

Eurocode 1: Basis of design and actions on structures —

Part 3: Traffic loads on bridges

**(together with United Kingdom
National Application Document)**

ICS 35.100

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/10, Bridges, upon which the following bodies were represented:

- Association of Consulting Engineers
- British Cement Association
- British Constructional Steelwork Association
- British Precast Concrete Federation
- Concrete Society
- Construction User Group
- County Surveyors' Society
- Department of the Environment, Transport and the Regions - Highways Agency
- Institution of Civil Engineers
- Institution of Structural Engineers
- Railtrack PLC
- Steel Construction Institute
- UK Steel Association
- Welding Institute
- Construction User Group

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National foreword

This publication comprises the English language version of ENV 1991-3:1995, Eurocode 1 — Basis of design and actions on structures — Part 3: Traffic loads on bridges, published by the European Committee for Standardization (CEN), and the National Application Document (NAD) for use with the ENV for traffic loads on bridges located in the United Kingdom (UK).

ENV 1991-3:1995 results from the programme of work initiated by the European Commission to make available a common set of rules for the design of bridges.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard (EN).

The values for certain parameters in the ENV Eurocode may be set by CEN members to conform to the requirements of national regulations. These parameters are designated by boxed values in the ENV. The values to be used in the UK are tabulated in this NAD.

During the ENV period, reference should be made to the supporting documents listed in this NAD.

The purpose of this NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for bridges constructed in the UK. The NAD sections/clauses are numbered in the same way as the ENV, and this NAD takes precedence over corresponding provisions in the ENV.

Users of this document are invited to comment on its technical content, ease of use, and any ambiguities or anomalies. Comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted to an EN.

Comments should be sent in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, if possible, a proposed revision within 2 years of the issue of this document.

For the purposes of this NAD, all references to clauses, tables, figures, and annexes are references to clauses, tables, figures, and annexes in this NAD, unless otherwise specified.

Compliance with ENV 1991-3:1995 and this NAD does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 44, the ENV title page, pages 2 to 130, an inside back cover and a back cover.

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National Application Document

**for use in the UK with
ENV 1991-3:1995**

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Introduction

This NAD has been developed from:

- a) a textual examination of ENV 1991-3:1995;
- b) a parametric calibration against BS 5400-2, DETR implementation and supporting standards and loading data;
- c) trial calculations.

NOTE The relevant authorities are the Technical Approval Authority of the Department of the Environment, Transport and the Regions/Highways Agency, Railtrack PLC, British Railways Board, British Waterways Board, London Underground Limited or other relevant UK authorities responsible for the projects to which the documents are applied.

This NAD covers all loads resulting from traffic on highway and railway bridges. UK requirements related to loads resulting from densities, self-weight and imposed loads (ENV 1991-2-1), fire actions (ENV 1991-2-2), snow loads (ENV 1991-2-3), wind actions (ENV 1991-2-4), thermal actions (ENV 1991-2-5), loads and deformations imposed during execution (ENV 1991-2-6), and accidental actions (ENV 1991-2-7) are specified in the NADs for ENV 1991-2 (to be published separately).

1 General

1.1 Scope

This NAD provides information to enable ENV 1991-3:1995 (EC1: Part 3) to be used to determine traffic loads on bridges to be located in the UK. Loading for highway bridges outside the scope of the ENV shall use loads derived from the relevant parts of the Design Manual for Roads and Bridges (DMRB) (see annex T). Loading for railway bridges outside the scope of the ENV shall be specified by the relevant authority. Departure from any of the requirements given in the NAD should be agreed with the relevant authority.

This NAD does not cover traffic loads required for structural appraisal and assessment of existing constructions, or loads required for assessing repairs and modifications, or for assessing change of use. Separate documentation is required for these (see annex T).

The design documents to be used in provisional application of this NAD are listed in annex T, together with any additional guidance on strength or design recommendations. The data in Tables 4.5.1 to 4.5.10 and the replacement boxed values given in annex X are compatible with these documents. If other documents are used for design aspects, the values given may not necessarily produce the required level of reliability.

Boxed values to be used in this NAD

Relevant ENV boxed values with their replacement NAD values applicable to highway bridges are given in Table X.1.

Replacement NAD boxed values applicable to railway bridges are specified in the text.

Additional annexes of this NAD

This NAD contains additional annexes that do not have equivalent annexes in ENV 1991-3:1995 (annexes S to Y). These are numbered to ensure there is clear distinction from existing (or NAD modified) ENV annexes, and allow for other ENV annexes that may follow.

1.2 Normative references

The following normative documents contain provisions that, through reference in this text, constitute provisions of this National Application Document. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

These documents shall be used in conjunction with the relevant UK NAD when published.

ENV 1991-1-1:1994, *Basis of design and actions on structures — Basis of design.*

ENV 1991-2-1:1995, *Actions on structures — Densities, self-weight and imposed loads.*

ENV 1991-2-2:1995, *Actions on structures exposed to fire.*

ENV 1991-2-3:1995, *Actions on structures — Snow loads.*

ENV 1991-2-4:1995, *Actions on structures — Wind actions.*

ENV 1991-2-5:1997, *Actions on structures — Thermal actions.*

ENV 1991-2-6:1997, *Actions on structures — Loads and deformations imposed during erection.*

ENV 1991-2-7:1998, *Actions on structures — Accidental actions due to impact and explosions.*

ENV 1997-1:1994, *Geotechnical design — General rules.*

1.3 Partial safety factors, combination factors and other values

1.3.1 Highway bridges

- a) The values for partial safety factors on actions γ should be the values given in Tables 4.5.1 and 4.5.2.
- b) The values for combination and reduction factors ψ should be the values given in Tables 4.5.3 and 4.5.4.
- c) The values of the adjustment factors a should be the values given in Tables 4.5.5 and 4.5.6.
- d) The combinations to be considered for design in terms of groups of loads (including the dominant and multi-component actions) are given in Tables 4.5.7, 4.5.8, 4.5.9, and 4.5.10.

1.3.2 Railway bridges

- a) The values for partial safety factors on actions γ should be the values given in Tables G1/2(a) and (b).
- b) The values for combination and reduction factors ψ should be the values given in Tables G1/2(a) and (b).
- c) The combinations to be considered for design in terms of groups of loads (including the dominant and multi-component actions) are given in Table 6.6(a).

1.4 Definitions

ENV 1991-3 uses terminology that may not be entirely familiar to UK engineers. Definitions of terms are given in ENV 1991-1.

1.5 Notation

ENV 1991-3 uses some notation that differs from notation generally used by UK engineers. The ENV notation should be adopted when using this NAD.

2 Classification of actions

2.1 General

Unless agreed otherwise by the relevant authority, the required design life shall be 120 years for highway bridges and 100 years for railway bridges. The design models for traffic actions given in the NAD shall be applied to all categories of bridges and classes of route or line, unless the relevant authority makes specific exclusions or additions for a particular project.

2.2 Variable actions

The design working life for variable actions shall be consistent with the design working life adopted for traffic actions. The basis of combinations of variable actions for the Ultimate Limit State in this NAD is such that equal probability of exceedance of loading is achieved for any combination, whether including frequent, infrequent or quasi-permanent values in the combination considered.

2.3 Accidental actions

The accidental actions given in this NAD shall apply to all projects, unless the relevant authority specifies more onerous requirements for particular projects.

3 Design situations and requirements

Design values shall generally be derived for all load cases for all design situations before selecting the critical load cases for the governing design situations. The governing combinations shall be identified for each element to be designed.

Account shall be taken in the total and coexistent load effects of the contributions resulting from consideration of adverse and relieving areas and effects.

The following effects or actions shall be considered with regard to the governing design situations.

- a) The effects of shrinkage, creep, residual stresses shall be considered.
- b) Differential settlement shall be considered, if required by the relevant authority. (The nominal values to be provided for, as well as load factors and combination requirements, if differential settlement is to be considered, shall be agreed with the relevant authority.)
- c) Considerations of seismic actions and combinations are not required, unless specifically stated by the relevant authority.
- d) Considerations of snow loads and combinations are not required, except for the execution condition relating to fabrication, erection and construction, or if exceptional loads are likely to be encountered.
- e) Overturning and stability of the structure and its parts shall be considered.
- f) Earth pressures and loads relating to the design of bridge foundations, abutments, retaining walls, and piers, etc., shall be based on the principles of ENV 1997-1 and the associated NAD.

4 Road traffic actions and other actions specifically for highway bridges

4.1 Field of application

For bridges with any individual span or loaded length greater than 200 m, or with carriageway widths greater than 42 m, the traffic models and other loading requirements shall be as agreed with the relevant authority.

Any specific models relating to weight limitations and construction sites or other special situations shall also be agreed with the relevant authority.

4.2 Representation of actions

The representative load models, loading classes, divisions into notional lanes, location and numbering of lanes, and application of the load models on the individual lanes given in this NAD shall apply unless otherwise agreed with the client and the relevant authority.

4.3 Vertical loads: characteristic values

4.3.1 General and associated design situations

Load models 1 and 2 shall be applied in all general and local verifications.

Load model 3 shall be applied to account for abnormal vehicle loads, in accordance with the classification of the bridges, unless the relevant authority specifies other special vehicles; and shall be applied in accordance with 4.3.4, unless any departures in the method of application or restriction or additional requirements are specified by the relevant authority.

Load model 4 shall be applied only to footways and to cycle tracks, unless specified otherwise by the relevant authority, and shall not be applied to the central reservation unless specified by the relevant authority.

4.3.2 Load model 1: main loading system

Adjustment factors a shall conform to Tables 4.5.5 and 4.5.6 and the minimum adjustment factor given in annex X.

The specified values shall apply unless agreed otherwise with the relevant authority. In certain circumstances, by agreement with the relevant authority, upper and lower bound values to a may be provided. The replacement simplifications in 4.3.2(6) of ENV 1991-3:1995 may be applied unless otherwise restricted by the relevant authority.

4.3.3 Load model 2: single axle model

The single axle adjustment factor β_Q shall be taken as equal to a_{Q1} and the contact areas shall conform to Figure 4.3 of ENV 1991-3:1995 unless otherwise specified by the relevant authority. This model should be considered for general as well as local verifications.

4.3.4 Load model 3: special vehicles

Unless specified otherwise by the relevant authority, the set of models to be applied for all design purposes shall consist of the following pairs of classes of special vehicles (see Table A.1 of ENV 1991-3:1995):

- a) for motorways and trunk road systems: 2 No. 900/150;
- b) for principal roads (not forming part of a motorway or trunk road system): 1 No. 900/150 + 1 No. 600/150;
- c) for other public roads: 2 No. 600/150.

In each of a) to c), the pair of vehicles shall be considered to be located anywhere in the most unfavourable lane, but shall not be closer together than 1.5 m, with each vehicle of the pair having the associated gap to LM1 loading as shown in Figure 4.5 of ENV 1991-3:1995.

Other classes may be required for special routes, and these should be specified by the relevant authority to account for special cases of abnormal loads. However, in all cases, whichever special vehicles are adopted, the description and axle arrangements for general verifications shall conform to Table A.2 of ENV 1991-3:1995 for each class of special vehicle used, and the positioning shall conform to Figures 4.4 and 4.5 of ENV 1991-3:1995 and to Figure 4.5.1. Similarly, for local variations, the loads from each axle-line shall conform to Figure A.1 of ENV 1991-3:1995.

In relation to 4.3.4(3)(c) of ENV 1991-3:1995, the coincident main loading system shall consist of 1.00 times the loading from the tandem system combined with 0.88 times the UDL, instead of the frequent values.

4.3.5 Load model 4: crowd loading

All elements supporting footway and cycle tracks shall have uniform loading applied in accordance with clause 5. The same loading shall be applied to carriageways for crowd loading.

The concentrated load given in 5.3.2.2 of ENV 1991-3:1995 shall only be considered in local verifications of elements.

4.3.6 Distribution of loading and dispersal of concentrated loads

The distribution analysis of the structure shall conform to the DMRB.

No allowance for dispersal of UDLs shall be made.

Dispersal of 1 horizontally and 1 vertically throughout any combination of pavement on concrete or orthotropic steel decks shall generally be applied. However, for asphalt surfacings the more onerous spread to depth ratio of 1 horizontally to 2 vertically through the asphalt thickness shall be used, unless specified otherwise by the relevant authority.

4.4 Horizontal forces: characteristic values

4.4.1 Braking, acceleration and skidding

Consideration of lateral or skew braking is not required unless specifically stated by the relevant authority (see also annex T).

The braking or acceleration load derived shall not be taken as less than 300 kN, to account for longitudinal skidding, unless specified otherwise by the relevant authority.

4.4.2 Centrifugal forces

Centrifugal forces may be derived and applied separately for each lane, i.e. using Q_v appropriate to each lane (see also annex T).

4.5 Groups of traffic loads on highway bridges

Table 4.5.7 (ULS) and Tables 4.5.8 to 4.5.10 (SLS) and associated notes give the common combinations and groups that shall be considered for most highway bridges, including, where relevant, reference to other loads resulting from ENV 1991-2. Footway and cycle track loads are incorporated. Reference is made to development of the combinations and groups required for the most common specific criteria considered in the design documents.

NOTE The relevant authority may require other specific combinations to be considered.

If special maintenance provisions are required to be considered for design, the required groups of loads shall be agreed with the relevant authority.

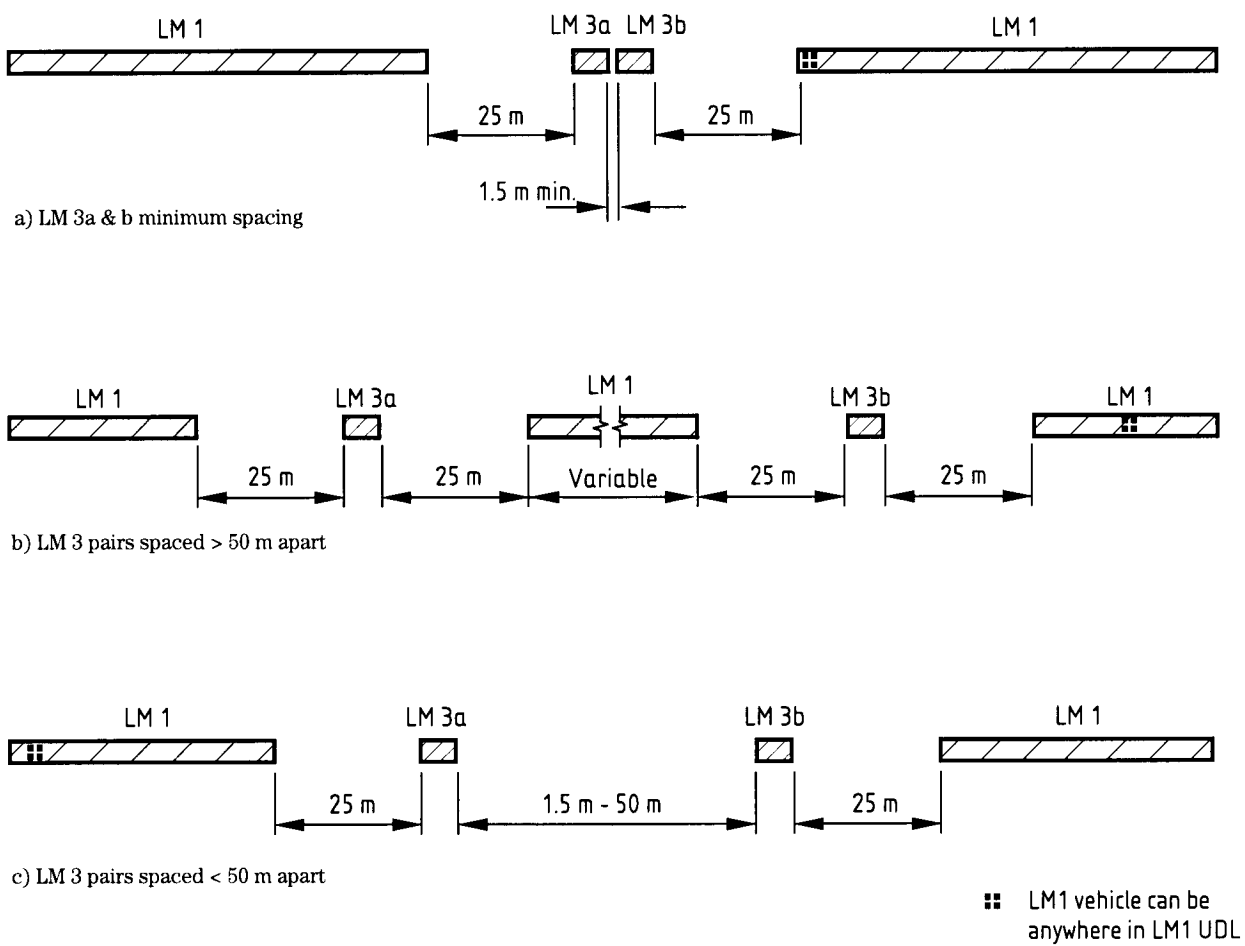


Figure 4.5.1 — Locations of special vehicles

The partial factors on actions in the ultimate limit state for highway bridges given in Table 4.5.1 shall replace ENV 1991-3:1995, C.2.3 and Table C.1.

Table 4.5.1 — Partial factors on actions in the ultimate limit state for highway bridges

Action	Symbol	Situation ^a	
		P/T	A
Permanent actions ^{bc}			
unfavourable	γ_{Gsup}	$\left\{ \begin{array}{l} 1.35 \text{ (superload)}^{def} \\ 1.20 \text{ (ground conditions)}^c \\ 1.35 \text{ (concrete)}^{def} \\ 1.20 \text{ (steel)}^{def} \end{array} \right.$	$\left\{ \begin{array}{l} 1 \\ 1 \\ 1 \\ 1 \end{array} \right.$
favourable	γ_{Ginf}	1^{def}	1
Prestress	γ_P	1^g	1
Settlement	γ_{Gset}	1^h	1
Traffic actions ⁱ	γ_Q		
unfavourable		1.50	1
favourable		0	0
Other variable actions ^j	γ_Q		
unfavourable		1.50	1
favourable		0	0
Accidental actions	γ_A	0	1
NOTE For considerations of static equilibrium and other similar stability cases, the criteria of ENV 1991-1:1995, C.2.3(2) shall be applied using $\gamma_{Gsup} = 1.05$ and $\gamma_{Ginf} = 0.95$.			
^a P: persistent situation; T: transient situation; A: accidental situation.			
^b i.e. Self-weight of structural and non-structural elements, permanent actions caused by ground, ground-water, and free water.			
^c Instead of using γ_G (1.35) and the usual γ_Q for lateral earth pressure actions, the design ground properties may be introduced in accordance with ENV 1997. A model factor γ_{Sd} is applied.			
^d In this verification the characteristic values of all permanent actions from one source are multiplied by γ_{Gsup} if the total resulting action effect is unfavourable, and by γ_{Ginf} if favourable [see also ENV 1991-1, 9.4.2(3)(a)].			
^e Unless otherwise specified, the factors apply to the appropriate characteristic values defined in ENV 1991-2.1 (especially for the weight of road pavement).			
^f In cases in which the limit state is sensitive to variations in space of permanent actions, the upper and lower characteristic values of these actions should be taken in accordance with ENV 1991-3:1995, 4.2(3)P.			
^g Unless otherwise specified. For prestress by tendons, this factor applies to the appropriate characteristic values defined in the relevant design Eurocode. Where prestress is induced by deformations imposed on the structure, the factors on G and the imposed deformations should be as defined in the relevant design Eurocode.			
^h Applicable only where settlements are to be assessed as the best estimate (see the design Eurocodes).			
ⁱ The components of traffic actions are introduced in combinations as one action, by the relevant group of loads g_{ri} , with the favourable components of these groups neglected.			
^j See Introduction.			

The partial factors on actions in the serviceability limit state for highway bridges given in Table 4.5.2 shall replace ENV 1991-3:1995, C.3.3.

Table 4.5.2 — Partial factors on actions in the serviceability limit state for highway bridges

Action	Symbol	Situation ^a	
		P/T	A
Permanent actions ^{bc}			
unfavourable	γ_{Gsup}	1 ^{def}	n/a
favourable	γ_{Ginf}	1 ^{def}	n/a
Prestress	γ_P	1 ^g	n/a
Settlement	γ_{Gset}	1 ^h	n/a
Traffic actions ⁱ	γ_Q		
unfavourable		1.10	n/a
favourable		0	n/a
Other variable actions ^j	γ_Q		
unfavourable		1.10	n/a
favourable		0	n/a
Accidental actions	γ_A	0	n/a

^a P: persistent situation; T: transient situation; A: accidental situation.

^b i.e. Self-weight of structural and non-structural elements, permanent actions caused by ground, ground-water, and free water.

^c Instead of using γ_G (1.35) and the usual γ_Q for lateral earth pressure actions, the design ground properties may be introduced in accordance with ENV 1997. A model factor γ_{sd} is applied.

^d In this verification the characteristic values of all permanent actions from one source are multiplied by γ_{Gsup} if the total resulting action effect is unfavourable, and by γ_{Ginf} if favourable [see also ENV 1991-1, 9.4.2(3)(a)].

^e Unless otherwise specified, the factors apply to the appropriate characteristic values defined in ENV 1991-2.1 (especially for the weight of road pavement).

^f In cases in which the limit state is sensitive to variations in space of permanent actions, the upper and lower characteristic values of these actions should be taken in accordance with ENV 1991-1, 4.2(3)P.

^g Unless otherwise specified. For prestress by tendons, this factor applies to the appropriate characteristic values defined in the relevant design Eurocode. Where prestress is induced by deformations imposed on the structure, the factors on G and the imposed deformations should be as defined in the relevant design Eurocode.

^h Applicable only where settlements are to be assessed as the best estimate (see the design Eurocodes).

ⁱ The components of traffic actions are introduced in combinations as one action, by the relevant group of loads g_{ri} , with the favourable components of these groups neglected.

^j See Introduction.

Combination and reduction factors for highway bridges for the ultimate limit state are given in Table 4.5.3.

Table 4.5.3 — Combination and reduction factors for highway bridges for the ultimate limit state

Action	Symbol	ψ_0	ψ'_1	ψ_1	ψ_2
Traffic loads	gr1 TS	0.94	0.98	0.94	0
	(LM1) UDL	0.60	0.70	0.60	0
	Single axle (LM2)	0	0.87	0.75	0
	gr2 (horizontal forces)	0	0	0	0
	gr3 (pedestrian loads)	0	0.80	0.40	0
	gr4 (LM4)	0	0.80	0	0
	gr5 (LM3)	0	1	0	0
Horizontal forces	—	0	0	0	0
Wind force	F_{Wk} or F_{Wn}	0.30	0.60	0.50	0
	F_W^*	1	0	0	0
Temperature effect	T_k	0 ^a	0.80 ^a	0.60 ^a	0.50 ^a

^a Subject to agreement with the relevant authority.

Combination and reduction factors for highway bridges for the serviceability limit state are given in Table 4.5.4.

Table 4.5.4 — Combination and reduction factors for highway bridges for the serviceability limit state

Action	Symbol	ψ_0	ψ'_1	ψ_1	ψ_2
Traffic loads	gr1 TS	0.94	0.98	0.94	0
	(LM1) UDL	0.60	0.70	0.60	0
	Single axle (LM2)	0	0.87	0.75	0
	gr2 (horizontal forces)	0	0	0	0
	gr3 (pedestrian loads)	0	0.80	0.40	0
	gr4 (LM4)	0	0.80	0	0
	gr5 (LM3)	0	1	0	0
Horizontal forces	—	0	0	0	0
Wind force	F_{Wk} or F_{Wn}	0.30	0.60	0.50	0
	F_W^*	1	0	0	0
Temperature effect	T_k	0.60 ^a	0.80 ^a	0.60 ^a	0.50 ^a

^a Subject to agreement with the relevant authority.

Adjustment factors a_Q for axle loads for highway bridges are given in Table 4.5.5.

NOTE Table 4.5.5 provides an opportunity to introduce site-specific loadings if required.

Table 4.5.5 — Adjustment factors a_Q for axle loads for highway bridges

Location	Adjustment factor a_Q (tandem system)
Lane 1 ^a	$a_{Q1} = 0.844a_{SL}$
Lane 2	$a_{Q2} = 1.267$
Lane 3	$a_{Q3} = 1.267$

NOTE $\beta_Q = a_{Q1}$, but β_Q may be increased above a_{Q1} or decreased down to a_{Qmin} to cover axle loads from special vehicles compatible with axle load limitations specified by the relevant authority.

^a For single lane carriageways, $a_{SL} = 0.69$; for multiple lane carriageways, $a_{SL} = 1.00$.

Adjustment factors a_q for distributed loads for highway bridges are given in Table 4.5.6.

NOTE This table provides the opportunity to introduce site-specific loading if required.

Table 4.5.6 — Adjustment factors a_q for distributed loads for highway bridges

Location	Adjustment factor a_q (uniform load system)
Lane 1 ^a	$a_{q1} = 0.40/a_{SL}$
Lane 2	$a_{q2} = 1.44$
Lane 3	$a_{q3} = 1.44$
Other lanes	$a_{qn} = 1.44$
Remaining area	$a_{qr} = 1.44$

^a For single lane carriageways, $a_{SL} = 0.69$; for multiple lane carriageways, $a_{SL} = 1.00$.

Notes to Tables 4.5.7, 4.5.8, 4.5.9 and 4.5.10

Double outline in ENV 1991-3:1995, Table 4.4, or bold indicates dominant component action [i.e. characteristic values (designated as component associated with group); see ENV 1991-3:1995, **4.5** and Table 4.4].

Table 4.5.7 (for ULS) and Tables 4.5.8, 4.5.9 and 4.5.10 (for SLS) specify the combined factors that shall be applied to the characteristic values of actions for the common design combinations of actions applicable to most bridges, using the relevant γ , ψ , and α factors from Tables 4.5.1 to 4.5.6 for the characteristic, frequent, infrequent, and quasi-permanent (see footnote ^m) values of the multi-component traffic actions, derived in accordance with ENV 1991-3:1995, **4.5**.

For ULS, the combinations of actions accord with the equation in ENV 1991-1:1994, **9.4.2**, using partial factors on actions given in Table 4.5.1. For SLS, the characteristic (rare) frequent and quasi-permanent combinations accord with the relevant equations in ENV 1991-1:1994, **9.5.2**, and the infrequent combination with the additional equation in ENV 1991-3:1995, **C.3.2(2)**, in each case multiplied by the relevant partial factor on actions given in Table 4.5.2.

The values in the tables are the unfavourable factor that shall be applied to the characteristic values given in the relevant clause of the ENV (i.e., the combined factor includes any α or ψ factor that shall be applied to the characteristic loads, such as given in Tables 4.5.3 to 4.5.6.

Reduced factors appropriate to favourable aspects of permanent actions are shown in parenthesis, if not equal to 1.0. There are no reduced values appropriate to variable actions, as these shall be taken as zero if favourable.

Footnotes

^a Two characteristic values should be taken to act simultaneously in group 2.

^b Group 3 is not relevant if the service vehicle is not specified and if group 4 is considered.

^c The special vehicles LM3 replace part of the LM1 loading. The factors on the LM3 vehicles and the coincident LM1 loading are included in the Group 5 combinations in accordance with **4.3.4**. Alternative use of LM2 also applies (because LM2 does not govern for lengths greater than 50 m, consideration of trailing load for this alternative is not relevant).

^d Factors for permanent actions have been provisionally taken as specified in the ENV [not as in the DMRB (BD37/88); see further notes in annex T].

^e Values use α for multiple lane carriageways, see footnote ^a to Tables 4.5.5 and 4.5.6 for single lane carriageways.

^f The single axle model (LM2) should be considered separately and should be considered not to act simultaneously with any other load model or variable load, except as specified in footnote ^c.

^g Only one footway/cycle track should be taken as loaded if the effect is more unfavourable than the effect of two loaded. The combined factors shown for the UDL on footways and cycletrack shall be applied to the characteristic value of 5 kN/m². This characteristic value shall only be considered for road bridges as specified in ENV 1991-3:1995, **5.3.2(3)**, and allowance shall be made for the reduced combination value to be taken as 2.5 kN/m², where appropriate, as specified in **4.5.1** of ENV 1991-3:1995.

^h For footbridges (see ENV 1991-3:1995, **D.2.1**) group 3 or group 4 only loads shall be taken in combination (see ENV 1991-3:1995, clause **5**).

ⁱ These actions shall be considered in turn with other actions. See also ENV 1991-3:1995, Table C.2, notes 4 and 5.

^j The wind only case shall also be considered at ULS and SLS (i.e. without traffic; see ENV 1991-3:1995, Table C.2, note 4).

^k The combination with temperature as the dominant variable action should also be considered, using relevant γ and ψ values from Tables 4.5.1 to 4.5.4.

^l See note to **C.2.1.2(4)** of ENV 1991-3:1995.

^m The quasi-permanent combination shall be as specified for frequent combination group 3, but with no footway and cycletrack loading.

Factors on characteristic actions that shall be taken for combination of actions for highway bridges, including appropriate values of γ and ψ (and α where relevant) (ULS) are given in Table 4.5.7.

Table 4.5.7 — Factors on characteristic actions for combination of actions for highway bridges (ULS)

Load type and system	Groups of loads									
	Group 1		Group 2 ^a		Group 3 ^b		Group 4		Group 5 ^c	
a) Permanent actions ^d										
Direct actions										
self-weight: concrete	1.35		1.35		1.35		1.35		1.35	
self-weight: steel	1.20		1.20		1.20		1.20		1.20	
superload: surfacing	1.35		1.35		1.35		1.35		1.35	
superload: other	1.35		1.35		1.35		1.35		1.35	
ground conditions	1.20		1.20		1.20		1.20		1.20	
Indirect actions										
prestress/creep	1.00		1.00		1.00		1.00		1.00	
settlement	1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)	
b) Variable actions										
Carriageway	TS	UDL	TS	UDL					TS	UDL
Vertical forces										
main loading system: LM1										
lane 1 ^e	1.27	0.60	1.19	0.36	0		0		1.27	0.53
lane 2	1.90	2.16	1.79	1.30	0		0		1.90	1.90
lane 3	1.90	2.16	1.79	1.30	0		0		1.90	1.90
other lanes	0	2.16	0	1.30	0		0		0	1.90
remaining area	0	2.16	0	1.30	0		0		0	1.90
special vehicles: LM3	0		0		0		0		1.50^c	
single axle: LM2 ^f	$1.50\beta_Q$		$1.50\beta_Q$		$1.50\beta_Q$		$1.50\beta_Q$		$1.50\beta_Q$	
crowd loading: LM4	0		0		0		1.50		0	
Horizontal forces										
braking and acceleration	0		1.50^a		0		0		0	
centrifugal forces	0		1.50^a		0		0		0	
Footway and cycletracks ^{gh}	UDL ^g	S veh.			UDL ^g	S veh.	UDL	S veh.		
	0.75	0	0	0	1.50	0	1.50	1.50	0	0
Wind ^{ij}										
F_{Wk} or F_{Wn}	0.45		0		0		0		0	
or F_W^*	1.50		0		0		0		0	
Temperature ^{ik}										
T_k	0		0		0		0		0	
c) Accidental actions										
Under bridge	0		1.00		1.00		0		1.00	
On bridge	0		0		0		1.00 ^l		0	
NOTE Sveh. = service vehicle.										

Factors on characteristic actions that shall be taken for the characteristic (rare) combination for actions for highway bridges, including appropriate values of γ and ψ (and α where relevant) (SLS) are given in Table 4.5.8.

Table 4.5.8 — Factors on characteristic actions for characteristic (rare) combination of actions for highway bridges (SLS)

Load type and system	Groups of loads									
	Group 1		Group 2 ^a		Group 3 ^b		Group 4		Group 5 ^c	
a) Permanent actions ^d										
Direct actions										
self-weight: concrete	1.00		1.00		1.00		1.00		1.00	
self-weight: steel	1.00		1.00		1.00		1.00		1.00	
superload: surfacing	1.00		1.00		1.00		1.00		1.00	
superload: other	1.00		1.00		1.00		1.00		1.00	
ground conditions	1.00		1.00		1.00		1.00		1.00	
Indirect actions										
prestress/creep	1.00		1.00		1.00		1.00		1.00	
settlement	1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)	
b) Variable actions										
Carriageway	TS	UDL	TS	UDL					TS	UDL
Vertical forces										
main loading system: LM1										
lane 1 ^e	0.93	0.44	0.87	0.26	0		0		0.93	0.39
lane 2	1.39	1.58	1.31	0.95	0		0		1.39	1.39
lane 3	1.39	1.58	1.31	0.95	0		0		1.39	1.39
other lanes	0	1.58	0	0.95	0		0		0	1.39
remaining area	0	1.58	0	0.95	0		0		0	1.39
special vehicles: LM3	0		0		0		0		1.10^c	
single axle: LM2 ^f	$1.1\beta_Q$		$0.83\beta_Q$		0		0		or $1.1\beta_Q$	
crowd loading: LM4	0		0		0		1.10		0	
Horizontal forces										
braking and acceleration	0		1.10^a		0		0		0	
centrifugal forces	0		1.10^a		0		0		0	
Footway and cycletracks ^{gh}	UDL ^g	S veh.			UDL ^g	S veh.	UDL ^g	S veh.		
	0.55	0	0	0	1.10	0	1.10	1.10	0	0
Wind ^{ij}										
F_{Wk} or F_{Wn}	0.33		0		0		0		0	
or F_{W^*}	1.10		0		0		0		0	
Temperature ^{ik}										
T_k	0.66		0.66		0.66		0.66		0.66	
c) Accidental actions										
Under bridge	n/a		n/a		n/a		n/a		n/a	
On bridge	n/a		n/a		n/a		n/a		n/a	

NOTE Sveh. = service vehicle.

Factors on characteristic actions that shall be taken for the infrequent combination of actions for highway bridges, including appropriate values of γ and ψ (and α where relevant) (SLS) are given in Table 4.5.9.

Table 4.5.9 — Factors on characteristic actions for infrequent combination of actions for highway bridges (SLS)

Load type and system	Groups of loads									
	Group 1		Group 2 ^a		Group 3 ^b		Group 4		Group 5 ^c	
a) Permanent actions ^d										
Direct actions										
self-weight: concrete	1.00		1.00		1.00		1.00		1.00	
self-weight: steel	1.00		1.00		1.00		1.00		1.00	
superload: surfacing	1.00		1.00		1.00		1.00		1.00	
superload: other	1.00		1.00		1.00		1.00		1.00	
ground conditions	1.00		1.00		1.00		1.00		1.00	
Indirect actions										
prestress/creep	1.00		1.00		1.00		1.00		1.00	
settlement	1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)		1.00 (0)	
b) Variable actions										
Carriageway	TS	UDL	TS	UDL					TS	UDL
Vertical forces										
main loading system: LM1										
lane 1 ^e	0.91	0.31	0.87	0.26	0		0		0.91	0.27
lane 2	1.36	1.11	1.31	0.95	0		0		1.36	0.98
lane 3	1.36	1.11	1.31	0.95	0		0		1.36	0.98
other lanes	0	1.11	0	0.95	0		0		0	0.98
remaining area	0	1.11	0	0.95	0		0		0	0.98
special vehicles: LM3	0		0		0		0		1.10^c	
single axle: LM2 ^f	0.96β _Q		0.83β _Q		0		0		or 0.96β _Q	
crowd loading: LM4	0		0		0		0.88		0	
Horizontal forces										
braking and acceleration	0		0^a		0		0		0	
centrifugal forces	0		0^a		0		0		0	
Footway and cycletracks ^{gh}	UDL ^g	S veh.			UDL ^g	S veh.	UDL	S veh.		
	0.55	0	0	0	0.88	0	0.88	0.88	0	0
Wind ^{ij}										
<i>F</i> _{Wk} or <i>F</i> _{Wn}	0.55		0		0		0		0	
or <i>F</i> _W [*]	0		0		0		0		0	
Temperature ^{ik}										
<i>T</i> _k	0.66		0.66		0.66		0.66		0.66	
c) Accidental actions										
Under bridge	n/a		n/a		n/a		n/a		n/a	
On bridge	n/a		n/a		n/a		n/a		n/a	
NOTE Sveh. = service vehicle.										

Factors on characteristic actions that shall be taken for the frequent combination of actions for highway bridges, including appropriate values of γ and ψ (and a where relevant) (SLS) are given in Table 4.5.10.

Table 4.5.10 — Factors on characteristic actions for frequent combination of actions for highway bridges (SLS)

Load type and system	Groups of loads						
	Group 1		Group 2 ^a	Group 3 ^b	Group 4	Group 5 ^c	
a) Permanent actions ^d			n/a		n/a	n/a	
Direct actions							
self-weight: concrete	1.00						1.00
self-weight: steel	1.00						1.00
superload: surfacing	1.00						1.00
superload: other	1.00						1.00
ground conditions	1.00						1.00
Indirect actions							
prestress/creep	1.00						1.00
settlement	1.00 (0)						1.00 (0)
b) Variable actions							
Carriageway	TS	UDL					
Vertical forces							
main loading system: LM1							
lane 1 ^e	0.87	0.26					0
lane 2	1.31	0.95					0
lane 3	1.31	0.95					0
other lanes	0	0.95					0
remaining area	0	0.95					0
special vehicles: LM3	0						0
single axle: LM2 ^f	0						0
crowd loading: LM4	0						0
Horizontal forces							
braking and acceleration	0						0
centrifugal forces	0		0				
Footway and cycletracks ^{gh}	UDL ^g	S veh.	UDL ^g	S veh.			
	0	0	0.44	0			
Wind ^{ij}							
F_{Wk} or F_{Wn}	0		0				
or F_W^*	0		0				
Temperature ^{ik}							
T_k	0.55		0.55				
c) Accidental actions							
Under bridge	n/a		n/a				
On bridge	n/a		n/a				
NOTE Sveh. = service vehicle.							

4.6 Fatigue load models

Traffic category and lane requirements shall be agreed with the relevant authority in accordance with Table 4.5 of ENV 1991-3:1995. Some further guidance is given in annex Y. Fatigue models 1 to 4 shall be applied in accordance with 4.6.1 to 4.6.6, unless specified otherwise by the relevant authority.

4.6.1 General

For fatigue verification the required design working life of highway bridges shall be 120 years.

4.6.2 Fatigue load model 1

Fatigue load model 1 may be used to check whether the maximum stress range exceeds the unlimited life criteria. Adjustment factors shall conform to annex X.

4.6.3 Fatigue load model 2

Fatigue load model 2 is used to check indefinite life in the same way as model 1. The axle loads given in Table 4.6 of ENV 1991-3:1995 may all be factored by 0.80.

4.6.4 Fatigue load model 3

Fatigue load model 3 is used for damage summation, in the event that model 1 or model 2 fail to demonstrate indefinite life. The axle loads defined for model 3 may all be factored by 0.70. For influence line lengths L in excess of 50 m, the number of load cycles shall be multiplied by a factor K_F , where:

$$K_F = (L/50)^{1/2}$$

4.6.5 Fatigue load model 4

Fatigue load model 4 is used if model 3 fails to provide satisfactory assurance of life in the fatigue life damage summation calculation. The equivalent axle loads given in Table 4.7 of ENV 1991-3:1995 may all be factored by 0.60.

4.6.6 Fatigue load model 5

Fatigue load model 5 and associated use of annex B of ENV 1991-3:1995 shall only be considered if such recorded data is available and current and only by agreement with the relevant authority.

4.7 Accidental actions

4.7.1 General

All the accidental actions shall be considered unless specifically excluded by the relevant authority.

Exclusion by protection due to effective safety barriers shall only be if the barrier is shown to be effective in accordance with the DMRB.

Any element or structure supporting the barrier shall be designed to resist an equivalent loading derived in accordance with the DMRB.

4.7.2 Collision forces from vehicles under the bridge (on bridge supports and superstructures)

Collision loads on highway bridge supports and superstructures shall conform to the DMRB requirements for vehicle collision loads on highway bridge supports and superstructures, unless specified otherwise by the relevant authority.

The collision loading for bridges that cross railway tracks, canals or navigable water shall be as specified by the relevant authority as agreed with the appropriate authority responsible for the underlying route. Bridges over railways shall conform to 6.7.1.3.

Any special requirements relating to exceptional loads (e.g. due to vandalism or other deliberate or accidental actions) shall be as agreed with the relevant authority.

4.7.3 Actions from vehicles on the bridge

If a rigid effective barrier is provided (see 4.7.1), or a kerb conforming to specific requirements provided by the relevant authority, consideration of the accidental axle load may not be required beyond (outside) the barrier. This similarly applies in relation to the central reservation area between inner barriers or kerbs in dual carriageway arrangements.

The accidental axle load shall be considered in all areas of the bridge, including footways, cycle tracks and the carriageway, where these areas are not protected by an effective barrier or kerb. Where the barrier is effective but deformable, application of the accidental axle load is only required within the bounds of the maximum deformation of the barrier, or within 1 m of the edge of the carriageway, whichever is greater.

The full specified collision loads from vehicles on kerbs or barriers shall be considered on carriageway kerbs or barriers only where they are effective. Where the barriers or kerbs are not effective, reduced collision loads may be considered, taking account of the height of the kerb or barrier. The loads so derived shall not be taken as less than the loads produced by application of the DMRB requirements relating to local and global effects, unless agreed otherwise with the relevant authority.

Collision forces on structural members shall be taken as the most onerous condition from above, taking account of the effectiveness or possible deformation of any kerb or barrier where so provided. In addition, the criteria of 4.7.2 shall be applied, unless additional effective protective measures are provided. The design of additional protection or requirements for effectiveness shall conform to the DMRB unless agreed otherwise with the relevant authority.

4.8 Actions on parapets

Actions on parapets shall not be taken as less than actions derived in accordance with the DMRB, unless agreed otherwise with the relevant authority.

4.9 Actions on embankments

The dispersal of load shall be taken as conforming to ENV 1997-1 and its associated NAD. The dispersal of load shall be used for considerations relating to embankments as well as for all parts of the bridge in contact with earth or fill.

5 Pedestrian, cycle and other actions specifically for footbridges

The service vehicle to be designed for shall be as specified by the relevant authority, if different to the vehicle shown in Figure 5.3 of ENV 1991-3:1995.

Partial factors on actions γ shall conform to Tables 4.5.1 and 4.5.2. The definition of groups of loads and the combination and reduction factors ψ shall conform to Tables D.1 and D.2 of ENV 1991-3:1995.

Accidental actions and actions on kerbs and barriers shall conform to the relevant actions from the requirements in 4.7 and 4.8, respectively.

Vibration serviceability shall be checked in accordance with the DMRB with respect to deliberately induced vibration or resonance with the movement of users. Foot and cycle track bridges shall be checked for aerodynamic effects in accordance with the DMRB.

6 Railway traffic actions and other actions specifically for railway bridges

6.1 Field of application

- 4) A light traffic mix is included in this NAD (see annex F).

6.3.1 Eccentricity of vertical loads (only load model 71)

- 3)P To clarify this subclause the text should be replaced by the following.

“The effect of lateral displacement of vertical loads shall be considered by taking the ratio of wheel loads on all axles as 1.25:1.00. The resulting eccentricity is shown in Figure 6.1. No eccentricity shall be considered in calculating fatigue effects.”

6.3.2 Load model

In applying load model 71 and characteristic values for vertical loads, the value of a shall be taken as 1.0 to obtain the classified vertical loads, except when considering the local effects from an individual axle load on a structural element. In these cases the value of a shall be taken as 1.1.

In exceptional cases, if safety and emergency working are not adversely affected, a lighter loading may be adopted. The provision also exists for adopting a heavier loading on restricted sections where appropriate.

The relevant authority shall determine the cases where a loading other than specified above shall be used.

In these cases the value of a shall be within the limits given in 6.3.2(3)P of ENV 1991-3:1995.

When a loading other than load model 71 is adopted, an appropriate traffic mix for fatigue shall be established taking account of the design life of the structure and any expected changes in the movement of traffic over the structure. Normal and abnormal operating conditions shall be considered.

The value of a used in conjunction with load model 71 shall be applied to the following load effects:

- nosing;
- centrifugal action;
- longitudinal loads due to traction or braking;
- accidental actions.

6.4.3.2 Dynamic factor Φ

- 2)P The dynamic factor shall be taken as Φ_3 unless otherwise specified by the relevant authority, except where a fatigue damage assessment is carried out. In this case, Φ_2 shall be used, as specified in F.2(2)P of ENV 1991-3:1995.

If Φ_3 is taken, Φ_2 may be used for evaluating shear effects if specified by the relevant authority.

6.4.3.3 Determinant lengths L_ϕ

The additional and amended values for the determinant length L_ϕ given in Table 6.2(a) shall be applied unless otherwise specified by the relevant authority.

In case 4.3 of Table 6.2 of ENV 1991-3:1995, the term “trough bridge” may be taken to mean “half through bridge”.

6.4.4 Dynamic effects when there is a risk of resonance or excessive vibration of the structure

No further consideration of excessive vibration or resonance of the structure is required where $V \leq 220$ km/h and the natural frequency is within the limits shown in Figure 6.9 of ENV 1991-3:1995.

In other cases, the effects shall be considered separately in accordance with the principles of annex H of ENV 1991-3:1995. A simplified method is given in the guidance on annex H in this NAD.

Table 6.2(a) — Additional and amended values of determinant length L_ϕ

Type of bridge	Structural element	Determinant length L_ϕ
Steel bridges	All end cross girders	4 m ^a
Steel bridges with longitudinal and cross ribs	Deck plate, longitudinal ribs	3 × cross girder spacing or 2 × cross girder spacing plus 3 m, whichever is greater ^a
Steel bridges with deck plate and cross girders only	Cross girders	Same as 2.1 ^a of ENV 1991-3:1995, Table 6.2
Concrete bridges	End zones of transverse spanning deck elements. [The end zone should extend a distance in the longitudinal direction of at least 1/4 of the span in the transverse direction (e.g. the distance between the webs in a box girder construction).]	4 m
Steel/concrete bridges with deck above steel beams	Concrete deck element except end zones of transverse spanning deck elements. End zones (as defined for concrete bridges)	Same as 4.1 of ENV 1991-3:1995, Table 6.2 4 m ^a
Steel concrete composite decks (composed of steel cross girders and concrete spanning longitudinally between cross girders)	Concrete element spanning longitudinally between cross girders. Cross girders	Same as 4.3 of ENV 1991-3:1995, Table 6.2 Same as 1.3 and 2.2 of ENV 1991-3:1995, Table 6.2, and above for end girders
Deckplate and other elements of closed (box) frames for one or more tracks (all cases)	All elements	Treat as a 4 span continuous bridge ^a
Bridges composed of $\cup$ and $\cap$ shaped units placed to form a closed (box) frame	Deck element	Same as 5.3 of ENV 1991-3:1995, Table 6.2 (i.e. treat as a portal)
Single arch bridges	Arch element	Same as 5.5 of ENV 1991-3:1995, Table 6.2
Series of arch bridges	Arch element	Same as 5.6 of ENV 1991-3:1995, Table 6.2
All	Suspension bars (without stiffening girders)	Same as for cross girders suspended from bars
Structure with more than one track	All	Generally shall be considered without a reduction in the dynamic increment ^a

^aAmended values.**6.5.1 Centrifugal forces**

3)P The maximum speed used in the calculation of the characteristic value of the centrifugal force shall be determined from the track layout and curvature on or adjacent to the bridge, assuming the maximum permitted cant and cant deficiency as specified by the relevant authority.

In exceptional cases, where safety and emergency working are not adversely affected, a lesser maximum speed may be adopted, as determined

by the relevant authority.

7)P The vertical load to be considered without centrifugal force shall be taken as the classified vertical load.

8)P Unless otherwise specified by the relevant authority, the following case shall also be considered:

$a \times$ load model 71 in conjunction with a line speed of 120 km/h, where $a \geq 1.0$.

6.5.4 Application of longitudinal actions

6.5.4.1 General and principles

3)P Unless otherwise specified by the relevant authority, the following actions shall be considered in the calculation of the longitudinal actions:

- earth pressure effects;
- effects due to inclined decks or bearing surfaces;
- anchorage forces from stressing or destressing or accidental breakage of continuous welded rails on or close to the bridge deck.

NOTE Simultaneous stressing or destressing or breakage of both rails of any one track shall be considered, but for bridge decks carrying two or more tracks the effects due to one track only need to be considered. Breakage of rails shall be considered as an accidental action (see 6.7.4).

6)P Unless otherwise specified by the relevant authority, connections shown between the rail and the structure/formation in Figure 6.11 of ENV 1991-3:1995 shall be replaced with “spring/friction” connections (i.e. connections that resist relative movement elastically up to a certain value and then yield at a constant resistance).

For ballasted track, the resistance per track per metre length shall be as follows unless otherwise specified by the relevant authority:

- for unloaded track, elastic resistance up to yield at a relative movement of 2.8 mm, followed by movement at a constant yield resistance of 20 kN;
- for loaded track, elastic resistance up to yield at a relative movement of 2.8 mm, followed by movement at a constant yield resistance of 40 kN.

See Figure 6.5.4.1(a).

For track directly fastened to concrete decks with standard fastenings, the resistance per track per metre length shall be taken as elastic resistance up to yield at a relative movement of 0.5 mm followed by movement at a constant yield resistance of 40 kN and 60 kN for unloaded and loaded track, respectively, unless otherwise specified by the relevant authority [see Figure 6.5.4.1(b)].

For other track conditions, the track resistance shall be specified by the relevant authority.

The maximum relative horizontal displacement between the rails and the deck due to traction/braking actions shall not exceed 4 mm, unless otherwise specified by the relevant authority.

6.5.4.2 Assessment of longitudinal actions

1)P Unless otherwise specified by the relevant authority, the assessment of actions transmitted to the structure for the following types of structures may also be based on the coefficients and equations given above, where the conditions specified in 6.5.4.2(2)P of ENV 1991-3:1995 apply:

- all spans up to 15 m for any system of articulation (for skew bridges, the span plus lead shall be considered);
- all monolithic structures (portal bridges or box type bridges), regardless of number of openings;
- the class of structure defined in 6.5.4.1(5)(c) of ENV 1991-3:1995, provided that the neutral axis of bending of the spans is not above the level of the rails.

For bridges carrying ballasted track other than where the above criteria apply, a particular calculation of the longitudinal forces as described in 6.5.4.1(6)P shall be carried out.

For bridges carrying directly fastened track, the assessment of actions transmitted to the structure shall conform to 6.5.4.1(3)P of ENV 1991-3:1995, together with the relevant clauses of this NAD.

2)P To clarify 6.5.4.2(2)P(a) of ENV 1991-3:1995, the following limits on the expansion length of the structure may be applied, unless otherwise specified by the relevant authority.

- a) If the track is continuous (i.e. without an expansion device), the expansion length of the structure shall be limited as follows:
 - 60 m for steel structures classified in 6.5.4.1(5)(a) and (b) of ENV 1991-3:1995, carrying ballasted track;
 - 90 m for concrete or composite structures classified in 6.5.4.1(5)(a) and (b) of ENV 1991-3:1995, carrying ballasted track;
 - 45 m for steel structures classified in 6.5.4.1(5)(c) of ENV 1991-3:1995, carrying ballasted track;
 - 60 m for concrete or composite structures classified in 6.5.4.1(5)(c) of ENV 1991-3:1995, carrying ballasted track.

For structures classified in 6.5.4.1(5)(c) of ENV 1991-3:1995, the expansion length may be taken as the individual span length, or the sum of the two adjacent span lengths, where an intermediate support carries the free ends of both spans.

- b) Unless otherwise specified by the relevant authority, the minimum value of track resistance to be taken into account shall be as given in 6.5.4.1(6)P.

Add after item c): “These limits may be deemed satisfied in UK conditions unless otherwise specified by the relevant authority”

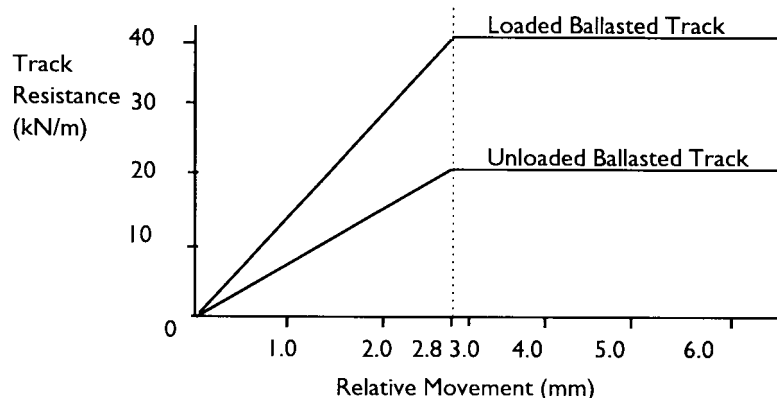


Figure 6.5.4.1(a) — Graph of relationship between track resistance and relative movement for ballasted track

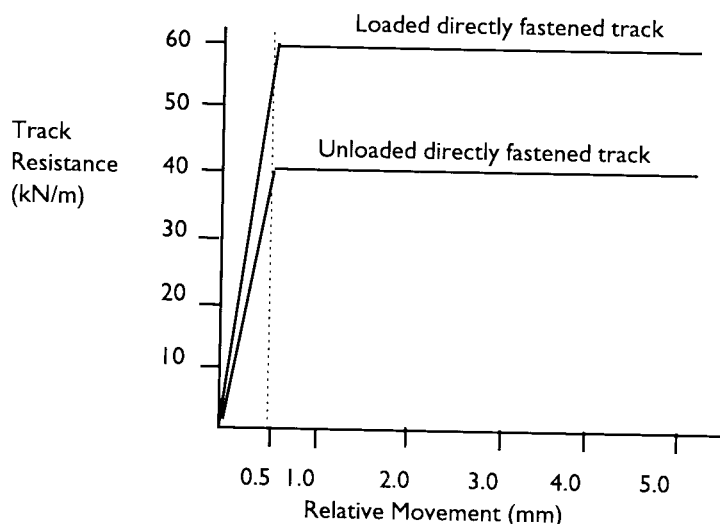


Figure 6.5.4.1(b) — Graph of relationship between track resistance and relative movement for directly fastened track

6.5.4.3 Longitudinal actions due to temperature variation

Add a new item 4)P as follows:

4)P Unless otherwise specified by the relevant authority, for single-span bridges not more than 15 m long, carrying standard directly fastened track that is continuous over both ends of the deck, the characteristic value of the longitudinal action to be taken into account at the bearings is given by:

$$F_{Tk} = \pm 40L_T \text{ (in kN) per track.}$$

For bridges carrying directly fastened track other than those covered by 6.5.4.3(4)P, or by 6.5.4.3(3) of ENV 1991-3:1995, a particular calculation of the longitudinal forces as described in 6.5.4.1(6)P shall be carried out.

To clarify equations 6.10, 6.11, 6.12, and 6.13 of ENV 1991-3:1995:

F_{Tk} = The characteristic value of the longitudinal action due to temperature variation that shall be taken into account at the bearing level. This action results from the expansion movement of the bridge relative to the continuous ballasted track.

6.5.4.4 Longitudinal actions due to traction and braking

Unless otherwise specified by the relevant authority, the values of the coefficients for traction/braking actions for continuous track given in Table 6.4 of ENV 1991-3:1995 are only valid when the expansion length of the structure given in 6.5.4.2(2)P(a) is not exceeded.

In clarification of Table 6.4 of ENV 1991-3:1995, in the case of a structure carrying continuous track, where the overall length of structure is less than 30 m, a coefficient for traction/braking actions of 0.5 may be used, unless otherwise specified by the relevant authority.

Linear interpolation may be used to obtain the coefficients for traction/braking where the overall length of the structure lies between the values given in Table 6.4 of ENV 1991-3:1995.

If only one expansion device is provided, the anchorage effects due to longitudinal forces in the rail shall be considered. Allowance shall be made for these forces to be resisted by the structure.

Add a new item 3) as follows:

- 3) Unless otherwise specified by the relevant authority, for all bridges carrying directly fastened track:

$$F_{bk} = Q_{lak}, Q_{lbk}$$

6.5.4.5 Longitudinal actions due to deflection of the structure

Unless otherwise specified by the relevant authority, the effect on longitudinal actions due to the deflection of the structure shall be considered only in cases where the depth between rail level and centre of rotation of the bridge bearing is greater than 1.5 m.

6.6 Slipstream effects from passing trains (aerodynamic effects)

This subclause relates to passing static pressure changes as a train passes a structure.

The values of q_{2k} given in Figure 6.14 of ENV 1991-3:1995 are based on continental rolling stock and structure gauge. The values generally over-predict the maximum pressure changes for rolling stock in common use in the UK, which is compatible with standard UK structure gauge. As an example, Figure 6.4(a) gives values of q_{2k} based on computer modelling (using USAERO simulations) for a class 90 power car.

The values of q_{2k} shall be agreed with the relevant authority.

Slipstream effects that relate to air velocities, relative to the ground, generated by the train pulling air along with it as it moves through the atmosphere (boundary layer) are not covered. These effects, that have implications for the safety of persons and/or equipment situated near passing trains, are outside the scope of this ENV but should be considered.

The characteristic values of action q_{2K} for simplified horizontal surface may also be used for station canopies without curved soffits that have one side closed by a wall or station building, where the distance from the centre of the track to the face of the closed wall or station building is more than or equal to 4 m.

For station canopies with curved soffits a particular assessment of the likely aerodynamics loads shall be considered.

6.6.3 Simple horizontal surfaces above the track

- 5) This item is more clearly expressed as follows.
“The action acting on edge strips that cross the track may be multiplied by a factor of 0.75 over a width up to 1.50 m.”

6.7 Accidental actions

6.7.1.2 Derailment on bridges: structural requirements and equivalent loads

- 1)P Unless otherwise specified by the relevant authority, the following design situation shall be considered in addition to the two design situations identified.

All deck plates and other similar local elements shall be designed to support a nominal point load of 250 kN applied at the top of the deck surface. This load shall be deemed to include all allowances for dynamic effects.

- 2)P To clarify this clause, the two vertical line loads shall be considered 1.4 m apart in a direction transverse to the track (“s” in ENV 1991-3:1995, Figure 6.17).

Unless otherwise specified by the relevant authority, the appropriate parts of the structure shall be additionally designed for the following equivalent loads at the ultimate state:

- eight individual concentrated vertical loads, each with a design value of 180 kN, arranged on two lines 1.4 m apart, with each of the four loads 1.6 m apart on line, applied anywhere on the deck.

- 5)P For structural elements above rail level, the protective measures provided shall be based on the following considerations, unless otherwise specified by the relevant authority.

- 1) For deck type bridges, a nominal concentrated horizontal force of 100 kN applied perpendicular to and at the top of the deck kerb anywhere along the length of the bridge shall be considered.

- 2) For half through bridges.

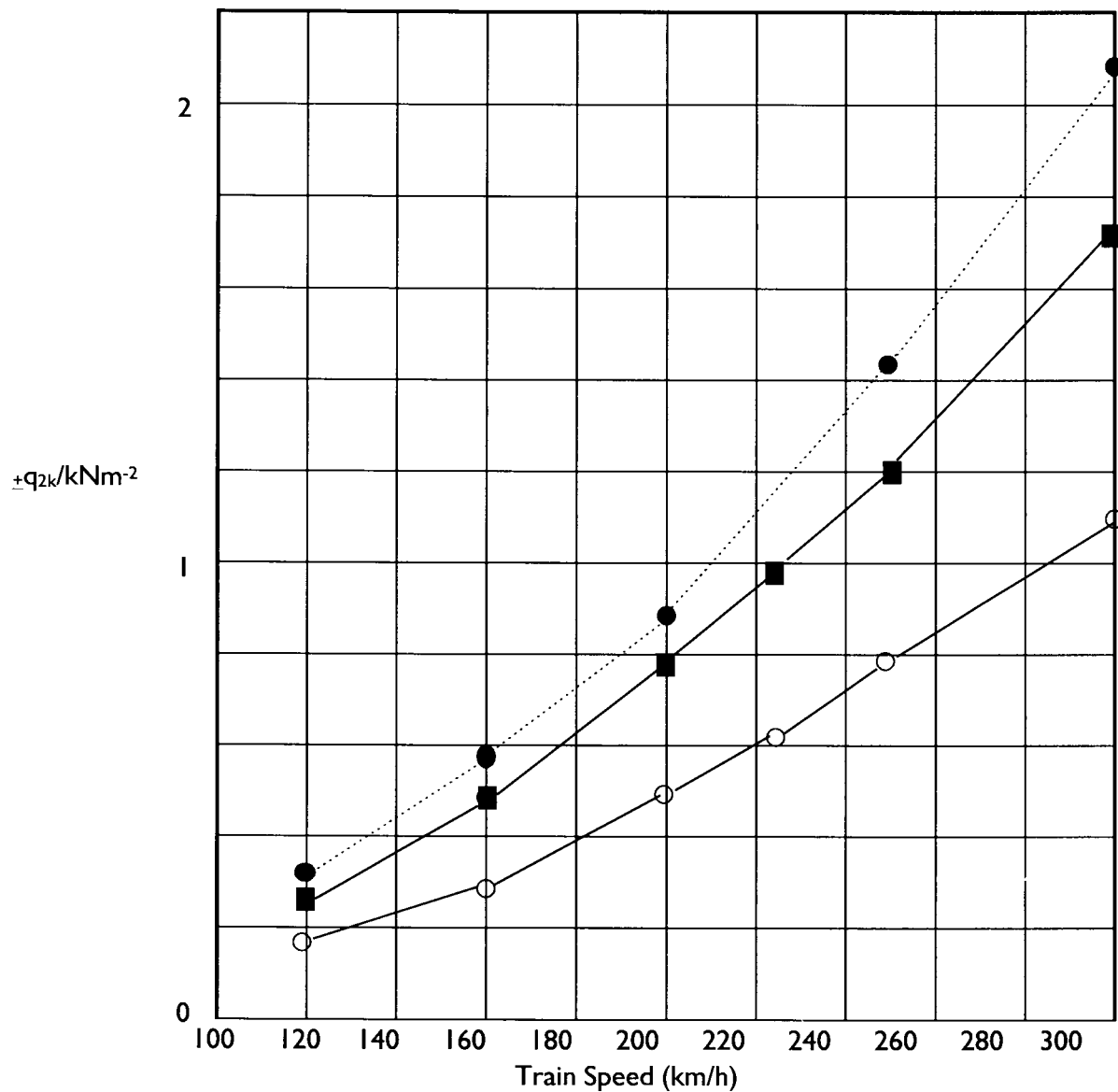
- If the horizontal resistance of the main structural members provides comparable protection to the protection for deck type bridges, consideration of further nominal horizontal concentrated load is not generally required.

- If the main structural members do not provide comparable protection to the protection for deck type bridges, provision of alternative comparable methods shall be considered as for through bridges.

- 3) For through bridges, provision of adequate protection shall be considered on an individual basis, taking into account:

- the speed, type and frequency of rail traffic;
- the geometry of the track;
- the probable consequences of any derailment.

- ENV 1991-3:1995, 6.6.3, with $h_g = 5.0$ m ($k_1 = 0.85$)
- USAERO Calculations for class 90 with $h_g = 4.5$ m
- USAERO Calculations for class 90 with $h_g = 5.0$ m



NOTE $k_1 = 0.85$, $h_g = 5.0$ m.

Figure 6.4(a) — Characteristic values of actions $\pm q_{2k}$ for class 90 power car passing under simple horizontal surfaces 4.5 m and 5.0 m above the track, compared with ENV 1991-3:1995, 6.6.3

6.7.1.3 Derailment under structures

1)P Unless otherwise specified by the relevant authority, for supporting structures for new bridges or similar structures constructed over or alongside railway tracks, the following shall be considered.

— Wherever reasonably practicable, supports carrying any structure over or alongside railway tracks shall be placed outside the hazard zone. In general, the hazard zone shall be assumed to extend for a width of 4.5 m from the edge of the outside rails of the nearest operational track.

— Consideration shall be given to extending the hazard zone in the following situations:

- if the support structure is situated within an embankment that carries rail traffic;
- if the support structure is adjacent to the outside of a track curve.

— If supports are placed inside the hazard zone, they should be monolithic piers and not individual columns. All supports located between railway tracks shall be considered to be inside the hazard zone.

— If individual columns are used within the hazard zone, the design of the structure above these shall incorporate a degree of continuity so that removal of any column does not cause the remainder of the structure to collapse under permanent loads together with the appropriate variable traffic loads.

— To provide robustness against the effect of light impacts, all piers and columns and their restraints, within the hazard zone, shall be designed to withstand without collapse a single horizontal design force of 2 000 kN acting at a design height of 1.2 m above the adjacent ground level, and a single horizontal design force of 500 kN acting at a height of 3 m. The two loads may act in any direction but it is not required to consider that they act simultaneously.

— These requirements shall be applied wherever reasonably practicable to new footbridges, taking into account the nature of the rail traffic and the track layout adjacent to the bridge.

— These requirements do not apply to lineside railway infrastructure (e.g. overhead line masts, signal gantries).

— Requirements for particular structures shall be confirmed with the relevant authority.

NOTE The requirements of this subclause are under review. Reference should be made to ENV 1991-2-7.

Add a new subclause **6.7.4** as follows.

6.7.4 Accidental actions due to rail breakage

1) Actions due to accidental breakage of continuous welded rails on or close to the bridge deck should be considered based on the requirements of **6.5.4.1**, and in agreement with the relevant authority. Collapse of a major part of the structure shall not occur as a result of such actions, but local damage is acceptable.

6.8 Assessment of traffic loads on railway bridges

6.8.1 General

1)P Unless otherwise specified by the relevant authority, the minimum spacing between centre lines of the tracks may be taken as the existing spacing or 3 400 mm, whichever is lesser.

NOTE The spacing provided shall be calculated from the dimensions associated with the chosen vehicle kinematic envelope with an allowance made for passing clearances, taking into consideration the effects of track curvature where appropriate. When calculating the required dimension, the expected rail traffic shall be taken into consideration so that the aerodynamic effects can be taken into account.

For bridges that carry more than one track, the checks for the limits of deflection and vibration, particularly with regard to safety, may not be sufficient if only one track is loaded.

6)P Unless otherwise specified by the relevant authority, the checks for the limits of deflection and vibration shall be made with the tracks loaded in accordance with the clauses of this NAD that relate to annex G.3.

6.8.2 Groups of loads: characteristic values of the multi-component action

In Table 6.6 of ENV 1991-3:1995, and Table 6.6(a), the groups of loads identified in Table 6.6(a) by “gr11”, “gr12”, etc., shall be applied as a single variable action. The ψ factors given in Tables G1/2(a) and (b) shall be applied to the group of loads as a whole.

Unless otherwise specified by the relevant authority, Table 6.6 of ENV 1991-3:1995 shall be replaced by Table 6.6(a). The note at the head of Table 6.6 of ENV 1991-3:1995 shall apply to Table 6.6(a). Values that differ from the boxed values given in Table 6.6 of ENV 1991-3:1995 are shown in bold in Table 6.6(a).

6.9 Fatigue load models

2)P An additional light traffic mix that may be used, subject to agreement with the relevant authority, is given in Table F.3 of this NAD.

**Table 6.6(a) — Assessment of groups of traffic loads
(characteristic values of the multi-component actions)**

Group of actions (4)		Type of action					
		Load model 71 (3) + SW/0	Unloaded train	Traction braking	Centrifugal force	Nosing	Comment
Bridge with single track	gr11 Track 1	1.0	—	0.75 (1)	1.0 (1)	1.0 (1)	11 Maximum vertical and lateral effects
	gr12 Track 1	—	1.0 (6)	—	1.0 (1) (5)	0.5 (1)	12 Lateral stability
	gr13 Track 1	1.0 to 0.7 (2)	—	1.0	1.0 (1)	1.0 (1)	13 Maximum longitudinal effects
	gr14 Track 1	1.0	—	1.0 (1)	1.0	1.0 (1)	14 Maximum lateral effects
	gr15 Track 1	1.0	—	1.0 (1)	1.0 (1)	1.0	15 Local lateral effects
Bridge with two tracks	gr11 Track 1	1.0	—	0.75 (1)	1.0 (1)	1.0 (1)	11 Maximum vertical and lateral effects
	gr11 Track 2	1.0	—	0.75 (1)	1.0 (1)	1.0 (1)	
	gr12 Track 1	—	1.0 (6)	—	1.0 (1) (5)	0.5 (1)	12 Lateral stability
	gr12 Track 2	—	—	—	—	—	
	gr13 Track 1	1.0 to 0.7 (2)	—	1.0	0.75 (1)	0.5 (1)	13 Maximum longitudinal effects
	gr13 Track 2	1.0 to 0.7 (2)	—	1.0	0.75 (1)	0.5 (1)	
	gr14 Track 1	1.0	—	0.75 (1)	1.0	0.5 (1)	14 Maximum lateral effects
	gr14 Track 2	1.0	—	0.75 (1)	1.0	0.5 (1)	
	gr15 Track 1	0.5 (1)	—	0.75 (1)	0.75 (1)	1.0	15 Local lateral effects
	gr15 Track 2	0.5 (1)	—	0.75 (1)	0.75 (1)	1.0	

Table 6.6(a) — Assessment of groups of traffic loads
(characteristic values of the multi-component actions) (continued)

Group of actions (4)		Type of action								Comment
		Load model 71 (3) + SW/0	Unloaded train	Traction braking		Centrifugal force		Nosing		
Bridge with three or more tracks	gr11 Any one track (A)	1.0 0.75	— —	0.75 (1)	0.75 (1)	1.0 (1)	0.75 (1)	1.0 (1)	0.75 (1)	11 Maximum vertical and lateral effects
	Any other track (B)	1.0 0.75	— —	0.75 (1)	0.75 (1)	1.0 (1)	0.75 (1)	1.0 (1)	0.75 (1)	
	All other tracks (C)	— 0.75	— —	— —	— 0.75 (1)	— 0.75 (1)				
	gr12 (A)	—	1.0 (Φ = 1.0)	—	—	1.0 (5)	—	0.5 (1)	12 Lateral stability	
	gr12 (B)	—	—	—	—	—	—			
	gr12 (C)	—	—	—	—	—	—			
	gr13 (A)	1.0 to 0.7 (2)	—	1.0	0.75 (1)	0.5 (1)	13 Maximum longitudinal effects			
	gr13 (B)	1.0 to 0.7 (2)	—	1.0	0.75 (1)	0.5 (1)				
	gr13 (C)	—	—	—	—	—				
	gr14 (A)	1.0 0.75	—	0.75 (1)	0.75 (1)	1.0 0.75	0.5 (1) 0.375 (1)	14 Maximum lateral effects		
	gr14 (B)	1.0 0.75	—	0.75 (1)	0.75 (1)	1.0 0.75	0.5 (1) 0.375 (1)			
	gr14 (C)	— 0.75	—	— —	— 0.75	— 0.375 (1)				
	gr15 (A)	0.5 (1) 0.375 (1)	—	0.75 (1)	0.375 (1)	0.75 (1) 0.375 (1)	1.0 0.75	15 Local lateral effects		
	gr15 (B)	0.5 (1) 0.375 (1)	—	0.75 (1)	0.375 (1)	0.75 (1) 0.375 (1)	1.0 0.75			
	gr15 (C)	— 0.375 (1)	—	— —	0.375 (1)	— 0.75				

Values in bold are different to “boxed” values in ENV 1991:1995, Table 6.6.

☐ Dominant component action

(1) These non-dominant values may be zero where the effect is favourable.

(2) For a favourable effect this non-dominant value cannot be zero. A minimum value of 0.7 is proposed in combination with dominant traction and braking action.

(3) Including all relevant factors ($\Phi, f, \dots$).

(4) The simultaneity of two or three characteristic values, although unlikely, has been proposed as a simplification for groups 11, 12, 13 and 14, since no significant consequences in the design arise

(5) Based on unloaded train.

(6) The static effects from the “Unloaded Train” shall not be enhanced by the dynamic factor Φ but shall take into account the factor “ f ” (ENV 1991-3:1995, equation 6.6).

Annex A

Annex A of ENV 1991-3:1995 (applicable to highway bridges) is generally applicable but shall be used as indicated in clauses 4 and 5, as appropriate to highway bridges.

Annex B

Annex B of ENV 1991-3:1995 (applicable to highway bridges) is generally applicable but shall be used as indicated in clauses 4 and 5, as appropriate to highway bridges.

Annex C

Annex C of ENV 1991-3:1995 (applicable to highway bridges) is generally applicable but shall be used as indicated in clauses 4 and 5, as appropriate to highway bridges.

Annex D

Annex D of ENV 1991-3:1995 (applicable to highway bridges) is generally applicable but shall be used as indicated in clauses 4 and 5, as appropriate to highway bridges.

Annex E

Annex E of ENV 1991-3:1995 (applicable to railway bridges) is generally applicable but shall be used as indicated in clause 6, as appropriate to railway bridges.

The method for calculating dynamic factors ($1 + \phi$) for actual trains may be used, unless otherwise specified by the relevant authority.

Annex F (normative)

Basis for the fatigue assessment of railway structures

Annex F of ENV 1991-3:1995 (applicable to railway bridges) is generally applicable, unless specified by the relevant authority, but shall be used as specified in clause 6 as appropriate to railway bridges.

Table F.3 gives a light traffic mix that may be used, subject to agreement with the relevant authority.

Table F.3 — Light traffic mix

Train type	Number of trains per day	Weight per train t	Traffic volume 10^6 t/year
5	2	2 160	1.4
9	190	296	20.5
1	10	663	2.4
2	5	530	1.0
Total	207	—	25.3

F.2 General design method

If damage equivalence factors λ are used for the fatigue assessment for traffic volumes that differ from those given in ENV 1991-3:1995, Tables F.1 and F.2, and in Table F.3 of this NAD, and/or a design life other than 100 years is used, appropriate values for λ_2 and λ_3 shall be verified. The verification may be based on the load spectra given in Figure F.3.1 and Tables F.3.1 to F.3.9, or as otherwise specified by the relevant authority.

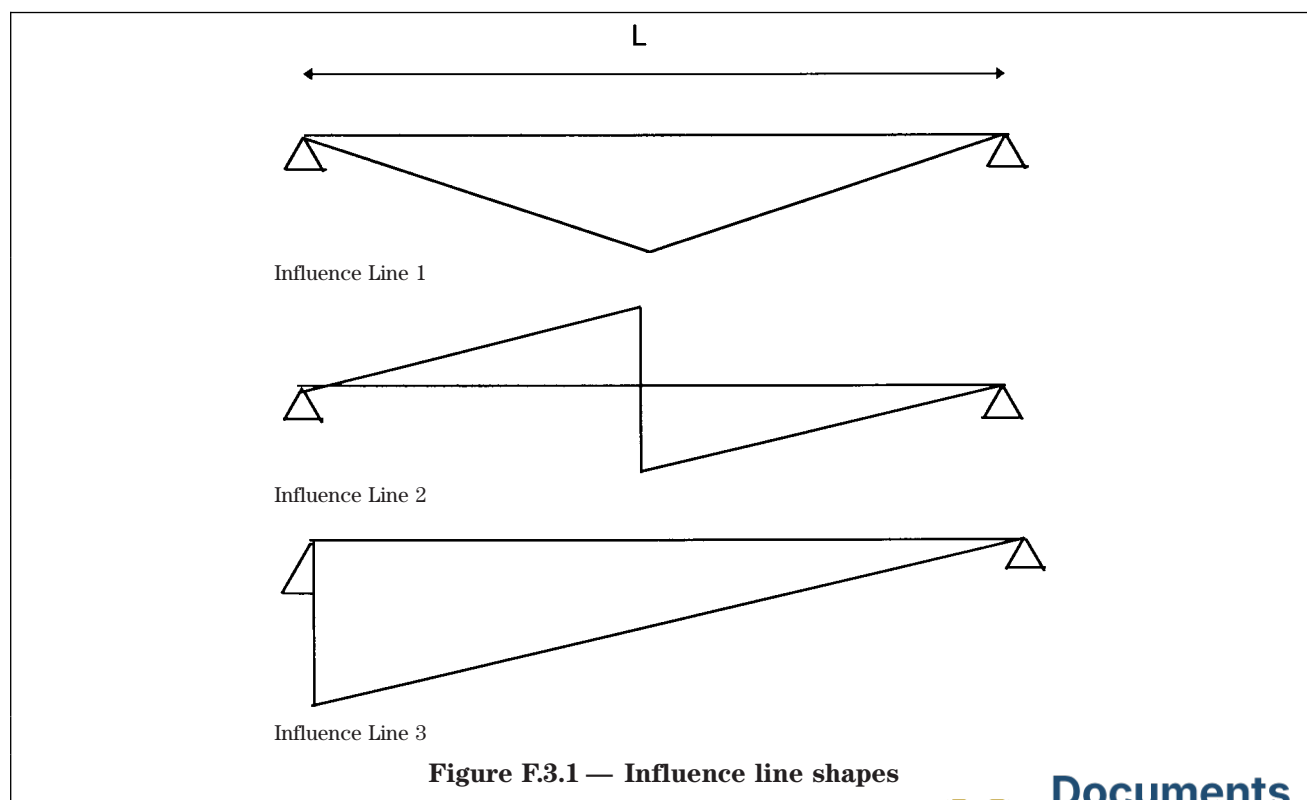


Table F.3.1 — Load spectra for heavy traffic mix, influence line 1, giving number of cycles per year (in thousands) expressed as proportion of load model $71 \times \phi_2, P_{L71}$, for various influence line lengths

Influence line length L m	Influence line 1: thousands of cycles per year for load proportion of load model $71 \times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	0	84.2	0	0	251.3	296.6	460.8	0	0
3	0	335.5	188.6	0	0	69.1	499.7	0	0	0
4	73.4	84.2	51.8	29.2	40.3	447.5	0	0	0	0
5	70.2	65.9	29.2	126.7	152.3	174.2	34.6	0	0	0
7	171.4	84.6	121.0	39.2	28.1	116.3	64.4	0	0	0
10	171.0	32.4	6.8	60.5	54.7	41.8	51.8	0	0	0
15	308.9	69.8	24.5	14.0	19.8	70.6	7.9	0	0	0
20	330.3	60.8	2.3	65.9	14.0	5.8	12.6	0	0	0
30	333.0	4.7	4.7	14.0	0	0	16.2	2.2	0	0
50	208.1	4.7	14.0	0	0	0	16.2	0	2.2	0
75	332.6	4.7	0	0	0	4.7	11.5	0	0	2.2
100	248.4	0	0	0	0	4.7	11.5	0	0	2.2

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F.3.2 — Load spectra for heavy traffic mix, influence line 2, giving number of cycles per year (in thousands) expressed as proportion of load model $71 \times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 2: thousands of cycles per year for load proportion of load model $71 \times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	0	84.2	18.4	0	0	538.7	460.8	0	0
3	0	0	85.3	17.3	0	0	538.7	460.8	0	0
4	0	84.2	15.5	2.9	6.8	360.9	631.8	0	0	0
5	0	84.2	9.2	136.6	289.4	266.4	203.9	112.3	0	0
7	0	88.9	125.6	328	2.9	227.5	265.9	63.4	0	0
10	24.5	145.4	347.9	134.1	132.1	107.6	178.0	32.4	0	0
15	134.3	378.0	219.8	122.6	73.6	68.6	56.3	49.0	0	0
20	163.4	430.6	139.1	156.1	56.5	52.2	41.9	51.8	0	0
30	342.5	386.1	125.5	111.2	57.1	16.9	62.8	0	0	0
50	538.2	232.6	194.8	56.9	65.9	7.6	5.2	1.1	0	0
75	639.9	306.7	61.4	73.4	11.5	0	8.1	0	1.1	0
100	763.9	230.8	78.1	18.0	2.2	2.3	5.8	0	1.1	0

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F.3.3 — Load spectra for heavy traffic mix, influence line 3, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 3: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	84.2	0	11.6	179.6	220.9	527.4	0	0	0
3	0	88.9	318.8	17.1	329.8	33.3	295.9	0	0	0
4	70.2	268.0	88.2	103.5	243.4	220.5	90.0	0	0	0
5	89.6	298.3	236.5	86.6	133.9	142.0	96.8	0	0	0
7	239.6	209.5	265.7	124.4	78.8	110.0	55.8	0	0	0
10	240.8	453.6	170.1	36.0	76.1	15.1	29.7	51.8	0	0
15	445.9	286.2	187.2	35.8	33.3	89.1	5.2	1.1	0	0
20	468.5	397.1	110.7	59.9	18.0	20.3	8.1	1.1	0	0
30	624.6	310.9	54.7	61.2	14.0	8.1	5.2	4.0	1.1	0
50	766.1	225.2	69.5	4.7	0	5.2	8.1	2.9	1.1	1.1
75	943.7	112.3	9.4	0	0	5.2	8.1	2.9	1.1	1.1
100	981.2	72.7	4.7	0	0	5.2	8.1	2.9	0	2.2

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F.3.4 — Load spectra for standard traffic mix, influence line 1, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 1: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	0	77.8	380.2	0	400.3	471.6	57.6	0	0
3	0	384.5	237.6	254.2	46.8	134.6	329.8	0	0	0
4	301.7	82.1	223.6	101.5	166.0	205.9	0	0	0	0
5	207.0	79.9	235.4	77.8	115.6	126.7	40.3	0	0	0
7	166.3	173.5	95.0	98.3	33.1	123.5	17.6	0	0	0
10	183.6	63.7	101.9	108.7	57.6	52.9	4.3	0	0	0
15	200.5	78.5	113.8	50.8	43.9	20.2	9.0	0	0	0
20	226.8	114.8	76.7	36.4	22.7	7.2	9.0	0	0	0
30	326.9	64.8	7.9	14.8	6.1	2.9	10.8	2.5	0	0
50	455.4	7.9	13.0	7.9	0	4.3	9.4	0	2.5	0
75	334.4	4.3	0	7.9	4.3	7.2	2.2	0	0	2.5
100	276.5	0	6.1	1.8	4.3	7.2	2.2	0	0	2.5

NOTE Load effect from traffic mix includes dynamic factor $(1 + \varphi)$ defined in ENV 1991-3:1995, equation F.1.

Table F.3.5 — Load spectra for standard traffic mix, influence line 2, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 2: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	4.3	78.7	394.7	0	98.8	771.5	51.5	0	0
3	0	4.3	79.9	393.5	0	98.8	771.5	51.5	0	0
4	0	82.1	185.2	210.4	107.8	539.3	274.7	0	0	0
5	0	82.1	387.9	163.3	342.5	177.7	246.1	0	0	0
7	0	275.2	208.6	421.9	221.9	16.4	227.3	28.1	0	0
10	28.1	371.9	393.8	207.7	142.7	77.2	135.9	42.1	0	0
15	211.3	559.4	224.3	140.4	125.5	71.8	64.6	2.2	0	0
20	387.7	487.6	195.3	109.6	102.2	54.2	56.3	0	0	0
30	596.2	378.0	155.2	133.9	87.7	34.4	9.9	0	0	0
50	873.7	234.0	153.7	100.1	25.6	6.8	4.3	1.3	0	0
75	940.7	306.4	99.4	30.8	14.2	2.2	4.7	0	1.3	0
100	1 035.4	285.1	51.1	16.4	5.6	3.6	1.1	0	1.3	0

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F.3.6 — Load spectra for standard traffic mix, influence line 3, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 3: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	77.8	172.8	289.8	317.9	268.7	248.4	0	0	0
3	0	274.3	479.3	49.1	317.2	17.8	237.6	0	0	0
4	142.6	395.1	337.9	155.5	106.0	194.0	44.3	0	0	0
5	221.4	414.5	326.7	76.0	126.9	164.3	45.5	0	0	0
7	366.5	388.3	273.6	109.6	106.6	123.1	7.7	0	0	0
10	481.1	465.5	202.7	75.1	81.4	15.5	47.7	0	0	0
15	743.6	234.9	221.0	47.3	65.5	52.0	5.4	1.3	0	0
20	779.6	279.4	212.8	56.2	19.8	19.6	6.8	1.3	0	0
30	980.5	251.1	75.1	35.6	16.9	6.8	5.8	2.3	1.3	0
50	1 059.7	247.0	42.5	9.2	4.5	6.8	3.2	0	1.3	1.3
75	1 238.9	103.7	11.7	6.1	3.1	5.0	4.3	0	1.3	1.3
100	1 291.0	52.6	9.5	7.0	0	5.0	4.3	0	0	2.5

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F3.7 — Load spectra for light traffic mix, influence line 1, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2, P_{L71}$, for various influence line lengths

Influence line length <i>L</i> m	Influence line 1: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	1 752.0	0	0	294.9	13.1	30.7	36.5	24.1	0	0
3	1 752.0	43.8	150.4	163.5	0	0	41.6	0	0	0
4	1 918.4	0	147.5	5.1	5.5	31.0	0	0	0	0
5	1 725.4	49.3	103.3	3.3	17.5	4.0	11.7	0	0	0
7	922.7	27.0	73.7	16.8	0	15.0	4.7	0	0	0
10	960.0	13.5	74.5	11.0	6.6	0.7	4.0	0	0	0
15	536.2	20.1	67.2	0	2.6	0	4.7	0	0	0
20	543.9	74.1	0	0	2.6	4.0	0.7	0	0	0
30	924.5	23.0	0	0	2.6	0	4.0	0.7	0	0
50	1 341.4	0	0	2.6	0	4.0	0	0	0.7	0
75	324.5	0	0	2.6	4.0	0	0	0	0	0.7
100	401.1	0	2.6	0	4.0	0	0	0	0	0.7

NOTE Load effect from traffic mix includes dynamic factor (1 + ϕ) defined in ENV 1991-3:1995, equation F.1.

Table F.3.8 — Load spectra for light traffic mix, influence line 2, giving number of cycles per year (in thousands) expressed as proportion of load model 71 $\times \phi_2$, P_{L71} , for various influence line lengths

Influence line length L m	Influence line 2: thousands of cycles per year for load proportion of load model 71 $\times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	0	0	610.6	1 429.7	13.1	77.7	20.1	0	0	0
3	0	0	2 040.4	0	82.3	27.7	0	0	0	0
4	0	0	2 040.4	56.9	33.9	20.1	0	0	0	0
5	0	438.0	1 602.4	67.9	41.6	1.5	0	0	0	0
7	0	1 023.5	1 084.8	15.7	23.4	4.0	0	0	0	0
10	1 016.9	173.0	924.9	9.1	12.4	15.0	0	0	0	0
15	1 020.2	674.9	422.3	18.2	11.7	4.0	0	0	0	0
20	1 361.8	170.8	596.4	17.5	4.7	0	0	0	0	0
30	1 522.8	89.4	521.6	10.2	6.6	0.7	0	0	0	0
50	1 532.6	576.7	34.7	2.6	0	4.0	0.7	0	0	0
75	1 629.0	431.4	83.6	2.6	0	4.0	0	0.7	0	0
100	1 881.9	185.1	77.0	2.6	4	0	0	0	0.7	0

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Table F3.9 — Load spectra for light traffic mix, influence line 3, giving number of cycles per year (in thousands) expressed as proportion of load model $71 \times \phi_2, P_{L71}$, for various influence line lengths

Influence line length L m	Influence line 3: thousands of cycles per year for load proportion of load model $71 \times \phi_2$, where $P_{L71} =$									
	0.0 to 0.1	0.1 to 0.2	0.2 to 0.3	0.3 to 0.4	0.4 to 0.5	0.5 to 0.6	0.6 to 0.7	0.7 to 0.8	0.8 to 0.9	0.9 to 1.0
2	1 752.0	0	294.9	0	40.9	63.5	0	0	0	0
3	1 752.0	51.1	267.2	8.0	41.6	1.5	29.9	0	0	0
4	1 752.0	166.8	159.5	28.5	13.1	5.5	25.9	0	0	0
5	1 848.7	78.1	179.9	8.0	5.1	15.7	15.7	0	0	0
7	1 853.5	156.6	100.7	4.0	20.8	11.0	4.7	0	0	0
10	1 900.9	140.2	86.5	1.5	13.5	4.0	4.7	0	0	0
15	1 974.7	76.3	78.1	5.5	12.0	0	4.0	0.7	0	0
20	2 013.0	46.7	74.1	10.2	2.6	0	4.0	0.7	0	0
30	2 052.4	80.7	11.0	0	2.6	0	4.0	0	0.7	0
50	2 067.7	76.3	0	2.6	0	4.0	0	0	0.7	0
75	2 134.5	9.5	0	2.6	4.0	0	0	0	0	0.7
100	2 137.4	6.6	2.6	4.0	0	0	0	0	0	0.7

NOTE Load effect from traffic mix includes dynamic factor $(1 + \phi)$ defined in ENV 1991-3:1995, equation F.1.

Annex G (normative)

Basis of design: supplementary clauses to ENV 1991-1 for railway bridges including serviceability criteria

To clarify Tables G.1 and G.2 and requirements **G.2.3** and **G.2.4** of ENV 1991-3:1995, the values for the γ and ψ factors are given in Table G.1/2(a) for persistent and transient situations, and Table G.1/2(b) for accidental situations.

Values that differ from the boxed values in ENV 1991-3:1995 are shown in bold.

G.3.1 Performance criteria regarding deformations and vibrations

G.3.1.1 General

The limits of the bridge deformation for safety purposes take precedence over the limits for passenger comfort. Even if the criteria for vertical deflections for passenger comfort are conformed to, additional checks for twist, rotation, etc., shall be carried out.

G.3.1.2 Limit states for traffic safety

G.3.1.2.1 Vertical acceleration of the deck

2) To assess the vertical acceleration of the deck, the track(s) shall be loaded as specified in Table 6.6(a).

A simplified method for assessing the vertical acceleration of the deck is given in annex H.

G.3.1.2.2 Deck twist

2)P To assess the maximum permitted twist in accordance with equation G.1 of ENV 1991-3:1995, only one track shall be loaded with load model 71.

For bridges that carry more than one track, the maximum twist measured over a length of 3 m shall not exceed 7.5 mm, with the tracks loaded as specified in Table 6.6(a).

This limit relates to the total twist due to any design twist that may be present if the track is unloaded (e.g. in a transition curve) and any twist due to deformation of the bridge.

G.3.1.2.3 Rotations at the end of the deck (for ballasted tracks)

1)P For double track bridges, only one track shall be loaded.

For bridges that carry more than two tracks, the criteria for double track bridges with one track loaded may be used, unless otherwise specified by the relevant authority.

Notes to Tables G.1/2(a) and G.1/2(b)

(1) The consideration of the effect of wind may be required for slender structures.

(2) In addition to the partial safety factor γ , the variation in depth of ballast shall be considered in accordance with **5.3.3.1(2)** of ENV 1991-2-1:1995, and agreed with the relevant authority.

Key to abbreviations in Table G.1/2(a) and G.1/2(b)

- fav., f.: favourable;
- unfav., u.: unfavourable;
- dom. dominant;
- other: other (co-existing actions);
- max. vert./lat.: maximum vertical and lateral effects;
- lat. stab.: lateral stability;
- max. long.: maximum longitudinal effects;
- max. lat.: maximum lateral effects;
- loc. lat.: local lateral effects.

G.3.1.2.4 Horizontal deflections of the deck

To assess the horizontal deflection of the deck, the track(s) shall be loaded in accordance with Table 6.6(a).

G.3.1.3 Limiting values for the maximum deflection for passenger comfort

8)P The provisional limiting values for vertical deflection given in Figures G.3 and G.4 of ENV 1991-3:1995, and the limiting values of δ/L specified in Table G.5 of ENV 1991-3:1995 shall be replaced by the values specified in Section 8 of UIC¹⁾ Leaflet 776-3R, or by values specified by the relevant authority.

In assessing the vertical deflection of the bridge, the track(s) shall be loaded in accordance with Table 6.6(a) unless otherwise specified by the relevant authority.

The span/deflection ratios are only valid for use in conjunction with load model 71, because they take into account that the load from passenger trains is less severe than the load represented by load model 71.

¹⁾ UIC, 16 rue Jean Rey, F-75015 Paris, France.

Table G.1/2(a) — Numerical values for γ and ψ factors for persistent and transient situations
(excluding accidental situations)

			Persistent and transient design situations													
			Ultimate limit state				Serviceability limit state									
Combination			Global stability		Resistance		Rare	Infrequent		Frequent		Quasi-permanent		Fatigue		
Permanent actions			γ_{inf} (f.), γ_{sup} (u.)		γ_{inf} (f.), γ_{sup} (u.)		γ_{inf} (f.), γ_{sup} (u.)	γ_{inf} (f.), γ_{sup} (u.)		γ_{inf} (f.), γ_{sup} (u.)		γ_{inf} (f.), γ_{sup} (u.)		γ_{inf} (f.), γ_{sup} (u.)		
Direct actions	Self-weight, soil	Fav.	0.85		1.0		1.0	1.0		1.0		1.0		1.0		
		Unfav.	1.15		1.35 ^a		1.0	1.0		1.0		1.0		1.0		
	Ballast (2)	Fav.	0.85 (2)		0.85 (2)		1.0 (2)	1.0 (2)		1.0 (2)		1.0 (2)		1.0 (2)		
		Unfav.	1.15 (2)		1.35 (2)		1.0 (2)	1.0 (2)		1.0 (2)		1.0 (2)		1.0 (2)		
Indirect actions	Settlement	Fav.	0.0		0.0		0.0	0.0		0.0		0.0		0.0		
		Unfav.	1.50		1.35		1.0	1.0		1.0		1.0		1.0		
	Prestress relaxation creep	Fav.	—		1.0		1.0	1.0		1.0		1.0		1.0		
		Unfav.	—		1.0		1.0	1.0		1.0		1.0		1.0		
Variable actions			Dom. γ	Other $\gamma \times \psi_0$	Dom. γ	Other $\gamma \times \psi_0$	Other ψ_0	Dom. ψ'_1	Other ψ_1	Dom. ψ_1	Other ψ_2	Dom. ψ_2	Other ψ_2	Dom. γ	Other ψ_1	
Traffic	Bridge with one track	gr11 max. vert./lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr12 lat. stab.	1.45	$1.45 \times \mathbf{1.0}$	1.45	$1.45 \times \mathbf{1.0}$	1.0	—	—	—	—	—	0.0	—	—	
		gr13 max. long.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr14 max. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr15 loc. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
	Bridge with two tracks	gr11 max. vert./lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr12 lat. stab.	1.45	$1.45 \times \mathbf{1.0}$	1.45	$1.45 \times \mathbf{1.0}$	1.0	—	—	—	—	—	0.0	—	—	
		gr13 max. long.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr14 max. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr15 loc. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
	Bridge with three or more tracks	gr11 max. vert./lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr12 lat. stab.	1.45	$1.45 \times \mathbf{1.0}$	1.45	$1.45 \times \mathbf{1.0}$	1.0	—	—	—	—	—	0.0	—	—	
		gr13 max. long.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr14 max. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
		gr15 loc. lat.	1.45	1.45×0.8	1.45	1.45×0.8	0.8	1.0	0.8	0.8	0.0	—	0.0	—	—	
	Fatigue traffic load		—	—	—	—	—	—	—	—	—	—	—	—	1.0	—
	Non-public footpath loads		1.50	0.8	1.50	0.8	0.8	0.8	0.5	0.5	0.0	—	0.0	—	—	—
Other	Aerodynamics		1.50	0.8	1.50	0.8	0.8	1.0	0.8	0.8	0.0	—	—	—	—	
	Natural: wind		1.50	0.6	1.50	0.6	0.6	0.6	0.5	0.5	0.0	—	—	1.0	—	
	Natural: temperature		1.50	0.6	1.50	0.6	0.6	0.8	0.6	0.6	0.5	—	0.6	—	0.6	
	Seismic actions		—	—	—	—	—	—	—	—	—	—	—	—	—	
	Accidental actions		—	—	—	—	—	—	—	—	—	—	—	—	—	

NOTE Values in bold differ from the “boxed” values in Tables G1 and G2 of ENV 1991-3:1995. The term “boxed” is specified in the foreword to ENV 1991-3:1995.

^aIf the self-weight based on final design, a value of 1.2 may be taken for steel bridges, subject to agreement with the relevant authority.

NOTE Values in bold differ from the “boxed” values in Tables G1 and G2 of ENV 1991-3:1995. The term “boxed” is specified in the foreword to ENV 1991-3:1995.

^a If the self-weight based on final design, a value of 1.2 may be taken for steel bridges, subject to agreement with the relevant authority.

Table G.1/2(b) — Numerical values for γ and ψ factors for accidental situations

Actions			Accidental situation				
1) Permanent actions			Global stability		Resistance		
Direct actions	Self-weight, soil	Fav.	0.85		1.0		
		Unfav.	1.15		(1.35) ^a		
	Ballast (2)	Fav.	0.85 (2)		0.85 (2)		
		Unfav.	1.15 (2)		[1.35 (2)]		
Indirect actions	Settlement	Fav.	0.0		0.0		
		Unfav.	1.50		(1.35)		
	Prestress	Fav.	—		1.0		
		Relaxation creep	Unfav.	—		1.0	
2) Variable actions			Dom. ψ ₁	Other ψ ₂	Dom. ψ ₁	Other ψ ₂	
	Bridge with one track	gr11 max. vert./lat.	0.8	0.0	0.8	0.0	
		gr12 lat. stab.	—	—	—	—	
		gr13 max. long.	0.8	0.0	0.8	0.0	
		gr14 max. lat.	0.8	0.0	0.8	0.0	
		gr15 loc. lat.	0.8	0.0	0.8	0.0	
	Bridge with two tracks	gr11 max. vert./lat.	0.8	0.0	0.8	0.0	
		gr12 lat. stab.	—	—	—	—	
		gr13 max. long.	0.8	0.0	0.8	0.0	
		gr14 max. lat.	0.8	0.0	0.8	0.0	
		gr15 loc. lat.	0.8	0.0	0.8	0.0	
	Bridge with three or more tracks	gr11 max. vert./lat.	0.8	0.0	0.8	0.0	
		gr12 lat. stab.	—	—	—	—	
		gr13 max. long.	0.8	0.0	0.8	0.0	
		gr14 max. lat.	0.8	0.0	0.8	0.0	
		gr15 loc. lat.	0.8	0.0	0.8	0.0	
	Fatigue traffic load		—	—	—	—	
	Non-public footpath loads		0.5	0.0	0.5	0.0	
	Other	Aerodynamics		0.8	0.0	0.8	0.0
		Natural actions: wind		0.5	0.0	0.5	0.0
Natural actions: temperatures		0.6	0.5	0.6	0.5		
Seismic action		—	—	—	—		
Accidental actions		Rail traffic	b	—	b	—	
		Road traffic	—	—	b	—	
		Severance of overhead line	—	—	b	—	

^a If the self-weight is based on the final design, a value of 1.2 may be taken for steel bridges, if agreed with the relevant authority.

^b Accidental action Ad (design value).

Annex H (informative)

Dynamic analysis where there is a risk of resonance or excessive vibrations of railway structures: basis of the supplementary calculations

H.1 General

Annex H may be used as the basis of the supplementary calculations unless otherwise specified by the relevant authority.

Subject to agreement with the relevant authority, the simplified method given in **H.2**, **H.3** and **H.4** may be used for dynamic analysis, where:

- $220 \text{ km/h} < v < 350 \text{ km/h}$, and/or the natural frequency is outside the limits in Figure 6.9 of ENV 1991-3:1995; and
- the bridge is simply supported with a skew less than 20° .

The simplified method only covers structural members where the direction of travel of the train is parallel to the longitudinal axis of the member and where the influence line for deflection is equal to the span of the member.

NOTE More precise methods are currently undergoing development by the International Union of Railways (UIC)¹⁾.

H.2 Deck acceleration

H.2.1 Additional check for deck acceleration at value of k ($k_{\max}$) corresponding to $1.2 \times$ maximum line speed ($= 1.2v_{\max}/2Ln_0$).

For values of $k_{\max} < 0.5$:

$$A_{\text{deck}} = 2g \frac{F_{\text{hag}}^{**}}{W_b} \left(\frac{2k_{\max}}{1 - k_{\max}^2} \right) < 0.35g$$

or

$$A_{\text{deck}} = 2g \frac{F_{\text{hag}}^{**}}{W_b} \left(\frac{2k_e}{1 - k_e^2} \right) \cos \left(\frac{\pi}{2k_e} \right) < 0.35g$$

where

k_e is given by $0.9k_{\max} \leq k_e \leq 1.1k_{\max}$, and chosen to give the minimum upper limit of A_{deck} ;
 $\cos(\pi/2k_e)$ is taken as positive.

For values of $k_{\max} > 0.5$:

$$A_{\text{deck}} = 2g \frac{F_{\text{hag}}^{**}}{W_b} \left(\frac{2k_{\max}}{1 - k_{\max}^2} \right) \cos \left(\frac{\pi}{2k_{\max}} \right) < 0.35g$$

where

$\cos(\pi/2k_{\max})$ is taken as positive; and

A_{deck} is the acceleration of the deck (m/s^2);

$v_{\max}$ is the maximum speed of the train (m/s);

L is the span of the bridge (m);

n_0 is the natural frequency of the unloaded bridge (Hz);

F_{hag}^{**} is a single force representing the heaviest axles of the train (kN);

W_b is the weight of the bridge (kN);

g is the acceleration due to gravity (m/s^2).

¹⁾ UIC, 16 Rue Jean Rey, F-75015 Paris, France.

H.2.2 Additional check for deck acceleration at k (k_{res}) corresponding to resonance.

For values of $k_{\text{res}} < 0.5$:

$$A_{\text{deck}} = \frac{2gk_{\text{res}}}{1 - k_{\text{res}}^2} \times \frac{F_{\text{rag}}^{**}}{W_b} (\eta k_{\text{res}} + 2D_{\xi i}) < 0.35 g$$

or

$$A_{\text{deck}} = \frac{2gk_{\text{eres}}}{1 - k_{\text{eres}}^2} \times \frac{F_{\text{rag}}^{**}}{W_b} \left[\eta k_{\text{res}} + 2D_{\xi i} \cos \left(\frac{\pi}{2k_{\text{eres}}} \right) \right] < 0.35g$$

where

k_{eres} is given by $0.9k_{\text{res}} \leq k_{\text{eres}} \leq 1.1k_{\text{res}}$, and chosen to give the minimum upper limit of A_{deck} ;
 $\cos(\pi/2 k_{\text{eres}})$ is taken as positive.

For values of $k_{\text{res}} > 0.5$:

$$A_{\text{deck}} = \frac{2gk_{\text{res}}}{1 - k_{\text{res}}^2} \times \frac{F_{\text{rag}}^{**}}{W_b} \left[\eta k_{\text{res}} + 2D_{\xi i} \cos \left(\frac{\pi}{2k_{\text{res}}} \right) \right] < 0.35g$$

where

$\cos(\pi/2k_{\text{res}})$ is taken as positive; and

$k_{\text{res}} = \frac{d}{2L} \times \frac{1}{i}$, with $i = 1, 2, 3$ or 4 but considering only values of $k_{\text{res}} < k_{\text{max}}$;

$$D_{\xi i} = \sum_{n=1}^m (R^i)^{n-1} = \frac{1 - (R^i)^m}{1 - R^i} \quad (\text{coefficient for repetitive loads with damping});$$

F_{rag}^{**} is a single force representing the equally spaced group of axles of a train (kN);

R is the ratio of amplitude of successive amplitude peaks (agreed with the relevant authority);

d is the spacing of the group of axles (m);

L is the span of the bridge (m);

m is the number of equally spaced groups of axles;

$$\eta = F_{\text{hag}}^{**} + F_{\text{rag}}^{**}$$

H.3 Dynamic load effects**H.3.1** Additional check for load effects at k_{res} (as above $k_{\text{res}} < k_{\text{max}}$).

For values of $k_{\text{res}} < 0.5$:

$$(1 + \varphi' + \varphi'') \text{EUDL}_{\text{hag}} + 1.6 \left(\frac{2k_{\text{res}}}{1 - k_{\text{res}}^2} \right) \times D_{\xi i} \times F_{\text{rag}}^{**} < \Phi \times \text{EUDL}_{\text{LM71}}$$

or

$$(1 + \varphi' + \varphi'') \text{EUDL}_{\text{hag}} + 1.6 \left(\frac{2k_{\text{eres}}}{1 - k_{\text{eres}}^2} \right) \cos \left(\frac{\pi}{2k_{\text{eres}}} \right) \times D_{\xi i} \times F_{\text{rag}}^{**} < \Phi \times \text{EUDL}_{\text{LM71}}$$

where

k_{eres} is given by $0.9k_{\text{res}} \leq k_{\text{eres}} \leq 1.1k_{\text{res}}$, and chosen to give the maximum value of

$$\left(\frac{2k_{\text{eres}}}{1 - k_{\text{eres}}^2} \right) \cos \left(\frac{\pi}{2k_{\text{eres}}} \right), \text{ and } \cos \left(\frac{\pi}{2k_{\text{eres}}} \right) \text{ is taken as positive.}$$

For values of $k_{\text{res}} > 0.5$:

$$(1 + \varphi' + \varphi'') \text{EUDL}_{\text{hag}} + 1.6 \left(\frac{2k_{\text{res}}}{1 - k_{\text{res}}^2} \right) \cos \left(\frac{\pi}{2k_{\text{res}}} \right) \times D_{\xi i} \times F_{\text{rag}}^{**} < \Phi \times \text{EUDL}_{\text{LM71}}$$

where

$\cos \left(\frac{\pi}{2k_{\text{res}}} \right)$ is taken as positive; and

- φ' and φ'' are as defined in ENV 1991-3:1995, annex E;
- φ' is calculated from k_{res} (see **H.2.2**);
- F_{rag}^{**} is a single force representing the equally spaced group of axles of a train (kN);
- EUDL_{hag} is the equivalent uniformly distributed load representing the heaviest group of axles of a train (kN);
- $\text{EUDL}_{\text{LM71}}$ is the equivalent uniformly distributed load representing load model 71 (kN).
- Φ is as defined in ENV 1991-3:1995, **6.4.3.2**.

H.3.2 Additional check for load effects of heaviest group of axles of a train at k_{max} :

$$(1 + \varphi' + \varphi'') \text{EUDL}_{\text{hag}} < \Phi \times \text{EUDL}_{\text{LM71}}$$

where

- φ' and φ'' are as defined in ENV 1991-3:1995, annex E, and φ' is calculated from k_{max} ;
- Φ is as defined in ENV 1991-3:1995, **6.4.3.2**.

H.4 General conditions

H.4.1 All conditions shall be conformed to in applying the simplified method, otherwise a special study should be undertaken.

At least train types 3 and 4 (see annex F), together with any other planned or foreseeable trains for which $v > 220$ km/h, should be considered.

The value of $\cos(\pi/2k_{\text{res}})$ should be taken as positive.

When calculating F_{rag} or F_{hag} for a group of axle loads, only the axles within a group that can be placed on the bridge span should be taken into account.

NOTE ** If the group of axles comprises more than one axle, the equivalent single force representing the group of axles should be obtained by summation of the individual axle loads. If the conditions do not apply, the summation of the individual axles may be represented by the expressions in **H.4.3** and **H.4.4**. The expressions are derived from a sinusoidal influence line of length L for forced vibrations and of length kL for free vibrations. The centre of gravity of the group of axles is assumed to coincide with the position of the maximum value of the appropriate influence line.

H.4.2 For symmetric and equal groups of axles:

- C_{rf2} $\times$ repeated axle load (two axles);
- C_{rf4} $\times$ repeated axle load (four axles);

where

C_{rf2} and C_{rf4} are defined in **H.4.3** and **H.4.4** respectively.

H.4.3 For axle loads comprising a single two axle bogie (with equal axle loads):

$$C_{\text{rf2}} = 2 \cos \beta$$

where

$$\beta = \frac{\pi}{kL} S_1 \quad \text{when considering free vibrations (repetitive axle loads);}$$

$$\beta = \frac{\pi}{L} S_1 \quad \text{when considering forced vibrations (heaviest axle loads);}$$

S_1 is the half distance between axles.

H.4.4 For axle loads comprising two sets of two axle bogies (with equal and symmetrical axle loads):

$$C_{rf4} = 4 \cos \left(\frac{2\beta + \gamma}{2} \right) \cos \left(\frac{-\gamma}{2} \right)$$

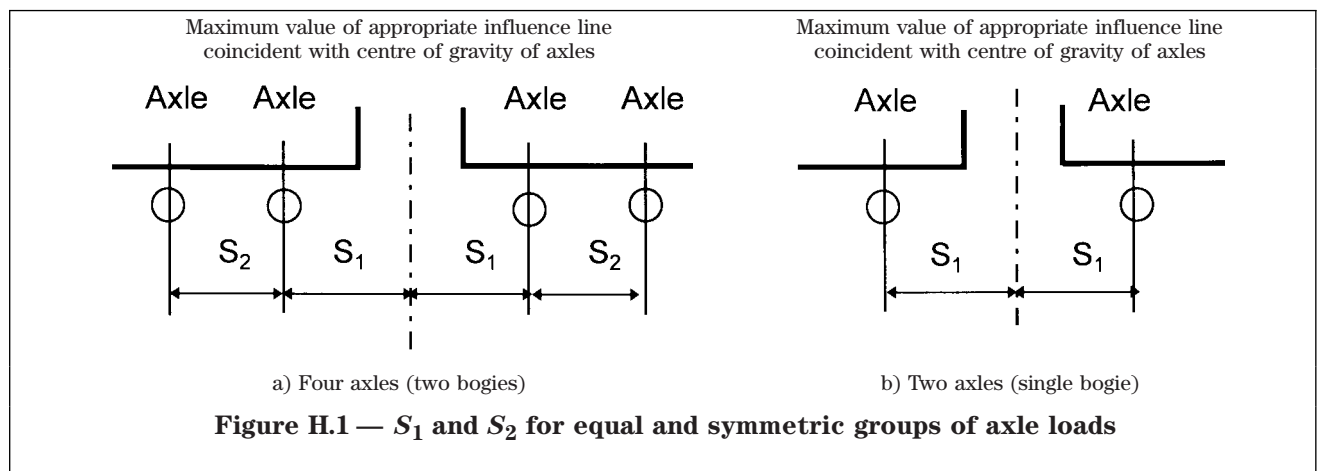
where

$$\gamma = \frac{\pi}{kL} S_2 \text{ and } \beta = \frac{\pi}{kL} S_1 \quad \text{when considering free vibrations (repetitive axle loads);}$$

$$\gamma = \frac{\pi}{L} S_2 \text{ and } \beta = \frac{\pi}{L} S_1 \quad \text{when considering forced vibrations (heaviest axle loads).}$$

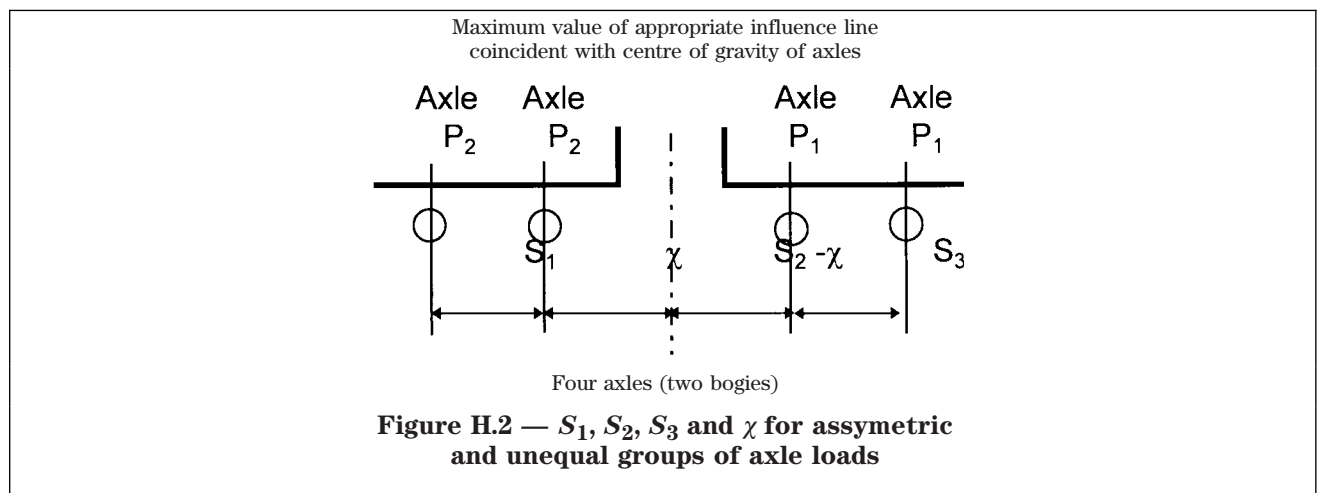
S_1 is the half distance between axles closest to the centre of gravity of the four axles (m) (see Figure H.1);

S_2 is the distance between bogie axles (m) (see Figure H.1).



For asymmetric groups of axles (see Figure H.2):

$$F_{\text{hag}} = P_2 \left[\cos \left(\frac{\pi \chi}{L} \right) + \cos \left(\frac{\pi}{L} (\chi + S_1) \right) \right] + P_1 \left[\cos \left(\frac{\pi}{L} (S_2 - \chi) \right) + \cos \left(\frac{\pi}{L} (S_3 + S_2 - \chi) \right) \right]$$



Annex J (normative)

Models for traffic loads in transient situations

The characteristic values specified in annex J shall be used for load model 71 for transient situations, unless otherwise specified by the relevant authority.

Annex S (informative)

Additional guidance

The *Design Manual for Roads and Bridges* (DMRB) applies to the design of highway bridges in the UK in versions published by the Department of the Environment, Transport and the Regions — Highways Agency, the Scottish Executive, the National Assembly for Wales and the Department for Regional Development — Northern Ireland. The DMRB includes loading aspects and may be used for additional guidance on loading.

For railway bridges, additional references and/or guidance should be obtained from the relevant railway authority.

Annex T (normative)

Design documents

T.1 General

Until completion of the ENVs and NADs for design of steel, concrete and composite bridges, the design of highway bridges of these materials shall conform to the strength requirements of the *Design Manual for Roads and Bridges* of the Department of the Environment, Transport and the Regions — Highways Agency, the Scottish Executive, the National Assembly for Wales and the Department for Regional Development — Northern Ireland, or as otherwise agreed by the relevant highways authority.

For railway bridges, design documents shall be as specified by the relevant railway authority.

NOTE Whether the DMRB or the NAD and associated ENV is used for strength, a compatible basis shall be agreed with the relevant authority until final issue of the EN documents and all partial factors on strength and actions are finally established.

T.2 Additional guidance for highway bridges

In relation to T.1 and specific clauses of this NAD the following should be noted.

Assessment and appraisal (1.1.1(4) of ENV 1991-3:1995)

Assessment and appraisal should follow the guidance in the DMRB, but with loading aspects related to the ENV and NAD. Assessment should only be carried out in accordance with the DMRB until all aspects of assessment and appraisal of existing structures are addressed by the relevant EN documents.

Definitions (1.4 of ENV 1991-3:1995)

Differences in terminology, particularly with respect to “carriageways” and “lanes”, and with regard to use of persistent/transient “situations” with permanent/variable actions, are defined in ENV 1991-1.

Design situations and requirements (clause 3 of ENV 1991-3:1995)

Coexistence of loads should generally follow the DMRB requirements. Execution conditions relating to fabrication, erection and construction do not form part of this ENV or NAD, but general aspects should be considered in development of the design prior to specific execution requirements. Additional guidance on design and loading aspects of bridge foundations, abutments, retaining walls, and piers may be found in the BRE/DTP publication *Bridge Foundations and Substructures*, The Stationery Office, 1979.

Representation of actions (4.2 of ENV 1991-3:1995)

The differences between the variable notional lanes of the DMRB (i.e. 2.50 m min. to 3.65 m max.) and the NAD/ENV fixed 3.00 m lanes and remaining areas should be noted.

The expression $3.6 a_{q1} = a_{qn} = a_r$ used in this NAD implies that areas loaded by load model 1 are effectively loaded by a uniform intensity irrespective of width and length plus associated tandem system defined by a_{Q1} , a_{Q2} and a_{Q3} .

The NAD/ENV figures are at maximum disparity with the DMRB figures where the numbers of notional lanes change in the DMRB (e.g. around 7.5 m, 11 m, 14.6 m, and 18 m). For single lane carriageways (up to 5.0 m width in the DMRB; 5.4 m width in the NAD/ENV), use of a_{SL} (= 0.69), produces the same factor on the tandem system and the UDL (i.e. $a_{Q1SL} = a_{Q1SL} = 0.58$).

Vertical loads: characteristic values (4.3 of ENV 1991-3:1995)

As with the paragraph relating to 4.2 of ENV 1991-3:1995, certain aspects of DMRB loading cannot be matched at all spans or load lengths. In particular with regard to LM3, the variable separation of the HB vehicle axles in the DMRB is not replicated in any of the annex A special vehicles considered alone, but the pair of vehicles given in 4.3.4a), b) or c) cover most arrangements of loaded lengths and continuous span influence lines with regard to total vehicle loads. It should also be noted that LM3 does not cater for axle loads greater than 150 kN, and heavier axle loads are covered by LM1 and/or LM2 under 4.3.2 and 4.3.3 of ENV 1991-3:1995 respectively. However, pairs of axle loads up to 450 kN each relevant to DMRB 45HB loads cannot be catered for by any of the ENV load systems, and special considerations may be required if vehicles with such pairs of axles are not restricted from entry onto the structure.

Horizontal forces: characteristic values (4.4 of ENV 1991-3:1995)

NAD/ENV loads may differ considerably from DMRB values, but generally their secondary nature should not require further consideration as the NAD/ENV loads are compatible with the vehicle loading considered. If horizontal forces are of prime concern, some further considerations may be required. In particular it should be noted that the effects of NAD/ENV centrifugal forces can be significantly less onerous than the forces currently in the DMRB, and if these can affect the design, the difference in forces should be reviewed and a suitable basis agreed with the relevant authority. Similarly amended bases for other horizontal forces may also be required.

Groups of traffic loads (4.5 of ENV 1991-3:1995)

UK designers only familiar with “nominal” and “design” loads of the DMRB should ensure NAD/ENV loads are properly applied where these are stated as “characteristic” or “multi-component actions”, and “frequent”, “infrequent”, “characteristic” values or actions, particularly with regard to **T.1**, also noting use of “dominant” component action and associated/reduced group actions.

Fatigue load models (4.6 of ENV 1991-3:1995)

The appropriate traffic categories and models to be considered should be reviewed and agreed with the relevant authority at the outset of the design requirements. See annex Y for more guidance.

Accidental actions (4.7 of ENV 1991-3:1995)

The required basis for all accidental actions should be agreed with the relevant authority at the outset of the design requirements. This relates to vehicle collision forces on or under the bridge, as well as parapet and barrier requirements. The associated vehicular loading should also be considered or specified at the outset, as well as any specific requirements for service vehicles or maintenance requirements. The differences in NAD/ENV and DMRB terminology should be noted, especially with regard to “piers” and “supports”, “decks” and “superstructures”, “parapets” and “barriers”, etc., as well as the different partial factors, that have been allowed for in the tabulated NAD values, by adjustment of the collision forces given in annex X.

Actions on parapets (4.8 of ENV 1991-3:1995)

The comments and guidance on ENV 1991-3:1995, clause 3 and 4.7, given in this annex shall apply.

Actions on embankments (4.9 of ENV 1991-3:1995)

The comments and guidance on ENV 1991-3:1995, clause 3 and 4.7, given in this annex shall apply.

Pedestrian, cycle and other actions specifically for footbridges (clause 5 of ENV 1991-3:1995)

The comments and guidance on ENV 1991-3:1995, clause 3 and 4.7, given in this annex shall apply.

Annex X (normative)

Summary of boxed values indicated in ENV 1991-3:1995 and reference clause in this NAD

Table X.1 summarizes the boxed values indicated in ENV 1991-3:1995 together with the reference clause. Table X.1 also specifies the values that shall be used to replace the boxed values in application of this NAD.

Table X.1 — Boxed values indicated in ENV 1991-3:1995 and values specified in this NAD

Reference in ENV 1991-3:1995	Item	Unit	ENV	NAD
1.4.2	kerb height (minimum)	m	0.1	0.075 ^a
1.4.2	hard shoulder width (minimum)	m	2.0	2.75 ^a
1.4.2	hard strip width (minimum)	m	2.0	0.75 ^a
4.3.2, 3, 4	adjustment factors a_Q for axle loads	See Table 4.5.5		
4.3.2, 3, 4	adjustment factors a_q for UDLs	See Table 4.5.6		
4.3.2	span length for use of one axle replacement	m	10.0	10.0
4.3.2	minimum adjustment factor, a_{Qmin}	—	0.8	0.8 a_{SL}
4.3.3	single axle load (see note 1)	kN	400	450
4.3.3	single wheel load (see note 1)	kN	200	250
4.3.4	gap from standard vehicle for application of main loading (see note 2)	m	25.0	25.0
4.5.1	groups of traffic loads (ULS)	See Table 4.5.7		
4.5.2	groups of traffic loads (SLS)	See Tables 4.5.8 to 4.5.10		
4.6.2	fatigue model 1 factor for axle load	—	0.7	0.5
4.6.2	fatigue model 1 factor for UDL	—	0.3	0.1
4.7.2/5.6.2	collision force on support in direction of travel	kN	1 000	1 650 ^a
4.7.2/5.6.2	collision force on support perpendicular to travel	kN	500	825 ^a
5.3.2.2	concentrated footbridge load (see note 3)	kN	10	10
5.3.2.2	side of surface square of footbridge load	m	0.1	0.1
5.4	horizontal load from vertical UDL	%	10	10
5.4/5.6.3	horizontal load from service vehicle	%	60	60
C.2.1	wind speed with traffic (see note 4)	m/s	23 <i>m</i>	35 <i>g</i>
C.2.1	additional depth for traffic	m	2.0	2.5
Annex C/D.2.3	values of γ factors for ULS	See Table 4.5.1		
Annex C/D.3.3	values of γ factors for SLS	See Table 4.5.2		
Annex C/D.2.4	values of ψ factors for ULS	See Table 4.5.3		
Annex C/D.3.4	values of ψ factors for SLS	See Table 4.5.4		

NOTE 1 Assuming β_Q taken as a_{Q1} in accordance with 4.3.3. The value of β_Q may be increased to a maximum of $(1/a_{Q1})$ by agreement with the relevant authority, if single axle loads up to 450 kN are to be considered, or to a lower value (but not less than a_{Qmin} of this NAD) if specifically agreed with the relevant authority. (Each contact surface of the single wheel load corresponds to that of twin wheels.)

NOTE 2 The gap shall be applied around each part of the vehicle pairs as shown in Figure 4.5 of ENV 1991-3:1995.

NOTE 3 Assuming service vehicle prescribed by the relevant authority.

NOTE 4 *m* = 10 min or hourly mean; *g* = derived gust on carriageway.

^a Or as otherwise required in the DMRB or specified by the relevant authority (in accordance with the road classification if applicable).