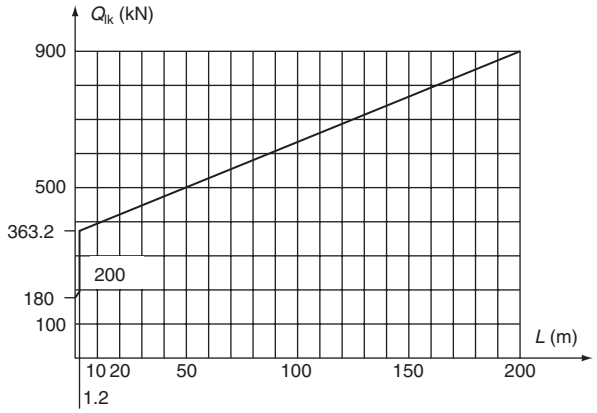


Fig. 4.14. Braking or acceleration force

Background documentation

This force intensity derives from studies using a simplified model based on the following assumptions, confirmed by tests carried out in Switzerland:

- A set of n identical lorries is considered with a uniform spacing, crossing the bridge in convoy with the same speed before the first vehicle brakes.
- The reaction time (the time between the braking of two consecutive lorries) is taken as the ratio of the distance between lorries over their initial speed (consequently the number of vehicles that brake simultaneously reaches a limit).
- The braking force of a lorry is proportional to its weight, with a factor that varies from 0.6 to 1 according to the type of lorry and its actual load.
- The dynamic lorry–bridge interaction is taken into account through the association of rheological models of springs, shock absorbers and friction elements in parallel.

Various simulations were carried out with various parameters and led to express the braking force as a function of the span length. The expression (4.6) in EN 1991-2 derives from these studies. The upper limit takes into account the braking force generated by military vehicles according to STANAG (military STANdardization AGreements – STANAG 2021).

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Table 4.7. Characteristic values of centrifugal forces (Data taken from EN 1991-2, Table 4.3; see EN 1991-2 for missing value)

$Q_{tk} = 0.2Q_v$ (kN)	if $r < 200$ m
$Q_{tk} =$ (kN)	if $200 \leq r \leq 1500$ m
$Q_{tk} = 0$	if $r > 1500$ m

4.4.2. Centrifugal force

cl. 4.4.2: EN 1991-2

EN 1991-2 defines the characteristic value of a transverse force, noted Q_{tk}, applicable at the finished carriageway level in a direction perpendicular to its axis, as given in Table 4.7.

where

r is the horizontal radius of the carriageway centreline (m)
Q_v is the total maximum weight of vertical concentrated loads of the tandem systems of LM1, i.e. $\sum_i \alpha_{Qi}(2Q_{ik})$

These formulae derive from the equation:

$$Q_t = \frac{V^2}{gr} Q_v$$

where

V is the vehicle speed (in m/s)
Q_v is the corresponding vertical force

$g = 9.81\ \text{m/s}^2$.

The value of Q_{tk} corresponds to a speed of around 70 kph. This speed has been chosen because the centrifugal force is mainly due to heavy vehicles. Individual cars do not give rise to significant centrifugal effects.

4.5. Groups of traffic loads on road bridges

cl. 4.5: EN 1991-2

As already mentioned in Section 4.1 above, the concept of a group of traffic loads has been defined in EN 1991-2 to facilitate the combinations of actions (see Chapter 8 of this Designers' Guide). A group of traffic loads is, in fact, something like a 'sub-combination' defining a 'global' traffic action for combination of non-traffic loads. The groups of loads are mutually exclusive and are used as 'global' variable actions in combinations of actions.

In EN 1991-2, the characteristic groups of traffic loads are defined in Table 4.4(a) and the frequent groups of traffic loads are defined in Table 4.4(b).

The characteristic groups of loads are explained in Fig. 4.15.

For the frequent groups of loads, see Table 4.8 of this Designers' Guide which is reproduced from Table 4.4(b) of EN 1991-2.

Attention is drawn to the fact that a frequent value is defined for the loads on footways and cycle tracks (gr3): the frequent value may be useful for the verification of some serviceability criteria, in particular for concrete members. However, no frequent value is foreseen for gr4 (crowd loading) and gr5 (special vehicles).

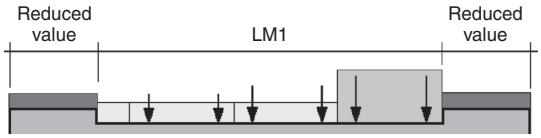
4.6. Models of vertical loads for fatigue verification

cl. 4.6: EN 1991-2

EN 1991-2 defines five load models for fatigue verification denoted FLM1 to FLM5. These models correspond, in principle, to various uses, in so far as it was decided, from inception, that the Eurocode should give:

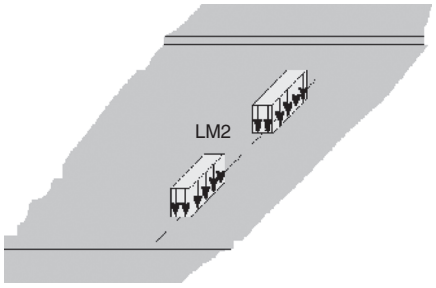
- one or more rather 'pessimistic' load models to quickly identify in which parts of the structure a problem of fatigue could appear

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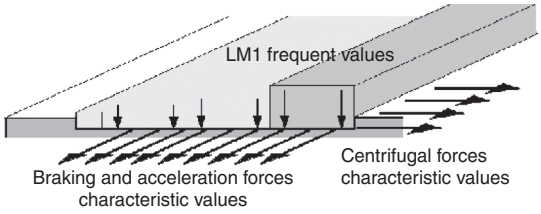


Group of loads gr1a
The carriageway is loaded with LM1 (characteristic values), the footways are loaded with a 'combination' or 'reduced' value. This value is determined by national choice, but the recommended value is 3.0 kN/m^2.

In practice, group of loads gr1a is the most important for general structural analysis of bridge decks and the verification of local effects.



Group of loads gr1b
This group includes only LM2 taken with its characteristic value.



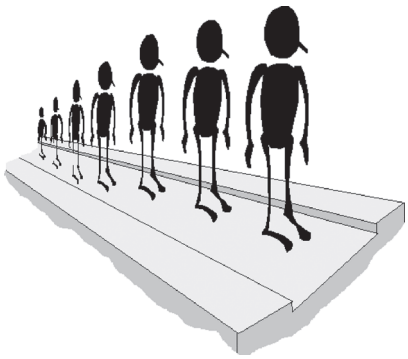
Group of loads gr2
This group is based on the characteristic values of horizontal forces due to braking/acceleration and due to centrifugal effects (in case of curved bridge). Vertical forces due to LM1, taken with the frequent values, are applied simultaneously with horizontal forces. It has to be noted that forces due to breaking (or acceleration) and centrifugal effects, which are independent variable actions, are simultaneously taken with their characteristic values only for simplicity for designers.



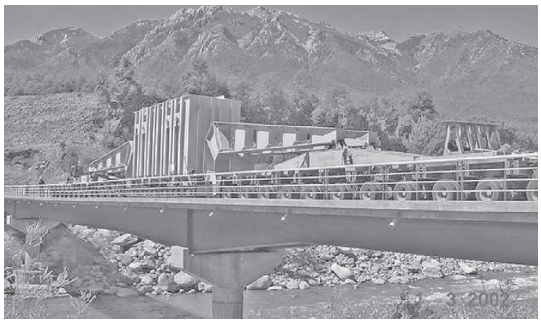
Group of loads gr3
This group includes only the vertical load (characteristic value) due to pedestrians or cyclists on footways or cycle tracks. The Eurocode specifies that one footway only should be loaded if the effect is more unfavourable than the effect of two loaded footways.

It is intended for the verification of the relevant structural members supporting footways and cycle tracks.

This group is not relevant if gr4 (see below) is taken into account.



Group of loads gr4
This group of loads corresponds to the loading of the bridge (carriageway + footways) by a crowd. It has to be taken into account when required by the client or the relevant authority.



Group of loads gr5
This group of loads is based on the consideration of special (abnormal) vehicles. The condition of taking account of these special vehicles on bridge decks, and particularly their simultaneity with normal road traffic, are defined at the national level (see 4.3.7 of this Designers' Guide).

Fig. 4.15. Description of groups of loads for road bridges

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Table 4.8. Assessment of groups of traffic loads (frequent values of multi-component action) (Data taken from EN 1991-2, Table 4.4-b)

		Carriageway		Footways and cycle tracks
Load type			Vertical forces	
EN 1991-2 reference		4.3.2	4.3.3	5.3.2(1)
Load system		LM1 (TS and UDL systems)	LM2 (single axle)	Uniformly distributed load
Load group	gr1a	Frequent values		
	gr1b		Frequent values	
	gr3			Frequent value[(a)]

[(a)] See 5.3.2.1(3). One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.

- one or more models to perform usual simple verifications
- one or more models to perform accurate verifications (based on a damage calculation).

Background documentation on the calibration of some fatigue load models can be found in the annex to this chapter.

cl. 4.6.2 and 4.6.3: EN 1991-2

4.6.1. FLM1 and FLM 2

cl. 4.6.2: EN 1991-2

FLM1 derives from LM1 with only 70% of the characteristic values of axle loads and 30% of the characteristic values of uniformly distributed loads. The α-factors are not applicable to this model. It is intended to be used to determine a maximum and a minimum stress for an individual verification (Table 4.9).

As mentioned in EN 1991-2, the load values for FLM1 are similar to the frequent values of Load Model LM1. However, adopting the frequent LM1 without adjustment would have been excessively conservative by comparison with the other models, especially for large loaded areas. Nevertheless, as it is defined, FLM1 is very conservative.

Fatigue Load Model No. 2 consists of a set of five lorries, denoted 'frequent lorries', the geometrical and weight characteristics of which are given in Tables 4.10 and 4.11.

FLM 2 is intended to be used for the determination of the maximum and minimum stresses that result from one of these lorries travelling on the slow lane of the bridge under consideration.

At the ENV stage of the Eurocodes, FLM1 and FLM2 were both intended to be used to check whether the fatigue lifetime of steel bridges might be considered as unlimited by reference to S–N curves that have a constant amplitude fatigue limit. In fact, only the S–N curves defined in EN 1993 Part 1.9: Fatigue have such a limit (Fig. 4.16) corresponding to 5.10^6 cycles.

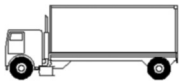
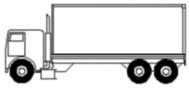
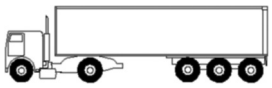
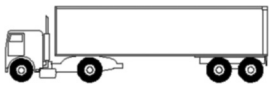
Thus, if the stress range resulting from a single application of FLM1 and/or FLM2 is less than the point of the S–N curves of abscissa $N = 5.10^6$, it is then assumed that no fatigue ultimate limit state may be reached for the detail under consideration. As a consequence,

Table 4.9. Fatigue Load Model No. 1

Location	Tandem system (TS) Axle loads $0.7Q_{ik}$ (kN)	UDL system $0.3q_{ik}$ (or $0.3q_{rk}$) (kN/m^2)
Lane No. 1	210	2.70
Lane No. 2	140	0.75
Lane No. 3	70	0.75
Other lanes	0	0.75
Remaining area (q_{rk})	0	0.75

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Table 4.10. Definition of frequent lorries (Data taken from EN 1991-2, Table 4.6; see EN 1991-2 for missing values)

1 Lorry silhouette	2 Axle spacing (m)	3 Frequent axle loads (kN)	4 Wheel type (see Table 4.11)
	4.5	90 190	A B
			
	3.20 5.20 1.30 1.30	90 180 120 120 120	A B C C C
			
	4.80 3.60 4.40 1.30	90 180 120 110 110	A B C C C

these two models have been calibrated with enough pessimism, so that their effects realistically match the effects of actual traffic.

FLM2 is intended to correct possible defects resulting from the use of FLM1 in the case of short influence lines. 'Frequent' lorries are normally calibrated to cover 99% of the damages due to free flowing traffic, such as the one recorded near Auxerre (France) for the calibration of LM1.

Attention is drawn to the following points:

- Only S–N curves related to frame steels have a constant amplitude fatigue limit; as a consequence, Fatigue Load Models 1 and 2 should not be used, for example for concrete bridges.
- Calibration tests did not precisely show whenever each model had to be used, considering that FLM1 may be used for large loaded surfaces.
- When using a constant amplitude fatigue limit, obscure discontinuities may occur in the design of the fatigue lifetime issued from the Eurocodes for similar structures.

For all the above reasons FLM1 and FLM2 should not be considered the models for the most common verifications.

cl. 4.6.4: EN 1991-2

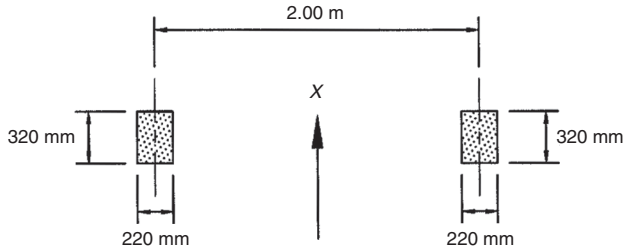
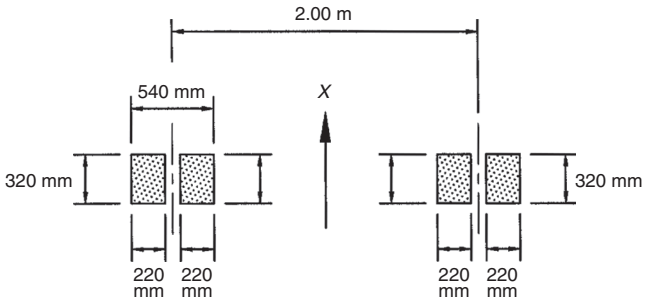
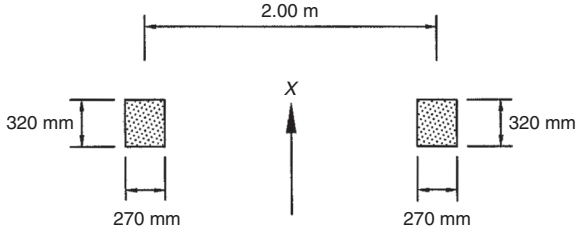
4.6.2. Description of Fatigue Load Model No. 3 (FLM3)

The main fatigue model is FLM3 (Fig. 4.17), which is intended for common verifications, without performing any damage calculation. It consists of four axles of 120 kN, each axle having two wheels with square contact areas of $0.40 \times 0.40\,m^2$.

For the definition of this model, the basic idea was originally to select a fatigue 'single vehicle' so that, assuming a conventional number of crossings of the bridge by this vehicle (e.g. 2.10^6), and after a numerical adaptation with appropriate factors, it led to the same damage as the real traffic during the intended lifetime of the bridge.

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Table 4.11. Definition of wheels and axles for FLM2 and FLM4 (Data taken from EN 1991-2, Table 4.8)

Wheel/axle type	Geometrical definition
A	
B	
C	

Thus, the designer calculates the extreme stresses (maximum and minimum) resulting from the crossing of the bridge by FLM3 in order to evaluate a stress range:

$$\Delta\sigma_{\mathrm{FLM}} = |\max \sigma_{\mathrm{FLM}} - \min \sigma_{\mathrm{FLM}}|$$

This stress range is then multiplied by a dynamic amplification factor φ_{fat} taking account of the carriageway roughness and a load factor λ_{e}, which gives an 'equivalent' stress range:

$$\Delta\sigma_{\mathrm{fat}} = \lambda_{\mathrm{e}}\varphi_{\mathrm{fat}}\Delta\sigma_{\mathrm{FLM}}$$

This stress range $\Delta\sigma_{\mathrm{fat}}$ is compared with the value $\Delta\sigma_{\mathrm{c}}$ of the S–N curve, corresponding to 2.10^6 applications (Fig. 4.18).

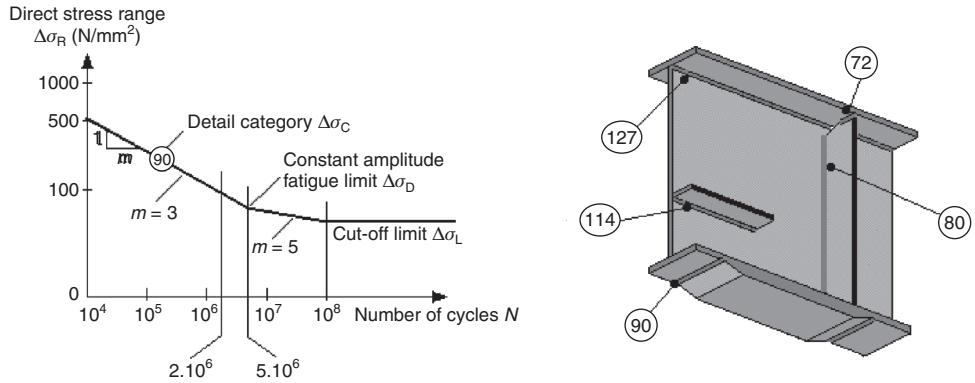


Fig. 4.16. Example of S–N curves related to normal stress

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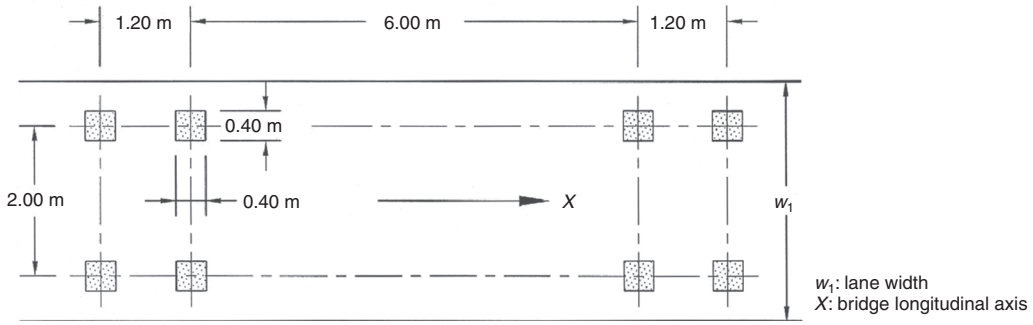


Fig. 4.17. Definition of FLM3 (See EN 1991-2, Figure 4.8)

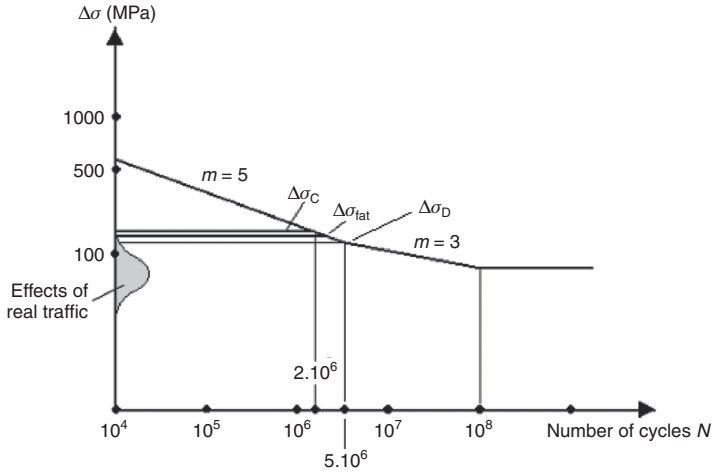


Fig. 4.18. Principle of fatigue verification with FLM3

The factor λ_e is obtained by multiplying four factors:

$$\lambda_e = \lambda_1\lambda_2\lambda_3\lambda_4$$

where

- λ_1 takes account of the damaging effect of traffic and depends on the length (span) of the influence line or surface
- λ_2 takes account of the expected annual traffic volume
- λ_3 is a function of the design working life of the bridge ($\lambda_3 = 1$ for 100 years)
- λ_4 takes account of multi-lane effects.

For the assessment of the expected annual traffic volume (factor λ_2), EN 1991-2 gives indicative numbers of heavy vehicles expected per year and per slow lane. These numbers are shown in Table 4.12 which is reproduced from Table 4.5(n) of EN 1991-2.

Table 4.12. Indicative number of heavy vehicles expected per year and per slow lane (Data taken from Table 4.5(n) of EN 1991-2; see EN 1991-2 for missing values)

Traffic categories		N_{obs} per year and per slow lane
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	2×10^6
2	Roads and motorways with medium flow rates of lorries	
3	Main roads with low flow rates of lorries	0.125×10^6
4	Local roads with low flow rates of lorries	

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Table 4.13. Indicative correspondence of notation between EN 1992-2 and EN 1993-2

Notation in this Designers' Guide	Notation in EN 1992-2	Notation in EN 1993-2
Stress range: $\Delta\sigma_{FLM} = \lvert\max \sigma_{FLM} - \min \sigma_{FLM}\rvert$	$\Delta\sigma_{s,Ecu}$	$\Delta\sigma_p = \lvert\sigma_{p,max} - \sigma_{p,min}\rvert$
'Equivalent' stress range: $\Delta\sigma_{fat} = \lambda_e \varphi_{fat} \Delta\sigma_{FLM}$	$\Delta\sigma_{s,equ} = \lambda_s \Delta\sigma_{s,EC}$	$\Delta\sigma_{E2} = \lambda\Phi_2\Delta\sigma_p$
$\lambda_e = \lambda_1\lambda_2\lambda_3\lambda_4$	$\lambda_s = \varphi_{fat}\lambda_{s,1}\lambda_{s,2}\lambda_{s,3}\lambda_{s,4}$	$\lambda = \lambda_1\lambda_2\lambda_3\lambda_4$

cl. 4.6.1(3): EN 1991-2

In this table, the traffic category for fatigue verifications is defined by:

- the number of slow lanes
- the number N_{obs} of heavy vehicles (maximum gross vehicle weight more than 100 kN) observed or estimated, per year and per slow lane.

On each fast lane, additionally, 10% of N_{obs} may be taken into account. *Note 1 to cl. 4.6.1(3): EN 1991-2*

The notation in the various Eurocodes is not equivalent, but the verification process is analogous. For illustration, Table 4.13 gives the correspondence between notation in Parts 2 of EN 1992 (concrete bridges) and EN 1993 (steel bridges).

For the assessment of action effects:

- the fatigue load models are positioned centrally on the appropriate notional lanes defined in the project specification for general effects *cl. 4.6.1(4): EN 1991-2*
- the fatigue load models are positioned centrally on the notional lanes assumed to be located anywhere on the carriageway and, moreover, for example for orthotropic decks, a statistical distribution of the transverse location of the vehicles within the notional lanes may be taken into account (Fig. 4.19). *cl. 4.6.1(5): EN 1991-2*

Fatigue Load Models (FLM1 to 4) include dynamic load amplification appropriate for pavements of good quality. It is recommended to apply to all loads an additional amplification factor $\Delta\varphi_{fat}$ near expansion joints, given by the following formula and represented in Fig. 4.20: *Annex B: EN 1991-2*

$$\Delta\varphi_{fat} = 1.30\left(1 - \frac{D}{26}\right)\Delta\varphi_{fat} \geq 1$$

where D is the distance (m) of the cross-section under consideration from the expansion joint.

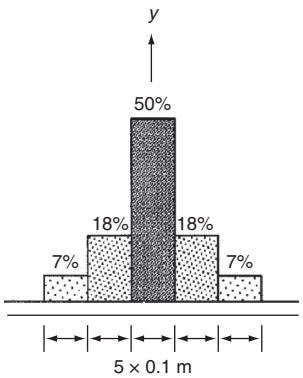


Fig. 4.19. Frequency distribution of transverse location of centre line of vehicle (See EN 1991-2, Figure 4.6)

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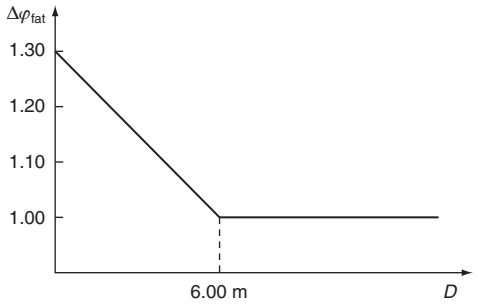


Fig. 4.20. Representation of the additional amplification factor (See EN 1991-2, Figure 4.7)

cl. 4.6.4 and 4.6.5: EN 1991-2

4.6.3. Description of Fatigue Load Models 4 and 5

Fatigue Load Models 4 and 5 are intended to be used for accurate verifications based on damage calculations using Palmgren-Miner's law. FLM 4 consists of a set of five lorries (called 'equivalent lorries') from which it is possible to simulate artificial traffic (by using probabilistic methods and by adjusting the proportion of each one in the global traffic). FLM5 is based on the direct use of recorded traffic. Table 4.14, reproduced from Table 4.7 of EN 1991-2, shows the set of equivalent lorries.

Note 3 to Table 4.7: EN 1991-2

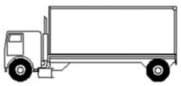
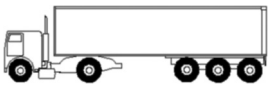
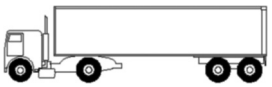
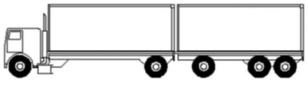
The wheel types are those defined in Table 4.11 above.

Note 3 to Table 4.7 of EN 1991-2 and hence this table gives the following information:

- 'long distance' means hundreds of kilometres
- 'medium distance' means 50–100 km
- 'local traffic' means distances less than 50 km

but in reality a mix of traffic types may occur.

Table 4.14. Set of equivalent lorries for FLM4 (Data taken from EN 1991-2 Table 4.7; see EN 1991-2 for missing values)

Vehicle type			Traffic type			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
Lorry	Axle spacing (m)	Equivalent axle loads (kN)	Lorry percentage	Lorry percentage	Lorry percentage	Wheel type
	4.5	70 130	20.0	40.0	80.0	A B
						
	3.20 5.20 1.30 1.30	70 150 90 90 90	50.0	30.0	5.0	A B C C C
						
	4.80 3.60 4.40 1.30	70 130 90 80 80	10.0	5.0	5.0	A B C C C

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4.6.4. Conditions of use of recorded traffic

The assessment of fatigue life based on recorded traffic needs specific application rules. Some of these rules are given in Informative Annex B to EN 1991-2.

The starting-point of the method is the determination of a stress history; in so far as the data are generally collected on the lanes of a highway or a motorway, it is necessary to apply to the data a dynamic amplification factor φ_{fat} taking into account the dynamic behaviour of the bridge and the effects of the expected roughness of the road surface. On the other hand, the records include an unavoidable dynamic magnification which has been roughly estimated equal to 10% (see the annex to this chapter).

For a more accurate approach, the Eurocode mentions the method given in ISO 8608[3] in which the road surface can be classified in terms of the power spectral density (PSD) of the vertical road profile displacement G_d, i.e. of the roughness. For a rough and quick estimation of the roughness quality, the following guidance is given:

- New roadway layers, such as, for example, asphalt or concrete layers, can be assumed to have a good or even a very good roughness quality.
- Old roadway layers which are not maintained may be classified as having a medium roughness.
- Roadway layers consisting of cobblestones or similar material may be classified as medium ('average') or bad ('poor', 'very poor').

In most common cases, it is possible to adopt the following values of φ_{fat}:

$\varphi_{fat} = 1.2$ for surfaces of good roughness
$\varphi_{fat} = 1.4$ for surfaces of medium roughness.

This dynamic amplification factor is independent of the local dynamic factor introduced in Section 4.6.2 and Fig. 4.19 above: the two factors apply when considering a cross-section within a distance of 6.00 m from an expansion joint.

If the data are recorded on one lane only, assumptions should be made concerning the traffic on other lanes. These assumptions may be based on records made at other locations for a similar type of traffic. The stress history should take into account the simultaneous presence of vehicles recorded on the bridge in any lane. A procedure should be developed to allow for this when records of individual vehicle loadings are used as a basis.

The numbers of cycles should be counted using the rainflow method or the reservoir method (Fig. 4.21).

If the duration of recordings is less than a full week, the records and the assessment of the fatigue damage rates may be adjusted taking into account observed variations of traffic flows and mixes during a typical week. An adjustment factor should also be applied to take into account any future changes of the traffic.

The cumulative fatigue damage calculated by use of records should be multiplied by the ratio between the design working life and the duration considered in the histogram. In the absence of detailed information, a factor of 2 for the number of lorries and a factor 1.4 for the load levels are recommended.

4.7. Actions for accidental design situations

cl. 4.7: EN 1991-2

This clause deals with:

- vehicle collision with bridge piers, soffit of bridge or decks
- the presence of heavy wheels or vehicle on footways
- vehicle collision with kerbs, vehicle parapets and structural components.

For collision forces from vehicles under the bridge, covering impact forces on piers and other supporting members, and impact on decks (Fig. 4.22), EN 1991-2 gives only recommendations or recommended values. This is due to the fact that EN 1991-2 was developed before EN 1991-1-7 (Accidental actions). Therefore, the questions related to impact from vehicles

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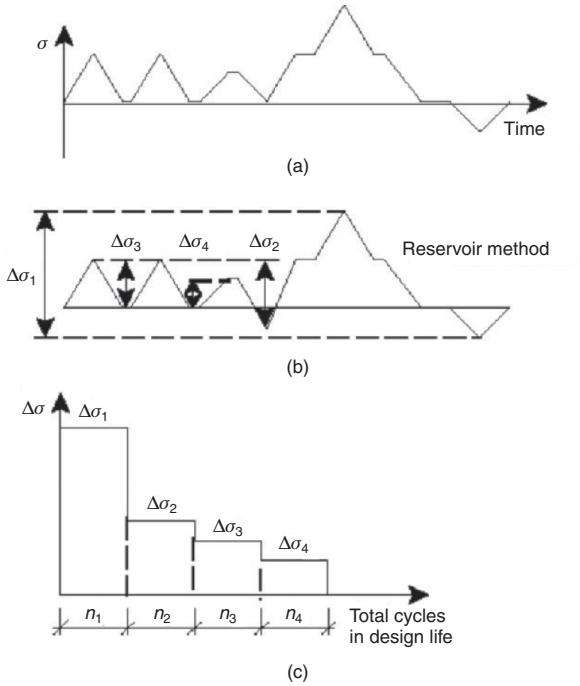


Fig. 4.21. Counting method of stress cycles: (a) Stress history at detail; (b) Cycle counting; (c) Stress-range spectrum

under the bridge are treated in Chapter 7 of this Designers' Guide. Hereafter actions from vehicles on the bridge are only evoked.

4.7.1. Vehicle on footways and cycle tracks on road bridges

The presence of heavy wheels or vehicles on footways is an accidental design situation and needs to be taken into account for all bridges where footways are not protected by a rigid road restraint system.

cl. 4.7.3.1(2): EN 1991-2

The accidental action is due to one axle load from the Tandem System corresponding to notional lane No. 2, i.e. $\alpha_{Q2}Q_{2k} = 200\alpha_{Q2}$ (see Section 4.3.5 of this Designers' Guide), to be applied and oriented on the unprotected parts of the deck so as to give the most adverse



Fig. 4.22. Example of impact on a bridge deck

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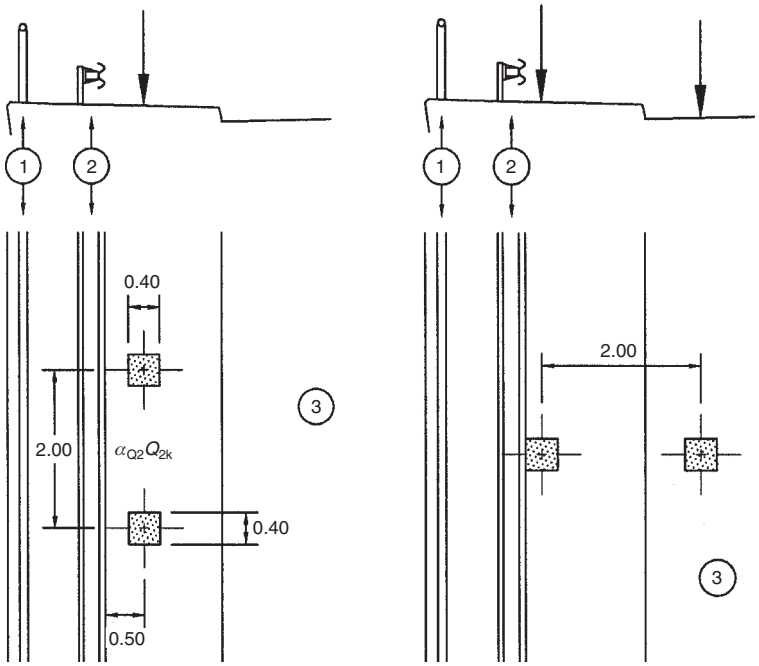


Fig. 4.23. Examples showing locations of loads from vehicles on footways and cycle tracks of road bridges (EN 1991-2, Figure 4.9)

effect. The design situations to be taken into account are defined by the designer in agreement with the client. Figure 4.23, that derives from ***Fig. 4.9 of EN 1991-2***, shows two examples of accidental design situations. *Fig. 4.9: EN 1991-2*

cl. 4.7.3.2: EN 1991-2

4.7.2. Collision forces on kerbs

The collision force is a horizontal force of 100 kN, perpendicular to the kerb and acting on a line 0.5 m long at a depth of 0.05 m below the top of the kerb. Where unfavourable, a vertical traffic load may be taken into account simultaneously, equal to $0.75\alpha_{Q1}Q_{1k} = 225\alpha_{Q1}$ kN. These forces are represented in Fig. 4.24 which derives from ***Fig. 4.10 of EN 1991-2***. *Fig. 4.10: EN 1991-2*

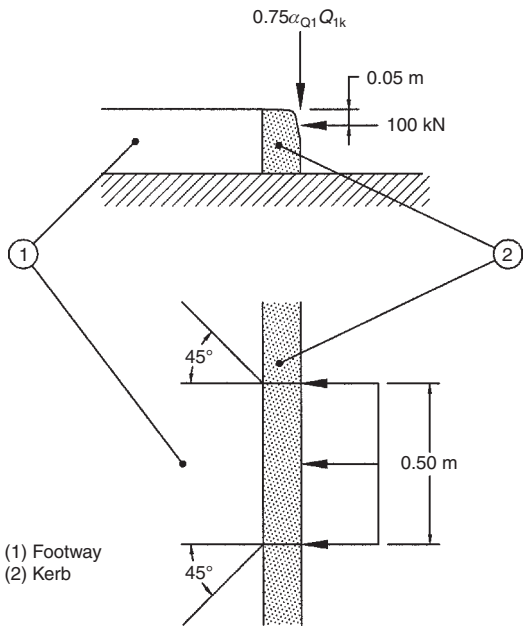


Fig. 4.24. Definition of vehicle collision forces on kerbs (EN 1991-2, Figure 4.10)

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Table 4.15. Recommended classes for the horizontal force transferred by vehicle restraint systems (Data taken from EN 1991-2, Table 4.9(n))

Recommended class	Horizontal force (kN)
A	100
B	200
C	400
D	600

The vehicle collision forces on kerbs have been introduced in the Eurocode to give a rule for the design of structural members supporting kerbs. And in rigid (concrete) members the angle of dispersal of the load may be taken equal to 45° as shown in Fig. 4.24.

cl. 4.7.3.3: EN 1991-2

4.7.3. Collision forces on vehicle restraint systems

For the detailed design of a bridge, precise rules have to be defined concerning the connection between the road restraint system and the relevant structural member of the bridge. However, in fact, in the British standard BS EN 1317, only performance classes are defined in its Part 2, and the performance is only defined by the containment level.

For the design of the connection, the Eurocode recommends four classes of values for the transferred horizontal force defined in Table 4.15. Of course, these recommended values may be replaced by more refined values in the National Annex, depending on test results obtained with commercial systems or devices.

These values globally cover the results of measurements during collision tests on real vehicle restraint systems used for bridges. The Eurocode mentions that there is no direct correlation between these values and the performance classes of vehicle restraint systems. The proposed values depend rather on the stiffness of the connection between the vehicle restraint system and the relevant structural member of the deck. Class D corresponds to a very strong connection, for example in the case of rigid steel road restraint systems. For the containment of heavy vehicles, the normal performance class of road restraint systems is performance class H. The most common performance classes are H2 and H3. Class C for the horizontal force may be associated with these performance classes. In that case, EN 1991-2 recommends applying the horizontal force, acting transversely, 100 mm below the top of the selected vehicle restraint system or 1.0 m above the level of the carriageway or footway, whichever is the lower, and on a line 0.5 m long. The recommended value of the vertical force acting simultaneously with the horizontal force is equal to $0.75\alpha_{Q1}Q_{1k}$ (see Fig. 4.25).

Of course, it is desirable to prevent deterioration of the structure in case of impact of a heavy vehicle on a vehicle parapet. For this reason, the Eurocode recommends designing the structure supporting the vehicle parapet to sustain locally an accidental load effect corresponding to at least 1.25 times the characteristic local resistance of the vehicle parapet (e.g. resistance of the connection of the parapet to the structure) without combination with any other variable load. More accurate values may be given in national annexes, based on real tests.

cl. 4.7.3.3(2): EN 1991-2

cl. 4.7.3.4: EN 1991-2

4.7.4. Collision forces on structural members

Of course, the vehicle collision forces on unprotected structural members above or beside the carriageway levels need to be taken into account; this is the case, for example, for bridges with lateral lattice girders (Fig. 4.26). The Eurocode recommends taking into account the same impact force as for piers, acting 1.25 m above the carriageway level. However, when additional protective measures between the carriageway and these members are provided, this force may be reduced for the individual project.

This force is an accidental action and, of course, should not be combined with any other variable load for the verifications (Fig. 4.27).

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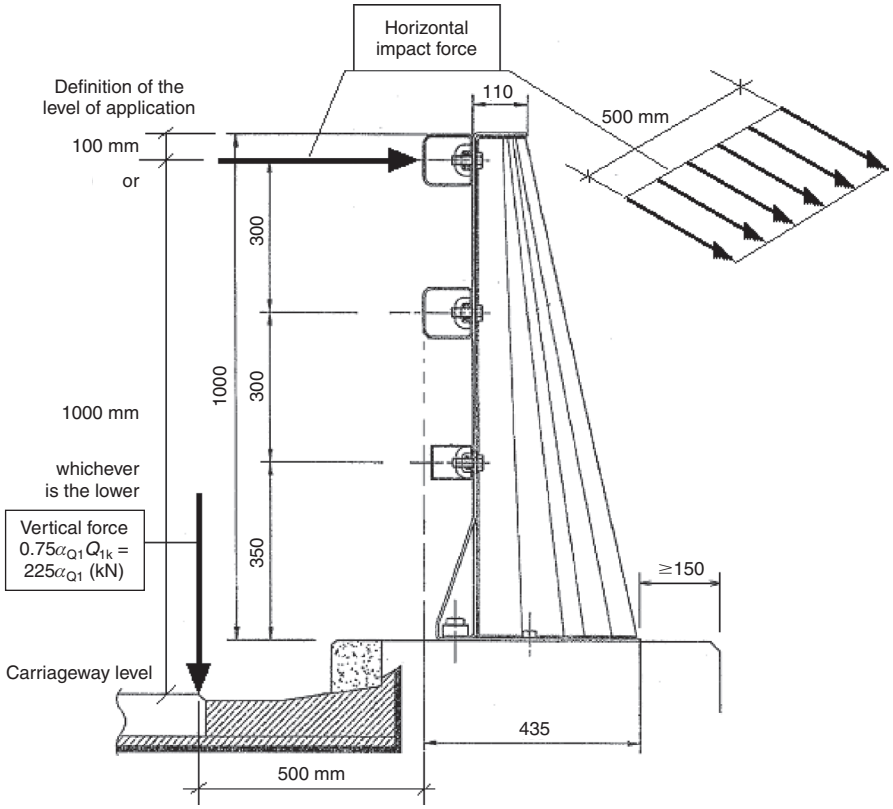


Fig. 4.25. Representation of the design forces to be applied to a vehicle parapet for heavy vehicles



Fig. 4.26. Example of bridge with protection of lateral girders

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Fig. 4.27. Example of accidental situation on a suspension bridge

cl. 4.8: EN 1991-2

4.8. Actions on pedestrian parapets

The European standard prEN 1317 Part 6[4] specifies geometrical and technical requirements and defines the requirements for design and manufacturing of pedestrian parapets on bridges with footways and/or cycle tracks. This standard defines traffic loads, acting in horizontal and vertical directions. The horizontal traffic actions as well as the vertical traffic actions comprise uniformly distributed loads and point loads. Concerning the horizontal uniformly distributed load, the European standard defines nine loading classes, the magnitude of the load being in the range $q_h = 0.4\,\text{kN/m}$ (class A) to $q_h = 3\,\text{kN/m}$ (class J).

EN 1991-2 recommends class C ($q_h = 1\,\text{kN/m}$) as the minimum class. The same minimum value is recommended for the vertical uniformly distributed load. For service side paths, the recommended minimum value is 0.8 kN/m, but exceptional and accidental cases are not covered by these recommended minimum values.

For the design of the supporting structure, the vertical action is normally not relevant. If pedestrian parapets are adequately protected against vehicle collision, the horizontal action on the parapet rail is taken into account simultaneously with the characteristic value of the uniformly distributed load on the footway or cycle track or footbridge (see Chapter 5 of this Designers' Guide). However, where pedestrian parapets cannot be considered as adequately protected against vehicle collisions, the Eurocode recommends designing the supporting structure in order to sustain an accidental load effect corresponding to 1.25 times the characteristic resistance of the parapet, exclusive of any other variable load.

cl. 4.8(3): EN 1991-2

cl. 4.9: EN 1991-2

4.9. Load models for abutments and walls adjacent to bridges

4.9.1. Vertical loads

EN 1991-2 recommends the application of LM1 on the carriageway located behind abutments for the design of wing walls, side walls and other parts of the bridge in contact with earth, but, for simplicity, the tandem system loads may be replaced by an equivalent uniformly distributed load, denoted q_{eq}, spread over a rectangular surface 3 m wide and 2.20 m long if, for a properly consolidated backfill, the dispersal angle from the vertical is taken equal to 30°.

It should be noted that the characteristic values of LM1 for the assessment of traffic action effects on bridges include a dynamic amplification which is not normally relevant for roads. Therefore, the characteristic values of LM1 may be multiplied by a reduction factor. Taking into account the values mentioned in the annex to this chapter, a factor of 0.7 may be commonly adopted.

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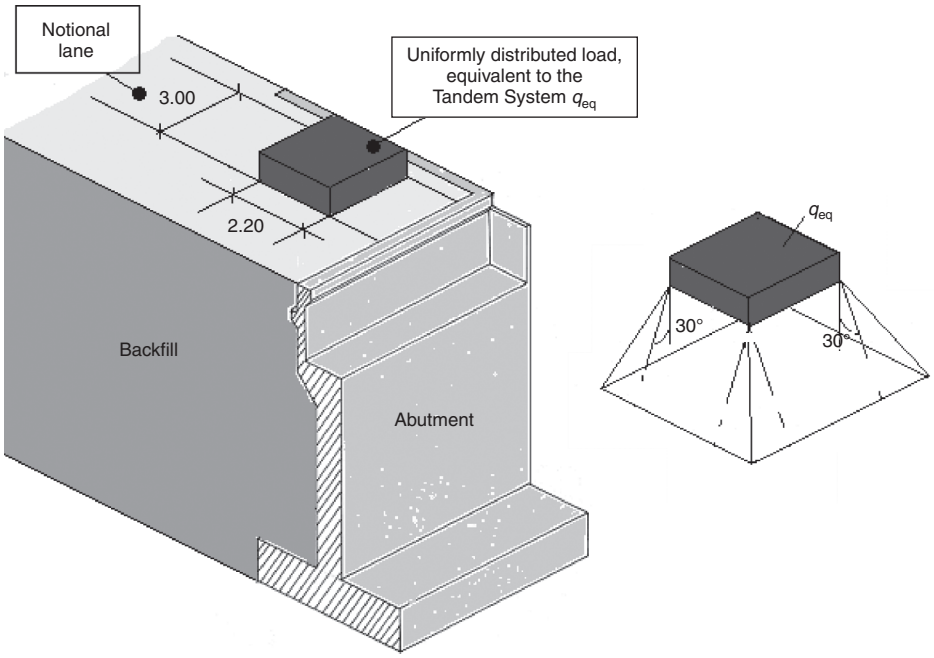


Fig. 4.28. Application of LM1 behind an abutment

For example (see Fig. 4.28), in the case of Lane No. 1 and for α factors equal to 1:

$$q_{eq} = \frac{600}{3 \times 2.2} \times 0.7 \cong 63.6\,\text{kN/m}^2$$

Outside this rectangle, the lane is loaded with a uniformly distributed load of $9 \times 0.7 = 6.3\,\text{kN/m}^2$.

4.9.2. Horizontal force

cl. 4.9.2: EN 1991-2

A horizontal force at the surfacing level of the carriageway over the backfill would be superfluous: for that reason, the Eurocode does not define any expression for such a force. On the other hand, a lorry may brake when arriving on the bridge. Therefore, for the design of upstand walls of abutments (see Fig. 4.29), a longitudinal braking force should be taken into account with a characteristic value equal to $0.6\alpha_{Q1}Q_{1k} = 180\alpha_{Q1}$ kN, acting simultaneously with the $\alpha_{Q1}Q_{1k} = 300\alpha_{Q1}$ kN axle loading of LM1 and with the earth pressure from the backfill. The backfill should be assumed not to be loaded simultaneously.

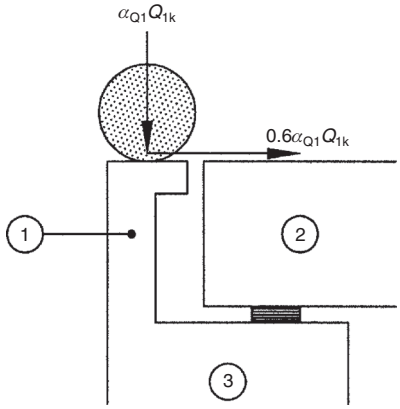


Fig. 4.29. Definition of loads on upstand walls: (1) Upstand wall; (2) Bridge deck; (3) Abutment (Reproduced from EN 1991-2, with permission from BSI)

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4.10. Worked examples

4.10.1. Example of LM1 arrangement for the study of the transverse bending of a bridge deck

We consider a very common composite steel–concrete bridge with two girders. Its cross-section is shown in Fig. 4.30. The carriageway width is divided into three notional lanes and a remaining area of 2 m width. The objective is to apply LM1 in order to obtain the most unfavourable bending moment in the transverse direction in sections S1 and S2.

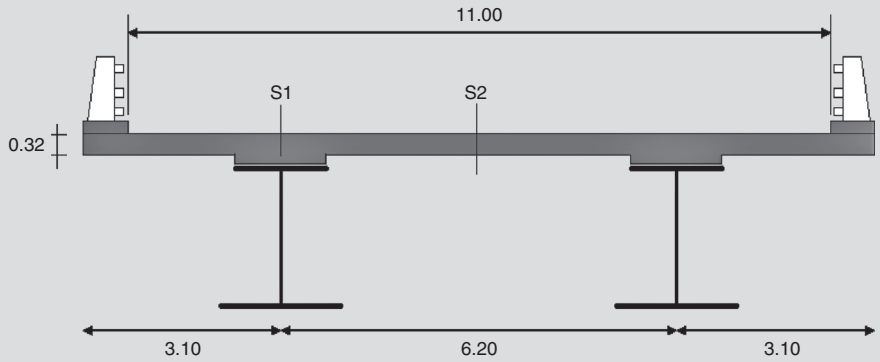


Fig. 4.30. Cross-section of the composite steel–concrete bridge deck

This bridge is designed, for example, for Class 2 traffic as defined in Table 4.3, which means that the axle loads in Lanes No. 1–2–3 are respectively equal to $0.9 \times 300 = 270\,\text{kN}$, $0.8 \times 200 = 160\,\text{kN}$, $0.8 \times 100 = 80\,\text{kN}$. Concerning UDL, the value in Lane No. 1 is $0.7 \times 9 = 6.3\,\text{kN/m}^2$; in the other lanes, the standard value $2.5\,\text{kN/m}^2$ is retained.

For this example, the cross-section is modelled as a slab simply supported along the girders to simplify the shape of the influence lines/surfaces.

Figure 4.31 shows the loading system corresponding to the most unfavourable bending moment over one girder. In this figure the wheels are represented by their contact area under the vertical force. In fact, the influence surface is more complex than the surface considered in this example, but the result is correct for the determination of the slab reinforcement.

Figure 4.32 shows the influence surface obtained by finite-element analysis of the bending moment in the transverse direction for a square slab.

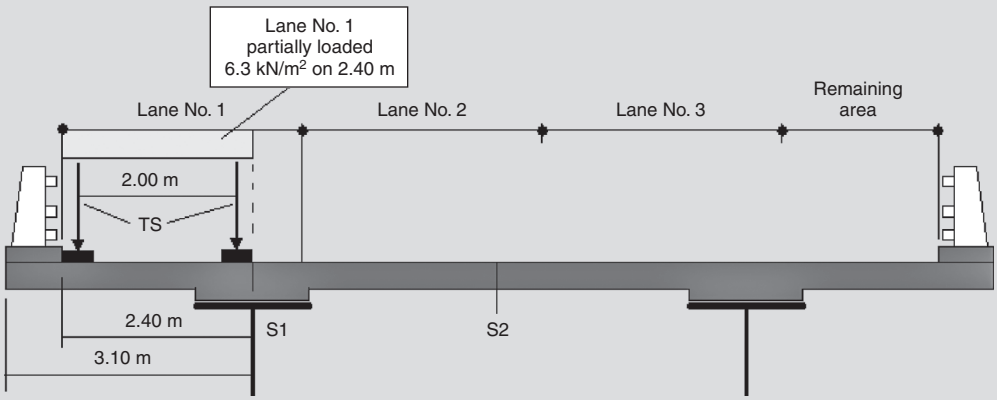


Fig. 4.31. Loading system for the maximum bending moment in section S1

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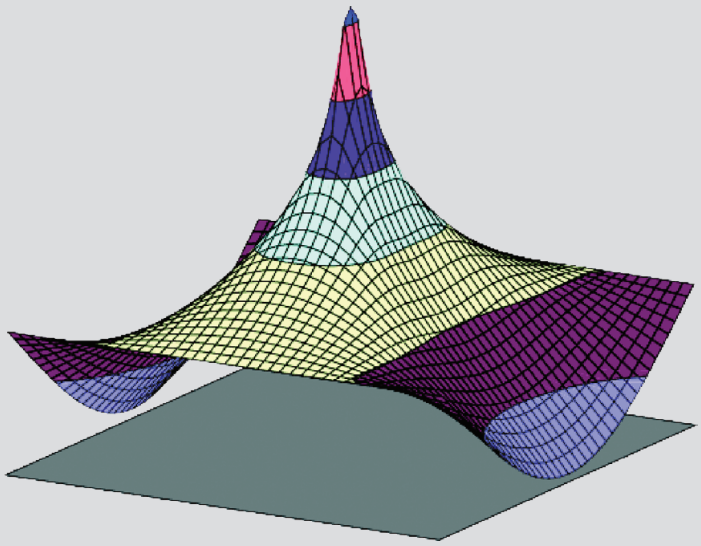


Fig. 4.32. Example of influence surface of the bending moment in the transverse direction for a square slab

The location of the loading system to obtain the most unfavourable effect is represented in Fig. 4.33.

The Tandem System of Lane No. 1 is positioned so that a line of loads is close to midspan. Lane No. 1 is positioned to obtain the most unfavourable effect, which implies the maximum excentricity between TS and UDL. Then Lanes No. 2 and No. 3 are positioned and partially loaded by UDL (only the positive part of the influence line is loaded).

For local effects, the position of loads is shown in Fig. 4.34.

The computed results (in kNm/m) are as follows:

	UDL	TS	Total
Lane 1	21.4	74.7	96.1
Lane 2	2.8	16.0	18.8
Lane 3	0.8	0.0	0.8
Total	24.9	90.7	115.6

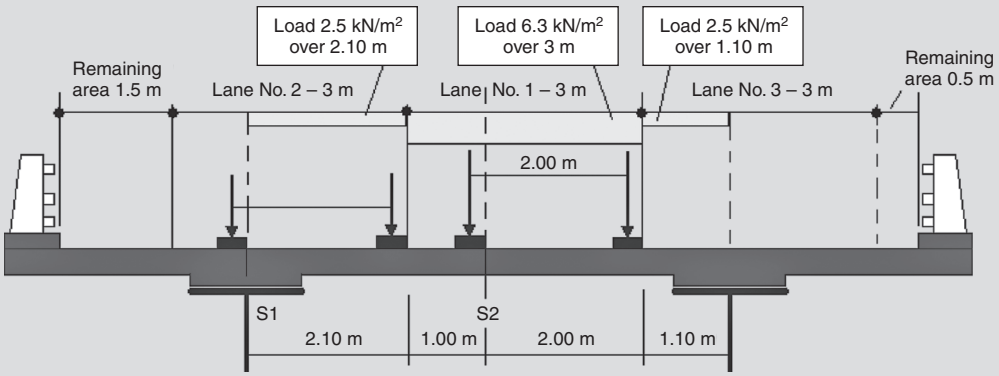


Fig. 4.33. Loading system for the maximum bending moment in section S2

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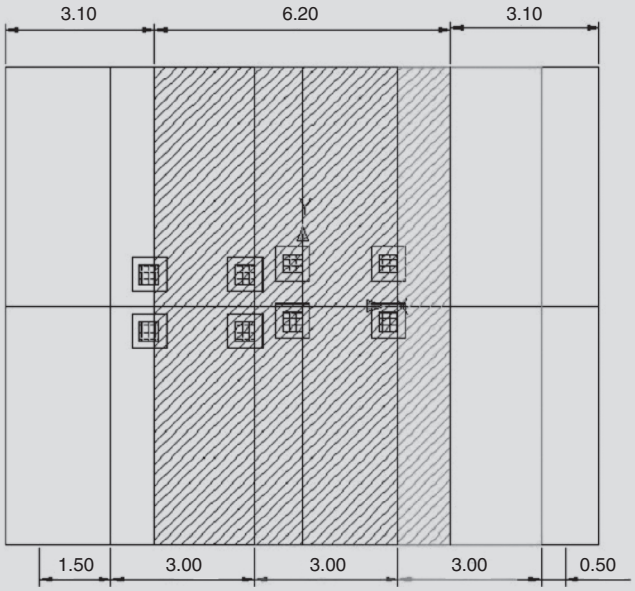


Fig. 4.34. Position of the loading system to obtain the most unfavourable effect

4.10.2. Example of application of loads on the backfill of a portal concrete bridge

The portal bridge is described in Fig. 4.35.

The purpose of this example is to show how the main load model LM1 may be applied to the road with regard to the backfill for the calculation of earth pressure on the vertical walls.

cl. 4.9.1: EN 1991-2

In accordance with the Eurocode, the same notional lanes are considered on the road as on the bridge deck. The uniformly distributed load UDL should be applied as for the bridge decks. However, for the Tandem Systems, it is suggested to replace them by an equivalent uniformly distributed load on the rectangular surface mentioned in Section 4.9.1 above. The example in Fig. 4.36 shows the loading of the lanes just behind the vertical wall.

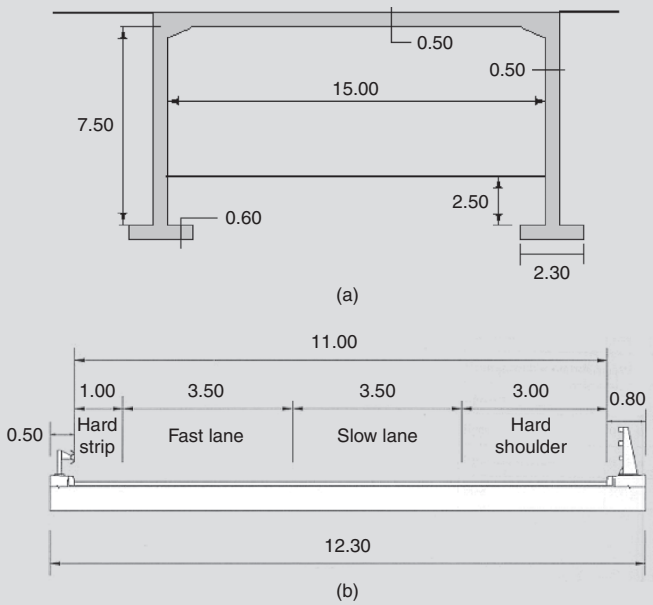


Fig. 4.35. Description of the portal concrete bridge: (a) View of the bridge in the longitudinal direction; (b) Cross-section of the upper slab

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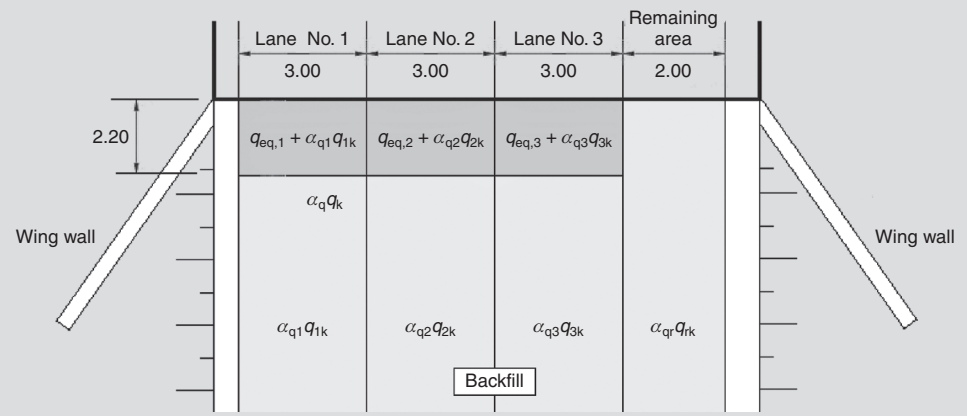


Fig. 4.36. Loading of the notional lanes on the backfill

Of course, these loads need to be distributed in the backfill with a dispersal angle. The recommended value of this dispersal angle from the vertical is 30°. Figure 4.37 shows the effect of the dispersal in the longitudinal direction.

Of course, the dispersal of the various equivalent loads for the tandem systems need to be considered in the transverse direction.

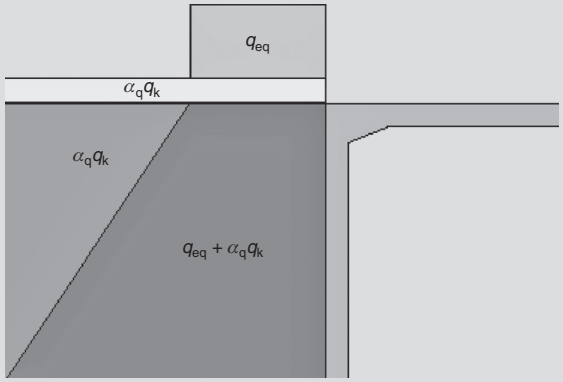


Fig. 4.37. Dispersal of the equivalent load in the backfill

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Annex to Chapter 4: Background information on the calibration of the main road traffic models in EN 1991-2

A4.1. Traffic data

The work for the development of EN 1991-2 (formerly ENV 1991-3) *Traffic loads on bridges* started in September 1987. The available traffic data provided by various countries included:

- Data collected from 1977 to 1982 in France, Germany, UK, Italy and the Netherlands
- More recent data mostly collected in 1986 and 1987 in several countries. Four countries (France, Germany, Italy and Spain) had full computerized records of traffic, including all the required information concerning the axle weights of heavy vehicles, the spacing between axles and between vehicles, and vehicle length.

Most of the data were recorded on the 'slow lane' (i.e. the lane supporting the heaviest traffic) of motorways or main roads. The duration of the records varied from a few hours to more than 800 hours. These traffic data have been used to define the main loading system (LM1) and the complementary loading system consisting of a single axle (LM2), and to check the possibility of practical use of the fatigue load model FLM3.

The results of the calibration have been checked with more recent data (mainly collected between 1996 and 1998): even if an increase in traffic was observed, this increase was rather limited and had no influence on the traffic load models which can be considered as perfectly fitted to the effects of actual traffic in the year 2000 in European countries.

A4.1.1. Traffic composition

The observed medium flow of heavy vehicles varied in general from 2500 to 4500 vehicles per day on the slow lane of motorways and main roads, and from 800 to 1500 per day on all other roads. On the 'fast' lanes of motorways or on secondary roads, this medium flow dropped to around 100–200 vehicles per day.

The distribution of the distance between lorries followed a 'gamma'-type law with a mode between 20 and 100 m, a mean value in the range 300 to +1000 m and a large coefficient of variation (2 to 4). For analysis of the traffic composition, four classes of vehicles were defined as follows:

- class 1: double-axle vehicles
- class 2: rigid vehicles with more than two axles
- class 3: articulated vehicles
- class 4: vehicles with trailers.

Although the traffic composition differed slightly from one European country to another, the most frequent types of vehicles were the double-axle and the articulated vehicles. Lorries with trailers were found most frequently in Germany.

The number of axles per vehicle, which depends on the manufacturer, varied widely, but histograms of their spacing revealed three persistent modes with peak values particularly constant:

- $d = 1.30$ m, corresponding to the double and triple axles with a very small standard deviation
- $d = 3.20$ m, corresponding to the tractor axles of the articulated lorries, with a small standard deviation
- $d = 5.40$ m, corresponding to the other spacings but with a widely scattered distribution.

A4.1.2. Axle and vehicle weights

The distribution of axle weights was very scattered, with a mean value around 60 kN. However, the maximum weight corresponding to a return period of 1 day was much more

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Table A4.1. Range of maximum weights per day

Types of axles	Single axles	Tandems	Tridems
Value range (kN) of the maximum in a day	140 to 200	220 to 340*	300 to 380

* Most of the values varied between 250 and 300 kN

stable from one location to the other. Table A4.1 gives full-ranging information on the observed maximum weight per axle type, corresponding to a return period of 1 day.

The maximum value of the total weight of vehicles for a return period of 1 day was fairly constant from one location to the other, mostly in the range 550–650 kN. All observed statistical distributions showed two modes: the first one around 150 kN and the second one (corresponding to 20 or 30% of the lorries) around 400 kN. Figures A4.1(a) to A4.1(d) show typical histograms of some traffic parameters.

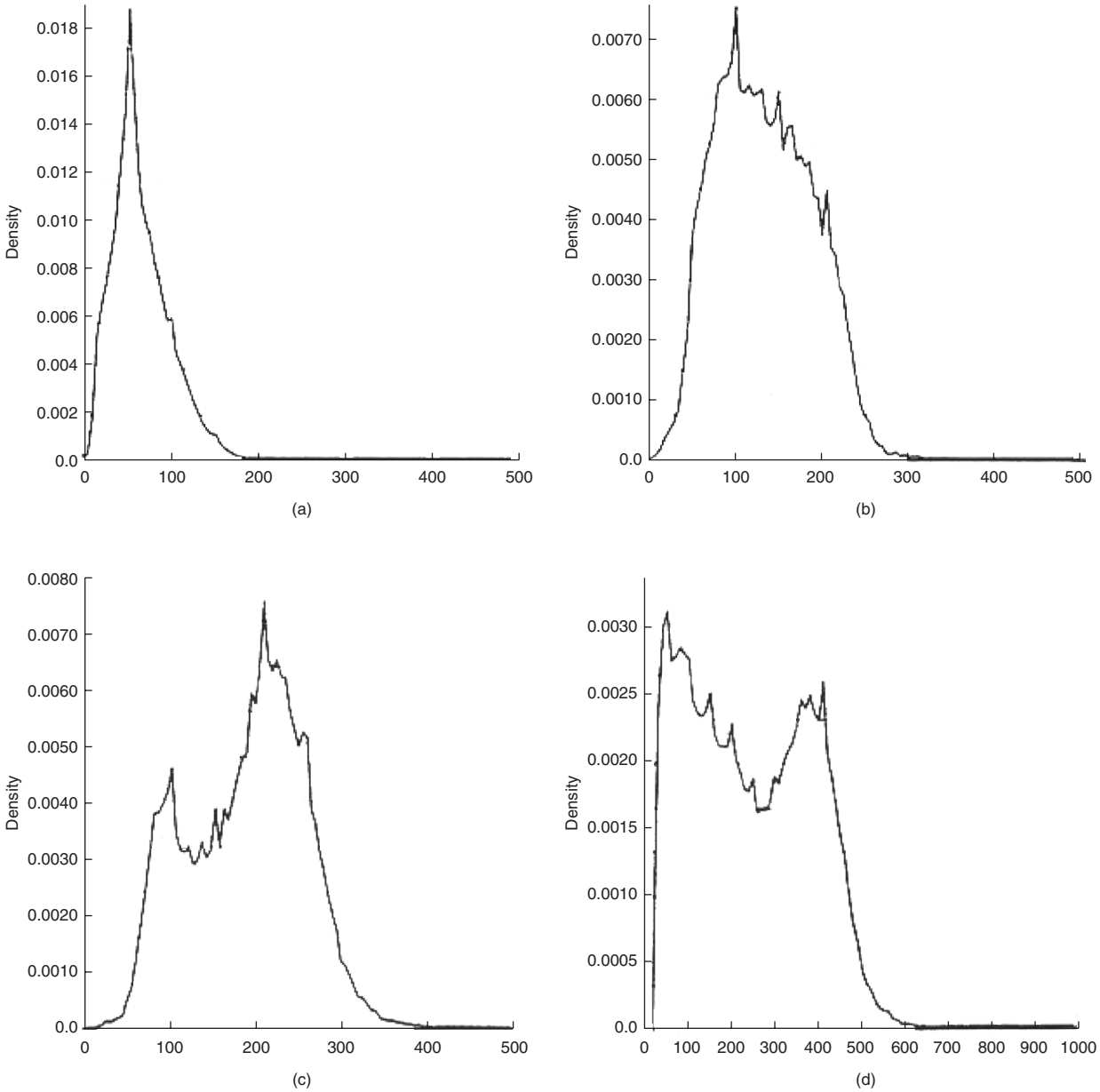


Fig. A4.1. Examples of histograms of typical traffic parameters: (a) Axle weights (kN); (b) Tandem weights (kN); (c) Tridem weights (kN); (d) Truck gross weights, *W* (all types) (kN)

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Finally, and despite some variations in the result of the measurements in the various countries (these variations resulted mostly from the choice of traffic samples), the road traffic parameters appeared to be numerically similar, in particular for the maximum daily values of axle weights and vehicle total weights. This was probably due to the fact that:

- the various national lorry manufacturers produce the same type of vehicles and export them widely in the European countries
- the transportation companies try to load their vehicles as heavily as possible in order to achieve lower costs
- the motorways and roads mainly used by the heaviest vehicles are used by long-distance traffic, which is increasingly international.

The majority of calibration studies were performed with traffic samples recorded on the French A6 motorway near the city of Auxerre, where the traffic is mainly international. This traffic was rather heavy for one loaded lane, but it was not the heaviest observed traffic; for example, the traffic on the slow lane of the Brohltal bridge in Germany was the most 'aggressive', and the recorded daily maximum axle weight was equal to 210 kN on the Paris ringroad while it was equal to 195 kN on the slow lane of the A6 motorway.

A4.2. Determination of the vertical effects of real traffic

A4.2.1. Influence lines and areas taken into account for calibration of Load Models LM1 and LM2

Preliminary studies showed that all national loading systems had both qualities and failings. Therefore it was decided to develop an original loading system with the following properties:

- Its effects had to reproduce very accurately the total utmost effects due to the actions of real traffic (or stem from the chosen representative values) for various shapes and dimensions of influence areas.
- Its effects should not vary significantly (i.e. a degree of robustness) if the system is only applied on a (significant) part of the relevant influence areas, so that the worst loading case can be easily determined.
- Its application rules should be as simple to understand and as unambiguous as possible.

The measured loads have been applied to the following theoretical influence areas, described as influence lines in Table A4.2 and represented diagrammatically in Fig. A4.2.

Influence areas of bending moments in the longitudinal and transverse directions of slab bridges (straight and skewed bridges) were also taken into account, but the calibration

Table A4.2. Influence lines/areas taken into account for the calibration of LM1 and LM2

Influence line No.	Nature of the influence line
I1	Maximum bending moment at midspan of a simply supported beam
I2	Maximum bending moment at midspan of a double fixed beam with an inertia that strongly varies between midspan and the ends
I3	Maximum bending moment on support of the former double fixed beam
I4	Minimum shear force at midspan of a simply supported beam
I5	Maximum shear force at midspan of a simply supported beam
I6	Total load
I7	Minimum bending moment at midspan of the first of the two-spans of a continuous beam (the second span only is loaded)
I8	Maximum bending moment at midspan of the first span of the former continuous beam
I9	Bending moment on central support of the former continuous beam

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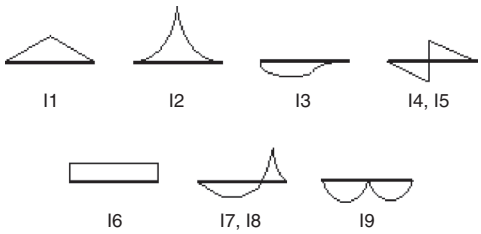


Fig. A4.2. Diagrammatic representation of the influence lines/areas

exercises were mainly based on influence areas of bridge decks globally represented as beams. In general, the loaded lengths were $L = 5$, 10, 20, 50, 100, 200 m.

A4.2.2. Extrapolation of traffic data for the calibration of LM1 and LM2

As previously explained, the real traffic was recorded at various locations and during periods of time that varied from a few hours to more than 800 hours. The project team experts decided to calibrate load models LM1 and LM2 so that the characteristic value of their effects would correspond to a return period of 1000 years (see Section 4.3.2 of this Designers' Guide). Therefore, it was necessary to extrapolate the effects determined from measured traffic.

Three extrapolation methods were used, with some variations. The first method assumed that the tail of the distribution of local extrema followed a Normal law. For the second method, the distribution of recorded data was replaced by a bi- or tri-modal Gumbel law. The last method was based on the use of Rice's formula for the idealization of the tail of the recorded data distribution (Fig. A4.3).

All the studies concerning the extrapolation of the observed road traffic effects showed that the various methods led to more or less equivalent results. The first idea was to mix all traffic records in order to get a 'European sample', but some of the extrapolation methods based on mathematical simulations of traffic needed a sample of homogeneous traffic. Starting from the fact that the traffic recorded on the French A6 motorway near the city of Auxerre was, in fact, 'European' traffic, it was decided that all the statistical developments would be performed solely with these traffic data.

Table A4.3 gives the extrapolated values of axle loads and gross weight of lorries corresponding to return periods of 20 weeks, 20 years and 2000 years. These values were established by the third method, but the two other methods gave similar results.

For the total effect of free-flowing traffic on one lane, the various methods also gave homogeneous results. Table A4.4 gives extrapolated values (averaged on the results of the three methods), for various loaded lengths, of the ratio total load/loaded length (in kN/m) on the same lane.

The extrapolated values of the total load divided by the loaded length increase by about 10% to 16% between the 20-year and 1000-year return periods, depending on the loaded length.

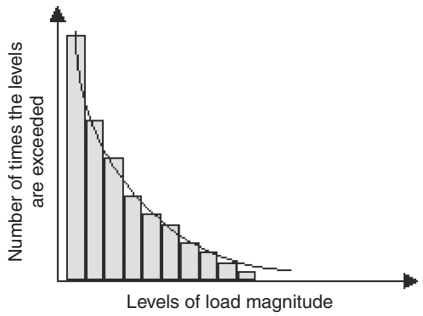


Fig. A4.3. Adjustment of Rice's formula to the tail of a histogram

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Table A4.3. Extrapolated values of axle loads and gross weight of lorries

Return period	Type of load	Extrapolated values (kN)
20 weeks	Single axle	252
	Tandem	332
	Tridem	442
	Gross weight	690
20 years	Single axle	273
	Tandem	355
	Tridem	479
	Gross weight	736
2000 years	Single axle	295
	Tandem	379
	Tridem	517
	Gross weight	782

Comments: The difference between 20-week and 20-year return periods is about 7–9%; the difference between 20-year and 2000-year return periods is again about 6–8%.

Table A4.4. Extrapolated values of the ratio total load/loaded length

L (m)	Extrapolation to 20 years	Extrapolation to 1000 years
20	45.65	50.37
50	29.43	33.03
100	20.45	23.73
200	13.52	15.70

Similar observations have been made for the effects of actions. For example, Table A4.5 gives the extrapolated values of the equivalent distributed load (kN/m) that produces, in a simply supported beam and for a single loaded lane, the maximum bending moment at midspan.

From all results of calculations, it has been possible to propose an empirical formula linking the value of a particular effect of road traffic loads corresponding to a return period of 20 weeks, denoted $E_{20\,\text{weeks}}$, to the value of the same effect corresponding to a return period T (in years), denoted E_{T}:

$$E_{\text{T}} = [1.05 + 0.116 \log_{10}(T)] E_{20\,\text{weeks}}$$

For example $E_{100\,\text{years}} = 1.28 E_{20\,\text{weeks}}$ and $E_{1000\,\text{years}} = 1.40 E_{20\,\text{weeks}}$, so that $E_{1000\,\text{years}} = 1.09 E_{100\,\text{years}}$: there is only a difference of 9% between effects (in general) for 100 years and 1000 years return periods.

Table A4.5. Extrapolated values of the equivalent distributed load (kN/m) producing the maximum bending moment at midspan of a beam

Span length (m)	Return period 20 weeks	Return period 20 months	Return period 20 years	Return period 1000 years
20	46.5	54.4	60.4	65.1
50	23.7	26.1	28.4	33.2
75	18.4	20.2	22.1	25.8
100	15.6	17.2	18.7	21.8
150	13.1	14.4	15.7	18.3
200	11.7	12.9	14.0	16.4

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Table A4.6. Comparison between effects of various traffic conditions

Span length (m)	Free-flowing traffic	Congested traffic with light vehicles	Congested traffic without light vehicles
20	60.34	51.42	52.87
50	34.26	40.45	42.40
100	22.76	35.70	36.50
200	17.70	31.33	33.63

Finally, any bridge can be subjected to various traffic situations: free-flowing traffic, condensed traffic, traffic jams, special situations due to social demonstrations ('snail' operations), etc. These situations have also been extrapolated, mostly with simulation software (based on the Monte-Carlo method) and starting from the observed traffic on the French A6 motorway near the city of Auxerre.

For example, Table A4.6 shows, for a return period of 1000 years, a comparison between the effects of free-flowing traffic, of congested traffic with light and heavy vehicles and of congested traffic without light vehicles. The values correspond to an equivalent distributed load (in kN/m) producing an utmost bending moment at midspan of a simply supported beam.

A4.3. Definition and determination of 'target' effects

The definition and the calibration of load models for road bridges is not only a matter of extrapolation of measured load effects: the load models also have to take into account the various foreseeable traffic situations that can occur on a bridge deck for the whole of its working life. Therefore, it was necessary to determine target values for several action effects, several loaded lanes and several loaded lengths, to achieve an accurate calibration of the load models.

Three questions had to be resolved:

- What dynamic amplification was probably included in the real traffic records?
- What types of traffic or traffic situations should be taken into account in the various lanes of a road?
- How to take into account the dynamic amplification of effects due to traffic.

Concerning the dynamic amplification included in the real traffic records, it was estimated equal to 10%, therefore all numerical values from measurements were divided by 1.10.

Two families of traffic type were considered: free-flowing traffic and congested traffic. The 'congested' traffic represented various scenarios such as traffic jam, a jam with successive movements of starting and stopping, or even a displacement at low speed. In the calculations, the conventional distance between two lorries to simulate a traffic jam situation was taken as equal to 5 m. For the free-flowing traffic, various percentages of lorries were taken into account in the two slowest lanes (motorway or highway).

Of course, the problem of dynamic amplification is relevant mainly for the free-flowing traffic. In fact, it has not been possible to assess the dynamic effects of traffic independently of the traffic situations and types taken into account. In particular, even for exactly the same traffic scenarios, the dynamic effects were different for bending moments and shear forces.

Finally, many numerical simulations have been performed, taking into account the dynamic behaviour of the vehicles and of the bridges, and based on some assumptions concerning the roughness and quality of the carriageway. For the determination of the characteristic load values, it was decided to consider an average roughness and, for spans shorter than 15 m, local irregularities represented by a 30 mm thick plank that could represent, for example, a localized defect of the carriageway surface or a missing carriageway joint element.

The drawings in Fig. A4.4 are only proposed to give an idea of the dynamic amplification of load effects, this dynamic amplification being represented by an equivalent dynamic

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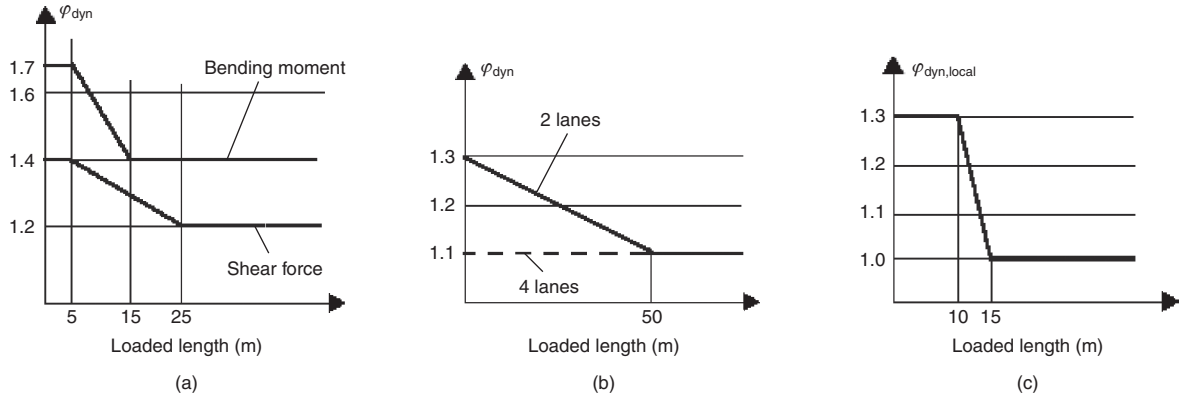


Fig. A4.4. Diagrammatic representation of the dynamic amplification of static traffic load effects: (a) Dynamic amplification factor for one loaded lane; (b) Dynamic amplification factor for 2 and 4 loaded lanes; (c) Complementary (multiplicative) dynamic amplification factor related to local effects

amplification factor. However, these diagrams have not been used for the determination of the target values.

In Fig. A4.4, the factor φ_{dyn} represents the dynamic amplification of the considered effect and depends, among other things, on the span length and on the type of influence area. It is assessed from a statistical comparison with the static effect; hence the maximum of the dynamic effect does not necessarily correspond to the maximum of the static effect. For that reason the 'target' values of the traffic effects have been determined for each influence surface and each action effect, by directly considering the results of particular dynamic calculations.

The 'congested' traffic has been considered either as a flowing traffic at very low speed or by simulation (random distribution of lorries and cars) in conditions estimated similar to flowing traffic.

The set of 'target values' of the action effects has been established:

- from the envelope of all the results related to free-flowing traffic (that includes the dynamic amplification) for short- and medium-span lengths (up to about 50 or 70 m)
- from the average value of all the results related to scenarios with congested traffic for long span lengths
- by smoothing some irregularities mainly due to the lack of results for some span lengths.

Moreover, it appeared that the target values corresponding to very short spans (1 to 10 m) were not satisfactory, especially for local effects. Specific studies led to correcting them by increasing their values: they form the origin of LM2.

For three or four loaded lanes the effects calculated by integrating scenarios of congested traffic on the first or two first lanes were dominant. For this reason the results corresponding to free-flowing traffic do not appear in these tables.

A4.4 Definition and calibration of the characteristic values of Load Models LM1 and LM2

The calibration of LM1 and LM2 was performed step by step, by using operational research methods. However, from the outset, it had been decided:

- to define load models (including automatically the dynamic amplification) associating concentrated and uniformly distributed loads in order to allow the possibility of performing simultaneously local and general verifications
- to fix the minimum value of the distributed load to 2.5 kN/m^2 (value adopted in many existing national standards).

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With the following notation

- E_{1i}, the target values of the selected effects for various span lengths and various influence lines or areas
- E_{2i}, the corresponding values deriving from the load model under calibration
- d_i the 'distance' between E_{1i} and E_{2i} defined by:

$$d_i = \left| \frac{E_{1i}}{E_{2i}} - 1 \right|$$

the optimization method consisted of finding, for various models depending on various parameters, a function E_2 such that

$$d_{\mathrm{m}} = \frac{\sum d_i}{n}$$

be minimum, or

$$d_{\max} = \max \left| \frac{E_{1i}}{E_{2i}} - 1 \right|$$

be minimum as well, or even d_{m} and $d_{\max}$ be minimum and

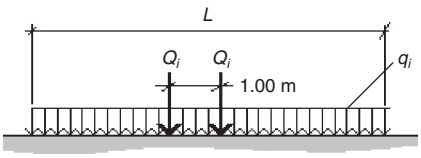
$$\frac{E_{1i}}{E_{2i}} \geq 1 \text{ or } 0.95$$

Many real and theoretical influence lines or areas, for bending, torsion and shear in girders as well as in slabs, were used for the calibration work, covering span lengths ranging between 5 and 200 m.

The calibration of LM1 was performed step by step, starting from Lane No. 1 (heaviest loaded lane, or 'slow' lane), then by adding successively Lane No. 2 and, simultaneously, Lanes No. 3 and 4. The calculations quickly revealed that the best fitted model was composed of both concentrated and uniformly distributed loads; two axle loads were needed, the distance between axles being equal to 1 m, and the intensity of the uniformly distributed load should be a decreasing function of the loaded length, denoted L. Table A4.7 summarizes the calibration steps after consideration of Lane 1, Lanes $1+2$ and Lanes $1+2+3+4$.

This solution was progressively modified for the purpose of simpler application conditions. The accuracy of the calibration was slightly decreased, but the load model became easier to use. In particular, the choice of the parameter L was somewhat ambiguous: it was better to avoid a law depending on the loaded length. With imposed uniformly distributed loads, the calibration studies led to a solution (the model described in this chapter) which gave acceptable results. Accurate calculations taking account of influence lines and areas of length less than 5 m led to an increase in the magnitude of the concentrated loads on the second lane, to correlatively decrease the magnitude of the distributed load on the same lane and to remove the concentrated loads after the third lane. Further, the distance

Table A4.7. Results of calibration studies for LM1

	Loaded lane (s)	Q_i (kN)	q_i (kN/m)
	1	$Q_1 = 185$	$q_1 = 29.3 + \frac{375.6}{L}$
	$1+2$	$Q_1 = 185$	$q_1 = 29.3 + \frac{375.6}{L}$
		$Q_2 = 100$	$q_2 = 0.417 q_1$
	$1+2+3+4$	$Q_1 = 185$	$q_1 = 29.3 + \frac{375.6}{L}$
		$Q_2 = 100$	$q_2 = 0.417 q_1$
		$Q_3 + Q_4 = 150$	$q_3 + q_4 = 0.56 q_1$

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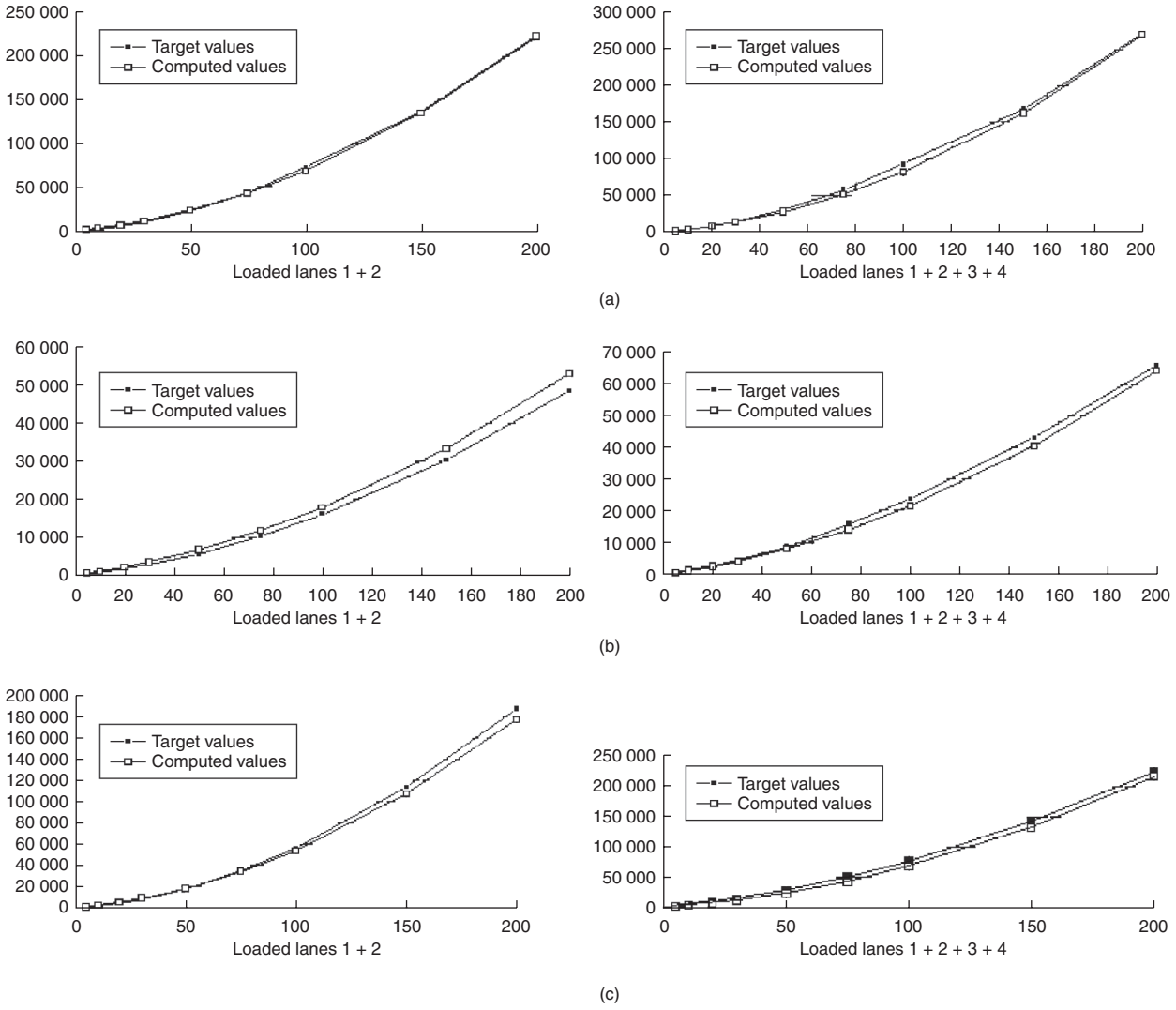


Fig. A4.5. Some comparisons between action effects of LM1 and the relevant target values: (a) Influence line I1 (bending moment at midspan of a simply supported beam); (b) Influence line I2 (bending moment at midspan of a double fixed beam); (c) Influence line I3 (maximum bending moment on support of a double fixed beam); (d) Influence line I7 (minimum bending moment at midspan of first span of a double-span continuous beam); (e) Influence line I8 (maximum bending moment at midspan of the first span of a double-span continuous beam); (f) Influence line I9 (bending moment on central support of a double-span continuous beam)

between concentrated loads in Lanes No. 1 to 3 was increased up to 1.20 m. This value seemed to fit better the real spacing between two axles of lorries, although the concentrated loads were not initially intended to represent the axles of real vehicles.

In order to see the quality of the calibration of LM1, Fig. A4.5(a)–(f) gives a direct comparison between some effects of LM1 and the relevant target values. The selected influence lines are lines I1, I2, I3, I7, I8, I9 as defined in A4.2.1 of this annex. The comparison is established for two and four loaded lanes. The loaded length is read in abscissa. The action effects are in kNm.

Further comments

For influence line I1 (Fig. A4.5(a)), LM1 gives results of very good quality. The most significant differences are obtained with influence line I2 (Fig. A4.5(b)): LM1 is rather conservative for two loaded lanes (+27% for $L = 50$ m and +9% for $L = 200$ m). This is due to the choice of an extreme variation of the moment of inertia of the cross-section of

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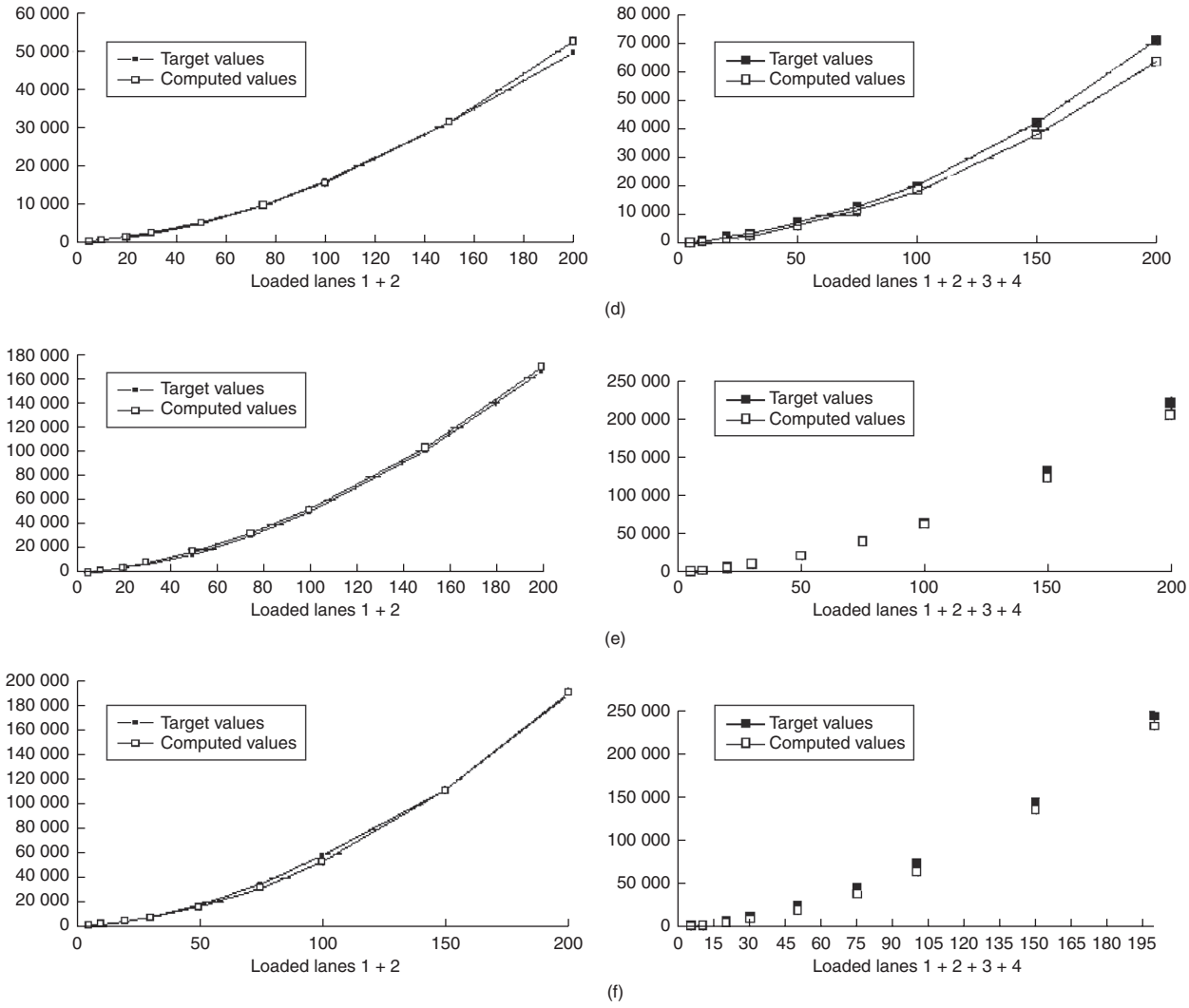


Fig. A4.5. continued

the beam between supports and midspan. For the other influence lines, the deviations between the computed and the target values are fairly insignificant.

A4.5. Calibration of the frequent values of Load Models LM1 and LM2

As mentioned in Section 4.3.2 of this Designers' Guide, the frequent values of LM1/LM2 effects correspond to a return period of one week. They only concern Load Model 1 (main loading system) and Load Model 2 (single axle).

Various simulations have been performed to assess, on the basis of the theoretical influence areas defined in Section A4.2.1 of this annex, the effects of traffic corresponding to a return period of one week to one year and by considering, as for the characteristic values, traffic scenarios of the carriageway. These scenarios envisaged:

- free-flowing traffic
- day traffic
- night traffic
- congested traffic.

The same database as for the determination of characteristic values was used.

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CHAPTER 5

Traffic loads on footbridges

5.1. General – field of application

This chapter is concerned with the description and the determination of traffic loads applicable to footways, cycle tracks and footbridges during permanent and transient design situations. The material in this chapter is covered in Section 5 of EN 1991-2 *Actions on structures – Traffic loads on bridges*.[1] The values of γ and ψ factors for the traffic components and the combinations of actions are given in Chapter 8 of this Designers' Guide, the material of which is covered in EN 1990 Annex A2.[2]

Modern society gives more and more consideration to the environment of people's life, especially in urban areas. One particular consequence of this is the development of footbridge construction for the crossing of obstacles of increasing size. Static loads due to pedestrians or cycles are very light compared to loads due to road or railway traffic. Therefore, long-span footbridges are very slender structures, especially when designed with innovative architectural ideas.

Some problems of dynamic stability, in connection with structural flexibility, have been highlighted in recent years, namely problems due to wind actions, but also due to footbridge–pedestrian interaction.

When crossing a footbridge, people can walk in a number of ways, run, jump or dance. On footbridges, these types of movement may give rise to vibrations which are not yet correctly covered by design standards. The number and location of people likely to be simultaneously on the bridge deck depend on the bridge under consideration, but also on external circumstances, more or less linked to its location; these parameters are commonly highly random and even uncertain.

Some accidental situations such as vandalism may occur. During such situations, the structural behaviour can be strongly modified: these scenarios are not explicitly considered in the Eurocodes, but simulations based on appropriate dynamic load models may be performed.

Forces exerted by several pedestrians in normal circumstances are usually not synchronised and have somewhat different frequencies. However, if one of the natural frequencies of the deck is close to the frequencies of the forces exerted by pedestrians, it is often the case that their perception of some movements of the bridge results in modifications to their gait: their steps tend to become synchronized and coincide with the vibrations of the bridge; resonance then occurs, increasing significantly the response of the bridge. In the case of horizontal vibrations, if the number N of pedestrians reaches one or several critical numbers, people may fully synchronize their movements with the footbridge.

At present, Section 5 of EN 1991-2 gives only static load models for pedestrian and cycle loads, and some general rules dealing with vibrational aspects. The field of application of these static load models is only slightly limited by the footbridge width, and a value of 6 m is suggested in a Note, but this value is rather conventional. In fact, various human

Note 2 cl. 5.1(2): EN 1991-2

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activities may take place on wide footbridges and expert analysis may be needed for individual projects. If there is any doubt, a dynamic analysis needs to be performed in order to determine if the consideration of static load models is sufficient.

5.2. Representation of actions

cl. 5.3.1(2): EN 1991-2

Three static models of vertical loads, which have to be taken into account independently, are defined in the Eurocode; they are not intended to be used for fatigue verifications:

- a vertical uniformly distributed load q_{fk}, applicable to footways, cycle tracks and footbridges
- a concentrated load Q_{fwk}, applicable to footways, cycle tracks and footbridges
- a load representing a service vehicle Q_{serv}, applicable only to footbridges as a 'normal' or an 'accidental' load.

Note 1 to cl. 5.1(2): EN 1991-2

In addition, horizontal forces are defined, accidental design situations are evoked and, as for road bridges, load models for embankments are defined. However, loads on access steps are not defined: a reference is made to EN 1991-1-1.

The effects of loads on construction sites are not intended to be covered by the load models given in Section 5 of EN 1991-2 and should be separately specified, where relevant.

cl. 5.2.3(1): EN 1991-2

It is important to emphasize that the models of vertical and horizontal loads, service vehicles excepted, are applicable to footbridges, on the areas of the deck of road bridges protected by pedestrian parapets, and on footpaths of railway bridges.

cl. 5.2.3(2): EN 1991-2

For inspection gangways located inside the bridge parts and for platforms on railway bridges, the definition of specific models is left to National Annexes or for the individual project, but a model is recommended consisting of a uniformly distributed vertical load equal to $2\,kN/m^2$ and a concentrated load of 3 kN applicable to a square surface of $0.2 \times 0.2\,m^2$. These actions are free actions and are not intended to be taken into account simultaneously.

5.3. Static load models for vertical loads – characteristic values

5.3.1. Uniformly distributed loads

cl. 5.3.2.1: EN 1991-2

Traffic actions to be taken into account for the *design of bridges supporting footways or cycle tracks* are represented by a uniformly distributed load; its recommended characteristic value is equal to $q_{fk} = 5\,kN/m^2$ (Fig. 5.1).

cl. 5.2.1(1): EN 1991-2

Loads due to cycle traffic are generally much lower than those due to pedestrian traffic, but it has been assumed that a frequent or occasional accumulation of pedestrians on cycle lanes may occur. Moreover, pedestrian loads on road or railway bridges give generally small effects compared to those due to road or railway traffic. Nevertheless, the Eurocode mentions that special consideration may need to be given to loads due to horses or cattle for individual projects.

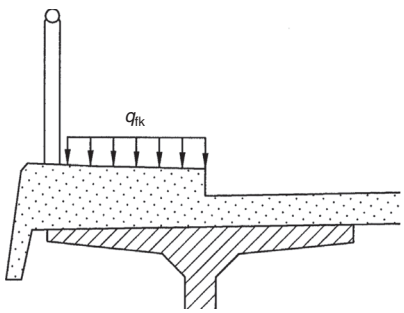


Fig. 5.1. Pedestrian load on a footway or cycle track (recommended value $5\,kN/m^2$)

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Background documentation
Background information on loads due to concentration of people on building floors is rather poor. Tests have been performed in the past with people dancing on a dynamometric platform. Depending on the type of music, the loads varied from 2.9 to $5\,kN/m^2$. With fast music, a magnitude of $5\,kN/m^2$ was reached approximately twice per second. The load corresponding to a concentrated crowd was about $5.5\,kN/m^2$ and a maximum dynamic load density of $8\,kN/m^2$ has been reached by several people jumping simultaneously. Experimental studies were performed for the design of the Stade de France. Dynamic tests were performed in the higher grandstand of Charlety Stadium in Paris, with a density of three people per square metre, but their purpose was to adjust the design in order to limit vertical accelerations and to avoid natural frequencies of the structure below or equal to 5 Hz. The reader should also refer to the TTL Designers' Guide to EN 1991: Buildings.[3]

The characteristic value $q_{fk} = 5\,kN/m^2$ represents a physical maximum load including a limited dynamic amplification (five heavy persons per square metre).

For the *design of footbridges*, the model for the assessment of general effects consists of a uniformly distributed load q_{fk} applicable to the unfavourable parts of the influence surface, longitudinally and transversally. The Eurocode leaves the choice of the characteristic value for the National Annex or for the individual project, but gives the following recommendations:

- Where the footbridge may carry (regularly or not) a continuous dense crowd (e.g. near the exit of a stadium or an exhibition hall), a characteristic value $q_{fk} = 5\,kN/m^2$ may be specified. *Note 1 to cl. 5.3.2.1(1): EN 1991-2*
- Where such a risk does not exist, it is possible to adopt a reduced value for long-span footbridges. The recommended value for q_{fk} is: *Note 2 to cl. 5.3.2.1(1): EN 1991-2*

$$q_{fk} = 2.0 + \frac{120}{L + 30}\ kN/m^2$$

$$q_{fk} \geq 2.5\,kN/m^2; q_{fk} \leq 5.0\,kN/m^2$$

where L is the loaded length in metres. This function is represented in Fig. 5.2.

5.3.2. Concentrated loads

cl. 5.3.2.2: EN 1991-2

The consideration of concentrated loads is required in order to check the resistance of a footbridge to local effects. In general, loads on footbridges may differ depending on their location and on the possible traffic flow of some vehicles. Three cases are envisaged by the Eurocode:

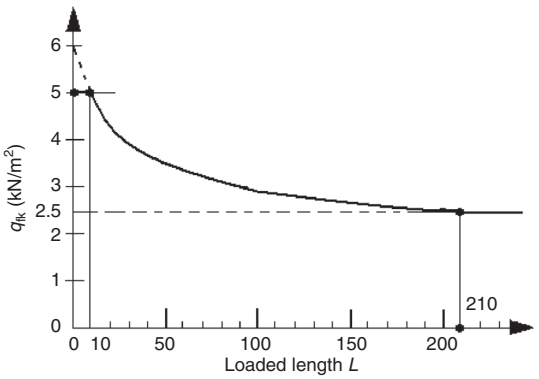


Fig. 5.2. Recommended model of uniformly distributed load for footbridges

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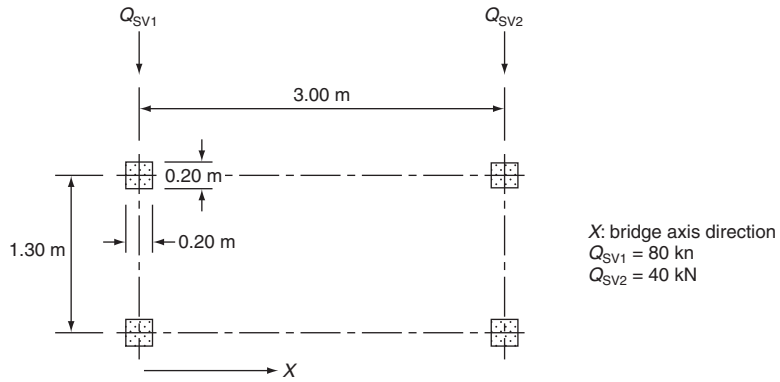


Fig. 5.3. Model for accidental presence of a vehicle on a footbridge deck (Reproduced from EN 1991-2, with permission from BSI)

- First case. Permanent provisions are made to prevent access of all vehicles to the footbridge.
- Second case. The presence of a 'heavy' vehicle on the footbridge is not normally foreseeable but no permanent obstacle prevents this presence: the Eurocode recommends strongly to take into account the accidental presence (accidental design situation) of a vehicle on the bridge deck.
- Third case. A 'heavy' vehicle is foreseen to be driven onto the footbridge deck: it may be a vehicle for maintenance, emergencies (e.g. ambulance, fire) or other services.

In the first case a concentrated load is to be taken into account to check the resistance as regards local effects due, for example, to small equipment for maintenance of the footbridge. The recommended characteristic value of the concentrated load Q_{fwk} is equal to 10 kN, acting on a square surface of sides 0.10 m. All figures may be adjusted in the National Annex. The concentrated load does not act simultaneously with the uniformly distributed load.

cl. 5.3.2.2(1): EN 1991-2

In the second case, the Eurocode defines a load model to be taken into account to represent the accidental presence (accidental design situation) of a vehicle on the bridge deck, consisting of a two-axle load group of 80 and 40 kN, separated by a wheel base of 3 m (Fig. 5.3), with a track (wheel-centre to wheel-centre) of 1.3 m and square contact areas of side 0.2 m at coating level. This model may be adjusted in the National Annex or for the individual project.

cl. 5.3.2.2(3): EN 1991-2

In the third case, a service vehicle Q_{serv} is defined. Its characteristics (axle weight and spacing, contact area of wheels, etc.), the dynamic amplification and all other appropriate loading rules may be defined for the individual project or in the National Annex. If no information is available, the vehicle previously defined for accidental design situations (second case) may be used as the service vehicle (characteristic load). Of course, the concentrated load Q_{fwk} does not act simultaneously with this load model. Where relevant, several service vehicles, mutually exclusive, may have to be taken into account and may be defined for the individual project.

cl. 5.3.2.3: EN 1991-2

5.4. Static model for horizontal forces (characteristic values)

cl. 5.4: EN 1991-2

No horizontal forces are associated with the uniformly distributed load on footways. However, for footbridges, the Eurocode recommends to associate:

- a horizontal force, to the uniformly distributed load, with a characteristic value equal to 10% of the total vertical load
- a horizontal force, due to the service vehicle, with a characteristic value equal to 60% of the total weight of this vehicle.

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Table 5.1. Definition of groups of loads (characteristic values)

Load type		Vertical forces		Horizontal forces
Load system		Uniformly distributed load	Service vehicle	
Groups of loads	gr1	q_{fk}	0	Q_{flk}
	gr2	0	Q_{serv}	Q_{flk}

The rule is as follows: a horizontal force, denoted Q_{flk}, acting along the footbridge axis at the pavement level, is taken into account, equal to the greater of the horizontal forces previously defined.

In the case where an accidental design situation is taken into account, a braking force is associated to the 'accidental' vehicle, equal to 60% of its total weight.

5.5. Groups of traffic loads on footbridges

cl. 5.5: EN 1991-2

As for load models for road traffic, groups of loads are defined for footbridges. Of course, these groups of loads are very simple and based on the load models previously defined. They are presented in Table 5.1, which correspond to Table 5.1 of EN 1991-2.

Each of these two groups of loads, which are mutually exclusive, should be considered as defining a single characteristic action for combination with non-traffic loads.

5.6. Actions for accidental design situations for footbridges

cl. 5.6: EN 1991-2

As for road bridges, such actions are due to:

- road traffic under the bridge (i.e. collision), or
- the accidental presence of a heavy vehicle on the bridge.

For collision forces from road vehicles under the bridge, see Chapter 7 of this Designers' Guide. Nevertheless, it has to be noted that footbridges (piers and decks) are generally much more sensitive to collision forces than are road bridges. Designing them for the same impact forces may be unrealistic. The most effective way to take collision into account generally consists of protecting the footbridges by measures defined in the project specification; for example:

- by establishing road restraint systems at appropriate distances from piers
- by giving the footbridges a higher clearance (for example 0.50 m) than for neighbouring road or railway bridges along the same road in the absence of intermediate access to the road.

The problem of the accidental presence of a 'heavy' vehicle on the bridge has already been discussed in Section 5.3.2 above.

5.7. Dynamic models of pedestrian loads

cl. 5.7: EN 1991-2

EN 1991-2 does not define dynamic load models of pedestrians. It only highlights the need to define appropriate dynamic models of pedestrian loads and comfort criteria, and gives a few recommendations intended to introduce the general comfort requirements defined in EN 1990 Annex A2 (and in Chapter 8 of this Designers' Guide). It is clear that a dynamic study starts with the determination of the relevant natural frequencies of the main structure of the footbridge deck from an appropriate structural model, depending on the dynamic characteristics of the structure. It is also clear that forces exerted by pedestrians with a frequency identical or close to one of the natural frequencies of the bridge can result in

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Fig. 5.4. The Millennium footbridge, London

resonance and needs be taken into account for limit state verifications in relation to vibrations (Fig. 5.4). In the absence of significant response of the bridge, a pedestrian walking normally exerts on it simultaneous periodic forces which are:

- vertical, with a frequency that can range between 1 and 3 Hz, and
- horizontal, with a frequency that can range between 0.5 and 1.5 Hz.

Groups of joggers may cross a footbridge with a frequency of 3 Hz.

Let us remember that footbridges may also be excited by wind, which is outside the scope of EN 1991-2.[1]

5.7.1. Dynamic characteristic of bridges

In Annex F to EN 1991-1-4: *Wind actions*,[4] simplified methods are given to estimate the fundamental frequencies of bridges. These are discussed below, and may be useful for a rough estimation of these fundamental frequencies in the case of footbridges.

Extract from EN 1991-1-4

(5) The fundamental vertical bending frequency $n_{1,B}$ of a plate or box girder bridge may be approximately derived from Expression (F.6).

$$n_{1,B} = \frac{K^2}{2\pi L^2}\sqrt{\frac{EI_b}{m}} \qquad (F.6)$$

where:

- L *is the length of the main span in m*
- E *is Young's Modulus in N/m^2*
- I_b *is the second moment of area of cross-section for vertical bending at mid-span in m^4*
- m *is the mass per unit length of the full cross-section at mid-span (covering dead and super-imposed dead loads) in kg/m*
- K *is a dimensionless factor depending on span arrangement defined below.*

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(a) For single span bridges:

$K = \pi$ *if simply supported or*
$K = 3.9$ *if propped cantilevered or*
$K = 4.7$ *if fixed end supports*

(b) For two-span continuous bridges:

K is obtained from Figure F.2 [reproduced here as Fig. 5.5], using the curve for two-span bridges, where

L_1 *is the length of the side span and* $L > L_1$.

(c) For three-span continuous bridges:

K is obtained from Figure F.2 [see Fig. 5.4 below], using the appropriate curve for three-span bridges, where

L_1 *is the length of the longest side span*
L_2 *is the length of the other side span and* $L > L_1 > L_2$

This also applies to three-span bridges with a cantilevered/suspended main span.

If $L_1 > L$ *then K may be obtained from the curve for two-span bridges, neglecting the shortest side span and treating the largest side span as the main span of an equivalent two-span bridge.*

(d) For symmetrical four-span continuous bridges (i.e. bridges symmetrical about the central support):

K may be obtained from the curve for two-span bridges in Figure F.2 [Fig. 5.5 below] treating each half of the bridge as an equivalent two-span bridge.

(e) For unsymmetrical four-span continuous bridges and continuous bridges with more than four spans:

K may be obtained from Figure F.2 [Fig. 5.5 below] using the appropriate curve for three-span bridges, choosing the main span as the greatest internal span.

Note 1 If the value of $\sqrt{EI_b/m}$ *at the support exceeds twice the value at mid-span, or is less than 80% of the mid-span value, then the Expression (F.6) should not be used unless very approximate values are sufficient.*

Note 2 A consistent set should be used to give $n_{1,B}$ *in cycles per second.*

(6) The fundamental torsional frequency of plate girder bridges is equal to the fundamental bending frequency calculated from Expression (F.6), provided the average longitudinal bending inertia per unit width is not less than 100 times the average transverse bending inertia per unit length.

(7) The fundamental torsional frequency of a box girder bridge may be approximately derived from Expression (F.7):

$$n_{1,T} = n_{1,B}\sqrt{P_1(P_2 + P_3)} \qquad (F.7)$$

with:

$$P_1 = \frac{mb^2}{I_p} \qquad (F.8)$$

$$P_2 = \frac{\sum r_j^2 I_j}{b^2 I_p} \qquad (F.9)$$

$$P_3 = \frac{L^2 \sum J_j}{2K_2 b^2 I_p (1+\nu)} \qquad (F.10)$$

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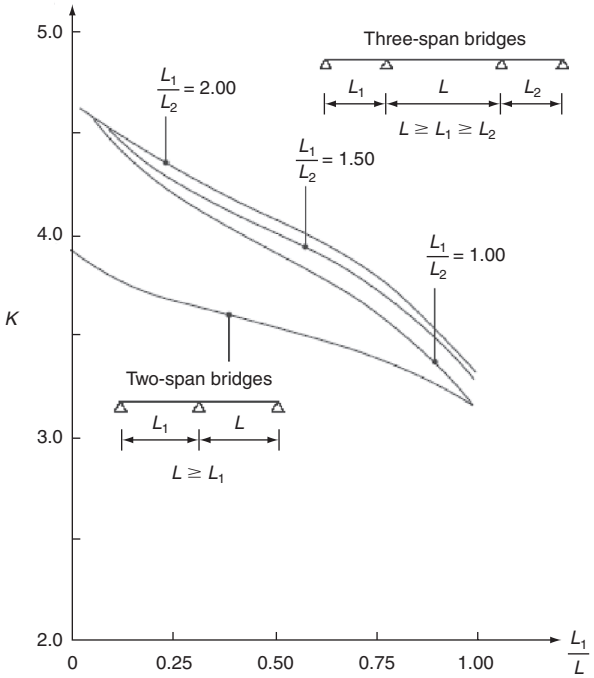


Fig. 5.5. Factor *K* used for the derivation of fundamental bending frequency

where:

$n_{1,B}$ *is the fundamental bending frequency in Hz*
b *is the total width of the bridge*
m *is the mass per unit length defined in F.2(5)*
ν *is Poisson's ratio of girder material*
r_j *is the distance of individual box centre-line from centre-line of bridge*
I_j *is the second moment of mass per unit length of individual box for vertical bending at mid-span, including an associated effective width of deck*
I_p *is the second moment of mass per unit length of cross-section at mid-span. It is described by Expression (F.11).*

$$I_p = \frac{m_d b^2}{12} + \sum (I_{pj} + m_j r_j^2) \qquad (F.11)$$

where:

m_d *is the mass per unit length of the deck only, at mid-span*
I_{pj} *is the mass moment of inertia of individual box at mid-span*
m_j *is the mass per unit length of individual box only, at mid-span, without associated portion of deck*
J_j *is the torsion constant of individual box at mid-span. It is described by Expression (F.12).*

$$J_j = \frac{4A_j^2}{\oint \frac{ds}{t}} \qquad (F.12)$$

where:

A_j *is the enclosed cell area at mid-span*
$\oint \frac{ds}{t}$ *is the integral around box perimeter of the ratio length/thickness for each portion of box wall at mid-span*

Note Slight loss of accuracy may occur if the proposed Expression (F.12) is applied to multibox bridges whose plan aspect ratio (= span/width) exceeds 6.

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Table 5.2. Examples of values of logarithmic decrement of structural damping

Structural type		Structural damping, δ_s
Steel bridges + lattice steel towers	Welded	0.02
	High-resistance bolts	0.03
	Ordinary bolts	0.05
Composite bridges		0.04
Concrete bridges	Prestressed without cracks	0.04
	Prestressed with cracks	0.10
Timber bridges*		0.06–0.12
Bridges, aluminium alloys		0.02
Bridges, glass- or -reinforced plastic		0.04–0.08
Cables	Parallel cables	0.006
	Spiral cables	0.020

Note 1: The values for timber and plastic composites are indicative only. In cases where aerodynamic effects are found to be significant in the design, more refined figures are needed through specialist advice (agreed if appropriate with the competent authority).
Note 2: For cable-stayed bridges the values given in Table F.2 need to be factored by 0.75.
* In EN 1995-2 (Design of timber bridges) the logarithmic decrement of structural damping is in the range $0.01 \times 2\pi = 0.063$ for structures without mechanical joints to $0.015 \times 2\pi = 0.094$ for structures with mechanical joints.

Moreover, in EN 1991-1-4 some approximate values of logarithmic decrement of structural damping in the fundamental mode are proposed (see the Table 5.2).

It should be remembered that the relationship between the structural damping ratio ζ and the logarithmic decrement due to structural damping δ_s is $\delta_s = 2\pi\varsigma$.

5.7.2. Dynamic models of pedestrians

In general, it seems accepted by many experts that the use of three dynamic models may be appropriate as follows:

- a model for a single pedestrian
- a model for a group of pedestrians, for example from 10 to 15
- a model for a dense crowd.

In the following, some background information is given concerning the first two models, but currently it is not possible to give a reliable model for a dense crowd. Many studies are being performed at the present time (2009), and results are expected in the future. The purpose of the following information is to give an idea of the directions adopted in current approaches.

With regard to comfort criteria, see Chapter 8 of this Designers' Guide.

Model for a single pedestrian

The model for a single pedestrian can be directly used for some verifications, but it is mostly used to define the dynamic excitation due to a group of pedestrians. The most basic model, but often agreed by experts, is a harmonic load:

$$Q_p(t) = G \times \sin(2\pi f t)$$

where f is the fundamental frequency under consideration.

For the vertical excitation by a pedestrian who is not running, G is taken equal to 280 N: it is the result of the multiplication of 700 N (representing the average pedestrian weight) by 0.4

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which derives from the development in Fourier's series of the action due to walking for $f = f_v = 2\,\text{Hz}$ and for a pedestrian velocity equal to $0.9 f_v$.

For the horizontal lateral excitation, G varies from 35 to 70 N and, in the previous formula, the frequency is the relevant horizontal frequency.

More sophisticated dynamic models for the single pedestrian have been proposed by several authors: these models associate, in general, several harmonic functions introducing several vibration modes.

In Annex B to EN 1995-2 (Vibrations caused by pedestrians),[5] which is only applicable to timber bridges with simply supported beams or truss systems excited by pedestrians, formulae give directly the vertical and horizontal (lateral) accelerations of the bridge.

(a) Vertical acceleration $a_{\text{vert},1}$:

$$a_{\text{vert},1} = \begin{cases} \dfrac{200}{M\varsigma} & \text{for } f_{\text{vert}} \leq 2.5\,\text{Hz} \\ \dfrac{100}{M\varsigma} & \text{for } 2.5\,\text{Hz} \leq f_{\text{vert}} \leq 5.0\,\text{Hz} \end{cases} \qquad \text{(B.1)}$$

where

M is the total mass of the bridge in kg, given by $M = ml$
ℓ is the span of the bridge
m is the mass per unit length (self-weight) of the bridge in kg/m
ς is the damping ratio
f_{vert} is the fundamental natural frequency for vertical deformation of the bridge.

(b) Horizontal acceleration $a_{\text{hor},1}$ of the bridge:

$$a_{\text{hor},1} = \frac{50}{M\varsigma} \quad \text{for } 0.5\,\text{Hz} \leq f_{\text{hor}} \leq 2.5\,\text{Hz}$$

where f_{hor} is the fundamental natural frequency for horizontal deformation of the bridge.

For example, in the formulae for vertical vibrations, the figure above M derives from $700 \times 0.4\alpha$ where α is the ratio between the structural response due to a pedestrian walking without moving forward and the structural response due to a pedestrian crossing the footbridge. This ratio depends on the structural response and it can only be given acceptable averaged values. For example, in the first case of vertical vibrations, $200 \cong 280 \times 0.7$.

For a jogger, some figures may be different.

Model for a group of pedestrians

The forces exerted by several pedestrians in common circumstances are normally not synchronized and have somewhat different frequencies. However, if one of the natural frequencies of the deck is close to the frequencies of the forces normally exerted by pedestrians, it commonly happens that their perception of some movements of the bridge result in modifications of their gait: their steps tend to become synchronized with the vibrations of the bridge; resonance then occurs, increasing considerably the response of the bridge.

In the absence of significant vibration, the number of persons contributing to the resonance is highly random; beyond about 10 persons on the bridge, it is a decreasing function of their number. For vertical vibrations, the resonance is in most cases mainly, but not solely, linked to the fundamental frequency of the bridge; for horizontal or torsional vibrations, the problem is more complex. However, correlation between forces exerted by pedestrians may increase with movements.

For a group of pedestrians, the model is more sophisticated than for a single pedestrian, but the most simplified rules give a generic expression such as:

$$Q_p(t) = n\psi G \times \sin(2\pi f t)$$

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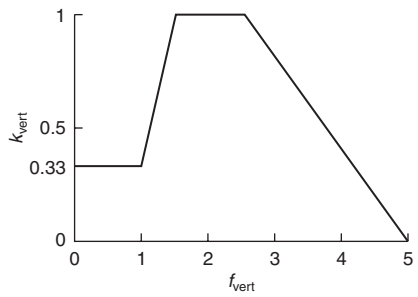


Fig. 5.6. Relationship between the vertical fundamental natural frequency f_{vert} and the coefficient k_{vert}

where

n is the equivalent number of pedestrians on the appropriate loaded surface
ψ is the reduction factor, a function of the difference between the real frequency of the pedestrian excitation and the natural structural frequency under consideration: in fact, it is a mathematical function, varying between 0 and 1, equal to 1 when the natural structural frequency can be excited by pedestrians.

As an example, in EN 1995-2, the following expressions are proposed for a group of people crossing a timber bridge:

(a) Vertical acceleration $a_{vert,n}$:

$$a_{vert,n} = 0.23 a_{vert,1} n k_{vert} \quad \text{(B.2)}$$

where

n is the number of pedestrians
k_{vert} is a coefficient according to Fig. 5.6
$a_{vert,1}$ is the vertical acceleration for one person crossing the bridge determined according to Expression (B.1)

The number of pedestrians, n, should be taken as:

- $n = 13$ for a distinct group of pedestrians
- $n = 0.6A$ for a continuous stream of pedestrians

where A is the area of the bridge deck in m^2.

It has to be noted that $0.23n$ is a good approximation of $\sqrt{n}$ for $12 < n < 20$: $0.23n \cong \sqrt{n}$ for $n \cong 19$.

(b) Horizontal (lateral) acceleration $a_{hor,n}$:

$$a_{hor,n} = 0.18 a_{hor,1} n k_{hor} \quad \text{(B.5)}$$

where k_{hor} is a coefficient according to Fig. 5.7.

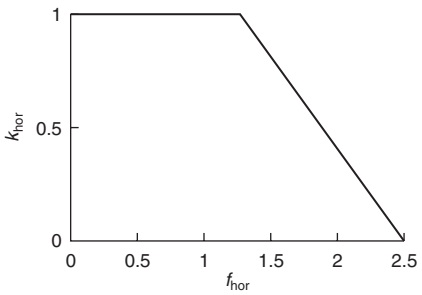


Fig. 5.7. Relationship between the horizontal fundamental natural frequency f_{hor} and the coefficient k_{hor}

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The number of pedestrians, n, should be taken as:

- $n = 13$ for a distinct group of pedestrians
- $n = 0.6A$ for a continuous stream of pedestrians

where A is the area of the bridge deck in m^2.

Other models

Several other models have been proposed by authors or scientific associations. They all have qualities and inadequacies. The concept of critical number of pedestrians sometimes appears. For example, according to an Arup consultant (pers. comm.), the critical number of pedestrians leading to lateral instability may be expressed according to the formula:

$$n_c = \frac{8\pi\zeta, f_i M_i}{k}$$

where

ζ is the damping ratio
f_i is the natural frequency (rad/s)
M_i is modal mass
k is the empirical factor equal, for example, to 300 Ns/m for frequencies in the range 0.5–1.0 Hz.

However, the concept of critical number of pedestrians still needs to be validated.[6]

cl. 5.8: EN 1991-2

5.8. Actions on parapets

The rules are exactly the same as those defined for road bridges. See Chapter 4 of this Designers' Guide.

cl. 5.9: EN 1991-2

5.9. Load model for abutments and walls adjacent to bridges

The Eurocode gives a very simple rule for the design of abutments and walls adjacent to bridges: the backfill or earth is loaded with a uniformly distributed load of $5\,kN/m^2$ which is not intended to cover the effects of heavy site vehicles. Of course, this (characteristic) value may be adjusted for the individual project.

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CHAPTER 6

Traffic loads on railway bridges

6.1. General

This chapter is concerned with the description and the assessment of traffic loads on railway bridges as well as earthworks during persistent and transient design situations. The material in this chapter is covered in the relevant clauses of EN 1991-2, *Eurocode 1: Actions on structures* – Part 2: *Traffic loads on bridges* (including Annexes C to H),[1] as well as in EN 1990 Annex A2.[2,3] Background is also taken from International Union of Railways (UIC) Codes listed in the Reference section of this chapter.

The structures must be designed in such a way that their deterioration, during the period of use of the construction, does not jeopardize their durability or performance within their environment and in relation to the level of maintenance defined for the individual project.

The rules about maximum permissibles deformations of bridges for speeds less than 200 km/h, given later in Chapter 8 (Table 8.12) of this Designers' Guide, differ from those given in EN 1990:2002/A1 (Annex A2), taking into account not only bridge but also track maintenance conditions. This is because, taking the load classification factor (see ***Clause 6.3.2(3)P: EN 1991-2***) with a value of $\alpha = 1.33$ as recommended in UIC Code 702[4] and in Section 6.7.2 below for ultimate limit states and for all new railway bridges, as well as the rules for permissible deformations given in Section 8.7.4 below, there is generally no need for a dynamic analysis for speeds less than 200 km/h.

cl. 6.3.2(3)P: EN 1991-2

The notes in this chapter should help the relevant authorities to establish their National Annexes for EN 1991-2 (Chapter 6) as well as for EN 1990: 2002/A1(Annex 2),[3] in order to obtain a uniform application of these Codes on all European rail networks with regard to bridge load capacity.

The logic diagram given in ***EN 1991-2, Fig. 6.9*** mentions cases where a dynamic analysis is required for sites with a maximum line speed less than 200 km/h. This analysis can be avoided by building stiffer bridges for cheaper track maintenance and by not attributing more expensive investment costs for the bridges when taking into account life-cycle cost analysis.

Fig. 6.9: EN 1991-2

6.2. Classification of actions: actions to be taken into account for railway bridges

As for all construction works, actions may be classified in several ways. The most common method for the establishment of combinations of actions is to adopt a classification depending on their variation with time:

- permanent actions that are either constant, vary very slowly with time or only occasionally, for example self-weight, imposed loads, uneven settlements etc.

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- variable actions, e.g. rail traffic actions, wind actions, temperature effects etc.
- accidental actions, e.g. from impact from derailed vehicles on bridge supports or superstructure, derailment loads on the bridge deck etc.

For the design of railway bridges, the following actions need to be taken into account where relevant.

(a) Permanent actions

Direct actions:

- Self-weight
- Horizontal earth pressure and, if relevant, other soil/structure interaction forces
- Track and ballast
- Movable loads:
  - self-weight of non-structural elements
  - loading from overhead line equipment (vertical and horizontal)
  - loading from other railway infrastructure equipment

Indirect actions:

- Differential settlement (including the effects of mining subsidence where required by the relevant authority)
- Shrinkage and creep for concrete bridges
- Prestress

(b) Variable actions – rail traffic actions

- Vertical traffic actions (based on UIC Codes 700,[5] 702,[4] 776-1[6]):
  - LM 71
  - LM SW/0
  - LM SW/2
  - Load Model HSLM (High-Speed Load Model in accordance with Eurocode EN 1991-2 where required by the Technical Specification for Interoperability of High Speed Traffic in accordance with the relevant EU Directive and/or the relevant authority, based on UIC Code 776-2[7]).
  - Load Model 'unloaded train' for checking lateral stability in conjunction with the leading lateral wind actions on the bridge.
  - load effects from real trains (where required by the relevant authority).
- Centrifugal forces
- Traction and braking
- Nosing
- Longitudinal forces (based on UIC Code 774-3[8] for load effects generated by the interaction between track and structure).
- Load effects generated by the interaction between train, track and structure to variable actions and in particular speed (based on UIC Code 776-2[7]).
- Live load surcharge horizontal earth pressure.
- Aerodynamic actions (slipstream effects from passing rail traffic etc., based on UIC Code 779-1[9]).

(c) Variable actions – other traffic actions

- Loads on non public footpaths (uniformly distributed and point loads).

(d) Variable actions – other

- Other operating actions:
  - stressing or destressing continuous welded rails

(e) Accidental actions

- Actions corresponding to derailment of rail traffic on the bridge.
- Actions corresponding to derailment of rail traffic beneath or adjacent to the bridge (based on UIC Codes 777-1[10] and 777-2[11]).
- Accidental loading from errant road vehicles beneath the bridge.
- Accidental loading from over-height road vehicles beneath the bridge.

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- Ship impact
- Actions due to the rupture of catenaries
- Accidental loadings during construction

(f) Seismic actions

- Actions due to earthquake loading

6.3. Notation, symbols, terms and definitions

Notation, symbols, terms and definitions are those given in EN 1991-2. Only Fig. 6.1, *EN 1991-2, Fig. 1.1*, is reproduced here, and some definitions are given to aid understanding of some concepts of this chapter.

Fig. 1.1: EN 1991-2

cl. 1.4.3: EN 1991-2

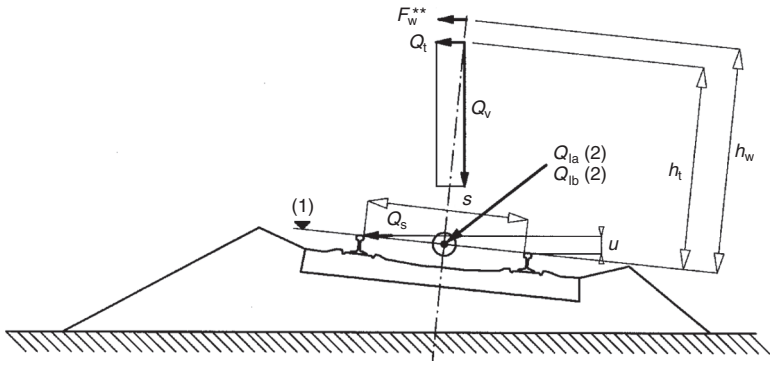


Fig. 6.1. Notation and dimensions specifically for railways (EN 1991-2, Fig. 1.1)

Glossary

Term	Definition
Footpath	Strip located alongside the tracks between the tracks and the parapets
Frequent operating speed	Most probable speed at the site for a particular type of real train (used for fatigue considerations)
Maximum design speed	*Generally 1.2 × maximum nominal speed*
Maximum line speed at the site	Maximum permitted speed of traffic at the site specified for the individual project (generally limited by characteristics of the infrastructure or railway operating safety requirements)
Maximum nominal speed	*Generally the maximum line speed at the site. Where specified for the individual project, a reduced speed may be used for checking individual real trains for their associated maximum permitted vehicle speed*
Maximum permitted vehicle speed	Maximum permitted speed of real trains due to vehicle considerations and generally independent of the infrastructure
Maximum train commissioning speed	Maximum speed used for testing a new train before the new train is brought into operational service and for special tests etc. The speed generally exceeds the maximum permitted vehicle speed and the appropriate requirements are to be specified for the individual project
Resonant speed	Traffic speed at which a frequency of loading (or a multiple thereof) matches a natural frequency of the structure (or a multiple thereof)
Tracks	Tracks include rails and sleepers. They are laid on a ballast bed or are directly fastened to the decks of bridges. The tracks may be equipped with expansion joints at one end or both ends of a deck. The position of tracks and the depth of ballast may be modified during the lifetime of bridges, for the maintenance of tracks

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cl. 6.3: EN 1991-2
cl. 6.4: EN 1991-2
cl. 6.5.1: EN 1991-2
cl. 6.5.2: EN 1991-2
cl. 6.5.3: EN 1991-2
cl. 6.5.4: EN 1991-2
cl. 6.6: EN 1991-2

6.4. General comments for the design of railway bridges

Railway bridges should be designed for the relevant rail traffic actions defined in ***Clause 6.3: EN 1991-2***. General rules are given for the calculation of the associated dynamic effects including resonance, centrifugal forces, nosing force, traction and braking forces, interaction of structure and track and aerodynamic actions due to passing rail traffic.

6.4.1. Design situations

Appropriate combinations of actions should be taken into account for the design of railway bridges that correspond to the real conditions occurring during the corresponding time period, corresponding to:

- Persistent design situations, generally covering the conditions of normal use with a return period equal to the intended design working life of the structure.
- Transient design situations, corresponding to temporary conditions applicable to the structure with a return period much shorter than the design working life of the structure (including consideration of the execution of the structure, where a structure is brought into use in stages to carry railway traffic loading etc. before construction is completed and loading requirements associated with maintenance of the bridge and tracks etc.).
- Accidental design situations, including exceptional conditions, applicable to the structure including consideration of derailment on or in the vicinity of the bridge, impact from errant road traffic on the bridge etc. and other relevant international and national requirements.
- Seismic design situations, where required in accordance with national requirements.
- Any other design situations as required by the relevant authority. The relevant authority should specify:
  - requirements relating to temporary bridges
  - the intended design working life of a structure which should generally be at least 100 years.

6.4.2. Combinations of actions

Annex 2: EN 1991-2

Generally, the design of a railway bridge should be verified using the partial factor method in accordance with ***EN 1990 Annex A2***.[3] Guidance on appropriate combinations of actions to be taken into account when using the Eurocodes is given in Chapter 8 of this Designers' Guide. Generally each action is considered in turn as a leading action with other actions taken as accompanying actions. Groups of loads for rail traffic actions are covered in Section 6.12.2 below.

6.4.3. Additional loading considerations

In addition, the design of a railway bridge should take into account the relevant loading:

- associated with the construction of the bridge
- appropriate to the stage of construction
- appropriate to the use of the bridge where the structure is brought into use in stages prior to the completion of construction
- requirements for temporary loading situations defined by the relevant authority associated with track maintenance, replacement of bearings etc.

6.4.4. Design acceptance criteria and limit states

Basic requirements relating to the design of railway bridges should be in accordance with the structural resistance, serviceability, durability, fitness for intended use, avoidance of damage from events not disproportionate to original cause etc.

Generally the design of a railway bridge should consider the following limit states:

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- the ultimate limit states associated with collapse of all or part of the structure and other similar forms of structural failure (e.g. buckling failure, loss of equilibrium, rupture, excessive deformation, failure or excessive deformation of the supporting ground etc.)
- fatigue failure of all or part of the structure
- serviceability limit states
- checks on design criteria relating to ensuring the safety of railway traffic.

6.5. General comments regarding characteristic values of railway actions

Rail loads have been developed using deterministic methods.

The values of γ and ψ factors given in Chapter 8 of this Designers' Guide are based on comparing calibration studies against a selection of European codes using the limit states method, which in turn have been generally based on empirical and historical (including permissible stress design codes) methods.

The comparative studies were carried out to support the drafting of the provisional version of the Eurocode (ENV 1991-3) and no further comparative studies have been carried out by the UIC to support the conversion of ENV 1991-3 to EN 1991-2 and EN 1990 Annex A2.

In Section 6.6 below, nominal values of actions due to rail traffic are given.

Subject to the loadings specified in Section 6.6 being enhanced by appropriate partial factors, the **nominal loadings are considered as characteristic values**.

Requirements for either considering:

- a mean value of an action,
- or where the variability is significant, upper and lower bound values

should be in accordance with the relevant international or national requirements.

Example 6.1. Variability of an action which is significant for railway bridges (see *1991-1-1, 5.2.3(2)*)

cl. 5.2.3(2): EN 1991-1-1

To take account of the variability of ballast depth, an additional factor of either 1.30 (ballast load effect unfavourable) or 0.70 (ballast load effect favourable) should be applied to the nominal depth of ballast beneath the underside of the sleeper.

The minimum and maximum nominal depths of ballast beneath the sleeper to be taken into account should be specified by the relevant authority.

Any additional ballast provided below the nominal depth of ballast may be considered as an imposed movable load. Additionally, the ballast density (or range of ballast densities) to be taken into account should be specified by the relevant authority.

6.6. Rail traffic actions and other actions for railway bridges

6.6.1. Field of application

This clause applies to rail traffic on the standard and wide track gauge.

The load models defined in this section do not describe actual loads. They have been selected so that their effects, with dynamic increments taken into account separately, represent the effects of service traffic. Where traffic outside the scope of the load models specified in this section needs to be considered, then alternative load models, with associated combination rules, should be specified for the particular project.

The load models are not applicable for action effects due to:

- narrow-gauge railways
- tramways and other light railways
- preservation railways

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- rack-and-pinion railways
- funicular railways.

Designers should pay special attention to temporary bridges because of the very low stiffness of the usual types of such structures. The loading and requirements for the design of temporary bridges should be specified in the National Annex.

6.6.2. Representation of actions – nature of rail traffic loads

In this Designers' Guide load models due to railway traffic are given for:

- vertical loads: LM71, LM SW (SW/0 and SW/2), and 'unloaded train'
- vertical loads for earthworks
- dynamic effects
- centrifugal forces
- nosing force
- traction and braking forces
- track–bridge interaction (based on UIC Code 774-3[8])

cl. 6.6: EN 1991-2

- aerodynamic effects are only mentioned (Design values see Clause 6.6: EN 1991-2)
- actions due to overhead line equipment and other railway infrastructure and equipment (note that these are also only mentioned without giving design values)
- derailment (accidental design situations):
  - the effect of rail traffic derailment on a structure carrying rail traffic (based on UIC Code 776-1[6])

cl. 4.6: EN 1991-1-7

  - for the effect of rail traffic derailment under or adjacent to a structure see Clause 4.6: EN 1991-1-7 and UIC Code 777-2[11].

6.7. Vertical loads – characteristic values (static effects) and eccentricity and distribution of loading

Recommendations concerning the application of traffic loads on railway bridges are given in Section 6.12 below.

6.7.1. General

Rail traffic actions are defined by means of load models. Four models of railway loading are given:

- LM71 and LM SW/0 (for continuous bridges) to represent normal rail traffic on mainline railways (passenger and heavy freight traffic)
- LM SW/2 to represent abnormal loads or waggons
- LM 'unloaded train' to represent the effect of an unloaded train
- LM HSLM (comprising HSLM-A and HSLM-B) to represent the loading from passenger trains at speeds exceeding 200 km/h.

cl. 6.3.2: EN 1991-2

6.7.2. Load Model 71

LM71 represents the static effect of vertical loading due to normal rail traffic.

The load arrangement and the characteristic values for vertical loads have to be taken as shown in Fig. 6.2.

cl. 6.3.2.3P: EN 1991-2

The characteristic values given in Fig. 6.1 needs to be multiplied by a factor α, on lines carrying rail traffic which is heavier or lighter than normal rail traffic. When multiplied by the factor α the loads are called 'classified vertical loads'. This factor α is one of the following:

0.75, 0.83, 0.91, 1.00, 1.10, 1.21, ***1.33****, 1.46*

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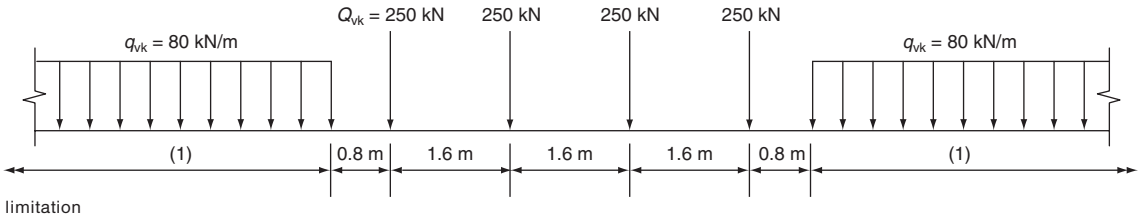


Fig. 6.2. Load Model 71 and characteristic values for vertical loads (Reproduced from EN 1991-2, with permission from BSI)

For international lines, it is recommended that a value of $\alpha \geq 1.0$ is adopted. The factor α may be specified in the National Annex or for the individual project.

This freedom of choice of the factor α could lead to a non-uniform railway network in Europe! Therefore in UIC Code 702[4] α = 1.33 is generally recommended for all new bridges constructed for the international freight network, but unfortunately is not compulsory! So all European railway authorities should immediately recommend this value in their National Annexes to develop a uniform European network for the next 100 years. This value takes into account the gradual increase of axle loads from 25 t today (2009) up to 30 t in the coming decades.

The actions listed below, associated with LM71, have to be multiplied by the same factor α: *cl. 6.3.2.3P: EN 1991-2*

- equivalent vertical loading for earthworks and earth pressure effects
- centrifugal forces
- nosing force (multiplied by α for $\alpha \geq 1$ only)
- traction and braking forces
- derailment actions for accidental design situations
- Load Model SW/0 for continuous span bridges.

The following should also be noted:

- **Attention to a mistake in EN 1991-2:** the combined response (interaction) of structure and track to variable actions has to be calculated with $\alpha = 1.0$, see remarks below and in Section 6.9.4.
- For checking limits of deformations, like twist, classified vertical loads and other actions are in general enhanced by α (except for passenger comfort where α is be taken as unity); however, for checking limits of deflections due to the strong and simplified method given in Section 8.7.4 of this Designer's Guide, for speeds up to 200 km/h, α is be taken equal to 1, even if other calculations (see above) are undertaken with $\alpha = 1.33$.

Specific and practical recommendations for using the classification factor α:
Ultimate limit states (ULS):
For the design of new bridges $\alpha = 1.33$ shall be adopted. Reductions should only be allowed by the relevant authority where justified.

For the assessment of existing bridges with a residual life of about 50 years $\alpha = 1.0$ should generally be adopted when they are strengthened. For bridges with a longer residual life, $\alpha = 1.33$ should be adopted.

Interaction track – bridge:
Theoretically this is a seviceability limit state (SLS) for the bridge and an ultimate limit state (*railway traffic safety*) for the rail. For bridge–track interaction the permissible additional rail stresses and deformations are calibrated on the existing practice. Forces and displacements must be calculated using the partial safety factors of the loads concerned. However, as the given permissible rail stresses and deformations were obtained by deterministic design methods, calibrated on the existing practice, the calculations for interaction should

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not be carried out with $\alpha = 1.33$ but – contrary to EN 1991-2 – always with $\alpha = 1.0$. Axle loads of 30 t will come only in a hundred years' time and we do not know what the track characteristics will be so far ahead in the future. The calculations with $\alpha = 1.0$ have sufficient reserves, so that in the foreseeable future no supplementary expansion joints will be necessary for bridges calculated with $\alpha = 1.0$ today.

Seviceability limit states (SLS) for permissible deflections:
With the severe (it will be explained later that this will not increase the price of the structure) permissible deflection recommended in Section 8.7.4 below, the value $\alpha = 1.0$ must be adopted together with LM71 (and SW/0 if relevant), even if $\alpha = 1.33$ is adopted for ULS design.

Fatigue:
All verifications should be performed with LM71, the basic load model for fatigue considerations, and with a value $\alpha = 1.0$, even if $\alpha = 1.33$ is adopted for ULS design.

Background information to the above-mentioned practical recommendations
Concerning heavy haul and higher axle loads on bridges, the following can be reported concerning the existing situation inside UIC.

In the actual UIC Code 700[5] (March 2003) one can find axle loads of 25 t and nominal loads per metre of 8.8 t/m (see class E5 in the following Table 6.1). These are currently the maximum loads for regular traffic.

Table 6.1. Existing classification of lines and load limits for wagons (Simplified presentation, not showing the importance of spaces between the axle loads)

Classification due to UIC Leaflet 700		Mass per axle = P				
Mass per length = p		A 16 t	A 18 t	C 20 t	D 22 t	E 25 t
1	5.0 t/m^2	A	B1			
2	6.4 t/m^2		B2	C2	D2	
3	7.2 t/m^2			C3	D3	
4	8.0 t/m^2			C4	D4	E4
5	8.8 t/m^2					E5

Due to the 100-year lifetime of bridges it is necessary to take into account long-term considerations. Having made a decision about future loads, in terms of new bridges there are no significant design or cost problems. More significant problems arise however when it is necessary to upgrade existing lines where there is a need to modify or strengthen bridges. Nevertheless, the step up to 25 t nominal axle load and 8 t/m (class E4) is in this case covered by the existing UIC Load Model 71 (with $\alpha = 1.0$). For nominal loads greater than 25 t and 8 t/m, completely new considerations have to be taken into account and the renewal of existing constructions will be necessary in most cases. In 1991 the ERRI (European Rail Research Institute of the UIC) expert group D192 commenced research into long-term considerations of bridge loading and ERRI D192/RP1[12] contains an initial forecast of expected future loads in Europe. The maximum values predicted by the different railway administrations were 30 t axle loads and a mass per length of 15 t/m. These values were at that time revolutionary, but nowadays (2009) axle loads of 30 t already exist in a few parts of the European network and heavy abnormal waggons with a mass per length of 15 t/m are reality. The ERRI expert group D192 also carried out a profitability study (D192/RP4[13]) to determine the effect of higher axle loads on the overall costs of bridges. Fifteen existing bridges were designed for two load cases, the first using LM71, the second using a 40% ($\alpha \cong 1.4$) higher design load. The overall

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costs (project and survey, temporary works, overhead work, signalling installations, site overhead costs, site equipment, foundations, piers, abutments, superstructure, bridge equipment) were compared. The results are shown in Fig. 6.3.

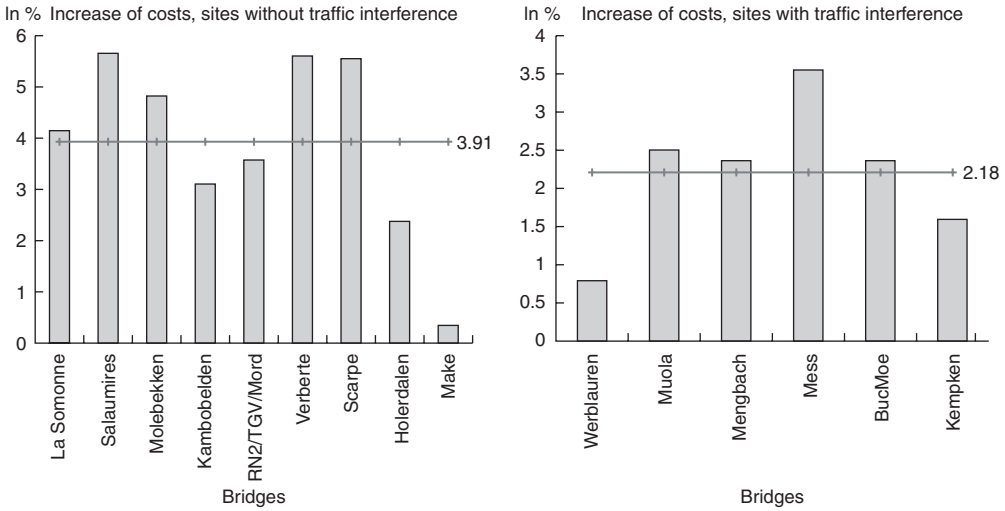


Fig. 6.3. ERRI D192/RP4: Construction costs increase due to a mean load increase of 40%

The cost increase was about 4% for bridges built without traffic interference and about 2% for bridges built with traffic interference (see Fig. 6.3). The overall initial investment costs for bridges therefore only changes slightly. Taking into account the fact that the 30 t axle loads will not be introduced for some decades, life-cycle cost (LCC) considerations give a neutral cost result. A slightly overdesigned bridge has less fatigue problems if the loadings are increasing slowly or not at all. A second study was undertaken in Switzerland in 2002, where all bridges for the two new alpine lines (St Gotthard and Lötschberg) were calculated with LM71 and $\alpha = 1.33$. The additional amount for investments gave an increase in costs of 3% mean value and the decision was taken to adopt $\alpha = 1.33$, not only for all the bridges of the new alpine lines but also for all future bridges on all other lines in Switzerland ('Swisscodes', SIA 261, SN 505 261[14]).

The results of the ERRI D192 expert group have not sufficiently influenced the Eurocodes and UIC Codes developed later. The classification factor of $\alpha = 1.0$ or 1.1 specified for LM71 is a minimum solution and corresponds to a maximum nominal load of 22.5 t or 25 t and a mass of 8 t/m or 8.8 t/m, which correspond to class D4/E5 of UIC Code 700.[5] Most railways wanted to have the same α classification factor greater than 1.0 for the whole of Europe, but unfortunately there was no consensus between railway administrations for the introduction of a uniform higher design load for Europe. The introduction of a new 30 t UIC Load Model 2000 is foreseen for future revision of the Eurocodes. It will be a difficult exercise with high costs. Nevertheless, some countries wanted to take account of the trend towards higher axle loads and therefore already apply an α value greater than 1.0. This could lead to future non-uniformity for heavy haul in the European railway network, as Fig. 6.4 shows. Therefore a clear definition of the European rail freight network has to be worked out, fixing both the maximum load and speed.

In 2003, an important recommendation was given in UIC Code 702: *Static loading diagrams to be taken into consideration for the design of rail-carrying structures on lines used by international services*.[4] In this recently revised version it gives clear recommendation for higher axle loads. For the future rail freight network it is recommended that the UIC LM 2000 is used. This has no basis in current Eurocodes, so for the present, **1.33** $\times$ LM71 is recommended (Fig. 6.5).

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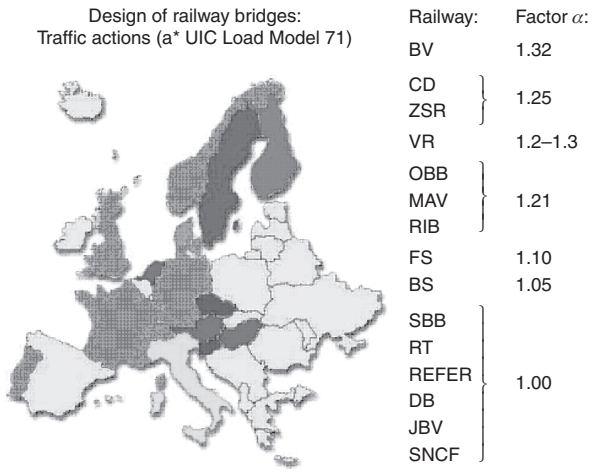


Fig. 6.4. Characteristic vertical traffic loads ($\alpha \times LM71$) for railway bridges in Europe, situations in the year 2002, note the inhomogeneous network

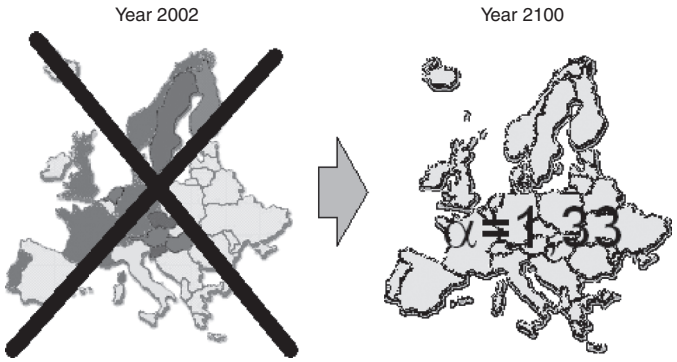


Fig. 6.5. Vision of future European railway network

This vision is of great importance for the interoperability and efficiency of the European rail infrastructure in the future.

Bridges represent just one element of the infrastructure and their upgrading could be called into question if there is no commercial thinking behind it. However, on the basis of

- the growing trend towards heavier and ever increasing numbers of traffic
- the EU policy of moving transport away from roads and onto the railways
- the axle loads permitted, for instance in North America,

it can be expected that, as in the past, traffic load, speed and frequency will increase in the medium term.

Conclusion

Heavier loads do not significantly influence the investment costs of bridges and the influence is zero taking life-cycle costs into consideration.

For the reasons mentioned above, the factor $\alpha = 1.33$ should be adopted for all the European freight railway network.

cl. 6.3.3: EN 1991-2

6.7.3. Load Models SW/0 and SW/2

Load Model SW/0 represents the static effect of vertical loading due to normal rail traffic on continuous beams.

Load Model SW/2 represents the static effect of vertical loading due to heavy abnormal rail traffic.

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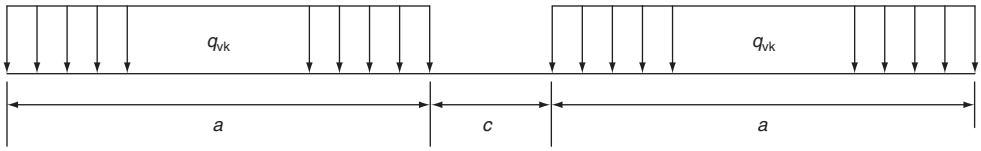


Fig. 6.6. Load Models SW/0 and SW/2 (Reproduced from EN 1991-2, with permission from BSI)

Table 6.2. Characteristic values for vertical loads for Load Models SW/0 and SW/2

Load model	q_{vk} (kN/m)	a (m)	c (m)
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

The load arrangement is as shown in Fig. 6.6, with the characteristic values of the vertical loads according to Table 6.2.

The lines or sections of line over which heavy abnormal rail traffic may operate where Load Model SW/2 needs to be taken into account have to be chosen by the relevant authority. *cl. 6.3.3(4)P: EN 1991-2*

Note: It is better if the relevant authority designates the sections of line for which LM SW/2 needs ***not*** *to be taken into account, or, even better, that LM SW/2 has to be adopted on all the lines. Remember: it costs not more if heavier loads are taken into consideration for building new bridges. We do not know the future evolution of freight traffic, but traffic with 30 t axle loads should be possible in the next 100 years. Life-cycle cost studies have proved that this can be done in an economic way.*

6.7.4. Load Model 'unloaded train'

cl. 6.3.4: EN 1991-2

For some specific verification purposes a specific load model is used, called 'unloaded train'. The Load Model 'unloaded train' consists of a vertical uniformly distributed load with a characteristic value of 10.0 kN/m.

Note: This case can be determinant for single-track bridges with small width and large height, when considering the limit state of static equilibrium of the whole bridge and with wind as a leading action.

6.7.5. Eccentricity of vertical loads (Load Models 71 and SW/0)

cl. 6.3.5: EN 1991-2

The effect of lateral displacement of vertical loads (unbalanced or asymmetric loading of waggons) needs to be considered by taking the ratio of wheel loads on all axles as up to 1.25:1.0 on any one track.

The above criteria may be used to determine the eccentricity of loading with respect to the centre-line of the track.

Note: See ***Clause 6.8.1: EN 1991-2*** for requirements relating to the geometric position of the tracks, eventually giving supplementary eccentricities. *cl. 6.8.1: EN 1991-2*

6.7.6. Distribution of axle loads by rails, sleepers and ballast

The distribution of axle loads by the rails, sleepers and ballast is clearly defined in ***Clause 6.3.6: EN 1991-2***. *cl. 6.3.6: EN 1991-2*

Note (1): For the design of local floor elements (longitudinal and transverse ribs of orthotropic deck plates, thin concrete slabs, etc.), the longitudinal distribution beneath sleepers as shown in EN 1991-2, Fig. 6.5 should be taken into account. For that, the single axles of LM71 (250 kN) must be taken as point loads.

Note (2): For the load distribution in the transverse direction, full-length sleepers may be adopted in general, when not specified by the relevant authority.

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cl. 6.3.6.4: EN 1991-2

6.7.7. Equivalent vertical loading for earthworks and earth pressure effects

For global effects, the equivalent characteristic vertical loading due to rail traffic actions for earthworks under or adjacent to the track may be taken as the appropriate load model (LM71, or classified vertical load where required, and SW/2 where required) uniformly distributed over a width of 3.00 m at a level 0.70 m below the running surface of the track.

No dynamic factor or increment needs to be applied to the above uniformly distributed load.

For the design of local elements close to a track (e.g. ballast retention walls), a special calculation should be carried out taking into account the maximum local vertical, longitudinal and transverse loading on the element due to rail traffic actions.

cl. 6.3.7: EN 1991-2

6.7.8. Actions for non-public footpaths

Non-public footpaths are those designated for use by only authorized persons. Pedestrian, cycle and general maintenance loads should be represented by a uniformly distributed load with a characteristic value $q_{fk} = 5\,kN/m^2$.

For the design of local elements a concentrated load $Q_k = 2.0\,kN$ acting alone should be taken into account and applied on a square surface with a 200 mm side.

Horizontal forces on parapets, partition walls and barriers due to persons should be taken as category B and C1 of EN 1991-1-1.

6.7.10. Loading for public railway platforms

The loading for public railway platforms should be in accordance with the requirements of the railway authority.

Note: The platforms should sustain all actions and influences likely to occur during use. If the possibility exists that road vehicles can gain access, this should be considered for the design.

6.8. Dynamic effects

6.8.1. General

Three dynamic factors/dynamic enhancements are defined in EN 1991-2:

Annex C (normative): EN 1991-2

- Dynamic factor $1 + \varphi$
  This is a physically determined dynamic factor for real trains. The dynamic enhancement φ is a function of the speed of the train, the natural frequency of the non-loaded bridge, as well as the determinant length (see Table 6.3 below). It is the dynamic factor for real trains to assess existing bridges, a basis for determining the dynamic factor Φ for LM71, SW/0 and SW/2 and also for calculating damage equivalent factors for fatigue. It is normally not directly used for designing new bridges.

Table 6.2: EN 1991-2

cl. 6.4.5: EN 1991-2

- Dynamic factor Φ
  This is used for designing new bridges, together with load models LM71, SW/0 and SW/2. It takes into account static and dynamic effects of different real trains. It is defined as a function of the determinant length and depends on the quality of track.

cl. 6.4.6.5.(3): EN 1991-2

- Dynamic enhancement $\varphi'_{dyn} = \max|y_{dyn}/y_{stat}| - 1$
  This enhancement is only used when dynamic analysis is necessary to check if the calculated load effects from high-speed rail traffic are greater than the load effects due to normal rail bridge loading.

The name dynamic factor for Φ is misleading because it covers not only dynamic effects but also a part of the static loads of the six standard trains defined in UIC Code 776-1,[6] which are represented in Annex A6.1 of this chapter. The relation between the dynamic enhancement $1 + \varphi$ and the dynamic factor Φ is given by:

$$(1 + \varphi)S_{\text{real trains } 1-6} \leq \Phi S_{LM71}$$

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where S as an elastomechanical action effect for M (moment), Q (shear force), y (deflection), σ (normal stress), τ (shear stress), ε (strain) and γ (shear deformation) at a point of the structural component.

So the determination of Φ is arrived at over the inequality:

$$\Phi \geq S_{\text{real train } 1-6}(1+\varphi_{1-6})/S_{\text{LM71}}$$

Annex C (normative): EN 1991-2

6.8.2. Dynamic factors $1+\varphi$ for real trains

An ORE (Office of Research and Experiments of the UIC, later called ERRI) Specialists' Committee provided the basis for determining the dynamic enhancement φ and the dynamic factor Φ. Its work was supplemented by model tests and theoretical studies, especially in those areas which were not covered by line tests. The accuracy of the results of the theoretical studies was confirmed by tests (see ORE Report D128/RP3[15]).

The laws were deduced from the behaviour of a simply supported beam. They cover most of the effects in continuous girders and other structures; where this is not the case, they are taken into account by the values given for the so-called determinant length L_Φ.

When service trains pass over a bridge, the resulting oscillations increase the load by a quantity φ made up of two components as follows:

φ' is the proportion applicable for a perfect level track
φ'' is the proportion representing the effects of vertical track irregularities and the response of vehicle unsprung mass.

The static load due to real trains at v (m/s) has to be multiplied by:

$1+\varphi = \varphi' + \varphi''$ for track with standard maintenance — EN 1991-2, (C1)

$1+\varphi = \varphi' + 0.5\varphi''$ for carefully maintained track — EN 1991-2, (C2)

The value φ' is given by the following formula:

with

$$\varphi' = \frac{K}{1-K+K^4} \text{ for } K < 0.76 \qquad \text{EN 1991-2, (C3)}$$

and

$$\varphi' = 1.325 \text{ for } K \geq 0.76 \qquad \text{EN 1991-2, (C4)}$$

where

$$K = \frac{v}{2L_\Phi \times n_0} \qquad \text{EN 1991-2, (C5)}$$

The following formula was established on the basis of theoretical studies to take account of the track irregularities:

$$\varphi'' = \frac{\alpha}{100}\left[56\,\mathrm{e}^{-(L_\Phi/10)^2} + 50\left(\frac{L_\Phi n_0}{80}-1\right)\mathrm{e}^{-(L_\Phi/20)^2}\right] \qquad \text{EN 1991-2, (C6)}$$

$$\varphi'' \geq 0$$

$$\alpha = \frac{v}{22} \quad \text{if } v \leq 22\,\text{m/s}\ (\sim 80\,\text{km/h}) \qquad \text{EN 1991-2, (C7)}$$

$$\alpha = 1 \quad \text{if } v > 22\,\text{m/s}$$

where

v is speed in m/s
L_Φ in the case of a main simple beam with two bearings, is the span in m
in other cases, the value L_Φ in ***EN 1991-2, Table 6.2*** should be used instead of L in the calculation. This also applies to the assessment of old bridges if service trains are used as live loads — *Table 6.2: EN 1991-2*
n_0 is the natural frequency of the unloaded bridge (s^{-1})
e base of natural logarithms (2.71828 . . .)

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C3: EN 1991-2 The term φ' in equation ***EN 1991-2, (C3)*** covers about 95% of the values studied, giving a statistical confidence limit of 95% (approximately mean value plus two standard deviations).

C6: EN 1991-2 The term φ'' in equation ***EN 1991-2, (C6)*** has been fixed by assuming a vertical dip in the track of 2 mm over a length of 1 m or 6 mm over a length of 3 m, and an unsprung mass of 2 t per axle.

The equations given represent upper bounds which may, however, be exceeded by at the most 30% in particular cases, such as very high-speed trains or long wheelbase vehicles, while only half these values are reached in the case of special vehicles with closely spaced axles.

Generally speaking, these effects are not predominant but they should be taken into account when calculating bridges for the acceptance of actual trains. It is particularly important to take this fact into account for short-span bridges.

The dynamic factors Φ for the LM71 are calculated from the dynamic enhancement φ for the chosen service trains given in Annex 1 of this chapter, so that the loads of LM71 multiplied by Φ cover the loads of actual trains multiplied by $(1 + \varphi)$ with sufficient safety (see also the equation in Section 6.8.1 above).

The values $\varphi = \varphi' + \varphi''$ have been calculated for bridges with high and low natural frequencies, taking the most unfavourable values. The frequencies used are given below and shown in ***EN 1991-2, Fig. 6.10***.

Fig. 6.10: EN 1991-2

The limit of validity for φ' is the lower limit of natural frequency and 200 km/h. For all other cases φ' should be determined by a dynamic analysis in accordance with Annex B of this chapter (see also UIC Code 776-2[7]).

Fig. 6.10: EN 1991-2 The limit of validity for φ'' is the upper limit of natural frequency in ***EN 1991-2, Fig. 6.10***. For all other cases φ'' may be determined by a dynamic analysis taking into account mass interaction between the unsprung axle masses of the train and the bridge in accordance with Annex B of this chapter.

The values of $\varphi' + \varphi''$ have to be determined using upper and lower limiting values of n_0, unless they are being undertaken for a particular bridge of known first natural frequency.

The upper limit of n_0 is given by:

$$n_0 = 94.76 L_\Phi^{-0.748} \qquad \text{EN 1991-2, (C8)}$$

and the lower limit is given by:

$$n_0 = \frac{80}{L_\Phi} \quad \text{for } 4\,\text{m} < L_\Phi \leq 20\,\text{m} \qquad \text{EN 1991-2, (C9)}$$

$$n_0 = 23.58 L_\Phi^{-0.592} \quad \text{for } 20\,\text{m} < L_\Phi \leq 100\,\text{m} \qquad \text{EN 1991-2, (C10)}$$

Damping was taken to correspond to logarithmic decrements from 0.0 to 1.0.

Service trains have been divided into six representative types for which standard speeds have been set. These six types of service train are given in Annex A6.1 of this chapter. The maximum loadings in relation to span were obtained for three of the six standard trains. However, the effects of all six standard trains should be taken into account for checking purposes.

The values of L_Φ were based on the influence line for the deflection of the member to which the calculations refer. In the case of asymmetrical influence lines, the formula to be applied is as given in Fig. 6.7. The definition of $l_\Phi = 2 \times (a + 1.5)$ is based on the assumption that a structure with a symmetrical influence line and the same maximum value will produce the same dynamic effect. This follows from the fact that the dynamic effects depend on the slope of the influence line at the bearing. To allow for the effect of distribution of the load by the rails, the value is increased by $2 \times 1.50 = 3.00$ m.

The following should be noted:

- ***Dynamic enhancement for the assessment of existing bridges***

  C3 to C6: EN 1991-2 In assessing existing bridges, equations ***EN 1991-2, C3 to C6*** can be used to determine dynamic factors $1 + \varphi$ of Real Trains.

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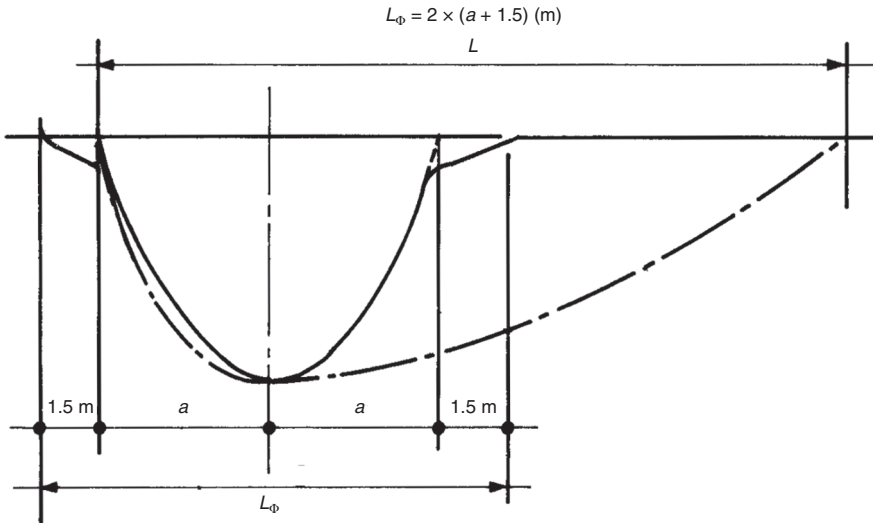


Fig. 6.7. L_Φ for asymmetrical influence lines

When assessing the strength of old lattice girder bridges, account must be taken of the fact that secondary vibrations occur in flexible diagonals (formed of flats) which result in stress increases at the extreme fibres. To allow for this, it is recommended that a stress of $5\,N/mm^2$ for speeds of $V < 50\,km/h$ and a stress of $10\,N/mm^2$ for higher speeds be added to the stresses calculated for the live load and the dynamic effect.

For special trains with a large number of axles and a total weight of more than 400 t, a dynamic enhancement φ of 0.10 to 0.15 may be added if more accurate calculations are not carried out and if such trains travel at speeds of 40 km/h or less.

- ***Dynamic enhancement for fatigue assessment, e.g. for calculating damage equivalent values λ with real trains***

  To take account of the average effect over the assumed 100-year life of the structure, the dynamic enhancement for each real train may be reduced to medium values of dynamic enhancements, as follows:

  $\varphi = 1 + \frac{1}{2}(\varphi' + \frac{1}{2}\varphi'')$ for carefully maintained track

6.8.3. Dynamic factor $\Phi(\Phi_2, \Phi_3)$

cl. 6.4.5: EN 1991-2

The dynamic factor Φ takes account of the dynamic magnification of stresses and vibration effects in the structure but does not take account of resonance effects.

The natural frequency of the structure should be within the frequency limits given in ***EN 1991-2, Fig. 6.10***. Where the criteria specified are not satisfied there is a risk that resonance or excessive vibration of the bridge may occur (with a possibility of excessive deck accelerations leading to ballast instability etc. and excessive deflections and stresses etc.). For such cases a dynamic analysis has to be carried out to calculate impact and resonance effects (see Annex B of this chapter).

Fig. 6.10: EN 1991-2

Structures carrying more than one track should be considered without any reduction of dynamic factor Φ.

Generally the dynamic factor Φ is taken as either Φ_2 or Φ_3 according to the quality of track maintenance as follows:

(a) For carefully maintained track:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \qquad \text{EN 1991-2, (6.4)}$$

with $1.00 \leq \Phi_2 \leq 1.67$.

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(b) For track with standard maintenance:

$$\Phi_3 = \frac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73 \qquad \text{EN 1991-2, (6.5)}$$

with $1.00 \leq \Phi_3 \leq 2.0$; where L_Φ is the 'determinant' length (length associated with Φ) in metres as defined in Table 6.3 below (EN 1991-2, Table 6.2).

The following comments should be noted:

- The dynamic factors were established for simply supported girders. The length L_Φ allows these factors to be used for other structural members with different support conditions.
- If no dynamic factor is specified, Φ_3 is be used.

For steel bridges with so-called open deck, i.e. with wooden sleepers on rail bearers and cross-girders, Φ_3 should be taken for the end cross girders and cantilevers of rail bearers, even for carefully maintained track.

- The dynamic factor Φ must not be used with:
  - the loading due to real trains
  - the Load Model 'unloaded train'.
- The determinant lengths L_Φ to be used are given in Table 6.3 below. Where no value for L_Φ is specified in the table, the length of the influence line for deflection of the element being considered may be taken as the determinant length.
  If the resultant stress in a structural member depends on several effects, each of which relates to a separate structural behaviour, then each effect should be calculated using the appropriate determinant length.

Permissible reductions of dynamic factors Φ:

In the case of arch bridges and concrete bridges of all types with a cover of more than 1.0 m, Φ_2 and Φ_3 may be reduced as follows:

$$\Phi_{2.3} = \Phi_{2.3} - \frac{h - 1.00}{10} \geq 1.0 \qquad \text{EN 1991-2, (6.8)}$$

where h is the height of cover including the ballast from the top of the deck to the top of the sleeper (for arch bridges, from the crown of the extrados) (in metres).

The effects of rail traffic actions on columns with a slenderness (buckling length/radius of gyration) <30, abutments, foundations, retaining walls and ground pressures may be calculated without taking into account dynamic effects.

6.8.4. Dynamic enhancement $\varphi'_{\mathbf{dyn}} = \mathbf{max}\left|\mathbf{y_{dyn}}/\mathbf{y_{stat}}\right| - \mathbf{1}$

This enhancement is determined by a dynamic study (see Annex B of this Chapter).

One part consists in checking whether the calculated load effects from high-speed traffic are greater than corresponding load effects due to normal rail bridge loading. For the design of the bridge, taking into account all the effects of vertical traffic loads, the most unfavourable value of:

$$\left(1 + \varphi'_{\mathrm{dyn}} + \varphi''/2\right) \times \begin{pmatrix} \text{HSLM} \\ \text{or} \\ \text{RT} \end{pmatrix} \text{ or } \Phi \times \left(\text{LM71}'' + '' \text{SW/0}\right) \qquad \text{EN 1991-2, (6.15 and 6.16)}$$

should be used.

The following dynamic enhancement is determined from the dynamic analysis:

$$\varphi'_{\mathrm{dyn}} = \max\left|y_{\mathrm{dyn}}/y_{\mathrm{stat}}\right| - 1 \qquad \text{EN 1991-2, (6.14)}$$

where

y_{dyn} is the maximum dynamic response and y_{stat} the corresponding maximum static response at any particular point in the structural element due to a real train (RT) or high-speed load model (HSLM)

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Table 6.3. Determinant lengths L_Φ (Data taken from EN 1991-2, Table 6.2)

Case	Structural element	Determinant length L_Φ
Steel deck plate: closed deck with ballast bed (orthotropic deck plate) (for local and transverse stresses)		
	Deck with cross-girders and continuous longitudinal ribs:	
1.1	Deck plate (for both directions)	3 times cross-girder spacing
1.2	Continuous longitudinal ribs (including small cantilevers up to 0.50 m)[a]	3 times cross-girder spacing
1.3	Cross-girders	Twice the length of the cross-girder
1.4	End cross-girders	3.6 m[b]
	Deck plate with cross-girders only:	
2.1	Deck plate (for both directions)	Twice cross-girder spacing + 3 m
2.2	Cross-girders	Twice cross-girder spacing + 3 m
2.3	End cross-girders	3.6 m[b]
Steel grillage: open deck without ballast bed[b] (for local and transverse stresses)		
3.1	Rail bearers:	
	• as an element of a continuous grillage	3 times cross-girder spacing
	• simply supported	Cross-girder spacing + 3 m
3.2	Cantilever of rail bearer[a]	3.6 m
3.3	Cross-girders (as part of cross-girder/continuous rail bearer grillage)	Twice the length of the cross-girder
3.4	End cross-girders	3.6 m[b]
Concrete deck slab with ballast bed (for local and transverse stresses)		
4.1	Deck slab as part of box girder or upper flange of main beam:	
	• spanning transversely to the main girders	3 times span of deck plate
	• spanning in the longitudinal direction	3 times span of deck plate
	• cross girders	Twice the length of the cross-girder
	• transverse cantilevers supporting railway loading	• $e \leq 0.5$ m: 3 times the distance between the webs • $e > 0.5$ m[a]
4.2	Deck slab continuous (in main girder direction) over cross-girders	Twice the cross-girder spacing
4.3	Deck slab for half-through and trough bridges:	
	• spanning perpendicular to the main girders	Twice span of deck slab + 3 m
	• spanning in the longitudinal direction	Twice span of deck slab
4.4	Deck slabs spanning transversely between longitudinal steel beams in filler beam decks	Twice the determinant length in the longitudinal direction
4.5	Longitudinal cantilevers of deck slab	• $e \leq 0.5$ m: 3.6 m[b] • $e > 0.5$ m[a]
4.6	End cross-girders or trimmer beams	3.6 m[b]

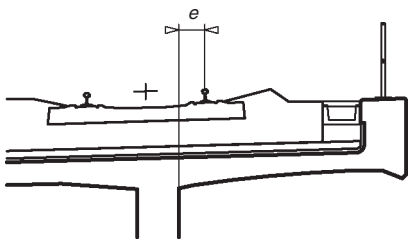


Fig. 6.8. Transverse cantilever supporting railway loading (Reproduced from EN 1991-2, with permission from BSI)

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Table 6.3 (continued)

Case	Structural element	Determinant length L_Φ
Main girders		
5.1	Simply supported girders and slabs (including steel beams embedded in concrete)	Span in main girder direction
5.2	Girders and slabs continuous over n spans with $L_m = 1/n(L_1 + L_2 + \ldots + L_n)$	$L_\Phi = k \times L_m$, but not less than max L_i $(i = 1, \ldots, n)$ $n = 2 \quad 3 \quad 4 \quad 5$ $k = 1.2 \quad 1.3 \quad 1.4 \quad 1.5$
5.3	Portal frames and closed frames or boxes:	
	• Single-span	Consider as three-span continuous beam (use 5.2, with vertical and horizontal lengths of members of the frame or box)
	• Multi-span	Consider as multi-span continuous beam (use 5.2, with lengths of end vertical members and horizontal members)
5.4	Single arch, arch rib, stiffened girders of bowstrings	Half span
5.5	Series of arches with solid spandrels retaining fill	Twice the clear opening
5.6	Suspension bars (in conjunction with stiffening girders)	4 times the longitudinal spacing of the suspension bars
Structural supports		
6	Columns, trestles, bearings, uplift bearings, tension anchors and for the calculation of contact pressures under bearings	Determinant length of the supported members

[a] In general all cantilevers greater than 0.50 m supporting rail traffic actions need a special study in accordance with EN 1991-2, 6.4.6 and with the loading agreed with the relevant authority specified in the National Annex.
[b] It is recommended to apply Φ_3.
Note: For Cases 1.1 to 4.6 inclusive L_Φ is subject to a maximum of the determinant length of the main girders.

LM71″+″SW/0	is Load Model 71 and if relevant Load Model SW/0 for continuous bridges (classified vertical load where required)
$\varphi''/2$	is defined in Section 6.8.2 above
Φ	is the dynamic factor in accordance with Section 6.8.3 above.

cl. 6.5: EN 1991-2

6.9. Horizontal forces – characteristic values

cl. 6.5.1: EN 1991-2

6.9.1. Centrifugal forces

Where the track on a bridge is curved over the whole or part of the length of the bridge, the centrifugal force and the track cant need to be taken into account.

The centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1.80 m above the running surface. For some traffic types, e.g. double stacked containers, the particular project should specify an increased value of h_t.

The centrifugal force should always be combined with the vertical traffic load. The centrifugal force must not be multiplied by the dynamic factor Φ_2 or Φ_3.

When considering the vertical effects of centrifugal loading, the vertical load effect of centrifugal loading less any reduction due to cant is enhanced by the relevant dynamic factor.

The characteristic value of the centrifugal force has to be determined according to the following equations:

$$Q_{tk} = \frac{v^2}{g \times r}(f \times Q_{vk}) = \frac{V^2}{127r}(f \times Q_{vk}) \qquad \text{EN 1991-2, (6.17)}$$

$$q_{tk} = \frac{v^2}{g \times r}(f \times q_{vk}) = \frac{V^2}{127r}(f \times q_{vk}) \qquad \text{EN 1991-2, (6.18)}$$

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where

Q_{tk}, q_{tk} are the characteristic values of the centrifugal forces (kN, kN/m)
Q_{vk}, q_{vk} are the characteristic values of the vertical loads specified in Section 6.7 above (excluding any enhancement for dynamic effects) for Load Models 71, SW/0, SW/2 and 'unloaded train'. For Load Model HSLM the characteristic value of centrifugal force should be determined using Load Model 71
f is the reduction factor (see below)
v is the maximum line speed at the site (in m/s). In the case of Load Model SW/2 an alternative maximum speed may be used (max. 22.22 m/s ($= 80$ km/h))
V is the maximum line speed at the site, as above, but in km/h
g is acceleration due to gravity (9.81 m/s^2)
r is the radius of curvature (m).

In the case of a curve of varying radii, suitable mean values may be taken for the value r.

The calculations have to be based on the maximum line speed at the site specified for the particular project.

In the case of Load Model SW/2 a maximum speed of 80 km/h may be assumed.

In addition, for bridges located in a curve, the case of the loading specified in Section 6.7.2 and, if applicable, in Section 6.7.3 need also to be considered without centrifugal force.

For Load Model 71 (and where required Load Model SW/0) and a maximum line speed at the site higher than 120 km/h, the following cases should be considered (see Table 6.4):

Case (a) Load Model 71 (and where required Load Model SW/0) with its dynamic factor and the centrifugal force for $V = 120$ km/h, with $f = 1$.
Case (b) A reduced Load Model 71 ($f \times Q_{vk}$, $f \times q_{vk}$) (and where required $f \times$ Load Model SW/0) with its dynamic factor and the centrifugal force for the maximum speed V specified, with a value for the reduction factor f given below.

For Load Model 71 (and where required Load Model SW/0) the reduction factor f is given by:

$$f = \left[1 - \frac{V - 120}{1000}\left(\frac{814}{V} + 1.75\right)\left(1 - \sqrt{\frac{2.88}{L_f}}\right)\right] \qquad \text{EN 1991-2, (6.19)}$$

subject to a minimum value of 0.35

where

L_f is the influence length of the loaded part of curved track on the bridge, which is most unfavourable for the design of the structural element under consideration (m)
V is the maximum line speed at the site

$$\left.\begin{array}{l} f = 1 \text{ for either } V \le 120\,\text{km/h or } L_f \le 2.88\,\text{m} \\ f < 1 \text{ for } 120\,\text{km/h} < V \le 300\,\text{km/h} \\ f_{(V)} = f_{(300)} \text{ for } V > 300\,\text{km/h} \end{array}\right\} \text{and } L_f > 2.88\,\text{m}$$

Table 6.7 or Fig. 6.16 or equation 6.19: EN 1991-2

For the Load Models SW/2 and 'unloaded train' the value of the reduction factor f should be taken as 1.0.

The criteria in the above paragraph are not valid for heavy freight traffic with a maximum permitted vehicle speed exceeding 120 km/h. For heavy freight traffic with a speed exceeding 120 km/h additional requirements should be specified.

6.9.2. Nosing force

cl. 6.5.2: EN 1991-2

The nosing force has to be taken as a concentrated force acting horizontally, at the top of the rails, perpendicular to the centre-line of track. It needs to be applied on both straight track and curved track.

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Table 6.4. Load cases for centrifugal force corresponding to values of α and maximum line speed at site (Data taken from EN 1991-2, Table 6.8)

Value of α	Maximum line speed at site (km/h)	Centrifugal force based on:*				Associated vertical traffic action based on:†
		V (km/h)	α	f		
$\alpha < 1$	>120	V	1‡	f	$1‡ \times f \times$ (LM71″+″SW/0) for case (b) above	$\Phi \times 1‡ \times f \times$ (LM71″+″SW/0)
		120	α	1	$\alpha \times 1 \times$ (LM71″+″SW/0) for case (a) above	$\Phi \times \alpha \times 1 \times$ (LM71″+″SW/0)
		0	–	–	–	
	≤120	V	α	1	$\alpha \times 1 \times$ (LM71″+″SW/0)	
		0	–	–	–	
$\alpha = 1$	>120	V	1	f	$1 \times f \times$ (LM71″+″SW/0) for case (b) above	$\Phi \times 1 \times 1 \times$ (LM71″+″SW/0)
		120	1	1	$1 \times 1 \times$ (LM71″+″SW/0) for case (a) above	$\Phi \times 1 \times 1 \times$ (LM71″+″SW/0)
		0	–	–	–	
	≤120	V	1	1	$1 \times 1 \times$ (LM71″+″SW/0)	
		0	–	–	–	
$\alpha > 1$	>120§	V	1	f	$1 \times f \times$ (LM71″+″SW/0) for case (b) above	$\Phi \times 1 \times 1 \times$ (LM71″+″SW/0)
		120	α	1	$\alpha \times 1 \times$ (LM71″+″SW/0) for case (a) above	$\Phi \times \alpha \times 1 \times$ (LM71″+″SW/0)
		0	–	–	–	
	≤120	V	α	1	$\alpha \times 1 \times$ (LM71″+″SW/0)	
		0	–	–	–	

* See the third paragraph of Section 6.9.1 regarding vertical effects of centrifugal loading. Vertical load effect of centrifugal loading less any reduction due to cant should be enhanced by the relevant dynamic factor. When determining the vertical effect of centrifugal force, factor f is to be included as shown above.
† $0.5 \times$ (LM71″+″SW/0) instead of (LM71″+″SW/0) where vertical traffic actions favourable.
‡ $\alpha = 1$ to avoid double-counting the reduction in mass of train with f.
§ Valid for heavy freight traffic limited to a maximum speed of 120 km/h

where

V	is the maximum line speed at site (km/h)
f	is the reduction factor
α	is the factor for classified vertical loads in accordance with Section 6.7.2
LM71″+″SW/0	is Load Model 71 and if relevant Load Model SW/0

The characteristic value of the nosing force is to be taken as $Q_{sk} = 100\,\text{kN}$. It must not be multiplied by the dynamic factor Φ or by the factor f in Section 6.9.1.

cl. 6.3.2(3)P: EN 1991-2

The characteristic value of the nosing force should be multiplied by the factor α in accordance with values of $\alpha \geq 1$.

The nosing force must always be combined with a vertical traffic load.

cl. 6.5.3: EN 1991-2

6.9.3. Actions due to traction and braking

Traction and braking forces act at the top of the rails in the longitudinal direction of the track. They have to be considered as uniformly distributed over the corresponding influence length $L_{a,b}$ for traction and braking effects for the structural element considered. The direction of the traction and braking forces has to take account of the permitted direction(s) of travel on each track.

The characteristic values of traction and braking forces are to be taken as follows:

Traction force: $Q_{lak} = 33\,(\text{kN/m}),\ L_{a,b}\,(\text{m}) \leq 1000\,(\text{kN})$ EN 1991-2, (6.20)
for Load Models 71, SW/0 and SW/2 and HSLM

Braking force: $Q_{lbk} = 20\,(\text{kN/m}),\ L_{a,b}\,(\text{m}) \leq 6000\,(\text{kN})^*$ EN 1991-2, (6.21)
for Load Models 71, SW/0 and HSLM

* *Note: For loaded lengths greater than 300 m, additional requirements should be specified by the relevant authority for taking into account the*

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effects of long trains and modern braking systems and simultaneous braking of the wagons.

$Q_{lbk} = 35\,(kN/m),\ L_{a,b}\,(m)$ EN 1991-2, (6.22)
for Load Model SW/2

The characteristic values of traction and braking forces must not be multiplied by the factor Φ or by the factor f in Section 6.9.1.

Note 1: For Load Models SW/0 and SW/2 traction and braking forces need only be applied to those parts of the structure that are loaded, according to Fig. 6.6 and Table 6.2.
Note 2: Traction and braking may be neglected for the Load Model 'unloaded train'.

These characteristic values are applicable to all types of track construction, e.g. continuous welded rails or jointed rails, with or without expansion devices.

The traction and braking forces for Load Models 71 and SW/0 have to be multiplied by the factor α in accordance with the requirements of Section 6.7.2.

For lines carrying special traffic (e.g. restricted to high-speed passenger traffic) the traction and braking forces may be taken as equal to 25% of the sum of the axle loads (real train) acting on the influence length of the action effect of the structural element considered, with a maximum value of 1000 kN for Q_{lak} and 6000 kN for Q_{lbk} where specified by the relevant authority.

Traction and braking forces need to always be combined with the corresponding vertical traffic loads.

When the track is continuous at one or both ends of the bridge only a proportion of the traction or braking force is transferred through the deck to the bearings, the remainder of the force being transmitted through the track where it is resisted behind the abutments. The proportion of the force transferred through the deck to the bearings should be determined by taking into account the combined response of the structure and track in accordance with ***Clause 6.5.4: EN 1991-2 and Annex G*** as well as with UIC Code 774-3.[8]

cl. 6.5.4: EN 1991-2 and Annex G

Note: In the case of a bridge carrying two or more tracks the braking forces on one track have to be considered with the traction forces on the other track. Where two or more tracks have the same permitted direction of travel either traction on two tracks or braking on two tracks has to be taken into account.

6.9.4. Track–bridge interaction

cl. 6.5.4: EN 1991-2

General

Relative displacements of the track and of the bridge, caused by a possible combination of the effects of thermal variations, train braking, as well as deflection of the deck under vertical traffic loads, lead to the track–bridge phenomenon that results in additional stresses to the bridge and the track. Where the rails are continuous over discontinuities in the support to the track (e.g. between a bridge structure and an embankment), longitudinal actions are transmitted partly by the rails to the embankment behind the abutment and partly by the bridge bearings and the substructure to the foundations. It is important to underline that the limit states for the track depend on its design and state of maintenance.

It is also important to minimize the forces lifting the rail fastening systems (vertical displacement at deck ends), as well as horizontal displacements (under braking/starting) which could weaken the ballast and destabilize the track. It is also essential to limit angular discontinuity at expansion joints and switches near the abutments in order to reduce any risk of derailment.

Note: In principle, interaction should be taken into account as a serviceability limit state (SLS) as regards the bridge, as well as being an ultimate limit state (railway traffic safety) as regards the rail. Forces and displacements should therefore theoretically be calculated using the partial safety factors as well as load factors for the loads concerned. That is the principle set out in ***Clause 6.3.2(3)P: EN 1991-2***. The permissible limit values given in UIC Code 774-3,[8] whether for displacements or additional stresses in the rail, due to interaction

cl. 6.3.2(3)P: EN 1991-2

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phenomena were however not determined using ULS procedures but calibrated with the old method of permissible strength design with the simple characterisitic values of Load Model 71. The values given are widely permitted for standard track components in a good state of maintenance and, what is very important, for the traffic and the rails existing today. As the recommended factor $\alpha = 1.33$ is taken for traffic loads in 100 years, where the track components are not known, ***the calculations for interaction have always to be carried out with $\alpha = 1.00$. This is in contradiction to the rule given in Clause 6.3.2.(3)P: EN 1991-2!***

cl. 6.3.2(3)P: EN 1991-2

To ensure track stability during compression (risk of buckling of the track, especially at bridge ends in summertime) or traction (risk of rail breakage in wintertime), the following permissible additional rail stresses are given in ***Clause 6.5.4.5.1: EN 1991-2****.*

cl. 6.5.4.5.1: EN 1991-2

For rails on the bridge and on the adjacent abutment the permissible additional rail stresses due to the combined response of the structure and track to variable actions are as follows:

- *The maximum permissible additional compressive rail stress is 72 N/mm^2.*
- *The maximum permissible additional tensile rail stress is 92 N/mm^2.*

Note: The limiting values for the rail stresses given above are valid for track complying with Rail UIC 60 of a steel grade of at least 900 N/mm^2 strength, minimum curve radius 1500 m, laid on ballasted track with concrete sleepers, the ballast well-consolidated, min. 30 cm deep under the sleepers.

When the above criteria are not satisfied special studies should be carried out or additional measures provided. ***However, there is a problem: normally the bridge design engineer does not have computer programs for calculating track–bridge interaction.***

The requirements for non-ballasted tracks have to be specified by the relevant authority, in function of the chosen track system. The disposition of the expansion joints has to be discussed as soon as possible with the relevant authority.

Computer programs for track–bridge interaction analyses should be validated before use, by analysing the test cases reported in Appendix D of UIC Code 774-3.[8] **But for most practical cases, if the limits of expansion lengths given below can be respected, no calculations of track–bridge interaction are necessary.**

Important principles

cl. 6.5.4: EN 1991-2

- **Expansion devices in the rails must be avoided wherever possible! This can be done in most cases without calculating track–bridge interaction. In these cases a lot of rules given in *Clause 6.5.4: EN 1991-2* and especially *EN 1991-2 Annex G* are not needed!**

EN 1991-2 Annex G

- **Using the possibility of locating the fixed support in the middle part of a deck, it is possible to increase the length of a single deck carrying continuously welded rails without expansion devices.**

Limits of expansion length to allow continuously welded rails (CWR)

The resulting maximum expansion length L_T (see Fig. 6.9) for a single deck carrying CWR without expansion joint will be:

- 60 m for steel structures carrying ballasted track (note: maximum length of deck with fixed bearing in the middle is 120 m)
- 90 m for structures in concrete or steel with concrete slab (composite girders) carrying ballasted track (note: maximum length of deck with fixed bearing in the middle is 180 m).

Note: Experience has shown that for rail UIC 54 with well-consolidated ballasted track, the permissible expansion lengths mentioned above for UIC rail 60 can be adopted.

For track curve radius $r \leq 1500$ m the permissible rail stresses have to be as agreed with the relevant authority.

When the maximum expansion length L_T is only marginally over the limits given, it is recommended that calculations using a track–bridge computer program are carried out, to avoid the expansion joints if possible.

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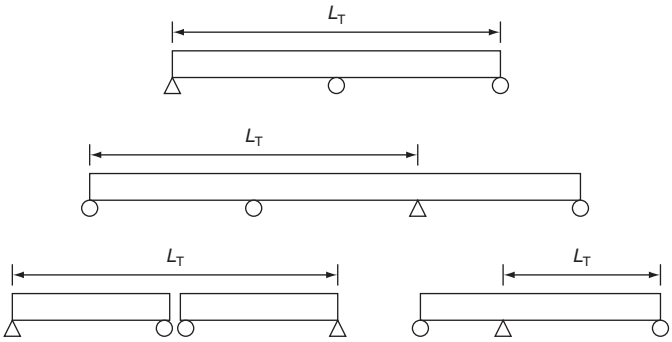


Fig. 6.9. Examples of expansion length L_T

When the maximum expansion length is over the limits given, expansion devices will be necessary.

Limiting values for longitudinal displacements of multi-span portal frame systems under braking/traction
In the case of a deck carrying expansion devices at both ends, e.g. in the case of a continuous multi-span portal frame without a special rigidly fixed bearing against horizontal longitudinal forces, the maximum permissible displacement of the multi-span portal frame system due to braking/traction (with $\alpha = 1.00$) on two tracks is 30 mm (calculated without a track–bridge interaction program).

Vertical displacement of the upper surface of a deck relative to the adjacent construction (abutment or another deck)
The deflection of the deck under traffic loads causes the end of the deck behind the support structures to lift. This lifting must be reduced. *cl. 6.5.4.5.2(P): EN 1991-2*

The vertical displacement of the upper surface of a deck relative to the adjacent construction (abutment or another deck) δ_V (mm) due to characteristic traffic loads ($\alpha = 1$) must not exceed the following values:

- 3 mm for a maximum line speed at the site of up to 160 km/h
- 2 mm for a maximum line speed at the site over 160 km/h.

6.10. Other actions for railway bridges

The following actions also need to be considered in the design of the structure:

- load effects from other railway infrastructure and equipment
- effects due to inclined decks or inclined bearing surfaces
- **aerodynamic actions** from passing trains on structures adjacent to the track; these actions **are defined in *Clause 6.6: EN 1991-2***. *cl. 6.6: EN 1991-2*
  *Note: The dynamic amplification factor mentioned in **Clause 6.6.1(5): EN 1991-2** must be considered at the start and end of these structures. It is recommended to check fatigue for these elements and their anchorages.* *cl. 6.6.1(5): EN 1991-2*
- action effects from catenaries and other overhead line equipment attached to the structure.

The relevant national and international requirements should be applied in terms of:

- wind actions
- temperature variations and temperature gradient effects etc.
- bearing friction
- snow, avalanche and ice loads
- water pressure effects from groundwater, free water, flowing water etc.

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- waterborne debris and scour effects
- settlement
- differential settlements.

cl. 6.7: EN 1991-2

6.11. Derailment

Railway structures have to be designed in such a way that, in the event of a derailment, the resulting damage to the bridge (in particular overturning or the collapse of the structure as a whole) is limited to a minimum.

6.11.1. Derailment actions from rail traffic on a railway bridge

Derailment of rail traffic on a railway bridge has to be considered as an accidental design situation. Two design situations have to be considered:

- Design Situation I: Derailment of railway vehicles, with the derailed vehicles remaining in the track area on the bridge deck with vehicles retained by the adjacent rail or an upstand wall.
- Design Situation II: Derailment of railway vehicles, with the derailed vehicles balanced on the edge of the bridge and loading the edge of the superstructure (excluding non-structural elements such as walkways).

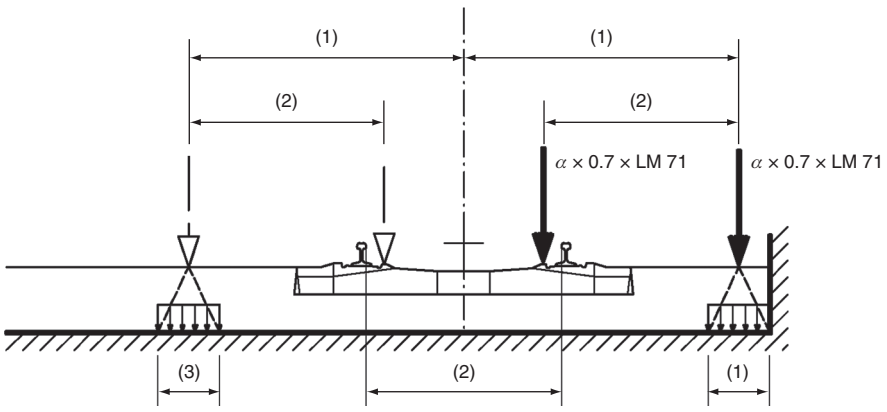
Note: The relevant authority may specify additional requirements.

For Design Situation I, collapse of a major part of the structure must be avoided. Local damage, however, may be tolerated. The parts of the structure concerned need to be designed for the following design loads in the Accidental Design Situation:

$\alpha \times 1.4 \times$ LM71 (both point loads and uniformly distributed loading, Q_{A1d} and q_{A1d}) parallel to the track in the most unfavourable position inside an area of width 1.5 times the track gauge on either side of the centre-line of the track (Fig. 6.10).

Note: It should be noted that the factor 1.4 is not considered a safety factor as laid down generally in the Eurocodes.

For Design Situation II, the bridge should not overturn or collapse. For the determination of overall stability a maximum total length of 20 m of $q_{A2d} = \alpha \times 1.4 \times$ LM71 should be taken as a uniformly distributed vertical line load acting on the edge of the structure under consideration.



(1) Max 1.5*s* or less if against wall
(2) Track gauge *s*
(3) For ballasted decks the point forces may be assumed to be distributed on a square of side 450 mm at the top of the deck

Fig. 6.10. Design Situation I – equivalent load Q_{A1d} and q_{A1d}

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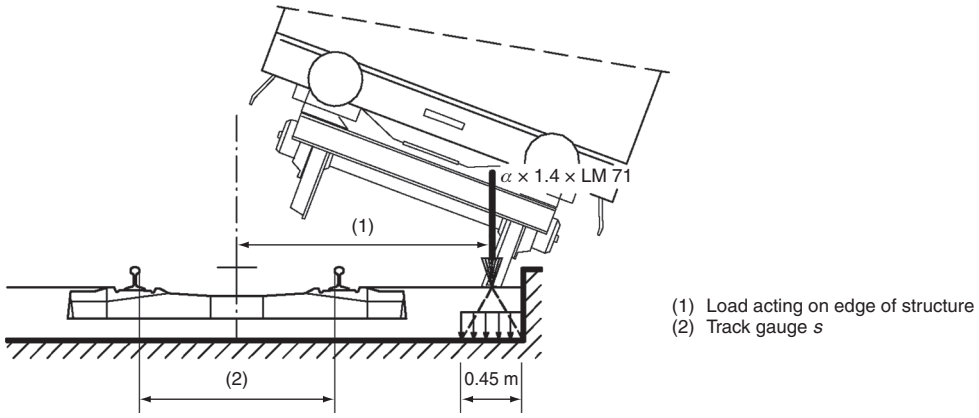


Fig. 6.11. Design Situation II – equivalent load q_{A2d}

The above-mentioned equivalent load is only to be considered for determining the ultimate strength or the stability of the structure as a whole. The cantilever and minor structural elements need not be designed for this load.

Example 6.2. Uniformly distributed equivalent line load for Design Situation II

For a bridge span of 8 m, take the four individual loads of 250 kN plus (8.0 m − 6.4 m) 80 kN/m = 1128 kN, which can be distributed along the whole length of 8 m, which gives 141 kN/m. With $\alpha = 1.33$ and the factor 1.4 one obtains $q_{A2d} = 262$ kN/m. For a span greater than 20 m, one obtains $q_{A2d} = 194$ kN/m, to be distributed along a length of 20 m.

Design Situations I and II have to be examined separately. A combination of these loads need not be considered.

For Design Situations I and II other rail traffic actions should be neglected for the track subjected to derailment actions.

For structural elements which are situated above the level of the rails, measures to mitigate the consequences of a derailment have to be in accordance with the requirements specified by the relevant authority.

6.11.2. Derailment under or adjacent to a structure and other actions for other Accidental Design Situations

When a derailment occurs, there is a risk of collision between derailed vehicles and structures over or adjacent to the track. The requirements for collision loading and other design requirements are specified in EN1991-1-7 and in UIC-Code 777-2.[11]

Other actions for other Accidental Design Situations should be taken into account in accordance with the requirements specified by the relevant authority.

6.12. Application of traffic loads on railway bridges

6.12.1. General

cl. 6.8.1: EN 1991-2

The bridge has to be designed for the required number and position(s) of the tracks in accordance with the track positions and tolerances specified for the particular project.

Each structure should also be designed for the greatest number of tracks geometrically and structurally possible in the least favourable position, irrespective of the position of the

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intended tracks, taking into account the minimum spacing of tracks and structural gauge clearance requirements specified for the particular project.

The effects of all actions have to be determined with the traffic loads and forces placed in the most unfavourable positions. Traffic actions which produce a relieving effect are to be neglected (see Example 6.3).

Example 6.3. Rules for application of LM71

For the application of influence lines, the two following examples shown for LM71 may be used as specimens (Fig. 6.12).

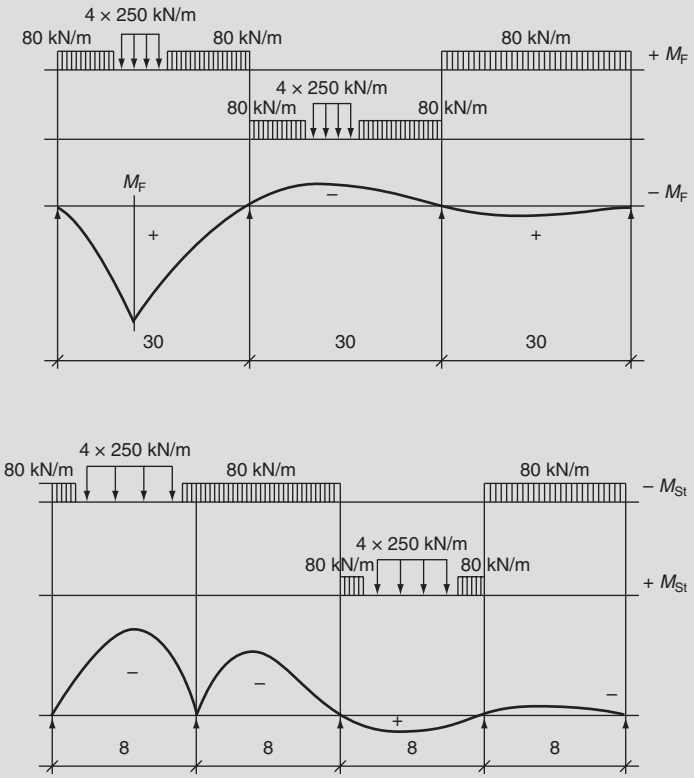


Fig. 6.12. LM71 placed in the most unfavourable position for calculating two different bending moments in continuous bridges

cl. 6.8.2: EN 1991-2

For the determination of the most adverse load effects from the application of Load Model 71:

- Any number of lengths of the uniformly distributed load q_{vk} have to be applied to a track and up to four of the individual concentrated loads Q_{vk} have to be applied once per track.
- For elements carrying two tracks, Load Model 71 has to be applied to either track or both tracks.
- For bridges carrying three or more tracks, Load Model 71 has to be applied to any one track, any two tracks or 0.75 times Load Model 71 to three or more of the tracks.

For the determination of the most adverse load effects from the application of Load Model SW/0:

- The loading has to be applied once per track.
- For elements carrying two tracks, Load Model SW/0 has to be applied to either track or both tracks.

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- For bridges carrying three or more tracks, Load Model SW/0 has to be applied to any one track, any two tracks or 0.75 times Load Model SW/0 to three or more of the tracks.

For the determination of the most adverse load effects from the application of Load Model SW/2:

- The loading has to be applied once per track.
- For elements carrying more than one track, Load Model SW/2 has to be applied to any one track only with Load Model 71 or Load Model SW/0 applied to the other tracks as specified above.

For the determination of the most adverse load effects from the application of Load Model 'unloaded train':

- Any number of lengths of the uniformly distributed load q_{vk} have to be applied to a track.
- Generally Load Model 'unloaded train' need only be considered in the design of structures carrying one track.

All continuous beam bridges designed for Load Model 71 have to be checked additionally for Load Model SW/0.

Where a dynamic analysis is required in accordance with Annex B to Chapter 6 of this Designers' Guide and UIC Code 776-2[7] all bridges need also to be designed for the loading from real trains and Load Model HSLM where required.

6.12.2. Groups of loads – characteristic values of the multi-component action

As stated in ***EN1991-2, 6.8.2*** the simultaneity of the loading systems can be taken into account by considering the groups of loads defined in Table 6.5 below. **Each of these groups of loads**, which are mutually exclusive, **should be considered as defining a single variable action for combination with non-traffic loads**. This means the following:

cl. 6.8.2: EN 1991-2

- A group of loads is a multi-component traffic action like defined in Table 6.5.
- In each group of loads one component is considered as dominant, other components as accompanying. For the assessment of the characteristic value of this group of loads the dominant component action is taken into account with its full characteristic value, the other accompanying component actions with generally reduced values.
- For defining representative values of the multi-component traffic action (group of loads) defined in Table 6.5, all values assigned to the different components in a group have to be multiplied by the same value of factor ψ (ψ_0, ψ_1 or ψ_2, depending on the representative value to be obtained). This representative value will, when necessary, be taken into account with other actions in the considered combinations.
- All values given to the different components in a group are multiplied by the same value of partial factor γ_Q for verification at ULS.
- The values of ψ and γ_Q to be used correspond to the values to be used for the component considered as dominant in the group when the dominant component is considered alone.
- If two components are designated as dominant in the same group, for simplification purposes it is the most unfavourable of the two values of ψ (and/or γ_Q) which should be used for the whole.

Note: It is not necessary to consider the group of loads technique, if no simplification of the design process can be obtained. The group of loads technique is not safe for use in all circumstances (e.g. for the design of bearings, for the assessment of maximum lateral and minimum vertical traffic loading, design of bearing restraints, the assessment of maximum overturning effects on abutments, especially for continuous bridges, etc.).

In general it is easier to take individual actions into account for the design of a bridge, thinking in hazard scenarios and taking leading and accompanying actions for the load combinations given in Chapter 8. They can be combined with the help of Table 6.5.

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Table 6.5. Assessment of groups of loads for rail traffic (characteristic values of multi-component actions) (Data taken from EN 1991-2, Table 6.11)

Number of tracks on structure			Groups of loads			Vertical forces			Horizontal forces			Comment
			Reference: sections of this Guide			6.7.2/6.7.3	6.7.3	6.7.4	6.9.3	6.9.1	6.9.2	
			Reference: EN 1991-2			6.3.2/6.3.3	6.3.3	6.3.4	6.5.3	6.5.1	6.5.2	
1	2	≥3	Number of tracks loaded	Load group[8]	Loaded track	LM71[1] SW/0[1],[2] HSLM[6],[7]	SW/2[1],[3]	Unloaded train	Traction, braking[1]	Centrifugal force[1]	Nosing force[1]	
			1	gr 11	T_1	1			1[5]	0.5[5]	0.5[5]	Max. vertical 1 with max. longitudinal
			1	gr 12	T_1	1			0.5[5]	1[5]	1[5]	Max. vertical 2 with max. transverse
			1	gr 13	T_1	1[4]			1	0.5[5]	0.5[5]	Max. longitudinal
			1	gr 14	T_1	1[4]			0.5[5]	1	1	Max. lateral
			1	gr 15	T_1			1		1[5]	1[5]	Lateral stability with "unloaded train"
			1	gr 16	T_1		1		1[5]	0.5[5]	0.5[5]	SW/2 with max. longitudinal
			1	gr 17	T_1		1		0.5[5]	1[5]	1[5]	SW/2 with max. transverse
			2	gr 21	T_1	1			1[5]	0.5[5]	0.5[5]	Max. vertical 1 with max longitudinal
					T_2	1			1[5]	0.5[5]	0.5[5]	
			2	gr 22	T_1	1			0.5[5]	1[5]	1[5]	Max. vertical 2 with max. transverse
					T_2	1			0.5[5]	1[5]	1[5]	
			2	gr 23	T_1	1[4]			1	0.5[5]	0.5[5]	Max. longitudinal
					T_2	1[4]			1	0.5[5]	0.5[5]	
			2	gr 24	T_1	1[4]			0.5[5]	1	1	Max. lateral
					T_2	1[4]			0.5[5]	1	1	
			2	gr 26	T_1		1		1[5]	0.5[5]	0.5[5]	SW/2 with max. longitudinal
					T_2	1			1[5]	0.5[5]	0.5[5]	
			2	gr 27	T_1		1		0.5[5]	1[5]	1[5]	SW/2 with max. transverse
					T_2	1			0.5[5]	1[5]	1[5]	
			≥ 3	gr 31	T_i	0.75			0.75[5]	0.75[5]	0.75[5]	Additional load case

(1) All relevant factors (α, Φ, f, …) have to be taken into account.

(2) SW/0 has only to be taken into account for continuous span bridges.

(3) SW/2 needs to be taken into account only if it is stipulated for the line.

(4) Factor may be reduced to 0.5 if favourable effect; it cannot be zero.

(5) In favourable cases these non-dominant values have be taken equal to zero.

(6) HSLM and real trains where required in accordance with EN 1991-2, 6.4.4 and 6.4.6.1.1.

(7) If a dynamic analysis is required in accordance with EN 1991-2, 6.4.4 see also 6.4.6.5(3) and 6.4.6.1.2.

(8) See also EN 1990: 2002/A1, *Table A.2.3*.[3]

Dominant component action as appropriate

to be considered in designing a structure supporting one track (Load Groups 11–17)

to be considered in designing a structure supporting two tracks (Load Groups 11–27 except 15). Each of the two tracks have to be considered as either T_1 (Track 1) or T_2 (Track 2)

to be considered in designing a structure supporting three or more tracks; (Load Groups 11 to 31 except 15). Any one track has to be taken as T_1, any other track as T_2 with all other tracks unloaded. In addition the Load Group 31 has to be considered as an additional load case where all unfavourable lengths of track T_i are loaded.

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6.13. Fatigue

Reference fatigue loading for all railway bridges and all materials

The fatigue assessment, in general a stress range verification, has to be carried out according to ***EN 1991-2, Annex D (normative)*** and the specifications in the Design Codes ***EN 1992, EN 1993*** and ***EN 1994***. For new bridges, fatigue calculations have to be done with the reference fatigue loading LM71 and with $\alpha = 1.0$ (even if taking $\alpha = 1.33$ for ULS). For structures carrying more than one track, this reference fatigue loading has to be applied to a maximum of two tracks in the most unfavourable positions.

Annex D (normative): EN 1991-2 EN 1992 EN 1993 EN 1994

Traffic mix (train types for fatigue) for fatigue considerations

Where the fatigue assessment is based on the damage equivalent factors λ, for instance for constructional steel, for reinforcing steel or for prestressing steel, one of the traffic mixes set out in ***EN 1991-2, Annex D3 (normative)*** should be used. However, as 250 kN axles are foreseen, and, as noted in Section 6.7.2, heavier loads do not significantly influence the investment costs of bridges, it is recommended that fatigue assessment should be carried out choosing also train types for fatigue with 250 kN axle loads, see also second Note below.

Annex D3 (normative): EN 1991-2

For structural members in steel the safety verification has to be carried out by ensuring that the following condition is satisfied:

$$\gamma_{Ff}\lambda\Phi_2\Delta\sigma_{71} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \qquad \text{EN 1991-2, (D.6)}$$

where

γ_{Ff} is the partial safety factor for the fatigue loading *(Note: The recommended value is $\gamma_{Ff} = 1.00$.)*

Λ is the damage equivalence factor for fatigue which takes account of the span, the service traffic, the annual traffic volume, the intended design life of the structural element and the number of tracks.

$$\lambda = \lambda_1\lambda_2\lambda_3\lambda_4$$

where

λ_1 is a factor accounting for the structural member type (e.g. a continuous beam) and takes into account the damaging effect of the chosen service traffic (e.g. heavy traffic mix), depending on the length of the influence line or area, and on function of the slopes (in general lines in a double logarithmic scale) of the different Wöhler curves

λ_2 is a factor that takes into account the annual traffic volume

λ_3 is a factor that takes into account the intended design life of the structural member

λ_4 is a factor that denotes the effect of loading from more than one track

Φ_2 is the dynamic factor

$\Delta\sigma_{71}$ is the stress range due to the Load Model 71 (and where required SW/0), always calculated with $\alpha = 1$ and the loadings being placed in the most unfavourable position for the element under consideration

$\Delta\sigma_c$ is the reference value of the fatigue strength

γ_{Mf} is the partial safety factor for fatigue strength in the design codes

Note:

- *For new bridges (even if taking $\alpha = 1.33$ for ULS), fatigue calculations have to be done with the fatigue loading LM71 and with $\alpha = 1.0$.*
- *The fatigue assessment should be carried out on the basis of 'traffic with 250 kN axles'. It is the heavy traffic mix (i.e. a traffic mix with 250 kN axle loads) mentioned in* ***EN 1991-2, Annex D3 (normative)*** that should be taken into account for calculating the damage equivalent factor λ_1.

Annex D3 (normative): EN 1991-2

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Alternatively, if the standard traffic mix represents the actual traffic more closely than the heavy traffic mix, the standard traffic mix could be used, but with the calculated λ_1 values enhanced by a factor of 1.1 to allow for the influence of 250 kN axle loads.

For reinforcing and prestressing steel the damage equivalent stress range is calculated in manner similar to that for steel.

For concrete subjected to compression, adequate fatigue resistance may be assumed to follow the rules given in EN 1992-2.

It cannot be stressed enough that railway bridges must be designed and constructed in a fatigue-resistant way. To attain optimal life-cycle costs and for reaching the intended design life (in general minimum 100 years), all important structural members need to be designed for fatigue, so that there is an acceptable level of probability that their performance will be satisfactory throughout their intended design life:

For steel bridges this means that constructional details have to be chosen which give the maximum possible fatigue detail categories $\Delta\sigma_c$; for example:

- Composite girders: detail category 71
- Welded plate girders: detail category 71
- Truss bridges: detail category 71 at sites where fatigue is a risk, detail category 36 at sites where fatigue is no risk.
- Orthotropic decks: detail category 36 at sites where orthogonal ribs are crossing better detail category 71 which is only possible when ribs are constructed only in the transverse direction under a thick plate. This latter type of orthotropic deck is possible if self-weight is not critical. This is the case if the spans are not long

For prestressed bridges fully prestressing under service loads is the best design to avoid fatigue problems. For structures not fully prestressed the permissible fatigue strength categories $\Delta\sigma_s$ for prestressing and reinforcing bars must be observed.

Plastic ducts and electrically isolated tendons can increase fatigue resistance of prestressing steel.

Anchorages and couplers for prestressing tendons have to be so placed that they are in a region of low stress variation.

For reinforced structures, the fatigue strength caregories $\Delta\sigma_s$ must of course be observed. Welded joints of reinforcing bars should be avoided in regions of high stress variation. The bending radii of reinforcing bars must be respected to avoid too much loss of fatigue strength.

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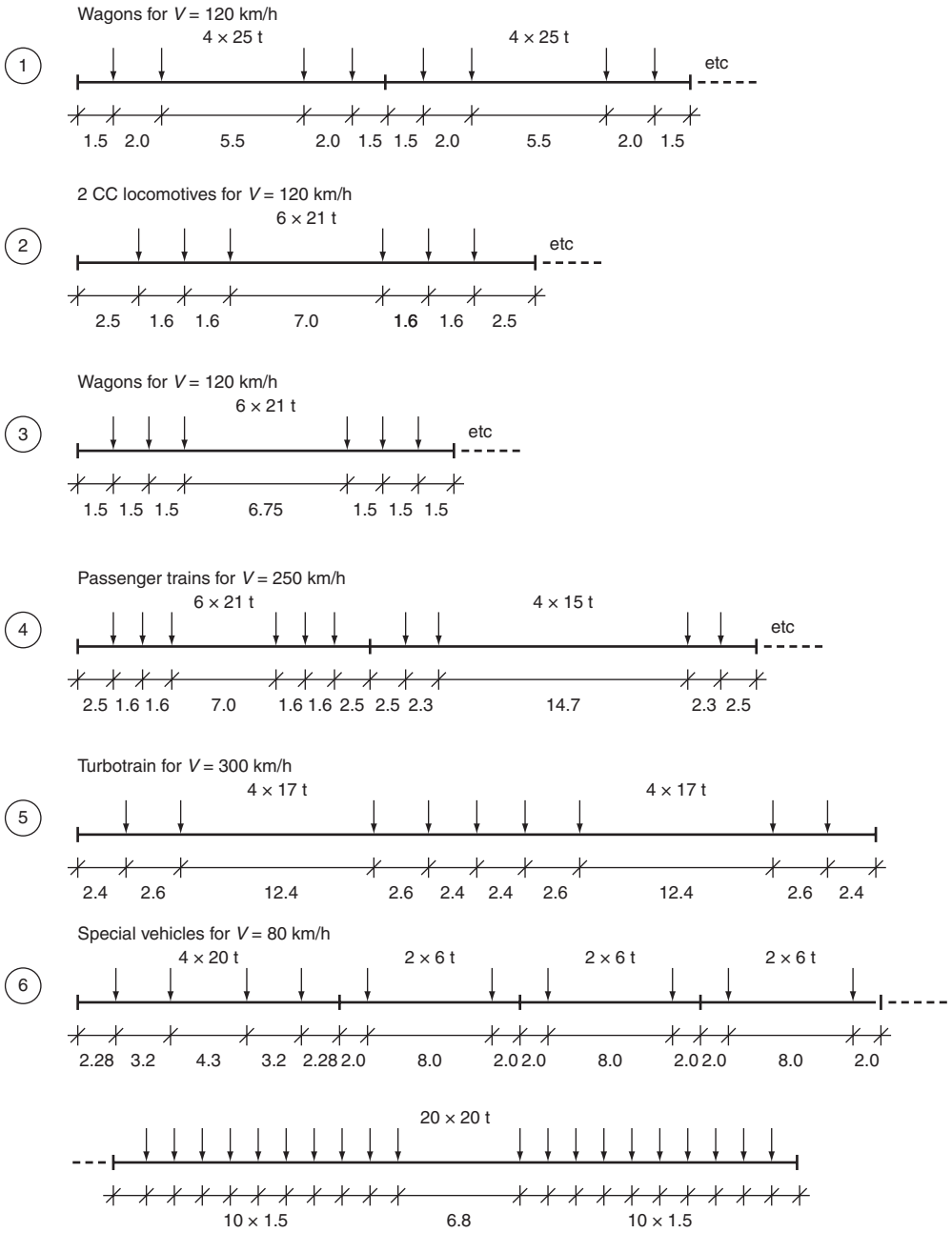
Annex A to Chapter 6: Background information on the determination of the main rail load models and the verification procedures for additional dynamic calculations

A6.1. Determination of rail load models

Table A6.1 shows the six standard real trains given in UIC Code 776-1[6] which represent the basis for determining Load Model 71.

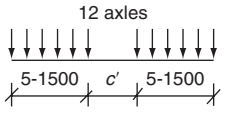
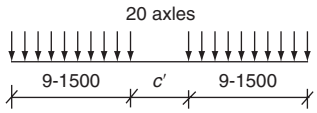
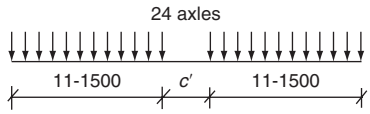
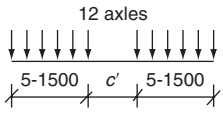
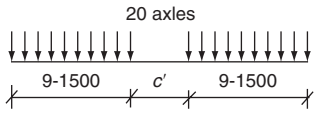
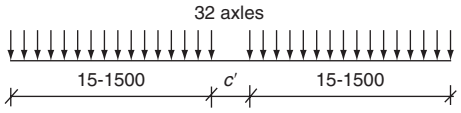
The dynamic factor Φ covers not only dynamic effects but also a part of the static loads of the six standard real trains defined in Table A6.1. The relationship between the

Table A6.1. Characteristical values of service trains



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Table A6.2. Allocation of heavy wagons to load classifications

Load model	Diagram of heavy wagons	Axle loads (t)	c' in m
SW/0		20	≥3.0
		22.5	≥6.0
		20	≥6.8
		19	≥9.0
SW/2		17	≥3.0
		19	≥6.0
		17	≥5.0
SW/2		22.5	≥8.5

dynamic factor for real trains $1 + \varphi$ (see Section 6.8.2) and the dynamic factor Φ (see Section 6.8.3) for LM71, SW/O and SW/2 is as follows:

$$(1 + \varphi)S_{\text{real trains } 1-6} \leq \Phi S_{\text{LM7}}$$

where S is an elastomechanical action effect for M (moment), Q (shear force), y (deflection), σ (normal stress), τ (shear stress), ε (strain) and γ (shear deformation) at a point of the structural component.

Therefore the determination of Φ is by way of the inequality:

$$\Phi \geq S_{\text{real trains } 1-6}(1 + \varphi_{1-6})/S_{\text{LM71}}$$

Table A6.2 shows the different heavy wagons given in UIC Code 776-1[6] which were the basis for determining Load Models SW/0 and SW/2.

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Annex B to Chapter 6: Dynamic studies for speeds >200 km/h*

Annexes E and F, cl. 6.4.6: EN 1991-2

Background documents: nine ERRI reports D214[16]

B6.1. Verification procedures for additional dynamic calculations

B6.1.1. General, risk of resonance, requirements for a dynamic analysis

The bridges on high-speed lines are to be designed by taking into account the resonance phenomenon which is generated by the crossing over of successions of axles with more or less uniform spacing. Excessive deformation of the bridge can jeopardize train traffic safety by causing unacceptable changes in the vertical and horizontal geometry of the track, excessive rail stresses and excessive vibrations in the bridge support structures. In the case of ballasted bridges, excessive vibrations and vertical accelerations could destabilize the ballast. Excessive deformation may also affect the loads imposed on the train/track/bridge system, as well as create conditions that lead to passenger discomfort.

The dynamic behaviour of a bridge depends on the:

- traffic speed across the bridge
- number of axles, their loads and distribution
- suspension characteristics of the vehicle
- span L of the bridge
- mass of the structure
- natural frequencies of the entire structure
- damping of the structure
- regularly spaced supports of the deck slabs and of the construction
- wheel defects (flats, out-of-roundness)
- vertical track defects
- dynamic characteristics of the track.

When a train crosses a bridge at a certain speed, the deck will deform as a result of excitation generated by the moving axle loads. At low speeds, structural deformation is similar to that corresponding to the equivalent static load case. At higher speeds, deformation of the deck exceeds the equivalent static values. The increase in deformation is also due to the regular excitation generated by evenly spaced axle loads. A risk of resonance exists at critical speeds, when the excitation frequency (or a multiple of the excitation frequency) coincides with the natural frequency of the structure. When this happens there is a rapid increase in structural deformation and acceleration (especially for low damping values of the structure) and may cause:

- loss of wheel–rail contact
- destabilization of the ballast.

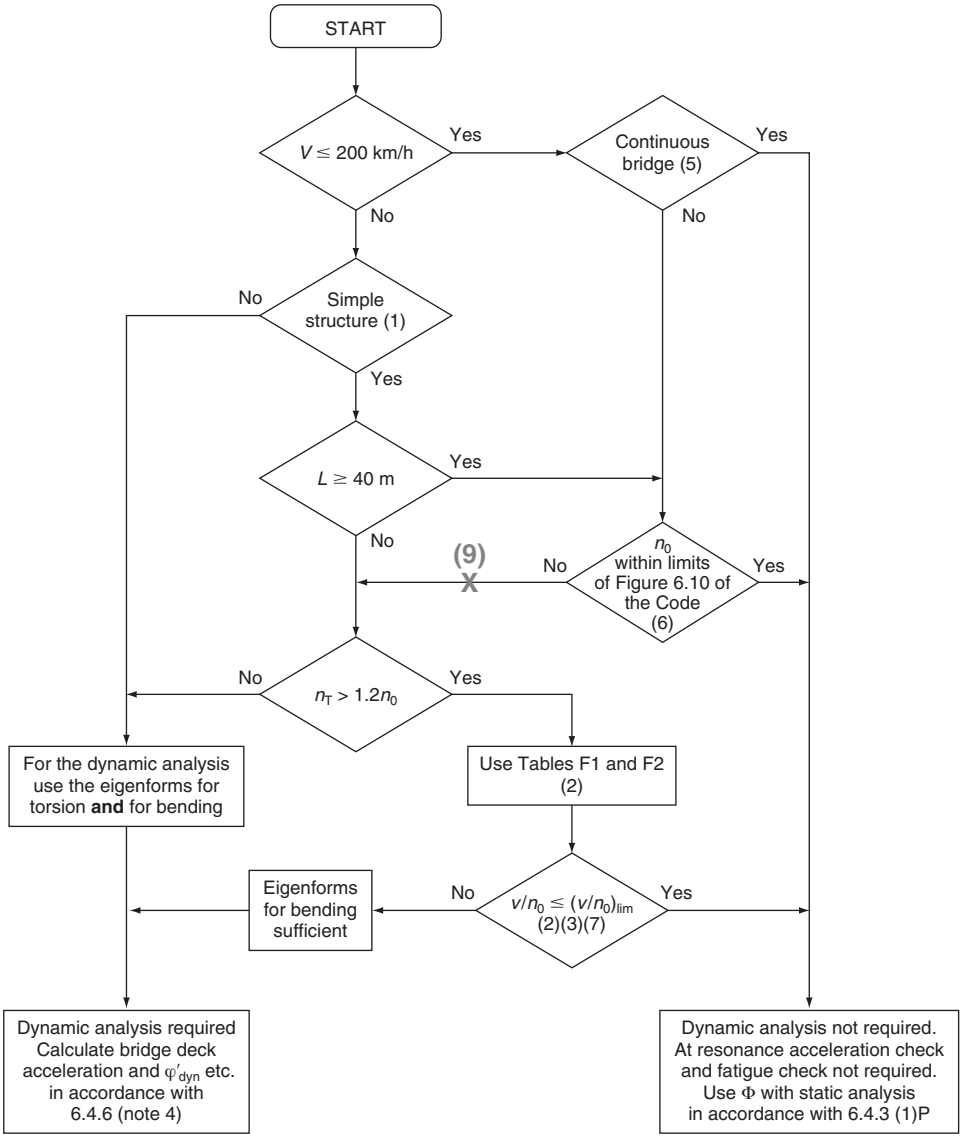
In such situations, train traffic safety on the bridge is compromized. In view of the potential risk outlined, calculations need to be done to determine the extent of deformations at resonance. Furthermore, accelerations of the structure cannot be determined by static analysis. Even though deck accelerations are low at low speeds, they can reach unacceptable values at higher speeds.

Note: In practice, the acceleration criterion will, in most cases, be the decisive factor.

In principle, the dynamic analysis has to be undertaken using the real high speed trains specified. The selection of real trains has to take into account each permitted or envisaged

* See remarks in Section 6.1 of this Designers' Guide.

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where:

V	is the maximum line speed at the site (km/h)
L	is the span length (m)
n_0	is the first natural bending frequency of the bridge loaded by permanent actions (Hz)
n_T	is the first natural torsional frequency of the bridge loaded by permanent actions (Hz)
v	is the maximum nominal speed (m/s)
$(v/n_0)_{lim}$	is given in EN 1991-2, Annex F.

Note (1) Valid for simply supported bridges with only longitudinal line beam or simple plate behaviour with negligible skew effects on rigid supports.

Note (2) For Tables F1 and F2 and associated limits of validity see EN 1991-2, Annex F.

Note (3) A dynamic analysis is required where the frequent operating speed of a real train equals a resonant speed of the structure. See 6.4.6.6 and Annex F of EN 1991-2.

Note (4) φ'_{dyn} is the dynamic impact component for real trains for the structure given in EN 1991-2, 6.4.6.5(3).

Note (5) Valid providing the bridge meets the requirements for resistance, deformation limits given in EN 1990: 2002/A1, A2.4.4 and the maximum coach body acceleration (or associated deflection limits) corresponding to a very good standard of passenger comfort given in EN 1990: 2002/A1 (Annex 2).

Note (6) For bridges with a first natural frequency n_0 within the limits given by Fig. B6.2 and a maximum line speed at the site not exceeding 200 km/h, a dynamic analysis is not required.

Note (7) For bridges with a first natural frequency n_0 exceeding the upper limit (1) in Fig. B6.2, a dynamic analysis is required. Also see EN 1991-2, 6.4.6.1.1(7).

Fig. B6.1. Logic diagram to determine whether a specific dynamic analysis is required (Reproduced from EN 1991-2, with permission from BSI), footnote (9) added by the author

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train formation for every type of high-speed train permitted or envisaged (see B6.1.3 below) to use the structure at speeds over 200 km/h. *cl. 6.4.6.1.1: EN 1991-2*

Note: The loading should be defined by the individual axle loads and spacings for each configuration of each required real train.

The dynamic analysis needs to also be undertaken using Load Model HSLM (high-speed load models) on bridges designed for international lines where European high-speed interoperability criteria TSI (Technical Specifications for Interoperability) are applicable. *cl. 6.4.6.1.1(2)P: EN 1991-2*

Note: The trains that were used to obtain Load Model HSLM were Eurostar, ICE2, Thalys and ETR. Other trains appeared afterwards (Virgin, Talgo), with different dynamic signatures. Moreover, bridges on interoperable lines are to be designed also for future high-speed trains. The research of Committee ERRI D214[16] permitted to design a simplified method to compute acceleration and to define a universal load model for dynamic calculations being able to cover the dynamic effect of all existing trains mentioned above, but also of all future trains corresponding to the technical specifications mentioned in Table B6.1.

Load Model HSLM comprises two separate universal trains with variable coach lengths, HSLM-A and HSLM-B. They are defined in Section B6.1.3.3.

Note: HSLM-A and HSLM-B together represent the dynamic load effects of articulated, conventional and regular high-speed passenger trains, in accordance with the requirements of the European Technical Specification for Interoperability.

B6.1.2. Logic diagram

cl. 6.4.4: EN 1991-2

The logic diagram in Fig. B6.1 is used to determine whether a static or a dynamic analysis is required.

The diagram shows:

V = traffic speed (km/h)
L = span (m)
n_0 = first natural bending frequency of the unloaded bridge (Hz)
n_T = first natural torsion frequency of the unloaded bridge (Hz)
V_{lim}/n_0 and $(V/n_0)_{lim}$ are defined in EN 1991-2, Annex F.

Note: The logic diagram of Fig. B6.1 also mentions cases where a dynamic analysis is required for a maximum line speed at sites less than 200 km/h. This analysis can be avoided if the recommended values for permissible deformations given later in Chapter 8 are chosen. In these cases the application of Annex B is not necessary.

Fig. B6.1. *Continued*

Note (8) For a simply supported bridge subjected to bending only, the natural frequency may be estimated using the formula:

$$n_0\,(\text{Hz}) = \frac{17.75}{\sqrt{\delta_0}} \qquad \text{EN 1991-2, (6.3)}$$

where δ_0 is the deflection at midspan due to permanent actions (mm) and is calculated, using a short term modulus for concrete bridges, in accordance with a loading period appropriate to the natural frequency of the bridge.

Note (9) **(Added by the author)** If the permissible deformations recommended in Table 8.12 of this Designers' Guide are respected, no dynamic study is necessary for speeds $\leq$ 200 km/h.

General note (summary when the maximum line speed at the site is $\leq$200 km):

Permissible deformations conforming to the recommended values given in Table 8.12 of this Designers' Guide:

- *There is no need for dynamic analysis if the speed of the line is less than or equal to 200 km/h.*

Permissible deformations not conforming to the recommended values given in Table 8.12 of this Designers' Guide:

- *For simple beams there is no need for dynamic analysis if the first natural bending frequency is within the limits of domain given in Fig. B6.2. Otherwise, an additional verification is required, considering:*
  - *train types 1 to 12 given in EN 1991-2, Annex D.* The load models for fatigue assessment in EN 1991-2, Annex D, are representative of mixed traffic that runs on conventional lines at speeds up to 200 km/h.
  - *real trains specified.*
- *For continuous beams no dynamic analysis is required.*

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The upper limit of n_0 is governed by dynamic enhancements due to track irregularities and is given by:

$$n_0 = 94.76L^{-0.748} \qquad \text{EN 1991-2, (6.1)}$$

The lower limit of n_0 is governed by dynamic impact criteria and is given by:

$$n_0 = 80/L \quad \text{for } 4\,\text{m} \leq L \leq 20\,\text{m}$$

$$n_0 = 23.58L^{-0.592} \quad \text{for } 20\,\text{m} < L \leq 100\,\text{m} \qquad \text{EN 1991-2, (6.2)}$$

where

n_0 is the first natural frequency of the bridge taking account of mass due to permanent actions

L is the span length for simply supported bridges or L_Φ for other bridge types

(1) Upper limit of natural frequency

(2) Lower limit of natural frequency

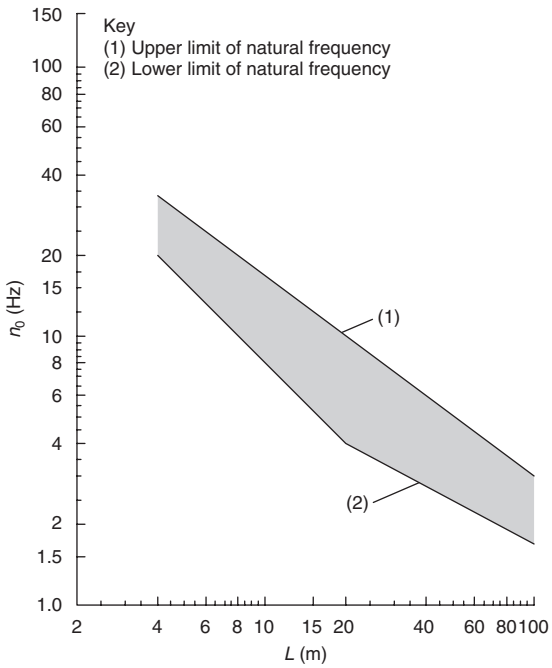


Fig. B6.2. Limits of bridge natural frequency n_0 (Hz) as a function of L (m) (Reproduced from EN 1991-2, with permission from BSI)

B6.1.3. Train models

Annex E: EN 1991-2

B6.1.3.1. Hypotheses relating to rolling stock

The concept of a 'universal train' was proposed on the basis of dynamic train signatures. A 'universal train' must be representative of both existing trains and future trains required to run on the European network. The 'universal train' signature, for a given bridge, is used to perform a dynamic calculation giving a midspan acceleration upper bound. It will thus considerably limit the number of calculations. However, it must be ensured that future rolling stock remains compatible with the dimensioning of bridges. Technical Specifications for Interoperability will make it possible to design rolling stock to be compatible with the criteria for structural safety of bridges (see B6.1.3.2 below).

It is possible to classify all current and future high-speed trains into three major categories, as shown below in Figs B6.3 to B6.5.

Annex E.1: EN 1991-2

B6.1.3.2. Rolling stock for interoperability

High-speed trains now run on international lines in different countries and their numbers will most probably increase in the future. It is therefore essential to establish minimum technical specifications for projects relating to bridges and rolling stock so as to allow high-speed trains to travel throughout the European network in safety and without being obliged to recalculate existing bridges in function of new high-speed trains.

The Technical Specifications for Interoperability relating to rolling stock can be outlined as follows.

Load Model HSLM is valid for passenger trains conforming to the following criteria:

- individual axle load P (kN) limited to 170 kN and for conventional trains also limited to the value in accordance with equation EN 1991-2, (E.2)
- the distance D (m) corresponding to the length of the coach or to the distance between regularly repeating axles in accordance with EN 1991-2, Table E.1
- the spacing of axles within a bogie, d_{BA} (m) in accordance with:

$$2.5\,\text{m} \leq d_{BA} \leq 3.5\,\text{m} \qquad \text{EN 1991-2, (E.1)}$$

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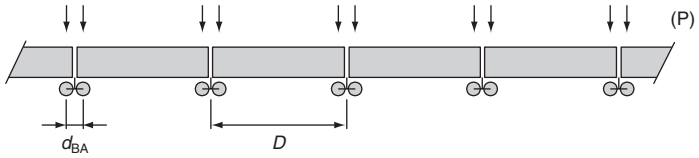


Fig. B6.3. Articulated train (Reproduced from EN 1991-2, with permission from BSI)

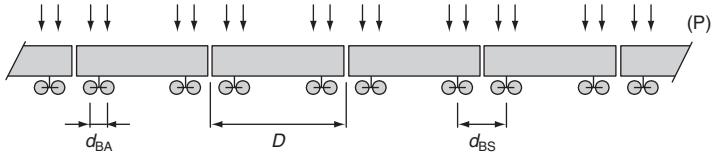


Fig. B6.4. Conventional train (Reproduced from EN 1991-2, with permission from BSI)

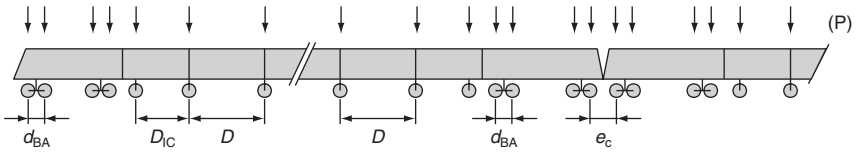


Fig. B6.5. Regular train (Reproduced from EN 1991-2, with permission from BSI)

- for conventional trains the distance between the centres of bogies between adjacent vehicles d_{BS} (m) in accordance with:

$$4P\cos\left(\frac{\pi d_{BS}}{D}\right)\cos\left(\frac{\pi d_{BA}}{D}\right) \leq 2P_{HSLMA}\cos\left(\frac{\pi d_{HSLMA}}{D_{HSLMA}}\right) \qquad \text{EN 1991-2, (E.2)}$$

- for regular trains with coaches with one axle per coach (e.g. train type E in EN 1991-2, Appendix F2) the intermediate coach length D_{IC} (m) and distance between adjacent axles across the coupling of two individual trainsets e_c (m) in accordance with EN 1991-2, Table E.1
- D/d_{BA} and $(d_{BS} - d_{BA})/d_{BA}$ should not be close to an integer value
- maximum total weight of train 10 000 kN
- maximum train length 400 m
- maximum unsprung axle mass of 2 t.

In order to ensure that high-speed trains crossing bridges or viaducts do not generate stresses incompatible with their dimensioning – whether they are strength characteristics or operating criteria – these trains should be designed to comply with the criteria listed in the first column of Table B6.1 below.

B6.1.3.3. Load Models HSLM

cl. 6.4.6.1.1: EN 1991-2

As previously mentioned in Section B6.1.1, Load Model HSLM comprises two separate universal trains with variable coach lengths. In order to ensure that they deliver dynamic behaviour with regard to current and future train traffic, bridges should be calculated using the Universal Dynamic Train (HSLM) consisting of HSLM-A and/or HSLM-B. These are defined as follows:

- For the definition of train HSLM-A, a set of ten reference trains A1 to A10: see Fig. B6.6 and Table B6.2 below.
- For the definition of train HSLM-B: see Figs B6.7 and B6.8 below.

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Table B6.1. Technical Specifications for Interoperability of rolling stock

Regular trains Type TALGO	$10\,\text{m} \leq D \leq 14\,\text{m}$ $P \leq 170\,\text{kN}$ $7\,\text{m} \leq e_c \leq 10\,\text{m}$ $8 \leq D_{1C} \leq 11\,\text{m}$ where D_{1C} = coupling distance between power car and coach e_c = coupling distance between two train sets
Articulated trains Type EUROSTAR, TGV	$18\,\text{m} \leq D \leq 27\,\text{m}$ $P \leq 170\,\text{kN}$ $2.5\,\text{m} \leq d_{BA} \leq 3.5\,\text{m}$
Conventional trains Type ICE, ETR, VIRGIN	$18\,\text{m} \leq D \leq 27\,\text{m}$ and $P < 170\,\text{kN}$ or values translating the inequality below: $4P\cos\left(\frac{\pi d_{BS}}{D}\right)\cos\left(\frac{\pi d_{BA}}{D}\right) \leq 2P_{HSLMA}\cos\left(\frac{\pi d_{HSLMA}}{D_{HSLMA}}\right)$ (EN 1991-2, (E.2))
All types	$L < 400\,\text{m}$ $\Sigma P \leq 10\,000\,\text{kN}$

Note: where D, D_{1C}, P, d_{BA}, d_{BS} and e_c are defined for articulated, conventional and regular trains in Figs B6.3 to B6.5 above.

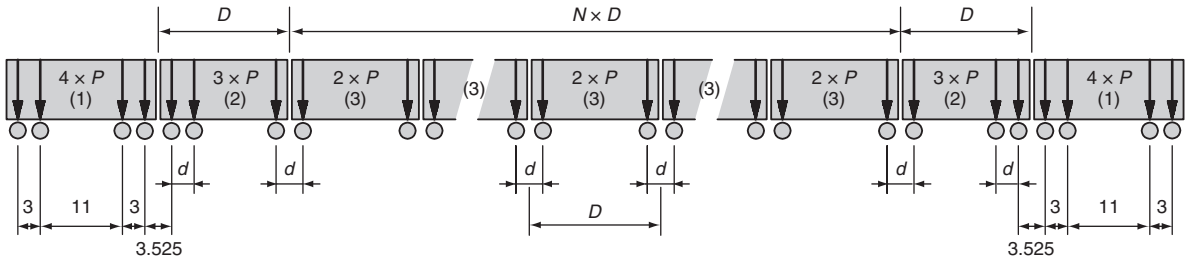


Fig. B6.6. Diagram of Universal Dynamic Train HSLM-A (Reproduced from EN 1991-2, with permission from BSI)

This Load Model comprises N number of point forces of 170 kN at regular spacing d(m) (Fig. B6.7) where N and d are defined in Fig. B6.8.

Table B6.3 illustrates how HSLM-A and HSLM-B are applied and indicates the trains to be used for dynamic bridge calculations.

Table B6.2. HSLM-A, definition of the ten trains (Data taken from EN 1991-2, Table 6.3; see EN 1991-2 for missing values)

Universal train	Number of intermediate coaches, N	Coach length D (m)	Bogie axle spacing d (m)	Point force P (kN)
A1	18	18	2.0	170
A2	17	19	3.5	200
A3				
A4	15	21	3.0	190
A5	14	22	2.0	170
A6				
A7	13	24	2.0	190
A8	12	25	2.5	190
A9				
A10	11	27	2.0	210

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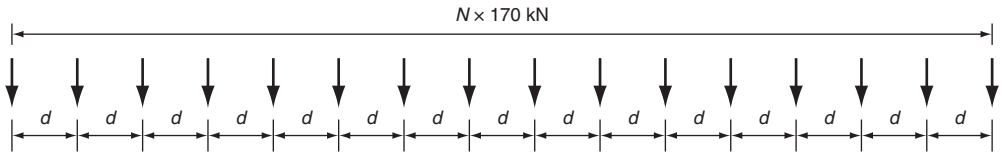


Fig. B6.7. Diagram of Universal Dynamic Train HSLM-B (Reproduced from EN 1991-2, with permission from BSI)

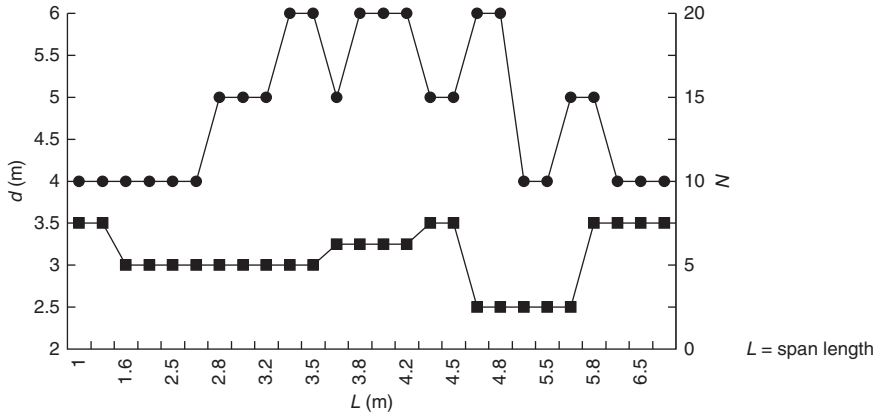


Fig. B6.8. Universal Dynamic Train HSLM-B (Reproduced from EN 1991-2, with permission from BSI)

B6.1.3.4. Load distribution

cl. 6.4.6.4(3): EN 1991-2

The representation of each axle by a single point force tends to overestimate dynamic effects for loaded lengths of less than 10 m. In such cases, the load distribution effects of rails, sleepers and ballast may be taken into account, not only for real trains but also for load models HSLM. This leads for example to a reduction of the calculated accelerations.

B6.1.3.5. Load combinations and partial factors

cl. 6.4.6.1.2: EN 1991-2

For dynamic analysis the calculation of the value of mass associated with self-weight and removable loads (ballast etc.) should use nominal values of density.

Table B6.3. Application of HSLM-A and HSLM-B (Data taken from EN 1991-2, Table 6.4)

Structural configuration	Span	
	$L < 7$ m	$L \geq 7$ m
Simply supported span[a]	HSLM-B[b]	HSLM-A[c]
Continuous structure[a] or Complex structure[e]	HSLM-A Trains A1 to A10 inclusive[d]	HSLM-A Trains A1 to A10 inclusive[d]

[a] Valid for bridges with only longitudinal line beam or simple plate behaviour with negligible skew effects on rigid supports.
[b] For simply supported spans with a span of up to 7 m, a single critical Universal Train from HSLM-B may be used for the analysis in accordance with 6.4.6.1.1(5).
[c] For simply supported spans with a span of 7 m or greater a single *(Note: only one)* critical Universal Train from HSLM-A may be used for the dynamic analysis in accordance with EN 1991-2, Annex E. (Alternatively Universal trains A1 to A10 inclusive may be used.)
[d] All Trains A1 to A10 inclusive should be used in the design.
[e] Any structure that does not comply with Note a above. For example, a skew structure, bridge with significant torsional behaviour, half-through structure with significant floor and main girder vibration modes etc. In addition, for complex structures with significant floor vibration modes (e.g. half-through or through-bridges with shallow floors), HSLM-B should also be applied.
Note: The National Annex or the individual project may specify additional requirements relating to the application of HSLM-A and HSLM-B to continuous and complex structures.

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Example B6.1. Determination of the critical Universal Train HSLM-A (*EN 1991-2, Annex E*)

$L = 15\,\text{m}$, simple supported bridge
$f_0 = 6\,\text{Hz}$
$\zeta = 1\%$
$v_{\max} = 420 \times 1.2 = 500\,\text{km/h}$ (maximum design speed)

so that $\lambda_{\max} = v_{\max}/f_0 = 500/3.6/6 = 23\,\text{m}$.

The aggressiveness curve is plotted on the top of Fig. B6.9.

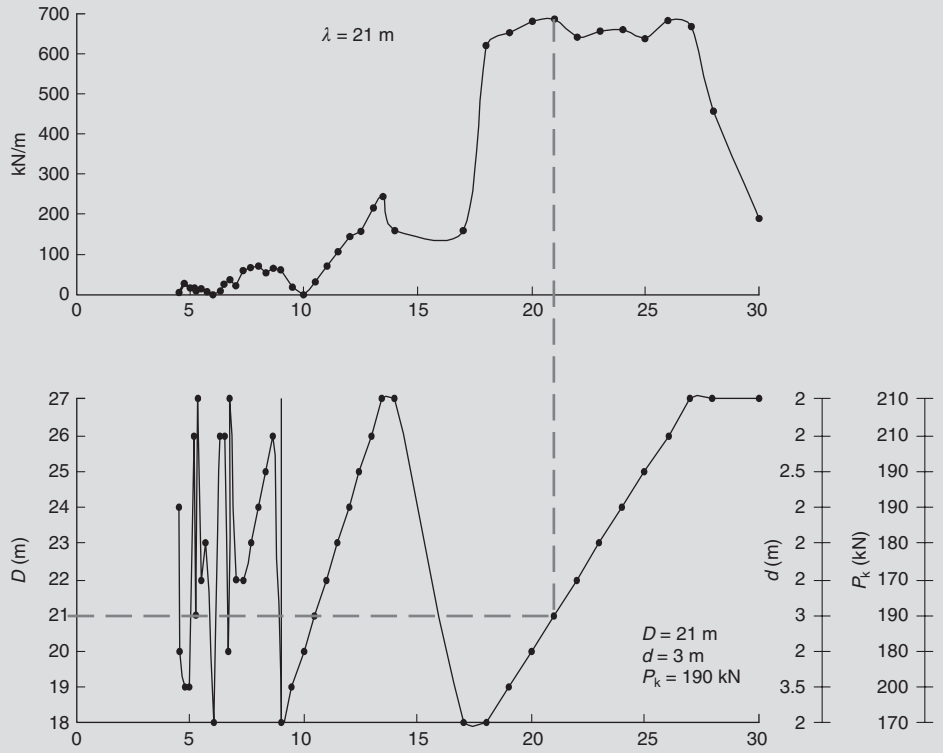


Fig. B6.9. Example of calculation, using agressiveness of trains for $L = 15\,\text{m}$ (See EN 1991-2, Fig. E.7) and the wavelength–train relationship parameters for defining the critical Universal Train HSLM-A (Reproduced from EN 1991-2, with permission from BSI)

On the aggressiveness curve given, the maximum is located at $\lambda = 21\,\text{m}$. The bottom curve shows the values D, d and P_k allowing this maximum to be reached:

$D = 21\,\text{m}$
$d = 3\,\text{m}$
$P_k = 190\,\text{kN}$

The dynamic calculation will be performed with the HSLM-A train corresponding to these values.

The dynamic analysis shall be undertaken using characteristic values of the loading from real trains specified. The dynamic analysis shall also be undertaken using Load Model HSLM on bridges designed for international lines, where European high speed inter-operability criteria are applicable.

Only one track (the most adverse) on the structure should be loaded in accordance with Table B6.4.

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Table B6.4. Summary of additional load cases depending upon number of tracks on bridge (data taken from EN 1991-2, Table 6.5)

Number of tracks on bridge	Loaded track	Loading for dynamic analysis
1	One	Each real train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel
2 (trains normally travelling in opposite directions)[a]	Either track	Each real train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel
	Other track	None

[a] For bridges carrying two tracks with trains normally travelling in the same direction or carrying three or more tracks with a maximum line speed at the site exceeding 200 km/h, the loading should be agreed with the relevant authority specified in the National Annex.

Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures) on a track, the load effects from a dynamic analysis should be combined with:

- the load effects from horizontal forces on the track subject to the loading in the dynamic analysis
- the load effects from vertical and horizontal loading on the other track(s), in accordance with the requirements given in 6.12.1 and Table 6.5 of this Designers' Guide.

Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures), the dynamic rail loading effects (bending moments, shears, etc., excluding acceleration) determined from the dynamic analysis have to be enhanced by the partial factors given.

A1, A2: EN 1990: 2002

Partial factors need not be applied to the loadings of real trains and the Load Model HSLM when determining bridge deck accelerations. The calculated values of acceleration have to be directly compared with the design values in B6.1.4.

A2.4.4.2.1(4)P: EN 1990/A1

B6.1.3.6. Speeds to be considered

cl. 6.4.6.2: EN 1991-2

For each real train and Load Model HSLM a series of speeds up to the maximum design speed need to be considered. The maximum design speed is taken to be generally 1.2 × maximum line speed at the site.

cl. 6.4.6.2(1)P: EN 1991-2

The maximum line speed at the site needs to be specified (see also Notes 1 to 5).

Calculations should be made for a series of speeds from 40 m/s up to the maximum design speed. Smaller speed steps should be made in the vicinity of resonant speeds.

For simply supported bridges that may be modelled as a line beam, the resonant speeds may be estimated using:

$$v_i = n_0 \lambda_i \qquad \text{EN 1991-2, (6.9)}$$

and

$$40\,\text{m/s} \leq v_i \leq \text{maximum design speed} \qquad \text{EN 1991-2, (6.10)}$$

where

v_i is the resonant speed (m/s)
n_0 is the first natural frequency of the unloaded structure
λ_i is the principal wavelength of frequency of excitation and may be estimated by:

$$\lambda_i = \frac{d}{i} \qquad \text{EN 1991-2, (6.11)}$$

d is the regular spacing of groups of axles
i = 1, 2, 3 or 4.

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B6.1.4. Principal supplementary design checks

The following additional dynamic verifications are always carried out under real trains or under universal dynamic loaded trains (HSLM) incremented by the corresponding dynamic coefficient. In comparison to the design of railway bridges on conventional routes, the additional principal design rules that often dictate the design of a railway bridge on a high-speed route are as follows.

- **Verification of maximum peak deck acceleration along each track**

  To ensure traffic safety the verification of maximum peak deck acceleration due to rail traffic actions needs to be regarded as a traffic safety requirement checked at the serviceability limit state (railway traffic safety) for the prevention of track instability. In cases where the bridges have ballasted tracks, intense accelerations of the deck create the risk of destabilizing the ballast.

  A2.4.4.2.1(1)P: EN 1990: 2002/A1

  The maximum peak values of bridge deck acceleration calculated along each track must not exceed the following design values:

  A2.4.4.2.1(4)P: EN 1990: 2002/A1

  – γ_{bt} for ballasted track
  – γ_{df} for direct fastened tracks

  for all members supporting the track, considering frequencies (including consideration of associated mode shapes) up to the greater of:

  – 30 Hz
  – 1.5 times the frequency of the fundamental mode of vibration of the member being considered
  – the frequency of the third mode of vibration of the member.

  Note: The recommended values are:

  $\gamma_{bt} = 0.35\,g\ (3.43\,m/s^2)$
  $\gamma_{df} = 0.50\,g\ (4.91\,m/s^2)$

- **Verification of whether the calculated load effects from high-speed rail traffic, including HSLM on high-speed interoperable routes, are greater than those of normal rail traffic loading (LM71″+″SW/0)**

  For the design of the bridge, taking into account all the effects of vertical traffic loads, the most unfavourable value of:

  $$\left(1+\varphi'_{dyn}+\varphi''/2\right)\times\begin{pmatrix}\text{HSLM}\\ \text{or}\\ \text{RT}\end{pmatrix}\ \text{or}\ \Phi\times(\text{LM71}''+''\text{SW}/0)\quad \text{EN 1991-2, (6.15 + 6.16)}$$

  has to be used.

  The following dynamic enhancement is determined from the dynamic analysis:

  $$\varphi'_{dyn}=\max\left|y_{dyn}/y_{stat}\right|-1 \qquad \text{EN 1991-2, (6.14)}$$

  where

y_{dyn}	is the maximum dynamic response and y_{stat} the corresponding maximum static response at any particular point in the structural element due to a real train (RT) or high-speed load model (HSLM)
LM71″+″SW/0	is Load Model 71 and if relevant Load Model SW/0 for continuous bridges (and classified vertical load where required for ULS)
$\varphi''/2$	is defined in Annex C of EN 1991-2 (here written for carefully maintained track)
Φ	is the dynamic factor given in accordance with Section 6.8.3.

  The following should be checked: all elastomechanical action effects such as M (moments), Q (shear forces), y (deflections), σ (normal stresses), δ deformations, τ (shear stresses), ε (strains) and γ (shear deformations) at any point of the structure.

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Table B6.5. Recommended levels of comfort (Data taken from EN 1990: 2002/A1, Table A2.9)

Level of comfort	Vertical acceleration b_v (m/s^2)
Very good	1.0
Good	1.3
Acceptable	2.0

Note: Alternatively the vertical acceleration b_v may be determined by a dynamic vehicle–bridge interaction analysis (see EN 1990: 2002/A1, A2.4.4.3(3)), but this is only possible for real trains and not for HSLM, where no car characteristics are given.

- **Additional verification for fatigue where dynamic analysis is required** *cl. 6.4.6.6: EN 1991-2*

  First of all, the fatigue assessment, a stress range verification, is carried out according to Section 6.13, with the reference fatigue loading LM71 and with $\alpha = 1.0$. The traffic mix given in EN 1991-2, Annex D.3 contains two high-speed passenger trains with speeds of 250 km/h.

  Fatigue increases not only with the number and the weight of trains but also with the speed of the trains. Conventional railway bridge design fatigue calculations based on live load stress ranges due to $\Phi \times$ LM71 etc. are therefore not necessarily sufficient.

  For bridges designed for HSLM, a fatigue approach is likely to be impracticable. In such cases it is recommended that the design takes into account the best estimate of actual and anticipated future high speed traffic. However, if the frequent operating speed of a chosen high-speed train at a site is near to a resonant speed, the static system of the bridge should be changed. This is in contradiction to the rule given in ***Clause 6.4.6.6(2)P: EN 1991-2***, where a fatigue check will also allow for the additional fatigue loading at resonance cycles of stress caused by the dynamic loading and the associated bridge response at resonance. *cl. 6.4.6.6(2)P: EN 1991-2*

- **Verification of limiting values for the maximum vertical deflection for passenger comfort** *A2.4.4.3: EN 1990: 2002/A1*

  In order to establish a maximum value that effectively translates the accelerations within the vehicle, it is important to know how vibrations impact passenger comfort and well-being. A certain number of physiological criteria linked to frequency, intensity of acceleration, steering relative to the spinal column and time of exposure (duration of vibrations) make it possible to assess vibrations and their influence on individuals. The limit exposure time to reduced comfort represents the limit of comfort adopted. These paragraphs characterize the flexibility of bridges with regard to comfort.

  Passenger comfort depends on the vertical acceleration b_v inside the coach during travel on the approach to, passage over and departure from the bridge.

  The maximum acceleration in the coach for ensuring the required level of passenger comfort may be defined for the individual project. Recommended levels of comfort are given in Table B6.5.

  Deflection criteria for checking passenger comfort are defined as follows.

  The maximum permissible vertical deflection δ along the centre-line of the track of railway bridges is a function of:

  - the span length
  - the train speed V (km/h)
  - the number of spans
  - the number of spans and the configuration of the bridge (simply supported beam, continuous beam).

  To limit vertical vehicle acceleration to the values given in Table B6.4, values for permissible deflections are given in ***EN 1990: 2002/A1, A.2.4.4.3.2***, and especially in ***EN 1990: 2002/A1, Fig. A.2.3***. *A2.4.4.3.2: EN 1990: 2002/A1* *Fig. A.2.3: EN 1990: 2002/A1*

  Note: There is no need to check vertical deflection for passenger comfort, if the severe permissible deformations to avoid excessive track maintenance mentioned in Chapter 8

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(Table 8.12) of this Designers' Guide are respected. This choice gives no more expensive investment costs for the bridges when taking into account life-cycle cost analysis.

- **Verification of twist**
  Twist also takes a different value under the dynamic effect of operating loads. This is expressed as dynamic twist t_{dyn}.

  In Section 8.7.4 of this Designers' Guide, twist of the deck is calculated with the characteristic value of Load Model 71 (and where required Load Model SW/0), multiplied by Φ and α, as well as with Load Model SW/2 multiplied by Φ, when heavy abnormal rail traffic may operate. The permissible values are given in Table 8.11 of this Designers' Guide.

  When HSLM or real trains are determinant for the design of a bridge, due to the draft of UIC Code 776-2,[7] an additional check is necessary as follows:

  $$t_{dyn} \leq 1.2\,\text{mm}/3\,\text{m}$$

  This must take into consideration the vertical traffic loads on one track, including the effects of centrifugal forces.

cl. 6.4.6.3.1: EN 1991-2

B6.1.5. Bridge parameters

B6.1.5.1. Structural damping

Structural damping is a key parameter in dynamic analysis. The magnitude of the vibrations depends heavily on structural damping, especially in proximity to resonance.

cl. 6.4.6.1.3(3): EN 1991-2

Only lower-bound estimates should be used in the dynamic analysis. Table B6.6 gives the lower limits of the percentage values of critical damping $\zeta(\%)$ based on a certain number of past measurements (see also ERRI reports D214[16]).

For spans less than 30 m dynamic vehicle–bridge mass interaction effects tend to reduce the peak response at resonance. Account may be taken of these effects by:

- carrying out a dynamic vehicle–structure interactive analysis
- increasing the value of damping assumed for the structure according to EN 1991-2, Fig. 6.15. For continuous beams, the smallest value $\Delta\zeta$ for all spans should be used. The total damping to be used is given by:

$$\zeta_{TOTAL} = \zeta + \Delta\zeta \qquad \text{EN 1991-2, (6.12)}$$

where

$$\Delta\zeta = \frac{0.0187L - 0.00064L^2}{1 - 0.0441L - 0.0044L^2 + 0.000255L^3}(\%) \qquad \text{EN 1991-2, (6.13)}$$

ζ is the lower limit of percentage of critical damping (%) defined above.

cl. 6.4.6.3.2: EN 1991-2

B6.1.5.2. Mass of the bridge

Maximum dynamic effects occur at resonance peaks, where a multiple of the load frequency coincides with the natural frequency of the structure. Underrating the mass will lead to overestimation of the natural frequency of the structure and of the speed at which resonance occurs.

Table B6.6. Percentage values of critical damping ζ (%) for different bridge types and span lengths L (Data taken from EN 1991-2, Table 6.6; see EN 1991-2 for missing values)

cl. 7.4.3: EN 1992-1-1

Type of bridge	Lower limit of the percentage of critical damping ζ (%)	
	Span length $L < 20$ m	Span length $L \geq 20$ m
Steel and composite	$\zeta = 0.5 + 0.125(20 - L)$	$\zeta = 0.5$
Filler beams and reinforced concrete		
Prestressed concrete	$\zeta = 1.0 + 0.07(20 - L)$	$\zeta = 1.0$

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At resonance, the maximum acceleration of a structure is inversely proportional to the distributed mass of the structure. *Therefore two extreme cases for the mass of the structure and the ballast must be considered in the dynamic analysis*: *cl. 6.4.6.3.2(2): EN 1991-2*

- A lower limit of the mass of the structure, together with the minimum density and thickness of the clean ballast, to obtain the maximum possible acceleration of the bridge deck.
- An upper limit of the mass of the structure, together with the maximum density and thickness of the saturated ballast (ballast with slag and with allowance for future track lifts), to obtain the lowest possible estimation of the fundamental frequency and speed at which the resonance can occur.

The density of materials should be taken from EN 1991-1-1. The minimum density of ballast may be taken as $1700\,kg/m^3$.

cl. 6.4.6.3.3: EN 1991-2

B6.1.5.3. Stiffness of the bridge

Maximum dynamic load effects are likely to occur at resonant peaks when a multiple of the frequency of loading and a natural frequency of the structure coincide. Any overestimation of bridge stiffness will overestimate the natural frequency of the structure and speed at which resonance occurs; it provides conservative results.

A lower-bound estimate of the stiffness throughout the structure has to be used.

The stiffness of the whole structure including the determination of the stiffness of elements of the structure may be determined in accordance with EN 1992 to EN 1994.

Values of Young's modulus may be taken from EN 1992 to EN 1994.

In ***Clause 6.4.6.3.3(3): EN1991-2***, concerning concrete, the following subclause with its first Note is written as follows: *cl. 6.4.6.3.3(3): EN 1991-2*

For concrete compressive cylinder strength $f_{ck} \geq 50\,N/mm^2$ (compressive cube strength $f_{ck,cube} \geq 60\,N/mm^2$) the value of static Young's modulus (E_{cm}) should be limited to the value corresponding to a concrete of strength $f_{ck} = 50\,N/mm^2$ ($f_{ck,cube} = 60\,N/mm^2$).

Note 1: Owing to the large number of parameters which can affect E_{cm} it is not possible to predict enhanced Young's modulus values with sufficient accuracy for predicting the dynamic response of a bridge. Enhanced E_{cm} values may be used when the results are confirmed by trial mixes and the testing of samples taken from site in accordance with EN 1990, EN 1992 and ISO 6784 subject to the agreement of the relevant authority specified in the National Annex.

Note: Where an assessment of existing concrete or composite bridges is undertaken, the increase in the magnitude of Young's modulus of concrete with time should be considered.

Members that are expected to crack, such as in reinforced concrete bridges, but may not be fully cracked, will behave in a manner intermediate between the uncracked and fully cracked conditions. For members subjected to bending an adequate prediction of behaviour is given in ***Clause 7.4.3: EN1992-1-1***. *cl. 7.4.3: EN 1991-1-1*

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CHAPTER 7

Accidental actions

This chapter is concerned with the determination of accidental actions and actions for the accidental design situations in accordance with EN 1990 applicable to bridges. The material in this chapter is covered in EN 1991-2 *Traffic loads on bridges* and EN 1991-1-7 Accidental actions.[1] Both these Parts of EN 1991 are intended to be used in conjunction with EN 1990, the other Parts of EN 1991 and EN 1992 to EN 1999 for the design of structures.

Actions for accidental design situations due to vehicles on bridge decks are defined in EN 1991-2 and are already developed in Chapters 4 and 6 of this Designers' Guide.

In this chapter, the following actions are more specifically developed:

- actions due to vehicle impact on bridge piers and decks (road vehicles and trains)
- actions due to ship impact on bridge piers and decks.

Notional values for identified accidental actions (e.g. in the case of internal explosions and impact) are proposed in EN 1991-2. These values may be altered in the National Annex or for an individual project and agreed for the design by the client and/or the relevant authority.

7.1. Accidental actions – general aspects

EN 1990 *Basis of structural design*, based on semi-probabilistic concepts, gives several classifications of actions. For common combinations of actions, the classification of actions distinguishes permanent, variable and accidental actions.

A permanent action is an action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value. A variable action is an action for which the variation in magnitude with time is neither negligible nor monotonic. And an accidental action is an action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life.

Accidental actions include mainly forces due to impact, explosions, soil subsidence, exceptional snow falls or earth avalanches, and tornados in countries that are normally not subject to such climatic phenomena. In common language, an accidental action corresponds to a rather rare phenomenon, unforeseeable, and with possible severe or catastrophic consequences unless an appropriate protection is ensured.

An action may not be accidental in itself. An action is often considered as an accidental action because it corresponds to a rare event, therefore the lack of data does not permit a satisfactory application of statistical treatments, and also for economic reasons because the cost of a systematic protection would not be reasonable. A good example is given by

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snow loads: it has been necessary to introduce in EN 1991-1-3 not only characteristic values but also accidental values to take into account exceptional snow falls.

In conclusion, in many cases, it is more appropriate to consider a relevant accidental situation rather than an accidental action. This means that before defining an accidental ultimate limit state, one has to consider if the corresponding situation is really accidental, i.e. if it is really a situation for which it is not intended to ensure the structural integrity, but only to avoid loss of human life.

The transmission of impact forces to the various members of the structure is determined by the use of models, including models for ground–structure interaction. Structural analysis in the case of impact is outside the scope of EN 1991-1-7, but some dynamic aspects are evoked.

Obviously, the actions due to impact and the mitigating measures provided should take into account, among other things, the type of traffic on and under the bridge and the consequences of the impact.

cl. 1.5.1.5: EN 1991-1-7

Robustness is defined in EN 1991-1-7 as the ability of a structure to withstand events such as fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause. Robustness is not specifically evoked for bridges, but some measures are often adopted when designing some types of bridges. For example, in the case of cable-stayed bridges, the structural resistance is often checked assuming that two or three stays are removed (accidental rupture or normal maintenance). Of course, the dynamic effects depend on the type of suspension break.

EN 1991-1-7 does not specifically deal with accidental actions caused by external explosions, warfare and terrorist activities, or the residual stability of buildings or other civil engineering works damaged by seismic action or fire, etc. Nevertheless, such situations may have to be taken into account for the design of bridges, depending on their exposure in some special locations (e.g. a strategic bridge located in the vicinity of a factory producing dangerous products).

cl. 1.5.1.3: EN 1991-1-7

EN 1991-1-7 gives the very important definition of risk as a measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence (see Table 7.9 later). EN 1990 introduces only the concept of consequence class as a function of the consequences of failure of the structure or part of it. Certainly, there is a strong link between risk and class of consequences, but the risk has a quantification aspect.

In any case, a zero risk level cannot be reached and in most cases it is necessary to accept a certain risk level. Such a risk level can be determined by various factors, such as the potential number of casualties, the economic consequences and the cost of safety measures, etc.

7.2. Accidental design situations

cl. 3.1(2): EN 1991-1-7

EN 1991-1-7 introduces the concept of a strategy to avoid accidental situations or to control the consequences of the various accidental design situations selected by the designer and agreed by the client or the relevant authority. Two types of strategies are envisaged: strategies based on identified accidental actions; and strategies based on limiting the extent of localized failure. They are summarized in Fig. 7.1 (Fig. 3.1 of EN 1991-1-7).

The Eurocode does not give an accurate definition of identified (and subsequently unidentified) accidental actions. However, it is possible to define identified accidental actions as accidental actions that can physically occur, of course with a very low probability, but without being associated with an exceptional situation. In other words, an identified accidental action has a statistical reality when considering a large number of construction works of the same type.

In the case of bridges, the following actions or situations may be considered as identified actions or situations:

- an impact from road vehicles, trains or ships on piers, decks, or other structural members (Figs 7.2 and 7.3) located near the infrastructure under consideration

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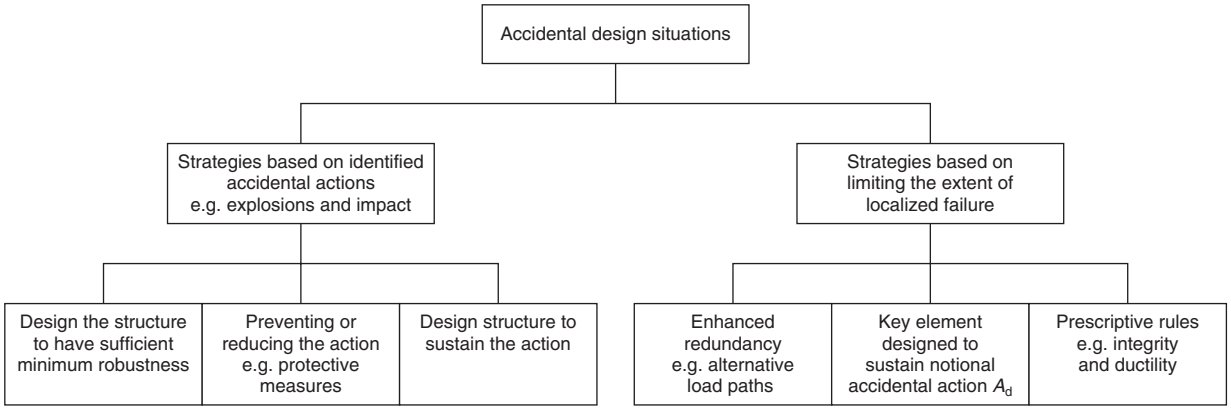


Fig. 7.1. Strategies for accidental design situations (Reproduced from EN 1991-1-7, with permission from BSI)

- the effects of fire, for example due to a lorry carrying flammable products, exploding or burning over or under a bridge deck (Fig. 7.4)
- scour effects around bridge piers or abutments for a bridge crossing a river
- overloading due to very heavy vehicles not authorized to cross the bridge or for which the bridge has not been designed.

Unidentified accidental actions may have various origins:

- actions or situations due to vandalism, for example a voluntary deterioration of cables of a cable-stayed bridge
- actions developing in exceptional conditions (impact from a plane on the masts of a suspension or cable-stayed bridge).



Fig. 7.2. Lorry impact on structural members of a suspension bridge

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Fig. 7.3. Example of protection of the lateral truss beams of a bridge with appropriate road restraint systems

Strictly speaking these actions may be identified actions which may not be considered, as the risk of them occurring may be very low. If the strategy for an unidentified action (i.e. limiting the amount of damage) is adopted, some protection may be assured from exceptional actions which have not been designed for.

At the design stage, the designer has to:

- establish a set of accidental design situations, including identified and possibly unidentified accidental actions, in agreement with the client and the relevant authority for the individual project
- adopt protection measures as far as possible



Fig. 7.4. Fire accident at the Wiehltal bridge (near Köln, Germany), 26 August 2004 (Courtesy of Anja Langner, Udo Langner, Georg Madalinsky, PSP)

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Table 7.1. Definition of consequences classes (Data taken from EN 1990 (Annex B), Table B.1)

Consequence class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

- ensure a robust structure if some accidental situations cannot be avoided for various reasons (physical, economical, etc.).

The concept of localized failure, which is defined as that part of a structure that is assumed to have collapsed, or been severely disabled, by an accidental event, may be relevant for a bridge. However, in general, the concept of a key element, defined as a structural member upon which the stability of the remainder of the structure depends after a localized failure, is mostly applicable to buildings. See the TTL *Designers' Guide to Eurocode 1: Actions on Buildings*.[2]

cl. 1.5.1.2: EN 1991-1-7

cl. 1.5.10: EN 1991-1-7

Examples of design measures to ensure a minimum robustness in the case of bridges include:

- providing adequate clearances between the trafficked lanes and the structure
- reducing the effects of the action on the structure, by protective bollards, safety barriers, cables to stop ships before a collision, etc.
- avoiding fragile or very light bridge decks if the risk of impact (e.g. by a mobile crane) is not negligible
- imposing some serviceability criteria for a cable-stayed bridge in the absence of one or several stays, under reduced loading
- limiting the accepted damaged length for a long bridge in case of collision with a seagoing vessel (the accepted damaged length may be reduced to 0).

If during the execution of a bridge it is subjected to an extreme event (e.g. a bridge located in a cyclonic country), where there is no risk to human life, and where economic, social or environmental consequences are negligible, the complete collapse of the structure caused by this extreme event may be preferable to over-dimensioning, superfluous when the structure is completed. Such a design strategy may be adopted in other circumstances and it is always the result of an accurate process and a motivated decision.

From a general viewpoint, EN 1991-1-7 suggests the adoption of strategies for accidental design situations based on the consequence classes defined in Table 7.1 which derives from Table B.1 of EN 1990 (Annex B).

In general, bridges belong to class CC2, but some of them may be considered as belonging to class CC3. When classified in CC2 consequence class, and depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied. In any case, the safety levels have to be accurately defined, depending on the level of the quality control for the design or for the execution. Of course, it is generally appropriate to treat some parts of the structure as belonging to a different consequence class, in particular for parts that may be replaced, such as cable stays or structural bearings. When classified into CC3 consequence class, a risk analysis and the use of refined methods such as dynamic analyses, non-linear models and interaction between the load and the structure may be needed.

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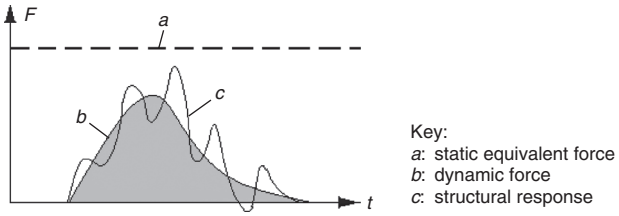


Fig. 7.5. Definitions related to actions due to impact (Reproduced from EN 1991-1-7, with permission from BSI)

7.3. Actions due to impact – general aspects

Impact loading is the result of a collision between two objects. In the case of bridges, the most common colliding objects are vehicles, ships, or even airplanes, that have an intended course. However, the occurrence of a human or mechanical failure may lead to a deviation of the intended course: these occurrences may be described by a probabilistic approach (e.g. a homogeneous Poisson process). After the initial failure, the course of the object will depend on its properties and the environment.

In principle, the mechanical effects of an impact should be determined by a dynamic analysis, taking into account the effects of time and the real behaviour of materials. In fact, this problem is very difficult and needs very complex and high-level numerical calculations (e.g. the study of the crash of a ship bow needs a finite-element model of about 10 000 elements and the results depend on the selected boundary conditions, especially for the assessment of instability aspects).

Therefore, sophisticated models of greater or lesser complexity are needed to study impact loading. A collision force is a dynamic force, i.e. a force, with an associated contact area at the point of impact, that varies in time and which may cause significant dynamic effects on the structure. It depends on the interaction between the impacting object and the structure. However, in common cases, actions due to impact are represented by an equivalent static force, i.e. an alternative representation for the dynamic force intended to cover the dynamic response of the structure without refined calculations.

cl. 1.5.5: EN 1991-1-7

cl. 4.2.1: EN 1991-1-7

This simplified representation gives acceptable results for the verification of static equilibrium, as well as for strength verifications and for the determination of deformations of the impacted structure. Figure 7.5 gives a simplified representation of a dynamic force, the structural response and the static equivalent force.

C.2(1): EN 1991-1-7

The Eurocode defines the concepts of **hard and soft impact**.

Hard impact corresponds to collision effects in the case of structures for which the energy is mainly dissipated by the impacting body.

Soft impact corresponds to collision effects in the case of structures which are designed to absorb impact energy by elastic-plastic deformations of members.

In fact, in many cases, collision effects are intermediate between hard and soft impact (Fig. 7.6): for simplicity, the impact load is determined using the 'rigid structure' assumption, i.e. using a 'hard impact' model. The impacting force may be represented by an equivalent static force.

cl. 1.5.6: EN 1991-1-7

7.4. Accidental actions caused by road vehicles

7.4.1. Impact on supporting substructures – simplified approach

(Definition: In EN 1991-1-7, the substructure is defined as that part of a building structure that supports the superstructure, i.e. foundations, abutments, piers and columns etc. The superstructure is defined as that part that usually relates to the bridge deck.)

The supporting substructures of bridges are their piers and abutments. EN 1991-1-7 envisages impact from lorries and cars for road bridges. Annually, along main routes in all European countries, several severe impacts from road vehicles against bridge piers are observed.

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Fig. 7.6. Impact on a bridge pier

As defined in the Eurocodes, a 'lorry' is a vehicle with maximum gross weight greater than 3.5 t and impact from lorries and cars is envisaged in courtyards and parking garages. In this Designers' Guide, only lorry impact is envisaged. For hard impact from road traffic, EN 1991-1-7 gives indicative values of equivalent static design force and recommended conditions. The proposed rules are represented in Fig. 7.7.

The reader's attention is drawn to the fact that the same symbol, h, is used for the height of the collision force above the level of the carriageway and for the physical clearance between the road surface and the underside of the bridge deck.

The model of hard impact on supporting substructures consists of two forces, F_{dx} in the direction of normal travel and F_{dy} in the direction perpendicular to the direction of normal travel. These two forces are normally not taken into account simultaneously.

Their position is defined by the height h above the level of the carriageway or higher where certain types of protective barriers are provided. Figure 7.8 shows the collision of a lorry against a bridge pier on the French motorway A11; the lorry slipped on a concrete safety barrier and impacted the pier at a rather high level.

The recommended application area of the impact force is a rectangle of height a and width b. In Fig. 7.7, the application area of F_{dx} only is represented.

Indicative values for F_{dx} and F_{dy} are given in Table 7.2 which derives from Table 4.1 of EN 1991-1-7. *cl. 4.3.1: EN 1991-1-7*

For various reasons, the design values given in Table 7.2 are indicative only. Indeed, the choice of values may take account of:

- the distance s of the centre-line of the nearest trafficked lanes to the structural member (see Fig. 7.9). Information on the effect of the distance s, where applicable, can be found in Annex C of the Eurocode *C.3: EN 1991-1-7*

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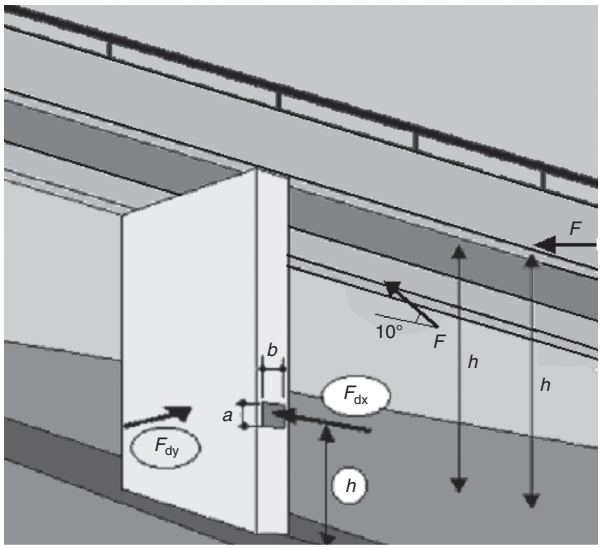


Fig. 7.7. Representation of impact forces from road vehicles



Fig. 7.8. Accident on the French motorway A11 (28 June 1997). The lorry slid on the concrete safety barrier and impacted a pier at a rather high level

Table 7.2. Indicative equivalent static design forces due to vehicular impact on members supporting structures over or adjacent to roadways

Category of traffic	Force F_{dx} (kN)	Force F_{dy} (kN)	Height h of collision force (m)	Dimensions of impact area (m)
Motorways and country national and main roads	1000	500	$0.50 \leq h \leq 1.50$ (or more for special circumstances)	$a = 0.50$ m $b =$ min. of 1.5 m or member width
Country roads in rural area	750	375		
Roads in urban area	500	250		

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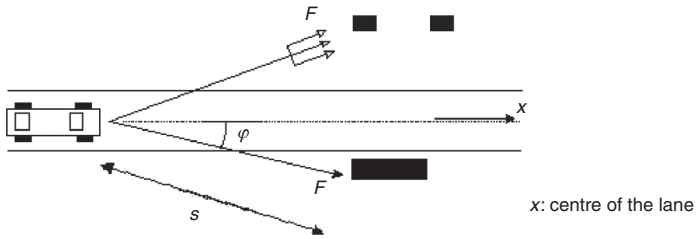


Fig. 7.9. Collision force on supporting substructures near traffic lanes (Reproduced from EN 1991-1-7, with permission from BSI)

- the consequences of the impact
- the expected volume and type of traffic
- any mitigating measures provided.

The design values may be defined on the basis of a risk analysis: **they may be lower (this option is not recommended by the authors) or higher than the values given in Table 4.1 of EN 1991-1-7**.

The UK National Annex to EN 1991-1-7 applies a factor to the values in EN 1991-1-7 which is determined by a comprehensive risk analysis explained in the National Annex.

As previously mentioned, a height h above the carriageway level more than 1.5 m may be specified where certain types of protective barriers are provided.

In the case of accidental actions caused by road vehicles on bridges also carrying rail traffic, the Eurocode recommends the UIC leaflet 777.1.[3] *Note 5 to cl. 4.3.1(1): EN 1991-1-7*

7.4.2. Impact on superstructures

Impact on members of the superstructure from road traffic (lorries and/or loads carried by the lorries) is to be taken into account unless adequate clearances or suitable protection measures to avoid impact are provided. It should be remembered that the clearance is measured perpendicular to the road (Fig. 7.10) and that allowance should be made for any possible future reduction caused by the resurfacing of the roadway under the bridge. In general, a complementary thickness equal to 10 cm is taken into account at the design stage.

EN 1991-1-7 gives indicative values of equivalent static impact forces on bridge decks. Of course the risk depends on the vertical clearance (Fig. 7.11). *cl. 4.3.2(1): EN 1991-1-7*

The idea is that the indicative values given in Table 7.3 (see below) apply for a value of the clearance below a value h_0 to be defined at the national level, the recommended value being

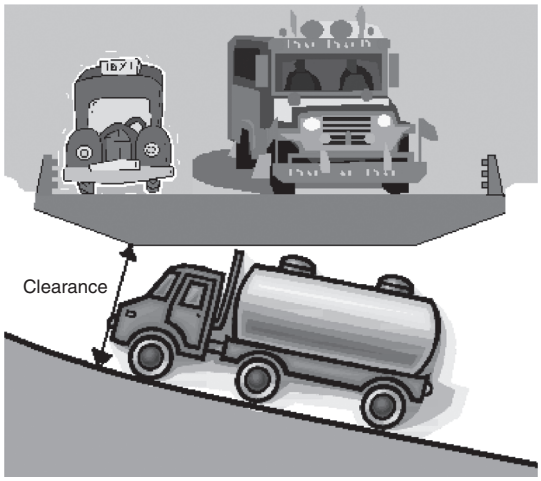


Fig. 7.10. Clearance under a bridge deck

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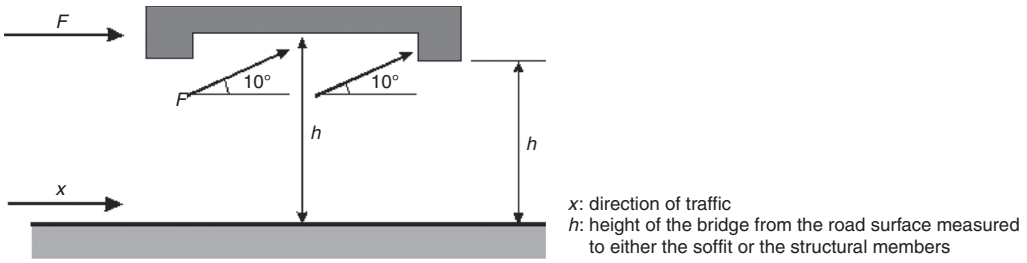


Fig. 7.11. Definition of impact force on members of the superstructure (Reproduced from EN 1991-1-7, with permission from BSI)

Table 7.3. Indicative equivalent static design forces due to impact on superstructures (Data taken from EN 1991-1-7, Table 4.2; see EN 1991-1-7 for missing values)

Category of traffic	F_{dx} (kN)
Motorways and country national and main roads	500
Country roads in rural area	
Roads in urban area	250

5.00 m. No impact needs to be considered for a vertical clearance beyond an upper limit equal to $h_0 + b$, b being defined at the national level. The recommended value is $b = 1$ m.

For $h_0 \leq h \leq h_1 = h_0 + b$ the magnitude of the impact force may be reduced linearly. Figure 7.12, deriving from Fig. 4.2 of the EN 1991-1-7, shows the law of the recommended reduction factor r_F, applicable to F_{dx} between h_0 and h_1.

In the UK National Annex to EN 1991-1-7 r_F is taken as 1 until $h = 5.7$ m and $h = 0$ for $h > 5.7$ m.

Figure 7.13 gives a representation of the impact force based on the recommended values of the Eurocode.

From a practical point of view, the Eurocode defines only an impact force in the direction of normal travel, noted F_{dx}. It was considered unnecessary to introduce more sophisticated models. Nevertheless, the Eurocode indicates that, where appropriate, forces perpendicular to the direction of normal travel, F_{dy}, should also be taken into account. In such a case, it is recommended that F_{dy} does not act simultaneously with F_{dx}. The indicative value of the impact force is given in Table 7.3, derived from Table 4.2 of EN 1991-1-7. The values given in the UK National Annex are about 60% greater than those given in Table 7.3.

cl. 4.3.2(2): EN 1991-1-7

The Eurocode recommends to take into account on the underside surfaces of bridge decks the same impact loads F_{dx} as above with an upward inclination, the recommended value of

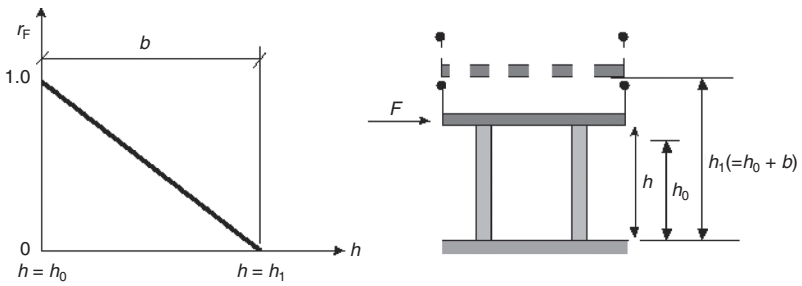


Fig. 7.12. Recommended value of factor r_F for vehicular collision forces on horizontal structural members above roadways, depending on clearance height h (Reproduced from EN 1991-1-7, with permission from BSI)

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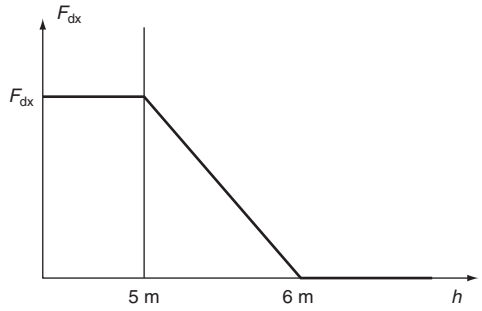


Fig. 7.13. Representation of the vehicular collision force on horizontal structural members above roadways, based on the recommended values

upward inclination being 10° – see Fig. 7.11. This rule is intended to cover the risk of lifting of a crane under a bridge and to impose a minimum robustness to the deck structure. *Note 4 to cl. 4.3.2(1): EN 1991-1-7*

Concerning the area of application of the impact force(s) on the members of the superstructure, a square area of impact is recommended, namely a square with sides 25 cm (Fig. 7.14). *cl. 4.3.2(3): EN 1991-1-7*

Of course, the impact area is located in order to produce the most unfavourable (general or local) effect.

7.4.3. Impact on supporting structures – simplified dynamic model

Annex C to EN 1991-1-7 provides some guidance for an approximate dynamic design of structures subject to accidental impact, for example by road vehicles.

The static forces given in Tables 7.2 and 7.3 above may be considered as corresponding to hard impact, but a basic dynamic analysis is possible.

The structure is assumed rigid and immovable, and the deformation of the colliding object is assumed to develop linearly during the impact phase. The maximum resulting dynamic interaction force is given by Expression (7.1):

$$F = v_r\sqrt{km} \qquad (7.1)\ (\text{EN 1991-1-7, C.2.1, C.1})$$

where

- v_r is the object velocity at impact
- k is the equivalent elastic stiffness of the object (i.e. the ratio between force F and total deformation)
- m is the mass of the colliding object.

If the force due to impact is represented by a rectangular pulse (without rise time, but this assumption is not essential, see Fig. 7.15) on the surface of the structure, the duration of

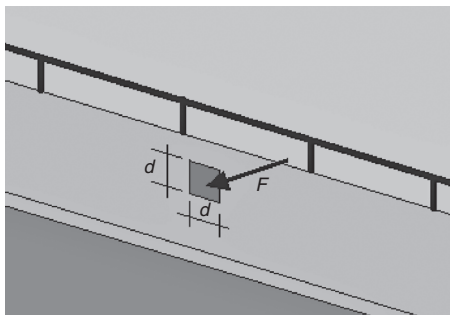


Fig. 7.14. Impact area on a bridge superstructure due to a road vehicular collision: recommended value $d = 0.25$

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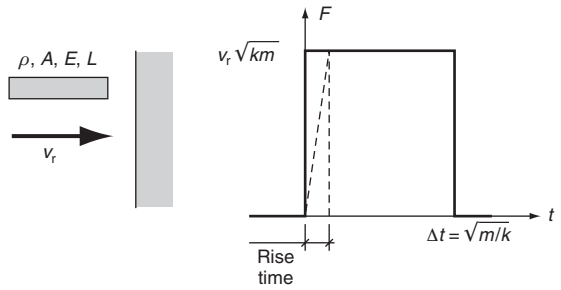


Fig. 7.15. Impact model, F = dynamic interaction force (Reproduced from EN 1991-1-7, with permission from BSI)

the pulse is given by the following formula:

$$F\Delta t = mv \;\Rightarrow\; \Delta t = \sqrt{m/k} \qquad (7.2)\ (\text{EN 1991-1-7, C.2.1, C.2})$$

If the colliding object of mass m (density ρ) is modelled as an equivalent impacting object of uniform cross-section A (see Fig. 7.15), length L and modulus of elasticity E then:

$$k = EA/L \quad \text{and} \quad m = \rho AL$$

cl. 2.1(3): EN 1991-1-7

EN 1991-1-7 mentions that Expression (7.1) gives the maximum dynamic force value on the outer surface of the structure. However, it draws the designer's attention to the fact that, within the structure, this force may give rise to dynamic effects which may be taken into account via a dynamic amplification factor (i.e. the ratio between dynamic and static response). The value of this dynamic amplification factor ranges from below 1.0 up to 1.8 depending on the dynamic characteristics of the structure and the object. In the absence of an accurate dynamic analysis, conservative values may be adopted, but the 'hard impact' model is, by itself, rather pessimistic.

C.2.2: EN 1991-1-7

In the case of soft impact (the structure is assumed elastic and the colliding object perfectly rigid), the expressions given above apply and may be used, k being the stiffness of the structure.

In the limit case of rigid-plastic response of the structure, the following condition needs to be checked:

$$\tfrac{1}{2}mv_r^2 \le F_0 y_0 \qquad (\text{EN 1991-1-7, C.2.2, C.5})$$

where

- F_0 is the plastic strength of the structure, i.e. the limit value of the static force F
- y_0 is its deformation capacity, i.e. the displacement of the point of impact that the structure can undergo.

For the application to the impact from an aberrant road vehicle on a structural member, the Eurocode suggests using the following expression of the velocity of impact v_r in Expression (7.1):

$$v_r = \sqrt{v_0^2 - 2as} = v_0\sqrt{1 - d/d_b}\ (\text{for } d < d_b) \qquad (7.3)\ (\text{EN 1991-1-7, C.3, C.6})$$

where (see Fig. 7.16):

- v_0 is the velocity of the lorry leaving the trafficked lane
- a is the average deceleration of the lorry after leaving the trafficked lane
- s is the distance from the point where the lorry leaves the trafficked lane to the structural member
- d is the distance from the centre of the trafficked lane to the structural member
- d_b is the braking distance $= d_b = (v_0^2/2a)\sin\varphi$, where φ is the angle between the trafficked lane and the course of the impacting vehicle.

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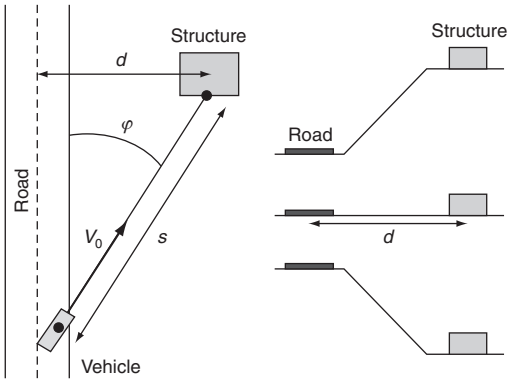


Fig. 7.16. Situation sketch for impact by vehicles (top view and cross-sections for upward slope, flat terrain and downward slope) (Reproduced from EN 1991-1-7, with permission from BSI)

C.3(3), Expression C.7: EN 1991-1-7

The following expression, established from some probabilistic considerations, is given as an approximate design value for the dynamic interaction force due to impact:

$$F_d = F_0\sqrt{1 - d/d_b}$$

Table C.2: EN 1991-1-7

where F_0 is the impact force, d and d_b are as before.

The reader's attention is drawn to the fact that EN 1991-1-7 suggests a design value of the impact force equal to 2400 kN for bridge piers on motorways, which is somewhat different from the indicative value mentioned in Table 7.2 of this Designers' Guide. Of course, this design value is based on rather pessimistic assumptions, but it is clear, as previously explained, that the impact forces may be different from the indicative values, which means that it is the responsibility of the client or the relevant authority to fix the 'accepted' risk level.

7.5. Accidental actions caused by derailed rail traffic under or adjacent to structures

cl. 4.5: EN 1991-1-7

7.5.1. Structures spanning across or alongside operational railway lines

cl. 4.5.1: EN 1991-1-7

When designing structures that are built over tracks, the reasonably foreseeable development of railway infrastructure, particularly the track layout and the structural clearances, should be taken into consideration.

EN 1991-1-7 gives rules to calculate the design values for actions due to impact on supporting members (e.g. piers and columns) caused by derailed trains passing under or adjacent to structures. In general, impact on the superstructure (deck structure) from derailed rail traffic under or on the approach to a structure need not be taken into account. More extensive guidance on accidental actions related to rail traffic may be found in UIC-Code 777-2.[4]

Of course, the strategy for design must also include other appropriate measures (both preventive and protective) to reduce, as far as is reasonably practicable, the effects of an accidental impact from a derailed train against supports of structures located above or adjacent to the tracks.

Recommended preventive and protective measures are as follows:

- Increasing the lateral distance between support and centre-line of the track.
- Increasing the longitudinal distance between the structure and any switch or crossing on the approach to the structure.
- Provision of a continuous superstructure, so that the superstructure remains standing if one of the columns is removed.

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- Avoidance of supports located on a line that is crossed by a line extended in the direction of the turnout of a switch. If this is not reasonably practicable, the provision of dwarf walls should be considered, taking into account their effect on other adjacent infrastructure.
- Provision of continuous walls or wall-type supports instead of columns.
- Provision of deflecting devices or absorbing devices.

cl. 4.5.1.2: EN 1991-1-7

7.5.2. Classification of structures

The Eurocode distinguishes two classes of permanent structures that may be subject to impact from derailed railway traffic (rules concerning temporary structures may be given at the national level). These classes are defined in Table 7.4, which derives from Table 4.3 of the Eurocode.

For class A structures, where the maximum speed of rail traffic at the location is less than or equal to 120 km/h, the Eurocode gives indicative design values for the static equivalent forces due to impact on supporting structural members.

As for impact on bridge piers from road traffic, only 'indicative' values are given, which means that other values may have to be considered for other circumstances.

Table 7.5, deriving from Table 4.4 of the Eurocode, gives the indicative values.

These values may be reduced where supporting structural members are protected, for example by solid plinths or platforms with a minimum height of 38 cm above the top of the rail. The values given in Table 7.5 are rather low; in fact, they correspond to impact due to derailment at low speed. They do not cover a direct impact by a high-speed train derailing at full velocity. Where the maximum permitted speed of rail traffic at the location is greater than 120 km/h, the Eurocode recommends providing preventive and/or protective measures and determining equivalent static forces assuming that consequence class CC3 applies.

cl. 4.5.1.4(3): EN 1991-1-7

In any case, the forces F_{dx} and F_{dy} are taken into account separately and applied at the specified height above track level. The recommended value of this height is 1.8 m.

Table 7.4. Classes of structure subject to impact from derailed railway traffic (Data taken from EN 1991-1-7, Table 4.3)

Class A	Structures that span across or near to the operational railway that are either permanently occupied or serve as a temporary gathering place for people (such as theatres and cinemas) or consist of more than one storey (such as car parks and warehouses)
Class B	Massive structures that span across the operational railway such as bridges carrying vehicular traffic or single-storey buildings that are not permanently occupied or do not serve as a temporary gathering place for people

Table 7.5. Indicative horizontal static equivalent design forces due to impact for class A structures over or alongside railways (Data taken from EN 1991-1-7, Table 4.4)

Distance '*d*' from structural elements to the centre-line of the nearest track (m)	Force F_{dx}[a] (kN)	Force F_{dy}[a] (kN)
Structural elements: $d < 3$ m	To be specified for the individual project Further information is set out in Annex B (of EN 1991-1-7)	To be specified for the individual project Further information is set out in Annex B (of EN 1991-1-7)
For continuous walls and wall type structures: 3 m $\leq d \leq$ 5 m	4000	1500
$d > 5$ m	0	0

[a] x = track direction; y = perpendicular to track direction.

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cl. 4.5.1.5: EN 1991-1-7

For class B structures, particular requirements need to be specified at the national level or for the individual project. These particular requirements may be based on a risk assessment.

Supporting structural members should generally not be located in the area immediately beyond the track ends. However, where supporting structural members are required to be located near to track ends, an end impact wall should be provided in the area immediately beyond the track ends in addition to any buffer stop.

cl. 4.6: EN 1991-1-7

7.6. Accidental actions caused by ship traffic

7.6.1. General

EN 1991-1-7 defines methods for the assessment of accidental actions due to collisions on bridge piers (Fig. 7.17) and decks from ships on inland waterways or from seagoing vessels.

Naturally, the magnitude of these actions depends on the flood conditions, the type and draught of vessels and their impact behaviour, and the type of the structures and their energy dissipation characteristics.

In both cases, the simplified approach to take into account the effects of ship impact on inland waterways and from sea vessels is the same: impact by ships against solid structures is normally considered as *hard impact*, with the kinetic energy being dissipated by elastic or plastic deformation of the ship itself.

cl. 4.6.1: EN 1991-1-7

The effects are calculated from equivalent static forces:

- a frontal force F_{dx} on piers
- a lateral force with a component F_{dy} acting perpendicular to the frontal impact force and a friction component F_R parallel to F_{dx}, on piers
- frontal force F on decks.

The frontal and lateral forces on bridge piers are assumed to be mutually exclusive.

EN 1991-1-7 is not applicable to structures designed to accept ship impact in normal operating conditions (e.g. quay walls and breasting dolphins).

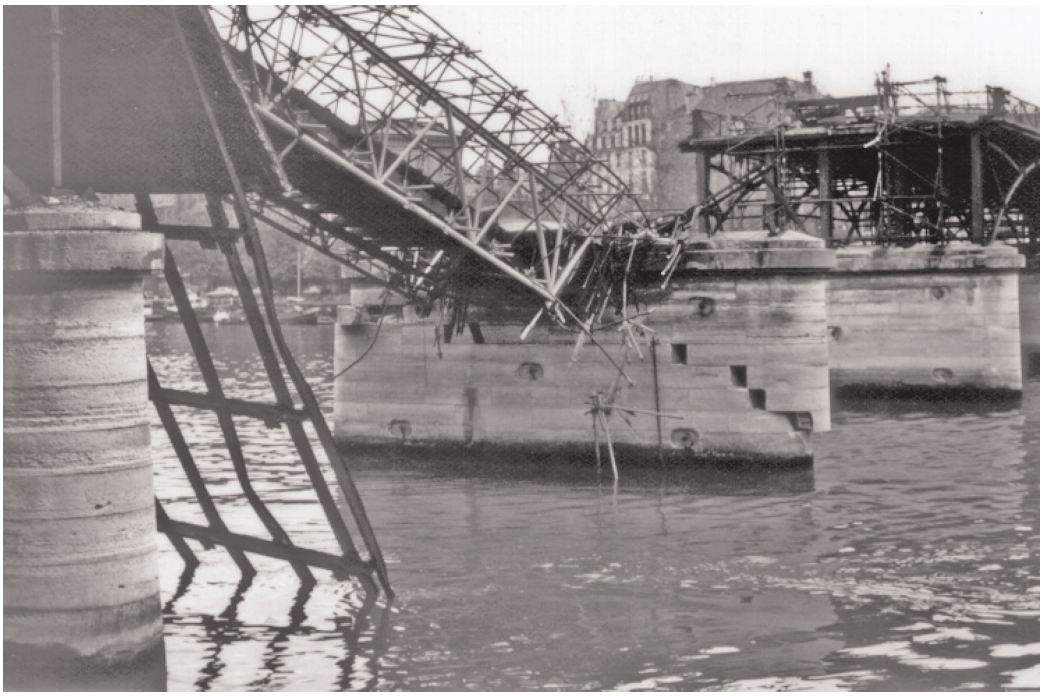


Fig. 7.17. Ship collision on the former Ponts des Arts – Paris, River Seine

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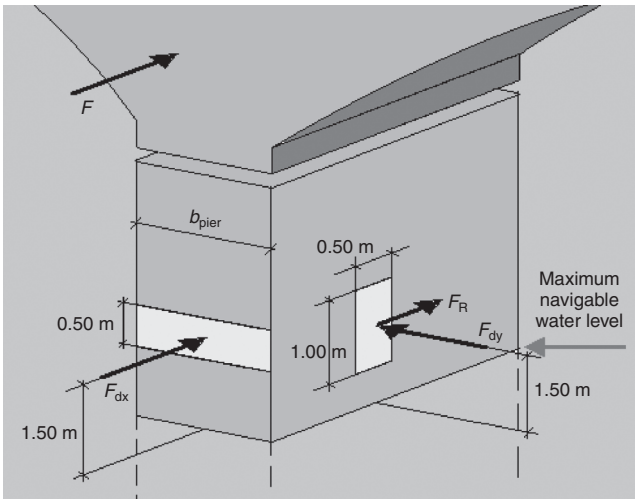


Fig. 7.18. Definition of static forces and impact conditions due to ship collision on bridge piers on inland waterways

An advanced approach is proposed in Annex C of EN 1991-1-7: dynamic design for impact.

Advanced design of structures to sustain actions due to impact may include explicitly one or several of the following aspects:

- dynamic effects
- non-linear material behaviour.

The results of calculations from refined methods may be different from the values defined using the simplified approach. For this reason, the proposed values are not recommended values, and not even minimum recommended values. This means that the responsibility of the reliability level for a bridge is selected by the designer, with the agreement of the client or of the relevant authority. A probabilistic modelling of a ship collision is described in Annex B to EN 1991-1-7, but such an approach may be adopted only by specialists with the agreement of the client.

cl. 4.6.2: EN 1991-1-7

7.6.2. Impact from river and canal traffic

The types of ships on inland waterways are selected depending on the classification of the individual waterways. This classification is established by the relevant authority according to the CEMT[5] classification system.

The various forces in case of adoption of the simplified approach are represented in Fig. 7.18. The impact force due to friction F_R acting simultaneously with the lateral impact force F_{dy} may be calculated from the following formula:

$$F_R = \mu F_{dy} \qquad (7.4)\ (\text{EN } 1991\text{-}1\text{-}7,\ (4.1))$$

where μ is the friction coefficient; its recommended value is 0.4.

The recommended impact area $b \times h$ has the following dimensions: $b = b_{pier}$ (b_{pier} being the width of the bridge pier) and $h = 0.5$ m for frontal impact; $h = 1.0$ m and $b = 0.5$ m for lateral impact.

The CEMT classification, given in Annex C to EN 1991-1-7, is reproduced in the following Table 7.6.

This table is a simplification of the table given in the official document agreed by the Council of the European Union. In particular, and for information, the following Table 7.7 gives the minimum height under bridges for the various classes.

For example, the River Seine in France is classified Vb.

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Table 7.6. Indicative values for the dynamic forces due to ship impact on inland waterways (Data taken from EN 1991-1-7, Table C.3; see EN 1991-1-7 for missing values)

CEMT class	Reference type of ship	Length l (m)	Mass m (t)[a]	Force F_{dx}[b] (kN)	Force F_{dy}[b] (kN)
I	Barge				
II	Campine-Barge	50–60	400–650	3000	1500
III	'Gustav König'				
IV	Class 'Europe'	80–90	1000–1500	5000	2500
Va	Big ship				
Vb	Tow + 2 barges	110–180	3000–6000	10 000	4000
VIa	Tow + 2 barges	110–180			
VIb	Tow + 4 barges	110–190	6000–12 000	14 000	5000
VIc	Tow + 6 barges	190–280			
VII	Tow + 9 barges	300	14 000–27 000	20 000	10 000

[a] The mass m in tons (1 t = 1000 kg) includes the total mass of the vessel, including the ship structure, the cargo and the fuel. It is often referred to as the displacement tonnage.
[b] The forces F_{dx} and F_{dy} include the effect of hydrodynamic mass and are based on background calculations, using expected conditions for every waterway class.

Where relevant, the deck of a bridge should also be designed to sustain an equivalent static force due to impact from a ship acting in a transverse direction to the longitudinal (span) axis of the bridge. Such a scenario may occur when ships can move outside the defined sailing zone, with a bridge deck rather low over the waterway level. Of course, a value for the equivalent static force cannot be defined for all cases because it depends on many mechanical and geometrical parameters. Nevertheless, the Eurocode gives an indicative value equal to 1 MN if the designer has no accurate idea.

The Eurocode states that in the absence of a dynamic analysis, the impact forces given in Table 7.6, which may be adjusted depending upon the consequences of failure of the ship impact, should be multiplied by an appropriate dynamic amplification factor. Indeed, these values include the dynamic effects in the colliding object, but not in the structure. Indicative values of the dynamic amplification factor are proposed: 1.3 for frontal impact and 1.7 for lateral impact. However, the values given in Table 7.6 correspond more or less to 'hard impact' and are probably pessimistic. Therefore, the recommended dynamic amplification factors look rather conservative and should not be used unless there is evidence to the contrary.

In harbour areas the forces given in Table 7.6 may be reduced by a factor of 0.5.

cl. 4.6.3: EN 1991-1-7

7.6.3. Impact from seagoing vessels

In the case of maritime waterways, the dimensions and gross weight of ships are much larger than in the case of inland waterways. In general, it will not be possible to design economically

Table 7.7. Minimum height under bridges

CEMT class	Reference type of ship	Minimum height under bridges (m)
I	Barge	4.00
II	Campine-Barge	4.00–5.00
III	'Gustav König'	4.00–5.00
IV	Class 'Europe'	5.25 or 7.00
Va	Big ship	5.25 or 7.00 or 9.10
Vb	Tow + 2 barges	
VIa	Tow + 2 barges	7.00 or 9.10
VIb	Tow + 4 barges	7.00 or 9.10
VIc	Tow + 6 barges	9.10
VII	Tow + 9 barges	9.10

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Table 7.8. Indicative values for the dynamic interaction forces due to ship impact for sea waterways (Data taken from EN 1991-1-7, Table C.4; see EN 1991-1-7 for missing values)

Class of ship	Length l (m)	Mass m[a] (t)	Force F_{dx}[b,c] (kN)	Force F_{dy}[b,c] (kN)
Small	50	3000	30 000	15 000
Medium				
Large	200	40 000	240 000	120 000
Very large				

[a] The mass m in tons (1 t = 1000 kg) includes the total mass of the vessel, including the ship structure, the cargo and the fuel. It is often referred to as the displacement tonnage. It does not include the added hydraulic mass.
[b] The forces given correspond to a velocity of about 5.0 m/s. They include the effects of added hydraulic mass.
[c] Where relevant, the effect of bulbs should be accounted for.

acceptable structures to resist the forces that can develop in the case of ship collision. Table 7.8 gives only an estimate of the magnitude of collision forces on rigid obstacles, but, in practice, protective measures should be taken.

For adoption of this simplified approach, the various forces are represented in Fig. 7.19. The impact force due to friction F_R acting simultaneously with the lateral impact force F_{dy} may be calculated from formula (7.4):

$$F_R = \mu F_{dy} \quad \text{(EN 1991-1-7, (4.2))}$$

where μ is the friction coefficient; its recommended value is 0.4, as for ship impact on inland waterways.

EN 1991-1-7 recommends, in the absence of a dynamic analysis for the impacted structure, to multiply the indicative dynamic values given in Table 7.8 by an appropriate dynamic amplification factor. Indicative values of the dynamic amplification factor are 1.3 for frontal impact and 1.7 for lateral impact, as for ships on inland waterways; in harbour areas the forces may be reduced by a factor of 0.5. However, as previously stated, it would not be reasonable to design bridge piers to resist large effects.

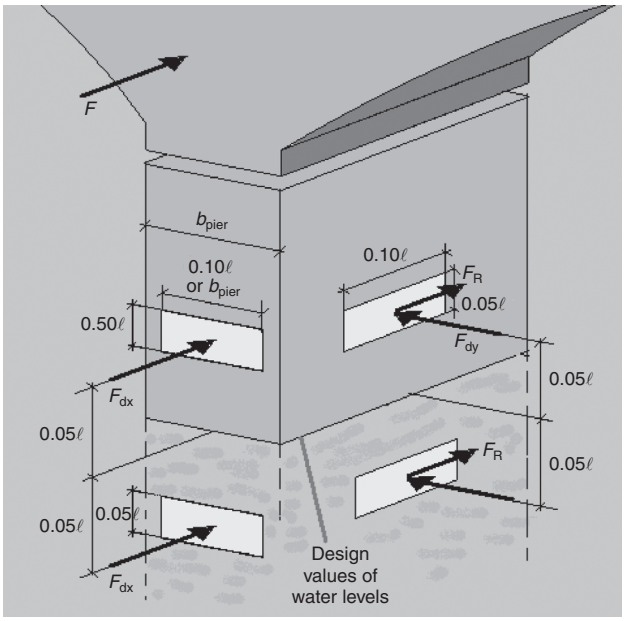


Fig. 7.19. Definition of static forces and impact conditions due to ship collision on bridge piers on sea waterways

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For side and stern impact, the impact forces are far lower than for frontal impact forces and EN 1991-1-7 suggests multiplying the forces given in Table 7.8 by a factor of 0.3, mainly because of reduced velocities. Side impact may govern the design in narrow waters where head-on impact is not feasible.

The point and area of impact depend upon the geometry of the structure and the size and geometry (e.g. with or without bulb) of the vessel, the vessel draught and trim, and tidal variations. The recommended values of the vertical range of the point of impact are $\pm 0.05l$ (l being ship length). The impact area is rectangular: its height is $0.05l$ and its width is equal to $0.1l$ or b_{pier}, whichever is the smaller.

Bow, stern and broad-side impact should be considered where relevant. Bow impact should be considered for the main sailing direction with a maximum deviation of 30°. *cl. 4.6.3(2): EN 1991-1-7*

The designer should examine the possibility that the bridge deck may be hit by the upper part of a ship. In general, the force on the superstructure of the bridge will be limited by the yield strength of the ship's superstructure. The Eurocode indicates that a range of 5–10% of the bow impact force may be considered as a guideline. In cases where only the mast is likely to impact on the superstructure, an indicative design load is 1 MN.

Of course, where the design values of actions due to ship impact are determined by advanced methods, the effects of hydrodynamic added mass should be taken into account.

Guidance is given in Annex B to EN 1991-1-7 for a risk analysis based on a probabilistic approach.

7.6.4. Advanced ship impact analysis for inland waterways

C.4.3: EN 1991-1-7

Informative Annex C to EN 1991-1-7 gives guidance on dynamic design for impact. The dynamic impact force F_{d} may be calculated from Expressions (7.5) to (7.7).

For elastic deformations (when $E_{\text{def}} \leq 0.21$ MNm), the dynamic design impact force may be calculated from Expression (7.5):

$$F_{\text{dyn,el}} = 10.95\sqrt{E_{\text{def}}}\ (\text{MN}) \qquad (7.5)\ (\text{EN 1991-1-7, C.4.3, C.8})$$

For plastic deformations (when $E_{\text{def}} > 0.21$ MNm), the dynamic design impact force may be calculated from Expression (7.6):

$$F_{\text{dyn,pl}} = 5.0\sqrt{1 + 0.128E_{\text{def}}}\ (\text{MN}) \qquad (7.6)\ (\text{EN 1991-1-7, C.4.3, C.9})$$

The deformation energy E_{def} (MNm) is equal to the available total kinetic energy E_{a} for the case of frontal impact, while in the case of lateral impact with angle $\alpha < 45°$, a sliding impact may be assumed and the deformation energy taken equal to:

$$E_{\text{def}} = E_{\text{a}}(1 - \cos\alpha) \qquad (7.7)\ (\text{EN 1991-1-7, C.4.3, C.10})$$

The kinetic energy is calculated with the average mass value for the relevant ship class, a design velocity v_{rd} equal to 3 m/s increased by the water velocity, and, where relevant, a hydrodynamic mass equal to 10% of the mass of displaced water for bow and 40% for side impact (all these values are recommended values).

If a dynamic structural analysis is performed, the impact forces may be modelled as a half-sine-wave pulse for $F_{\text{dyn}} < 5$ MN (elastic impact) and a trapezoidal pulse for $F_{\text{dyn}} > 5$ MN (plastic impact); load durations and other details are presented in Fig. 7.20.

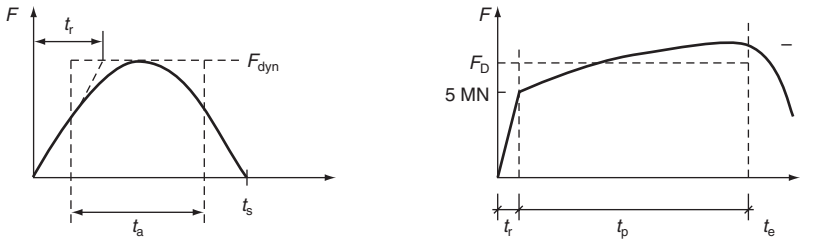
When a design value for the impact force is given, for example taken from Table 7.6, and the load duration has to be calculated, the mass m^* may be determined as follows:

if $F_{\text{dyn}} > 5$ MN: by setting E_{def}, Expression (7.6), equal to the kinetic energy $E_{\text{a}} = 0.5m^*v_{\text{n}}^2$

if $F_{\text{dyn}} \leq 5$ MN: directly by $m^* = (F_{\text{dyn}}/v_{\text{n}})^2 \times (1/c)$ (MN s^2/m)

When not specified for the individual project, a design velocity v_{rd} equal to 3 m/s increased by the water velocity is recommended; in harbours the velocity may be assumed as 1.5 m/s. The angle α may be taken as 20°.

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(a) Elastic impact ($F_{dyn} \leq 5$ MN) (b) Plastic impact ($F_{dyn} > 5$ MN)

Key:
t_r: elastic elapsing time (s)
t_p: plastic impact time (s)
t_e: elastic response time (s)
t_a: equivalent impact time (s)
t_s: total impact time (s), $t_s = t_r + t_p + t_e$
c: elastic stiffness of the ship (=60 MN/m)
F_0: elastic-plastic limit force = 5 MN
x_e: elastic deformation (≈0.1 m)
v_n: (a) the sailing speed v_r, for frontal impact
(b) velocity of the colliding ship normal to the impact point $v_n = v_r \sin \alpha$ for lateral impact

The mass m^* to be taken into account is:
(a) for frontal impact: the total mass of the colliding ship/barge
(b) for lateral impact: $m^* = (m_1 + m_{hydr})/3$, with m_1 the mass of the directly colliding ship or barge and m_{hyd} the hydraulic added mass.

Fig. 7.20. Load–time function for ship collision, respectively for elastic and plastic ship response (Reproduced from EN 1991-1-7, with permission from BSI)

C.4.4: EN 1991-1-7

7.6.5. Advanced ship impact analysis for sea waterways

Informative Annex C to EN 1991-1-7 gives guidance on dynamic design for impact. The dynamic impact force F_d in the case of ship impact in sea waterways may be derived from Expressions (7.8) to (7.10). In harbours the velocity may be assumed as 1.5 m/s, and 5 m/s at full sea.

The dynamic design impact force for sea-going merchant vessels between 500 dead weight tons (DWT) and 300 000 DWT may be determined from Expression (7.8):

$$F_{bow} = \begin{cases} F_0 \overline{L}[\overline{E}_{imp} + (5.0 - \overline{L})\overline{L}^{1.6}]^{0.5} & \text{for } \overline{E}_{imp} \geq \overline{L}^{2.6} \\ 2.24 F_0 [\overline{E}_{imp}\overline{L}]^{0.5} & \text{for } \overline{E}_{imp} < \overline{L}^{2.6} \end{cases}$$

(7.8) (EN 1991-1-7, C.4.4, C.11)

where

$$\overline{L} = L_{pp}/275\,\text{m}$$

$$\overline{E}_{imp} = E_{imp}/1425\,\text{MNm}$$

$$E_{imp} = \tfrac{1}{2} m_x v_0^2$$

F_{bow} is the maximum bow collision force (MN)
F_0 is the reference collision force = 210 MN
E_{imp} is the energy to be absorbed by plastic deformations
L_{pp} is the length of vessel (m)
m_x is the mass plus added mass with respect to longitudinal motion (10^6 kg)
v_0 is the initial speed of vessel, $v_0 = 5$ m/s (in harbours: 2.5 m/s).

From the energy balance the maximum indentation s_{max} is determined using:

$$s_{max} = \frac{\pi E_{imp}}{2 P_{bow}} \qquad (7.9)\ (\text{EN 1991-1-7, C.4.4, C.12})$$

The associated impact duration, T_0, is represented by Expression (7.10):

$$T_0 \approx 1.67 \frac{s_{max}}{V_0} \qquad (7.10)\ (\text{EN 1991-1-7, C.4.4, C.13})$$

When not specified by the project, a design velocity v_{rd} equal to 5 m/s increased by the water velocity is recommended; in harbours the velocity may be assumed as 2.5 m/s.

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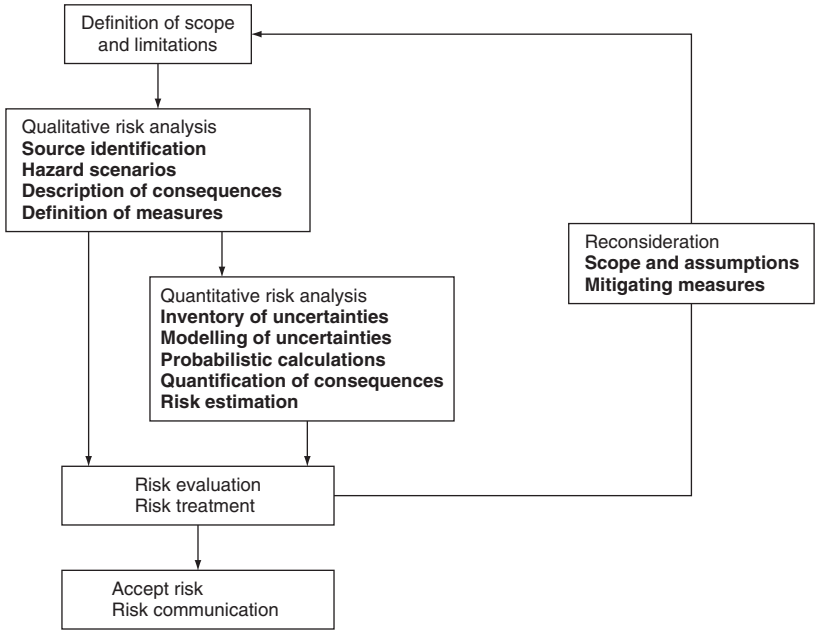


Fig. 7.21. Overview of risk analysis (Reproduced from EN 1991-1-7, with permission from BSI)

Annex B: EN 1991-1-7

7.7. Risk assessment

Information on risk assessment is given in informative Annex B to EN 1991-1-7. A general overview is presented in Fig. 7.21.

Moreover, this Annex B gives additional definitions to those introduced in Clause 1.5 of the Eurocode. These definitions are listed in the following Table 7.9.

Table 7.9. Definitions relating to risk analysis

Term	Definition	Reference in EN 1991-1-7
Consequence	A possible result of an (in risk analysis usually unwanted) event. Consequences may verbally or numerically be expressed in terms of loss of life, injury, economic loss, environmental damage, disruption to users and the public, etc. Both immediate consequences and those that arise after a certain time has elapsed are to be included.	B.2.1
Hazard scenario	A critical situation at a particular time consisting of a leading hazard together with one or more accompanying conditions which lead to an unwanted event (e.g. complete collapse of the structure).	B.2.2
Risk	A measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.	1.5.13
Risk acceptance criteria	Acceptable limits to probabilities of certain consequences of an undesired event and are expressed in terms of annual frequencies. These criteria are normally determined by the authorities to reflect the level of risk considered to be acceptable by people and society.	B.2.4
Risk analysis	A systematic approach for describing and/or calculating risk. Risk analysis involves the identification of undesired events, and the causes, likelihoods and consequences of these events (see Figure B.1).	B.2.5
Risk evaluation	A comparison of the results of a risk analysis with the acceptance criteria for risk and other decision criteria.	B.2.6
Risk management	Systematic measures undertaken by an organization in order to attain and maintain a level of safety that complies with defined objectives.	B.2.7
Undesired event	An event or condition that can cause human injury or environmental or material damage.	B.2.8

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The methods of risk analysis are described, in Annex B, as a 'short course'. For more information, reference should be made to Annex B of EN 1991-1-7 and specialized documentation. See also the TTL Designers' Guide to EN 1991.[6]

Concerning bridge design, a few applications are described in very general terms:

- impact from road vehicles
- impact from ships
- impact from rail traffic.

For impact from rail traffic, the methodology is based on recommendations and guidance given for Class A and Class B structures in UIC Code 777-2).[4] UIC Code 777-2 includes specific recommendations and guidance on the following:

- carrying out a risk assessment for Class B structures
- measures (including construction details) to be considered for Class A structures, including situations where the maximum line speed at the site is less than 50 km/h
- measures to be considered for Class A structures where the distance from the nearest structural support and the centre-line of the nearest track is 3 m or less.

Guidance is given in the EN 1991-1-7 for Class B structures.

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CHAPTER 8

Combinations of actions for road bridges, footbridges and railway bridges

8.1. General

The material in this chapter is covered in EN 1990 Annex A2.[1]

Chapter 8 is concerned with combinations of actions for the design of the most common road bridges, footbridges and railway bridges, for serviceability and ultimate limit state verifications (except fatigue verifications) with the recommended design values of permanent, variable and accidental actions and ψ factors to be used in the design of these bridges. It is also concerned with combinations of actions during execution.

The seismic combinations of actions are outside the scope of this chapter.

Some types of bridge are not, or not fully, covered by EN 1991-2 *Traffic loads on bridges* (e.g. bridges under an airport runway, mechanically movable bridges, roofed bridges, bridges carrying water). Nevertheless, the principles for establishing the combinations of actions explained in this chapter may be adopted.

A2.1.1: EN 1990: 2002/A1

For bridges carrying both road and rail traffic and for other civil engineering structures carrying traffic loads (e.g. backfill behind a retaining wall), specific rules or requirements need to be defined in the project specification.

The general format of combinations of actions is described in Section 6 of EN 1990. In particular, for ultimate limit states STR/GEO, the choice between Expressions 6.10 and 6.10a/b is left for national decision. Therefore, in the present Designers' Guide, the combinations of actions are detailed for both cases (see Designers' Guide to EN 1990 *Eurocode: Basis of Structural Design*[2]).

When referring to Expression 6.10 of EN 1990 for the fundamental combination of actions or to Expression 6.14b of EN 1990 for the characteristic combination of actions, one variable action is considered as the leading variable action of the combination. This means that:

- its representative value is the characteristic value
- all other variable actions which can physically act simultaneously are the accompanying actions and taken with their combination value
- unfavourable and favourable permanent actions are distinguished whether they act as, or opposite, the leading variable action and whether they have stabilizing or destabilizing effects on the member etc. under consideration.

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For persistent design situations, the leading variable action may be, according to the effect under consideration, one of the groups of loads defined in Section 4.5 of this Designers' Guide for road traffic, 5.5 for footbridge traffic and 6.12.2 for rail traffic. When one of these actions is the leading action, the effects of wind actions, of snow loads or of thermal actions are considered as accompanying in the persistent design situation load combination.

When referring to Expressions 6.10a/b for the fundamental combination of actions, a leading variable action is identified only in Expression 6.10b. In Expression 6.10a, all variable actions are taken with their combination value.[3]

Note 3 to A2.1.1(1): EN 1990: 2002/A1

Concerning the design working life, the Eurocode mentions that guidance may be given in the National Annex with regard to the use of Table 2.1 of EN 1990 (design working life). In normal circumstances, the design working life for road bridges, footbridges and railway bridges may be taken equal to 100 years. The UK National Annex for EN 1990 stipulates 120 years for bridges. This design working life may be extended to some road and railway retaining structures. In the case of timber footbridges, a design working life of 50 years may be adopted. For temporary structures, the recommended value of 10 years may be considered as a pertinent value.

It should be remembered that the design working life of the bridge does not apply systematically to replaceable structural or non-structural members or devices. Some elements are easily replaceable or repairable; the order of magnitude of their required working life is 10 years. If they are not easily replaceable or repairable, a working life of 25 years may be reasonably required. With regard to cable-stay bridges, see EN 1993-1-11.

8.2. General rules for combinations of actions

cl. 6.4.3.1(1)P

Before explaining the principles and the simplified rules given in EN 1990 to establish the various combinations of actions for the calculation of bridges, the distinction between a **combination of actions** and a **load case** is now explained in order to avoid any misunderstanding.

cl. 1.5.2.11: EN 1990

A combination of actions is a set of design values used for the verification of structural reliability for a limit state under the simultaneous influence of different actions. A load case describes compatible **load arrangements** (i.e. identification of the position, magnitude and direction of a free action), sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for an individual verification. **Several load cases may correspond to a unique combination of actions**.

cl. 1.5.2.10: EN 1990

Simplified rules are defined by EN 1990 Annex A2 in order to limit reasonably the number of calculations for designers. Of course, it is reminded that the relevant design situations shall be taken into account where a bridge is brought into use in stages (Fig. 8.1).

Note 4 to A2.1.1: EN 1990: 2002/A1

Where relevant, specific construction loads need to be taken into account simultaneously in the appropriate combination of actions; for example, effects of more or less controlled deformations due to the use of launching girders between two statically different stages.

A2.2.1(8): EN 1990: 2002/A1



Fig. 8.1. Example of bridge built by the cantilever method

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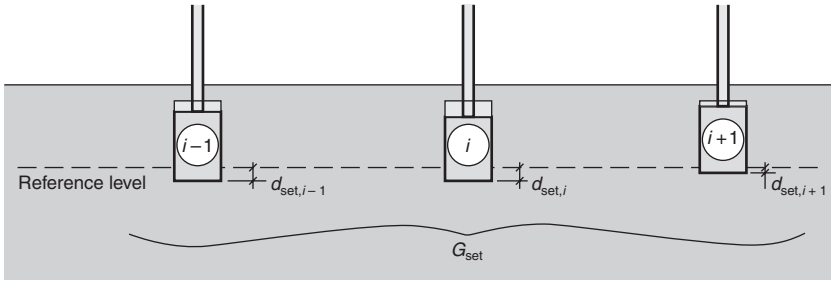


Fig. 8.2. Representation of the action of uneven settlements G_{set}

For road bridges as well as for footbridges and railway bridges, any group of loads, as defined in EN 1991-2, is to be taken into account in combinations of actions as a unique variable action.

A2.2.1(9): EN 1990: 2002/A1

In general, snow loads and wind actions need not be considered simultaneously with loads arising from construction activity Q_{ca} (i.e. loads due to working personnel) for an obvious reason: that is, people do not work on construction sites during severe snow or wind conditions (close, for example, to the characteristic values). Nevertheless, there is a possibility of the physical presence of snow loads and some construction loads (e.g. actions due to heavy equipment or cranes) during some transient design situations. See also Chapter 3 of this Designers' Guide.

A2.2.1(10): EN 1990: 2002/A1

A few other general rules are given that are common-sense rules concerning the simultaneous presence of various variable actions; these rules do not need any further explanation.

Prestressing actions are taken into account in accordance with rules given in EN 1992 to EN 1999 and in EN 1990: 2002/A1 Clause A2.3.1(8).

A2.2.1(12): EN 1990: 2002/A1

On the other hand, rules covering settlements are far more detailed. First of all, bridge decks may be very sensitive to differential settlements between the various parts of its bearing substructure. If the value of the differential settlement between two successive bridge piers is too high compared to the deck stiffness, damage may result – for example, cracks in concrete members.

A2.2.1(13) to (17): EN 1990: 2002/A1

Except in the case of swelling clay, the loading of a soil generates settlements which vary monotonically (in the same direction) with time and need to be taken into account from the time they give rise to effects in the structure (i.e. after the structure, or a part of it, becomes statically indeterminate). Physically, settlements are mainly caused by permanent actions: for bridges piers, the dominant permanent actions are actions due to self-weight and permanent actions transmitted by the bridge deck (including actions due to the interaction between the development of settlements and creep of concrete members in the case of prestressed bridge decks). In the case of abutments, settlements may be mainly caused by the weight of backfill. In general, variable actions (in particular traffic actions) have no or very little influence on the total settlement. EN 1990: 2002/A1, A2.2.1(15) defines a global permanent action due to soil subsidence, G_{set}, which is represented by a set of values corresponding to differences (compared to a reference level) of settlements between individual foundations or parts of foundations, $d_{set,i}$ (i being the number of the individual foundation or part of foundation). This action is represented in Fig. 8.2. The reference level, represented by a straight line for simplicity, is the level beyond which uneven settlements cause action effects in the deck structure.

The values of $d_{set,i}$ may be the 'final' values (i.e. long-term values) or 'intermediate values', for example during execution. In any case, effects of uneven settlements are to be taken into account if they may be significant compared to the effects from direct actions. The values of $d_{set,i}$ are the best-estimate predicted values in accordance with EN 1997 with due regard for the construction process of the structure.

Requirements concerning total settlement may have to be defined for a railway bridge (to limit the deformation of the track). In general, differential settlements may have structural

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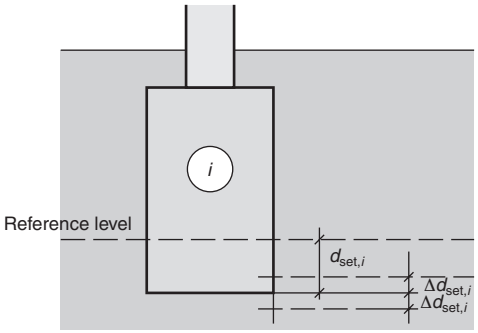


Fig. 8.3. Definition of settlement uncertainty for foundation No. *i*

consequences on a bridge deck. The design of foundations may depend on the requirements concerning differential settlements.

In any case, where the structure is very sensitive to uneven settlements, uncertainty in the assessment of these settlements should be taken into account. EN 1990: 2002/A1, A.2.2.1(17) suggests taking into account this uncertainty by a positive or negative variation of the settlement value between only two individual foundations or parts of an individual foundation. For foundation No. *i*, the settlement expresses as $d_{set,i} \pm \Delta d_{set,i}$, where $\Delta d_{set,i}$ takes account of uncertainties attached to the assessment of settlements (Fig. 8.3).

In practice, attention is drawn to the fact that prestressed concrete box girders of constant depth are very sensitive to settlements.

A2.2.2: EN 1990: 2002/A1

8.3. Combination rules for actions for road bridges

8.3.1. Simplified combination rules

As stated in Section 8.2, the following combination rules are simplified rules intended to avoid needlessly complicated calculations. This means they may be adopted in most cases, but, of course, more accurate combinations of actions may be needed in special cases. The simplifications mainly consist of limiting the number of variable actions to be taken into account, but EN 1990 authorizes national adjustments, in particular for geographical reasons or local climatic conditions. In the most common cases, the simplified rules may be summarized as follows:

- Snow loads are never combined with any group of traffic loads, except of course for roofed bridges.
- Wind and thermal actions are not taken into account simultaneously with any group of traffic loads.

cl. 4.5: EN 1991-2

- Wind actions need only be taken into account simultaneously with load group gr1a.
- No variable non-traffic action is taken into account simultaneously with load group gr1b.
- The combination of non-traffic actions with load group gr5 (special vehicles) is to be decided at national level (national annexes).

The practical application of these rules is detailed in Section 8.6.3. of this Designers' Guide.

8.3.2. Combination, frequent and quasi-permanent values of variable actions

In accordance with the principles given in EN 1990, the combination, frequent and quasi-permanent values of variable actions are obtained from the characteristic values by application of reduction factors:

- ψ_0 for combination values

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Table 8.1. Recommended values of ψ factors for road bridges (Data taken from EN 1990: 2002/A1, Table A2.1)

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads	gr1a	TS	0.75	0.75	0
(see EN 1991-2, Table 4.4)	(LM1 + pedestrian or cycle-track loads)[a]	UDL	0.40	0.40	0
		Pedestrian + cycle-track loads[b]	0.40	0.40	0
	gr1b (single axle)		0	0.75	0
	gr2 (horizontal forces)		0	0	0
	gr3 (pedestrian loads)		0	0	0
	gr4 (LM4 – (crowd loading))		0	0.75	0
	gr5 (LM3 – (special vehicles))		0	0	0
Wind forces	F_{Wk}				
	• Persistent design situations		0.6	0.2	0
	• Execution		0.8	–	0
	F_W^*		1.0	–	–
Thermal actions	T_k		0.6[c]	0.6	0.5
Snow loads	$Q_{Sn,k}$ (during execution)		0.8	–	–
Construction loads	Q_c	1.0		–	1.0

[a] The recommended values of ψ_0, ψ_1 and ψ_2 for gr1a and gr1b are given for road traffic corresponding to adjusting factors α_{Qi}, α_{qi}, α_{qr} and β_Q equal to 1. Those relating to UDL correspond to common traffic scenarios, in which a rare accumulation of lorries can occur. Other values may be envisaged for other classes of routes, or of expected traffic, related to the choice of the corresponding α factors. For example, a value of ψ_2 other than zero may be envisaged for the UDL system of LM1 only, for bridges supporting severe continuous traffic. See also EN 1998.
[b] The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a 'reduced' value. ψ_0 and ψ_1 factors are applicable to this value.
[c] The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

- ψ_1 for frequent values
- ψ_2 for quasi-permanent values.

The recommended values of these reduction factors are given in Table 8.1.

Additional comments and background information

(a) As mentioned in Section 4.3.2 of this Designers' Guide, the frequent values of road traffic loads are based on a return period of one week.

In Annex B to Chapter 4 of this Designers' Guide (Section B2.2), an 'empirical' formula is proposed to link the values of a specific effect for various values of the return period; see also the TTL Designers' Guide for Actions on Buildings.[4]

$$E_T = [1.05 + 0.116 \log_{10}(T)] E_{20\,\text{weeks}}$$

where

E_T is the effect corresponding to a return period T, expressed in years
$E_{20\,\text{weeks}}$ is the effect corresponding to a return period of 20 weeks.

Assuming that this formula remains usable for a return period of 1 week = 0.02 year, it gives:

$$E_{1\,\text{week}} = [1.05 + 0.116 \log_{10}(0.02)] E_{20\,\text{weeks}} = 0.85 E_{20\,\text{weeks}}$$

However

$$E_{1000\,\text{years}} = 1.40 E_{20\,\text{weeks}}$$

Thus

$$E_{1\,\text{week}} = 0.85 \times \frac{E_{1000\,\text{year}}}{1.40} = 0.61 E_{1000\,\text{years}}$$

Considering this calculation, it was agreed by the experts not to reduce uniformly the two components of the main loading system, TS and UDL. In order to ensure a good design

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of members to resist local effects, it was decided to apply a factor equal to 0.75 to concentrated loads and a factor equal to 0.4 to uniformly distributed loads.

As concerns the combination values, it was considered that it would not be useful to define other ψ values, between 1 and 0.75, for concentrated loads (axle loads) and between 1 and 0.4 for uniformly distributed loads.

(b) As explained in Section 2.3.5 of Chapter 2 of this Designers' Guide, it may be decided to ignore the concept corresponding to wind forces F_W^* and F_W^{**}. Therefore, the line giving the combination value (1.00) for F_W^* may be ignored.

Table 4.4(b): EN 1991-2

(c) The recommended frequent value of gr3 (pedestrian loads) is 0. However, the frequent model of gr3 is mentioned in ***Table 4.4(b) of EN 1991-2***, and in Table 4.8 of Chapter 4 of this Designers' Guide. A frequent value equal to 0 is not reasonable for bridges located in towns, with wide footways. We consider that $\psi_1 = 0.4$ for load group gr3 is a reasonable value. On the other hand, the frequent value of the crowd loading (gr4) should be taken equal to 0. In special circumstances, it may be useful to define a frequent value for special vehicles (gr5) if it is envisaged that a certain type of such vehicles will cross the bridge regularly. In that case, ψ_1 may be taken equal to 1.

(d) Concerning snow loads, the ψ_0 value is only defined for execution situations: as previously explained, snow loads are not combined with any other traffic or non-traffic action during persistent design situations. For traffic classes other than the basic traffic class (corresponding to adjusting factors equal to 1), it is recommended to adopt the same ψ factors.

Editorial note

At the ENV stage an additional set of values for traffic loads was introduced: the 'infrequent' values. These values were calibrated to correspond to a return period of 1 year and were introduced only for the design of concrete road bridges; no infrequent values were defined for pedestrian and rail traffic actions. The use of the infrequent values is no longer defined in EN 1992-2 (Design of concrete bridges), but EN 1990 Annex A2 leaves it to be decided at the national level (National Annex) and only for certain serviceability limit states of concrete road bridges.

A2.2.2(1): EN 1990: 2002/A1

In such a case, the expression of this combination of actions is:

A2.1a: EN 1990: 2002/A1

$$E_d = E\{G_{k,j}; P; \psi_{1,infq} Q_{k,1}; \psi_{1,i} Q_{k,i}\} \; j \geq 1; i > 1$$

in which the combination of actions in brackets { } may be expressed as:

A2.1b: EN 1990: 2002/A1

$$\sum_{j \geq 1} G_{k,j} \,''+''\, P \,''+''\, \psi_{1,infq} Q_{k,1} \,''+''\, \sum_{i>1} \psi_{1,i} Q_{k,i}$$

Note 2 to EN 1990: 2002/A1, Table A2.1 (Table 8.1 of this chapter) gives recommended values of $\psi_{1,infq}$ when the National Annex allows the use of infrequent values:

- 0.80 for gr1a (LM1), gr1b (LM2), gr3 (pedestrian loads), gr4 (LM4, crowd loading) and T (thermal actions)
- 0.60 for F_{Wk} in persistent design situations
- 1.00 in other cases (i.e. the characteristic value is used as the infrequent value).

A2.2.3: EN 1990: 2002/A1

8.4. Combination rules for footbridges

8.4.1. Simplified combination rules

A2.2.2: EN 1990: 2002/A1

For footbridges, only two groups of loads (see Chapter 5 of this Designers' Guide) plus a concentrated load Q_{fwk} are specified. The simplified rules concerning footbridges are very similar to the rules defined for road bridges. In particular:

- The concentrated load Q_{fwk} is not to be combined with any other non-traffic variable action.

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Table 8.2. Recommended values of ψ factors for footbridges (Data taken from EN 1990: 2002/A1, Table A2.2)

Action	Symbol	ψ_0	ψ_1	ψ_2
	gr1	0.40	0.40	0
Traffic loads	Q_{fwk}	0	0	0
	gr2	0	0	0
Wind forces	F_{Wk}	0.3	0.2	0
Thermal actions	T_k	0.6[a]	0.6	0.5
Snow loads	$Q_{Sn,k}$ (during execution)	0.8	–	0
Construction loads	Q_c	1.0	–	1.0

[a] The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

- Snow loads are not combined with any group of traffic loads, except for special geographical areas and certain types of footbridges (in particular roofed footbridges).
- Wind and thermal actions are not taken into account simultaneously with any group of traffic loads.

In the case of roofed footbridges, the Eurocode allows a definition of the appropriate combinations of actions in the National Annex. The combinations of actions are normally similar to those for buildings, the imposed loads being replaced by the relevant group of loads and the ψ factors for traffic actions being in accordance with Table 8.2.

A2.2.3(4): EN 1990: 2002/A1

8.4.2. Combination, frequent and quasi-permanent values of variable actions

The combination, frequent and quasi-permanent values of variable actions for pedestrian bridges are obtained from the characteristic values by application of reduction factors:

- ψ_0 for combination values
- ψ_1 for frequent values
- ψ_2 for quasi-permanent values.

The recommended values of these reduction factors are given in Table 8.2.

8.5. Combination rules for railway bridges

A2.2.4: EN 1990: 2002/A1

8.5.1. Simplified combination rules

Actions should be combined in accordance with the methods defined in EN 1990 using appropriate partial factors.

Generally for railways, the following applies:

- Snow loads need not be taken into account in any combination for persistent design situations nor for any transient design situation after the completion of the bridge unless otherwise specified for particular geographical areas and certain types of railway bridges (roofed bridges).
- The combinations of actions to be taken into account when rail traffic actions and wind actions act simultaneously should include:
  - vertical rail traffic actions including dynamic factor, horizontal rail traffic actions and wind forces, with each action being considered as the leading action of the combination of actions one at a time
  - vertical rail traffic actions excluding dynamic factor, lateral rail traffic actions from the 'unloaded train' defined in Section 6.7.4 of Chapter 6 of this Designers' Guide and wind forces for checking overall stability.

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- Wind action need not be combined with (see Chapter 6):
  - load groups gr13 or gr23 (maximum longitudinal effect)
  - load groups gr16, gr17, gr26, gr27 and the individual traffic action Load Model SW/2 (load groups containing SW/2)

  (See Section 6.12.2 and Table 6.5 of Chapter 6 of this Designers' Guide).
- Requirements for taking wind actions and snow loads into account with construction loads should be in accordance with the relevant international or national requirements.

cl. 6.6: EN 1991-2

- Actions due to aerodynamic effects of rail traffic and wind actions should be combined. Each action should be considered individually as a leading variable action.
- If a structural member is not directly exposed to wind, the action q_{ik} due to aerodynamic effects should be determined for train speeds enhanced by the speed of the wind.
- Where groups of loads are not used for rail traffic loading (normal case), rail traffic loading should be considered as a unique multi-directional variable action with individual components of rail traffic actions taken as the maximum unfavourable and minimum favourable values as appropriate.
- Where groups of loads are used to represent the combined load effects of rail traffic actions, the combinations of rail traffic actions given in Section 6.12.2 of this Designers' Guide should be used. A unique ψ value should be applied to one of the load groups, with ψ taken as equal to the ψ value applicable to the leading component of the group.
- Requirements for combining actions for accidental design situations and seismic design situations should be in accordance with the relevant international or national requirements (generally only one accidental action is taken into account at any one time) and excluding wind actions or snow loads. For combinations including derailment loading, rail traffic actions should be taken into account as accompanying actions in the combinations with their combination value.

(a) **Accidental action** (derailment, design situations I and II; see Section 6.11.1 of this Designers' Guide):

$$\sum_{j\geq 1} G_{k,j} \,''+''\, P \,''+''\, A_d \,''+''\, (\psi_{1,1} \text{ or } \psi_{2,1}) Q_{k1} \,''+''\, \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad \text{EN 1990, (6.11)}$$

Note: For railway bridges with more than one track, only the tracks not loaded with derailment actions can be loaded with other rail traffic loads. Specific rules or requirements need to be defined in the project specification. With the choice given in the equation above, freedom to think in hazard scenarios is given; for example:

$\psi_{1,1} = 0.8$ if one supplement track is loaded with LM71, or
$\psi_{2,1} = 0$ if only the derailment loads specified in Section 6.11.1 of this Designers' Guide is taken into account.

(b) **Seismic action**

$$\sum_{j\geq 1} G_{k,j} \,''+''\, P \,''+''\, A_{Ed} \,''+''\, \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad \text{EN 1990, (6.12)}$$

Table A2.3 footnote 4: EN 1990: 2002/A1

Note: For railway bridges, only one track need be loaded with LM71, and LM SW/2 may be neglected, see footnote a of Table 8.3 and third footnote of Table 8.9 and Table A2.5.

Recommended value: $\psi_{2,j} \geq 0.8$.

The minimum coexistent favourable vertical load with centrifugal, traction or braking individual components of rail traffic actions is 0.50LM71 (see footnote c in Table 8.3 below).

- In cases where the limit state is very sensitive to variations in magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken into account, with appropriate combinations of favourable and unfavourable actions.

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Table 8.3. Recommended values of ψ factors for railway bridges (Data taken from EN 1990: 2002/A1, Table A2.3)

Actions			ψ_0	ψ_1	ψ_2[a]
Individual components of traffic actions[c]	LM71		0.80	[b]	0
	SW/0		0.80	[b]	0
	SW/2		0	1.00	0
	Unloaded train		1.00	–	–
	HSLM		1.00	1.00	0
	Traction and braking		Individual components of traffic actions in design situations where the traffic loads are considered as a single (multi-directional) leading action and not as groups of loads should use the same values of ψ factors as those adopted for the associated vertical loads		
	Centrifugal forces				
	Interaction forces due to deformation under vertical traffic loads				
	Nosing forces		1.00	0.80	0
	Non-public footpath loads		0.80	0.50	0
	Real trains		1.00	1.00	0
	Horizontal earth pressure due to traffic load surcharge		0.80	[b]	0
	Aerodynamic effects		0.80	0.50	0
Main traffic actions (groups of loads)	gr11 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal			
	gr12 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr13 (braking/traction)	Max. longitudinal			
	gr14 (centrifugal/nosing)	Max. lateral	0.80	0.80	0
	gr15 (unloaded train)	Lateral stability with 'unloaded train'			
	gr16 (SW/2)	SW/2 with max. longitudinal			
	gr17 (SW/2)	SW/2 with max. transverse			
	gr21 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal			
	gr22 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr23 (braking/traction)	Max. longitudinal	0.80	0.70	0
	gr24 (centrifugal/nosing)	Max. lateral			
	gr26 (SW/2)	SW/2 with max. longitudinal			
	gr27 (SW2)	SW/2 with max. transverse			
	gr31 (LM71 + SW/0)	Additional load cases	0.80	0.60	0
Other operating actions	Aerodynamic effects		0.80	0.50	0
	General maintenance loading for non-public footpaths		0.80	0.50	0
Wind forces	F_{Wk}		0.75	0.50	0
Thermal actions[d]	T_k		0.60	0.60	0.50
Snow loads	$Q_{Sn,k}$ (during execution)		0.8	–	0
Construction loads	Q_c		1.0	–	1.0

[a] If deformation is being considered for persistent and transient design situations, ψ_2 should be taken equal to 1.00 for rail traffic actions. **For seismic design situations, see Table 8.9 of this Designers' Guide (EN 1990: 2002/A1, Table A2.5).**
[b] 0.8 if 1 track only is loaded; 0.7 if 2 tracks are simultaneously loaded; 0.6 if 3 or more tracks are simultaneously loaded.
[c] Minimum coexistent favourable vertical load with individual components of rail traffic actions (e.g. centrifugal, traction or braking) is 0.5LM71, etc.
[d] See EN 1991-1-5.

- For the design of structural members subject to geotechnical actions and for other geotechnical design situations, the combinations of loading and design philosophy should be in accordance with the relevant national and international requirements.

For bridges carrying both rail and road traffic, the combinations of actions to be taken into account should be decided at the national level (National Annex or requirements of the relevant authorities).

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In accordance with Chapter 2 of this Designers' Guide, the wind action denoted F_W^{**} has been ignored.

8.5.2. Combination of frequent and quasi-permanent values of variable actions

The recommended values of ψ factors for railway bridges are given in Table 8.3 established from EN 1990: 2002/A1, Table A2.3. All references to F_W^{**} have been removed (see Chapter 2 of this Designers' Guide).

8.6. Combination of actions for ultimate limit states

Fatigue verifications are defined in the material-dependent Eurocodes EN 1992 to EN 1994: the combinations of actions, associated with the relevant verification rules, are specific for each material (see Chapters 4 and 6 of this Designers' Guide).

8.6.1. Reminder of the general format of combinations of actions and verification rules for persistent and transient design situations

As for buildings, three categories of ultimate limit state are envisaged. These categories are called EQU (static equilibrium), STR (structural member resistance) and GEO (geotechnical limit states). Remember that limit states correspond to an idealization of structural phenomena to be avoided. Figure 8.4 gives an illustration of these categories of limit states for a bridge built by the cantilever method during execution.

For each limit state (EQU, STR, GEO), the design values are to be taken from one or several of the three tables which are given in the following paragraphs (i.e. Tables 8.4 to 8.6).

The general expressions of combinations of actions for ultimate limit states (ULS) and serviceability limit states (SLS) are recalled in Tables 8.4 and 8.5.

The general formats for verification are summarized in Table 8.5. Concerning equation (6.11), in general there is no variable action taken with its frequent value. Therefore, the accidental combination of actions includes only variable actions, accompanying permanent actions and the accidental action, taken with their quasi-permanent value.

It should be remembered that three approaches are defined for the verification of structural members (footings, piles, piers, side walls, wing walls, flank walls and front walls of abutments, ballast retention walls, etc.) (STR) involving geotechnical actions and the resistance of the ground (GEO), supplemented, for geotechnical actions and resistances, by EN 1997:

A2.3.1(5): EN 1990: 2002/A1

- Approach 1: Applying in separate calculations design values from ***Table A2.4(C) and Table A2.4(B) of EN 1990 Annex A2*** (reproduced as Tables 8.8 and 8.7 respectively in

Table A2.4(C) and Table A2.4(B): EN 1990 Annex 2

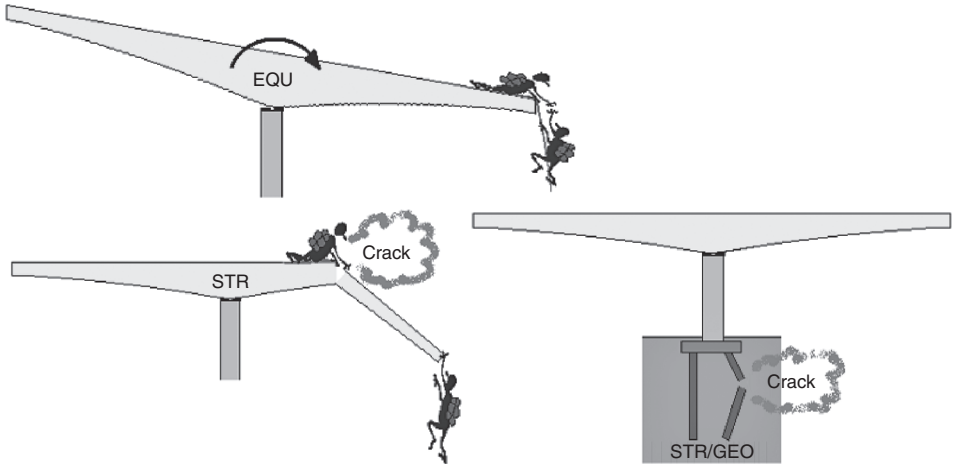


Fig. 8.4. Ultimate limit states EQU, STR and GEO for a bridge during execution

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Table 8.4. General expressions of combinations of actions for ultimate limit states, except fatigue

Combination	Reference: EN 1990	General expression
Fundamental (for persistent and transient design situations)	(6.10)	$\sum_{j\geq 1}\gamma_{Gj}G_{kj}\,''+''\,\gamma_P P\,''+''\,\gamma_{Q,1}Q_{k,1}\,''+''\,\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
	(6.10 a/b)	$\begin{cases}\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\,''+''\,\gamma_P P\,''+''\,\gamma_{Q,1}\psi_{0,1}Q_{k,1}\,''+''\,\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}\\ \sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}\,''+''\,\gamma_P P\,''+''\,\gamma_{Q,1}Q_{k,1}\,''+''\,\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}\end{cases}$ $0.85 \leq \xi_j \leq 1.00$ for unfavourable permanent actions G
Accidental (for accidental design situations)	(6.11)	$\sum_{j\geq 1}G_{kj}\,''+''\,P\,''+''\,A_d\,''+''\,(\psi_{1,1}\text{ or }\psi_{2,1})Q_{k1}\,''+''\,\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$
Seismic (for seismic design situations)	(6.12)	$\sum_{j\geq 1}G_{k,j}\,''+''\,P\,''+''\,A_{Ed}\,''+''\,\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$

this Designers' Guide) to the geotechnical actions as well as the actions on/from the structure.

Table A2.4(B): EN 1990 Annex A2

- Approach 2: Applying design values of actions from ***Table A2.4(B) of EN 1990 Annex A2*** (reproduced as Table 8.7 in this Designers' Guide) to the geotechnical actions as well as the actions on/from the structure.

Table A2.4(C): EN 1990

- Approach 3: Applying design values of actions from ***Table A2.4(C) of EN 1990*** Annex A2 (reproduced as Table 8.8 in this Designers' Guide) to the geotechnical actions and, simultaneously, applying design values of actions from Table A2.4(B) to the actions on/from the structure.

Tables A2.4(A), A2.4(B), A2.4(C): EN 1990 Annex A2

The choice of approach 1, 2 or 3 is left for national determination (National Annex). Figure 8.5 shows a diagrammatic representation of the use of ***Tables A2.4(A), A2.4(B) and A2.4(C) of EN 1990 Annex A2*** (reproduced as Tables 8.6, 8.7 and 8.8 in this Designers' Guide) of the Eurocode for the various ultimate limit states.

As for buildings, choices are left open at the national level concerning:

- the use of Expressions 6.10 or 6.10a/b
- the selection of the approach for verifications relating to limit states STR with geotechnical actions and limit states GEO.

Table 8.5. General formats for ULS and SLS verifications

Ultimate limit states (ULS)	EQU (static equilibrium)	$E_{d,dst} \leq E_{d,stb}$	$E_{d,dst}$ is the design value of the effect of destabilizing actions $E_{d,stb}$ is the design value of the effect of stabilizing actions
	STR/GEO (rupture or excessive deformation)	$E_d \leq R_d$	E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments R_d is the design value of the corresponding resistance
Serviceability limit states (SLS)		$E_d \leq C_d$	C_d is the limiting design value of the relevant serviceability criterion E_d is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination

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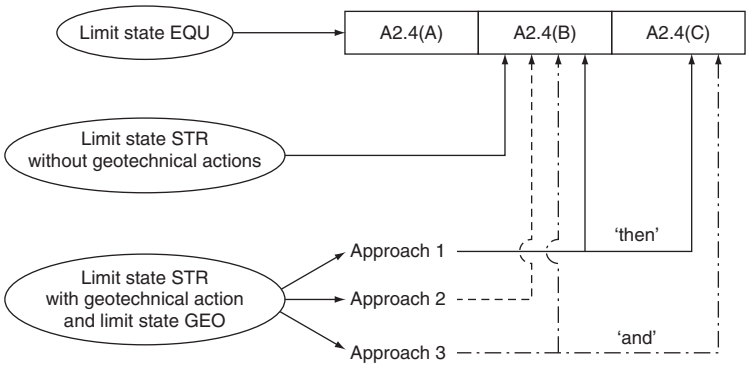


Fig. 8.5. Diagrammatic representation of the use of Tables A2.4(A), A2.4(B) and A2.4(C)

Concerning the use of Expressions 6.10 or 6.10a/b for bridges, it may be recommended to use only Expression 6.10 at the present stage. Indeed, many calculations experienced considerable difficulties in the application of Expressions 6.10a/b; one major difficulty is that the most unfavourable combination of actions, for a given cross-section, may be different depending on the effect under consideration (e.g. bending moment, shear force or torsion). Moreover, the economy is slight when using 6.10a/b instead of 6.10.

The UK National Annex to EN 1990 only allows the use of Expression 6.10 for the design of bridges in the UK.

Concerning the 'geotechnical' approach, in general, for the foundations of bridge piers (shallow or piled foundations), approach No. 2 may be adopted; this means that verification of the foundations may be performed with the same combinations of actions as for other parts of the structure. In some cases, for bridge abutments, it may be more appropriate to adopt Approach 3: it is a matter of expert judgement.

The UK National Annex requires the use of Approach 1, see Fig. 8.5. where the design applies in separate calculations design values from Table 8.7 and Table 8.8 of this Designers' Guide to the geotechnical actions as well as the other actions on/from the structure. In common cases, the sizing of foundations is governed by Table 8.8 and the structural resistance is governed by Table 8.7.

cl. 4.1.2(2)P: EN 1990

From a general point of view, in applying Tables 8.6 to 8.8 in cases where the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken.

A2.3.1(2): EN 1990: 2002/A1

For geotechnical problems (site stability, hydraulic and buoyancy failure, etc.), see EN 1997. It should be remembered that water actions and debris effects are covered in EN 1991-1-6 (see Chapter 3 of this Designers' Guide), and prestressing actions with the relevant values of γ_P partial factors are taken in accordance with EN 1990 to EN 1999, in particular EN 1992-1-1 (Clause 2.4.2.2), EN 1993-1-11 for tension elements (Clauses 2.2.(2), 5.2(3) and 5.3(2)), and EN 1994-2 (Clause 2.4.1.1). In the cases where γ_P values are not provided in the relevant design Eurocodes, these values may be defined as appropriate in the National Annex or for the individual project. They depend, among other things, on:

A2.3.1(8): EN 1990: 2002/A1

- the type of prestress
- the classification of prestress as a direct or an indirect action
- the type of structural analysis
- the unfavourable or favourable character of the prestressing action and the leading or accompanying character of prestressing in the combination.

For prestressing effects during the execution of the works, see also EN 1991-1-6 and Chapter 3 of this Designers' Guide.

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8.6.2. Design values and combinations of actions in persistent and transient design situations for EQU limit states

For EQU limit states, the design values of actions are taken from EN 1990: 2002/A1, Table A2.1 reproduced as Table 8.6 below, with some additional explanations.

The first remark in Table 8.6 concerns the reduction of the recommended values of γ factors for permanent actions (1.05 and 0.95) compared to the corresponding factors for buildings (1.10 and 0.90). The reason for this is that the magnitude of these actions is normally better controlled for a bridge than for a common type of building. For example, measurements have been performed in the case of bridge decks built by the cantilever method in a position different from the final position (e.g. when the final position is obtained by a rotation around a vertical axis): these measurements showed a difference of less than 2% between the self-weight of the two parts of the arms. It is possible to differentiate $G_{k,sup}$ and $G_{k,inf}$ or even to slightly reduce the recommended values of partial factors $\gamma_{G,sup}$ and $\gamma_{G,inf}$ in some cases.

In general, the risk of loss of static equilibrium is quite impossible for bridges during persistent design situations (i.e. when they have been fully completed) and even during some transient design situations corresponding to maintenance operations. However, the risk of loss of static equilibrium exists during execution (see Fig. 8.6).

Table 8.6. Design values of actions (EQU) (Set A) (Data taken from EN 1990: 2002/A1, Table A2.4(A))

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3 of EN 1990.

Note 1: The γ values for the persistent and transient design situations may be set by the National Annex.

For persistent design situations, the recommended set of values for γ are:

$\gamma_{G,sup} = 1.05$

$\gamma_{G,inf} = 0.95$[1]

$\gamma_Q = 1.35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)

$\gamma_Q = 1.45$ for rail traffic actions, where unfavourable (0 where favourable)

$\gamma_Q = 1.50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable)

γ_P = recommended values defined in the relevant design Eurocode.

For transient design situations during which there is a risk of loss of static equilibrium, $Q_{k,1}$ represents the dominant destabilizing variable action and $Q_{k,i}$ represents the relevant accompanying destabilizing variable actions.

During execution, if the construction process is adequately controlled, the recommended set of values for γ are:

$\gamma_{G,sup} = 1.05$

$\gamma_{G,inf} = 0.95$[1]

$\gamma_Q = 1.35$ for construction loads where unfavourable (0 where favourable)

$\gamma_Q = 1.50$ for all other variable actions, where unfavourable (0 where favourable)

[1] Where a counterweight is used, the variability of its characteristics may be taken into account, for example by one or both of the following recommended rules:

- applying a partial factor $\gamma_{G,inf} = 0.8$ where the self-weight is not well defined (e.g. containers)
- by considering a variation of its project-defined position specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined. For steel bridges during launching, the variation of the counterweight position is often taken equal to ± 1 m.

Note 2: For the verification of uplift of bearings of continuous bridges or in cases where the verification of static equilibrium also involves the resistance of structural elements (e.g. where the loss of static equilibrium is prevented by stabilizing systems or devices, e.g. anchors, stays or auxiliary columns), as an alternative to two separate verifications based on Tables A2.4(A) and A2.4(B), a combined verification, based on Table A2.4(A), may be adopted. The National Annex may set the γ values. The following values of γ are recommended:

$\gamma_{G,sup} = 1.35$

$\gamma_{G,inf} = 1.25$

$\gamma_Q = 1.35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)

$\gamma_Q = 1.45$ for rail traffic actions, where unfavourable (0 where favourable)

$\gamma_Q = 1.50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable)

$\gamma_Q = 1.35$ for all other variable actions, where unfavourable (0 where favourable)

provided that applying $\gamma_{G,inf} = .00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

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Fig. 8.6. Example of loss of static equilibrium of a prestressed concrete bridge deck built by the cantilever method

For the reason mentioned above, a Note to Table 8.6 draws the designer's attention to additional uncertainty on permanent actions during execution when a counterweight is used, in particular in the case of steel bridges during launching. This uncertainty may be taken into account by way of a specific γ factor on the weight of the counterweight, or through an imperfection of the location of the counterweight (± 1 m).

In some cases, the verification of static equilibrium also involves the resistance of some structural elements (Fig. 8.7).

Normally, the resistance of these structural members should be checked with combinations of actions corresponding to an ultimate limit state STR. However, the primary phenomenon is a risk of loss of static equilibrium. As for buildings, in order to avoid a double verification for which there is no real justification, the Eurocode allows a combined verification with a unique combination of actions in which the recommended values of the γ factors on permanent actions are taken equal to 1.35 ($=1.05+0.30$) and 1.25 ($=0.95+0.30$). More clearly, the general recommended combination of actions is:

$$1.35G_{\mathrm{kj,sup}} \,''+''\, 1.25G_{\mathrm{kj,inf}} \,''+''\, \gamma_{\mathrm{P}}P_{\mathrm{k}} \,''+''\, \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} \,''+''\, \sum_{i>1} \gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i}$$

but provided that applying $\gamma_{\mathrm{G,inf}} = 1.00$ to both the favourable and the unfavourable parts of permanent actions does not give a more unfavourable effect, i.e. with the following combination of actions:

$$G_{\mathrm{kj,sup}} \,''+''\, G_{\mathrm{kj,inf}} \,''+''\, \gamma_{\mathrm{P}}P_{\mathrm{k}} \,''+''\, \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} \,''+''\, \sum_{i>1} \gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i}$$

8.6.3. Design values and combinations of actions in persistent and transient design situations for STR/GEO limit states

As previously recalled, the design values of actions may be taken from EN 1990: 2002/A1, Table A2.4(B) and Table A2.4(C), depending on the limit state under consideration and the selected approach. Table 8.7 below gives set B of design values of actions (STR/GEO)

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Fig. 8.7. Examples of devices or members stabilizing bridge decks to prevent a loss of static equilibrium during execution: (a) Fastening of a concrete segment over a bridge pier; (b) Stabilization of an arm with cables; (c) Stabilization of an arm with auxiliary supporting columns

from EN 1990: 2002/A1, Table A2.4(B). For practical editorial reasons, and because it is recommended to use at present only Expression 6.10 for the verifications of resistance, Expressions 6.10 and 6.10a/b are not presented at the same level in this Designers' Guide.

Attention is drawn to Note 3: all permanent actions from **one source** represent a unique permanent action; a unique value of the partial factor is applicable to this permanent action, which may be $\gamma_{G,inf}$ or $\gamma_{G,sup}$ depending on its favourable or unfavourable character. It is, in particular, the case for self-weight: different partial factors shall not be applied to the spans of a multi-span bridge deck. Nevertheless, in cases when the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken according to 4.1.2(2)P of EN 1990. The single source principle is comprehensively explained in Part 1 of the TTL Designers' Guide for EN 1991: Actions on Buildings[4] and the TTL Designers' Guide to EN 1990.[2]

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Table 8.7. Design values of actions (STR/GEO) (set B) (Data taken from EN 1990: 2002/A1, Table A2.4(B))

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3. (Tables 8.1 to 8.3 of this Designers' Guide)
Note 1: The choice between 6.10, or 6.10a and 6.10b will be in the National Annex. In the case of 6.10a and 6.10b, the National Annex may in addition modify 6.10a to include permanent actions only.
Note 2: The γ and ξ values may be set by the National Annex. The following values for γ and ξ are recommended when using Expressions 6.10, or 6.10a and 6.10b:
$\gamma_{G,sup} = 1.35$[(1)]
$\gamma_{G,inf} = 1.00$
$\gamma_Q = 1.35$ when Q represents unfavourable actions due to road or pedestrian traffic (0 when favourable)
$\gamma_Q = 1.45$ when Q represents unfavourable actions due to rail traffic, for load groups 11 to 31 (except 16, 17, 26[(3)] and 27[(3)]), load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions (0 when favourable)
$\gamma_Q = 1.20$ when Q represents unfavourable actions due to rail traffic, for load groups 16 and 17 and SW/2 (0 when favourable)
$\gamma_Q = 1.50$ for other traffic actions and other variable actions[(2)]
$\xi = 0.85$ (so that $\xi\gamma_{G,sup} = 0.85 \times 1.35 \cong 1.15$)
$\gamma_{G,set} = 1.20$ in the case of linear elastic analysis, and $\gamma_{G,set} = 1.35$ in the case of non-linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.
See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.
γ_P = recommended values defined in the relevant design Eurocode.
(1) This value covers self-weight of structural and non-structural elements, ballast, soil, groundwater and free water, removable loads, etc.
(2) This value covers variable horizontal earth pressure from soil, groundwater, free water and ballast, traffic load surcharge earth pressure, traffic aerodynamic actions, wind and thermal actions, etc.
(3) For rail traffic actions for load groups 26 and 27 $\gamma_Q = 1.20$ may be applied to individual components of traffic actions associated with SW/2 and $\gamma_Q = 1.45$ may be applied to individual components of traffic actions associated with load models LM71, SW/0 and HSLM, etc.
Note 3: The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self-weight of the structure may be considered as coming from one source; this also applies if different materials are involved. See however A2.3.1(2).
Note 4: For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1.0–1.15 may be used in most common cases and may be modified in the National Annex.
Note 5: Where actions due to water are not covered by EN 1997 (e.g. flowing water), the combinations of actions to be used may be specified for the individual project.

With the recommended values of Table 8.7, the simplified combination rules detailed in Section 8.3.1 and the recommended values of Table 8.1, the most common combinations of actions for **road bridges** in **persistent design situations** can be expressed as follows:

$$\left\{\sum_{j\geq 1}(1.35G_{kj,sup}\ ''+''\ 1.00G_{kj,inf})\right\}$$

$$''+''\ \gamma_P P_k\ ''+''\ \begin{cases} 1.35(TS + UDL + q_{fk}^*) + 1.5 \times 0.6F_{Wk,traffic} \\ 1.35gri_{i=1b,2,3,4,5} \\ 1.5T_k + 1.35(0.75TS + 0.4UDL + 0.4q_{fk}^*) \\ 1.5F_{Wk} \\ 1.5Q_{Sn,k} \end{cases}$$

In these expressions, q_{fk}^* represents the 'combination value' (or 'reduced value') of vertical loads on footways and cycle tracks of load group gr1a: its recommended value is $3\,kN/m^2$. Expressions $(TS + UDL + q_{fk}^*)$ and $(0.75TS + 0.4UDL + 0.4q_{fk}^*)$ correspond respectively to 'gr1a' and to '$\psi_0$gr1a'. Concerning the prestressing force P_k, in most cases this force is used with its mean value P_m and $\gamma_P = 1$. $F_{Wk,traffic}$ represents wind actions taking into

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Table 8.8. Design values of actions (STR/GEO) (set C) (Data taken from EN 1990: 2002/A1, Table A2.4(C))

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3 (Tables 8.1 to 8.3 of this Designers' Guide).
Note: The γ values may be set by the National Annex. The recommended set of values for γ are:
$\gamma_{G,sup} = 1.00$
$\gamma_{G,inf} = 1.00$
$\gamma_{G,set} = 1.00$
$\gamma_Q = 1.15$ for road and pedestrian traffic actions where unfavourable (0 where favourable)
$\gamma_Q = 1.25$ for rail traffic actions where unfavourable (0 where favourable)
$\gamma_Q = 1.30$ for the variable part of horizontal earth pressure from soil, groundwater, free water and ballast, for traffic load surcharge horizontal earth pressure, where unfavourable (0 where favourable)
$\gamma_Q = 1.30$ for all other variable actions where unfavourable (0 where favourable)
$\gamma_{G,set} = 1.00$ in the case of linear elastic or non-linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.
γ_P = recommended values defined in the relevant design Eurocode.

account the presence of road traffic on the bridge deck (see Chapter 2 of this Designers' Guide).

Finally, where relevant, two values are recommended for $\gamma_{G,set}$: 1.20 in the case of a linear elastic analysis, and 1.35 in the case of a non-linear analysis, but only where the effects of settlements are unfavourable. The explanation is rather simple: a linear elastic analysis is rather unfavourable concerning phenomena which develop progressively with time, with the possibility of redistribution of efforts. Therefore, a reduced value of the partial factor is proposed, compared to the 'normal' value for permanent actions (1.35).

In the case of **footbridges** in persistent design situations, for application of the simplified combination rules, the recommended values of Tables 8.2 and 8.8 allow the following combinations of actions for STR/GEO Ultimate Limit States to be written:

$$\left\{\sum_{j\geq 1}(1.35G_{kj,sup}\ ''+''\ 1.00G_{kj,inf})\right\}\ ''+''\ \gamma_P P_k\ ''+''\ \begin{cases}1.35gr1\ ''+''\ 1.5\times 0.3F_{Wk}\\ 1.35gr2\ ''+''\ 1.5\times 0.3F_{Wk}\\ 1.35Q_{fwk}\\ 1.5T_k\ ''+''\ 1.35\times 0.4gr1\\ 1.5F_{Wk}\\ 1.5Q_{Sn,k}\end{cases}$$

The same remarks apply for the prestressing force, settlements and the relevant partial factors as for road bridges.

In the case of **railway bridges**, generally the approach described in EN 1990, equation (6.10), see Table 8.4, should be used for persistent and transient design situations, unless specified otherwise by the relevant authority. The number of practical combinations of actions is greater than for road bridges or footbridges. For that reason, the whole set of possibilities with the various load groups will not be given here. However, the way to establish the combinations of actions follows rules, which are very similar to those for road bridges or footbridges.

Table 8.7 gives set B of design values of actions (STR/GEO) taken from EN 1990: 2002/A1, Table A2.4(B).

8.6.4. Design values and combinations of actions in the accidental and seismic design situations

All recommended values of partial factors for actions for the ultimate limit states in the accidental and seismic design situations (Expressions 6.11a to 6.12b of EN 1990) are equal

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Table 8.9. Design values of actions for use in accidental and seismic combinations of actions (Data taken from EN 1990: 2002/A1, Table A2.5)

Design situation	Permanent actions		Prestress	Accidental or seismic action	Accompanying variable actions (†)	
	Unfavourable	Favourable			Main (if any)	Others
Accidental (*) (Eq. 6.11a/b)	$G_{kj,sup}$	$G_{kj,inf}$	P	A_d	$\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$	$\psi_{2,i}\,Q_{k,i}$
Seismic(‡) (Eq. 6.12a/b)	$G_{kj,sup}$	$G_{kj,inf}$	P	$A_{Ed} = \gamma_I A_{Ek}$		$\psi_{2,i}\,Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National Annex, depending on the accidental action under consideration.
(†) Variable actions are those considered in Tables A2.1 to A2.3 (i.e. Tables 8.1 to 8.3 of this Designers' Guide).
(‡) The National Annex or the individual project may specify particular **seismic design situations. For railway bridges only one track need be loaded and load model SW/2 may be neglected**.
Note: The design values in this Table A2.5 may be changed in the National Annex. The recommended values are $\gamma = 1.0$ for all non-seismic actions.

to 1.00. This is represented symbolically in Table 8.9 which reproduces Table A2.5 of EN 1990 Annex A2.

One or several variable actions need to be considered simultaneously with the accidental action in very special circumstances. In any case, no variable action with its frequent value is taken as a 'main' action.

Accidental design situations may have to be taken into account during execution. For example, in the case of bridges built by the cantilever method, a severe accidental situation may be the fall of a travelling form during its displacement or of a prefabricated unit during its fastening to the structure. Some variable actions (construction loads) may have to be taken into account simultaneously with the accidental action.

The accidental combination of actions in the case of loss of static equilibrium during execution is expressed as follows in common cases:

$$\sum_{j \geq 1} G_{kj,sup} \,''+''\, \sum_{j \geq 1} G_{kj,inf} \,''+''\, P \,''+''\, A_d \,''+''\, \psi_2 Q_{c,k} \qquad \text{EN 1990: 2002/A1, (A2.2)}$$

where $Q_{c,k}$ is the characteristic value of construction loads as defined in EN 1991-1-6 (i.e. the characteristic value of the relevant combination of groups Q_{ca}, Q_{cb}, Q_{cc}, Q_{cd}, Q_{ce} and Q_{cf}) – see Chapter 3 of this Designers' Guide.

The UK National Annex to EN 1990 stipulates the use of ψ_1 to be used for the main accompanying variable action in the accidental design situation.

8.7. Combinations of actions and criteria for serviceability

8.7.1. General

The expressions of combinations of actions for serviceability limit states are given in Table 8.10.

In these expressions, the values of γ factors are equal to 1, which is a recommended value. In most cases, there is no reason to alter this value: the fact that all γ factors are equal to 1 in combinations of actions for serviceability limit states is a consequence of the general principles of the semi-probabilistic format of verification of constructions.

The verifications are symbolically represented by the following equation:

$$E_d \leq C_d$$

where

C_d is the limiting design value of the relevant serviceability criterion
E_d is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination.

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Table 8.10. General expressions of combinations of actions for serviceability limit states (Data taken from EN 1990: 2002/A1, Table A2.6)

Combination	Reference: EN 1990	General expression
Characteristic	(6.14)	$\sum_{j\geq 1} G_{k,j} \,''+''\, P \,''+''\, Q_{k,1} \,''+''\, \sum_{i>1} \psi_{0,i} Q_{k,i}$
Frequent	(6.15)	$\sum_{j\geq 1} G_{k,j} \,''+''\, P \,''+''\, \psi_{1,1} Q_{k,1} \,''+''\, \sum_{i>1} \psi_{2,i} Q_{k,i}$
Quasi-permanent	(6.16)	$\sum_{j\geq 1} G_{k,j} \,''+''\, P \,''+''\, \sum_{i\geq 1} \psi_{2,i} Q_{k,i}$

The serviceability criteria depend on serviceability requirements which are defined either in EN 1990 Annex A2 or in the design Eurocodes EN 1992 to EN 1999. Specific serviceability requirements may also be defined for the individual project. Hereafter, only serviceability criteria defined in EN 1990 Annex A2 are mentioned and, where relevant, commented upon.

From a general point of view, serviceability criteria for bridges are mainly connected with deformations and vibrations.

With the recommended expressions of Table 8.10, the simplified combination rules detailed in Section 8.3.1 and the recommended values of Table 8.1, the most common characteristic combinations of actions for serviceability limit states concerning **road bridges** in **persistent design situations** are expressed as follows:

- Characteristic combinations of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup} \,''+''\, G_{kj,inf})\right\} \,''+''\, P_k \,''+''\, \begin{cases} (TS + UDL + q^*_{fk}) \,''+''\, 0.6F_{Wk,traffic} \\ gri_{i=1b,2,3,4,5} \,''+''\, 0.6T_k \\ gr1b \\ T_k \,''+''\, (0.75TS + 0.4UDL + 0.4q^*_{fk}) \\ F_{Wk} \\ Q_{Sn,k} \end{cases}$$

  Symbols and notation have the same meaning as for ultimate limit states.

- Frequent combinations of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup} \,''+''\, G_{kj,inf})\right\} \,''+''\, P_k \,''+''\, \begin{cases} (0.75TS + 0.4UDL) \,''+''\, 0.5T_k \\ 0.75gr1b \\ 0.75gr4 \,''+''\, 0.5T_k \\ 0.6T_k \\ 0.2F_{Wk} \\ 0.5Q_{Sn,k} \end{cases}$$

- Quasi-permanent combinations of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup} \,''+''\, G_{kj,inf})\right\} \,''+''\, P_k \,''+''\, 0.5T_k$$

In the case of **footbridges** in persistent design situations, for the application of the simplified combination rules, the recommended values of Tables 8.2 and 8.8 allow the following combinations of actions to be written:

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- Characteristic combination of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup}\,''+''\,G_{kj,inf})\right\}\,''+''\,P_k\,''+''\begin{cases} gr1\,''+''\,0.3F_{Wk} \\ gr2\,''+''\,0.3F_{Wk} \\ gr1\,''+''\,0.6T_k \\ gr2\,''+''\,0.6T_k \\ T_k+0.4gr1 \\ F_{Wk} \\ Q_{Sn,k} \end{cases}$$

- Frequent combinations of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup}\,''+''\,G_{kj,inf})\right\}\,''+''\,P_k\,''+''\begin{cases} 0.4gr1\,''+''\,0.5T_k \\ 0.6T_k \\ 0.2F_{Wk} \\ 0.8Q_{Sn,k} \end{cases}$$

- Quasi-permanent combination of actions

$$\left\{\sum_{j\geq 1}(G_{kj,sup}\,''+''\,G_{kj,inf})\right\}\,''+''\,P_k\,''+''\,0.5T_k$$

The same remarks apply for the prestressing force, settlements and the relevant partial factors as for road bridges.

8.7.2. Serviceability criteria regarding deformation and vibration for road bridges

Notes A2.4.2(3): EN 1990: 2002/A1

As mentioned in EN 1990), vibrations of road bridges may have various origins, in particular traffic actions and wind actions. For vibrations due to wind actions, see EN 1991-1-4 and Chapter 2 of this Designers' Guide. For vibrations due to traffic actions, comfort criteria may have to be defined. Fatigue effects may also have to be taken into account, in particular fatigue effects on stays or suspension cables. However, the verification of serviceability limit states concerning deformation and vibration needs to be considered only in exceptional cases for road bridges when completed. In such cases, the frequent combination of actions is recommended for the assessment of deformation.

The designer's attention is drawn to the risks induced by uplift of the bridge deck at supports, risks for traffic safety and for mechanical integrity of structural elements such as bearings. Concerning structural bearings, it should be borne in mind that there is a risk of displacement and, consequently, of malfunctioning when the bridge deck has a significant general slope. The vibrations due to road traffic are transmitted to the structural bearings and induce displacements.

Finally, the problems of deformation and vibration for road bridges are not solved by a good standard, but by a good design!

8.7.3. Verification concerning vibration of footbridges due to pedestrian traffic

The main sources of vibration of footbridges are wind actions and actions due to pedestrian traffic. As explained in Chapter 5 of this Designers' Guide, to date (2009) it has not been possible to define universal well-fitted models of pedestrian traffic for various circumstances, in particular the presence of streams of pedestrians. EN 1990 Annex A2 gives examples of some common situations: footbridges in highly populated urban areas, in the vicinity of railway and bus stations, schools, or any other places where crowds may congregate, or any important building with public admittance, etc.

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In fact, EN 1990 Annex A2 states that pedestrian comfort criteria should be defined in terms of maximum acceptable acceleration of any part of the deck. Motion sensitivity is also seen to be strongly dependent on damping.

Only recommended maximum values of acceleration (m/s^2) are proposed for any part of the deck: *A2.4.3.2: EN 1990: 2002/A1*

- 0.7 for vertical vibrations
- 0.2 for horizontal vibrations due to normal use
- 0.4 for exceptional crowd conditions.

Additionally, EN 1990 Annex A2 states that a verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than:

- 5 Hz for vertical vibrations
- 2.5 Hz for horizontal (lateral) and torsional vibrations.

However, this does not mean that, for some footbridges or parts of footbridges, a sophisticated verification of the comfort criteria has not to be envisaged beyond the mentioned values.

The most advanced reference document concerning the variation of frequency dependency of response perception is ISO 2631.[5] For information, Annex C (Examples of vibration criteria) of ISO/DIS 10137[6] (*Bases for design of structures – Serviceability of buildings and walkways against vibrations*) mentions, in its paragraph C.1.2 'Walkways':

> *The design situations should be selected depending on the pedestrian traffic to be admitted on the individual footbridge during its design working life. It is recommended to consider the following scenarios:*
>
> - *One person walking across the walkway and another (the receiver) standing at mid-span.*
> - *An average pedestrian flow based on a daily occurrence rate, e.g. a group size of 8 to 15 people, depending on the length and the width of the walkway.*
> - *The presence of streams of pedestrians (significantly more than 15 persons).*
> - *Occasional festive or choreographic events (when relevant).*
>
> *In the absence of more definitive data, the level of vibrations in vertical direction (z-axis) for walkways over road or waterways should not exceed those obtained by a multiplying factor of 60 to the relevant base curve, figures C.1, except where one or more person standing still on the walkway has to be accounted for (such as the first scenario), in which case a multiplying factor of 30 should be applicable. Horizontal vibrations induced by pedestrian traffic or wind should not exceed 60 times the base curve for the horizontal direction (x- and y-axis), Figure C.2.*

The figures C.1 and C.2 mentioned in the above quotation are reproduced below as Fig. 8.8.

8.7.4. Verifications regarding deformations and vibrations for railway bridges

General

The control of deformations and vibrations is a major problem for railway bridges because excessive bridge deformations can endanger traffic by creating unacceptable changes in vertical and horizontal track geometry, excessive rail stresses and vibrations in bridge structures. Likewise, excessive vibrations can lead to ballast instability and unacceptable reduction in wheel rail contact forces. Excessive deformations can also affect the loads imposed on the track–bridge system, and create conditions which cause passenger discomfort.

EN 1990 Annex A2 gives a list of points to be checked. In the following, two major points only are developed: the deck twist for normal track gauge and the vertical deformation of the deck (permissible deflections). *A2.4.4.2.2: EN 1990: 2002/A1* *A2.4.4.2.3: EN 1990: 2002/A1*

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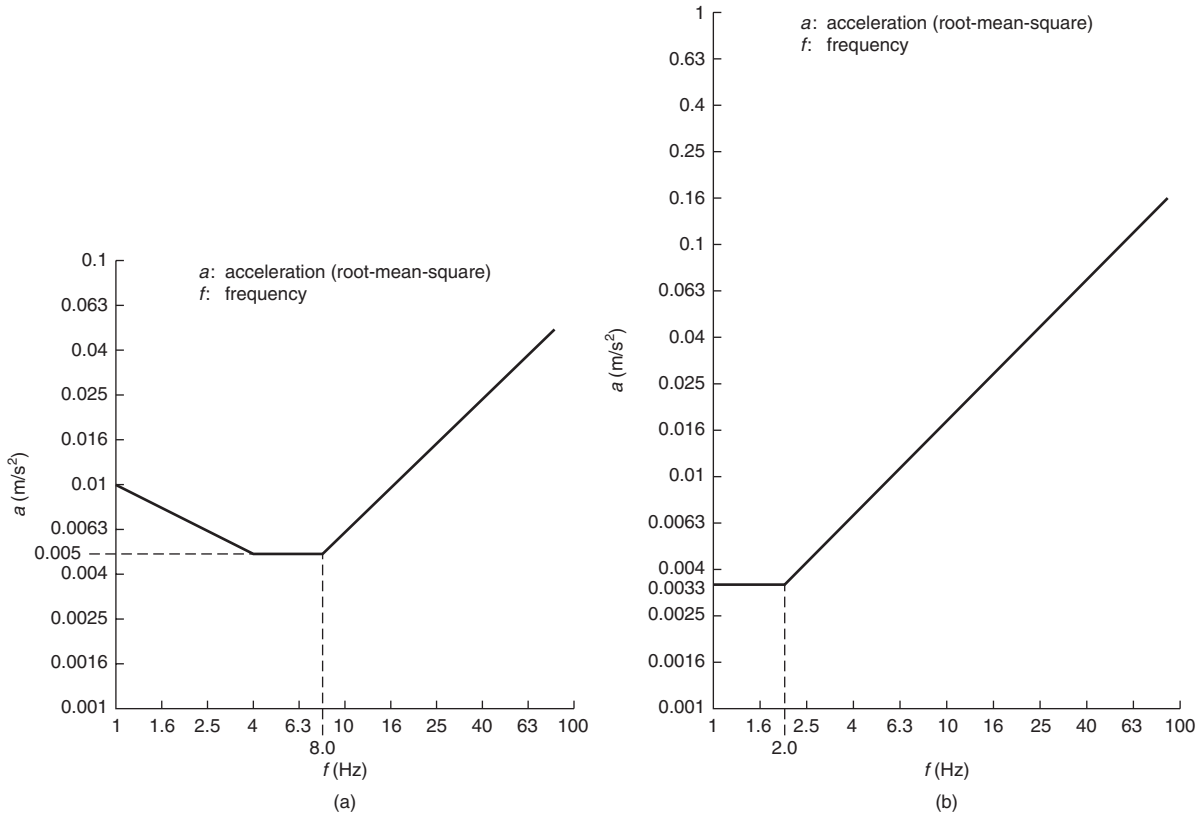


Fig. 8.8. Vibrations in buildings according to ISO/DIS 10137[6]: (a) (ISO/DIS 10137): Building vibration z-axis base curve for acceleration; (b) (ISO/DIS 10137): Building vibration x- and y-axis base curve for acceleration

A2.4.4.2.3(1): EN 1990: 2002/A1

It should be noted that only minimum conditions for vertical bridge deformations are given in EN 1990: 2002/A1, A2.4.4.2.3(1). If these conditions would be determinant in the design of a bridge, this could lead to bridges with insufficient stiffness, provoking premature track maintenance at the ends of the bridges. It is important to bear in mind what was pointed out earlier in Section 6.8.2 stiffness afforded to bridges costs nothing when considering life-cycle costs.

A2.4.4.2.2: EN 1990: 2002/A1

Deck twist for normal track gauge

The twist of the bridge deck shall be calculated taking into account the characteristic values of Load Model 71, as well as SW/0 or SW/2 as appropriate multiplied by Φ and α and Load Model HSLM (for speeds over 200 km/h) including centrifugal effects all in accordance with EN 1991-2 Section 6. Twist shall be checked on the approach to the bridge, across the bridge and for the departure from the bridge.

Note: The check of the twist is an important condition for rail traffic safety. Therefore the value $\alpha = 1.33$ has to be taken with Load Model 71 or SW/0 where relevant.

The maximum twist t (mm/3 m) of a track gauge s (m) of 1.35 m measured over a length of 3 m (Fig. 8.9) should not exceed the values given in Table 8.11.

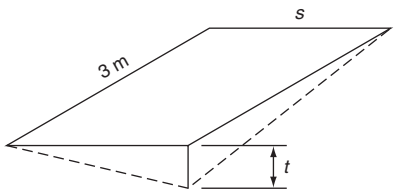


Fig. 8.9. Definition of deck twist (Reproduced from EN 1990:2002/A1, with permission from BSI)

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Table 8.11. Limiting values of deck twist (EN 1990: 2002/A1, Table A2.7)

Speed range V (km/h)	Maximum twist t (mm/3 m)
$V \leq 120$	$t \leq t_1$
$120 < V \leq 200$	$t \leq t_2$
$V > 200$	$t \leq t_3$

With the following recommended values for the set of t:

$t_1 = 4.5$
$t_2 = 3.0$
$t_3 = 1.5$

The total track twist due to any twist which may be present in the track when the bridge is not subject to rail traffic actions (e.g. in a transition curve), plus the track twist due to the total deformation of the bridge resulting from rail traffic actions, shall not exceed t_T, with a recommended value $t_T = 7.5\,\text{mm}/3\,\text{m}$. See also B6.1.4 of this Designers' Guide, if Load Model HSLM or real trains are leading for the design of the bridge.

Vertical deformation of the deck (permissible deflections)

The vertical traffic loads applied to the bridge cause the deck to bend, resulting in a vertical displacement of every point on the surface of the deck. In general, maximum displacement occurs at the point in the middle of the deck, or at midspan. This displacement is known as the deflection of the deck.

cl. A2.4.4.2.3(1): EN 1990: 2002/A1

Note: The condition in ***Clause A2.4.4.2.3(1): EN 1990: 2002/A1****, that the maximum total vertical deflection measured along any track due to rail traffic actions should not exceed L/600 does not take into account track maintenance! A simplified rule is given hereafter to avoid the need for excessive track maintenance. In addition the following simplified rules have the advantage that no dynamic analysis is necessary for speeds <200 km/h.*

For all classified lines with $\alpha > 1.0$ (that means also if $\alpha = 1.33$ is adopted), the permissible values for deflections in Table 8.12 are recommended, always calculated with LM71 (SW/0) and with $\boldsymbol{\alpha = 1}$.

The deflection of the deck also causes rotation of the ends of the deck. For a succession of simple beams, see Fig. 8.10, the permissible values for deflections may therefore be reduced, to avoid the permissible total relative rotation between the adjacent ends of two decks being doubled.

The deflection of the deck under traffic loads causes the end of the deck behind the support structures to lift. This lifting must be reduced to:

cl. 6.5.4.5: EN 1991-2

$V \leq 160\,\text{km/h} \quad \leq 3\,\text{mm}$
$160 < V \leq 200\,\text{km/h} \quad \leq 2\,\text{mm}$

taking LM71 (SW/0) with $\boldsymbol{\alpha = 1.00}$.

Table 8.12. Permissible vertical deflections to avoid excessive track maintenance

$V < 80$ km/h	$\delta_{stat} \leq L/800$ Note: Due to what is said above, namely that the maximum total deflection measured along any track due to rail traffic actions should not exceed $L/600$, please note that 600 multiplied with 1.33 gives approximately 800.
$80 \leq V \leq 200$ km/h	$\delta_{stat} \leq L/(15V - 400)$ Note: The upper limit $L/2600$ for 200 km/h is the permissible deflection which DB (Deutsche Bundesbahn – German railways) has taken following many years of designing bridges for high-speed lines in Germany, a value which gave satisfaction.
$V > 200$ km/h	$\delta_{dyn} \leq$ value given by the dynamic study, but $\delta_{stat} \leq L/2600$

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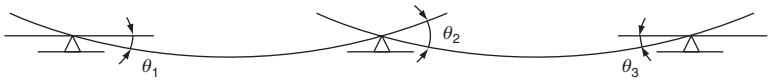


Fig. 8.10. Angular rotations at the end of decks (Reproduced from EN 1990:2002/A1, with permission from BSI)

In general, additional limits of angular rotations at the end of decks in the vicinity of expansion devices, switches and crossings are not necessary with the permissible deformations in Table 8.12 respected.

The requirements for non-ballasted structures have to be specified by the relevant authority, in relation to the function of the system.

cl. A.2.4.4.2.4: EN 1990: 2002/A1

Permissible tranverse deformations and vibrations of the deck are given in ***Clause A.2.4.4.2.4: EN 1990: 2002/A1***.

cl. A.2.4.4.3: EN 1990: 2002/A1

Note: The passenger comfort criteria given in ***Clause A.2.4.4.3: EN 1990: 2002/A1*** has no significance, when the vertical deformations are in accordance with the permissible values given in Table 8.12.

8.8. Worked example of combinations of actions during execution

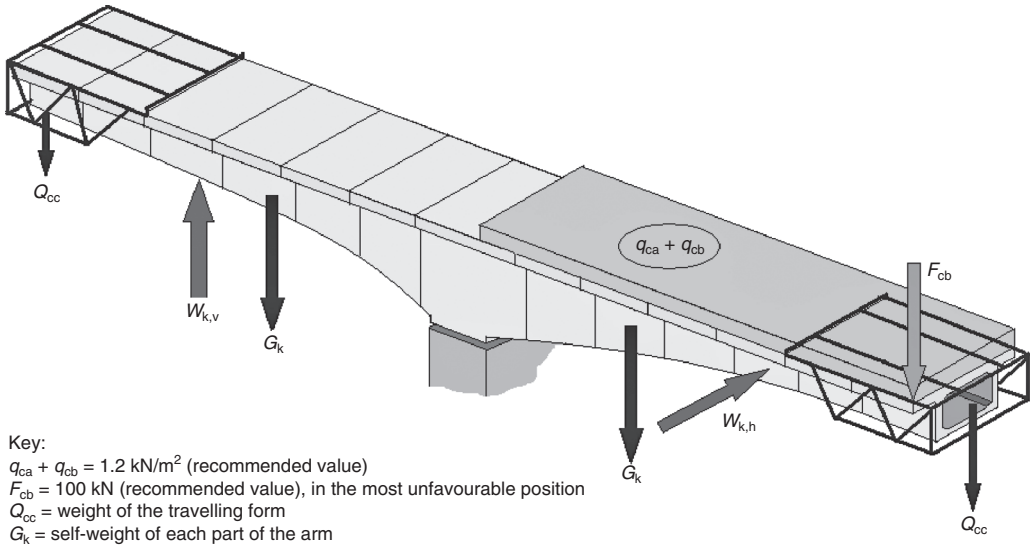
The following example is intended to illustrate the method for establishing combinations of actions during execution in the case of a prestressed concrete bridge deck built by the cantilever method. The design situations to be taken into account are:

- a transient design situation for the verification of devices and structural members associated with the stability and resistance of the bridge deck during execution
- an accidental design situation corresponding to the fall of a precast unit or of a travelling form

For the transient design situation, two cases may have to be envisaged:

- the arm is not symmetrical because one segment is being poured on one side
- the arm is symmetrical but a storm is arriving and execution personnel or visitors leave the site and small-scale equipment is removed.

These two cases are shown in Fig. 8.11.



Key:
$q_{ca} + q_{cb}$ = 1.2 kN/m^2 (recommended value)
F_{cb} = 100 kN (recommended value), in the most unfavourable position
Q_{cc} = weight of the travelling form
G_k = self-weight of each part of the arm
$W_{k,v}$ = characteristic value of the wind force corresponding to unbalanced uplift
$W_{k,h}$ = characteristic value of the wind force corresponding to unbalanced drag.

Fig. 8.11. Stability of a bridge deck built by the cantilever method during execution

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In the following equations, the symbol F_{Wk} covers both actions ($W_{k,v}$; $W_{k,h}$) of Fig. 8.10.

(a) EQU limit-state with only permanent and variable actions
Preliminary note:

$\psi_0 = 1$ is the recommended value for construction loads and $\psi_0 = 0.8$ is the recommended value for wind actions during execution (see Tables 8.1 to 8.3 of Chapter 8 of this Designers' Guide). With these recommended values, it is obvious that construction loads should be systematically taken as accompanying actions to obtain the most unfavourable combination of actions.

The most unfavourable combination of actions is:

$$1.05G_{kj,sup}\,''+''\,0.95G_{kj,inf}\,''+''\,P''+''\,1.5F_{Wk}\,''+''\,1.35Q_{ck}$$

In the case of combined resistance–static equilibrium verification, the combination of actions is:

$$1.35G_{kj,sup}\,''+''\,1.25G_{kj,inf}\,''+''\,P''+''\,1.35F_{Wk}\,''+''\,1.35Q_{ck}$$

if the following combination of actions is not more unfavourable:

$$G_{kj,sup}\,''+''\,G_{kj,inf}\,''+''\,P''+''\,1.35F_{Wk}\,''+''\,1.35Q_{ck}$$

(b) EQU limit-state with an accidental action

$$G_{kj,sup}\,''+''\,G_{kj,inf}\,''+''\,P''+''\,A_d\,''+''\,Q_{ck}$$

A_d represents, for example, the fall of a travelling form.

(c) EQU limit-sate in seismic design situation

$$G_{kj,sup}\,''+''\,G_{kj,inf}\,''+''\,P''+''\,A_{Ed}[=\gamma_I A_{Ek}]\,''+''\,Q_{ck}$$

(d) STR/GEO ultimate limit states

$$1.35G_{kj,sup}\,''+''\,G_{kj,inf}\,''+''\,P''+''\,1.5F_{Wk}\,''+''\,1.5Q_{ck}$$

$$1.35G_{kj,sup}\,''+''\,G_{kj,inf}\,''+''\,P''+''\,1.5Q_{ck}\,''+''\,1.5\times 0.8F_{Wk}$$

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