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EN 1991-2 (2003) (English): Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

EN 1991-2

September 2003

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English version

Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges

Eurocode 1: Actions sur les structures - Partie 2: Actions sur les ponts, dues au trafic

Eurocode 1: Einwirkungen auf Tragwerke - Teil 2: Verkehrslasten auf Brücken

This European Standard was approved by CEN on 28 November 2002.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Management Centre or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Slovakia, Spain, Sweden, Switzerland and United Kingdom.

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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Contents

Foreword

This document (EN 1991-2:2003) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by March 2004, and conflicting national standards shall be withdrawn at the latest by [AC1⟩ March 2010 ⟨AC1].

This document supersedes ENV 1991-3:1995.

CEN/TC 250 is responsible for all Structural Eurocodes.

[AC1⟩ According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and the United Kingdom. ⟨AC1]

Background of the Eurocode Programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

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[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

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

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and 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 N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;

– as a basis for specifying contracts for construction works and related engineering services ;

– as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving ~~a~~ full compatibility of these technical specifications with the Eurocodes.

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National Annex.

The National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.* :
- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), *e.g.* snow map,
- procedure to be used where alternative procedures are given in the Eurocode.

It may also contain
- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1991-2

EN 1991-2 defines models of traffic loads for the design of road bridges, footbridges and railway bridges. For the design of new bridges, EN 1991-2 is intended to be used, for direct application, together with Eurocodes EN 1990 to 1999.

The bases for combinations of traffic loads with non-traffic loads are given in EN 1990, A2.

[4] see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1 (Interpretative Document Nr. 1).

Complementary rules may be specified for individual projects :

- when traffic loads need to be considered which are not defined in this Part of Eurocode 1 (*e.g.* site loads, military loads, tramway loads) ;
- for bridges intended for both road and rail traffic ;
- for actions to be considered in accidental design situations ;
- for masonry arch bridges.

For road bridges, Load Models 1 and 2, defined in 4.3.2 and 4.3.3, and taken into account with adjustment factors α and β equal to 1, are deemed to represent the most severe traffic met or expected in practice, other than that of special vehicles requiring permits to travel, on the main routes of European countries. The traffic on other routes in these countries and in some other countries may be substantially lighter, or better controlled. However it should be noted that a great number of existing bridges do not meet the requirements of this EN 1991-2 and the associated Structural Eurocodes EN 1992 to EN 1999.

It is therefore recommended to the national authorities that values of the adjustment factors α and β be chosen for road bridge design corresponding possibly to several classes of routes on which the bridges are located, but remain as few and simple as possible, based on consideration of the national traffic regulations and the efficiency of the associated control.

For railway bridges, Load Model 71 (together with Load Model SW/0 for continuous bridges), defined in 6.3.2, represent the static effect of standard rail traffic operating over the standard-gauge or wide-gauge European mainline-network. Load Model SW/2, defined in 6.3.3, represents the static effect of heavy rail traffic. The lines, or sections of lines, over which such loads shall be taken into account are defined in the National Annex (see below) or for the individual project.

Provision is made for varying the specified loading to cater for variations in the type, volume and maximum weight of rail traffic on different railways, as well as for different qualities of track. The characteristic values given for Load Models 71 and SW/0 may be multiplied by a factor α for lines carrying rail traffic which is heavier or lighter than the standard.

In addition two other load models are given for railway bridges :

- load model "unloaded train" for checking the lateral stability of single track bridges and
- load model HSLM to represent the loading from passenger trains at speeds exceeding 200 km/h.

Guidance is also given on aerodynamic actions on structures adjacent to railway tracks as a result of passing trains and on other actions from railway infrastructure.

Bridges are essentially public works, for which :

- the European Directive 89/440/EEC on contracts for public works is particularly relevant, and
- public authorities have responsibilities as owners.

Public authorities may also have responsibilities for the issue of regulations on authorised traffic (especially on vehicle loads) and for delivery and control dispensations when relevant, *e.g.* for special vehicles.

EN 1991-2 is therefore intended for use by :
- committees drafting standards for structural design and related product, testing and execution standards ;
- clients (*e.g.* for the formulation of their specific requirements on traffic and associated loading requirements) ;
- designers and constructors ;
- relevant authorities.

AC1) Where a Table or a Figure (AC1 are part of a NOTE, the Table or the Figure number is followed by (n) (*e.g.* Table 4.5(n)).

National Annex for EN 1991-2

This Standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices have to be made. Therefore the National Standard implementing EN 1991-2 should have a National Annex containing all Nationally Determined Parameters to be used for the design of bridges to be constructed in the relevant country.

National choice is allowed in EN 1991-2 through the following clauses :

Section 1 : General	
1.1(3)	Complementary rules for retaining walls, buried structures and tunnels.

Section 2 : Classification of actions	
2.2(2) NOTE 2	Use of infrequent values of loading for road bridges
2.3(1)	Definition of appropriate protection against collisions
2.3(4)	Rules concerning collisions forces from various origins

Section 3 : Design situations	
(5)	Rules for bridges carrying both road and rail traffic

Section 4 : Road traffic actions and other actions specifically for road bridges	
4.1(1) NOTE 2	Road traffic actions for loaded lengths greater than 200m
4.1(2) NOTE 1	Specific load models for bridges with limitation of vehicle weight
4.2.1(1) NOTE 2	Definition of complementary load models
4.2.1(2)	Definition of models of special vehicles
4.2.3(1)	Conventional height of kerbs
4.3.1(2) NOTE 2	Use of LM2
4.3.2(3) NOTES 1 & 2	Values of α factors

4.3.2(6)	Use of simplified alternative load models
4.3.3(2)	Values of β factor
4.3.3(4) NOTE 2	Selection of wheel contact surface for LM2
4.3.4(1)	Definition of Load Model 3 (special vehicles)
4.4.1(2) NOTE 2	Upper limit of the braking force on road bridges
AC1 *Text deleted* AC1	AC1 *Text deleted* AC1
4.4.1(3)	Horizontal forces associated with Load Model 3
4.4.1(6)	Braking force transmitted by expansion joints
4.4.2(4)	Lateral forces on road bridge decks
4.5.1 – Table 4.4a Notes a and b	Consideration of horizontal forces in gr1a
AC1 4.5.2(1) NOTE 3 AC1	Use of infrequent values of variable actions
AC1 4.6.1(2) NOTE 2 and NOTE 4 AC1	Use of Fatigue Load Models
4.6.1(3) NOTE 1	Definition of traffic categories
4.6.1(6)	Definition of additional amplification factor (fatigue)
4.6.4(3)	Adjustment of Fatigue Load Model 3
4.6.5(1) NOTE 2	Road traffic characteristics for the use of Fatigue Load Model 4
4.6.6(1)	Use of Fatigue Load Model 5
4.7.2.1(1)	Definition of impact force and height of impact
4.7.2.2(1) NOTE 1	Definition of collision forces on decks
4.7.3.3(1) NOTE 1	Definition of collision forces on vehicle restraint systems
4.7.3.3(1) NOTE 3	Definition of vertical force acting simultaneously with the horizontal collision force
4.7.3.3(2)	Design load for the structure supporting a vehicle parapet
4.7.3.4(1)	Definition of collision forces on unprotected vertical structural members
4.8(1) NOTE 2	Definition of actions on pedestrian parapets
4.8(3)	Definition of design loads due to pedestrian parapets for the supporting structure
4.9.1(1) NOTE 1	Definition of load models on embankments

Section 5 : Actions on footways, cycle tracks and footbridges	
5.2.3(2)	Definition of load models for inspection gangways
5.3.2.1(1)	Definition of the characteristic value of the uniformly distributed load
5.3.2.2(1)	Definition of the characteristic value of the concentrated load on footbridges
5.3.2.3(1)P NOTE 1	Definition of service vehicles for footbridges
5.4(2)	Characteristic value of the horizontal force on footbridges

5.6.1(1)	Definition of specific collision forces
5.6.2.1(1)	Collision forces on piers
5.6.2.2(1)	Collision forces on decks
5.6.3(2) NOTE 2	Definition of a load model for accidental presence of a vehicle on a footbridge
5.7(3)	Definition of dynamic models of pedestrian loads

Section 6 : Rail traffic actions and other actions specifically for railway bridges	
6.1(2)	Traffic outside the scope of EN1991-2, alternative load models
6.1(3)P	Other types of railways
6.1(7)	Temporary bridges
6.3.2(3)P	Values of α factor
6.3.3(4)P	Choice of lines for heavy rail traffic
6.4.4	Alternative requirements for a dynamic analysis
6.4.5.2(3)P	Choice of dynamic factor
6.4.5.3(1)	Alternative values of determinant lengths
6.4.5.3 Table 6.2	Determinant length of cantilevers
6.4.6.1.1(6)	Additional requirements for the application of HSLM
6.4.6.1.1(7)	Loading and methodology for dynamic analysis
6.4.6.1.2(3) Table 6.5	Additional load cases depending upon number of tracks
6.4.6.3.1(3) Table 6.6	Values of damping
6.4.6.3.2(3)	Alternative density values of materials
6.4.6.3.3(3) NOTE 1 NOTE 2	 Enhanced Young's modulus Other material properties
6.4.6.4(4)	Reduction of peak response at resonance and alternative additional damping values
6.4.6.4(5)	Allowance for track defects and vehicle imperfections
6.5.1(2)	Increased height of centre of gravity for centrifugal forces
6.5.3(5)	Actions due to braking for loaded lengths greater than 300 m
6.5.3(9)P	Alternative requirements for the application of traction and braking forces
6.5.4.1(5)	Combined response of structure and track, requirements for non-ballasted track
6.5.4.3.(2) NOTES 1 & 2	Alternative requirements for temperature range
6.5.4.4(2) NOTE 1	Longitudinal shear resistance between track and bridge deck
6.5.4.5	Alternative design criteria
6.5.4.5.1(2)	Minimum value of track radius
6.5.4.5.1(2)	Limiting values for rail stresses
6.5.4.6	Alternative calculation methods
6.5.4.6.1(1)	Alternative criteria for simplified calculation methods
6.5.4.6.1(4)	Longitudinal plastic shear resistance between track and bridge deck
6.6.1(3)	Aerodynamic actions, alternative values
6.7.1(2)P	Derailment of rail traffic, additional requirements

6.7.1(8)P	Derailment of rail traffic, measures for structural elements situated above the level of the rails and requirements to retain a derailed train on the structure
6.7.3(1)P	Other actions
6.8.1(11)P Table 6.10	Number of tracks loaded when checking drainage and structural clearances
6.8.2(2) Table 6.11	Assessment of groups of loads
6.8.3.1(1)	Frequent values of multi-component actions
6.8.3.2(1)	Quasi-permanent values of multi-component actions
6.9(6)	Fatigue load models, structural life
6.9(7)	Fatigue load models, special traffic
Annex C(3)P	Dynamic factor
Annex C(3)P	Method of dynamic analysis
Annex D2(2)	Partial safety factor for fatigue loading

Section 1 General

1.1 Scope

(1) EN 1991-2 defines imposed loads (models and representative values) associated with road traffic, pedestrian actions and rail traffic which include, when relevant, dynamic effects and centrifugal, braking and acceleration actions and actions for accidental design situations.

(2) Imposed loads defined in EN 1991-2 are intended to be used for the design of new bridges, including piers, abutments, upstand walls, wing walls and flank walls etc., and their foundations.

(3) The load models and values given in EN 1991-2 should be used for the design of retaining walls adjacent to roads and railway lines.

NOTE For some models only, applicability conditions are defined in EN 1991-2. For the design of buried structures, retaining walls and tunnels, provisions other than those in EN 1990 to EN 1999 may be necessary. Possible complementary conditions may be defined in the National Annex or for the individual project.

(4) EN 1991-2 is intended to be used in conjunction with EN 1990 (especially A2) and EN 1991 to EN 1999.

(5) Section 1 gives definitions and symbols.

(6) Section 2 defines loading principles for road bridges, footbridges (or cycle-track bridges) and railway bridges.

(7) Section 3 is concerned with design situations and gives guidance on simultaneity of traffic load models and on combinations with non-traffic actions.

(8) Section 4 defines :

- imposed loads (models and representative values) due to traffic actions on road bridges and their conditions of mutual combination and of combination with pedestrian and cycle traffic (see section 5) ;

- other actions specifically for the design of road bridges.

(9) Section 5 defines :

- imposed loads (models and representative values) on footways, cycle tracks and footbridges ;

- other actions specifically for the design of footbridges.

(10) Sections 4 and 5 also define loads transmitted to the structure by vehicle restraint systems and/or pedestrian parapets.

(11) Section 6 defines :

– imposed actions due to rail traffic on bridges ;

– other actions specifically for the design of railway bridges and structures adjacent to the railway.

1.2 Normative references

This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications, are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

EN 1317	Road restraint systems Part 1 : Terminology and general criteria for test methods Part 2 : Performance classes, impact test acceptance criteria and test methods for safety barriers Part 6 : Pedestrian restraint systems, pedestrian parapetparpets

NOTE The Eurocodes were published as European Prestandards. The following European Standards which are published or in preparation are cited in normative clauses or in NOTES to normative clauses :

EN 1990	Eurocode : Basis of Structural Design
EN 1991-1-1	Eurocode 1 : Actions on structures : Part 1-1 : General actions - Densities, self-weight imposed loads for buildings
EN 1991-1-3	Eurocode 1 : Actions on structures : Part 1-3 : General actions - Snow loads
prEN 1991-1-4	Eurocode 1 : Actions on structures : Part 1-4 : General actions - Wind actions
prEN 1991-1-5	Eurocode 1 : Actions on structures : Part 1-5 : General actions - Thermal actions
prEN 1991-1-6	Eurocode 1 : Actions on structures : Part 1-6 : General actions - Actions during execution
prEN 1991-1-7	Eurocode 1 : Actions on structures : Part 1-7 : General actions - Accidental actions
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 1997	Eurocode 7 : Geotechnical design
EN 1998	Eurocode 8 : Design of structures for earthquake resistance
EN 1999	Eurocode 9 : Design of aluminium structures

1.3 Distinction between Principles and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in EN 1991-2 between Principles and Application Rules.

(2) The Principles comprise :
- general statements and definitions for which there is no alternative, as well as ;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The Principles are identified by the letter P following the paragraph number.

(4) The Application Rules are generally recognised rules which comply with the Principles and satisfy their requirements.

(5) It is permissible to use alternative design rules different from the Application Rules given in EN 1991-2 for works, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the structural safety, serviceability and durability which would be expected when using the Eurocodes.

NOTE If an alternative design rule is substituted for an Application Rule, the resulting design cannot be claimed to be wholly in accordance with EN 1991-2 although the design will remain in accordance with the Principles of EN 1991-2. When EN 1991-2 is used in respect of a property listed in an annex Z of a product standard or an ETAG[5], the use of an alternative design rule may not be acceptable for CE marking.

(6) In EN 1991-2, the Application Rules are identified by a number in brackets *e.g.* as this clause.

1.4 Terms and definitions

NOTE 1 For the purposes of this European Standard, general definitions are provided in EN 1990 and additional definitions specific to this Part are given below.

NOTE 2 Terminology for road restraint systems is derived from EN 1317-1.

1.4.1 Harmonised terms and common definitions

1.4.1.1
deck
parts of a bridge which carry the traffic loading over piers, abutments and other walls, pylons being excluded

1.4.1.2
road restraint system
general name for vehicle restraint system and pedestrian restraint system used on the road

NOTE Road restraint systems may be, according to use :
- permanent (fixed) or temporary (demountable, *i.e.* they are removable and used during temporary road works, emergencies or similar situations),
- deformable or rigid,
- single-sided (they can be hit on one side only) or double-sided (they can be hit on either side).

[5] ETAG : European Technical Approval Guideline

1.4.1.3
safety barrier
road vehicle restraint system installed alongside, or on the central reserve, of a road

1.4.1.4
vehicle parapet
safety barrier installed on the edge, or near the edge, of a bridge or on a retaining wall or similar structure where there is a vertical drop and which may include additional protection and restraint for pedestrians and other road users

1.4.1.5
pedestrian restraint system
system installed to retain and to provide guidance for pedestrians

1.4.1.6
pedestrian parapet
pedestrian or "other user" restraint system along a bridge or on top of a retaining wall or similar structure and which is not intended to act as a road vehicle restraint system

1.4.1.7
pedestrian guardrail
pedestrian or "other user" restraint system along the edge of a footway or footpath intended to restrain pedestrians and other users from stepping onto or crossing a road or other area likely to be hazardous

NOTE "Other user" may include provision for equestrians, cyclists and cattle.

1.4.1.8
noise barrier
screen to reduce transmission of noise

1.4.1.9
inspection gangway
permanent access for inspection, not open for public traffic

1.4.1.10
movable inspection platform
part of a vehicle, distinct from the bridge, used for inspection

1.4.1.11
footbridge
bridge intended mainly to carry pedestrian and/or cycle-track loads, and on which neither road traffic loads, except those permitted vehicles *e.g.* maintenance vehicles, nor any railway load are permitted

1.4.2 Terms and definitions specifically for road bridges

1.4.2.1
carriageway
for application of sections 4 and 5, the part of the road surface, supported by a single structure (deck, pier, etc.), which includes all physical traffic lanes (*i.e.* as may be marked on the road surface), hard shoulders, hard strips and marker strips (see 4.2.3(1))

1.4.2.2
hard shoulder
surfaced strip, usually of one traffic lane width, adjacent to the outermost physical traffic lane, intended for use by vehicles in the event of difficulty or during obstruction of the physical traffic lanes

1.4.2.3
hard strip
surfaced strip, usually less than or equal to 2 m wide, located alongside a physical traffic lane, and between this traffic lane and a safety barrier or vehicle parapet

1.4.2.4
central reservation
area separating the physical traffic lanes of a dual-carriageway road. It generally includes a median strip and lateral hard strips separated from the median strip by safety barriers.

1.4.2.5
notional lane
strip of the carriageway, parallel to an edge of the carriageway, which in section 4 is deemed to carry a line of cars and/or lorries

1.4.2.6
remaining area
difference, where relevant, between the total area of the carriageway and the sum of the areas of the notional lanes (see Figure 4.1)

1.4.2.7
tandem system
assembly of two consecutive axles considered to be simultaneously loaded

1.4.2.8
abnormal load
vehicle load which may not be carried on a route without permission from the relevant authority

1.4.3 Terms and definitions specifically for railway bridges

1.4.3.1
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.

1.4.3.2
footpath
strip located alongside the tracks, between the tracks and the parapets

1.4.3.3
resonant speed
traffic speed at which a frequency of loading (or a multiple of) matches a natural frequency of the structure (or a multiple of)

1.4.3.4
frequent operating speed
most probable speed at the site for a particular type of Real Train (used for fatigue considerations)

1.4.3.5
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)

1.4.3.6
maximum permitted vehicle speed
maximum permitted speed of Real Trains due to vehicle considerations and generally independent of the infrastructure

1.4.3.7
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.

1.4.3.8
maximum design speed
generally 1,2 × Maximum Nominal Speed

1.4.3.9
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.

1.5 Symbols

For the purposes of this European Standard, the following symbols apply.

1.5.1 Common symbols

NOTE Symbols used in one place only are not systematically repeated below.

Latin upper case letters

L	In general, loaded length

Latin lower case letters

gri	Group of loads, i is a number (i = 1 to n)
r	Horizontal radius of a carriageway or track centre-line, distance between wheel loads (Figure 6.3)

1.5.2 Symbols specifically for sections 4 and 5

Latin upper case letters

Q_{ak}	Characteristic value of a single axle load (Load Model 2) for a road bridge (see 4.3.3)
Q_{flk}	Characteristic horizontal force on a footbridge
Q_{fwk}	Characteristic value of the concentrated load (wheel load) on a footbridge (see 5.3.2.2)
Q_{ik}	Magnitude of characteristic axle load (Load Model 1) on notional lane number i (i = 1, 2...) of a road bridge
Q_{lk}	Magnitude of the characteristic longitudinal forces (braking and acceleration forces) on a road bridge
Q_{serv}	Load model corresponding to a service vehicle for footbridges
Q_{tk}	Magnitude of the characteristic transverse or centrifugal forces on road bridges
Q_{trk}	Transverse braking force on road bridges
TS	Tandem system for Load Model 1
UDL	Uniformly distributed load for Load Model 1

Latin lower case letters

f_h	In general, natural horizontal frequency of a bridge
f_v	In general, natural vertical frequency of a bridge
n_l	Number of notional lanes for a road bridge
q_{eq}	Equivalent uniformly distributed load for axle loads on embankments (see 4.9.1)
q_{fk}	Characteristic vertical uniformly distributed load on footways or footbridges
q_{ik}	Magnitude of the characteristic vertical distributed load (Load Model 1) on notional lane number i (i = 1, 2...) of a road bridge
q_{rk}	Magnitude of the characteristic vertical distributed load on the remaining area of the carriageway (Load Model 1)
w	Carriageway width for a road bridge, including hard shoulders, hard strips and marker strips (see 4.2.3(1))
w_l	Width of a notional lane for a road bridge

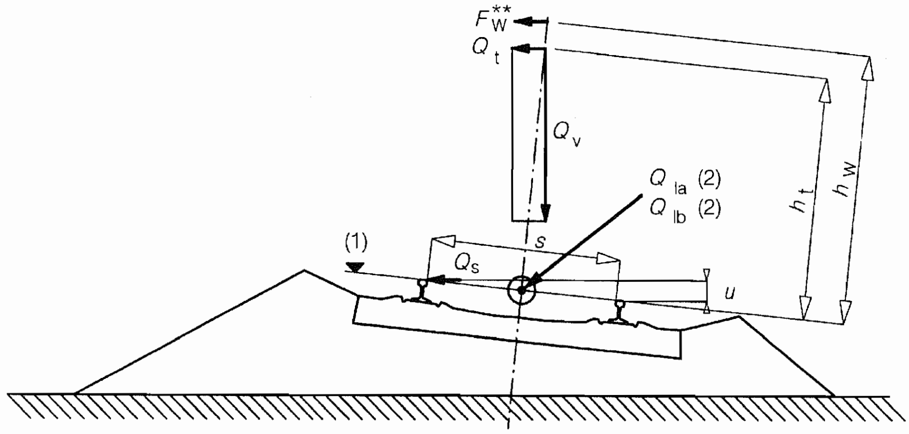
Greek upper case letters

$\Delta\varphi_{fat}$	Additional dynamic amplification factor for fatigue near expansion joints (see 4.6.1(6))

Greek lower case letters

α_{Qi}, α_{qi}	adjustment factors of some load models on lanes i (i = 1, 2...), defined in 4.3.2
α_{qr}	Adjustment factor of load models on the remaining area, defined in 4.3.2
β_Q	Adjustment factor of Load Model 2 defined in 4.3.3
φ_{fat}	Dynamic amplification factor for fatigue (see annex B)

1.5.3 Symbols specifically for section 6



Key

(1) Running surface

(2) Longitudinal forces acting along the centreline of the track

Figure 1.1 - Notation and dimensions specifically for railways

Latin upper case letters

$A_{(L/\lambda)}G_{(\lambda)}$	Aggressivity (see Equations E.4 and E.5)
D	Coach or vehicle length
D_{IC}	Intermediate coach length for a Regular Train with one axle per coach
E_{cm}	Secant modulus of elasticity of normal weight concrete
F_L	Total longitudinal support reaction
F_{Qk}	Characteristic longitudinal force per track on the fixed bearings due to deformation of the deck
F_{Tk}	Longitudinal force on a fixed bearing due to the combined response of track and structure to temperature
F_W^{**}	Wind force compatible with rail traffic
F_{li}	Individual longitudinal support reaction corresponding to the action i
G	Self-weight (general)
H	Height between (horizontal) axis of rotation of the (fixed) bearing and the upper surface of the deck (underside of ballast beneath tracks)
K	Total longitudinal support stiffness
K_2	Longitudinal support stiffness per track per m, 2E3 kN/m
K_5	Longitudinal support stiffness per track per m, 5E3 kN/m
K_{20}	Longitudinal support stiffness per track per m, 20E3 kN/m
L	Length (general)
L_T	Expansion length
L_{TP}	Maximum permissible expansion length
L_f	Influence length of the loaded part of curved track
L_i	Influence length
L_Φ	"determinant" length (length associated with Φ)

M	Number of point forces in a train
N	Number of regularly repeating coaches or vehicles, or number of axles, or number of equal point forces
P	Point force Individual axle load
Q	Concentrated force or variable action (general)
Q_{A1d}	Point load for derailment loading
Q_h	Horizontal force (general)
Q_k	Characteristic value of a concentrated force or a variable action (*e.g.* characteristic value of a vertical loading on a non-public footpath)
Q_{lak}	Characteristic value of traction force
Q_{lbk}	Characteristic value of braking force
Q_r	Rail traffic action (general, *e.g.* resultant of wind and centrifugal force)
Q_{sk}	Characteristic value of nosing force
Q_{tk}	Characteristic value of centrifugal force
Q_v	Vertical axle load
Q_{vi}	Wheel load
Q_{vk}	Characteristic value of vertical load (concentrated load)
ΔT	Temperature variation
ΔT_D	Temperature variation of the deck
ΔT_N	Temperature variation
ΔT_R	Temperature variation of the rail
V	Speed in km/h Maximum Line Speed at the Site in km/h
X_i	Length of sub-train consisting of i axles

Latin lower case letters

a	Distance between rail supports, length of distributed loads (Load Models SW/0 and SW/2)
a_g	Horizontal distance to the track centre
a'_g	Equivalent horizontal distance to the track centre
b	Length of the longitudinal distribution of a load by a sleeper and ballast
c	Space between distributed loads (Load Models SW/0 and SW/2)
d	Regular spacing of groups of axles Spacing of axles within a bogie Spacing of point forces in HSLM-B
d_{BA}	Spacing of axles within a bogie
d_{BS}	Spacing between centres of adjacent bogies
e	Eccentricity of vertical loads, eccentricity of resulting action (on reference plane)
e_c	Distance between adjacent axles across the coupling of two individual regular trainsets
f	Reduction factor for centrifugal force
f_{ck}, $f_{ck, cube}$	Concrete compressive cylinder/ cube strength
g	Acceleration due to gravity

h	Height (general)
	Height of cover including ballast from the top of the deck to the top of a sleeper
h_g	Vertical distance from the running surface to the underside of the structure above the track
h_t	Height of centrifugal force over the running surface
h_w	Height of wind force over the running surface
k	Longitudinal plastic shear resistance of the track
k_1	Train shape coefficient
k_2	Multiplication factor for slipstream actions on vertical surfaces parallel to the tracks
k_3	Reduction factor for slipstream actions on simple horizontal surfaces adjacent to the track
k_4	Multiplication factor for slipstream actions on surfaces enclosing the tracks (horizontal actions)
k_5	Multiplication factor for slipstream actions on surfaces enclosing the tracks (vertical actions)
k_{20}	Longitudinal plastic shear resistance of track, 20kN per m of track
k_{40}	Longitudinal plastic shear resistance of track, 40kN per m of track
k_{60}	Longitudinal plastic shear resistance of track, 60kN per m of track
n_0	First natural bending frequency of the unloaded structure
n_T	First natural torsional frequency of the structure
q_{A1d}, q_{A2d}	Distributed loading for derailment loading
q_{fk}	Characteristic value of vertical loading on non-public footpath (uniformly distributed load)
q_{ik}	Characteristic value of equivalent distributed aerodynamic action
q_{lak}	Characteristic value of distributed traction force
q_{lbk}	Characteristic value of distributed braking force
q_{tk}	Characteristic value of distributed centrifugal force
q_{v1}, q_{v2}	Vertical load (uniformly distributed load)
q_{vk}	Characteristic value of vertical load (uniformly distributed load)
r	Radius of track curvature
	Transverse distance between wheel loads
s	Gauge
u	Cant, relative vertical distance between the uppermost surface of the two rails at a particular location along the track
v	Maximum Nominal Speed in m/s
	Maximum Permitted Vehicle Speed in m/s
	Speed in m/s
v_{DS}	Maximum Design Speed in m/s
v_i	Resonant speed in m/s
y_{dyn} , y_{stat}	Maximum dynamic response and maximum corresponding static response at any particular point

Greek upper case letters

Θ	End rotation of structure (general)
$\Phi(\Phi_2, \Phi_3)$	Dynamic factor for railway Load Models 71, SW/0 and SW/2

Greek lower case letters

α	Load classification factor Coefficient for speed Linear temperature coefficient for thermal expansion
β	Ratio of the distance between the neutral axis and the surface of the deck relative to height H
δ	Deformation (general) Vertical deflection
δ_0	Deflection at midspan due to permanent actions
δ_B	Longitudinal relative displacement at the end of the deck due to traction and braking
δ_H	Longitudinal relative displacement at the end of the deck due to deformation of the deck
δ_h	Horizontal displacement Horizontal displacement due to the longitudinal displacement of the foundations of the substructure
δ_p	Horizontal displacement due to the longitudinal deformation of the substructure
δ_V	Vertical relative displacement at the end of the deck
δ_φ	Horizontal displacement due to longitudinal rotation of foundation
γ_{Ff}	Partial safety factor for fatigue loading
γ_{Mf}	Partial safety factor for fatigue strength
$\varphi, \varphi', \varphi''$	Dynamic enhancement of static loading for Real Trains
φ'_{dyn}	Dynamic enhancement of static loading for a Real Train determined from a dynamic analysis
κ	Coefficient relating to the stiffness of an abutment relative to the piers
λ	Damage equivalent factor for fatigue Excitation wavelength
λ_C	Critical wavelength of excitation
λ_i	Principal wavelength of excitation
λ_v	Wavelength of excitation at the Maximum Design Speed
ρ	Density
σ	Stress
σ_A, σ_B, σ_M	Pressure on the upper surface of the deck from rail traffic actions
$\Delta\sigma_{71}$	Stress range due to the Load Model 71 (and where required SW/0)
$\Delta\sigma_C$	Reference value of fatigue strength
ξ	Reduction factor for the determination of the longitudinal forces in the fixed bearings of one-piece decks due to traction and braking
ζ	Lower limit of percentage of critical damping (%), or damping ratio
ζ_{TOTAL}	Total damping (%)
$\Delta\zeta$	Additional damping (%)

Section 2 Classification of actions

2.1 General

(1) The relevant traffic actions and other specific actions on bridges should be classified in accordance with EN 1990, section 4 (4.1.1).

(2) Traffic actions on road bridges, footbridges and railway bridges consist of variable actions and actions for accidental design situations, which are represented by various models.

(3) All traffic actions should be classified as free actions within the limits specified in sections 4 to 6.

(4) Traffic actions are multi-component actions.

2.2 Variable actions

(1) For normal conditions of use (*i.e.* excluding any accidental situation), the traffic and pedestrian loads (dynamic amplification included where relevant) should be considered as variable actions.

(2) The various representative values are :

- characteristic values, which are either statistical, *i.e.* corresponding to a limited probability of being exceeded on a bridge during its design working life, or nominal, see EN 1990, 4.1.2(7) ;
- frequent values ;
- quasi-permanent values.

NOTE 1 In Table 2.1, some information is given on the bases for the calibration of the main Load Models (fatigue excluded) for road bridges and footbridges. Rail loading and the associated γ and ψ factors have been developed using Method (a) in Figure C.1 of EN 1990.

Table 2.1 – Bases for the calibration of the main Load Models (fatigue excluded)

Traffic Load Models	Characteristic values	Frequent values	Quasi-permanent values
Road bridges			
LM1 (4.3.2)	1000 year return period (or probability of exceedance of 5% in 50 years) for traffic on the main roads in Europe (α factors equal to 1, see 4.3.2).	1 week return period for traffic on the main roads in Europe (α factors equal to 1, see 4.3.2).	Calibration in accordance with definition given in EN 1990.
LM2 (4.3.3)	1000 year return period (or probability of exceedance of 5% in 50 years) for traffic on the main roads in Europe (β factor equal to 1, see 4.3.3).	1 week return period for traffic on the main roads in Europe (β factor equal to 1, see 4.3.3).	Not relevant
LM3 (4.3.4)	Set of nominal values. Basic values defined in annex A are derived from a synthesis based on various national regulations.	Not relevant	Not relevant
LM4 (4.3.5)	Nominal value deemed to represent the effects of a crowd. Defined with reference to existing national standards.	Not relevant	Not relevant
Footbridges			
Uniformly distributed load (5.3.2.1)	Nominal value deemed to represent the effects of a crowd. Defined with reference to existing national standards.	Equivalent static force calibrated on the basis of 2 pedestrians/m^2 (in the absence of particular dynamic behaviour). It can be considered, for footbridges in urban areas, as a load of 1 week return period.	Calibration in accordance with definition given in EN 1990.
Concentrated load (5.3.2.2)	Nominal value. Defined with reference to existing national standards.	Not relevant	Not relevant
Service vehicle (5.3.2.3)	Nominal value. As specified or given in 5.6.3.	Not relevant	Not relevant

NOTE 2 For road bridges, the National Annex may impose the use of infrequent values which are intended to correspond approximately to a mean return period of one year for traffic on the main roads in Europe. See also EN 1992-2, EN1994-2 and EN 1990, A2.

(3) For calculation of fatigue lives, separate models, associated values and, where relevant, specific requirements are given in 4.6 for road bridges, in 6.9 for railway bridges, and in the relevant annexes.

2.3 Actions for accidental design situations

(1) Road vehicles and trains may generate actions due to collision, or their accidental presence or location. These actions should be considered for the structural design where appropriate protection is not provided.

NOTE Appropriate protection may be defined in the National Annex or for the individual project.

(2) Actions for accidental design situations described in this Part of EN 1991 refer to common situations. They are represented by various load models defining design values in the form of static equivalent loads.

(3) For actions due to road vehicles under road bridges, footbridges and railway bridges during accidental design situations, see AC1 4.7.2, 5.6.2 and 6.7.2 AC1.

(4) Collision forces due to boats, ships or aeroplanes, for road bridges, footbridges and railway bridges (*e.g.* over canals and navigable water), should be defined where appropriate.

NOTE The National Annex may define the collision forces. Recommended values for boat and ship impacts are given in EN 1991-1-7. Additional requirements may be specified for the individual project.

(5) Actions for accidental design situations due to road vehicles on road bridges and footbridges are defined in 4.7.3 and 5.6.3 respectively.

(6) Actions for accidental design situations due to trains or railway infrastructure are defined in 6.7. They are applicable where relevant to road bridges, footbridges and railway bridges.

Section 3 Design situations

(1)P Selected design situations shall be taken into account and critical load cases identified. For each critical load case, the design values of the effects of actions in combination shall be determined.

NOTE For bridges for which signalling is used to limit the weight of vehicles, an accidental design situation may have to be taken into account, corresponding to the crossing of the bridge by one vehicle in breach of warnings.

(2) The various traffic loads to be taken into account as simultaneous when using groups of loads (combinations of action components) are given in the following sections ; each of which should be considered in design calculations, where relevant.

(3)P The combination rules, depending on the calculation to be undertaken, shall be in accordance with EN 1990.

NOTE For seismic combinations for bridges and associated rules, see EN 1998-2.

(4) Specific rules for the simultaneity with other actions for road bridges, footbridges, and railway bridges are given in EN 1990, A2.

(5) For bridges intended for both road and rail traffic, the simultaneity of actions and the particular required verifications should be specified.

NOTE The particular rules may be defined in the National Annex or for the individual project.

Section 4 Road traffic actions and other actions specifically for road bridges

4.1 Field of application

(1) Load models defined in this section should be used for the design of road bridges with loaded lengths less than 200 m.

NOTE 1 200 m corresponds to the maximum length taken into account for the calibration of Load Model 1 (see 4.3.2). In general, the use of Load Model 1 is safe-sided for loaded lengths over 200 m.

NOTE 2 Load models for loaded lengths greater than 200 m may be defined in the National Annex or for the individual project.

(2) The models and associated rules are intended to cover all normally foreseeable traffic situations (*i.e.* traffic conditions in either direction on any lane due to the road traffic) to be taken into account for design (see however (3) and the notes in 4.2.1).

NOTE 1 Specific models may be defined in the National Annex or for the individual project to be used for bridges equipped with appropriate means including road signs intended to strictly limit the weight of any vehicle (*e.g.* for local, agricultural or private roads).

NOTE 2 Load models for abutments and walls adjacent to bridges are defined separately (see 4.9). They derive from the road traffic models without any correction for dynamic effects. For frame bridges, loads on road embankments may also give rise to action effects in the bridge structure.

(3) The effects of loads on road construction sites (*e.g.* due to scrapers, lorries carrying earth, etc.) or of loads specifically for inspection and tests are not intended to be covered by the load models and should be separately specified, where relevant.

4.2 Representation of actions

4.2.1 Models of road traffic loads

(1) Loads due to the road traffic, consisting of cars, lorries and special vehicles (*e.g.* for industrial transport), give rise to vertical and horizontal, static and dynamic forces.

NOTE 1 The load models defined in this section do not describe actual loads. They have been selected and calibrated so that their effects (with dynamic amplification included where indicated) represent the effects of the actual traffic in the year 2000 in European countries.

NOTE 2 The National Annex may define complementary load models, with associated combination rules where traffic outside the scope of the load models specified in this section needs to be considered.

NOTE 3 The dynamic amplification included in the models (fatigue excepted), although established for a medium pavement quality (see annex B) and pneumatic vehicle suspension, depends on various parameters and on the action effect under consideration. Therefore, it cannot be represented by a unique factor. In some unfavourable cases, it may reach 1,7 (local effects), but still more unfavourable values can be reached for poorer pavement quality, or if there is a risk of resonance. These cases can be avoided by appropriate quality and design measures. Therefore, an additional dynamic amplification may have to be taken into account for particular calculations (see 4.6.1.(6)) or for the individual project.

(2) Where vehicles which do not comply with National regulations concerning limits of weights and, possibly, dimensions of vehicles not requiring special permits, or military loads, have to be taken into account for the design of a bridge, they should be defined.

NOTE The National Annex may define these models. Guidance on standard models for special vehicles and their application is given in annex A. See 4.3.4.

4.2.2 Loading classes

(1) The actual loads on road bridges result from various categories of vehicles and from pedestrians.

(2) Vehicle traffic may differ between bridges depending on its composition (*e.g.* percentages of lorries), its density (*e.g.* average number of vehicles per year), its conditions (*e.g.* jam frequency), the extreme likely weights of vehicles and their axle loads, and, if relevant, the influence of road signs restricting carrying capacity.

These differences should be taken into account through the use of load models suited to the location of a bridge (*e.g.* choice of adjustment factors α and β defined in 4.3.2 for Load Model 1 and in 4.3.3 for Load Model 2 respectively).

4.2.3 Divisions of the carriageway into notional lanes

(1) The carriageway width, w, should be measured between kerbs or between the inner limits of vehicle restraint systems, and should not include the distance between fixed vehicle restraint systems or kerbs of a central reservation nor the widths of these vehicle restraint systems.

NOTE The National Annex may define the minimum value of the height of the kerbs to be taken into account. The recommended minimum value of this height is 100 mm.

(2) The width w_l of notional lanes on a carriageway and the greatest possible whole (integer) number n_l of such lanes on this carriageway are defined in Table 4.1.

Table 4.1 - Number and width of notional lanes

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_1 = 1$	3 m	w - 3 m
$5{,}4 \text{ m} \leq w < 6$ m	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$
NOTE For example, for a carriageway width equal to 11m, $n_1 = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

(3) For variable carriageway widths, the number of notional lanes should be defined in accordance with the principles used for Table 4.1.

NOTE For example, the number of notional lanes will be :
- 1 where $w < 5{,}4$ m
- 2 where $5{,}4 \leq w < 9$ m
- 3 where $9 \text{ m} \leq w < 12$ m, etc.

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

(a) 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 ;

(b) the whole carriageway, central reservation included, should be divided into notional lanes if the parts are separated by a temporary road restraint system.

NOTE The rules given in 4.2.3(4) may be adjusted for the individual project, allowing for envisaged future modifications of the traffic lanes on the deck, *e.g.* for repair.

4.2.4 Location and numbering of the lanes for design

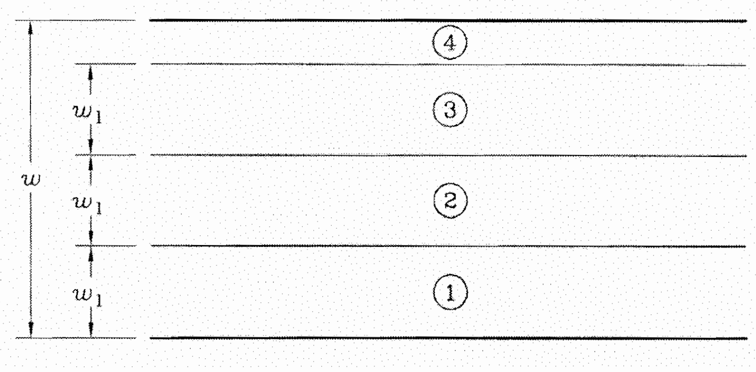
The location and numbering of the lanes should be determined in accordance with the following rules :

(1) The locations of notional lanes should not be necessarily related to their numbering.

(2) For each individual verification (*e.g.* for a verification of the ultimate limit state of resistance of a cross-section to bending), the number of lanes to be taken into account as loaded, their location on the carriageway and their numbering should be so chosen that the effects from the load models are the most adverse.

(3) For fatigue representative values and models, the location and the numbering of the lanes should be selected depending on the traffic to be expected in normal conditions.

(4) The lane giving the most unfavourable effect is numbered Lane Number 1, the lane giving the second most unfavourable effect is numbered Lane Number 2, etc. (see Figure 4.1).



Key
w Carriageway width
w_1 Notional lane width
1 Notional Lane Nr. 1
2 Notional Lane Nr. 2
3 Notional Lane Nr. 3
4 Remaining area

Figure 4.1 - Example of the Lane Numbering in the most general case

(5) Where the carriageway consists of two separate parts on the same deck, only one numbering should be used for the whole carriageway.

NOTE Hence, even if the carriageway is divided into two separate parts, there is only one Lane Number 1, which can [AC1) be considered alternatively (AC1] on the two parts.

(6) Where the carriageway consists of two separate parts on two independent decks, each part should be considered as a carriageway. Separate numbering should then be used for the design of each deck. If the two decks are supported by the same piers and/or abutments, there should be one numbering for the two parts together for the design of the piers and/or the abutments.

4.2.5 Application of the load models on the individual lanes

(1) For each individual verification, the load models, on each notional lane, should be applied on such a length and so longitudinally located that the most adverse effect is obtained, as far as this is compatible with the conditions of application defined below for each particular model.

(2) On the remaining area, the associated load model should be applied on such lengths and widths in order to obtain the most adverse effect, as far as this is compatible with particular conditions specified in 4.3.

(3) When relevant, the various load models should be combined together (see 4.5) and with models for pedestrian or cycle loads.

4.3 Vertical loads - Characteristic values

4.3.1 General and associated design situations

(1) Characteristic loads are intended for the determination of road traffic effects associated with ultimate limit state verifications and with particular serviceability verifications (see EN 1990 to EN 1999).

(2) The load models for vertical loads represent the following traffic effects :

a) Load Model 1 (LM1) : Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.

b) Load Model 2 (LM2) : A single axle load applied on specific tyre contact areas which covers the dynamic effects of the normal traffic on short structural members.

NOTE 1 As an order of magnitude, LM2 can be predominant in the range of loaded lengths up to 3m to 7m.

NOTE 2 The use of LM2 may be further defined in the National Annex.

c) Load Model 3 (LM3) : A set of assemblies of axle loads representing special vehicles (*e.g.* for industrial transport) which can travel on routes permitted for abnormal loads. It is intended for general and local verifications.

d) Load Model 4 (LM4) : A crowd loading, intended only for general verifications.

NOTE This crowd loading is particularly relevant for bridges located in or near towns if its effects are not covered by Load Model 1.

(3) Load Models 1, 2 and 3, where relevant, should be taken into account for any type of design situation (*e.g.* for transient situations during repair works).

(4) Load Model 4 should be used only for some transient design situations.

4.3.2 Load Model 1

(1) Load Model 1 consists of two partial systems :

(a) Double-axle concentrated loads (tandem system : TS), each axle having the following weight :

$$\alpha_Q Q_k \qquad (4.1)$$

where :

α_Q are adjustment factors.

– No more than one tandem system should be taken into account per notional lane.
– Only complete tandem systems should be taken into account.

- For the assessment of general effects, each tandem system should be assumed to travel centrally along the axes of notional lanes (see (5) below for local verifications and Figure 4.2b).
- Each axle of the tandem system should be taken into account with two identical wheels, the load per wheel being therefore equal to $0{,}5\alpha_Q Q_k$.
- The contact surface of each wheel should be taken as square and of side 0,40 m (see Figure 4.2b).

(b) Uniformly distributed loads (UDL system), having the following weight per square metre of notional lane :

$$\alpha_q q_k \tag{4.2}$$

where :

α_q are adjustment factors.

The uniformly distributed loads should be applied only in the unfavourable parts of the influence surface, longitudinally and transversally.

NOTE LM1 is intended to cover flowing, congested or traffic jam situations with a high percentage of heavy lorries. In general, when used with the basic values, it covers the effects of a special vehicle of 600 kN as defined in annex A.

(2) Load Model 1 should be applied on each notional lane and on the remaining areas. On notional lane Number *i*, the load magnitudes are referred to as $\alpha_{Qi} Q_{ik}$ and $\alpha_{qi} q_{ik}$ (see Table 4.2). On the remaining areas, the load magnitude is referred to as $\alpha_{qr} q_{rk}$.

(3) The values of adjustment factors α_{Qi}, α_{qi} and α_{qr} should be selected depending on the expected traffic and possibly on different classes of routes. In the absence of specification these factors should be taken equal to unity.

NOTE 1 The values of α_{Qi}, α_{qi} and α_{qr} factors are given in the National Annex. In all cases, for bridges without road signs restricting vehicle weights, the following minimum values are recommended :

$$\alpha_{Q1} \geq 0{,}8 \quad \text{and} \tag{4.3}$$

$$\text{for : } i \geq 2,\ \alpha_{qi} \geq 1 \text{ ; this restriction being not applicable to } \alpha_{qr}. \tag{4.4}$$

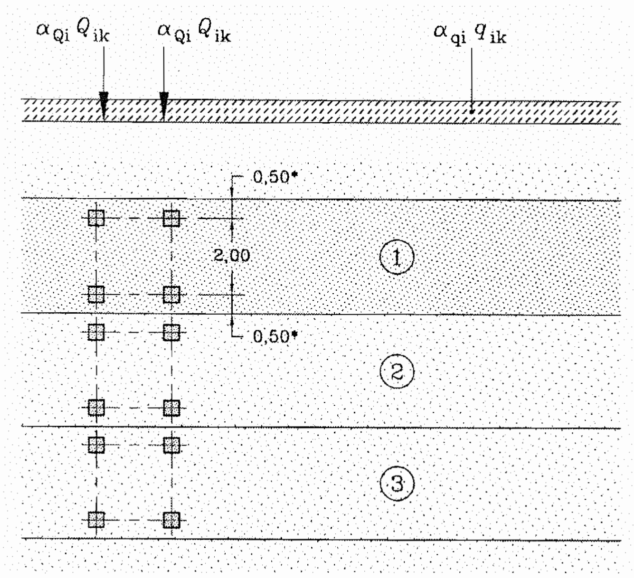
NOTE 2 Values of α factors may correspond, in the National Annex, to classes of traffic. When they are taken equal to 1, they correspond to a traffic for which a heavy industrial international traffic is expected, representing a large part of the total traffic of heavy vehicles. For more common traffic compositions (highways or motorways), a moderate reduction of α factors applied to tandems systems and the uniformly distributed loads on Lane 1 may be applied (10 to 20%).

(4) The characteristic values of Q_{ik} and q_{ik}, dynamic amplification included, should be taken from Table 4.2.

Table 4.2 - Load model 1 : characteristic values

Location	**Tandem system *TS***	***UDL* system**
	Axle loads Q_{ik} (kN)	[AC1] q_{ik} **(or** q_{rk}) (kN/m^2) [/AC1]
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

The details of Load Model 1 are illustrated in Figure 4.2a.



Key

(1) Lane Nr. 1 : Q_{1k} = 300 kN ; q_{1k} = 9 kN/m^2
(2) Lane Nr. 2 : Q_{2k} = 200 kN ; q_{2k} = 2,5 kN/m^2
(3) Lane Nr. 3 : Q_{3k} = 100 kN ; q_{3k} = 2,5 kN/m^2
[AC1] Tandem axle spacing = 1,2 m [/AC1]
* For w_l = 3,00 m

Figure 4.2a - Application of load Model 1

NOTE The application of 4.2.4-(2) and 4.3.2-(1) to (4) practically consists, for this model, of choosing the locations of the numbered lanes and the locations of the tandem systems (in most cases in the same cross-section). The length and width to be loaded by UDL are those of the relevant adverse parts of the influence surfaces.

(5) For local verifications, a tandem system should be applied at the most unfavourable location. Where two tandem systems on adjacent notional lanes are taken into account, they may be brought closer, with a distance between wheel axles not below 0,50 m (see Figure 4.2b).

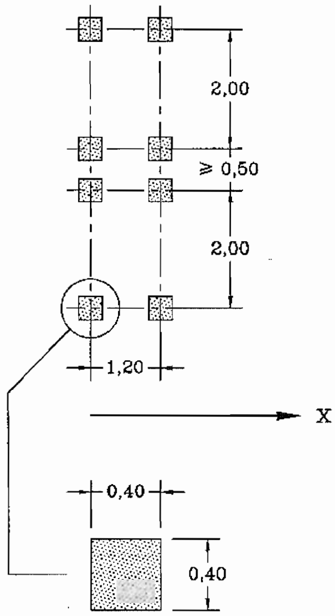


Figure 4.2b - Application of tandem systems for local verifications

(6) Where general and local effects can be calculated separately, the general effects may be calculated by using the following simplified alternative rules :

NOTE The National Annex may define the conditions of use of these alternative rules.

a) the second and third tandem systems are replaced by a second tandem system with axle weight equal to :

$$(200\ \alpha_{Q2} + 100\ \alpha_{Q3})\ \text{kN, or} \qquad (4.5)$$

b) for span lengths greater than 10 m, each tandem system are replaced in each lane by a one-axle concentrated load of weight equal to the total weight of the two axles.

NOTE In that case, the single axle weight is :

- 600 α_{Q1} kN on Lane Number 1
- 400 α_{Q2} kN on Lane Number 2
- 200 α_{Q3} kN on Lane Number 3

4.3.3 Load Model 2

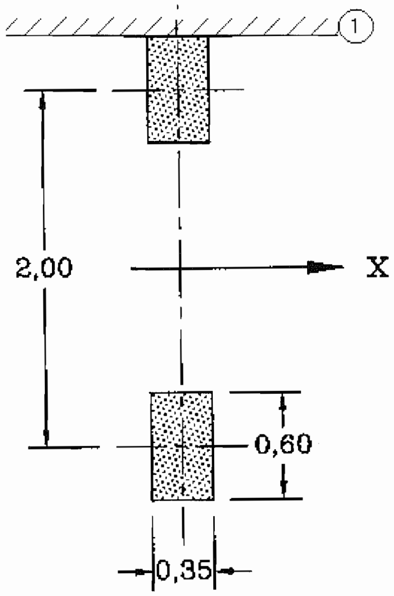
(1) Load Model 2 consists of a single axle load $\beta_Q Q_{ak}$ with Q_{ak} equal to 400 kN, dynamic amplification included, which should be applied at any location on the carriageway. However, when relevant, only one wheel of 200 β_Q (kN) may be taken into account.

(2) The value of β_Q should be specified.

NOTE The National Annex may give the value of β_Q. It is recommended that $\beta_Q = \alpha_{Q1}$.

(3) In the vicinity of expansion joints, an additional dynamic amplification factor equal to the value defined in 4.6.1(6) should be applied.

(4) The contact surface of each wheel should be taken into account as a rectangle of sides 0,35 m and 0,60 m (see Figure 4.3).



Key
X Bridge longitudinal axis direction
1 Kerb

Figure 4.3 - Load Model 2

NOTE 1 The contact areas of Load Models 1 and 2 are different, and correspond to different tyre models, arrangements and pressure distributions. The contact areas of Load Model 2, corresponding to twin tyres, are normally relevant for orthotropic decks.

NOTE 2 For simplicity, the National Annex may adopt the same square contact surface for the wheels of Load Models 1 and 2.

4.3.4 Load Model 3 (special vehicles)

(1) Where relevant, models of special vehicles should be defined and taken into account.

NOTE The National Annex may define Load Model 3 and its conditions of use. Annex A gives guidance on standard models and their conditions of application.

4.3.5 Load Model 4 (crowd loading)

(1) Crowd loading, if relevant, should be represented by a Load Model consisting of a uniformly distributed load (which includes dynamic amplification) equal to 5 kN/m^2.

NOTE The application of LM4 may be defined for the individual project.

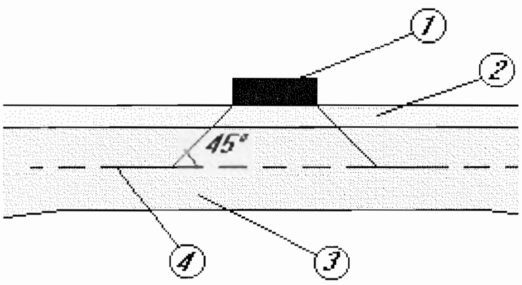
(2) Load Model 4 should be applied on the relevant parts of the length and width of the road bridge deck, the central reservation being included where relevant. This loading system, intended for general verifications, should be associated only with a transient design situation.

4.3.6 Dispersal of concentrated loads

(1) The various concentrated loads to be considered for local verifications, associated with Load Models 1 and 2, should be taken as uniformly distributed on their whole contact area.

(2) The dispersal through the pavement and concrete slabs should be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the level of the centroid of the slab (Figure 4.4).

NOTE In the case of dispersal through backfill or earth, see the NOTES in 4.9.1.



Key
1 Wheel contact pressure
2 Pavement
3 Concrete slab
4 Middle surface of concrete slab

Figure 4.4 - Dispersal of concentrated loads through pavement and a concrete slab

(3) The dispersal through the pavement and orthotropic decks should be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the level of the middle plane of the structural top plate (Figure 4.5).

NOTE The transverse distribution of the load among the ribs of the orthotropic deck is not considered here.

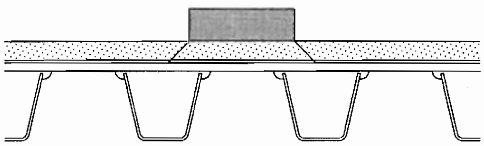


Figure 4.5 - Dispersal of concentrated loads through pavement and orthotropic decks

4.4 Horizontal forces - Characteristic values

4.4.1 Braking and acceleration forces

(1)P A braking force, Q_{lk}, shall be taken as a longitudinal force acting at the surfacing level of the carriageway.

(2) The characteristic value of Q_{lk}, limited to 900 kN for the total width of the bridge, should be calculated as a fraction of the total maximum vertical loads corresponding to the Load Model 1 likely to be applied on Lane Number 1, as follows :

$$Q_{lk} = 0{,}6\alpha_{Q1}(2Q_{1k}) + 0{,}10\alpha_{q1}q_{1k}w_l L$$
$$180\alpha_{Q1}\ (kN) \leq Q_{lk} \leq 900\ (kN) \qquad (4.6)$$

where :

L is the length of the deck or of the part of it under consideration.

NOTE 1 For example, $Q_{lk} = 360 + 2{,}7L$ ($\leq$ 900 kN) for a 3m wide lane and for a loaded length L>1,2 m, if α factors are equal to unity.

NOTE 2 The upper limit (900 kN) may be adjusted in the National Annex. The value 900 kN is normally intended to cover the maximum braking force of military vehicles according to STANAG[6].

(3) Horizontal forces associated with Load Model 3 should be defined where appropriate.

NOTE The National Annex may define horizontal forces associated with Load Model 3.

(4) This force should be taken into account as located along the axis of any lane. However, if the eccentricity effects are not significant, the force may be considered to be applied only along the carriageway axis, and uniformly distributed over the loaded length.

(5) Acceleration forces should be taken into account with the same magnitude as braking forces, but in the opposite direction.

NOTE Practically this means that Q_{lk} may be negative as well as positive.

(6) The horizontal force transmitted by expansion joints or applied to structural members that can be loaded by only one axle should be defined.

NOTE The National Annex may define the value for Q_{lk}. The recommended value is :

$$Q_{lk} = 0{,}6\alpha_{Q1}Q_{1k} \qquad (4.6a)$$

[6] STANAG : Military STANdardization AGreements (STANAG 2021)

4.4.2 Centrifugal and other transverse forces

(1) The centrifugal force Q_{tk} should be taken as a transverse force acting at the finished carriageway level and radially to the axis of the carriageway.

(2) The characteristic value of Q_{tk}, in which dynamic effects are included, should be taken from Table 4.3.

Table 4.3 - Characteristic values of centrifugal forces

$Q_{tk} = 0{,}2Q_v$ (kN)	if $r < 200$ m
$Q_{tk} = 40Q_v / r$ (kN)	if $200 \leq r \leq 1500$ m
$Q_{tk} = 0$	if $r > 1500$ m

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.* $\sum_i \alpha_{Qi}(2Q_{ik})$ (see Table 4.2).

(3) Q_{tk} should be assumed to act as a point load at any deck cross-section.

(4) Where relevant, lateral forces from skew braking or skidding should be taken into account. A transverse braking force, Q_{trk}, equal to 25% of the longitudinal braking or acceleration force Q_{lk}, should be considered to act simultaneously with Q_{lk} at the finished carriageway level.

NOTE The National Annex may define a minimum transverse loading. In most cases, forces resulting from wind effects and collisions on kerbs provide a sufficient transverse loading.

4.5 Groups of traffic loads on road bridges

4.5.1 Characteristic values of the multi-component action

(1) The simultaneity of the loading systems defined in 4.3.2 (Load Model 1), 4.3.3 (Load Model 2), 4.3.4 (Load Model 3), 4.3.5 (Load Model 4), 4.4 (horizontal forces) and the loads defined in section 5 for footways should be taken into account by considering the groups of loads defined in Table 4.4a. Each of these groups of loads, which are mutually exclusive, should be considered as defining a characteristic action for combination with non-traffic loads.

Table 4.4a - Assessment of groups of traffic loads (characteristic values of the multi-component action)

AC1

		CARRIAGEWAY						FOOTWAYS AND CYCLE TRACKS
Load type		**Vertical forces**				**Horizontal forces**		**Vertical forces only**
Reference		4.3.2	4.3.3	4.3.4	4.3.5	4.4.1	4.4.2	5.3.2-(1)
Load system		LM1 (TS and UDL systems)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces [a]	Centrifugal and transverse forces [a]	Uniformly Distributed load
Groups of Loads	gr1a	Characteristic values						Combination value [b]
	gr1b		Characteristic value					
	gr2	Frequent values				Characteristic value	Characteristic value	
	gr3 [d]							Characteristic value [c]
	gr4				Characteristic value			Characteristic value
	gr5	See annex A		Characteristic value				
(shaded)	Dominant component action (designated as component associated with the group)							

[a] May be defined in the National Annex (for the cases mentioned).
[b] May be defined in the National Annex. The recommended value is 3 kN/m^2.
[c] See 5.3.2.1-(2). One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.
[d] This group is irrelevant if gr4 is considered.

AC1

4.5.2 Other representative values of the multi-component action

(1) The frequent action should consist only of either the frequent values of LM1 or the frequent value of LM2, or the frequent values of loads on footways or cycle-tracks (taking the more unfavourable), without any accompanying component, as defined in Table 4.4b.

NOTE 1 For the individual components of the traffic action, these representative values are defined in EN 1990, A2.

NOTE 2 For quasi-permanent values (generally equal to zero), see EN 1990, A2.

NOTE 3 Where the National Annex refers to infrequent values of variable actions, the same rule as in 4.5.1 may be applied by replacing all characteristic values in Table 4.4 by infrequent values defined in EN 1990, A2, without modifying the other values mentioned in the Table. But the infrequent group gr2 is practically irrelevant for road bridges.

Table 4.4b - Assessment of groups of traffic loads (frequent values of the multi-component action)

		CARRIAGEWAY		FOOTWAYS AND CYCLE TRACKS
Load type		Vertical forces		
Reference		4.3.2	4.3.3	5.3.2(1)
Load system		LM1 (TS and UDL systems)	LM2 (single axle)	Uniformly distributed load
Groups of loads	gr1a	Frequent values		
	gr1b		Frequent value	
	gr3			Frequent value [a]

[a] One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.

4.5.3 Groups of loads in transient design situations

(1) The rules given in 4.5.1 and 4.5.2 are applicable with the following modifications given in 4.5.3(2).

(2) For verifications in transient design situations, the characteristic values associated with the tandem system should be taken equal to $0{,}8\alpha_{Qi}Q_{ik}$, and all other characteristic, frequent and quasi-permanent values and the horizontal forces are as specified for persistent design situations without any modification (*i.e.* they are not reduced proportionally to the weight of the tandems).

NOTE In transient design situations due to road or bridge maintenance, the traffic is commonly concentrated on smaller areas without being significantly reduced, and long lasting traffic jams are frequent. However, more reductions may be applied in cases where the heaviest lorries are diverted by appropriate measures.

4.6 Fatigue load models

4.6.1 General

(1) Traffic running on bridges produces a stress spectrum which may cause fatigue. The stress spectrum depends on the geometry of the vehicles, the axle loads, the vehicle spacing, the composition of the traffic and its dynamic effects.

(2) In the following, five fatigue load models of vertical forces are defined and given in 4.6.2 to 4.6.6.

NOTE 1 Horizontal forces may have to be taken into account simultaneously with vertical forces for the individual project : for example, centrifugal forces may occasionally need to be considered together with the vertical loads.

NOTE 2 The use of the various Fatigue Load Models is defined in EN 1992 to EN 1999 and further information is given as below :

a) Fatigue Load Models 1, 2 and 3 are intended to be used to determine the maximum and minimum stresses resulting from the possible load arrangements on the bridge of any of these models ; in many cases, only the algebraic difference between these stresses is used in EN1992 to EN1999.

b) Fatigue Load Models 4 and 5 are intended to be used to determine stress range spectra resulting from the passage of lorries on the bridge.

c) Fatigue Load Models 1 and 2 are intended to be used to check whether the fatigue life may be considered as unlimited when a constant stress amplitude fatigue limit is given. Therefore, they are appropriate for steel constructions and may be inappropriate for other materials. Fatigue Load Model 1 is generally conservative and covers multi-lane effects automatically. Fatigue Load Model 2 is more accurate than Fatigue Load Model 1 when the simultaneous presence of several lorries on the bridge can be neglected for fatigue verifications. If that is not the case, it should be used only if it is supplemented by additional data. The National Annex may give the conditions of use of fatigue load models 1 and 2.

d) Fatigue Load Models 3, 4 and 5 are intended to be used for fatigue life assessment by reference to fatigue strength curves defined in EN1992 to EN1999. They should not be used to check whether fatigue life can be considered as unlimited. For this reason, they are not numerically comparable to Fatigue Load Models 1 and 2. Fatigue Load Model 3 may also be used for the direct verification of designs by simplified methods in which the influence of the annual traffic volume and of some bridge dimensions is taken into account by a material-dependent adjustment factor λ_e.

e) Fatigue Load Model 4 is more accurate than Fatigue Load Model 3 for a variety of bridges and of the traffic when the simultaneous presence of several lorries on the bridge can be neglected. If that is not the case, it should be used only if it is supplemented by additional data, specified or as defined in the National Annex.

f) Fatigue Load Model 5 is the most general model, using actual traffic data.

NOTE 3 The load values given for Fatigue Load Models 1 to 3 are appropriate for typical heavy traffic on European main roads or motorways (traffic category Number 1 as defined in Table 4.5).

NOTE 4 The values of Fatigue Load Models 1 and 2 may be modified for the individual project or by the National Annex when considering other categories of traffic. In this case, the modifications made to both models should be proportional. For Fatigue Load Model 3 a modification depends on the verification procedure.

(3) A traffic category on a bridge should be defined, for fatigue verifications, at least, 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 (*i.e.* a traffic lane used predominantly by lorries).

NOTE 1 The traffic categories and values may be defined in the National Annex. Indicative values for N_{obs} are given in Table 4.5 for a slow lane when using Fatigue Load Models 3 and 4. On each fast lane (*i.e.* a traffic lane used predominantly by cars), additionally, 10% of N_{obs} may be taken into account.

Table 4.5(n) - Indicative number of heavy vehicles expected per year and per slow lane

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	$2,0 \times 10^6$
2	Roads and motorways with medium flow rates of lorries	$0,5 \times 10^6$
3	Main roads with low flow rates of lorries	$0,125 \times 10^6$
4	Local roads with low flow rates of lorries	$0,05 \times 10^6$

NOTE 2 Table 4.5 is not sufficient to characterise the traffic for fatigue verifications. Other parameters may have to be considered, for example :
- percentages of vehicle types (see, *e.g.*, Table 4.7), which depend on the "traffic type",
- parameters defining the distribution of the weight of vehicles or axles of each type.

NOTE 3 There is no general relation between traffic categories for fatigue verifications, and the loading classes and associated α factors mentioned in 4.2.2 and 4.3.2.

NOTE 4 Intermediate values of N_{obs} are not excluded, but are unlikely to have significant effect on the fatigue life.

(4) For the assessment of general action effects (*e.g.* in main girders) all fatigue load models should be placed centrally on the notional lanes defined in accordance with the principles and rules given in 4.2.4(2) and (3). The slow lanes should be identified in the design.

(5) For the assessment of local action effects (*e.g.* in slabs) the models should be centered on notional lanes assumed to be located anywhere on the carriageway. However, where the transverse location of the vehicles for Fatigue Load Models 3, 4 and 5 is significant for the studied effects (*e.g.* for orthotropic decks), a statistical distribution of this transverse location should be taken into account in accordance with Figure 4.6.

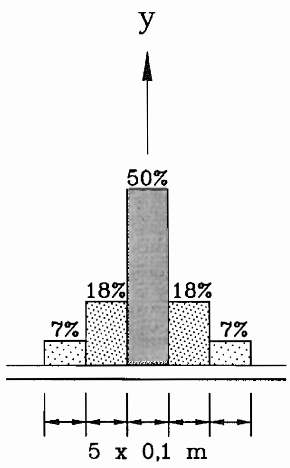


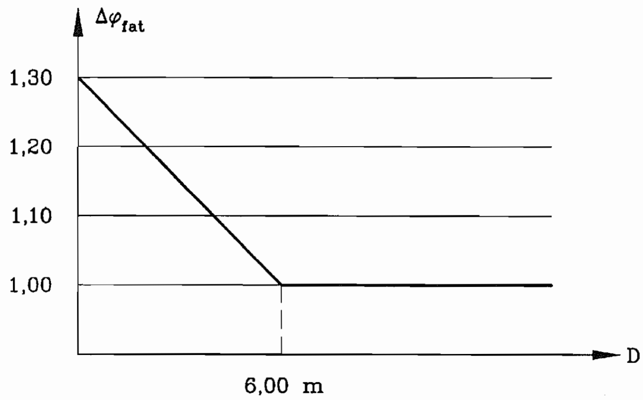
Figure 4.6 - Frequency distribution of transverse location of centre line of vehicle

(6) Fatigue Load Models 1 to 4 include dynamic load amplification appropriate for pavements of good quality (see annex B). An additional amplification factor $\Delta\varphi_{fat}$ should be taken into account near expansion joints and applied to all loads :

$$\Delta\varphi_{fat} = 1{,}30\left(1-\frac{D}{26}\right) \quad ; \quad \Delta\varphi_{fat} \geq 1 \qquad (4.7)$$

where :

D is the distance (m) of the cross-section under consideration from the expansion joint. See Figure 4.7.



Key
$\Delta\varphi_{fat}$: Additional amplification factor
D : Distance of the cross-section under consideration from the expansion joint

Figure 4.7 - Representation of the additional amplification factor

NOTE A conservative, often acceptable, simplification may consist of adopting $\Delta\varphi_{fat} = 1{,}3$ for any cross-section within 6m from the expansion joint. The dynamic additional amplification may be modified in the National Annex. Expression (4.7) is recommended.

4.6.2 Fatigue Load Model 1 (similar to LM1)

(1) Fatigue Load Model 1 has the configuration of the characteristic Load Model 1 defined in 4.3.2, with the values of the axle loads equal to $0{,}7Q_{\mathrm{ik}}$ and the values of the uniformly distributed loads equal to $0{,}3q_{\mathrm{ik}}$ and (unless otherwise specified) $0{,}3q_{\mathrm{rk}}$.

NOTE The load values for Fatigue Load Model 1 are similar to those defined for the Frequent Load Model. However adopting the Frequent Load Model without adjustment would have been excessively conservative by comparison with the other models, especially for large loaded areas. For individual projects, q_{rk} may be neglected.

(2) The maximum and minimum stresses ($\sigma_{\mathrm{FLM,max}}$ and $\sigma_{\mathrm{FLM,min}}$) should be determined from the possible load arrangements of the model on the bridge.

4.6.3 Fatigue Load Model 2 (set of "frequent" lorries)

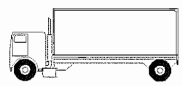
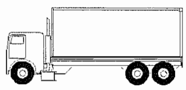
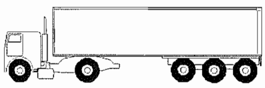
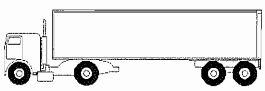
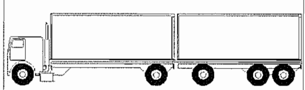
(1) Fatigue Load Model 2 consists of a set of idealised lorries, called "frequent" lorries, to be used as defined in (3) below.

(2) Each "frequent lorry" is defined by :
- the number of axles and the axle spacing (Table 4.6, columns 1+2),
- the frequent load of each axle (Table 4.6, column 3),
- the wheel contact areas and the transverse distance between wheels (column 4 of Table 4.6 and Table 4.8).

(3) The maximum and minimum stresses should be determined from the most severe effects of different lorries, separately considered, travelling alone along the appropriate lane.

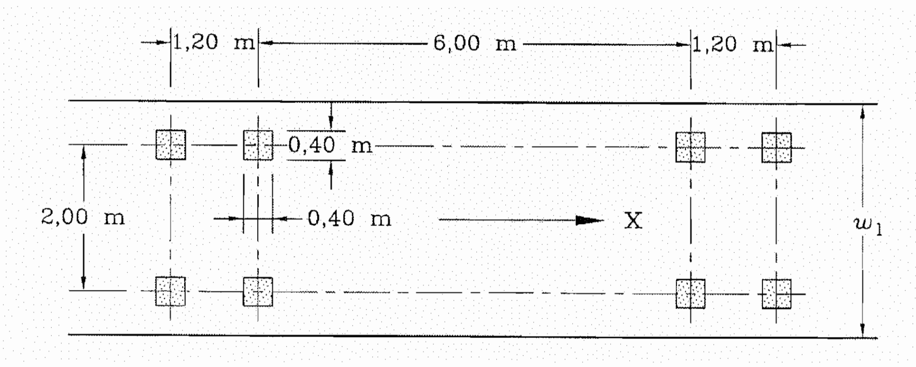
NOTE When some of these lorries are obviously the most critical, the others may be disregarded.

Table 4.6 - Set of "frequent" lorries

1	2	3	4
LORRY SILHOUETTE	**Axle spacing** (m)	**Frequent axle loads** (kN)	**Wheel type** (see Table 4.8)
	4,5	90 190	A B
	4,20 1,30	80 140 140	A B B
	3,20 5,20 1,30 1,30	90 180 120 120 120	A B C C C
	3,40 6,00 1,80	90 190 140 140	A B B B
	4,80 3,60 4,40 1,30	90 180 120 110 110	A B C C C

4.6.4 Fatigue Load Model 3 (single vehicle model)

(1) This model consists of four axles, each of them having two identical wheels. The geometry is shown in Figure 4.8. The weight of each axle is equal to 120 kN, and the contact surface of each wheel is a square of side 0,40 m.



Key
w_l : Lane width
X : Bridge longitudinal axis

Figure 4.8 - Fatigue Load Model 3

(2) The maximum and minimum stresses and the stress ranges for each cycle of stress fluctuation, *i.e.* their algebraic difference, resulting from the transit of the model along the bridge should be calculated.

(3) Where relevant, two vehicles in the same lane should be taken into account.

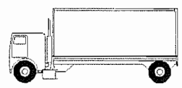
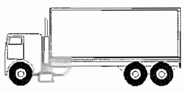
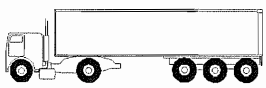
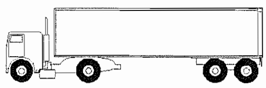
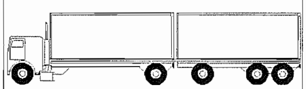
NOTE The conditions of application of this rule may be defined in the National Annex or for the individual project. Possible recommended conditions are given hereafter :

- one vehicle is as defined in (1) above ;
- the geometry of the second vehicle is as defined in (1) above and the weight of each axle is equal to 36 kN (instead of 120 kN) ;
- the distance between the two vehicles, measured from centre to centre of vehicles, is not less than 40 m.

4.6.5 Fatigue Load Model 4 (set of "standard" lorries)

(1) Fatigue Load Model 4 consists of sets of standard lorries which together produce effects equivalent to those of typical traffic on European roads. A set of lorries appropriate to the traffic mixes predicted for the route as defined in Tables 4.7 and 4.8 should be taken into account.

Table 4.7 - Set of equivalent lorries

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
	4,20 1,30	70 120 120	5,0	10,0	5,0	A B 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
	3,40 6,00 1,80	70 140 90 90	15,0	15,0	5,0	A B B B
	4,80 3,60 4,40 1,30	70 130 90 80 80	10,0	5,0	5,0	A B C C C

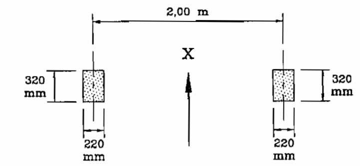
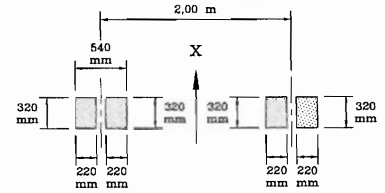
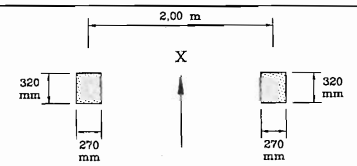
NOTE 1 This model, based on five standard lorries, simulates traffic which is deemed to produce fatigue damage equivalent to that due to actual traffic of the corresponding category defined in Table 4.5.

NOTE 2 Other standard lorries and lorry percentages may be defined for the individual project or in the National Annex.

NOTE 3 For the selection of a traffic type, it may broadly be considered that :
- "Long distance" means hundreds of kilometres,
- "Medium distance" means 50 to 100 km,
- "Local traffic" means distances less than 50 km.

In reality, mixture of traffic types may occur.

Table 4.8 - Definition of wheels and axles

WHEEL/ AXLE TYPE	GEOMETRICAL DEFINITION
A	
B	
C	

(2) Each standard lorry is defined by :

- the number of axles and the axle spacing (Table 4.7, columns 1+2),
- the equivalent load of each axle (Table 4.7, column 3)
- the wheel contact areas and the transverse distances between wheels, in accordance with column 7 of Table 4.7. and Table 4.8.

(3) The calculations should be based on the following procedure :

- the percentage of each standard lorry in the traffic flow should be selected from Table 4.7. columns 4, 5 or 6 as relevant ;
- the total number of vehicles per year to be considered for the whole carriageway $\sum N_{obs}$ should be defined ;

NOTE Recommended values are given in Table 4.5.

- each standard lorry is considered to cross the bridge in the absence of any other vehicle.

(4) The stress range spectrum and the corresponding number of cycles from each fluctuation in stress during the passage of individual lorries on the bridge should be the Rainflow or the Reservoir counting method.

NOTE For verification rules, see EN 1992 to EN 1999

4.6.6 Fatigue Load Model 5 (based on recorded road traffic data)

(1) Fatigue Load Model 5 consists of the direct application of recorded traffic data, supplemented, if relevant, by appropriate statistical and projected extrapolations.

NOTE For the use of this model, see the National Annex. Guidance for a complete specification and the application of such a model is given in annex B.

4.7 Actions for accidental design situations

4.7.1 General

(1)P Loads due to road vehicles in accidental design situations shall be taken into account where relevant, resulting from :

- vehicle collision with bridge piers, soffit of bridge or decks,
- the presence of heavy wheels or vehicle on footways (effects of heavy wheels or vehicle on footways shall be considered for all road bridges where footways are not protected by an effective rigid road restraint system),
- vehicle collision with kerbs, vehicle parapets and structural components (effects of vehicle collision with vehicle parapets and safety barriers shall be considered for all road bridges where such road restraint systems are provided on the bridge deck ; effects of vehicle collision with kerbs shall be considered in all cases).

4.7.2 Collision forces from vehicles under the bridge

NOTE See 5.6.2 and 6.7.2, and EN 1990, A2.

4.7.2.1 Collision forces on piers and other supporting members

(1) Forces due to the collision of abnormal height or aberrant road vehicles with piers or with the supporting members of a bridge should be taken into account.

NOTE The National Annex may define :

- rules to protect the bridge from vehicular collision forces,
- when vehicular collision forces are to be taken into account (*e.g.* with reference to a safety distance between piers and the edge of the carriageway),
- the magnitude and location of vehicular collision forces,
- and also the limit states to be considered.

For stiff piers the following minimum values are recommended :

a) Impact force : 1000 kN in the direction of vehicle travel or 500 kN perpendicular to that direction ;
b) Height above the level of adjacent ground surface : 1,25 m.

See also EN 1991-1-7.

4.7.2.2 Collision forces on decks

(1) If relevant the vehicle collision force should be specified.

NOTE 1 The National Annex may define the collision force on decks, possibly in relation to vertical clearance and other forms of protection. See EN 1991-1-7.

NOTE 2 Collision loads on bridge decks and other structural components over roads may vary widely depending on structural and non-structural parameters, and their conditions of applicability. The possibility of collision by vehicles having an abnormal or illegal height may have to be envisaged, as well as a crane swinging up while a vehicle is moving. Preventive or protective measures may be introduced as an alternative to designing for collision forces.

4.7.3 Actions from vehicles on the bridge

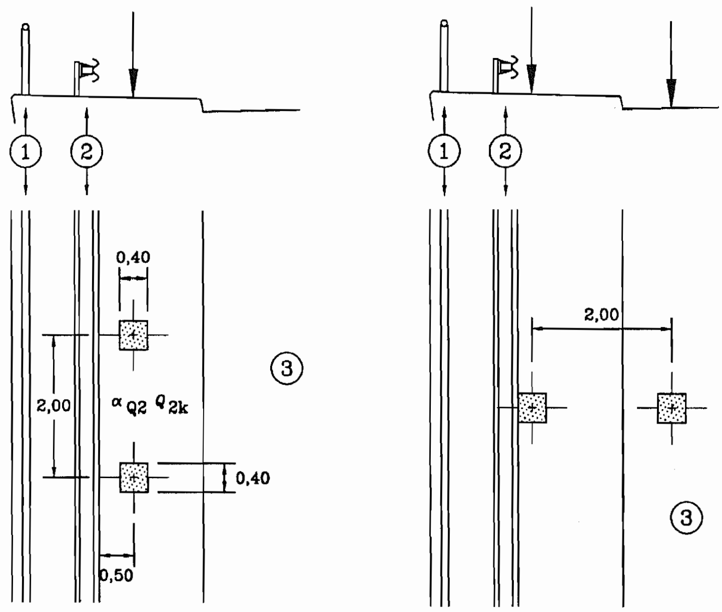
4.7.3.1 Vehicle on footways and cycle tracks on road bridges

(1) If a safety barrier of an appropriate containment level is provided, wheel or vehicle loading beyond this protection need not be taken into account.

NOTE Containment levels for safety barriers are defined in EN 1317-2.

(2) Where the protection mentioned in (1) is provided, one accidental axle load corresponding to $\alpha_{Q2}Q_{2k}$ (see 4.3.2) should be so placed and oriented on the unprotected parts of the deck so as to give the most adverse effect adjacent to the safety barrier as shown, for example, in Figure 4.9. This axle load should not be taken into account simultaneously with any other variable load on the deck. A single wheel alone should be taken into account if geometrical constraints make a two-wheel arrangement impossible.

Beyond the vehicle restraint system, the characteristic variable concentrated load defined in 5.3.2.2 should be applied, if relevant, separately from the accidental load.



Key
(1) Pedestrian parapet (or vehicle parapet if a safety barrier is not provided)
(2) Safety barrier
(3) Carriageway

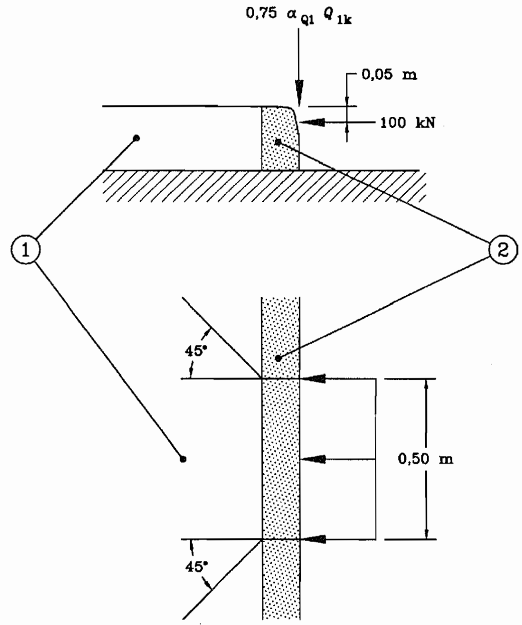
Figure 4.9 - Examples showing locations of loads from vehicles on footways and cycle tracks of road bridges

(3) In the absence of the protection mentioned in (1), the rules given in (2) are applicable up to the edge of the deck where a vehicle parapet is provided.

4.7.3.2 Collision forces on kerbs

(1) The action from vehicle collision with kerbs or pavement upstands should be taken as a lateral force equal to 100 kN acting at a depth of 0,05 m below the top of the kerb.

This force should be considered as acting on a line 0,5 m long and is transmitted by the kerbs to the structural members supporting them. In rigid structural members, the load should be assumed to have an angle of dispersal of 45°. When unfavourable, a vertical traffic load acting simultaneously with the collision force equal to $0{,}75\alpha_{Q1}Q_{1k}$ (see Figure 4.10) should be taken into account.



Key
(1) Footway
(2) Kerb

Figure 4.10 - Definition of vehicle collision forces on kerbs

4.7.3.3 Collision forces on vehicle restraint systems

(1) For structural design, horizontal and vertical forces transferred to the bridge deck by vehicle restraint systems should be taken into account.

NOTE 1 The National Annex may define and select classes of collision forces and associated conditions of application. In the following, 4 recommended classes of values for the transferred horizontal force are given :

Table 4.9 (n) – Recommended classes for the horizontal force transferred by vehicle restraint systems

Recommended class	Horizontal force (kN)
A	100
B	200
C	400
D	600

The horizontal force, acting transversely, may be applied 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.

NOTE 2 The values of the horizontal forces given for the classes A to D derive from measurements during collision tests on real vehicle restraint systems used for bridges. There is no direct correlation between these values and performance classes of vehicle restraint systems. The proposed values depend rather on the stiffness of the connection between the vehicle restraint system and the kerb or the part of the bridge to which it is connected. A very strong connection leads to the horizontal force given for class D. The lowest horizontal force derives from measurements for a vehicle restraint system with a weak connection. Such systems are frequently used for a steel vehicle restraint systems according to a performance class H2 according to EN 1317-2. A very weak connection may lead to the horizontal force given for class A.

NOTE 3 The vertical force acting simultaneously with the horizontal collision force may be defined in the National Annex. The recommended values may be taken equal to $0{,}75\alpha_{Q1}Q_{1k}$. The calculations taking account of horizontal and vertical forces may be replaced, when possible, by detailing measures (for example, design of reinforcement).

(2) The structure supporting the vehicle parapet should be designed to sustain locally an accidental load effect corresponding to at least 1,25 times the characteristic local resistance of vehicle parapet (*e.g.* resistance of the connection of the parapet to the structure) and need not be combined with any other variable load.

NOTE This design load effect may be defined in the National Annex. The value given in this clause (1,25) is a recommended minimum value.

4.7.3.4 Collision forces on structural members

(1) The vehicle collision forces on unprotected structural members above or beside the carriageway levels should be taken into account.

NOTE These forces may the defined in the National Annex. It is recommended that they may be the same as defined in 4.7.2.1(1), 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.

(2) These forces should not be considered to act simultaneously with any variable load.

NOTE For some intermediate members where damage to one of which would not cause collapse (*e.g.* hangers or stays), smaller forces may be defined for the individual project.

4.8 Actions on pedestrian parapets

(1) For structural design, forces that are transferred to the bridge deck by pedestrian parapets should be taken into account as variable loads and defined, depending on the selected loading class of the parapet.

NOTE 1 For loading classes of pedestrian parapets, see EN 1317-6. For bridges, class C is the recommended minimum class.

NOTE 2 The forces transferred to the bridge deck by pedestrian parapets may be defined with their classification for the individual project or in the National Annex in accordance with EN 1317-6. A line force of 1,0 kN/m acting, as a variable load, horizontally or vertically on the top of the parapet is a recommended minimum value for footways or footbridges. For service side paths, the recommended minimum value is 0,8 kN/m. Exceptional and accidental cases are not covered by these recommended minimum values.

(2) For the design of the supporting structure, if pedestrian parapets are adequately protected against vehicle collision, the horizontal actions should be considered as simultaneous with the uniformly distributed vertical loads defined in 5.3.2.1.

NOTE Pedestrian parapets can be considered as adequately protected only if the protection satisfies the requirements for the individual project.

(3) Where pedestrian parapets cannot be considered as adequately protected against vehicle collisions, the supporting structure should be designed to sustain an accidental load effect corresponding to 1,25 times the characteristic resistance of the parapet, exclusive of any variable load.

NOTE This design load effect may be defined in the National Annex. The value given in this clause (1,25) is recommended.

4.9 Load models for abutments and walls adjacent to bridges

4.9.1 Vertical loads

(1) The carriageway located behind abutments, wing walls, side walls and other parts of the bridge in contact with earth, should be loaded with appropriate models.

NOTE 1 These appropriate load models may be defined in the National Annex. The use of Load Model 1, defined in 4.3.2, is recommended, but, for simplicity, the tandem system loads may be replaced by an equivalent uniformly distributed load, noted q_{eq}, spread over an appropriate relevant rectangular surface depending on the dispersal of the loads through the backfill or earth.

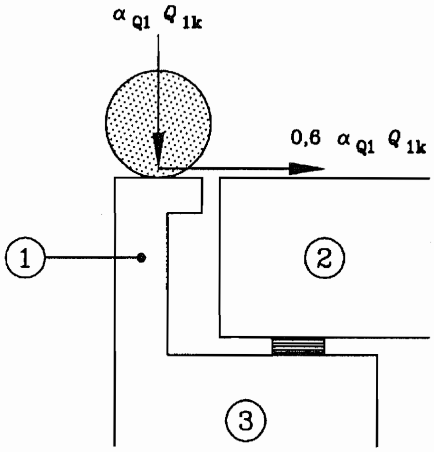
NOTE 2 For the dispersal of the loads through the backfill or earth, see EN 1997. In the absence of any other rule, if the backfill is properly consolidated, the recommended value of the dispersal angle [AC1) from the vertical (AC1] is equal to 30°. With such a value, the surface on which q_{eq} is applied may be taken as a rectangular surface 3 m wide and 2,20 m long .

(2) Representative values of the load model other than the characteristic values should not be considered.

4.9.2 Horizontal force

(1) No horizontal force should be taken into account at the surfacing level of the carriageway over the backfill.

(2) For the design of abutment upstand walls (see Figure 4.11), a longitudinal braking force should be taken into account with a characteristic value equal to $0{,}6\alpha_{Q1}Q_{1k}$, acting simultaneously with the $\alpha_{Q1}Q_{1k}$ axle loading of Load Model Number 1 and with the earth pressure from the backfill. The backfill should be assumed not to be loaded simultaneously.



Key
(1) Upstand wall
(2) Bridge deck
(3) Abutment

Figure 4.11 - Definition of loads on upstand walls

Section 5 Actions on footways, cycle tracks and footbridges

5.1 Field of application

(1) Load models defined in this section are applicable to footways, cycle tracks and footbridges.

(2) The uniformly distributed load q_{fk} (defined in 5.3.2.1) and the concentrated load Q_{fwk} (defined in 5.3.2.2) should be used for road and railway bridges as well as for footbridges, where relevant (see 4.5, 4.7.3 and 6.3.6.2(1)). All other variable actions and actions for accidental design situations defined in this section are intended only for footbridges.

NOTE 1 For loads on access steps, see 6.3 in EN 1991-1-1.

NOTE 2 For large footbridges (for example more than 6 m width) load models defined in this section may not be appropriate and then complementary load models, with associated combination rules, may have to be defined for the individual project. Indeed, various human activities may take place on wide footbridges.

(3) Models and representative values given in this section should be used for serviceability and ultimate limit state calculations excluding fatigue limit states.

(4) For calculations relating to the vibration of pedestrian bridges and based on dynamic analysis, see 5.7. For all other calculations of load effects to be performed for any bridge type, the models and values given in this section include the dynamic amplification effects, and the variable actions should be treated as static.

(5) The effects of loads on construction sites are not intended to be covered by the load models given in this section and should be separately specified, where relevant.

5.2 Representation of actions

5.2.1 Models of the loads

(1) The imposed loads defined in this section result from pedestrian and cycle traffic, minor common construction and maintenance loads (*e.g.* service vehicles), and accidental situations. These loads give rise to vertical and horizontal, static and dynamic forces.

NOTE 1 Loads due to cycle traffic are generally much lower than those due to pedestrian traffic, and the values given in this section are based on the frequent or occasional presence of pedestrians on cycle lanes. Special consideration may need to be given to loads due to horses or cattle for individual projects.

NOTE 2 The load models defined in this section do not describe actual loads. They have been selected so that their effects (with dynamic amplification included where mentioned) represent the effects of actual traffic.

(2) Actions for accidental design situations due to collision should be represented by static equivalent loads.

5.2.2 Loading classes

(1) Loads on footbridges may differ depending on their location and on the possible traffic flow of some vehicles. These factors are mutually independent and are considered in various clauses given below. Therefore no general classification of these bridges needs to be defined.

5.2.3 Application of the load models

(1) The same models, service vehicle excepted (see 5.3.2.3), should be used for pedestrian and cycle traffic on footbridges, on the areas of the deck of road bridges limited by pedestrian parapets and not included in the carriageway as defined in 1.4.2 (footways as defined in this Part of EN 1991) and on the footpaths of railway bridges.

(2) Other appropriate models should be defined for inspection gangways within the bridges and for platforms on railway bridges.

NOTE Such models can be defined in the National Annex or for the individual project. The recommended models, to be used separately in order to get the most unfavourable effects, are an uniformly distributed load of 2 kN/m^2 and a concentrated load of 3 kN applicable to a square surface of 0,20×0,20 m^2.

(3) For each individual application, the models of vertical loads should be applied anywhere within the relevant areas so that the most adverse effect is obtained.

NOTE In other terms, these actions are free actions.

5.3 Static models for vertical loads - characteristic values

5.3.1 General

(1) Characteristic loads are intended for the determination of pedestrian or cycle-track static load effects associated with ultimate limit-states verifications and particular serviceability verifications.

(2) Three models, mutually exclusive, should be taken into account, as relevant. They consist of :

- a uniformly distributed load, q_{fk}
- a concentrated load Q_{fwk} , and
- loads representing service vehicles, Q_{serv} .

(3) The characteristic values of these load models should be used for both persistent and transient design situations.

5.3.2 Load Models

5.3.2.1 Uniformly distributed load

(1) For road bridges supporting footways or cycle tracks, a uniformly distributed load q_{fk} should be defined (Figure 5.1).

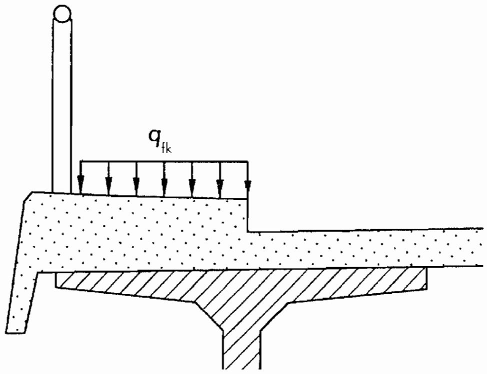


Figure 5.1 - Characteristic load on a footway (or cycle track)

NOTE The characteristic value q_{fk} may be defined in the National Annex or for the individual project. The recommended value is $q_{fk} = 5$ kN/m^2.

(2) For the design of footbridges, a uniformly distributed load q_{fk} should be defined and applied only in the unfavourable parts of the influence surface, longitudinally and transversally.

NOTE Load Model 4 (crowd loading) defined in 4.3.5, corresponding to $q_{fk} = 5$ kN/m^2, may be specified to cover the static effects of a continuous dense crowd where such a risk exists. Where the application of Load Model 4 defined in 4.3.5 is not required for footbridges, the recommended value for q_{fk} is :

$$q_{fk} = 2{,}0 + \frac{120}{L+30} \quad \text{kN/m}^2$$

$$q_{fk} \geq 2{,}5 \text{ kN/m}^2 ; \; q_{fk} \leq 5{,}0 \text{ kN/m}^2 \qquad (5.1)$$

where :

L is the loaded length in [m].

5.3.2.2 Concentrated load

(1) The characteristic value of the concentrated load Q_{fwk} should be taken equal to 10 kN acting on a square surface of sides 0,10 m.

NOTE The characteristic value of the load as well as the dimensions may be adjusted in the National Annex. The values in this clause are recommended.

(2) Where, in a verification, general and local effects can be distinguished, the concentrated load should be taken into account only for local effects.

(3) If, for a footbridge, a service vehicle, as mentioned in 5.3.2.3 is specified, Q_{fwk} should not be considered.

5.3.2.3 Service vehicle

(1)P When service vehicles are to be carried on a footbridge or footway, one service vehicle Q_{serv} shall be taken into account.

NOTE 1 This vehicle may be a vehicle for maintenance, emergencies (*e.g.* ambulance, fire) or other services. The characteristics of this vehicle (axle weight and spacing, contact area of wheels), 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 and if no permanent obstacle prevents a vehicle being driven onto the bridge deck, the use of the vehicle defined in 5.6.3 as the service vehicle (characteristic load) is recommended ; in this case, there will be no need to apply 5.6.3, *i.e.* to consider the same vehicle as accidental.

NOTE 2 Service vehicle needs not be considered if permanent provisions are made to prevent access of all vehicles to the footbridge.

NOTE 3 Several service vehicles, mutually exclusive, may have to be taken into account and may be defined for the individual project.

5.4 Static model for horizontal forces - Characteristic values

(1) For footbridges only, a horizontal force Q_{flk} should be taken into account, acting along the bridge deck axis at the pavement level.

(2) The characteristic value of the horizontal force should be taken equal to the greater of the following two values :
- 10 per cent of the total load corresponding to the uniformly distributed load (5.3.2.1),
- 60 per cent of the total weight of the service vehicle, if relevant (5.3.2.3-(1)P).

NOTE The characteristic value of the horizontal force may be defined in the National Annex or for the individual project. The values in this clause are recommended.

(3) The horizontal force is considered as acting simultaneously with the corresponding vertical load, and in no case with the concentrated load Q_{fwk} .

NOTE This force is normally sufficient to ensure the horizontal longitudinal stability of footbridges. It does not ensure horizontal transverse stability, which should be ensured by considering other actions or by appropriate design measures.

5.5 Groups of traffic loads on footbridges

(1)When relevant, the vertical loads and horizontal forces due to traffic should be taken into account by considering groups of loads defined in Table 5.1. Each of these groups of loads, which are mutually exclusive, should be considered as defining a characteristic action for combination with non–traffic loads.

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}

(2) For any combination of traffic loads together with actions specified in other Parts of EN 1991, any such group should be considered as one action.

NOTE For the individual components of the traffic loads on footbridges, the other representative values are defined in EN 1990, A2.

5.6 Actions for accidental design situations for footbridges

5.6.1 General

(1) Such actions are due to :

– road traffic under the bridge (*i.e.* collision) or

– the accidental presence of a heavy vehicle on the bridge.

NOTE Other collision forces (see 2.3) may be defined for the individual project or in the National Annex.

5.6.2 Collision forces from road vehicles under the bridge

(1) The measures to protect a footbridge should be defined.

NOTE Footbridges (piers and decks) are generally much more sensitive to collision forces than road bridges. Designing them for the same collision load may be unrealistic. The most effective way to take collision into account generally consists of protecting the footbridges :
– by road restraint systems at appropriate distances before piers,
– by a higher clearance than for neighbouring road or railway bridges over the same road in the absence of intermediate access to the road.

5.6.2.1 Collision forces on piers

(1) Forces due to the collision of abnormal height or aberrant road vehicles with piers or with the supporting members of a footbridge or ramps or stairs should be taken into account.

NOTE The National Annex may define :
– rules to protect the bridge from vehicular collision forces,
– when vehicular collision forces are to be taken into account (*e.g.* with reference to a safety distance between piers and the edge of the carriageway),
– the magnitude and location of vehicular collision forces,

– and also the limit states to be considered.

For stiff piers the following minimum values are recommended :

a) Impact force : 1000 kN in the direction of vehicle travel or 500 kN perpendicular to that direction ;

b) Height above the level of adjacent ground surface : 1,25 m.

See also EN 1991-1-7.

5.6.2.2 Collision forces on decks

(1) An adequate vertical clearance between the ground surface and the soffit of the deck above should be ensured in the design, when relevant.

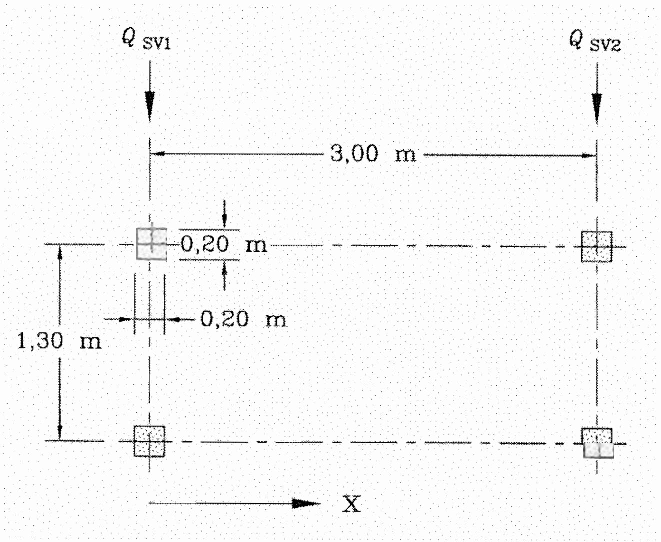
NOTE 1 The National Annex or the individual project may define collision forces depending on the vertical clearance. See also EN 1991-1-7.

NOTE 2 The possibility of collision by vehicles having an abnormal or illegal height may have to be taken into account.

5.6.3 Accidental presence of vehicles on the bridge

(1)P If no permanent obstacle prevents a vehicle from being driven onto the bridge deck, the accidental presence of a vehicle on the bridge deck shall be taken into account.

(2) For such a situation, the following load model should be used, consisting of a two-axle load group of 80 and 40 kN, separated by a wheel base of 3 m (Figure 5.2), with a track (wheel-centre to wheel-centre) of 1,3 m and square contact areas of side 0,2m at coating level. The braking force associated with the load model should be 60% of the vertical load.



Key
x : Bridge axis direction
Q_{sv1} = 80 kN
Q_{sv2} = 40 kN

Figure 5.2 - Accidental loading

NOTE 1 See the note in 5.3.2.3-(1)P.

NOTE 2 If relevant, other characteristics of the load model may be defined in the National Annex or for the individual project. The model defined in this clause is recommended.

(3) No variable action should be taken into account simultaneously with the load model defined in 5.6.3(2).

5.7 Dynamic models of pedestrian loads

(1) Depending on the dynamic characteristics of the structure, the relevant natural frequencies (corresponding to vertical, horizontal, torsional vibrations) of the main structure of the bridge deck should be determined from an appropriate structural model.

NOTE Vibrations of footbridges may have various origins, *e.g.* pedestrians, who can walk, run, jump or dance, wind, vandals, etc.

(2) Forces exerted by pedestrians with a frequency identical to one of the natural frequencies of the bridge can result into resonance and need to be taken into account for limit state verifications in relation with vibrations.

NOTE Effects of pedestrian traffic on a footbridge depend on various factors as, for example, the number and location of people likely to be simultaneously on the bridge, and also on external circumstances, more or less linked to the location of the bridge. In the absence of significant response of the bridge, a pedestrian normally walking exerts on it the following simultaneous periodic forces :
– in the vertical direction, with a frequency range of between 1 and 3 Hz, and
– in the horizontal direction, with a frequency range of between 0,5 and 1,5 Hz.
Groups of joggers may cross a footbridge with a frequency of 3 Hz.

(3) Appropriate dynamic models of pedestrian loads and comfort criteria should be defined.

NOTE The dynamic models of pedestrian loads and associated comfort criteria may be defined in the National Annex or for the individual project. See also EN 1990, A2.

5.8 Actions on parapets

(1) For footbridges, pedestrian parapets should be designed in accordance with rules given in 4.8.

5.9 Load model for abutments and walls adjacent to bridges

(1) The area external to a carriageway and located behind abutments, wing walls, side walls and other parts of the bridge in contact with earth, should be loaded with a uniformly distributed vertical load of 5 kN/m^2.

NOTE 1 This load does not cover the effects of heavy construction vehicles and other lorries commonly used for the placing of the backfill.

NOTE 2 The characteristic value may be adjusted for the individual project.

Section 6 Rail traffic actions and other actions specifically for railway bridges

6.1 Field of application

(1)P This section applies to rail traffic on the standard track gauge and wide track gauge European mainline network.

(2) The load models defined in this section do not describe actual loads. They have been selected so that their effects, with dynamic enhancements taken into account separately, represent the effects of service traffic. Where traffic outside the scope of the load models specified in this Part needs to be considered, then alternative load models, with associated combination rules, should be specified.

NOTE The alternative load models with associated combination rules may be defined in the National Annex or for the individual project.

(3)P This section is not applicable for actions due to:
- narrow-gauge railways,
- tramways and other light railways,
- preservation railways,
- rack and pinion railways,
- funicular railways.

The loading and characteristic values of actions for these types of railways should be specified.

NOTE The loading and characteristic values of actions for these types of railways may be defined in the National Annex or for the individual project.

(4) Requirements are specified in EN 1990 A2 for the limits of deformation of structures carrying rail traffic to maintain the safety of operations and to ensure the comfort of passengers etc..

(5) Three standard mixes of rail traffic are given as a basis for calculating the fatigue life of structures (see annex D).

(6) The self-weight of non-structural elements includes the weight of elements such as, for example, noise and safety barriers, signals, ducts, cables and overhead line equipment (except the forces due to the tension of the contact wire etc.).

(7) The design should pay special attention to temporary bridges because of the flexibility of some types of temporary structures. The loading and requirements for the design of temporary bridges should be specified.

NOTE The loading requirements for the design of temporary railway bridges, which may generally be based on this document, may be specified in the National Annex or for the individual project. Special requirements may also be given in the National Annex or for the individual project for temporary bridges depending upon the conditions in which they are used (*e.g.* special requirements are needed for skew bridges).

6.2 Representation of actions – nature of rail traffic loads

(1) General rules are given for the calculation of the associated dynamic effects, centrifugal forces, nosing force, traction and braking forces and aerodynamic actions due to passing rail traffic.

(2) Actions due to railway operations are given for:
- vertical loads: Load Models 71, SW (SW/0 and SW/2), “unloaded train” and HSLM (6.3 and 6.4.6.1.1),
- vertical loading for earthworks (6.3.6.4),
- dynamic effects (6.4),
- centrifugal forces (6.5.1),
- nosing force (6.5.2),
- traction and braking forces (6.5.3),
- aerodynamic actions from passing trains (6.6),
- actions due to overhead line equipment and other railway infrastructure and equipment (6.7.3).

NOTE Guidance is given on the evaluation of the combined response of structure and track to variable actions (6.5.4).

(3) Derailment actions for Accidental Design Situations are given for:
- the effect of rail traffic derailment on a structure carrying rail traffic (6.7.1).

6.3 Vertical loads - Characteristic values (static effects) and eccentricity and distribution of loading

6.3.1 General

(1) Rail traffic actions are defined by means of load models. Five models of railway loading are given:
- Load Model 71 (and Load Model SW/0 for continuous bridges) to represent normal rail traffic on mainline railways,
- Load Model SW/2 to represent heavy loads,
- Load Model HSLM to represent the loading from passenger trains at speeds exceeding 200 km/h,
- Load Model “unloaded train” to represent the effect of an unloaded train.

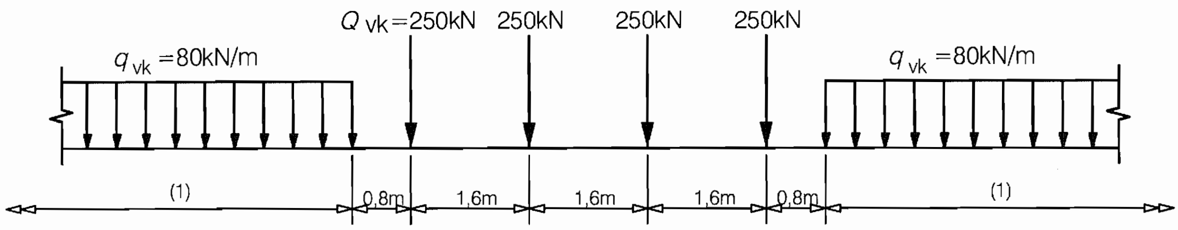
NOTE Requirements for the application of load models are given in 6.8.1.

(2) Provision is made for varying the specified loading to allow for differences in the nature, volume and maximum weight of rail traffic on different railways, as well as different qualities of track.

6.3.2 Load Model 71

(1) Load Model 71 represents the static effect of vertical loading due to normal rail traffic.

(2)P The load arrangement and the characteristic values for vertical loads shall be taken as shown in Figure 6.1.



Key
(1) No limitation

Figure 6.1 - Load Model 71 and characteristic values for vertical loads

(3)P The characteristic values given in Figure 6.1 shall 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 shall be one of the following:

0,75 - 0,83 - 0,91 - 1,00 - 1,10 - 1,21 - 1,33 - 1.46

The actions listed below shall be multiplied by the same factor α:
- equivalent vertical loading for earthworks and earth pressure effects according to 6.3.6.4,
- centrifugal forces according to 6.5.1,
- nosing force according to 6.5.2 (multiplied by α for $\alpha \geq 1$ only),
- traction and braking forces according to 6.5.3,
- combined response of structure and track to variable actions according to 6.5.4,
- derailment actions for Accidental Design Situations according to 6.7.1(2),
- Load Model SW/0 for continuous span bridges according to 6.3.3 and 6.8.1(8).

NOTE For international lines it is recommended to take $\alpha \geq 1,00$. The factor α may be specified in the National Annex or for the individual project.

(4)P For checking limits of deflection classified vertical loads and other actions enhanced by α in accordance with 6.3.2(3) shall be used (except for passenger comfort where α shall be taken as unity).

6.3.3 Load Models SW/0 and SW/2

(1) Load Model SW/0 represents the static effect of vertical loading due to normal rail traffic on continuous beams.

(2) Load Model SW/2 represents the static effect of vertical loading due to heavy rail traffic.

(3)P The load arrangement shall be taken as shown in Figure 6.2, with the characteristic values of the vertical loads according to Table 6.1.

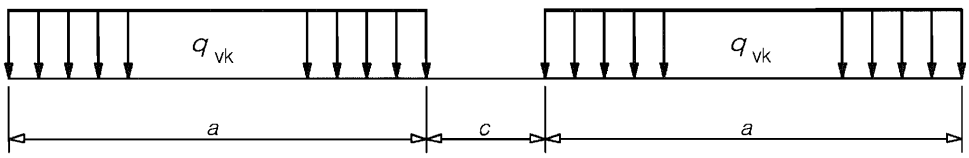


Figure 6.2 - Load Models SW/0 and SW/2

Table 6.1 - 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

(4)P The lines or section of line over which heavy rail traffic may operate where Load Model SW/2 shall be taken into account shall be designated.

NOTE The designation may be made in the National Annex or for the individual project.

(5)P Load Model SW/0 shall be multiplied by the factor α in accordance with 6.3.2(3).

6.3.4 Load Model "unloaded train"

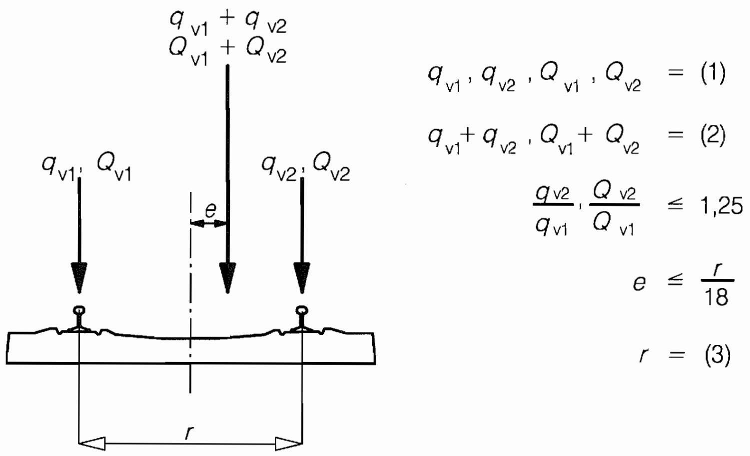
(1) For some specific verifications (see EN 1990 A2, § 2.2.4(2)) a particular load model is used, called "unloaded train". The Load Model "unloaded train" consists of a vertical uniformly distributed load with a characteristic value of 10,0 kN/m.

6.3.5 Eccentricity of vertical loads (Load Models 71 and SW/0)

(1)P The effect of lateral displacement of vertical loads shall be considered by taking the ratio of wheel loads on all axles as up to 1,25:1,00 on any one track. The resulting eccentricity e is shown in Figure 6.3.

Eccentricity of vertical loads may be neglected when considering fatigue.

NOTE Requirements for taking into account the position and tolerance in position of tracks are given in 6.8.1.



Key

(1) Uniformly distributed load and point loads on each rail as appropriate
(2) LM 71 (and SW/0 where required)
(3) Transverse distance between wheel loads

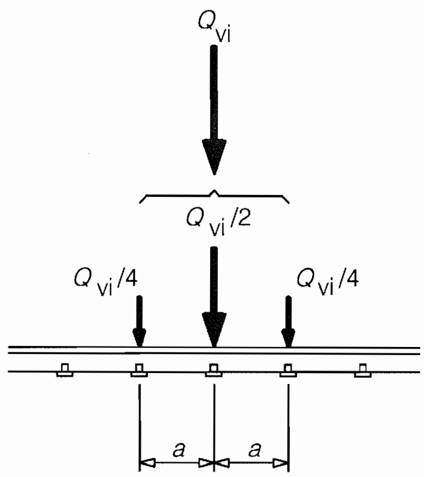
Figure 6.3 - Eccentricity of vertical loads

6.3.6 Distribution of axle loads by the rails, sleepers and ballast

(1) Subclauses 6.3.6.1 to 6.3.6.3 are applicable to Real Trains, Fatigue Trains, Load Models 71, SW/0, SW/2, the "unloaded train" and HSLM except where stated otherwise.

6.3.6.1 Longitudinal distribution of a point force or wheel load by the rail

(1) A point force in Load Model 71 (or classified vertical load in accordance with 6.3.2(3) where required) and HSLM (except for HSLM-B) or wheel load may be distributed over three rail support points as shown in Figure 6.4 below:



Key

Q_{vi} is the point force on each rail due to Load Model 71 or a wheel load of a Real Train in accordance with 6.3.5, Fatigue Train or HSLM (except for HSLM-B)

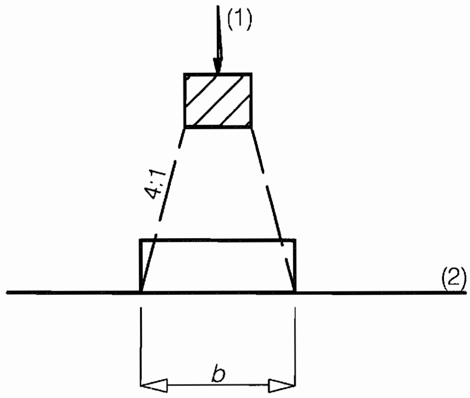
a is the distance between rail support points

Figure 6.4 - Longitudinal distribution of a point force or wheel load by the rail

6.3.6.2 Longitudinal distribution of load by sleepers and ballast

(1) Generally the point loads of Load Model 71 only (or classified vertical load in accordance with 6.3.2(3) where required) or an axle load may be distributed uniformly in the longitudinal direction (except where local load effects are significant, *e.g.* for the design of local floor elements, etc.).

(2) For the design of local floor elements etc. (*e.g.* longitudinal and transverse ribs, rail bearers, cross girders, deck plates, thin concrete slabs, etc.), the longitudinal distribution beneath sleepers as shown in Figure 6.5 should be taken into account, where the reference plane is defined as the upper surface of the deck.

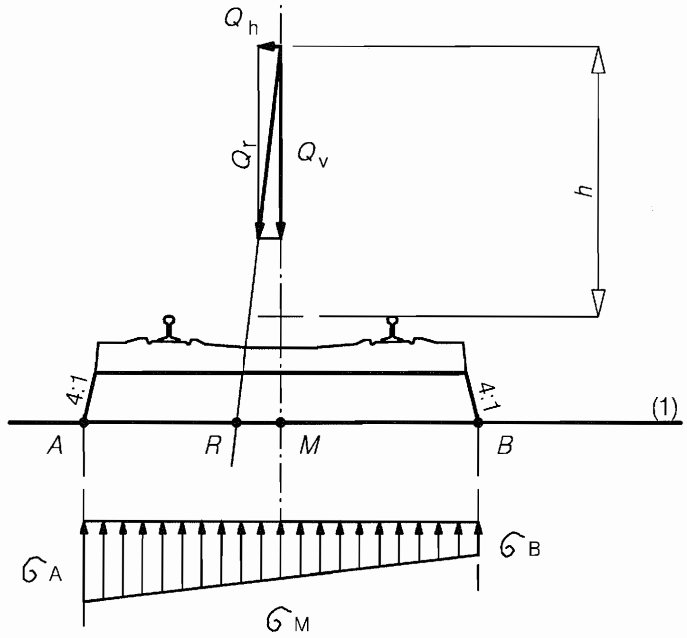


Key
(1) Load on sleeper
(2) Reference plane

Figure 6.5 - Longitudinal distribution of load by a sleeper and ballast

6.3.6.3 Transverse distribution of actions by the sleepers and ballast

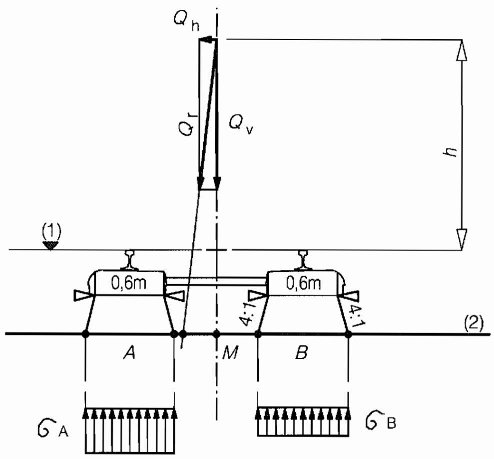
(1) On bridges with ballasted track without cant, the actions should be distributed transversely as shown in Figure 6.6.



Key
(1) Reference plane

Figure 6.6 - Transverse distribution of actions by the sleepers and ballast, track without cant (effect of eccentricity of vertical loads not shown)

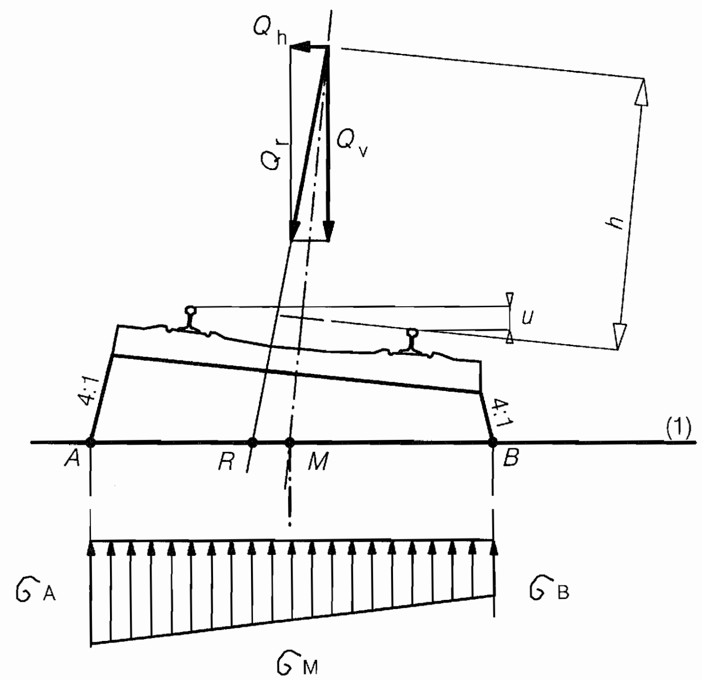
(2) On bridges with ballasted track (without cant) and full length sleepers, where the ballast is only consolidated under the rails, or for duo-block sleepers, the actions should be distributed transversely as shown in Figure 6.7.



Key
(1) Running surface
(2) Reference plane

Figure 6.7 - Transverse distribution of actions by the sleepers and ballast, track without cant (effect of eccentricity of vertical loads not shown)

(3) On bridges with ballasted track with cant the actions should be distributed transversely as shown in Figure 6.8.



Key
(1) Reference plane

Figure 6.8 - Transverse distribution of actions by the sleepers and ballast, track with cant (effect of eccentricity of vertical loads not shown)

(4) On bridges with ballasted track and cant and for full length sleepers, where the ballast is only consolidated under the rails, or for duo-block sleepers, Figure 6.8 should be modified to take into account the transverse load distribution under each rail shown in Figure 6.7.

(5) The transverse distribution to be used should be specified.

NOTE The individual project may specify the transverse distribution to be used.

6.3.6.4 Equivalent vertical loading for earthworks and earth pressure effects

(1) 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 in accordance with 6.3.2(3) 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.

(2) No dynamic factor or enhancement needs to be applied to the above uniformly distributed load.

(3) 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.

6.3.7 Actions for non-public footpaths

NOTE The individual project may specify alternative requirements for non-public footpaths, maintenance walkways or platforms etc.

(1) Non-public footpaths are those designated for use by only authorised persons.

(2) Pedestrian, cycle and general maintenance loads should be represented by a uniformly distributed load with a characteristic value $q_{fk} = 5$ kN/m².

(3) 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.

(4) Horizontal forces on parapets, partition walls and barriers due to persons should be taken as category B and C1 of EN 1991-1-1.

6.4 Dynamic effects (including resonance)

6.4.1 Introduction

(1) The static stresses and deformations (and associated bridge deck acceleration) induced in a bridge are increased and decreased under the effects of moving traffic by the following:

- the rapid rate of loading due to the speed of traffic crossing the structure and the inertial response (impact) of the structure,
- the passage of successive loads with approximately uniform spacing which can excite the structure and under certain circumstances create resonance (where the frequency of excitation (or a multiple there of) matches a natural frequency of the structure (or a multiple there of), there is a possibility that the vibrations caused by successive axles running onto the structure will be excessive),
- variations in wheel loads resulting from track or vehicle imperfections (including wheel irregularities.

(2)P For determining the effects (stresses, deflections, bridge deck acceleration etc.) of rail traffic actions the above effects shall be taken into account.

6.4.2 Factors influencing dynamic behaviour

(1) The principal factors which influence dynamic behaviour are:

i) the speed of traffic across the bridge,
ii) the span L of the element and the influence line length for deflection of the element being considered,
iii) the mass of the structure,
iv) the natural frequencies of the whole structure and relevant elements of the structure and the associated mode shapes (eigenforms) along the line of the track,
v) the number of axles, axle loads and the spacing of axles,
vi) the damping of the structure,
vii) vertical irregularities in the track,
viii) the unsprung/sprung mass and suspension characteristics of the vehicle,

ix) the presence of regularly spaced supports of the deck slab and/or track (cross girders, sleepers etc.),
x) vehicle imperfections (wheel flats, out of round wheels, suspension defects etc.),
xi) the dynamic characteristics of the track (ballast, sleepers, track components etc.).

These factors are taken into account in 6.4.4 to 6.4.6.

NOTE There are no specific deflection limits specified for avoiding resonance and excessive vibration effects. See EN 1990 A2 for deflection criteria for traffic safety and passenger comfort etc.

6.4.3 General design rules

(1)P A static analysis shall be carried out with the load models defined in 6.3 (LM71 and where required Load Models SW/0 and SW/2). The results shall be multiplied by the dynamic factor Φ defined in 6.4.5 (and if required multiplied by α in accordance with 6.3.2).

(2) The criteria for determining whether a dynamic analysis is required are given in 6.4.4.

(3)P Where a dynamic analysis is required:

- the additional load cases for the dynamic analysis shall be in accordance with 6.4.6.1.2.
- maximum peak deck acceleration shall be checked in accordance with 6.4.6.5.
- the results of the dynamic analysis shall be compared with the results of the static analysis multiplied by the dynamic factor Φ in 6.4.5 (and if required multiplied by α in accordance with 6.3.2). The most unfavourable values of the load effects shall be used for the bridge design in accordance with 6.4.6.5.
- a check shall be carried out according to 6.4.6.6 to ensure that the additional fatigue loading at high speeds and at resonance is covered by consideration of the stresses derived from the results of the static analysis multiplied by the dynamic factor Φ.

(4) All bridges where the Maximum Line Speed at the Site is greater than 200 km/h or where a dynamic analysis is required should be designed for characteristic values of Load Model 71 (and where required Load Model SW/0) or classified vertical loads with $\alpha \geq 1$ in accordance with 6.3.2.

(5) For passenger trains the allowances for dynamic effects in 6.4.4 to 6.4.6 are valid for Maximum Permitted Vehicle Speeds up to 350 km/h.

6.4.4 Requirement for a static or dynamic analysis

(1) The requirements for determining whether a static or a dynamic analysis is required are shown in Figure 6.9.

NOTE The National Annex may specify alternative requirements. The use of the flow chart in Figure 6.9 is recommended.

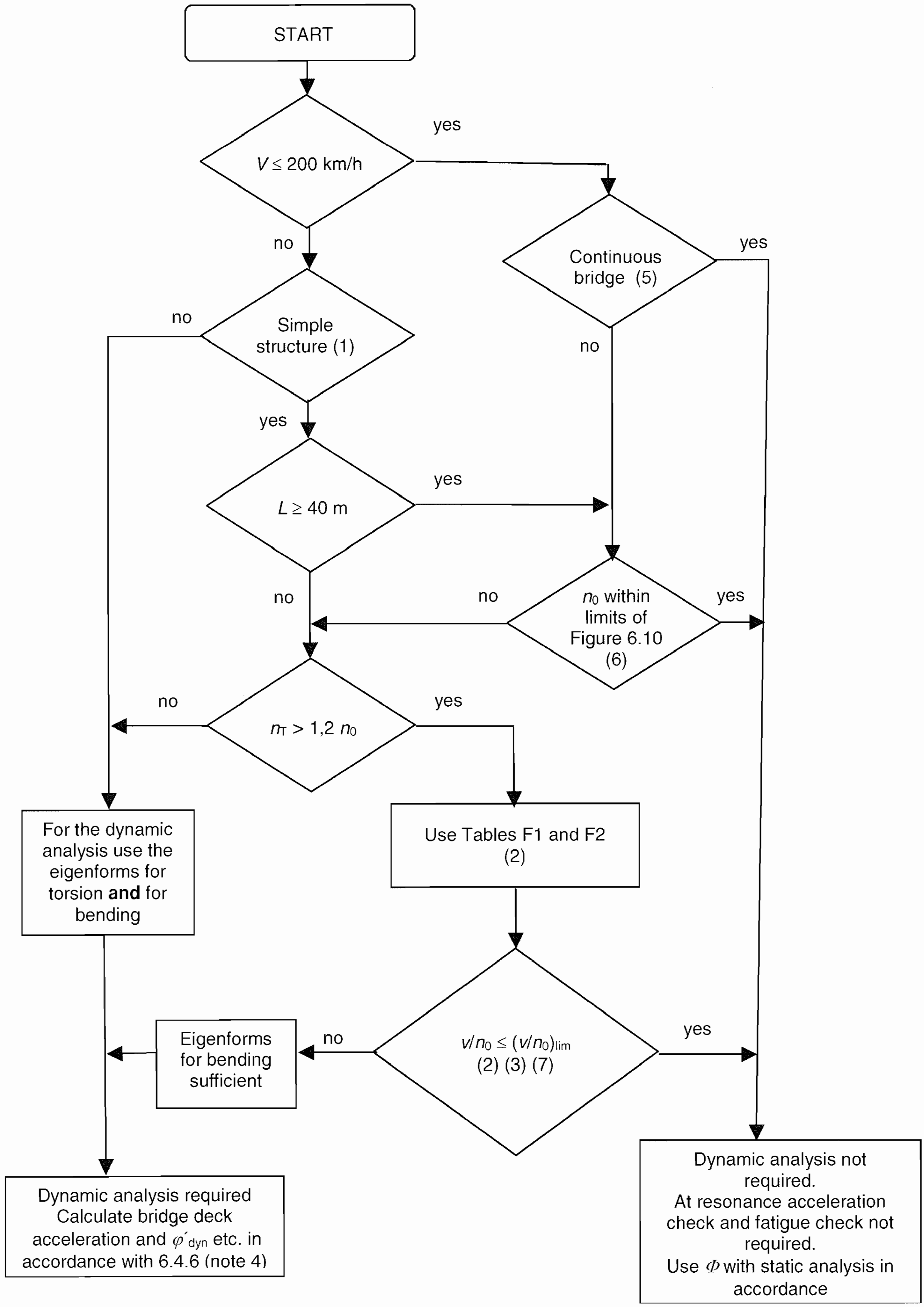


Figure 6.9 - Flow chart for determining whether a dynamic analysis is required

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 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 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.

NOTE 4 φ'_{dyn} is the dynamic impact component for Real Trains for the structure given in 6.4.6.5(3).

NOTE 5 Valid providing the bridge meets the requirements for resistance, deformation limits given in EN 1990 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 A2.

NOTE 6 For bridges with a first natural frequency n_0 within the limits given by Figure 6.10 and a Maximum Line Speed at the Site not exceeding 200km/h, a dynamic analysis is not required.

NOTE 7 For bridges with a first natural frequency n_0 exceeding the upper limit (1) in Figure 6.10 a dynamic analysis is required. Also see 6.4.6.1.1(7).

The upper limit of n_0 is governed by dynamic enhancements due to track irregularities and is given by :

$$n_0 = 94{,}76L^{-0{,}748} \quad (6.1)$$

The lower limit of n_0 is governed by dynamic impact criteria and is given by :

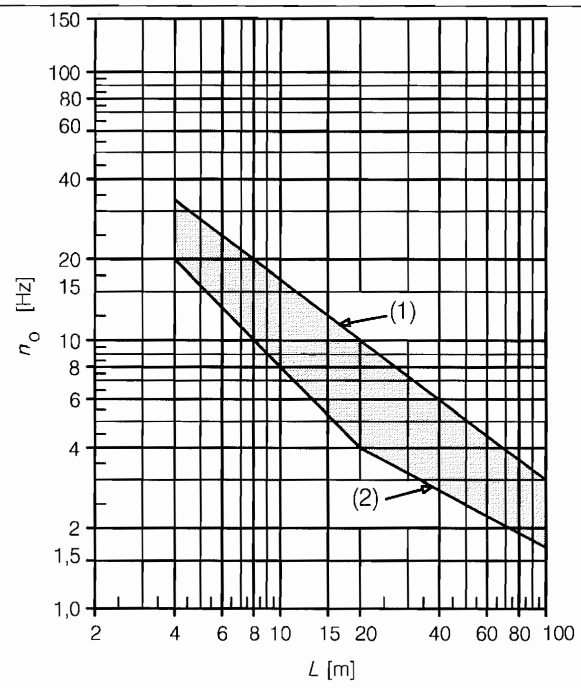
$$n_0 = 80/L \quad \text{for } 4\text{m} \leq L \leq 20\text{m}$$

$$n_0 = 23{,}58L^{-0{,}592} \quad \text{for } 20\text{m} < L \leq 100\text{m} \quad (6.2)$$

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.



Key

(1) Upper limit of natural frequency

(2) Lower limit of natural frequency

Figure 6.10 - Limits of bridge natural frequency n_0 [Hz] as a function of L [m]

NOTE 8 For a simply supported bridge subjected to bending only, the natural frequency may be estimated using the formula :

$$n_0\,[\mathrm{Hz}] = \frac{17{,}75}{\sqrt{\delta_0}} \tag{6.3}$$

where:

δ_0 is the deflection at mid span 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.

6.4.5 Dynamic factor Φ (Φ_2, Φ_3)

6.4.5.1 Field of application

(1) 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.

(2)P Where the criteria specified in 6.4.4 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 shall be carried out to calculate impact and resonance effects.

NOTE Quasi static methods which use static load effects multiplied by the dynamic factor Φ defined in 6.4.5 are unable to predict resonance effects from high speed trains. Dynamic analysis techniques, which take into account the time dependant nature of the loading from the High Speed Load Model (HSLM) and Real Trains (*e.g.* by solving equations of motion) are required for predicting dynamic effects at resonance.

(3) Structures carrying more than one track should be considered without any reduction of dynamic factor Φ.

6.4.5.2 Definition of the dynamic factor Φ

(1)P The dynamic factor Φ which enhances the static load effects under Load Models 71, SW/0 and SW/2 shall be taken as either Φ_2 or Φ_3.

(2) 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:

$$\Phi_2 = \frac{1{,}44}{\sqrt{L_\Phi} - 0{,}2} + 0{,}82 \tag{6.4}$$

with: $1{,}00 \leq \Phi_2 \leq 1{,}67$

(b) For track with standard maintenance:

$$\Phi_3 = \frac{2{,}16}{\sqrt{L_\Phi} - 0{,}2} + 0{,}73 \tag{6.5}$$

with: $1{,}00 \leq \Phi_3 \leq 2{,}0$

where:

L_Φ "Determinant" length (length associated with Φ) defined in Table 6.2 [m].

NOTE 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.

(3)P If no dynamic factor is specified Φ_3 shall be used.

NOTE The dynamic factor to be used may be specified in the National Annex or for the individual project.

(4)P The dynamic factor Φ shall not be used with:
- the loading due to Real Trains,
- the loading due to Fatigue Trains (annex D),
- Load Model HSLM (6.4.6.1.1(2)),
- the load model "unloaded train" (6.3.4).

6.4.5.3 Determinant length L_Φ

(1) The determinant lengths L_Φ to be used are given in the Table 6.2 below.

NOTE Alternative values of L_Φ may be specified in the National Annex. The values given in Table 6.2 are recommended.

(2) Where no value of L_Φ is specified in Table 6.2 the determinant length should be taken as the length of the influence line for deflection of the element being considered or alternative values should be specified.

NOTE The individual project may specify alternative values.

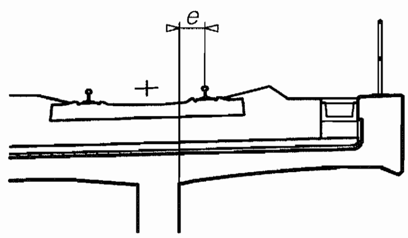
(3) If the resultant stress in a structural member depends on several effects, each of which relates to a separate structural behaviour, then each effect should be calculated using the appropriate determinant length.

Table 6.2 - Determinant lengths L_Φ

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	3,6m [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	3,6m [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 - simply supported	 3 times cross girder spacing Cross girder spacing + 3 m
3.2	Cantilever of rail bearer [a]	3,6m
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	3,6m [b]

[a] In general all cantilevers greater than 0,50 m supporting rail traffic actions need a special study in accordance with 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

Table 6.2 (continued)

Case	Structural element	Determinant length L_Φ
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] **Figure 6.11 - Transverse cantilever supporting railway loading**
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 + 3m
	- 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: 3,6m [b] - $e > 0,5$ m: [a]
4.6	End cross girders or trimmer beams	3,6m [b]

[a] In general all cantilevers greater than 0,50 m supporting rail traffic actions need a special study in accordance with 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.

Table 6.2 (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 $L_m = 1/n\ (L_1 + L_2 + .. + L_n)$ (6.6)	$L_\Phi = k \times L_m$, (6.7) but not less than max L_i ($i = 1,..., n$) $n =$ 2 3 4 ≥ 5 $k =$ 1,2 1,3 1,4 1,5
5.3	Portal frames and closed frames or boxes: - single-span - multi-span	 Consider as three-span continuous beam (use 5.2, with vertical and horizontal lengths of members of the frame or box) Consider as multi-span continuous beam (use 5.2, with lengths of end vertical members and horizontal members)
5.4	Single arch, archrib, 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

6.4.5.4 Reduced dynamic effects

(1) In the case of arch bridges and concrete bridges of all types with a cover of more than 1,00 m, Φ_2 and Φ_3 may be reduced as follows:

$$red\ \Phi_{2,3} = \Phi_{2,3} - \frac{h - 1{,}00}{10} \geq 1{,}0 \qquad (6.8)$$

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) [m].

(2) 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.

6.4.6 Requirements for a dynamic analysis

6.4.6.1 Loading and load combinations

6.4.6.1.1 Loading

(1)P The dynamic analysis shall be undertaken using characteristic values of the loading from the Real Trains specified. The selection of Real Trains shall take into account each permitted or envisaged train formation for every type of high speed train permitted or envisaged to use the structure at speeds over 200km/h.

NOTE 1 The individual project may specify the characteristic axle loads and spacings for each configuration of each required Real Train.

NOTE 2 Also see 6.4.6.1.1(7) for loading where a dynamic analysis is required for a Maximum Line Speed at the Site less than 200km/h.

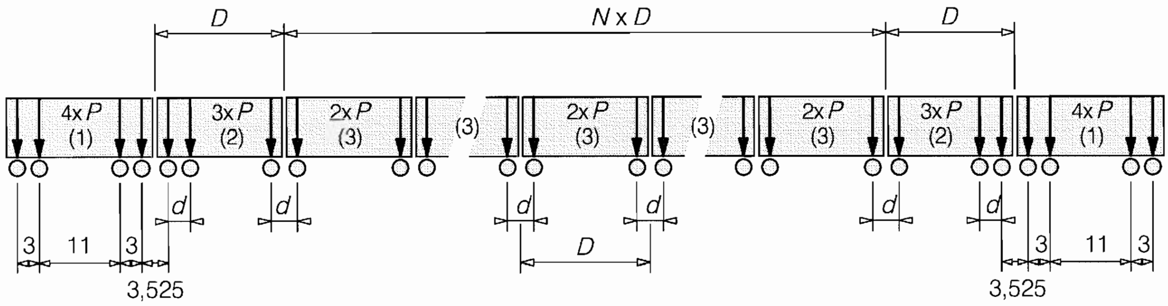
(2)P The dynamic analysis shall also be undertaken using Load Model HSLM on bridges designed for international lines where European high speed interoperability criteria are applicable.

NOTE The individual project may specify when Load Model HSLM is to be used.

(3) Load Model HSLM comprises of two separate Universal Trains with variable coach lengths, HSLM-A and HSLM-B.

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 for the European Technical Specification for Interoperability given in E.1.

(4) HSLM-A is defined in Figure 6.12 and Table 6.3:



Key

(1) Power car (leading and trailing power cars identical)

(2) End coach (leading and trailing end coaches identical)

(3) Intermediate coach

Figure 6.12 - HSLM-A

Table 6.3 - HSLM-A

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	16	20	2,0	180
A4	15	21	3,0	190
A5	14	22	2,0	170
A6	13	23	2,0	180
A7	13	24	2,0	190
A8	12	25	2,5	190
A9	11	26	2,0	210
A10	11	27	2,0	210

(5) HSLM-B comprises of N number point forces of 170 kN at uniform spacing d [m] where N and d are defined in Figures 6.13 and 6.14:

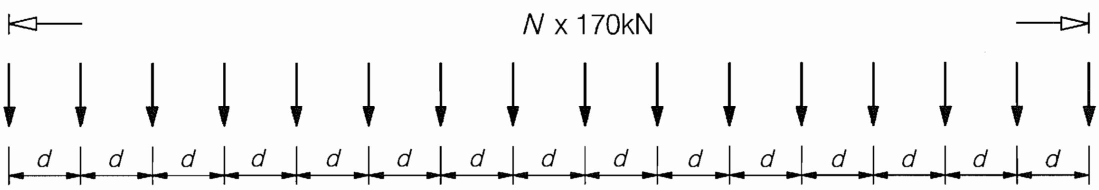


Figure 6.13 - HSLM-B

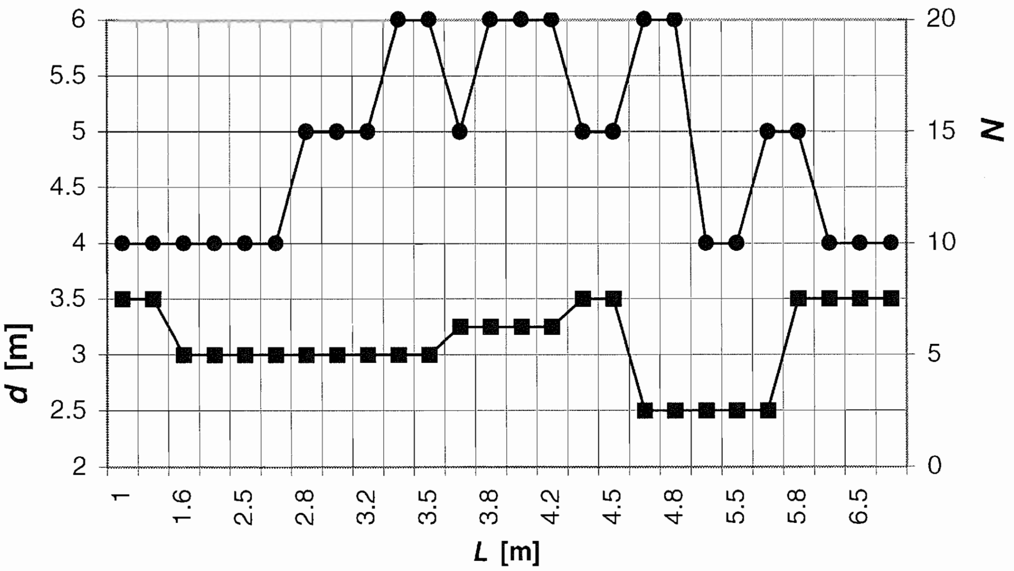


Figure 6.14 - HSLM-B

where L is the span length [m].

(6) Either HSLM-A or HSLM-B should be applied in accordance with the requirements of Table 6.4:

Table 6.4 - Application of HSLM-A and HSLM-B

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 critical Universal Train from HSLM-A may be used for the dynamic analysis in accordance with 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.

(7) Where the frequency limits of Figure 6.10 are not satisfied and the Maximum Line Speed at the Site is ≤ 200 km/h a dynamic analysis should be carried out. The analysis should take into account the behaviours identified in 6.4.2 and consider:

- Train Types 1 to 12 given in annex D,
- Real Trains specified.

NOTE The loading and methodology for the analysis may be specified for the individual project and should be agreed with the relevant authority specified in the National Annex.

6.4.6.1.2 Load combinations and partial factors

(1) For the dynamic analysis the calculation of the value of mass associated with self weight and removable loads (ballast etc.) should use nominal values of density.

(2)P For the dynamic analysis loads according to 6.4.6.1.1(1) and (2) and where required 6.4.6.1.1(7) shall be used.

(3) For the dynamic analysis of the structure only, one track (the most adverse) on the structure should be loaded in accordance with Table 6.5.

Table 6.5 - Summary of additional load cases depending upon number of tracks on bridge

Number of tracks on a 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 2 tracks with trains normally travelling in the same directions or carrying 3 or more tracks with a Maximum Line Speed at the Site exceeding 200km/h the loading should be agreed with the relevant authority specified in the National Annex.

(4) Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures) in accordance with 6.4.6.5(3) 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 of 6.8.1 and Table 6.11.

(5)P Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures) in accordance with 6.4.6.5(3) the dynamic rail loading effects (bending moments, shears, deformations etc. excluding acceleration) determined from the dynamic analysis shall be enhanced by the partial factors given in A2 of EN 1990.

(6)P Partial factors shall not be applied to the loading given in 6.4.6.1.1 when determining bridge deck accelerations. The calculated values of acceleration shall be directly compared with the design values in 6.4.6.5.

(7) For fatigue, a bridge should be designed for the additional fatigue effects at resonance from the loading in accordance with 6.4.6.1.1 on any one track. See 6.4.6.6.

6.4.6.2 Speeds to be considered

(1)P For each Real Train and Load Model HSLM a series of speeds up to the Maximum Design Speed shall be considered. The Maximum Design Speed shall be generally 1,2 × Maximum Line Speed at the site.

The Maximum Line Speed at the site shall be specified.

NOTE 1 The individual project may specify the Maximum Line Speed at the site.

NOTE 2 Where specified for the individual project a reduced speed may be used for checking individual Real Trains for 1,2 × their associated Maximum Permitted Vehicle Speed.

NOTE 3 It is recommended that the individual project specify an increased Maximum Line Speed at the Site to take into account potential modifications to the infrastructure and future rolling stock.

NOTE 4 Structures can exhibit a highly peaked response due to resonance effects. Where there is a likelihood of train overspeeding and exceeding either the Maximum Permitted Vehicle Speed or the current or envisaged Maximum Line Speed at the Site it is recommended that the individual project specify an additional factor for increasing the Maximum Design Speed to be used in the dynamic analysis.

NOTE 5 It is recommended that the individual project specify additional requirements for checking structures where there is a requirement for a section of line to be suitable for commissioning tests of a Real Train. The Maximum Design Speed used for the Real Train should be at least 1,2 × Maximum Train Commissioning Speed. Calculations are required to demonstrate that safety considerations (maximum deck accelerations, maximum load effects, etc.) are satisfactory for structures at speeds in excess of 200 km/h. Fatigue and passenger comfort criteria need not be checked at 1,2 × Maximum Train Commissioning Speeds.

(2) Calculations should be made for a series of speeds from 40m/s up to the Maximum Design Speed defined by 6.4.6.2(1). 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 Equation 6.9.

$$v_i = n_0 \lambda_i \tag{6.9}$$

and

$$40 \text{ m/s} \le v_i \le \text{Maximum Design Speed}, \quad (6.10)$$

where:

v_i is the Resonant Speed [m/sec]
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:

$$\lambda_i = \frac{d}{i} \quad (6.11)$$

d is the regular spacing of groups of axles
i = 1, 2, 3 or 4.

6.4.6.3 Bridge parameters

6.4.6.3.1 Structural damping

(1) The peak response of a structure at traffic speeds corresponding to resonant loading is highly dependent upon damping.

(2)P Only lower bound estimates of damping shall be used.

(3) The following values of damping should be used in the dynamic analysis:

Table 6.6 - Values of damping to be assumed for design purposes

Bridge Type	ζ Lower limit of percentage of critical damping [%]	
	Span L < 20m	Span $L \ge$ 20m
Steel and composite	$\zeta = 0{,}5 + 0{,}125\ (20 - L)$	$\zeta = 0{,}5$
Prestressed concrete	$\zeta = 1{,}0 + 0{,}07\ (20 - L)$	$\zeta = 1{,}0$
Filler beam and reinforced concrete	$\zeta = 1{,}5 + 0{,}07\ (20 - L)$	$\zeta = 1{,}5$

NOTE Alternative safe lower bound values may be used subject to the agreement of the relevant authority specified in the National Annex.

6.4.6.3.2 Mass of the bridge

(1) 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 and any underestimation of mass will overestimate the natural frequency of the structure and overestimate the traffic speeds at which resonance occurs.

At resonance the maximum acceleration of a structure is inversely proportional to the mass of the structure.

(2)P Two specific cases for the mass of the structure including ballast and track shall be considered:

- a lower bound estimate of mass to predict maximum deck accelerations using the minimum likely dry clean density and minimum thickness of ballast,
- an upper bound estimate of mass to predict the lowest speeds at which resonant effects are likely to occur using the maximum saturated density of dirty ballast with allowance for future track lifts.

NOTE The minimum density of ballast may be taken as 1700kg/ m^3. Alternative values may be specified for the individual project.

(3) In the absence of specific test data the values for the density of materials should be taken from EN 1991-1-1.

NOTE Owing to the large number of parameters which can affect the density of concrete it is not possible to predict enhanced density values with sufficient accuracy for predicting the dynamic response of a bridge. Alternative density 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.

6.4.6.3.3 Stiffness of the bridge

(1) 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.

(2)P A lower bound estimate of the stiffness throughout the structure shall be used.

(3) 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.

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 of $f_{ck} = 50$ 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 2 Other material properties may be used subject to the agreement of the relevant authority specified in the National Annex.

6.4.6.4 Modelling the excitation and dynamic behaviour of the structure

(1) The dynamic effects of a Real Train may be represented by a series of travelling point forces. Vehicle/structure mass interaction effects may be neglected.

The analysis should take into account variations throughout the length of the train in axle forces and the variations in spacing of individual axles or groups of axles.

(2) Where appropriate the analysis technique should allow for the following dynamic behaviours of the structure:

- for complex structures the proximity of adjacent frequencies and associated mode shapes of oscillation,
- interaction between bending and torsional modes,
- local deck element behaviour (shallow floors and cross girders of half-through type bridges or trusses etc.),
- the skew behaviour of slabs etc.

(3) The representation of each axle by a single point force tends to overestimate dynamic effects for loaded lengths of less than 10m. In such cases, the load distribution effects of rails, sleepers and ballast may be taken into account.

Notwithstanding 6.3.6.2(1) individual axle loads should not be distributed uniformly in the longitudinal direction for a dynamic analysis.

(4) 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,

NOTE The method used should be agreed with the relevant authority specified in the National Annex.

- increasing the value of damping assumed for the structure according to Figure 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 :

$$\zeta_{\mathrm{TOTAL}} = \zeta + \Delta\zeta \qquad (6.12)$$

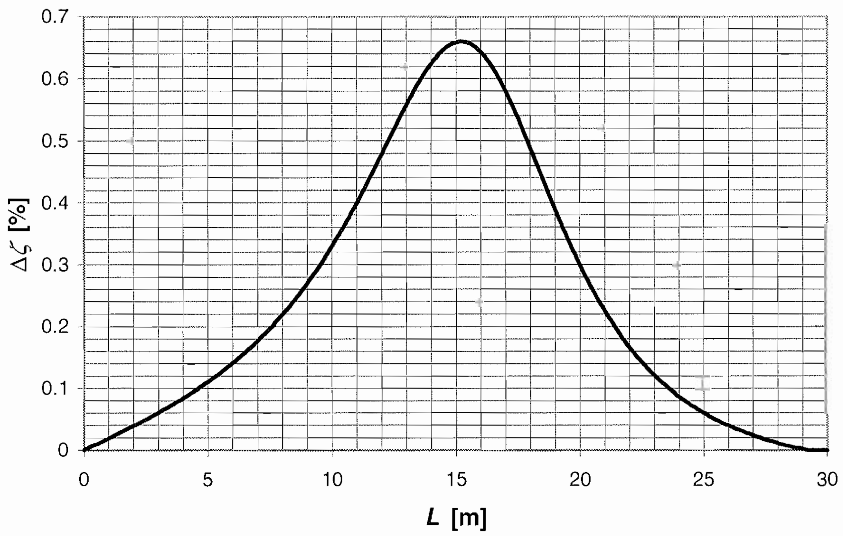


Figure 6.15 - Additional damping $\Delta\zeta$ [%] as a function of span length L [m]

where:

$$\Delta\zeta = \frac{0{,}0187L - 0{,}00064L^2}{1 - 0{,}0441L - 0{,}0044L^2 + 0{,}000255L^3}[\%] \qquad (6.13)$$

ζ is the lower limit of percentage of critical damping [%] defined in 6.4.6.3.1.

NOTE The National Annex may specify alternative values.

(5) The increase in calculated dynamic load effects (stresses, deflections, bridge deck accelerations, etc.) due to track defects and vehicle imperfections may be estimated by multiplying the calculated effects by a factor of:

$(1 + \varphi''/2)$ for carefully maintained track,

$(1 + \varphi'')$ for track with standard track maintenance,

where:

φ'' is in accordance with annex C and should not be taken as less than zero.

NOTE The National Annex may specify the factor to be used.

(6) Where the bridge satisfies the upper limit in Figure 6.10 the factors that influence dynamic behaviours (vii) to (xi) identified in 6.4.2 may be considered to be allowed for in Φ, $\varphi''/2$ and φ'' given in 6.4 and annex C.

6.4.6.5 Verifications of the limit states

(1)P To ensure traffic safety:

- The verification of maximum peak deck acceleration shall be regarded as a traffic safety requirement checked at the serviceability limit state for the prevention of track instability.
- The dynamic enhancement of load effects shall be allowed for by multiplying the static loading by the dynamic factor Φ defined in 6.4.5. If a dynamic analysis is necessary, the results of the dynamic analysis shall be compared with the results of the static analysis enhanced by Φ (and if required multiplied by α in accordance with 6.3.2) and the most unfavourable load effects shall be used for the bridge design.
- If a dynamic analysis is necessary, a check shall be carried out according to 6.4.6.6 to establish whether the additional fatigue loading at high speeds and at resonance is covered by consideration of the stresses due to load effects from Φ x LM71 (and if required $\Phi \times$ Load Model SW/0 for continuous structures and classified vertical load in accordance with 6.3.2(3) where required). The most adverse fatigue loading shall be used in the design.

(2)P The maximum permitted peak design values of bridge deck acceleration calculated along the line of a track shall not exceed the recommended values given in A2 of EN 1990 (see A2.4.4.2.1).

(3) A dynamic analysis (if required) should be used to determine the following dynamic enhancement :

$$\varphi'_{dyn} = \max \left| y_{dyn} / y_{stat} \right| - 1 \qquad (6.14)$$

where:

y_{dyn} is the maximum dynamic response and AC1 *text deleted* AC1
AC1 y_{stat} the corresponding maximum static response at any particular point in the structural element due to a Real Train or Load Model HSLM. AC1

For the design of the bridge, taking into account all the effects of vertical traffic loads, the most unfavourable value of:

$$\left(1+\varphi'_{dyn}+\varphi''/2\right)\times\begin{pmatrix} HSLM \\ \text{or} \\ RT \end{pmatrix} \qquad (6.15)$$

or

$$\Phi \times (\text{LM71"+"SW/0}) \qquad (6.16)$$

should be used where:

HSLM is the load model for high speed lines defined in 6.4.6.1.1(2),

LM71"+"SW/0 is Load Model 71 and if relevant Load Model SW/0 for continuous bridges (or classified vertical load in accordance with 6.3.2(3) where required).

RT is the loading due to all Real Trains defined in 6.4.6.1.1.

$\varphi''/2$ is the increase in calculated dynamic load effects (stresses, deflections, bridge deck accelerations, etc.) resulting from track defects and vehicle imperfections in accordance with annex C for carefully maintained track (φ'' to be used for track with standard maintenance).

Φ is the dynamic factor in accordance with 6.4.5.

6.4.6.6 Additional verification for fatigue where dynamic analysis is required

(1)P The fatigue check of the structure shall allow for the stress range resulting from elements of the structure oscillating above and below the corresponding permanent load deflection due to:

- additional free vibrations set up by impact effects from axle loads travelling at high speed,
- the magnitude of dynamic live loading effects at resonance,
- the additional cycles of stress caused by the dynamic loading at resonance.

(2)P Where the Frequent Operating Speed of a Real Train at a structure is near to a Resonant Speed the design shall allow for the additional fatigue loading due to resonance effects.

NOTE The individual project may specify the fatigue loading, *e.g.* details, annual tonnage and mix of Real Trains and associated Frequent Operating Speeds at the site to be taken into account in the design.

(3) Where the bridge is designed for Load Model HSLM in accordance with 6.4.6.1.1(2) the fatigue loading should be specified taking into account the best estimate of current and future traffic.

NOTE The individual project may specify the fatigue loading *e.g.* details, annual tonnage and mix of Real Trains and associated Frequent Operating Speeds at the site to be taken into account in the design.

(4) For structures that satisfy annex F the Resonant Speed may be estimated using equations 6.9 and 6.10.

(5) For the verification for fatigue a series of speeds up to a Maximum Nominal Speed should be considered.

NOTE It is recommended that the individual project specify an increased Maximum Nominal Speed at the Site to take into account potential modifications to the infrastructure and future rolling stock.

6.5 Horizontal forces - characteristic values

6.5.1 Centrifugal forces

(1)P 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 shall be taken into account.

(2) The centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1,80 m above the running surface (see Figure 1.1). For some traffic types, *e.g.* double stacked containers, an increased value of h_t should be specified.

NOTE The National Annex or individual project may specify an increased value of h_t.

(3)P The centrifugal force shall always be combined with the vertical traffic load. The centrifugal force shall not be multiplied by the dynamic factor Φ_2 or Φ_3.

NOTE 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.

(4)P The characteristic value of the centrifugal force shall be determined according to the following equations:

$$Q_{tk} = \frac{v^2}{g \times r}(f \times Q_{vk}) = \frac{V^2}{127r}(f \times Q_{vk}) \tag{6.17}$$

$$q_{tk} = \frac{v^2}{g \times r}(f \times q_{vk}) = \frac{V^2}{127r}(f \times q_{vk}) \tag{6.18}$$

where:

Q_{tk}, q_{tk}	Characteristic values of the centrifugal forces [kN, kN/m]
Q_{vk}, q_{vk}	Characteristic values of the vertical loads specified in 6.3 (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	Reduction factor (see below)
v	Maximum speed in accordance with 6.5.1(5) [m/s]
V	Maximum speed in accordance with 6.5.1(5) [km/h]
g	Acceleration due to gravity [9,81 m/s²]
r	Radius of curvature [m]

In the case of a curve of varying radii, suitable mean values may be taken for the value r.

(5)P The calculations shall be based on the specified Maximum Line Speed at the Site. In the case of Load Model SW/2 an alternative maximum speed may be assumed.

NOTE 1 The individual project may specify the requirements.

NOTE 2 For SW/2 a maximum speed of 80km/h may be used.

NOTE 3 It is recommended that the individual project specify an increased Maximum Line Speed at the Site to take into account potential modifications to the infrastructure and future rolling stock.

(6)P In addition, for bridges located in a curve, the case of the loading specified in 6.3.2 and, if applicable, 6.3.3, shall also be considered without centrifugal force.

(7) 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:

a) Load Model 71 (and where required Load Model SW/0) with its dynamic factor and the centrifugal force for V=120 km/h according to equations 6.17 and 6.18 with f = 1.
b) Load Model 71 (and where required Load Model SW/0) with its dynamic factor and the centrifugal force according to equations 6.17 and 6.18 for the maximum speed V specified, with a value for the reduction factor f given by 6.5.1(8).

(8) For Load Model 71 (and where required Load Model SW/0) the reduction factor f is given by:

$$f = \left[1 - \frac{V - 120}{1000}\left(\frac{814}{V} + 1{,}75\right)\left(1 - \sqrt{\frac{2{,}88}{L_f}}\right)\right] \qquad (6.19)$$

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 speed in accordance with 6.5.1(5).

f =1 for either $V \leq 120$ km/h or $L_f \leq 2{,}88$ m

f <1 for 120 km/h < $V \leq 300$ km/h (see Table 6.7 or Figure 6.16 or equation 6.19) and $L_f > 2{,}88$m
$f_{(V)} = f_{(300)}$ for V >300 km/h.

For the load models SW/2 and "unloaded train" the value of the reduction factor f should be taken as 1,0.

Table 6.7 - Factor *f* for Load Model 71 and SW/0

L_f [m]	Maximum speed in accordance with 6.5.1(5) [km/h]				
	≤ 120	160	200	250	≥ 300
≤ 2,88	1,00	1,00	1,00	1,00	1,00
3	1,00	0,99	0,99	0,99	0,98
4	1,00	0,96	0,93	0,90	0,88
5	1,00	0,93	0,89	0,84	0,81
6	1,00	0,92	0,86	0,80	0,75
7	1,00	0,90	0,83	0,77	0,71
8	1,00	0,89	0,81	0,74	0,68
9	1,00	0,88	0,80	0,72	0,65
10	1,00	0,87	0,78	0,70	0,63
12	1,00	0,86	0,76	0,67	0,59
15	1,00	0,85	0,74	0,63	0,55
20	1,00	0,83	0,71	0,60	0,50
30	1,00	0,81	0,68	0,55	0,45
40	1,00	0,80	0,66	0,52	0,41
50	1,00	0,79	0,65	0,50	0,39
60	1,00	0,79	0,64	0,49	0,37
70	1,00	0,78	0,63	0,48	0,36
80	1,00	0,78	0,62	0,47	0,35
90	1,00	0,78	0,62	0,47	0,35
100	1,00	0,77	0,61	0,46	0,35
≥150	1,00	0,76	0,60	0,44	0,35

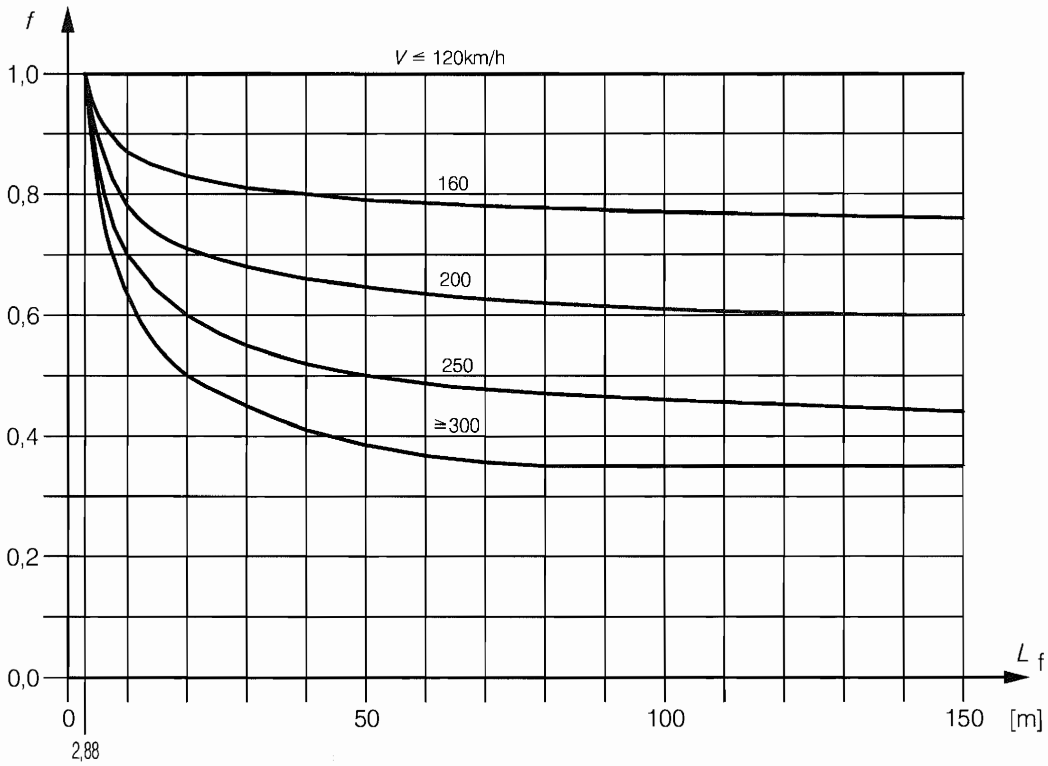


Figure 6.16 - Factor *f* for Load Model 71 and SW/0

(9) For LM71 and SW/0 centrifugal forces should be determined from equations 6.17 and 6.18 using classified vertical loads (see 6.3.2(3)) in accordance with the load cases given in Table 6.8:

Table 6.8 - Load Cases for centrifugal force corresponding to values of α and Maximum Line Speed at Site

Value of α	Maximum Line Speed at Site [km/h]	Centrifugal force based on : [d]				Associated vertical traffic action based on: [a]
		V [km/h]	α	f		
$\alpha < 1$	> 120	V	1^{c}	f	1^{c} x f x (LM71"+"SW/0) for case 6.5.1(7)b	Φ x α x 1 x (LM71"+"SW/0)
		120	α	1	α x 1 x (LM71"+"SW/0) for case 6.5.1(7)a	Φ x α x 1 x (LM71"+"SW/0)
		0	-	-	-	
	≤ 120	V	α	1	α x 1 x (LM71"+"SW/0)	
		0	-	-	-	
$\alpha = 1$	> 120	V	1	f	1 x f x (LM71"+"SW/0) for case 6.5.1(7)b	Φ x 1 x 1 x (LM71"+"SW/0)
		120	1	1	1 x 1 x (LM71"+"SW/0) for case 6.5.1(7)a	Φ x 1 x 1 x (LM71"+"SW/0)
		0	-	-	-	
	≤ 120	V	1	1	1 x 1 x (LM71"+"SW/0)	
		0	-	-	-	
$\alpha > 1$	> 120 [b]	V	1	f	1 x f x (LM71"+"SW/0) for case 6.5.1(7)b	Φ x 1 x 1 x (LM71"+"SW/0)
		120	α	1	α x 1 x (LM71"+"SW/0) for case 6.5.1(7)a	Φ x α x 1 x (LM71"+"SW/0)
		0	-	-	-	
	≤ 120	V	α	1	α x 1 x (LM71"+"SW/0)	
		0	-	-	-	

[a] 0,5 x (LM71"+"SW/0) instead of (LM71"+"SW/0) where vertical traffic actions favourable.
[b] Valid for heavy freight traffic limited to a maximum speed of 120 km/h.
[c] $\alpha = 1$ to avoid double counting the reduction in mass of train with f.
[d] See 6.5.1(3) 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 to be included as shown above.

where:

V	Maximum speed in accordance with 6.5.1(5) [km/h]
f	Reduction factor in accordance with 6.5.1(8)
α	Factor for classified vertical loads in accordance with 6.3.2(3).
LM71"+"SW/0	Load Model 71 and if relevant Load Model SW/0 for continuous bridges.

(10) The criteria in 6.5.1(5) and 6.5.1(7) to 6.5.1(9) 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.

NOTE The individual project may specify the additional requirements.

6.5.2 Nosing force

(1)P The nosing force shall be taken as a concentrated force acting horizontally, at the top of the rails, perpendicular to the centre-line of track. It shall be applied on both straight track and curved track.

(2)P The characteristic value of the nosing force shall be taken as Q_{sk} = 100 kN. It shall not be multiplied by the factor Φ (see 6.4.5) or by the factor f in 6.5.1(4).

(3) The characteristic value of the nosing force in 6.5.2(2) should be multiplied by the factor α in accordance with 6.3.2(3) for values of $\alpha \geq 1$.

(4)P The nosing force shall always be combined with a vertical traffic load.

6.5.3 Actions due to traction and braking

(1)P Traction and braking forces act at the top of the rails in the longitudinal direction of the track. They shall 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 shall take account of the permitted direction(s) of travel on each track.

(2)P The characteristic values of traction and braking forces shall be taken as follows:

Traction force: $Q_{lak} = 33\ [\text{kN/m}]\ L_{a,b}\ [\text{m}] \leq 1000\ [\text{kN}]$ (6.20)
for Load Models 71,
SW/0, SW/2 and HSLM

Braking force: $Q_{lbk} = 20\ [\text{kN/m}]\ L_{a,b}\ [\text{m}] \leq 6000\ [\text{kN}]$ (6.21)
for Load Models 71,
SW/0 and Load Model HSLM

$Q_{lbk} = 35\ [\text{kN/m}]\ L_{a,b}\ [\text{m}]$ (6.22)
for Load Model SW/2

The characteristic values of traction and braking forces shall not be multiplied by the factor Φ (see 6.4.5.2) or by the factor f in 6.5.1(6).

NOTE 1 For Load Models SW/0 and SW/2 traction and braking forces need only to be applied to those parts of the structure which are loaded according to Figure 6.2 and Table 6.1.

NOTE 2 Traction and braking may be neglected for the Load Model "unloaded train".

(3) These characteristic values are applicable to all types of track construction, *e.g.* continuous welded rails or jointed rails, with or without expansion devices.

(4) The above traction and braking forces for Load Models 71 and SW/0 should be multiplied by the factor α in accordance with the requirements of 6.3.2(3).

(5) For loaded lengths greater than 300m additional requirements for taking into account the effects of braking should be specified.

NOTE The National Annex or individual project may specify the additional requirements.

(6) 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}. The lines carrying special traffic and associated loading details may be specified.

NOTE 1 The individual project may specify the requirements.

NOTE 2 Where the individual project specifies reduced traction and braking loading in accordance with the above the specified loading should take into account other traffic permitted to use the line, *e.g.* trains for track maintenance etc.

(7)P Traction and braking forces shall be combined with the corresponding vertical loads.

(8) 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 6.5.4.

(9)P In the case of a bridge carrying two or more tracks the braking forces on one track shall be considered with the traction forces on one other track.
Where two or more tracks have the same permitted direction of travel either traction on two tracks or braking on two tracks shall be taken into account.

NOTE For bridges carrying two or more tracks with the same permitted direction of travel the National Annex may specify alternative requirements for the application of traction and braking forces.

6.5.4 Combined response of structure and track to variable actions

6.5.4.1 General principles

(1) Where the rails are continuous over discontinuities in the support to the track (*e.g.* between a bridge structure and an embankment) the structure of the bridge (bridge deck, bearings and substructure) and the track (rails, ballast etc.) jointly resist the longitudinal

actions due to traction or braking. 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.

NOTE References to embankment throughout 6.5.4 may also be taken as references to the track formation or ground beneath the track on the approaches to the bridge whether the track is on an embankment, level ground or in a cutting.

(2) Where continuous rails restrain the free movement of the bridge deck, deformations of the bridge deck (*e.g.* due to thermal variations, vertical loading, creep and shrinkage) produce longitudinal forces in the rails and in the fixed bridge bearings.

(3)P The effects resulting from the combined response of the structure and the track to variable actions shall be taken into account for the design of the bridge superstructure, fixed bearings, the substructure and for checking load effects in the rails.

(4) The requirements of 6.5.4 are valid for conventional ballasted track.

(5) The requirements for non-ballasted track should be specified.

NOTE The requirements for non-ballasted track may be specified in either the National Annex or for the individual project.

6.5.4.2 Parameters affecting the combined response of the structure and track

(1)P The following parameters influence the combined behaviour of the structure and track and shall be taken into account in the analysis:

a) Configuration of the structure:
   - simply supported beam, continuous beams or a series of beams,
   - number of individual decks and length of each deck,
   - number of spans and length of each span,
   - position of fixed bearings,
   - position of the thermal fixed point,
   - expansion length L_T between the thermal fixed point and the end of the deck.

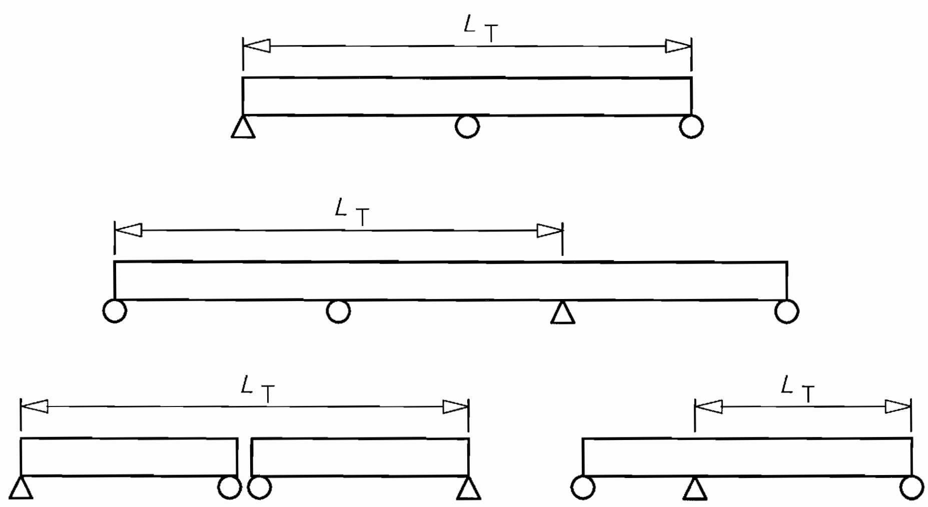


Figure 6.17 - Examples of expansion length L_T

b) Configuration of the track:
   - ballasted track or non-ballasted track systems,
   - vertical distance between the upper surface of the deck and the neutral axis of the rails,
   - location of rail expansion devices.

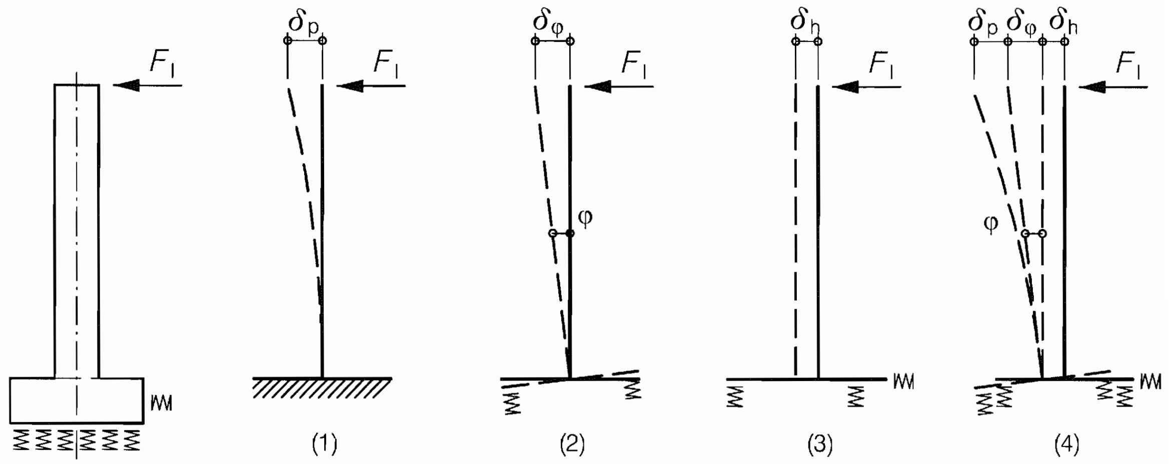
NOTE The individual project may specify requirements regarding the location of rail expansion devices taking into account requirements to ensure such devices are effective whilst ensuring that the rail expansion devices are not adversely affected by bending effects in the rail due to the close proximity of the end of a bridge deck etc.

c) Properties of the structure:
   - vertical stiffness of the deck,
   - vertical distance between the neutral axis of the deck and the upper surface of the deck,
   - vertical distance between the neutral axis of the deck and the axis of rotation of the bearing,
   - structural configuration at bearings generating longitudinal displacement of the end of the deck from angular rotation of the deck,
   - longitudinal stiffness of the structure defined as the total stiffness which can be mobilised by the substructure against actions in the longitudinal direction of the tracks taking into account the stiffness of the bearings, substructure and foundations.

For example the total longitudinal stiffness of a single pier is given by:

$$K = \frac{F_1}{(\delta_p + \delta_\varphi + \delta_h)} \tag{6.23}$$

for the case represented below as an example.



Key
(1) Bending of the pier
(2) Rotation of the foundation
(3) Displacement of the foundation
(4) Total displacement of the pier head

Figure 6.18 - Example of the determination of equivalent longitudinal stiffness at bearings

d) Properties of the track:
   - axial stiffness of the rail,
   - resistance of the track or the rails against longitudinal displacement considering either:
     - resistance against displacement of the track (rails and sleepers) in the ballast relative to the underside of the ballast, or
     - resistance against displacement of the rails from rail fastenings and supports *e.g.* with frozen ballast or with directly fastened rails,

   where the resistance against displacement is the force per unit length of the track that acts against the displacement as a function of the relative displacement between rail and the supporting deck or embankment.

6.5.4.3 Actions to be considered

(1)P The following actions shall be taken into account:
- traction and braking forces as defined in 6.5.3.

- Thermal effects in the combined structure and track system.
- Classified vertical traffic loads (including SW/0 and SW/2 where required). Associated dynamic effects may be neglected.

NOTE The combined response of the structure and track to the "unloaded train" and load model HSLM may be neglected.

- Other actions such as creep, shrinkage, temperature gradient etc. shall be taken into account for the determination of rotation and associated longitudinal displacement of the end of the decks where relevant.

(2) Temperature variations in the bridge should be taken as ΔT_N (see EN 1991-1-5), with γ and ψ taken as 1,0.

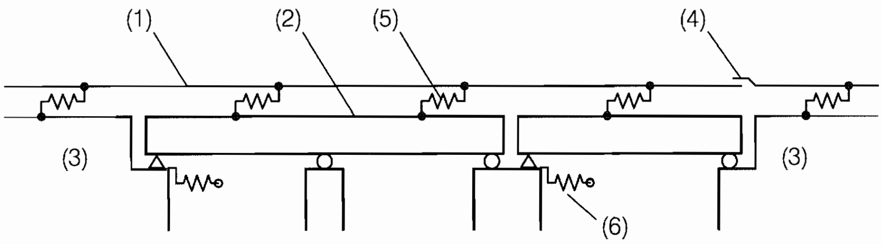
NOTE 1 The National Annex may specify alternative values of ΔT_N. The values given in EN 1991-1-5 are recommended.

NOTE 2 For simplified calculations a temperature variation of the superstructure of $\Delta T_N = \pm 35$ Kelvin may be taken into account. Other values may be specified in the National Annex or for the individual project.

(3) When determining the combined response of track and structure to traction and braking forces, the traction and braking forces should not be applied on the adjacent embankment unless a complete analysis is carried out considering the approach, passage over and departure from the bridge of rail traffic on the adjacent embankments to evaluate the most adverse load effects.

6.5.4.4 Modelling and calculation of the combined track/structure system

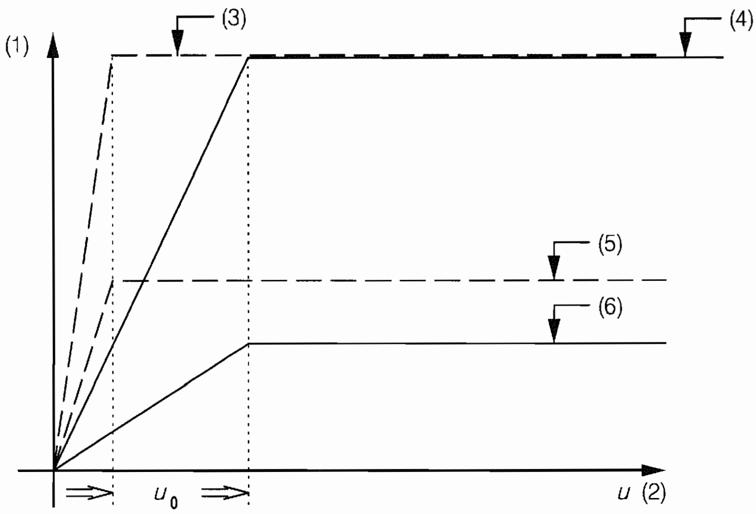
(1) For the determination of load effects in the combined track/structure system a model based upon Figure 6.19 may be used.



Key
(1) Track
(2) Superstructure (a single deck comprising two spans and a single deck with one span shown)
(3) Embankment
(4) Rail expansion device (if present)
(5) Longitudinal non-linear springs reproducing the longitudinal load/ displacement behaviour of the track
(6) Longitudinal springs reproducing the longitudinal stiffness K of a fixed support to the deck taking into account the stiffness of the foundation, piers and bearings etc.

Figure 6.19 - Example of a model of a track/structure system

(2) The longitudinal load/ displacement behaviour of the track or rail supports may be represented by the relationship shown in Figure 6.20 with an initial elastic shear resistance [kN/mm of displacement per m of track] and then a plastic shear resistance k [kN/m of track].



Key

(1) Longitudinal shear force in the track per unit length
(2) Displacement of the rail relative to the top of the supporting deck
(3) Resistance of the rail in sleeper (loaded track)
(frozen ballast or track without ballast with conventional fastenings)
(4) Resistance of sleeper in ballast (loaded track)
(5) Resistance of the rail in sleeper (unloaded track)
(frozen ballast or track without ballast with conventional fastenings)
(6) Resistance of sleeper in ballast (unloaded track)

Figure 6.20 - Variation of longitudinal shear force with longitudinal track displacement for one track

NOTE 1 The values of longitudinal resistance used for the analysis of rail/ballast/bridge stiffness may be given in the National Annex or agreed with the relevant authority specified in the National Annex.

NOTE 2 The behaviour described in Figure 6.20 is valid in most cases (but not for embedded rails without conventional rail fastenings etc.).

(3)P Where it can be reasonably foreseen that the track characteristics may change in the future, this shall be taken into account in the calculations in accordance with the specified requirements.

NOTE The individual project may specify the requirements.

(4)P For the calculation of the total longitudinal support reaction F_L and in order to compare the global equivalent rail stress with permissible values, the global [AC1⟩effect shall be calculated⟨AC1] as follows:

$$F_L = \sum \psi_{0i} F_{li} \qquad (6.24)$$

with:

F_{li} the individual longitudinal support reaction corresponding to the action i,

ψ_{0i} for the calculation of load effects in the superstructure, bearings and substructures the combination factors defined in EN 1990 A2 shall be used,

ψ_{0i} for the calculation of rail stresses, ψ_{0i} shall be taken as 1,0.

(5) When determining the effect of each action the non-linear behaviour of the track stiffness shown in Figure 6.20 should be taken into account.

(6) The longitudinal forces in the rails and bearings resulting from each action may be combined using linear superimposition.

6.5.4.5 Design criteria

NOTE Alternative requirements may be specified in the National Annex.

6.5.4.5.1 Track

(1) 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 should be limited to the following design values:
- Compression: 72 N/mm²,
- Tension: 92 N/mm².

(2) The limiting values for the rail stresses given in 6.5.4.5.1(1) are valid for track complying with:
- UIC 60 rail with a tensile strength of at least 900 N/mm²,
- straight track or track radius $r \geq 1\ 500$ m,

  NOTE For ballasted tracks with additional lateral restraints to the track and for directly fastened tracks this minimum value of track radius may be reduced subject to the agreement of the relevant authority specified in the National Annex.

- for ballasted tracks with heavy concrete sleepers with a maximum spacing of 65 cm or equivalent track construction,
- for ballasted tracks with at least 30 cm consolidated ballast under the sleepers.

When the above criteria are not satisfied special studies should be carried out or additional measures provided.

NOTE For other track construction standards (in particular those that affect lateral resistance) and other types of rail it is recommended that the maximum additional rail stresses is specified in the National Annex or for the individual project.

6.5.4.5.2 Limiting values for the deformation of the structure

(1)P Due to traction and braking δ_B [mm] shall not exceed the following values:
- 5 mm for continuous welded rails without rail expansion devices or with a rail expansion device at one end of the deck,
- 30 mm for rail expansion devices at both ends of the deck where the ballast is continuous at the ends of the deck,
- movements exceeding 30 mm shall only be permitted where the ballast is provided with a movement gap and rail expansion devices provided.

where δ_B [mm] is:

- the relative longitudinal displacement between the end of a deck and the adjacent abutment or,
- the relative longitudinal displacement between two consecutive decks.

(2)P For vertical traffic actions (up to two tracks loaded with load model LM 71 (and where required SW/0) δ_H [mm] shall not exceed the following values:
- 8 mm when the combined behaviour of structure and track is taken into account (valid where there is only one or no expansion devices per deck),
- 10 mm when the combined behaviour of the structure and track is neglected.

where δ_H [mm] is:

- the longitudinal displacement of the upper surface of the deck at the end of a deck due to deformation of the deck.

NOTE Where either the permissible additional stresses in the rail in 6.5.4.5.1(1) are exceeded or the longitudinal displacement of the deck in 6.5.4.5.2(1) or 6.5.4.5.2(2) is exceeded either change the structure or provide rail expansion devices.

(3)P The vertical displacement of the upper surface of a deck relative to the adjacent construction (abutment or another deck) δ_V [mm] due to variable actions shall 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.

(4)P For directly fastened rails the uplift forces (under vertical traffic loads) on rail supports and fastening systems shall be checked against the relevant limit state (including fatigue) performance characteristics of the rail supports and fastening systems.

6.5.4.6 Calculation methods

NOTE Alternative calculation methods may be specified in the National Annex or for the individual project.

(1) The following calculation methods enable the combined response of the track and structure to be checked against the design criteria given in 6.5.4.5. The design criteria for ballasted decks may be summarised as:

a) Longitudinal relative displacement at the end of the deck split into two components to enable comparison with the permitted values: δ_B due to braking and traction and δ_H due to vertical deformation of the deck,
b) Maximum additional stresses in the rails,
c) Maximum vertical relative displacement at the end of the deck, δ_V.

For directly fastened decks an additional check on uplift forces is required in accordance with 6.5.4.5.2(4).

(2) In 6.5.4.6.1 a simplified method is given for estimating the combined response of a simply supported or a continuous structure consisting of single bridge deck and track to variable actions for structures with an expansion length L_T of up to 40m.

(3) For structures that do not satisfy the requirements of 6.5.4.6.1 a method is given in annex G for determining the combined response of a structure and track to variable actions for:

- simply supported or a continuous structure consisting of a single bridge deck,
- structures consisting of a succession of simply supported decks,
- structures consisting of a succession of continuous single piece decks.

(4) Alternatively, or for other track or structural configurations, an analysis may be carried out in accordance with the requirements of 6.5.4.2 to 6.5.4.5.

6.5.4.6.1 Simplified calculation method for a single deck

(1) For a superstructure comprising of a single deck (simply supported, continuous spans with a fixed bearing at one end or continuous spans with an intermediate fixed bearing) it is not necessary to check the rail stresses providing:

- the substructure has sufficient stiffness, K to limit δ_B, the displacement of the deck in the longitudinal direction due to traction and braking, to a maximum of 5 mm under the longitudinal forces due to traction and braking defined in 6.5.4.6.1(2) (classified in accordance with 6.3.2(3) where required). For the determination of the displacements the configuration and properties of the structure given in 6.5.4.2(1) should be taken into account.
- for vertical traffic actions δ_H, the longitudinal displacement of the upper surface of the deck at the end of the deck due to deformation of the deck does not exceed 5mm,
- expansion length L_T is less than 40m,

NOTE Alternative criteria may be specified in the National Annex. The criteria given in this clause are recommended.

(2) The limits of validity of the calculation method in 6.5.4.6.1 are:

- track complies with the construction requirements given in 6.5.4.5.1(2).
- longitudinal plastic shear resistance k of the track is:
  - unloaded track: k = 20 to 40 kN per m of track,
  - loaded track: k = 60 kN per m of track.
- vertical traffic loading:
  - Load Model 71 (and where required Load Model SW/0) with $\alpha = 1$ in accordance with 6.3.2(3),
  - Load Model SW/2,

  NOTE The method is valid for values of α where the load effects from α x LM71 are less than or equal to the load effects from SW/2.

- actions due to braking for:
  - Load Model 71 (and where required Load Model SW/0) and Load Model HSLM:
  - q_{lbk} = 20 kN/m,
  - Load Model SW/2:
  - q_{lbk} = 35 kN/m.
- actions due to traction:
  - q_{lak} = 33 kN/m, limited to a maximum of Q_{lak} = 1000 kN.
- actions due to temperature:
  - Temperature variation ΔT_D of the deck: $\Delta T_D \leq 35$ Kelvin,

Temperature variation ΔT_R of the rail: $\Delta T_R \leq 50$ Kelvin,
Maximum difference in temperature between rail and deck:
$|\Delta T_D - \Delta T_R| \leq 20$ Kelvin. 6.25

(3) The longitudinal forces due to traction and braking acting on the fixed bearings may be obtained by multiplying the traction and braking forces by the reduction factor ξ given in Table 6.9.

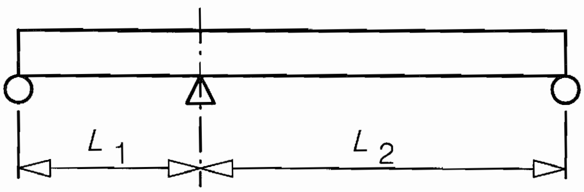
Table 6.9 - Reduction factor ξ for the determination of the longitudinal forces in the fixed bearings of one-piece decks due to traction and braking

Overall length of structure [m]	Reduction factor ξ		
	Continuous track	Rail expansion devices at one end of deck	Rail expansion devices at both ends of deck
≤ 40	0,60	0,70	1,00

NOTE For portal frames and closed frames or boxes it is recommended that the reduction factor ξ be taken as unity. Alternatively the method given in annex G or an analysis in accordance with 6.5.4.2 to 6.5.4.5 may be used.

(4) The characteristic longitudinal forces F_{Tk} per track due to temperature variation (according to 6.5.4.3) acting on the fixed bearings may be obtained as follows :

– for bridges with continuous welded rails at both deck ends and fixed bearings at one end of the deck :
  F_{Tk} [kN] = $\pm$ 0,6 $k\, L_T$ (6.26)
  with k [kN/m] the longitudinal plastic shear resistance of the track per unit length according to 6.5.4.4(2) for unloaded track and L_T [m] the expansion length according to 6.5.4.2(1).
– for bridges with continuous welded rails at both deck ends and fixed bearings situated in a distance L_1 from one end of the deck and L_2 from the other end :
  F_{Tk} [kN] = $\pm$ 0,6 $k\,(L_2 - L_1)$ (6.27)
  with k [kN/m] the longitudinal plastic shear resistance of the track per unit length according to 6.5.4.4(2) for unloaded track and L_1 [m] and L_2 [m] according to Figure 6.21.



N.B. (1) Deck corresponding to either L_1 or L_2 may comprise of one or more spans.

Figure 6.21 - Deck with fixed bearings not located at one end [(1)]

– for bridges with continuous welded rails at the deck end with fixed bearings and rail expansion devices at the free deck end:
  F_{Tk} [kN] = $\pm$ 20 L_T, but $F_{Tk} \leq$ 1100 kN (6.28)
  with L_T [m] expansion length according to 6.5.4.2.(1).

– for bridge decks with rail expansion devices at both ends:
$F_{Tk} = 0$ (6.29)

NOTE For track complying with 6.5.4.5.1(2) values of k may be taken from annex G2(3). Alternative values of k may be specified in the National Annex.

(5) The characteristic longitudinal forces F_{Qk} per track on the fixed bearings due to deformation of the deck may be obtained as follows:

– for bridges with continuous welded rails at both deck ends and fixed bearings on one end of the deck and with rail expansion devices at the free end of the deck:
F_{Qk} [kN] = $\pm$ 20 L (6.30)
with L [m] the length of the first span near the fixed bearing

– for bridges with rail expansion devices at both ends of the deck:
F_{Qk} [kN] = 0 (6.31)

(6) The vertical displacement of the upper surface of a deck relative to the adjacent construction (abutment or another deck) due to variable actions may be calculated ignoring the combined response of the structure and track and checked against the criteria in 6.5.4.5.2(3).

6.6 Aerodynamic actions from passing trains

6.6.1 General

(1)P Aerodynamic actions from passing trains shall be taken into account when designing structures adjacent to railway tracks.

(2) The passing of rail traffic subjects any structure situated near the track to a travelling wave of alternating pressure and suction (see Figures 6.22 to 6.25). The magnitude of the action depends mainly on:
– the square of the speed of the train,
– the aerodynamic shape of the train,
– the shape of the structure,
– the position of the structure, particularly the clearance between the vehicle and the structure.

(3) The actions may be approximated by equivalent loads at the head and rear ends of a train, when checking ultimate and serviceability limit states and fatigue. Characteristic values of the equivalent loads are given in 6.6.2 to 6.6.6.

NOTE The National Annex or the individual project may specify alternative values. The values given in 6.6.2 to 6.6.6. are recommended.

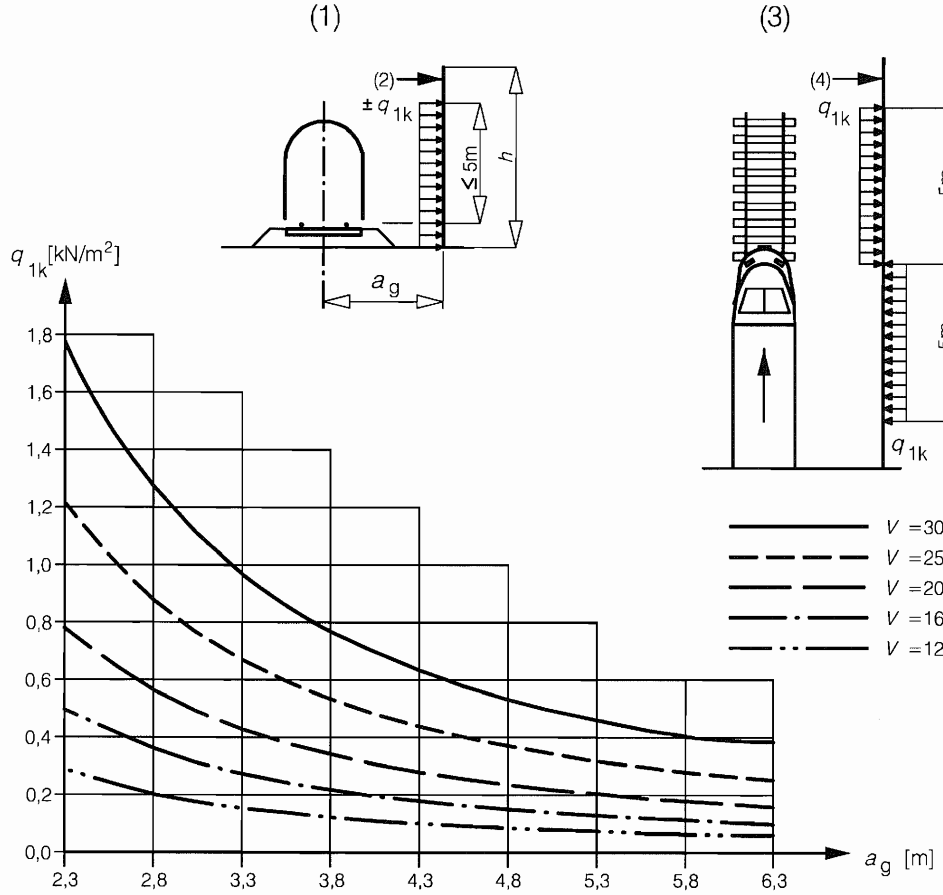
(4) In 6.6.2 to 6.6.6 the Maximum Design Speed V [km/h] should be taken as the Maximum Line Speed at the Site except for cases covered by EN 1990 A2.2.4(6).

(5) At the start and end of structures adjacent to the tracks, for a length of 5 m from the start and end of the structure measured parallel to the tracks the equivalent loads in 6.6.2 to 6.6.6 should be multiplied by a dynamic amplification factor of 2,0.

NOTE For dynamically sensitive structures the above dynamic amplification factor may be insufficient and may need to be determined by a special study. The study should take into account dynamic characteristics of the structure including support and end conditions, the speed of the adjacent rail traffic and associated aerodynamic actions and the dynamic response of the structure including the speed of a deflection wave induced in the structure. In addition, for dynamically sensitive structures a dynamic amplification factor may be necessary for parts of the structure between the start and end of the structure.

6.6.2 Simple vertical surfaces parallel to the track (*e.g.* noise barriers)

(1) The characteristic values of the actions, $\pm q_{1k}$, are given in Figure 6.22.



Key
(1) Section
(2) Surface of structure
(3) Plan view
(4) Surface of structure

Figure 6.22 - Characteristic values of actions q_{1k} for simple vertical surfaces parallel to the track

(2) The characteristic values apply to trains with an unfavourable aerodynamic shape and may be reduced by:

– a factor $k_1 = 0,85$ for trains with smooth sided rolling stock

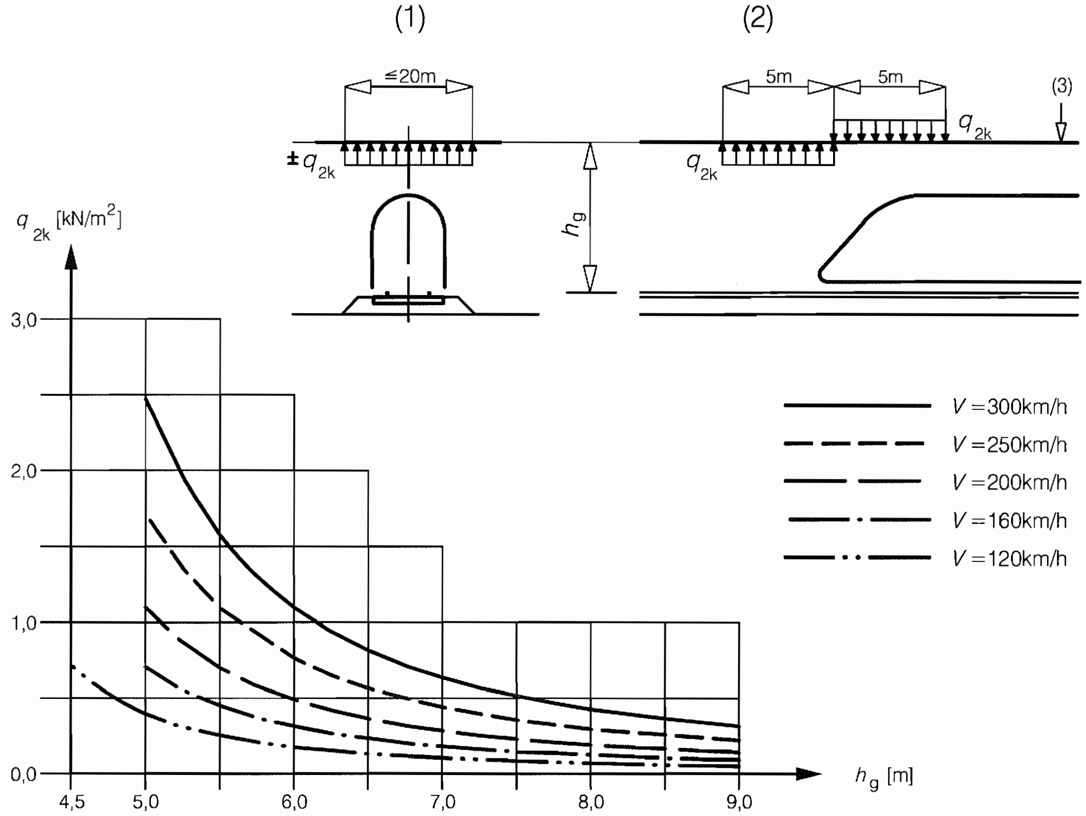
– a factor $k_1 = 0{,}6$ for streamlined rolling stock (*e.g.* ETR, ICE, TGV, Eurostar or similar)

(3) If a small part of a wall with a height ≤ 1,00 m and a length ≤ 2,50 m is considered, *e.g.* an element of a noise protection wall, the actions q_{1k} should be increased by a factor $k_2 = 1{,}3$.

6.6.3 Simple horizontal surfaces above the track (*e.g.* overhead protective structures)

(1) The characteristic values of the actions, $\pm q_{2k}$, are given in Figure 6.23.

(2) The loaded width for the structural member under investigation extends up to 10 m to either side from the centre-line of the track.



Key
(1) Section
(2) Elevation
(3) Underside of the structure

Figure 6.23 - Characteristic values of actions q_{2k} for simple horizontal surfaces above the track

(3) For trains passing each other in opposite directions the actions should be added. The loading from trains on only two tracks needs to be considered.

(4) The actions q_{2k} may be reduced by the factor k_1 as defined in 6.6.2.

(5) The actions acting on the edge strips of a wide structure which cross the track may be multiplied by a factor of 0,75 over a width up to 1,50 m.

6.6.4 Simple horizontal surfaces adjacent to the track (*e.g.* platform canopies with no vertical wall)

(1) The characteristic values of the actions, $\pm q_{3k}$, are given in Figure 6.24 and apply irrespective of the aerodynamic shape of the train.

(2) For every position along the structure to be designed, q_{3k} should be determined as a function of the distance a_g from the nearest track. The actions should be added, if there are tracks on either side of the structural member under consideration.

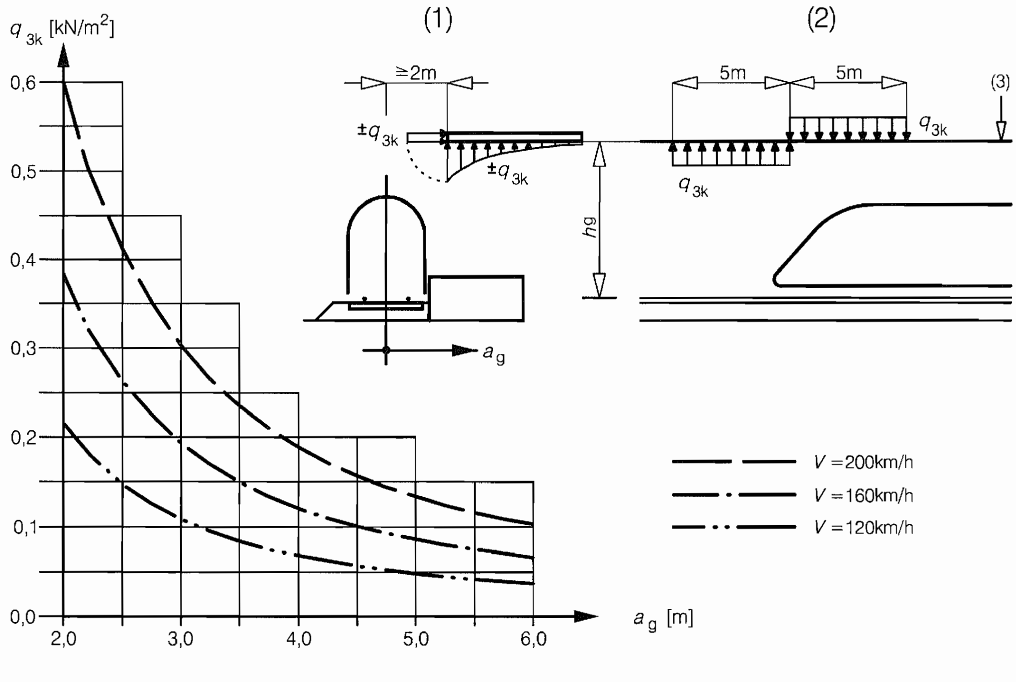
(3) If the distance h_g exceeds 3,80 m the action q_{3k} may be reduced by a factor k_3:

$$k_3 = \frac{(7{,}5 - h_g)}{3{,}7} \quad \text{for } 3{,}8 \text{ m} < h_g < 7{,}5 \text{ m} \qquad (6.32)$$

$$k_3 = 0 \qquad \text{for } h_g \geq 7{,}5 \text{ m} \qquad (6.33)$$

where:

h_g distance from top of rail level to the underside of the structure.



Key
(1) Section
(2) Elevation
(3) Underside of the structure

Figure 6.24 - Characteristic values of actions q_{3k} for simple horizontal surfaces adjacent to the track

6.6.5 Multiple-surface structures alongside the track with vertical and horizontal or inclined surfaces (*e.g.* bent noise barriers, platform canopies with vertical walls etc.)

(1) The characteristic values of the actions, $\pm q_{4k}$, as given in Figure 6.25 should be applied normal to the surfaces considered. The actions should be taken from the graphs in Figure 6.22 adopting a track distance the lesser of:

$a'_g = 0{,}6 \min a_g + 0{,}4 \max a_g$ or 6 m (6.34)

where distances min a_g and max a_g are shown in Figure 6.25.

(2) If max $a_g > 6$ m the value max $a_g = 6$ m should be used.

(3) The factors k_1 and k_2 defined in 6.6.2 should be used.

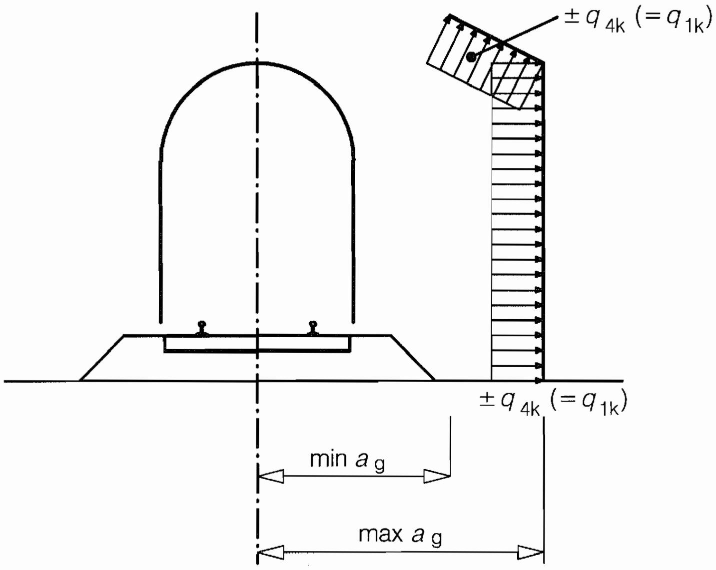


Figure 6.25 - Definition of the distances min a_g and max a_g from centre-line of the track

6.6.6 Surfaces enclosing the structure gauge of the tracks over a limited length (up to 20 m) (horizontal surface above the tracks and at least one vertical wall, *e.g.* scaffolding, temporary constructions)

(1) All actions should be applied irrespective of the aerodynamic shape of the train:
– to the full height of the vertical surfaces:

$\pm k_4 q_{1k}$ (6.35)

where:

q_{1k} is determined according to 6.6.2,
$k_4 = 2$

– to the horizontal surfaces:

$$\pm k_5 q_{2k} \tag{6.36}$$

where:

q_{2k} is determined according to 6.6.3 for only one track,
k_5 = 2,5 if one track is enclosed,
k_5 = 3,5 if two tracks are enclosed.

6.7 Derailment and other actions for railway bridges

(1)P Railway structures shall 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.

6.7.1 Derailment actions from rail traffic on a railway bridge

(1)P Derailment of rail traffic on a railway bridge shall be considered as an Accidental Design Situation.

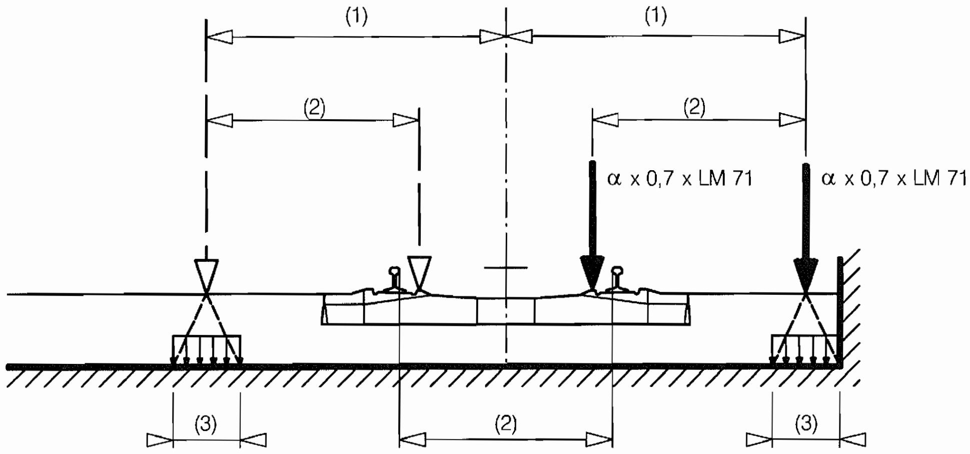
(2)P Two design situations shall be considered:

- 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 National Annex or individual project may specify additional requirements and alternative loading.

(3)P For Design Situation I, collapse of a major part of the structure shall be avoided. Local damage, however, may be tolerated. The parts of the structure concerned shall be designed for the following design loads in the Accidental Design Situation:

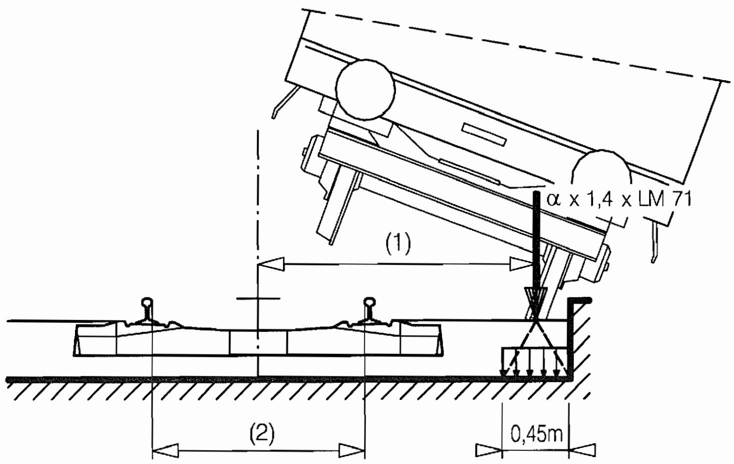
$\alpha \times 1{,}4 \times$ LM 71 (both point loads and uniformly distributed loading, Q_{A1d} and q_{A1d}) parallel to the track in the most unfavourable position inside an area of width 1,5 times the track gauge on either side of the centre-line of the track:



Key
(1) max. 1,5s 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 450mm at the top of the deck.

Figure 6.26 - Design Situation I - equivalent load Q_{A1d} and q_{A1d}

(4)P 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 shall be taken as a uniformly distributed vertical line load acting on the edge of the structure under consideration.



Key
(1) Load acting on edge of structure
(2) Track gauge s

Figure 6.27 - Design Situation II - equivalent load q_{A2d}

NOTE The above-mentioned equivalent load is only to be considered for determining the ultimate strength or the stability of the structure as a whole. Minor structural elements need not be designed for this load.

(5)P Design Situations I and II shall be examined separately. A combination of these loads need not be considered.

(6) For Design Situations I and II other rail traffic actions should be neglected for the track subjected to derailment actions.

NOTE See EN 1990 A2 for the requirements for application of traffic actions to other tracks.

(7) No dynamic factor needs to be applied to the design loads in 6.7.1(3) and 6.7.1(4).

(8)P For structural elements which are situated above the level of the rails, measures to mitigate the consequences of a derailment shall be in accordance with the specified requirements.

NOTE 1 The requirements may be specified in the National Annex or for the individual project.

NOTE 2 The National Annex or individual project may also specify requirements to retain a derailed train on the structure.

6.7.2 Derailment under or adjacent to a structure and other actions for Accidental Design Situations

(1) 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 EN 1991-1-7.

(2) Other actions for Accidental Design Situations are given in EN 1991-1-7 and should be taken into account.

6.7.3 Other actions

(1)P The following actions shall also be taken into account in the design of the structure:

- effects due to inclined decks or inclined bearing surfaces,
- longitudinal anchorage forces from stressing or destressing rails in accordance with the specified requirements.
- longitudinal forces due to the accidental breakage of rails in accordance with the specified requirements.
- actions from catenaries and other overhead line equipment attached to the structure in accordance with the specified requirements.
- actions from other railway infrastructure and equipment in accordance with the specified requirements.

NOTE The specified requirements including actions for any Accidental Design Situation to be taken into account may be specified in the National Annex or for the individual project.

6.8 Application of traffic loads on railway bridges

6.8.1 General

NOTE See 6.3.2 for the application of the factor α and 6.4.5 for the application of the dynamic factor Φ.

(1)P The structure shall be designed for the required number and position(s) of the tracks in accordance with the track positions and tolerances specified.

NOTE The track positions and tolerances may be specified for the individual project.

(2) 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 intended tracks taking into account the minimum spacing of tracks and structural gauge clearance requirements specified.

NOTE The minimum spacing of tracks and structural gauge clearance requirements may be specified for the individual project.

(3)P The effects of all actions shall be determined with the traffic loads and forces placed in the most unfavourable positions. Traffic actions which produce a relieving effect shall be neglected.

(4)P 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} shall be applied to a track and up to four of the individual concentrated loads Q_{vk} shall be applied once per track,
- for structures carrying two tracks, Load Model 71 shall be applied to one track or both tracks,
- for structures carrying three or more tracks, Load Model 71 shall be applied to one track or to two tracks or 0,75 times Load Model 71 to three or more of the tracks.

(5)P For the determination of the most adverse load effects from the application of Load Model SW/0:
- the loading defined in Figure 6.2 and Table 6.1 shall be applied once to a track,
- for structures carrying two tracks, Load Model SW/0 shall be applied to one track or both tracks,
- for structures carrying three or more tracks, Load Model SW/0 shall be applied to one track or to two tracks or 0,75 times Load Model SW/0 to three or more of the tracks.

(6)P For the determination of the most adverse load effects from the application of Load Model SW/2:
- the loading defined in Figure 6.2 and Table 6.1 shall be applied once to a track,
- for structures carrying more than one track, Load Model SW/2 shall be applied to one track only with Load Model 71 or Load Model SW/0 applied to one other track in accordance with 6.8.1(4) and 6.8.1(5).

(7)P 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} shall be applied to a track,
- generally Load Model "unloaded train" shall only be considered in the design of structures carrying one track.

(8)P All continuous beam structures designed for Load Model 71 shall be checked additionally for Load Model SW/0.

(9)P Where a dynamic analysis is required in accordance with 6.4.4 all bridges shall also be designed for the loading from Real trains and Load Model HSLM where required by 6.4.6.1.1. The determination of the most adverse load effects from Real Trains and the application of Load Model HSLM shall be in accordance with 6.4.6.1.1(6) and 6.4.6.5(3).

(10)P For the verification of deformations and vibrations the vertical loading to be applied shall be:

- Load Model 71 and where required Load Models SW/0 and SW/2,
- Load Model HSLM where required by 6.4.6.1.1,
- Real Trains when determining the dynamic behaviour in the case of resonance or excessive vibrations of the deck where required by 6.4.6.1.1.

(11)P For bridge decks carrying one or more tracks the checks for the limits of deflection and vibration shall be made with the number of tracks loaded with all associated relevant traffic actions in accordance with Table 6.10. Where required by 6.3.2(3) classified loads shall be taken into account.

Table 6.10 - Number of tracks to be loaded for checking limits of deflection and vibration

Limit State and associated acceptance criteria	Number of tracks on the bridge		
	1	2	≥ 3
Traffic Safety Checks:			
– Deck twist (EN 1990: A2.4.4.2.2)	1	1 or 2 [a]	1 or 2 or 3 or more [b]
– Vertical deformation of the deck (EN 1990: A2.4.4.2.3)	1	1 or 2 [a]	1 or 2 or 3 or more [b]
– Horizontal deformation of the deck (EN 1990: A2.4.4.2.4)	1	1 or 2 [a]	1 or 2 or 3 or more [b]
– Combined response of structure and track to variable actions including limits to vertical and longitudinal displacement of the end of a deck (6.5.4)	1	1 or 2 [a]	1 or 2 [a]
– Vertical acceleration of the deck (6.4.6 and EN 1990: A2.4.4.2.1)	1	1	1
SLS Checks:			
– Passenger comfort criteria (EN 1990: A2.4.4.3)	1	1	1
ULS Checks			
– Uplift at bearings (EN 1990: A2.4.4.1(2)P)	1	1 or 2 [a]	1 or 2 or 3 or more [b]

[a] Whichever is critical

[b] Where groups of loads are used the number of tracks to be loaded should be in accordance with Table 6.11. Where groups of loads are not used the number of tracks to be loaded should also be in accordance with Table 6.11.

NOTE Requirements for the number of tracks to be considered loaded when checking drainage and structural clearance requirements may be specified in the National Annex or for the individual project.

6.8.2 Groups of Loads - Characteristic values of the multicomponent action

(1) The simultaneity of the loading defined in 6.3 to 6.5 and 6.7 may be taken into account by considering the groups of loads defined in Table 6.11. Each of these groups of loads, which are mutually exclusive, should be considered as defining a single variable characteristic action for combination with non-traffic loads. Each Group of Loads should be applied as a single variable action.

NOTE In some cases it is necessary to consider other appropriate combinations of unfavourable individual traffic actions. See A2.2.6(4) of EN 1990.

(2) The factors given in the Table 6.11 should be applied to the characteristic values of the different actions considered in each group.

NOTE All the proposed values given for these factors may be varied in the National Annex. The values in Table 6.11 are recommended.

(3)P Where groups of loads are not taken into account rail traffic actions shall be combined in accordance with Table A2.3 of EN 1990.

Table 6.11 - Assessment of Groups of Loads for rail traffic (characteristic values of the multicomponent actions)

number of tracks on structure			Groups of loads			Vertical forces			Horizontal forces			Comment
			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	LM 71 (1) SW/0 (1), (2) HSLM (6)(7)	SW/2 (1),(3)	Unloaded train	Traction, Braking (1)	Centrifugal force (1)	Nosing force (1)	
			1	gr11	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 T_2	1 1			1 (5) 1 (5)	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	Max. vertical 1 with max longitudinal
			2	gr 22	T_1 T_2	1 1			0,5 (5) 0,5 (5)	1 (5) 1 (5)	1 (5) 1 (5)	Max. vertical 2 with max. transverse
			2	gr 23	T_1 T_2	1 (4) 1 (4)			1 1	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	Max. longitudinal
			2	gr 24	T_1 T_2	1 (4) 1 (4)			0,5 (5) 0,5 (5)	1 1	1 1	Max. lateral
			2	gr 26	T_1 T_2	 1	1 		1 (5) 1 (5)	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	SW/2 with max. longitudinal
			2	gr 27	T_1 T_2	 1	1 		0,5 (5) 0,5 (5)	1 (5) 1 (5)	1 (5) 1 (5)	SW/2 with max. transverse
			≥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, ...) shall be taken into account.
(2) SW/0 shall only be taken into account for continuous beam structures.
(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 shall be taken equal to zero.
(6) HSLM and Real Trains where required in accordance with 6.4.4 and 6.4.6.1.1.
(7) If a dynamic analysis is required in accordance with 6.4.4 see also 6.4.6.5(3) and 6.4.6.1.2.
(8) See also Table A2.3 of EN 1990

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 shall be considered as either T_1 (Track one) 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 shall 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.

6.8.3 Groups of Loads - Other representative values of the multicomponent actions

6.8.3.1 Frequent values of the multicomponent actions

(1) Where Groups of Loads are taken into account the same rule as in 6.8.2(1) above is applicable by applying the factors given in Table 6.11 for each Group of Loads, to the frequent values of the relevant actions considered in each Group of Loads.

NOTE The frequent values of the multicomponent actions may be defined in the National Annex. The rules given in this clause are recommended.

(2)P Where Groups of Loads are not used rail traffic actions shall be combined in accordance with Table A2.3 of EN 1990.

6.8.3.2 Quasi-permanent values of the multicomponent actions

(1) Quasi-permanent traffic actions should be taken as zero.

NOTE The quasi-permanent values of the multicomponent actions may be defined in the National Annex. The value given in this clause is recommended.

6.8.4 Traffic loads in Transient Design Situations

(1)P Traffic loads for Transient Design Situations shall be defined.

NOTE Some indications are given in annex H. The traffic loads for Transient Design Situations may be defined for the individual project.

6.9 Traffic loads for fatigue

(1)P A fatigue damage assessment shall be carried out for all structural elements, which are subjected to fluctuations of stress.

(2) For normal traffic based on characteristic values of Load Model 71, including the dynamic factor Φ, the fatigue assessment should be carried out on the basis of the traffic mixes, "standard traffic", "traffic with 250 kN-axles" or “light traffic mix” depending on whether the structure carries mixed traffic, predominantly heavy freight traffic or lightweight passenger traffic in accordance with the requirements specified. Details of the service trains and traffic mixes considered and the dynamic enhancement to be applied are given in annex D.

NOTE The requirements may be defined for the individual project.

(3) Where the traffic mix does not represent the real traffic (*e.g.* in special situations where a limited number of vehicle type(s) dominate the fatigue loading or for traffic requiring a value of α greater than unity in accordance with 6.3.2(3)) an alternative traffic mix should be specified.

NOTE The alternative traffic mix may be defined for the individual project.

(4) Each of the mixes is based on an annual traffic tonnage of 25×10^6 tonnes passing over the bridge on each track.

(5)P For structures carrying multiple tracks, the fatigue loading shall be applied to a maximum of two tracks in the most unfavourable positions.

(6) The fatigue damage should be assessed over the design working life.

NOTE The design working life may be specified in the National Annex. 100 years is recommended. See also EN 1990.

(7) Alternatively, the fatigue assessment may be carried out on the basis of a special traffic mix.

NOTE A special traffic mix may be specified in the National Annex or for the individual project.

(8) Additional requirements for the fatigue assessment of bridges where a dynamic analysis is required in accordance with 6.4.4 when dynamic effects are likely to be excessive are given in 6.4.6.6.

(9) Vertical rail traffic actions including dynamic effects and centrifugal forces should be taken into account in the fatigue assessment. Generally nosing and longitudinal traffic actions may be neglected in the fatigue assessment.

NOTE In some special situations, for example bridges supporting tracks at terminal stations, the effect of longitudinal actions should be taken into account in the fatigue assessment.

Annex A
(informative)
Models of special vehicles for road bridges

A.1 Scope and field of application

(1) This annex defines standardised models of special vehicles that can be used for the design of road bridges.

(2) The special vehicles defined in this annex are intended to produce global as well as local effects such as are caused by vehicles which do not comply with the national regulations concerning limits of weights and, possibly, dimensions of normal vehicles.

NOTE The consideration of special vehicles for bridge design is intended to be limited to particular cases.

(3) This annex also provides guidance in case of simultaneous application on a bridge carriageway of special vehicles and normal road traffic represented by Load Model 1 defined in 4.3.2.

A.2 Basic models of special vehicles

(1) Basic models of special vehicles are conventionally defined in Tables A.1 and A.2, and in Figure A.1.

NOTE 1 The basic models of special vehicles correspond to various levels of abnormal loads that can be authorised to travel on particular routes of the European highway network.

NOTE 2 Vehicle widths of 3,00 m for the 150 and 200 kN axle-lines, and of 4,50 m for the 240 kN axle-lines are assumed.

Table A1 - Classes of special vehicles

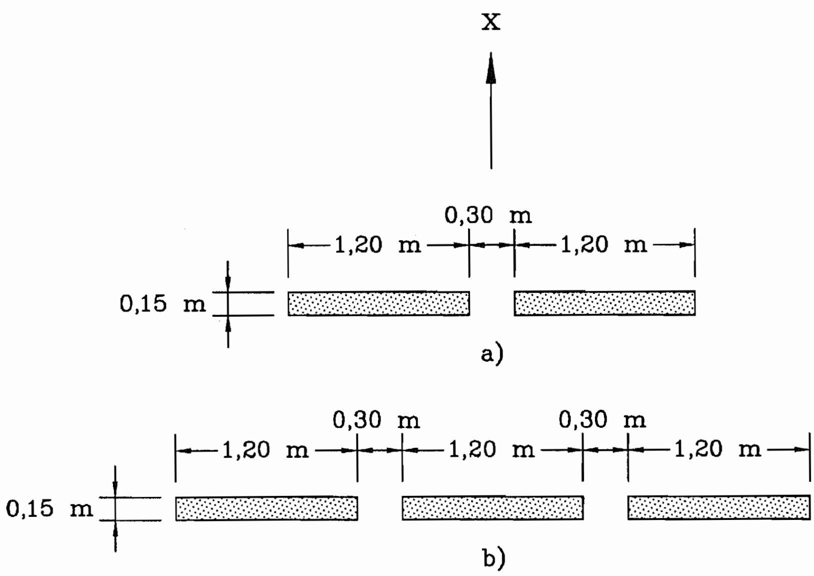
Total weight	Composition	Notation
600 kN	4 axle-lines of 150 kN	600/150
900 kN	6 axle-lines of 150 kN	900/150
1200 kN	8 axle-lines of 150 kN or 6 axle-lines of 200 kN	1200/150 1200/200
1500 kN	10 axle-lines of 150 kN or 7 axle-lines of 200 kN + 1 axle line of 100 kN	1500/150 1500/200
1800 kN	12 axle-lines of 150 kN or 9 axle-lines of 200 kN	1800/150 1800/200
2400 kN	12 axle-lines of 200 kN or 10 axle-lines of 240 kN or 6 axle-lines of 200 kN (spacing 12m) + 6 axle-lines of 200 kN	2400/200 2400/240 2400/200/200

3000 kN	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	3000/200 3000/240 3000/200/200
3600 kN	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

Table A2 - Description of special vehicles

	Axle-lines of 150 kN	Axle-lines of 200 kN	Axle-lines of 240 kN
600 kN	$n = 4\times150$ $e = 1{,}50$ m		
900 kN	$n = 6\times150$ $e = 1{,}50$ m		
1200 kN	$n = 8\times150$ $e = 1{,}50$ m	$n = 6\times200$ $e = 1{,}50$ m	
1500 kN	$n = 10\times150$ $e = 1{,}50$ m	$n = 1\times100 + 7\times200$ $e = 1{,}50$ m	
1800 kN	$n = 12\times150$ $e = 1{,}50$ m	$n = 9\times200$ $e = 1{,}50$ m	
2400 kN		$n = 12\times200$ $e = 1{,}50$ m $n = 6\times200 + 6\times200$ $e = 5\times1{,}5+12+5\times1{,}5$	[AC1⟩ $n = 10\times240$ ⟨AC1] $e = 1{,}50$ m
3000 kN		$n = 15\times200$ $e = 1{,}50$ m $n = 8\times200 + 7\times200$ $e = 7\times1{,}5+12+6\times1{,}5$	[AC1⟩ $n = 1\times120$ $+ 12\times240$ ⟨AC1] $e = 1{,}50$ m
3600 kN		$n = 18\times200$ $e = 1{,}50$ m	[AC1⟩ $n = 15\times240$ ⟨AC1] $e = 1{,}50$ m $n = 8\times240 + 7\times240$ $e = 7\times1{,}5+12+6\times1{,}5$

NOTE
n number of axles multiplied by the weight (kN) of each axle in each group
e axle spacing (m) within and between each group.



Key
x Bridge axis direction
a) 100 to 200 kN axle-lines
b) 240 axle-lines

Figure A.1 - Arrangement of axle-lines and definition of wheel contact areas

(1) One or more of the models of special vehicles may have to be taken into account.

NOTE 1 The models and the load values and dimensions may be defined for the individual project.

NOTE 2 The effects of the 600/150 standardised model are covered by the effects of Load Model 1 where applied with α_{Qi} and α_{qi} factors all equal to 1.

NOTE 3 Particular models, especially to cover the effects of exceptional loads with a gross weight exceeding 3600 kN, may have to be defined for the individual project.

(3) The characteristic loads associated with the special vehicles should be taken as nominal values and should be considered as associated solely with transient design situations.

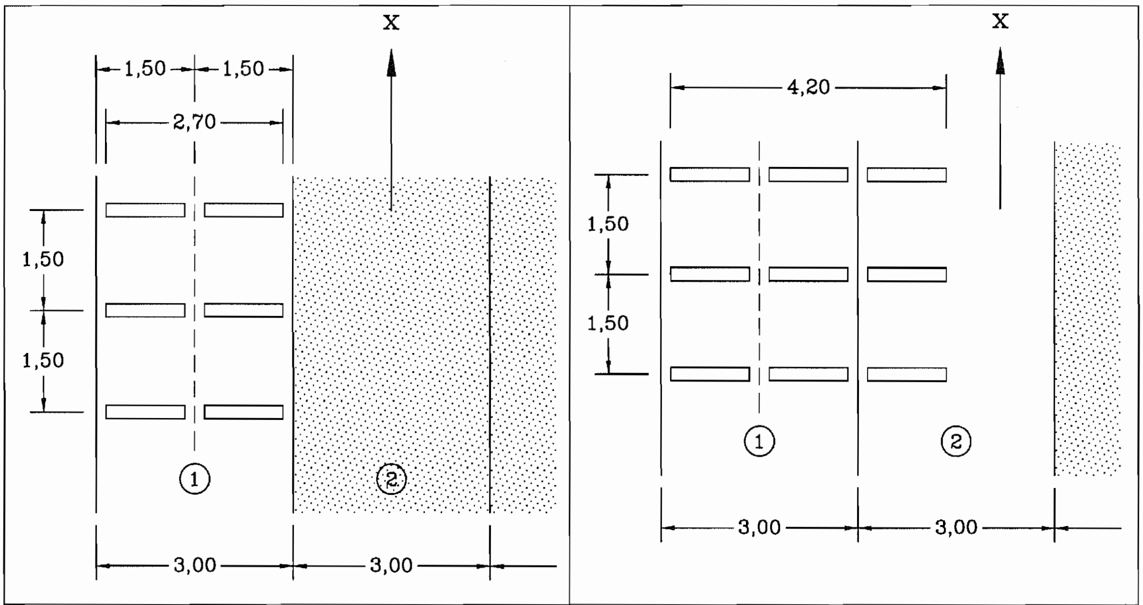
A.3 Application of special vehicle load models on the carriageway

Each standardised model should be applied :

– on one notional traffic lane as defined in 1.4.2 and 4.2.3 (considered as Lane Number 1) for the models composed of 150 or 200 kN axle-lines, or

– on two adjacent notional lanes (considered as Lanes Number 1 and 2 - see Figure A.2) for models composed of 240 kN axle-lines.

(2) 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.



Key	Key
Axle-lines of 150 or 200 kN (b = 2,70 m) X : Bridge axis direction (1) Lane 1 (2) Lane 2	Axle-lines of 240 kN (b = 4,20 m) X : Bridge axis direction (1) Lane 1 (2) Lane 2

Figure A.2 - Application of the special vehicles on notional lanes

(3) Depending on the models under consideration, these models may be assumed to move at low speed (not more than 5 km/h) or at normal speed (70 km/h).

(4) Where the models are assumed to move at low speed, only vertical loads without dynamic amplification should be taken into account.

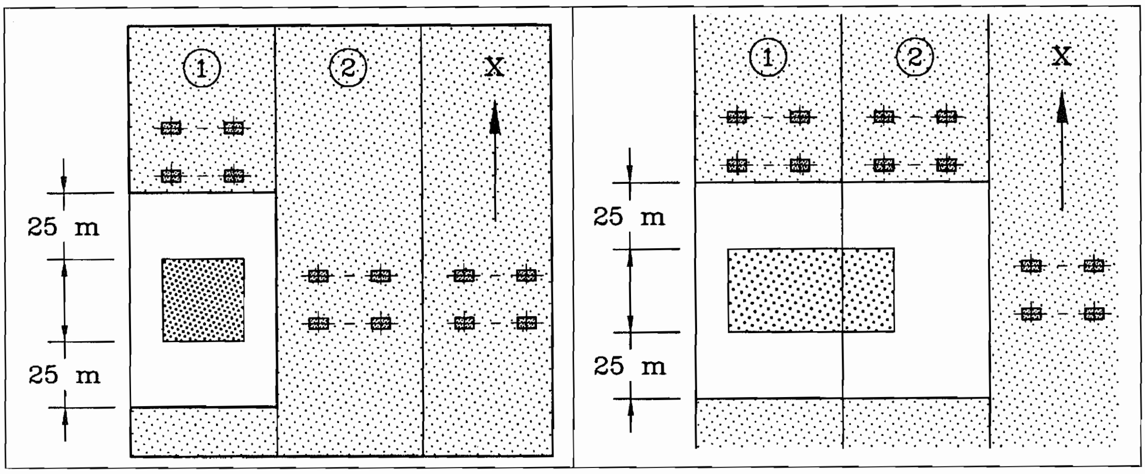
(5) Where the models are assumed to move at normal speed, a dynamic amplification should be taken into account. The following formula may be used :

$$\varphi = 1,40 - \frac{L}{500} \quad \varphi \geq 1$$

where :

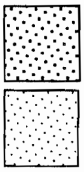
L influence length (m)

(6) Where the models are assumed to move at low speed, each notional lane and the remaining area of the bridge deck should be loaded by Load Model 1 with its frequent values defined in 4.5 and in A2 to EN 1990. On the lane(s) occupied by the standardised vehicle, this system should not be applied at less than 25 m from the outer axles of the vehicle under consideration (see Figure A.3).



Key
Axle-lines of 150 or 200 kN
X : Bridge axis direction
(1) Lane 1
(2) Lane 2

Key
Axle-lines of 240 kN
X : Bridge axis direction
(1) Lane 1
(2) Lane 2



Standardised vehicle

Area loaded with the frequent model of LM1

NOTE A more favourable transverse position for some special vehicles and a restriction of simultaneous presence of general traffic may be defined for the individual project.

Figure A.3 - Simultaneity of Load Model 1 and special vehicles

(7) Where special vehicles are assumed to move at normal speed, a pair of special vehicles should be used in the lane(s) occupied by these vehicles. On the other lanes and the remaining area the bridge deck should be loaded by Load Model 1 with its frequent values defined in 4.5 and in EN 1990, A2.

Annex B
(informative)
Fatigue life assessment for road bridges Assessment method based on recorded traffic

(1) A stress history should be obtained by analysis using recorded representative real traffic data, multiplied by a dynamic amplification factor φ_{fat}.

(2) This dynamic amplification factor should take into account the dynamic behaviour of the bridge and depends on the expected roughness of the road surface and on any dynamic amplification already included in the records.

NOTE In accordance with ISO 8608[7], 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. G_d is a function of the spatial frequency n, $G_d(n)$, or of the angular spatial frequency of the path Ω, $G_d(\Omega)$, with $\Omega=2\pi n$. The actual power spectral density of the road profile should be smoothed and then fitted, in the bi-logarithmic presentation plot, by a straight line in an appropriate spatial frequency range. The fitted PSD can be expressed in a general form as

$$G_d(n) = G_d(n_0)\left(\frac{n}{n_0}\right)^{-w} \qquad \text{or} \qquad G_d(\Omega) = G_d(\Omega_0)\left(\frac{\Omega}{\Omega_0}\right)^{-w}$$

where :

- n_0 is the reference spatial frequency (0,1 cycle/m),
- Ω_0 is the reference angular spatial frequency (1 rd/m),
- w is the exponent of the fitted PSD.

Often, instead of displacement PSD, G_d, it is convenient to consider velocity PSD, G_v, in terms of change of the vertical ordinate of the road surface per unit distance travelled. Since the relationships between G_v and G_d are :

$$G_v(n) = G_d(n)(2\pi n)^2 \qquad \text{and} \qquad G_v(\Omega) = G_d(\Omega)(\Omega)^2$$

When w=2 the two expressions of velocity PSD are constant.

Considering constant velocity PSD, 8 different classes of roads (A, B, ..., H) with increasing roughness are considered in ISO 8608. The class limits are graphed versus the displacement PSD in Figure B.1. For road bridge pavement classification only the first 5 classes (A, B, ..., E) are relevant.

Quality surface may be assumed very good for road surfaces in class A, good for surfaces in class B, medium for surfaces in class C, poor for surfaces in class D and very poor for surfaces in class E.

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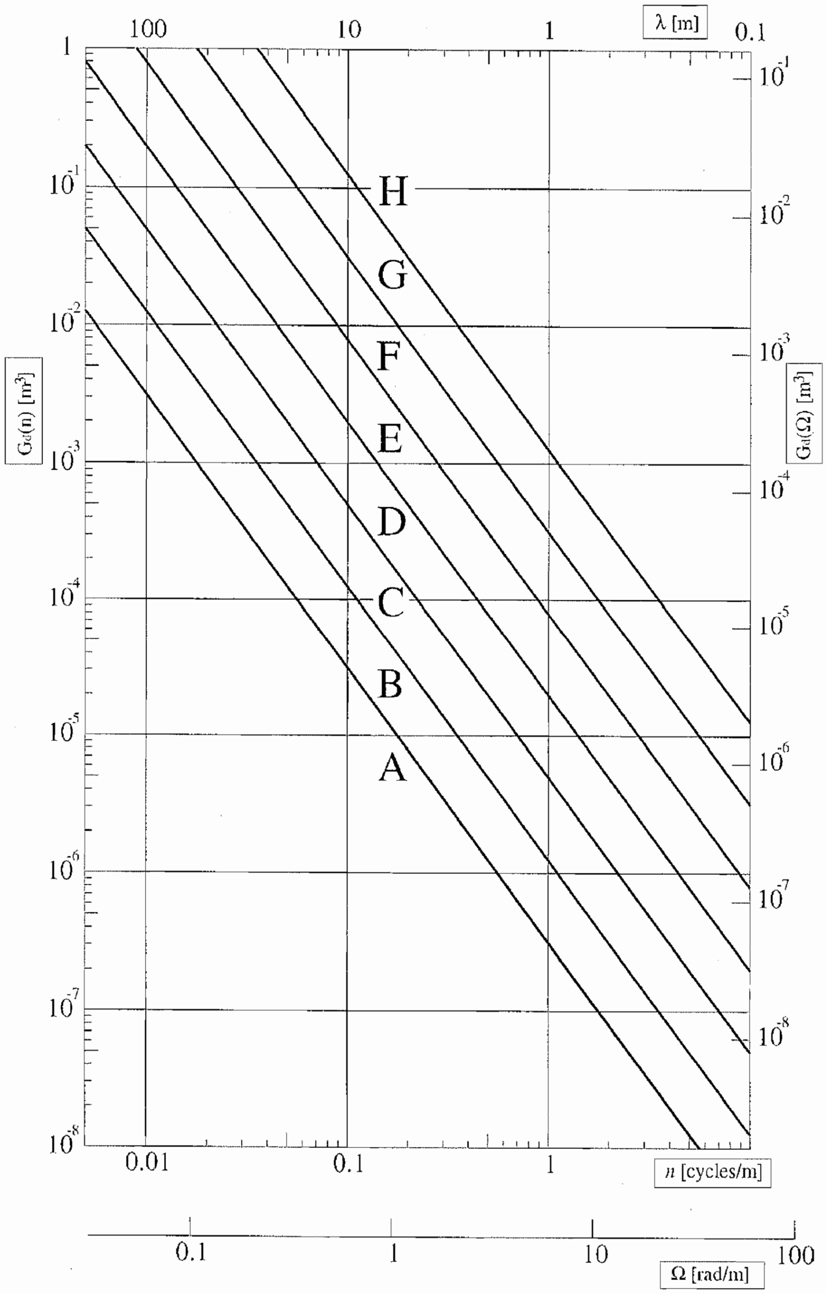
[7] ISO 8608:1995 – Mechanical vibration – Road surface profiles – Reporting of measured data



Key

AC1 G_d (n) [m^3] AC1 Displacement power spectral density,
λ [m] Wavelength,
AC1 G_d (Ω) [m^3] AC1 Displacement power spectral density,
n [cycles/m] Spatial frequency,
Ω [rad/m] Angular spatial frequency,

Figure B.1 – Road surface classification (ISO 8608)

The limit values of G_d and G_v for the first 5 road surface classes in terms of n and Ω are given in Tables B.1 and B.2, respectively.

Table B.1 – Degree of roughness expressed in terms of spatial frequency units, *n*

		Degree of roughness			
Road class	Pavement quality	$G_d\ (n_0)^a\ [10^{-6}\ m]$			$G_v\ (n)\ [10^{-6}\ m]$
		Lower limit	Geometric mean	Upper limit	Geometric mean
A	Very good	---	16	32	6,3
B	Good	32	64	128	25,3
C	Medium	128	256	512	101,1
D	Poor	512	1024	2048	404,3
E	Very poor	2048	4096	8192	1617,0

[a] n_0=0,1 cycle/m

Table B.2 – Degree of roughness expressed in terms of angular spatial frequency units, *Ω*

		Degree of roughness			
Road class	Pavement quality	$G_d\ (\Omega_0)^a\ [10^{-6}\ m]$			$G_v\ (\Omega)\ [10^{-6}\ m]$
		Lower limit	Geometric mean	Upper limit	Geometric mean
A	Very good	---	1	2	1
B	Good	2	4	8	4
C	Medium	8	16	32	16
D	Poor	32	64	128	64
E	Very poor	128	256	512	256

[a] Ω_0=1 rad/m

(3) Unless otherwise specified, the recorded axle loads should be multiplied by :

φ_{fat} = 1,2 for surface of good roughness

φ_{fat} = 1,4 for surface of medium roughness.

(4) In addition, when considering a cross-section within a distance of 6,00 m from an expansion joint, the load should be multiplied by the additional dynamic amplification factor $\Delta\varphi_{fat}$ derived from Figure 4.7.

(5) The classification of roadway roughness may be taken in accordance with ISO 8608.

(6) 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").

(7) The wheel contact areas and the transverse distances between wheels should be taken as described in Table 4.8, where relevant.

(8) 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.

(9) 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.

(10) The numbers of cycles should be counted using the rainflow method or the reservoir method.

(11) 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 on the traffic

(12) The cumulative fatigue damage calculated by use of records should be multiplied by the ratio between the design working life and the duration considered on the histogram. In the absence of detailed information, a factor 2 for the number of lorries and a factor 1,4 for the load levels are recommended.

Annex C
(normative)
Dynamic factors 1 + φ for Real Trains

(1)P To take account of dynamic effects resulting from the movement of actual service trains at speed, the forces and moments calculated from the specified static loads shall be multiplied by a factor appropriate to the Maximum Permitted Vehicle Speed.

(2) The dynamic factors 1 + φ are also used for fatigue damage calculations.

(3)P The static load due to a Real Train at v [m/s] shall be multiplied by:

either, $1 + \varphi = 1 + \varphi' + \varphi''$ for track with standard maintenance (C.1)

or, $1 + \varphi = 1 + \varphi' + 0{,}5\ \varphi''$ for carefully maintained track (C.2)

NOTE The National Annex may specify whether expression (C.1) or (C.2) may be used. Where the expression to be used is not specified, expression (C.1) is recommended.

with:

$$\varphi' = \frac{K}{1 - K + K^4} \quad \text{for } K < 0{,}76 \tag{C.3}$$

and

$$\varphi' = 1{,}325 \quad \text{for } K \geq 0{,}76 \tag{C.4}$$

$$\text{where: } K = \frac{v}{2L_\Phi \times n_0} \tag{C.5}$$

and

$$\varphi'' = \frac{\alpha}{100}\left[56\mathrm{e}^{-\left(\frac{L_\Phi}{10}\right)^2} + 50\left(\frac{L_\Phi n_0}{80} - 1\right)e^{-\left(\frac{L_\Phi}{20}\right)^2}\right] \tag{C.6}$$

$\varphi'' \geq 0$

$$\text{with: } \alpha = \frac{v}{22} \quad \text{if } v \leq 22 \text{ m/s} \tag{C.7}$$

$\alpha = 1$ if $v > 22$ m/s

where:

v is the Maximum Permitted Vehicle Speed [m/s]

n_0 is the first natural bending frequency of the bridge loaded by permanent actions [Hz]

L_Φ is the determinant length [m] in accordance with 6.4.5.3.

α is a coefficient for speed

The limit of validity for φ' defined by Equations (C.3) and (C.4) is the lower limit of natural frequency in Figure 6.10 and 200 km/h. For all other cases φ' should be determined by a dynamic analysis in accordance with 6.4.6.

NOTE The method used should be agreed with the relevant authority specified in the National Annex.

The limit of validity for φ'' defined by Equation (C.6) is the upper limit of natural frequency in Figure 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 6.4.6.

(4)P The values of $\varphi' + \varphi''$ shall be determined using upper and lower limiting values of n_0, unless it is being made for an individual bridge of known first natural frequency.

The upper limit of n_0 is given by:

$$n_0 = 94{,}76 L_\Phi^{-0{,}748} \qquad \text{(C.8)}$$

and the lower limit is given by:

$$n_0 = \frac{80}{L_\Phi} \qquad \text{for } 4 \text{ m} \le L_\Phi \le 20 \text{ m} \qquad \text{(C.9)}$$

$$n_0 = 23{,}58 L_\Phi^{-0{,}592} \qquad \text{for } 20 \text{ m} < L_\Phi \le 100 \text{ m} \qquad \text{(C.10)}$$

Annex D
(normative)
Basis for the fatigue assessment of railway structures

D.1 Assumptions for fatigue actions

(1) The dynamic factors Φ_2 and Φ_3 which are applied to the static Load Model 71 and SW/0 and SW/2, when clause 6.4.5 applies, represent the extreme loading case to be taken into account for detailing bridge members. These factors would be unduly onerous if they were applied to the Real Trains used for making an assessment of fatigue damage.

(2) To take account of the average effect over the assumed 100 years life of the structure, the dynamic enhancement for each Real Train may be reduced to:

$$1 + \tfrac{1}{2}(\varphi' + \tfrac{1}{2}\varphi'') \qquad \text{(D.1)}$$

where φ' and φ'' are defined below in equations (D.2) and (D.5).

(3) Equations (D.2) and (D.5) are simplified forms of equations (C.3) and (C.6) which are sufficiently accurate for the purpose of calculating fatigue damage and are valid for Maximum Permitted Vehicle Speeds up to 200km/h:

$$\varphi' = \frac{K}{1 - K + K^4} \qquad \text{(D.2)}$$

with:

$$K = \frac{v}{160} \qquad \text{for L} \leq 20 \text{ m} \qquad \text{(D.3)}$$

$$K = \frac{v}{47{,}16L^{0{,}408}} \qquad \text{for L} > 20 \text{ m} \qquad \text{(D.4)}$$

and

$$\varphi'' = 0{,}56e^{-\frac{L^2}{100}} \qquad \text{(D.5)}$$

where:

v is the Maximum Permitted Vehicle Speed [m/s]

L is the determinant length L_Φ [m] in accordance with 6.4.5.3

NOTE Where dynamic effects including resonance may be excessive and a dynamic analysis is required in accordance with 6.4.4 additional requirements for the fatigue assessment of bridges are given in 6.4.6.6.

D.2 General design method

(1)P The fatigue assessment, in general a stress range verification, shall be carried out according to EN 1992, EN 1993 and EN 1994.

(2) As an example for steel bridges the safety verification shall be carried out by ensuring that the following condition is satisfied:

$$\gamma_{\mathrm{Ff}} \lambda \Phi_2 \Delta\sigma_{71} \leq \frac{\Delta\sigma_{\mathrm{c}}}{\gamma_{\mathrm{Mf}}} \qquad \text{(D.6)}$$

where:

γ_{Ff} is the partial safety factor for fatigue loading

NOTE The value for γ_{Ff} may be given in the National Annex. The recommended value is $\gamma_{\mathrm{Ff}} = 1{,}00$.

λ is the damage equivalence factor for fatigue which takes account of the service traffic on the bridge and the span of the member. Values of λ are given in the design codes [AC1) (EN 1992 – EN 1999). (AC1]

Φ_2 is the dynamic factor (see 6.4.5)

$\Delta\sigma_{71}$ is the stress range due to the Load Model 71 (and where required SW/0) but excluding α) being placed in the most unfavourable position for the element under consideration

$\Delta\sigma_{\mathrm{C}}$ is the reference value of the fatigue strength (see EN 1993)

γ_{Mf} is the partial safety factor for fatigue strength in the design codes [AC1) (EN 1992 – EN 1999) (AC1]

D.3 Train types for fatigue

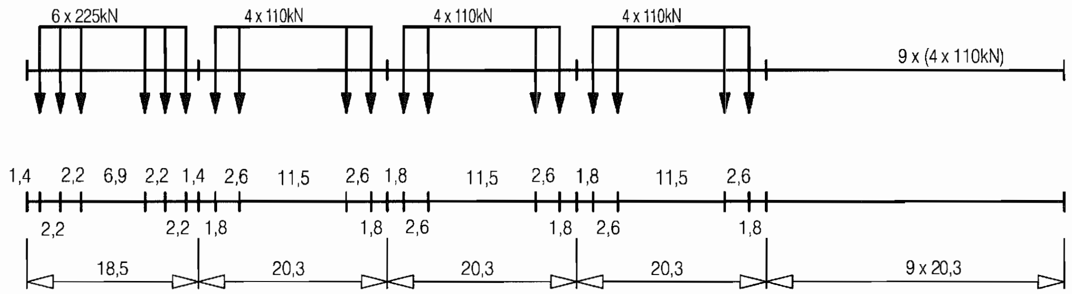
The fatigue assessment should be carried out on the basis of the traffic mixes, "standard traffic", "traffic with 250 kN-axles" or “light traffic mix”, depending on whether the structure carries standard traffic mix, predominantly heavy freight traffic or light traffic.

Details of the service trains and traffic mixes are given below.

(1) Standard and light traffic mixes

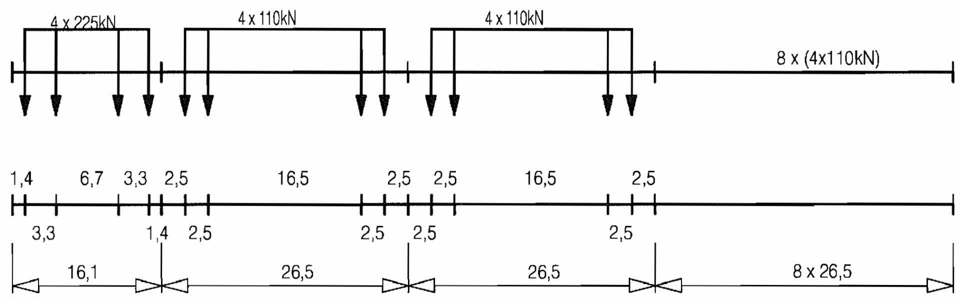
Type 1 Locomotive-hauled passenger train

$\Sigma Q = 6630\text{kN}$ $V = 200\text{km/h}$ $L = 262{,}10\text{m}$ $q = 25{,}3\text{kN/m'}$



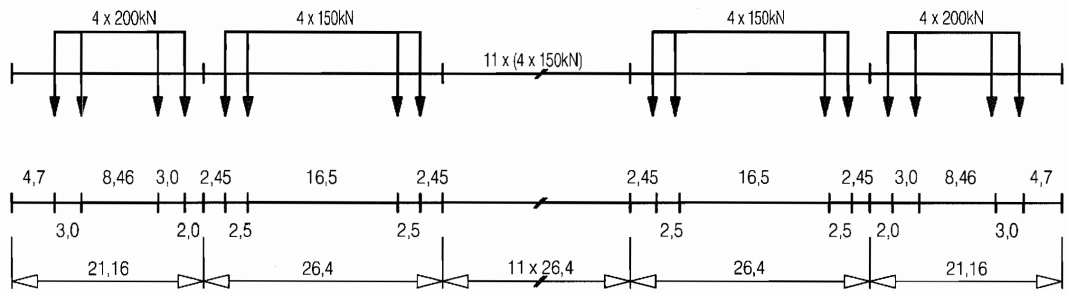
Type 2 Locomotive-hauled passenger train

$\Sigma Q = 5300\text{kN}$ $V = 160\text{km/h}$ $L = 281{,}10\text{m}$ $q = 18{,}9\text{kN/m'}$



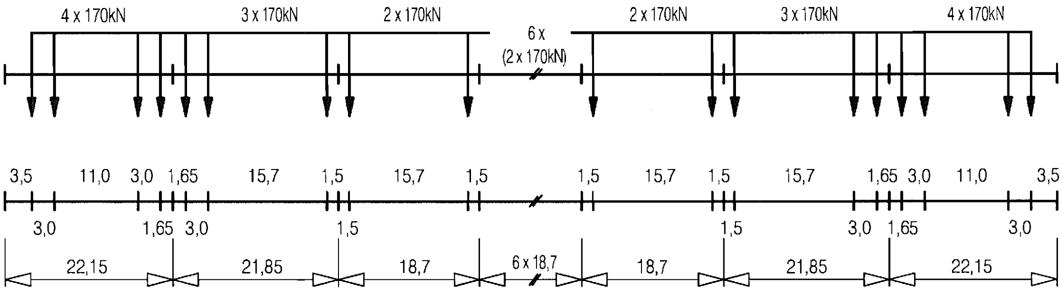
Type 3 High speed passenger train

$\Sigma Q = 9400\text{kN}$ $V = 250\text{km/h}$ $L = 385{,}52\text{m}$ $q = 24{,}4\text{kN/m'}$



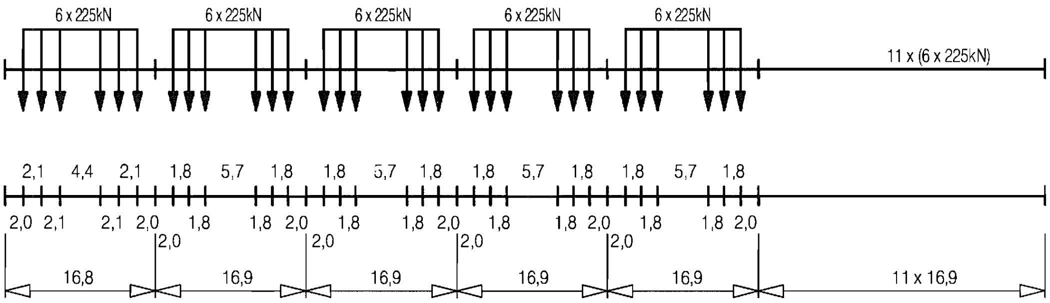
Type 4 High speed passenger train

$\Sigma Q = 5100\text{kN}$ $V = 250\text{km/h}$ $L = 237{,}60\text{m}$ $q = 21{,}5\text{kN/m'}$



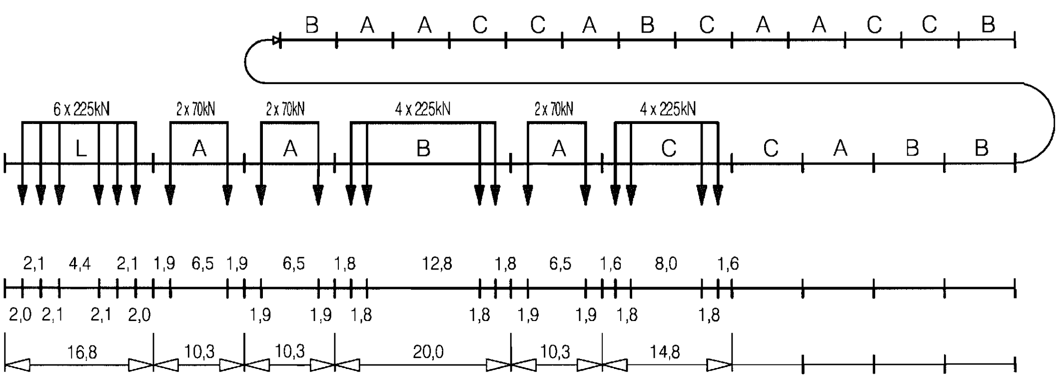
Type 5 Locomotive-hauled freight train

$\Sigma Q = 21600\text{kN}$ $V = 80\text{km/h}$ $L = 270{,}30\text{m}$ $q = 80{,}0\text{kN/m'}$



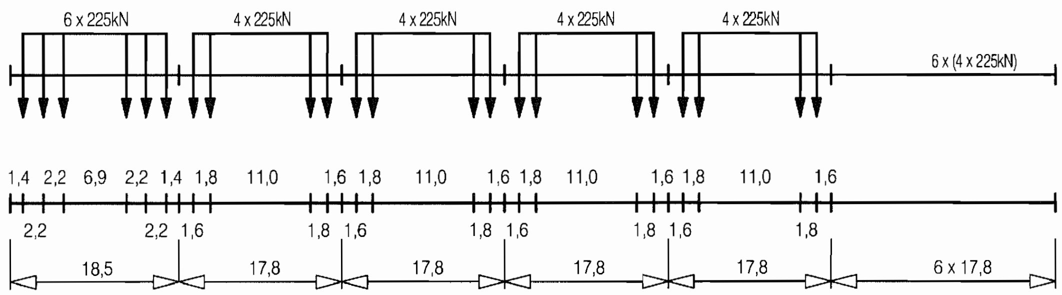
Type 6 Locomotive-hauled freight train

$\Sigma Q = 14310\text{kN}$ $V = 100\text{km/h}$ $L = 333{,}10\text{m}$ $q = 43{,}0\text{kN/m'}$



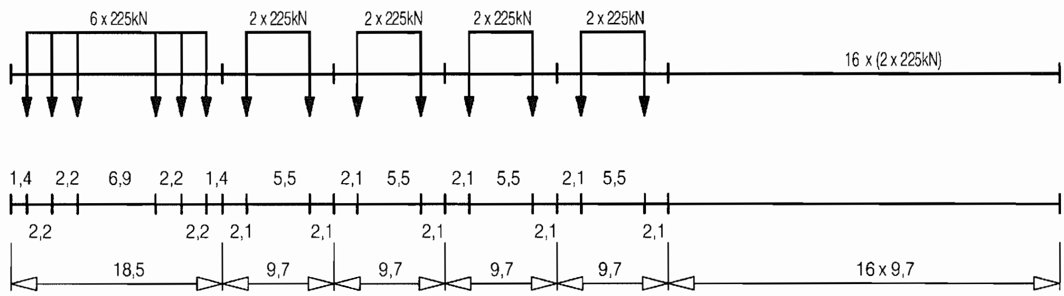
Type 7 Locomotive-hauled freight train

$\Sigma Q = 10350\text{kN}$ $V = 120\text{km/h}$ $L = 196{,}50\text{m}$ $q = 52{,}7\text{kN/m'}$



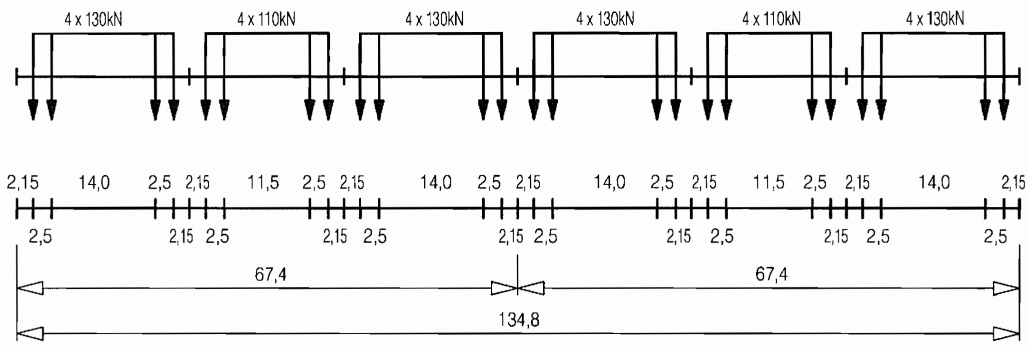
Type 8 Locomotive-hauled freight train

$\Sigma Q = 10350\text{kN}$ $V = 100\text{km/h}$ $L = 212{,}50\text{m}$ $q = 48{,}7\text{kN/m'}$



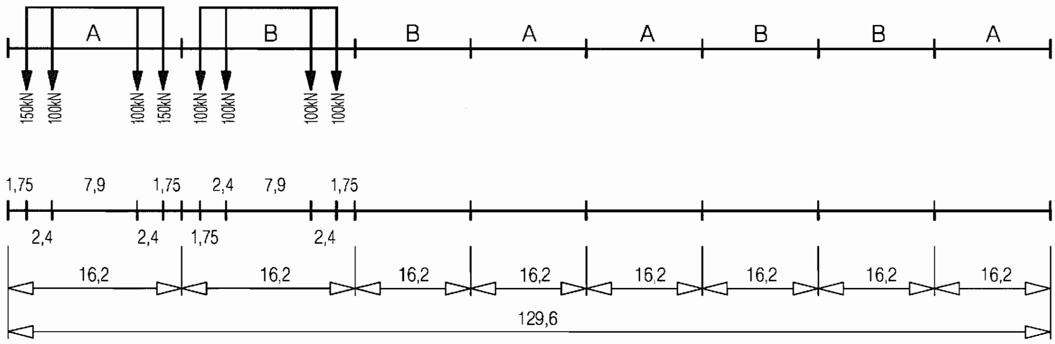
Type 9 Surburban multiple unit train

$\Sigma Q = 2960\text{kN}$ $V = 120\text{km/h}$ $L = 134{,}80\text{m}$ $q = 22{,}0\text{kN/m'}$



Type 10 Underground

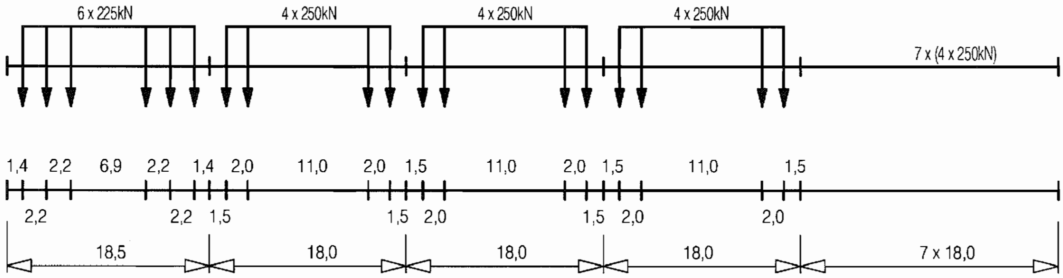
$\Sigma Q = 3600\text{kN}$ $V = 120\text{km/h}$ $L = 129{,}60\text{m}$ $q = 27{,}8\text{kN/m}'$



(2) Heavy traffic with 250 kN - axles

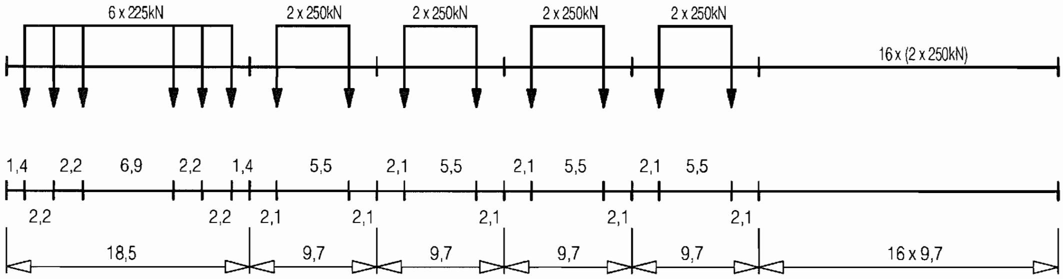
Type 11 Locomotive-hauled freight train

$\Sigma Q = 11350\text{kN}$ $V = 120\text{km/h}$ $L = 198{,}50\text{m}$ $q = 57{,}2\text{kN/m}'$



Type 12 Locomotive-hauled freight train

$\Sigma Q = 11350\text{kN}$ $V = 100\text{km/h}$ $L = 212{,}50\text{m}$ $q = 53{,}4\text{kN/m}'$



(3) Traffic mix:

Table D.1 - Standard traffic mix with axles ≤ 22,5 t (225 kN)

Train type	Number of trains/day	Mass of train [t]	Traffic volume [10^6t/year]
1	12	663	2,90
2	12	530	2,32
3	5	940	1,72
4	5	510	0,93
5	7	2160	5,52
6	12	1431	6,27
7	8	1035	3,02
8	6	1035	2,27
	67		24,95

Table D.2 - Heavy traffic mix with 25t (250 kN) axles

Train type	Number of trains/day	Mass of train [t]	Traffic volume [10^6t/year]
5	6	2160	4,73
6	13	1431	6,79
11	16	1135	6,63
12	16	1135	6,63
	51		24,78

Table D.3 - Light traffic mix with axles ≤ 22,5 t (225 kN)

Train type	Number of trains/day	Mass of train [t]	Traffic volume [10^6t/year]
1	10	663	2,4
2	5	530	1,0
5	2	2160	1,4
9	190	296	20,5
	207		25,3

Annex E
(informative)
Limits of validity of Load Model HSLM and the selection of the critical Universal Train from HSLM-A

E.1 Limits of validity of Load Model HSLM

(1) 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 E.2,
- the distance D [m] corresponding to the length of the coach or to the distance between regularly repeating axles in accordance with Table E.1,
- the spacing of axles within a bogie, d_{BA} [m] in accordance with:

$$2{,}5 \text{ m} \leq d_{BA} \leq 3{,}5 \text{ m} \qquad \text{(E.1)}$$

- for conventional trains the distance between the centres of bogies between adjacent vehicles d_{BS} [m] in accordance with Equation E.2,
- for regular trains with coaches with one axle per coach (*e.g.* Train type E in 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 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 of 10,000 kN,
- maximum train length of 400 m,
- maximum unsprung axle mass of 2 tonnes,

Table E.1 - Limiting parameters for high speed passenger trains conforming to Load Model HSLM

Type of train	P [kN]	D [m]	D_{IC} [m]	e_c [m]
Articulated	170	$18 \leq D \leq 27$	-	-
Conventional	Lesser of 170 or value corresponding to equation E.2 below.	$18 \leq D \leq 27$	-	-
Regular	170	$10 \leq D \leq 14$	$8 \leq D_{IC} \leq 11$	$7 \leq e_c \leq 10$

where:

$$4P\cos\left(\frac{\pi\, d_{BS}}{D}\right)\cos\left(\frac{\pi\, d_{BA}}{D}\right) \leq 2P_{HSLMA}\cos\left(\frac{\pi\, d_{HSLMA}}{D_{HSLMA}}\right) \qquad \text{(E.2)}$$

where:

P_{HSLMA}, d_{HSLMA} and D_{HSLMA} are the parameters of the Universal Trains in accordance with Figure 6.12 and Table 6.3 corresponding to the coach length D_{HSLMA} for:
- a single Universal Train where D_{HSLMA} equals the value of D,
- two Universal Trains where D does not equal D_{HSLMA} with D_{HSLMA} taken as just greater than D and just less than D,

and D, D_{IC}, P, d_{BA}, d_{BS} and e_C are defined as appropriate for articulated, conventional and regular trains in Figures E.1 to E.3:

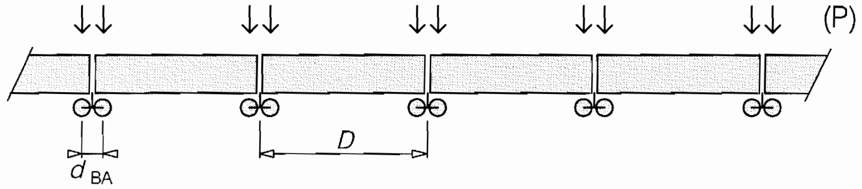


Figure E1 - Articulated train

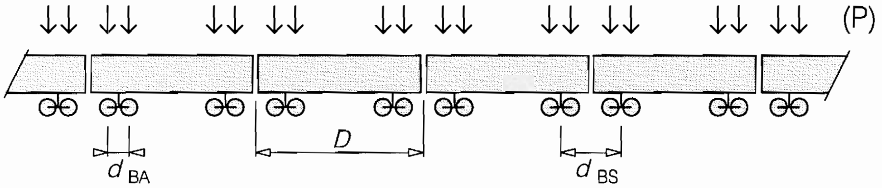


Figure E2 - Conventional train

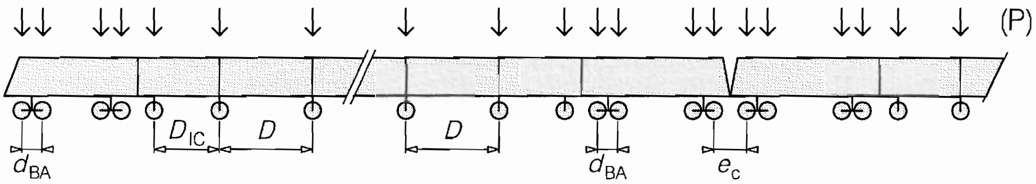


Figure E3 - Regular train

(2) The point forces, dimensions and lengths of the Universal Trains defined in 6.4.6.1.1 do not form part of the real vehicle specification unless referenced in E.1(1).

E.2 Selection of a Universal Train from HSLM-A

(1) For simply supported spans that exhibit only line beam dynamic behaviour and with a span of 7 m or greater a single Universal Train derived from the load model HSLM-A may be used for the dynamic analysis.

(2) The critical Universal Train is defined in E.2(5) as a function of:

– the critical wavelength of excitation λ_C [m] defined in E.2(4)

where the critical wavelength of excitation λ_C is a function of:

– the wavelength of excitation at the Maximum Design Speed λ_v [m] given in E.2(3),
– the span of the bridge L [m],
– the maximum value of aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ [kN/m] in the range of excitation wavelength from 4,5 m to L [m] given in E.2(4).

(3) The wavelength of excitation at the Maximum Design Speed λ_v [m] is given by:

$$\lambda_v = v_{DS} / n_0 \qquad \text{(E.3)}$$

where:

n_0 First natural frequency of the simply supported span [Hz]
v_{DS} Maximum Design Speed in accordance with 6.4.6.2(1) [m/s]

(4) The critical wavelength of excitation λ_C should be determined from Figures E.4 to E.17 as the value of λ corresponding to the maximum value of aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ for the span of length L [m] in the range of excitation wavelength from 4,5 m to λ_v.

Where the span of the deck does not correspond to the reference length L in figures E.4 to E.17, the two figures corresponding to the values of L taken as either just greater than the span or just less than the span of the deck should be taken into account. The critical wavelength of excitation λ_C should be determined from the figure corresponding to the maximum aggressivity. Interpolation between the diagrams is not permitted.

NOTE It can be seen from Figures E.4 to E.17 that in many cases $\lambda_C = \lambda_v$ but in some cases λ_C corresponds to a peak value of aggressivity at a value of λ less than λ_v. (For example in Figure E.4 for λ_v = 17m, λ_C = 13m)

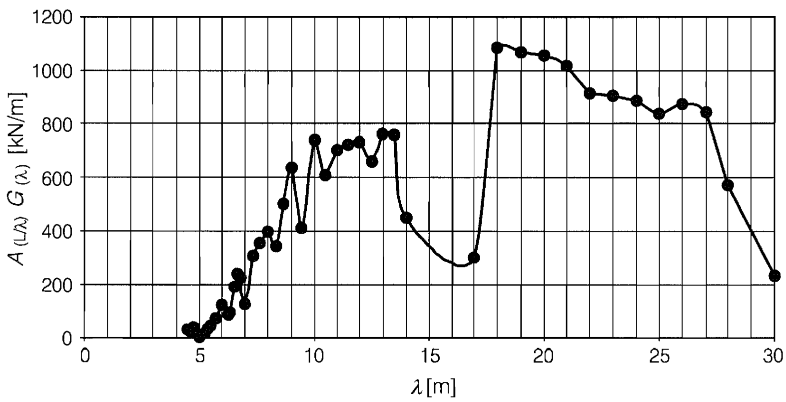


Figure E.4 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 7,5 m and damping ratio ζ = 0.01

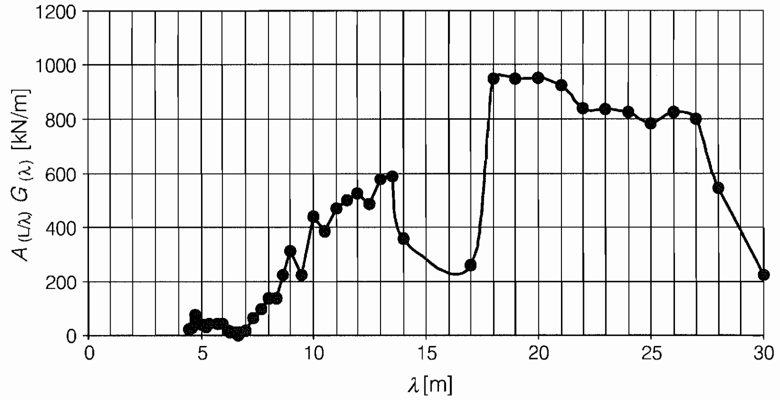


Figure E.5 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 10,0 m and damping ratio ζ = 0.01

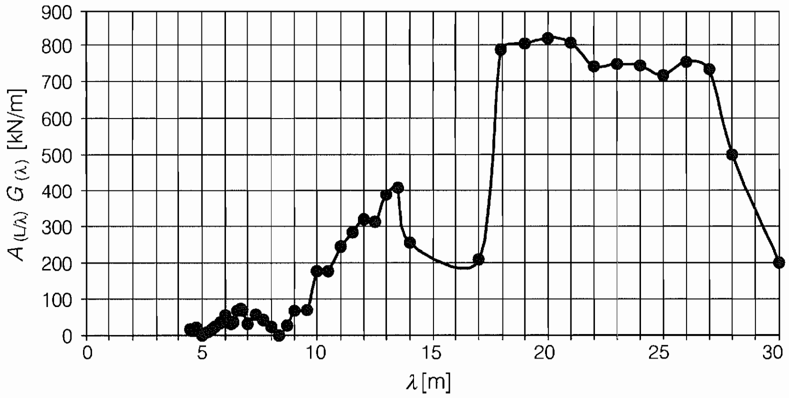


Figure E.6 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 12,5 m and damping ratio ζ = 0.01

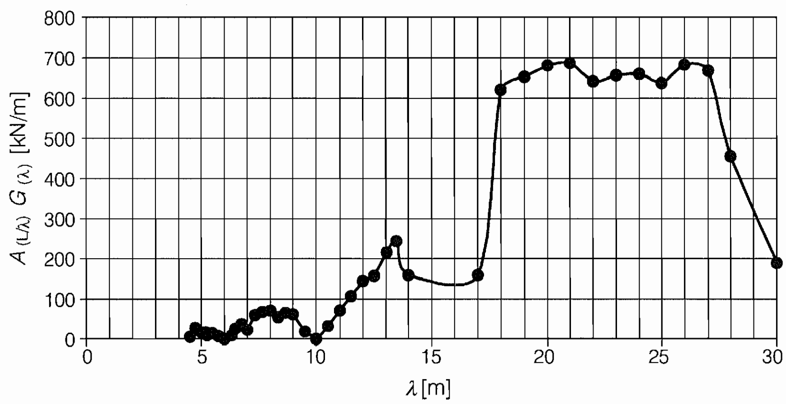


Figure E.7 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 15,0 m and damping ratio ζ = 0.01

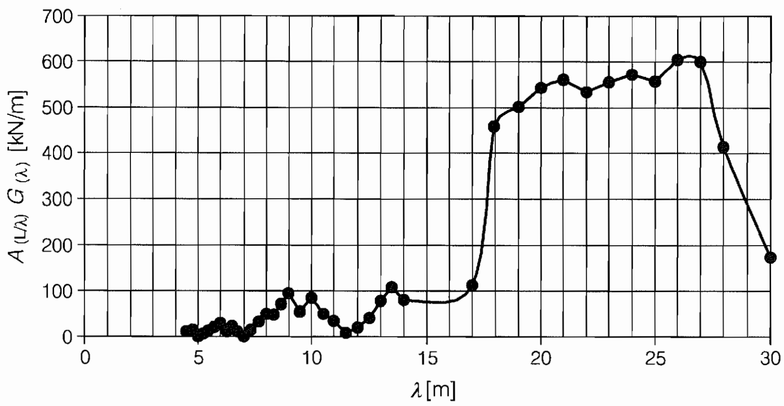


Figure E.8 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 17,5 m and damping ratio ζ = 0.01

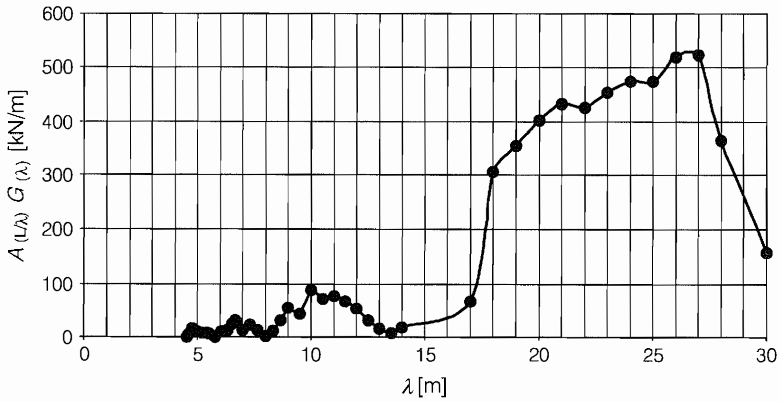


Figure E.9 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 20,0 m and damping ratio ζ = 0.01

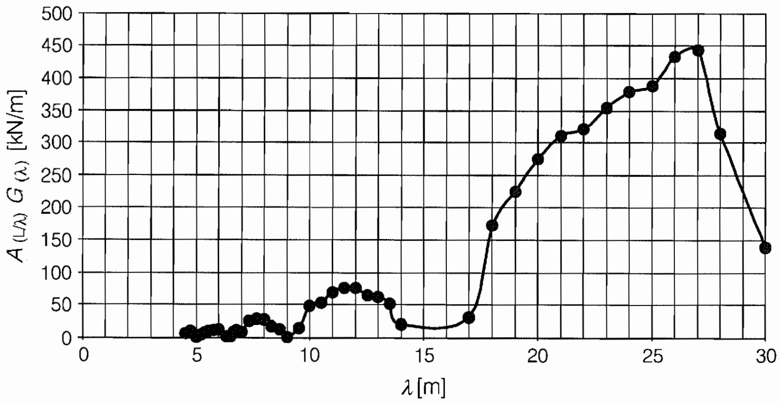


Figure E.10 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 22,5 m and damping ratio ζ = 0.01

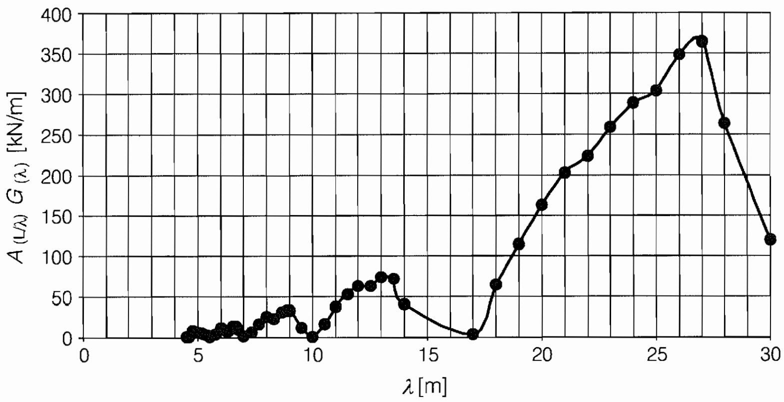


Figure E.11 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 25,0 m and damping ratio ζ = 0.01

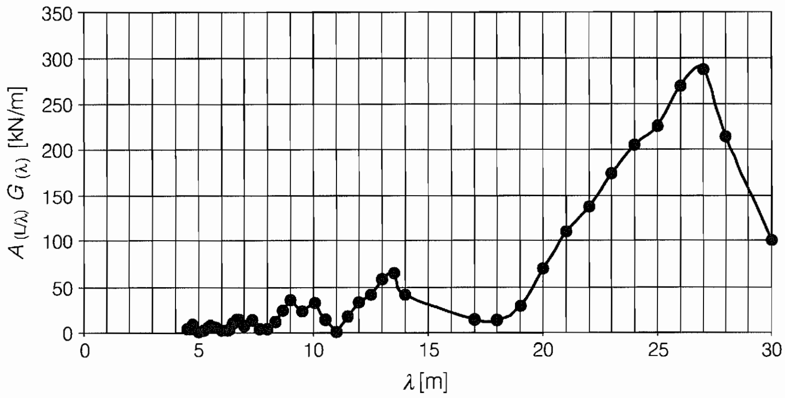


Figure E.12 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 27,5 m and damping ratio ζ = 0.01

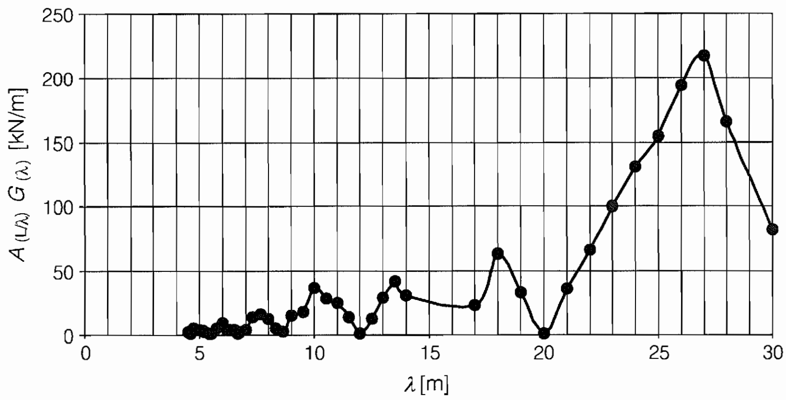


Figure E.13 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 30,0 m and damping ratio ζ = 0.01

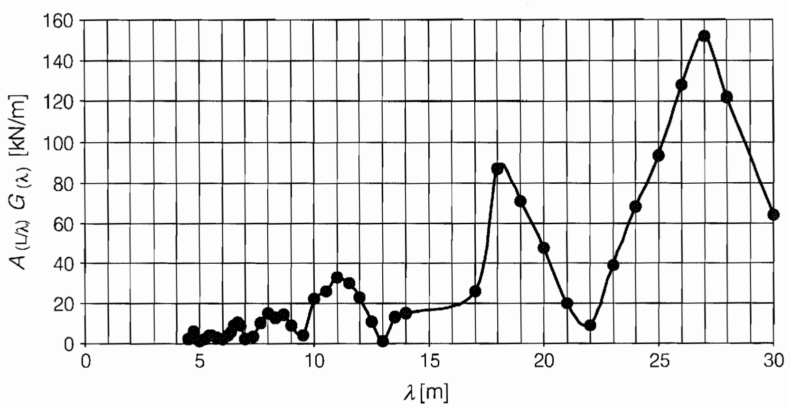


Figure E.14 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 32,5 m and damping ratio ζ = 0.01

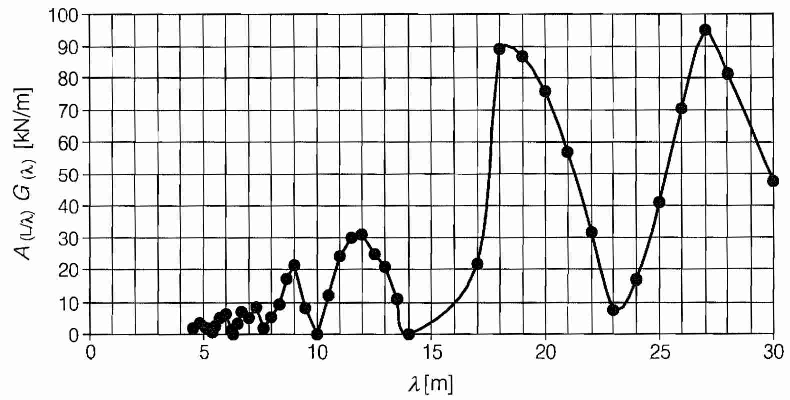


Figure E.15 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 35,0 m and damping ratio ζ = 0.01

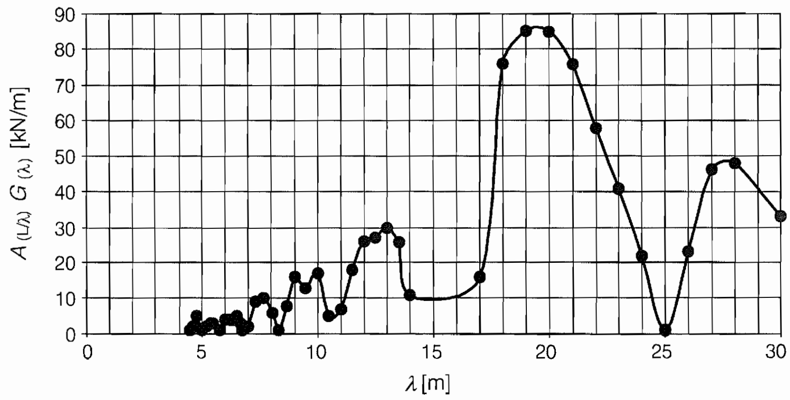


Figure E.16 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 37,5 m and damping ratio ζ = 0.01

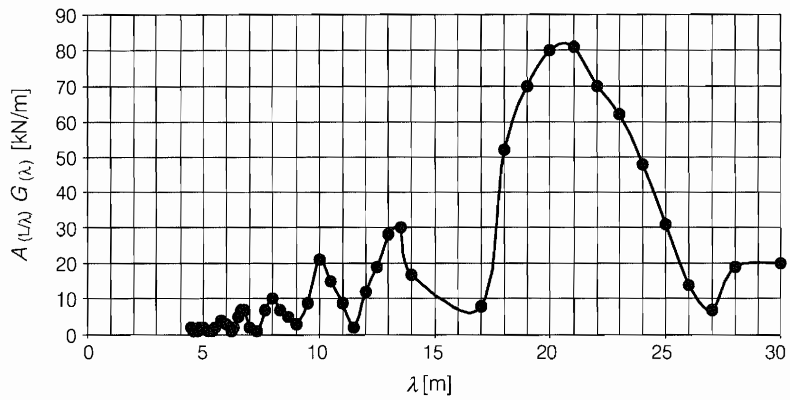


Figure E.17 - Aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ as a function of excitation wavelength λ for a simply supported span of L = 40,0 m and damping ratio ζ = 0.01

(5) The critical Universal Train in HSLM-A is defined in Figure E.18:

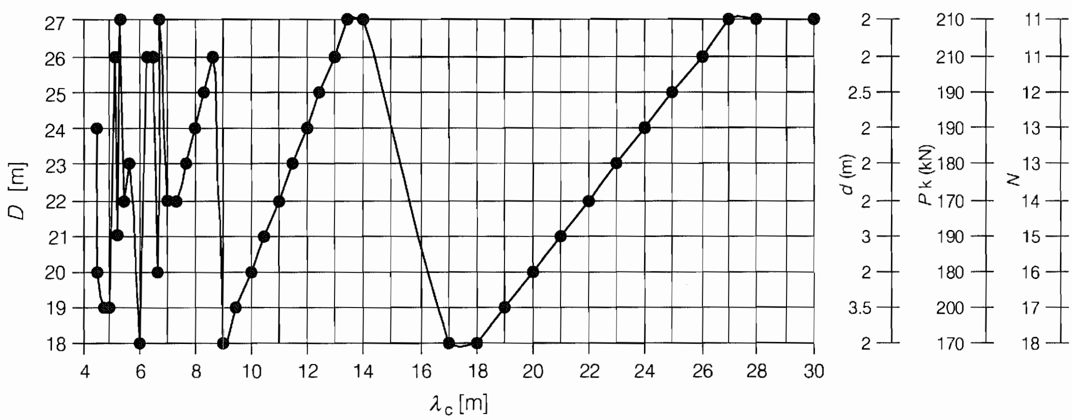


Figure E.18 - Parameters defining critical Universal Train in HSLM-A as a function of critical wavelength of excitation λ_C [m]

NOTE For values of [AC1⟩ λ_C ⟨AC1] < 7 m it is recommended that the dynamic analysis is carried out with Universal Trains A1 to A10 inclusive in accordance with Table 6.3.

Where:

- D Length of intermediate and end coaches defined in Figure 6.12 [m]
- d Spacing of bogie axles for intermediate and end coaches defined in Figure 6.12 [m]
- N Number of intermediate coaches defined in Figure 6.12
- P_k Point force at each axle position in intermediate and end coaches and in each power car as defined in Figure 6.12 [kN]
- λ_C Critical wavelength of excitation given in E.2(4) [m]

(6) Alternatively the aggressivity $A_{(L/\lambda)}G_{(\lambda)}$ [kN/m] is defined by equations E.4 and E.5:

$$A_{\left(L/\lambda\right)} = \left| \frac{\cos\left(\dfrac{\pi L}{\lambda}\right)}{\left(\dfrac{2L}{\lambda}\right)^2 - 1} \right| \tag{E.4}$$

$$G_{(\lambda)} \cong \underset{i=0 \text{ to } M-1}{\mathrm{MAX}} \; \frac{1}{\zeta X_i} \sqrt{\left(\sum_{k=0}^{i} P_k \cos\left(\frac{2\pi x_k}{\lambda}\right) \right)^2 + \left(\sum_{k=0}^{i} P_k \sin\left(\frac{2\pi x_k}{\lambda}\right) \right)^2} \left(1 - \exp\left(-2\pi\zeta \frac{X_i}{\lambda} \right) \right) \tag{E.5}$$

where i is taken from 0 to (M-1) to cover all sub-trains including the whole train and:

L	Span [m]
M	Number of point forces in train
P_k	Load on axle k [kN]
X_i	Length of sub-train consisting of i axles
x_k	Distance of point force P_k from first point force P_0 in train [m]
λ	Wavelength of excitation [m]
ζ	Damping ratio

Annex F
(informative)
Criteria to be satisfied if a dynamic analysis is not required

NOTE Annex F is not valid for Load Model HSLM (Annex F is valid for the trains given in F(4)).

(1) For simply supported structures satisfying the maximum value of $(v/n_0)_{lim}$ given in Tables F.1 and F.2:
- the maximum dynamic load effects (stresses, deflections etc.) and
- the fatigue loading at high speeds (except where the Frequent Operating Speed corresponds to a Resonant Speed and in such cases a specific dynamic analysis and fatigue check should be carried out in accordance with 6.4.6)

do not exceed the values due to $\Phi_2 \times$ Load Model 71 and no further dynamic analysis is necessary and
- the maximum deck acceleration is less than either 3.50m/s^2 or 5,0m/s^2 as appropriate.

Table F.1 - Maximum value of $(v/n_0)_{lim}$ for a simply supported beam or slab and a maximum permitted acceleration of a_{max}< 3.50m/s^2.

Mass m 10^3 kg/m		≥5,0 <7,0	≥7,0 <9,0	≥9,0 <10,0	≥10,0 <13,0	≥13,0 <15,0	≥15,0 <18,0	≥18,0 <20,0	≥20,0 <25,0	≥25,0 <30,0	≥30,0 <40,0	≥40,0 <50,0	≥50,0 -
Span $L\in$ m [a]	ζ %	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m
[5,00,7,50)	2	1,71	1,78	1,88	1,88	1,93	1,93	2,13	2,13	3,08	3,08	3,54	3,59
	4	1,71	1,83	1,93	1,93	2,13	2,24	3,03	3,08	3,38	3,54	4,31	4,31
[7,50,10,0)	2	1,94	2,08	2,64	2,64	2,77	2,77	3,06	5,00	5,14	5,20	5,35	5,42
	4	2,15	2,64	2,77	2,98	4,93	5,00	5,14	5,21	5,35	5,62	6,39	6,53
[10,0,12,5)	1	2,40	2,50	2,50	2,50	2,71	6,15	6,25	6,36	6,36	6,45	6,45	6,57
	2	2,50	2,71	2,71	5,83	6,15	6,25	6,36	6,36	6,45	6,45	7,19	7,29
[12,5,15,0)	1	2,50	2,50	3,58	3,58	5,24	5,24	5,36	5,36	7,86	9,14	9,14	9,14
	2	3,45	5,12	5,24	5,24	5,36	5,36	7,86	8,22	9,53	9,76	10,36	10,48
[15,0,17,5)	1	3,00	5,33	5,33	5,33	6,33	6,33	6,50	6,50	6,50	7,80	7,80	7,80
	2	5,33	5,33	6,33	6,33	6,50	6,50	10,17	10,33	10,33	10,50	10,67	12,40
[17,5,20,0)	1	3,50	6,33	6,33	6,33	6,50	6,50	7,17	7,17	10,67	12,80	12,80	12,80
[20,0,25,0)	1	5,21	5,21	5,42	7,08	7,50	7,50	13,54	13,54	13,96	14,17	14,38	14,38
[25,0,30,0)	1	6,25	6,46	6,46	10,21	10,21	10,21	10,63	10,63	12,75	12,75	12,75	12,75
[30,0,40,0)	1				10,56	18,33	18,33	18,61	18,61	18,89	19,17	19,17	19,17
≥40,0	1				14,73	15,00	15,56	15,56	15,83	18,33	18,33	18,33	18,33

[a] $L \in [a,b)$ means $a \le L < b$

NOTE 1 Table F.1 includes a safety factor of 1.2 on $(v/n_0)_{lim}$ for acceleration, deflection and strength criteria and a safety factor of 1,0 on the $(v/n_0)_{lim}$ for fatigue.

NOTE 2 Table F.1 includes an allowance of $(1+\varphi''/2)$ for track irregularities.

Table F.2 - Maximum value of $(v/n_0)_{lim}$ for a simply supported beam or slab and a maximum permitted acceleration of $a_{max} < 5.0$ m/s²

Mass m 10^3 kg/m		≥5,0 <7,0	≥7,0 <9,0	≥9,0 <10,0	≥10,0 <13,0	≥13,0 <15,0	≥15,0 <18,0	≥18,0 <20,0	≥20,0 <25,0	≥25,0 <30,0	≥30,0 <40,0	≥40,0 <50,0	≥50,0 -
Span $L \in$ m [a]	ζ %	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m
[5,00,7,50)	2	1,78	1,88	1,93	1,93	2,13	2,13	3,08	3,08	3,44	3,54	3,59	4,13
	4	1,88	1,93	2,13	2,13	3,08	3,13	3,44	3,54	3,59	4,31	4,31	4,31
[7,50,10,0)	2	2,08	2,64	2,78	2,78	3,06	5,07	5,21	5,21	5,28	5,35	6,33	6,33
	4	2,64	2,98	4,86	4,93	5,14	5,21	5,35	5,42	6,32	6,46	6,67	6,67
[10,0,12,5)	1	2,50	2,50	2,71	6,15	6,25	6,36	6,36	6,46	6,46	6,46	7,19	7,19
	2	2,71	5,83	6,15	6,15	6,36	6,46	6,46	6,46	7,19	7,19	7,75	7,75
[12,5,15,0)	1	2,50	3,58	5,24	5,24	5,36	5,36	7,86	8,33	9,14	9,14	9,14	9,14
	2	5,12	5,24	5,36	5,36	7,86	8,22	9,53	9,64	10,36	10,36	10,48	10,48
[15,0,17,5)	1	5,33	5,33	6,33	6,33	6,50	6,50	6,50	7,80	7,80	7,80	7,80	7,80
	2	5,33	6,33	6,50	6,50	10,33	10,33	10,50	10,50	10,67	10,67	12,40	12,40
[17,5,20,0)	1	6,33	6,33	6,50	6,50	7,17	10,67	10,67	12,80	12,80	12,80	12,80	12,80
[20,0,25,0)	1	5,21	7,08	7,50	7,50	13,54	13,75	13,96	14,17	14,38	14,38	14,38	14,38
[25,0,30,0)	1	6,46	10,20	10,42	10,42	10,63	10,63	12,75	12,75	12,75	12,75	12,75	12,75
[30,0,40,0)	1				18,33	18,61	18,89	18,89	19,17	19,17	19,17	19,17	19,17
≥40,0	1				15,00	15,56	15,83	18,33	18,33	18,33	18,33	18,33	18,33

[a] $L \in [a,b)$ means $a \le L < b$

NOTE 1 Table F.2 includes a safety factor of 1.2 on $(v/n_0)_{lim}$ for acceleration, deflection and strength criteria and a safety factor of 1,0 on the $(v/n_0)_{lim}$ for fatigue.

NOTE 2 Table F.2 include an allowance of $(1+\varphi''/2)$ for track irregularities.

where:

L is the span length of bridge [m],
m is the mass of bridge [10^3 kg/m],
ζ is the percentage of critical damping in [%],
v is the Maximum Nominal Speed and is generally the Maximum Line Speed at the site. A reduced speed may be used for checking individual Real Trains for their associated Maximum Permitted Vehicle Speed [m/s],
n_0 is the first natural frequency of the span [Hz].
Φ_2 and φ'' are defined in 6.4.5.2 and annex C.

(2) Tables F.1 and F.2 are valid for:

- simply supported bridges with insignificant skew effects that may be modelled as a line beam or slab on rigid supports. Tables F.1 and F.2 are not applicable to half through and truss bridges with shallow floors or other complex structures that may not be adequately represented by a line beam or slab,
- bridges where the track and depth of the structure to the neutral axis from the top of the deck is sufficient to distribute point axle loads over a distance of at least 2,50 m,
- the Train Types listed in F(4),
- structures designed for characteristic values of vertical loads or classified vertical loads with $\alpha \geq 1$ in accordance with 6.3.2,
- carefully maintained track,
- spans with a natural frequency n_0 less than the upper limit in Figure 6.10,

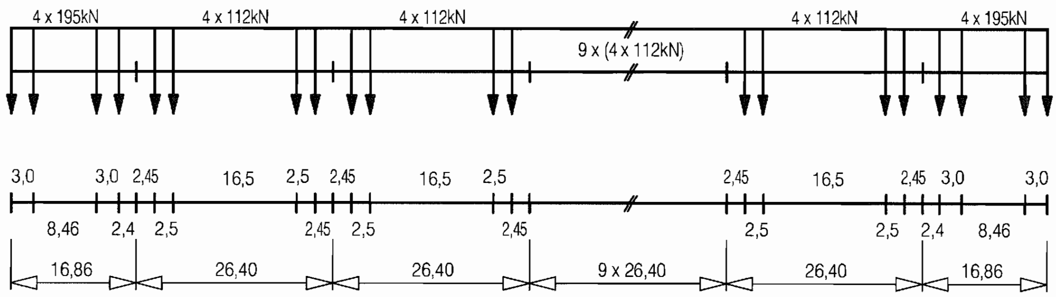
– structures with torsional frequencies n_T satisfying: $n_T > 1.2 \times n_0$

(3) Where the above criteria are not satisfied a dynamic analysis should be carried out in accordance with 6.4.6.

(4) The following Real Trains were used in the development of the criteria in 6.4 and annex F (except Load Model HSLM which is based upon the train types permitted by the relevant interoperability criteria).

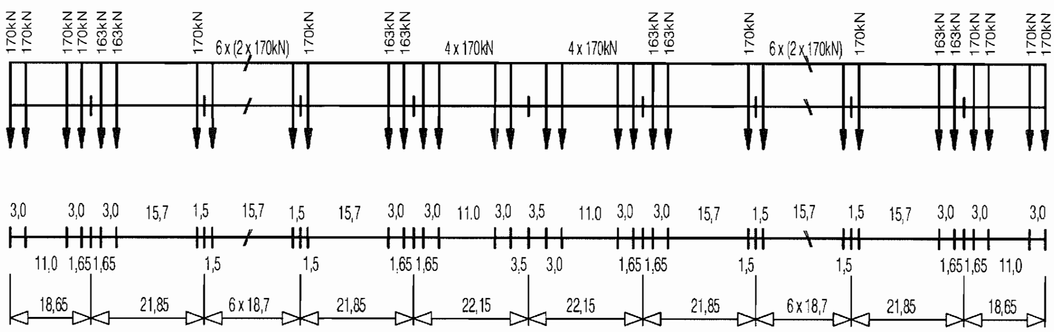
Type A

$\Sigma Q = 6936\text{kN}$ $V = 350\text{km/h}$ $L = 350{,}52\text{m}$ $q = 19{,}8\text{kN/m}'$



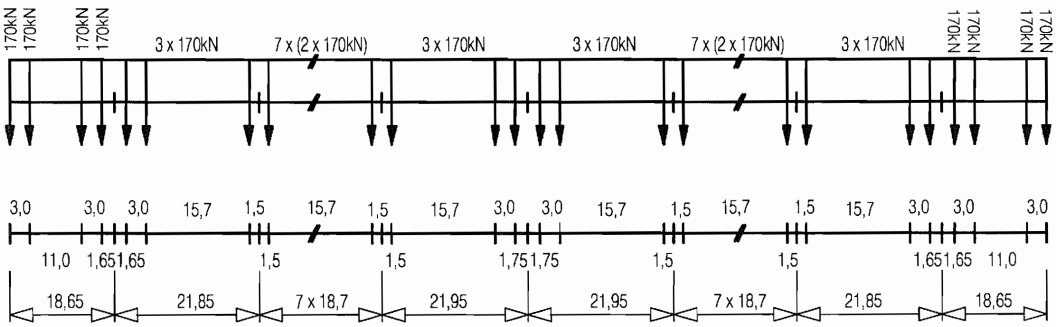
Type B

$\Sigma Q = 8784\text{kN}$ $V = 350\text{km/h}$ $L = 393{,}34\text{m}$ $q = 22{,}3\text{kN/m}'$



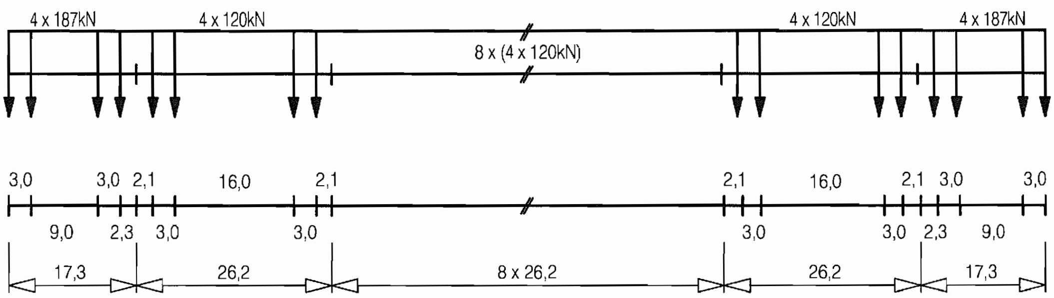
Type C

$\Sigma Q = 8160\text{kN} \quad V = 350\text{km/h} \quad L = 386{,}67\text{m} \quad q = 21{,}1\text{kN/m'}$



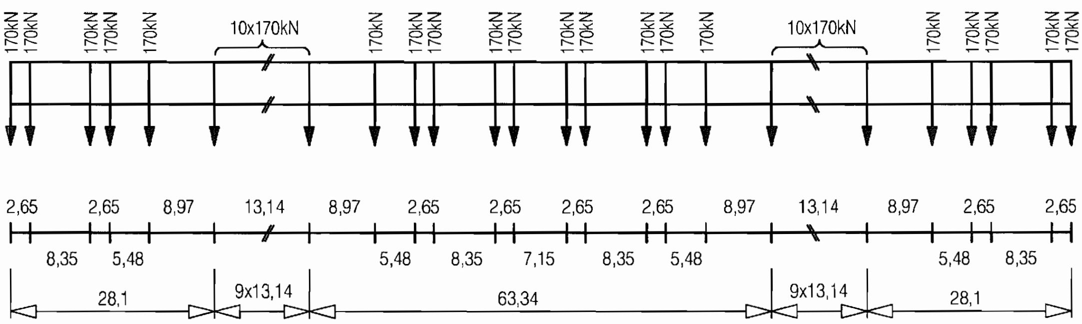
Type D

$\Sigma Q = 6296\text{kN} \quad V = 350\text{km/h} \quad L = 295{,}70\text{m} \quad q = 21{,}3\text{kN/m'}$



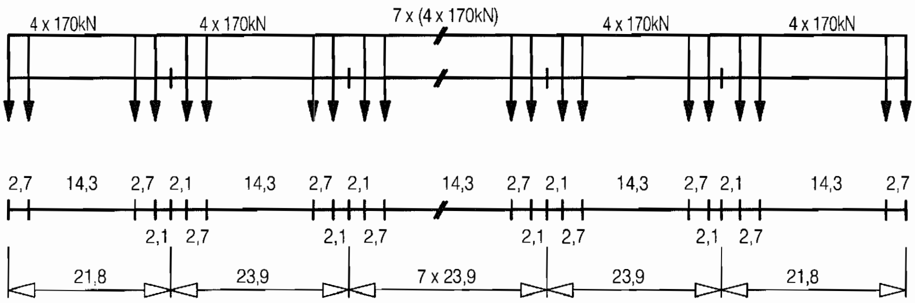
Type E

$\Sigma Q = 6800\text{kN} \quad V = 350\text{km/h} \quad L = 356{,}05\text{m} \quad q = 19{,}1\text{kN/m'}$



Type F

$\Sigma\ Q = 7480\text{kN}\quad V = 350\text{km/h}\quad L = 258{,}70\text{m}\quad q = 28{,}9\text{kN/m'}$



Annex G
(informative)
Method for determining the combined response of a structure and track to variable actions

G.1 Introduction

(1) A method for determining the combined response of a structure and track to variable actions is given below for:
- simply supported or continuous structures consisting of a single bridge deck (G3),
- structures consisting of a succession of simply supported decks (G4),
- structures consisting of a succession of continuous single piece decks (G4).

(2) In each case requirements are given for:
- determining the maximum permissible expansion length L_{TP} which corresponds to the maximum permissible additional rail stresses given in 6.5.4.5.1(1) or the maximum permissible deformation of the structure given in 6.5.4.5.2(1) due to traction and braking and 6.5.4.5.2(2) due to vertical traffic actions. Where the proposed expansion length L_T exceeds the permissible expansion length L_{TP}, rail expansion devices should be provided or a more refined calculation in accordance with the requirements of 6.5.4.1 to 6.5.4.5 carried out.
- determining the longitudinal actions on the fixed bearings due to:
  - traction and braking,
  - temperature variation,
  - end rotation of deck due to vertical traffic loads

(3) In all cases a separate check should be made for compliance with the maximum vertical displacement of the upper surface of a deck given in 6.5.4.5.2(3).

G.2 Limits of validity of calculation method

(1) Track construction:
- UIC 60 rail with a tensile strength of at least 900 N/mm²,
- heavy concrete sleepers with a maximum spacing of 65cm or equivalent track construction,
- at least 30 cm of well consolidated ballast under the sleepers,
- straight track or track radius $r \geq 1500$ m.

(2) Bridge configuration:
- expansion length L_T:
  - for steel structures: $L_T \leq 60$ m,
  - for concrete and composite structures: $L_T \leq 90$ m.

(3) Longitudinal plastic shear resistance k of the track:
- unloaded track: k = 20 to 40 kN per m of track,
- loaded track: k = 60 kN per m of track.

(4) Vertical traffic loading:
- Load Model 71 (and where required Load Model SW/0) with $\alpha = 1$ in accordance with 6.3.2(3),
- Load Model SW/2,

NOTE The method is valid for values of α where the load effects from α x LM71 are less than or equal to the load effects from SW/2.

(5) Actions due to braking:
- for Load Model 71 (and where required Load Model SW/0) and Load Model HSLM:
  $q_{lbk} = 20$ kN/m, limited to a maximum of $Q_{lbk} = 6000$ kN,
- for Load Model SW/2:
  $q_{lbk} = 35$ kN/m.

(6) Actions due to traction:
- $q_{lak} = 33$ kN/m, limited to a maximum of $Q_{lak} = 1000$ kN.

(7) Actions due to temperature:
- Temperature variation ΔT_D of the deck: $\Delta T_D \leq 35$ Kelvin,
- Temperature variation ΔT_R of the rail: $\Delta T_R \leq 50$ Kelvin,
- Maximum difference in temperature between rail and deck:

$$\left| \Delta T_D - \Delta T_R \right| \leq 20 \text{ Kelvin.} \tag{G.1}$$

G.3 Structures consisting of a single bridge deck

(1) Initially the following values should be determined neglecting the combined response of the structure and track to variable actions:
- expansion length L_T and check $L_T \leq \max L_T$ according to G.2(2) and Figure 6.17,
- stiffness K of substructures per track according to 6.5.4.2,
- longitudinal displacement of the upper edge of the deck due to deformation of the deck:

$$\delta = \Theta H \text{ [mm]} \tag{G.2}$$

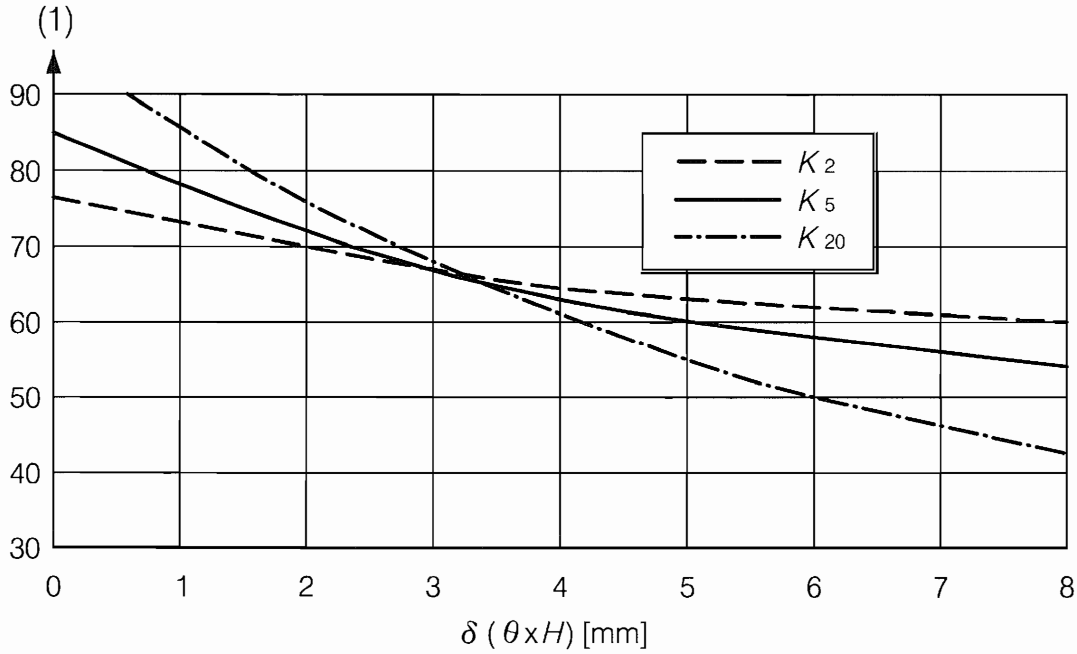
where:

Θ Rotation of the deck end [rad],
H height between (horizontal) axis of rotation of the (fixed) bearing and the surface of the deck [mm],

(2) For the couples of values (unloaded/loaded track) of the longitudinal plastic shear resistance of the track $k = 20/60$ kN per m of track and k=40/60 kN per m of track and the linear temperature coefficient α_T = 10E-6 1/Kelvin or α_T = 12E-6 1/Kelvin the maximum permissible expansion length L_{TP} [m] is given in Figure G.1 to G.4 as appropriate.

Where the point (L_T,δ) describing the expansion length of the deck and longitudinal displacement of the deck end due to vertical traffic actions lies below the corresponding or interpolated curve corresponding to the longitudinal stiffness of the substructure K, the maximum permissible additional rail stresses given in 6.5.4.5.1(1) and the maximum permissible deformation of the structure given in 6.5.4.5.2(1) due to traction and braking and 6.5.4.5.2(2) due to vertical traffic actions are satisfied.

Alternatively, if this condition is not met an analysis may be carried out in accordance with the requirements of 6.5.4.2 to 6.5.4.5 or rail expansion devices should be provided.



Key

(1) Maximum permissible expansion Length L_{TP} [m]

k longitudinal plastic shear resistance of the track [kN per m of track] :
for unloaded tracks:
- k_{20} = 20 kN per m of track and k_{40} = 40 kN per m of track,

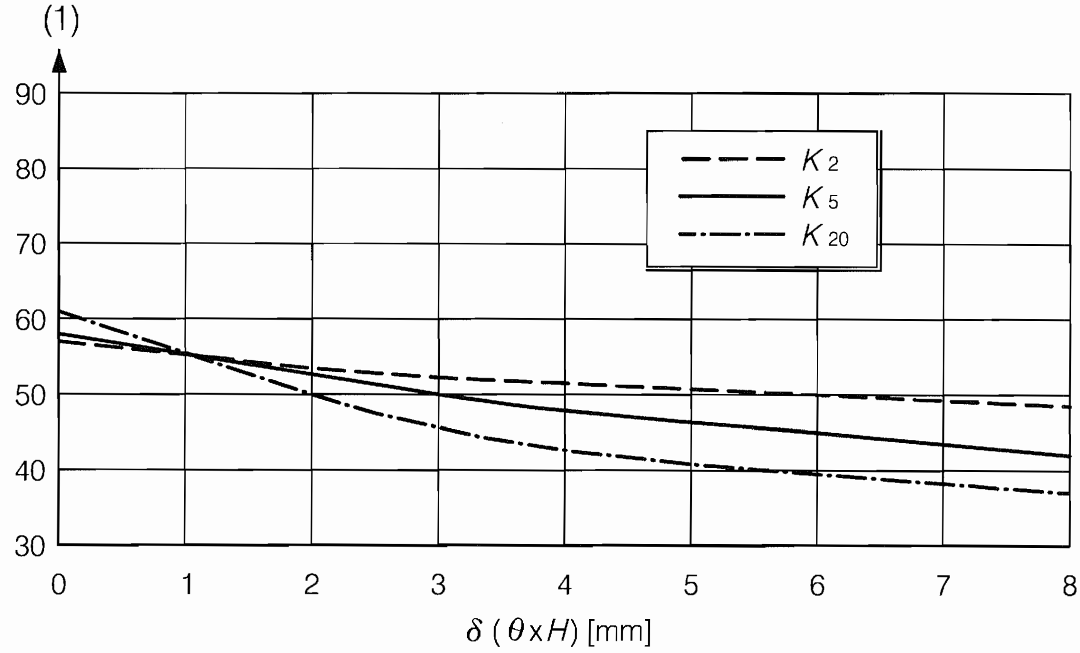
for loaded tracks:
- k_{60} = 60 kN per m of track,

K stiffness of substructure per track per m of deck (i.e. substructure stiffness divided by the number of tracks and by the deck length) [kN/m]:
K_2 = 2E3 kN/m
K_5 = 5E3 kN/m
K_{20} = 20E3 kN/m

α_T linear temperature coefficient [1/Kelvin],

$\delta(\Theta H)$ horizontal displacement of the upper deck edge due to end rotation [mm].

Figure G.1 - Permissible domain for rail stresses in simply supported deck bridges for α_T = 10E-6 [1/Kelvin], ΔT = 35 [Kelvin], k_{20}/k_{60} = 20/60 [kN/m]



Key

(1) Maximum permissible expansion Length L_{TP} [m]

k longitudinal plastic shear resistance of the track [kN per m of track] :
for unloaded tracks:
- k_{20} = 20 kN per m of track and k_{40} = 40 kN per m of track,

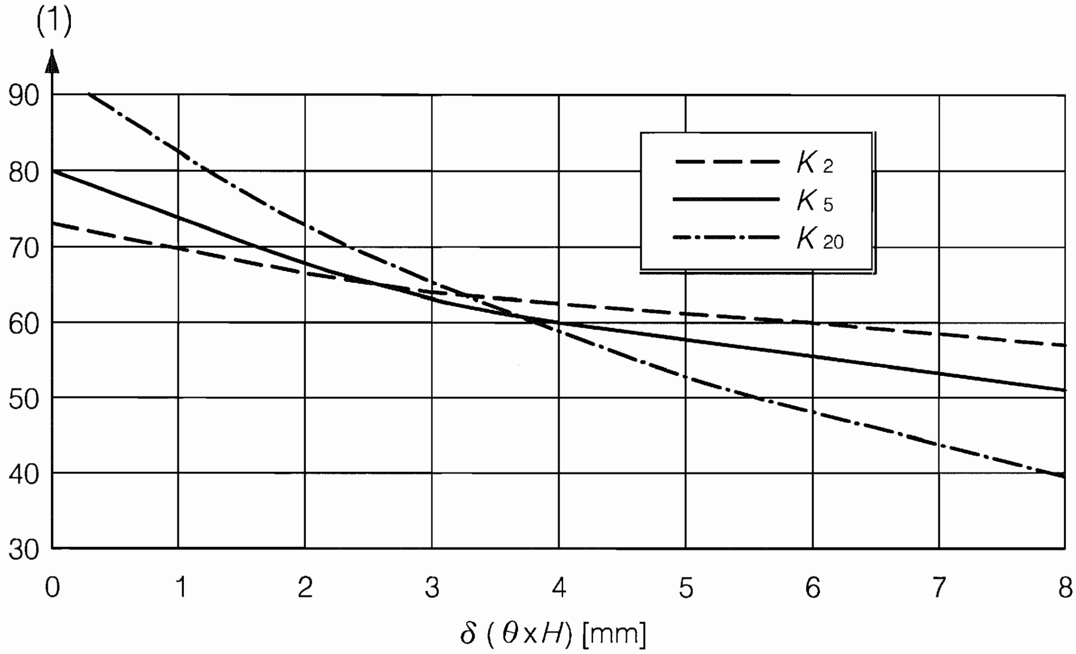
for loaded tracks:
- k_{60} = 60 kN per m of track,

K stiffness of substructure per track per m of deck (i.e. substructure stiffness divided by the number of tracks and by the deck length) [kN/m]:
K_2 = 2E3 kN/m
K_5 = 5E3 kN/m
K_{20} = 20E3 kN/m

α_T linear temperature coefficient [1/Kelvin],

$\delta(\Theta H)$ horizontal displacement of the upper deck edge due to end rotation [mm].

Figure G.2 - Permissible domain for rail stresses in simply supported deck bridges for α_T = 10E-6 [1/Kelvin], ΔT = 35 [Kelvin], k_{40}/k_{60} = 40/60 [kN/m]



Key

(1) Maximum permissible expansion Length L_{TP} [m]

k longitudinal plastic shear resistance of the track [kN per m of track] :
for unloaded tracks:
- k_{20} = 20 kN per m of track and k_{40} = 40 kN per m of track,

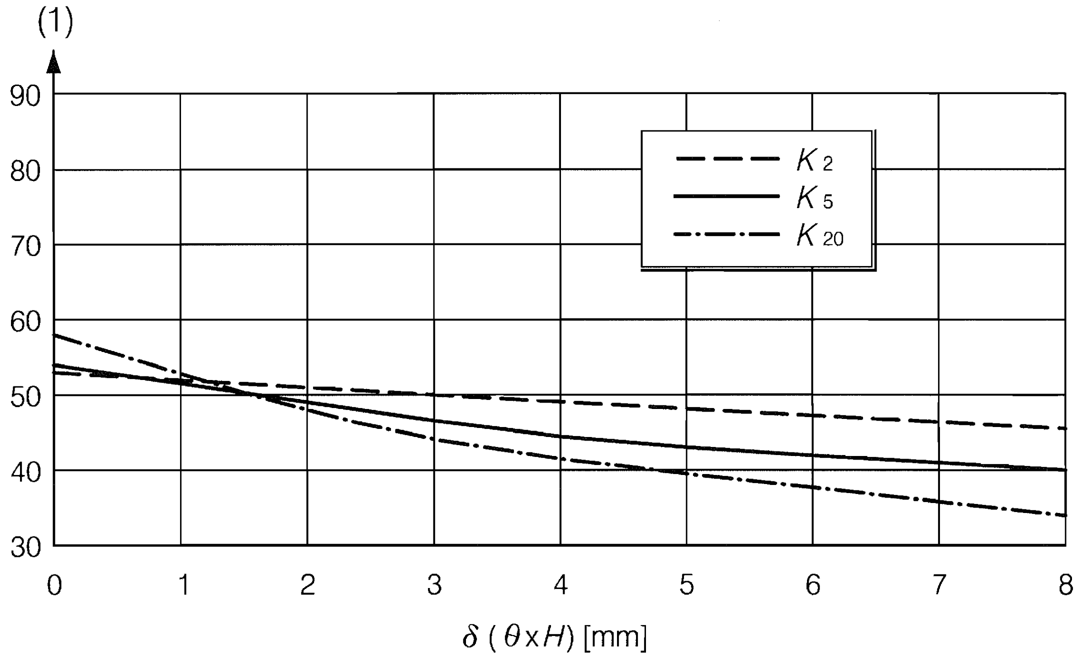
for loaded tracks:
- k_{60} = 60 kN per m of track,

K stiffness of substructure per track per m of deck (i.e. substructure stiffness divided by the number of tracks and by the deck length) [kN/m]:
K_2 = 2E3 kN/m
K_5 = 5E3 kN/m
K_{20} = 20E3 kN/m

α_T linear temperature coefficient [1/Kelvin],

$\delta(\Theta H)$ horizontal displacement of the upper deck edge due to end rotation [mm].

Figure G.3 - Permissible domain for rail stresses in simply supported deck bridges for α_T = 12E-6 [1/Kelvin], ΔT = 35 [Kelvin], k_{20}/k_{60} = 20/60 [kN/m]



Key
(1) Maximum permissible expansion Length L_{TP} [m]

k longitudinal plastic shear resistance of the track [kN per m of track] :
for unloaded tracks:
- k_{20} = 20 kN per m of track and k_{40} = 40 kN per m of track,

for loaded tracks:
- k_{60} = 60 kN per m of track,

K stiffness of substructure per track per m of deck (i.e. substructure stiffness divided by the number of tracks and by the deck length) [kN/m]:
K_2 = 2E3 kN/m
K_5 = 5E3 kN/m
K_{20} = 20E3 kN/m

α_T linear temperature coefficient [1/Kelvin],

$\delta(\Theta H)$ horizontal displacement of the upper deck edge due to end rotation [mm].

Figure G.4 - Permissible domain for rail stresses in simply supported deck bridges for α_T = 12E-6 [1/Kelvin], ΔT = 35 [Kelvin], k_{40}/k_{60} = 40/60 [kN/m]

(3) Actions in the longitudinal bridge direction on the (fixed) bearings due to traction and braking, to temperature variation and due to the deformation of the deck under vertical traffic loads should be determined with the formulae given in Table G.1. The formulae are valid for one track. For two or more tracks with a support stiffness of K_U the actions on the fixed bearings may be determined by assuming a support stiffness of $K = K_U /2$ and multiplying the results of the formulae for one track by 2.

Table G.1 - Actions on the fixed bearings in longitudinal bridge direction [a]

Load case	Limits of validity	Continuous welded rails	With one rail expansion device
Braking [e]	$L \geq 50$ m [d]	$82.10^{-3} \times L^{0,9} \times K^{0,4}$ [b]	$2{,}26 \times L^{1,1} \times K^{0,1}$ [b]
	$L \leq 30$ m [d]	$126.10^{-3} \times L^{0,9} \times K^{0,4}$	$3{,}51 \times L^{1,1} \times K^{0,1}$
Temperature	$20 \leq k$ [kN/m] ≤ 40	$(0{,}34 + 0{,}013k)L^{0,95} \times K^{0,25}$ [c]	$800 + 0{,}5L + 0{,}01\ K/L$ [c] for $L \geq 60$ m $20\ L$ for $L \leq 40$m Interpolated values for $40<L<60$ m
End rotation	Deck bridge	$0{,}11L^{0,22} \times K^{0,5} \times (1{,}1\text{-}\beta) \times \theta H^{0,86}$	Same as continuous welded rail
	Through and half through bridge	$0{,}11L^{0,22} \times K^{0,5} \times (1{,}1\text{-}\beta) \times \theta H$	Same as continuous welded rail

[a] Where rail expansion devices are provided at both ends of the deck all the traction and braking forces are resisted by the fixed bearings. Actions on the fixed bearings due to temperature variation and end rotation due to vertical deflection depend upon the structural configuration and associated expansion lengths.
[b] The braking force applied to the fixed bearings is limited to a maximum of 6000 kN per track.
[c] The force applied to the fixed bearings due to temperature is subject to a limit of 1340 kN where rail expansion devices are provided to all rails at one end of the deck.
[d] For values of L in the range $30 < L < 50$ m linear interpolation may be used to estimate braking effects.
[e] The formulae for braking take into account the effects of traction.

where:

K is the support stiffness as defined above [kN/m],

L depends upon the structural configuration and type of variable action as follows [m]:

- For a simply supported deck with fixed bearing at one end:
  $L = L_T$,
- For a multiple span continuous deck with a fixed bearing at one end:
  for “Braking”:
  $L = L_{Deck}$ (total length of the deck),
  for “Temperature”:
  $L = L_T$,
  for “End rotation due to vertical traffic loads”:
  L = length of the span next to the fixed bearing,
- For a multiple span continuous deck with a fixed bearing at an intermediate position:
  for “Braking”:
  $L = L_{Deck}$ (total length of the deck),
  for “Temperature”:
  the actions due to temperature variation can be determined as the algebraic sum of the support reactions of the two static arrangements obtained by dividing the deck at the fixed bearing section, each deck having the fixed bearing at the intermediate support,

for "End rotation due to vertical traffic loads":
L = length of the longest span at the fixed support,

β is the ratio of the distance between the neutral axis and the surface of the deck relative to the height H [ratio].

G.4 Structures consisting of a succession of decks

(1) In addition to the limits of validity given in G.3 the following limits of validity are applicable:

- the track on the bridge and for at least 100 m on the embankments at both sides consists of continuous welded rail without an expansion device,
- all the decks have the same static arrangement (fixed support at the same end and not on the same pier),
- one fixed bearing is situated on an abutment,
- the length of each deck does not differ more than 20% from the average value of deck length,
- the expansion length L_T of each deck is less than 30m if ΔT_D = 35 Kelvin, or less than 60 m if ΔT_D = 20 Kelvin and there is negligible possibility of frozen ballast. (If the maximum temperature variation of the decks is intermediate between 20 Kelvin and 35 Kelvin, with negligible possibility of frozen ballast, the maximum limit to L_T may be interpolated between 30 m and 60 m),
- the stiffness of the fixed supports is greater than 2E3 x L_T [m] [KN/m of track per track] for L_T = 30 m and 3E3 x L_T [m] [kN/m of track per track] for L_T = 60 m multiplied by the number of tracks, where L_T is in [m],
- the stiffness of each fixed support (with the exception of the fixed support at the abutment) does not differ more than 40% from the average value of the support stiffness,
- the maximum longitudinal displacement, due to deformation of the deck at the top of the slab supporting the track of the deck end with reference to the adjacent abutment, evaluated without taking into account the combined response of structure and track to variable loads, is less than 10 mm,
- the sum of the absolute displacements, due to deformation of the deck at the top of the slab supporting the track, of two consecutive deck-ends, evaluated without taking into account the combined response of structure and track to variable loads, is less than 15 mm.

(2) The longitudinal support reactions F_{Lj} due to temperature variations, traction and braking and deformation of the deck may be determined as follows:

Actions F_{L0} on the fixed bearing (j = 0) on the abutment:

- due to temperature variation:
  $F_{L0}(\Delta T)$ determined by assuming a single deck with the length L_1 of the first deck.

- due to braking and acceleration:
  $$F_{L0} = \kappa \cdot q_{lbk}\,(q_{lak}) \cdot L_1 \qquad \text{(G3)}$$

  where:

  $\kappa = 1$ if the stiffness of the abutment is the same as that of the piers,

$\kappa = 1{,}5$ if the stiffness of the abutment is at least five times greater than that of the piers,

κ may be interpolated for intermediate stiffness,

q_{lak}, q_{lbk} actions due to traction and braking according to clause G.2(5) and G.2(6),

L_i [m]length of the deck connected to the fixed support.

– due to deformation of the deck:

$$F_{L0}(q_V) = F_{L0}(\Theta H) \tag{G.4}$$

determined in accordance with G.3 for single deck bridges where ΘH is in [mm].

Finally, the actions on the fixed bearings on the piers should be determined in accordance with Table G.2.

Table G.2 - Formulae for the calculation of bearing reactions for a succession of decks

Support $j = 0 \dots n$	**Temperature variation** $F_{Lj}(\Delta T)$	**Traction/Braking** $F_{Lj}(q_L)$	**Deformation of the deck** $F_{Lj}(\Theta H)$
Abutment with first fixed bearing $j = 0$	$F_{L0}(\Delta T)$	$F_{L0}(q_L) = \kappa\, q_L\, L_0$	$F_{L0}(\Theta H)$
First pier $j = 1$	$F_{L1}(\Delta T) = 0{,}2\, F_{L0}(\Delta T)$	$F_{L2}(q_L) = q_L\, L_1$	$F_{L1}(\Theta H) = 0$
Intermediate piers $j = m$	$F_{Lm}(\Delta T) = 0$	$F_{Lm}(q_L) = q_L\, L_m$	$F_{Lm}(\Theta H) = 0$
(n-1)th pier $j = (n-1)$	$F_{L(n-1)}(\Delta T) = 0{,}1\, F_{L0}(\Delta T)$	$F_{L(n-1)}(q_L) = q_L\, L_{(n-1)}$	$F_{L(n-1)}(\Theta H) = 0$
(n)th pier $j = n$	$F_{Ln}(\Delta T) = 0{,}5\, F_{L0}(\Delta T)$	$F_{Ln}(q_L) = q_L\, L_n$	$F_{Ln}(\Theta H) = 0{,}5\, F_{L0}(\Theta H)$

NOTE 1 The formulae for braking take into account the effects of traction.

NOTE 2 The braking force applied to the fixed bearings is limited to a maximum of 6000 kN per track.

NOTE 3 The force applied to the fixed bearings due to temperature is subject to a limit of 1340 kN where one rail expansion device is provided.

Annex H
(informative)
Load models for rail traffic loads in Transient Design Situations

(1) When carrying out design checks for Transient Design Situations due to track or bridge maintenance, the characteristic values of Load Model 71, SW/0, SW/2, “unloaded train” and HSLM and associated rail traffic actions should be taken equal to the characteristic values of the corresponding loading given in Section 6 for the Persistent Design Situation.

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