UIC CODE 776-1

5th edition, August 2006

Original

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Loads to be considered in railway bridge design

Charges à prendre en considération dans le calcul des ponts-rails
Bei der Berechnung von Eisenbahnbrücken zu berücksichtigende Lasten

UNION INTERNATIONALE DES CHEMINS DE FER
INTERNATIONALER EISENBAHNVERBAND
INTERNATIONAL UNION OF RAILWAYS

Leaflet to be classified in Volume:

VII - Way and Works

Application:

With effect from 1. August 2006
All members of the International Union of Railways

Record of updates

1st edition, July 1974	First issue and 1 Amendment
2nd edition, January 1977	and 1 Amendment
3rd edition, July 1979	and 1 Amendment
4th edition, July 1994	Revised by Sub-committee "Bridges", January/February 1994
5th edition, August 2006	The leaflet has been extensively updated to reflect the studies carried out by the UIC in support of the preparation of Eurocodes EN1991-2 "Actions on structures: Traffic loads on bridges" and Annex A2 "Application for bridges" to EN1990 "Basis of design". Changes since the last edition are not shown.

The person responsible for this leaflet is named in the UIC Code

Contents

Summary

UIC Leaflet 776-1 describes the loads to be taken into account in the design of railway bridges.

The leaflet defines imposed loads (load models and characteristic values) associated with rail traffic which include:

- vertical loads for bridges,
- vertical loading for earthworks,
- dynamic effects,
- centrifugal actions,
- nosing action,
- braking and acceleration actions,
- and actions for Accidental Design Situations corresponding to the derailment of rail traffic on the bridge.

It gives also rules and methods for establishing combinations of actions and design values of actions to be taken into account in limit state design.

The commissioning party should specify additional requirements for the design of roofed bridges, moveable bridges or bridges carrying road and rail traffic or other structured carrying rail traffic loads (e.g. backfill behind a retaining wall).

1 - General principles

1.1 - Symbols

The following symbols apply:

1.1.1 - Latin upper case letters

A	Accompanying action
A_d	Design value of an accidental action
C_d	Design value of the relevant serviceability criterion
E_d	Design value of the effect of actions
$E_{d,\ dst}$	Design value of the effect of destabilising actions
$E_{d,\ stb}$	Design value of the effect of stabilising actions
R_d	Design value of the corresponding resistance of the structure
F_w^{**}	Wind force compatible with rail traffic
F_{wk}	Characteristic wind force
F_{wn}	Nominal wind force
G	Self-weight (general)
L	Length (general) Leading variable action
L_f	Influence length of the loaded part of curved track
L_i	Influence length
$L\Phi$	"Determinant" length (length associated with Φ)
M	Main accompanying variable action
O	Other accompanying variable action
P	Relevant representative value of a prestressing action
Q_{Ald}	Point load for derailment loading
Q_h	Horizontal force (general)
Q_k	Concentrated load
$Q\nu_k$	Concentrated vertical load
Q_{la}	Traction (acceleration) force
Q_{lb}	Braking force
Q_r	Rail traffic action (general, e.g. resultant of wind and centrifugal force)
Q_s	Nosing force
Q_t	Centrifugal force
Q_v	Vertical axle load
Q_{vi}	Wheel load
V	Speed in km/h Maximum Line Speed at the Site in km/h

1.1.2 - Latin lower case letters

a	Distance between rail supports, length of distributed loads (Load Models SW/0 and SW/2)
a_g	Horizontal distance to the track centre
c	Space between distributed loads (Load Models SW/0 and SW/2)
e	Eccentricity of vertical loads Eccentricity of resulting action (on reference plane) Base of natural logarithms
f	Reduction factor for centrifugal force
g	Acceleration due to gravity
h	Height (general)
h_t	Height of centrifugal force over running surface
m	Mass of structure per unit length
n_0	First natural bending frequency of the unloaded structure
q_{Ai}	Accidental line load
q_{A1d}, q_{A2d}	Distributed loading for derailment loading
q_f	Loading on non-public footpath
q_t	Centrifugal force
q_{vk}	Vertical distributed load
r	Radius of track curvature Transverse distance between wheel loads
s	Track gauge
u	Cant, relative vertical distance between the uppermost surface of the two rails at a particular location along the track
v	Speed in m/s
gri	Group of Loads, i is a number (i = 1 to n)

1.1.3 - Greek upper case letters

$\Phi(\Phi_2, \Phi_3)$	Dynamic factor for railway Load Models 71, SW/0 and SW/2

1.1.4 - Greek lower case letters

α	Load classification factor
γ	Partial factor (safety or serviceability)
φ, φ', φ''	Dynamic enhancement of static loading for Real Trains
ψ_0	Factor for the combination value of a variable action
ψ_1	Factor for the frequent value of a variable action
ψ_2	Factor for the quasi-permanent value of a variable action

1.2 - General

Railway bridges should be designed for the relevant actions associated with the types of loading listed in point 1.3 - page 6.

Recommendations for characteristic values of actions to be taken into account associated with rail traffic are given in point 2 - page 11.

The actions to be taken into account for loading other than due to rail traffic should be in accordance with the relevant international or national requirements.

1.2.1 - Design situations

Appropriate combinations of actions should be taken into account for the design of railway bridges, taking into account the circumstances under which the bridge is required to fulfil its function:

The following design situations should be taken into account:

- *Persistent design situations*, generally corresponding to conditions of normal use with a return period equal to the intended life of the structure;

- *Transient design situations*, corresponding to temporary conditions applicable to the structure with a return period much shorter than the life of the structure (including consideration of the execution of the structure, where a structure is brought into use in stages to carry railway traffic loading, etc. before construction is completed and loading requirements associated with maintenance of the bridge and tracks, etc.);

- *Accidental design situations*, including exceptional conditions, applicable to the structure including consideration of derailment on or in the vicinity of the bridge, impact from errant road traffic on the bridge, etc. and other relevant international and national requirements;

- *Seismic design situations*, where required in accordance with national requirements;

- any other design situations as required by the commissioning party;

- any other design situation as required by relevant international or national requirements.

NB : The commissioning party should specify:
- requirements relating to transient design situations,
- requirements relating to temporary bridges,
- the intended life of the structure which should generally be at least 100 years for a railway bridge.

1.2.2 - Combinations of actions

Guidance on appropriate combinations of actions to be taken into account is given in point 3 - page 30. Generally each action is considered in turn as a leading action with other actions taken as accompanying actions.

1.2.3 - Groups of loads

Point 3 also lists appropriate combinations and partial γ and ψ factors to be used when railway loading is considered using the group of loads technique. The group of loads technique has been developed to simplify the design process. Rail traffic loading is treated as a single multi-component variable action. The single multi-component action is then combined with other actions as a single variable action. Groups of loads that may be used for the design of railway bridges are defined in Table 4 - page 28.

The group of loads technique is not suitable for use in all situations. For example individual rail traffic actions should also be taken into account in the design of bearings, for the assessment of maximum lateral and minimum vertical traffic loading, design of bearing restraints, the assessment of maximum overturning effects on abutments (especially for continuous bridges), etc.

Further information on the group of loads technique is given in Appendix D - page 45.

1.2.4 - Additional loading considerations

In addition, the design of a railway bridge should take into account the relevant loading:

- associated with the construction of the bridge,
- appropriate to the stage of construction,
- appropriate to the use of the bridge where the structure is brought into use in stages prior to the completion of construction,
- requirements for temporary loading situations defined by the railway operator associated, for example, with track maintenance, replacement of bearings, etc.

1.2.5 - Design acceptance criteria and limit states

Guidance on the relevant performance requirements and design acceptance criteria for railway bridges are given in *UIC Leaflet 774-3 and 776-2* (see Bibliography - page 46) and relevant international and national requirements.

Basic requirements relating to the design of railway bridges should be in accordance with the relevant international and national requirements regarding structural resistance, serviceability, durability, fitness for intended use, avoidance of damage from events not disproportionate to original cause, etc.

This leaflet assumes that requirements relating to the design of the structure are in accordance with the requirements of relevant international and national requirements (e.g. *Eurocode EN 1990*, see Bibliography - page 46) and that the design of the structure is in accordance with limit state principles.

Generally, the design of a railway bridge should consider the following limit states:

- the ultimate limit states associated with collapse of all or part of the structure and other similar forms of structural failure (e.g. buckling failure, loss of equilibrium, rupture, excessive deformation, failure or excessive deformation of the supporting ground, etc.),
- fatigue failure of all or part of the structure (limit states corresponding to fatigue are outside the scope of this leaflet),

- serviceability limit states corresponding to conditions beyond which the specified service requirements for the structure are no longer met (e.g. for durability of the structure or for general deformation requirements, etc.) and *inter alia* deformation and vibration limits for railway bridges given in *UIC Leaflet 776-2*. To include consideration of both reversible and irreversible serviceability limit states,

- checks on design criteria relating to ensuring the safety of railway traffic (see *UIC Leaflet 774-3* for longitudinal forces and *UIC Leaflet 776-2* for deformation and vibration limits relating to interaction between train, track and bridge),

in accordance with the requirements of relevant international and national requirements.

1.3 - Actions

1.3.1 - Classification of actions

In accordance with the relevant international or national requirements, actions may generally be classified by the manner in which they vary with time:

- *permanent actions* that are either constant, vary very slowly with time or only occasionally change, for example self weight, imposed loads, uneven settlement, etc.,

- *variable actions*, e.g. rail traffic actions, wind, temperature effects, etc.,

- *accidental actions*, e.g. from impact from vehicles on bridge supports or superstructure, derailment loads on the bridge deck, etc.

1.3.2 - Actions to be taken into account

Railway bridges should be designed to take the following actions into account:

1.3.2.1 - Permanent actions

Direct actions:

- self weight,

- horizontal earth pressure and if relevant, other soil/ structure interaction forces,

- track and ballast,

- movable loads:
  - self weight of non structural elements,
  - loading from overhead line equipment (vertical and horizontal),
  - loading from other railway infrastructure equipment.

Indirect actions:

- settlement (for some structures consideration of absolute settlement can be critical when considering differential movement between a structure and the kinematic envelope of a train),

- differential settlement (including the effects of mining subsidence where required by the commissioning party),

- shrinkage and creep,

- prestress.

1.3.2.2 - Variable actions

1.3.2.2.1 - Rail traffic actions

- Vertical traffic actions (appropriate additional allowance to be made for dynamic effects):

  • UIC Load Model 71,
  • UIC Load Model SW/0,
  • UIC Load Model SW/2 (where required by the commissioning party),
  • Load Model HSLM (High Speed Load Model in accordance with *Eurocode 1991-2* (see Bibliography - page 46), where required by the Technical Specification for Interoperability of High Speed Traffic in accordance with the relevant EU Directive and/or the commissioning party (seel also *UIC Leaflet 776-2*),
  • Load Model "unloaded train" (for checking lateral stability in conjunction with lateral rail traffic actions and wind loading on the bridge and rail vehicles),
  • Load effects from Real Trains (where required by the commissioning party).

- Centrifugal;

- Traction and braking;

- Nosing;

- Longitudinal forces (see also *UIC Leaflet 774-3* for load effects generated by the interaction between track and structure in resisting variable actions);

- Load effects generated by the interaction between train, track and structure in presence of variable actions (see *UIC Leaflet 776-2*);

- Live load surcharge horizontal earth pressure;

- Aerodynamic actions (slipstream effects from passing rail traffic, etc.) (*UIC Leaflet 779-1*, see Bibliography - page 46).

1.3.2.2.2 - Other traffic actions

- Actions on public footpaths (uniformly distributed and point loads).

- Loads on non-public footpaths (uniformly distributed and point loads).

- Loads on platforms.

- Loads on areas where vehicular traffic permitted.

- Horizontal loads on pedestrian parapets.

- Horizontal loads on vehicle parapets due to vehicle containment, etc.

1.3.2.2.3 - Other actions

- Other operating actions:

  - stressing or destressing continuous welded rails.

- Construction loading:

  - plant,
  - personnel,
  - storage of materials,
  - actions associated with method of construction.

1.3.2.2.4 - Natural actions

- Wind.

NB : Where permitted by the commissioning party a reduced maximum wind speed compatible with rail traffic operation may be specified.

- Thermal (uniform, temperature, gradient, etc.).

- Thermal restraint from bearing friction.

- Water pressure:

  - ground water,
  - free water,
  - moving water,
  - uplift, etc.

- The effects of scour.

- Water borne debris.

- Ice loads (where required by the relevant authority).

- Ice pressure (where required by the relevant authority).

- Snow loading (where required by the relevant authority).

- Avalanche (where required by the relevant authority).

- Mud slides (where required by the relevant authority).

1.3.2.3 - Accidental actions

- Actions corresponding to derailment of rail traffic on the bridge.

- Actions corresponding to derailment of rail traffic beneath or adjacent to the bridge (*UIC Leaflet 777-1 and 777-2*, see Bibliography - page 46).

- Accidental loading from errant road vehicles beneath the bridge.

- Accidental loading from over height road vehicles beneath the bridge.

- Ship impact.

- Actions due to the rupture of catenaries.

- Actions due to the accidental breakage of rails.

- Accidental loadings during construction.

- Fire (where required by the relevant authority).

1.3.2.4 - Seismic actions

- Actions due to earthquake loading (where specified by the relevant international and national requirements).

1.4 - Characteristic values of actions

The rail loadings given in point 2 - page 11 have been developed using deterministic methods.

Subject to the loadings specified in point 2 being enhanced by appropriate partial factors the loadings may be considered as characteristic values.

The values of γ and ψ factors given in point 3 - page 30 are based on comparative calibration studies against a selection of European national limit state codes, which in turn are generally based on empirical and historical (including permissible stress design codes) methods.

NB : The comparative studies were carried out to support the drafting of *ENV 1991-3* and no further comparative studies have been carried out by the UIC to support the conversion of *ENV 1991-3* to *EN 1991-2* and *EN 1990, Annex A2*. The relevant authorities should consider the need for further comparative calculations before adopting the γ and ψ values given in Tables 1 to 3 of Appendix A.

Requirements for either considering:

- a mean value of an action,

- or where the variability is significant, an upper and lower bound value,

should be in accordance with the relevant international or national requirements.

To take account of the variability of ballast depth an additional factor of either 1,33 (ballast load effect unfavourable) or 0,75 (ballast load effect favourable) should be applied to the nominal depth of ballast beneath the underside of the sleeper. The minimum and maximum nominal depths of ballast beneath the sleeper to be taken into account should be specified by the commissioning party. Any additional ballast provided below the nominal depth of ballast may be considered as an imposed moveable load. Additionally, the ballast density (or range of ballast densities) to be taken into account should be specified by the commissioning party.

Generally, the design of a railway bridge should be verified using the partial factor method outlined in point 3 - page 30.

2 - Rail traffic actions and other actions for railway bridges

2.1 - Field of application

This point applies to rail traffic on the standard and wide track gauge.

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

This point is not applicable for actions due to:

- narrow-gauge railways,
- tramways and other light railways,
- preservation railways,
- rack and pinion railways,
- funicular railways.

Designers should pay special attention to temporary bridges because of the flexibility of some types of temporary structures. The loading and requirements for the design of temporary bridges should be specified by the commissioning party.

2.2 - Representation of actions - Nature of rail traffic loads

General rules are given for the calculation of the associated dynamic effects, centrifugal forces, nosing force, traction and braking forces.

Actions due to railway operations are given for:

- vertical loads: Load Models 71, SW (SW/0 and SW/2), and "unloaded train",
- vertical loading for earthworks,
- dynamic effects,
- centrifugal forces,
- nosing force,
- traction and braking forces,
- aerodynamic and slipstream actions from passing trains,
- actions due to overhead line equipment and other railway infrastructure and equipment.

Guidance on the evaluation of the combined response of structure and track to variable actions is given in *UIC Leaflet 774-3*.

Derailment actions for accidental design situations are given for the effect of rail traffic derailment on a structure carrying rail traffic.

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

Recommendations concerning the application of the following load models are given in point 2.8 - page 26.

2.3.1 - General

Rail traffic actions are defined by means of load models. Four models of railway loading are given:

- Load Model 71 and Load Model SW/0 (for continuous bridges) to represent normal rail traffic on mainline railways (see *UIC Leaflet 702*, see Bibliography - page 46 for the relevant rules of application),
- Load Model SW/2 to represent heavy loads,
- Load Model "unloaded train" to represent the effect of an unloaded train.

NB : In *UIC Leaflet 776-2*, a Load Model HSLM (comprising HSLM-A and HSLM-B) is given to represent the loading from passenger trains at speeds exceeding 200 km/h.

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

2.3.2 - Load Model 71

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

The load arrangement and the characteristic values for vertical loads shall be taken as shown in Fig. 1.

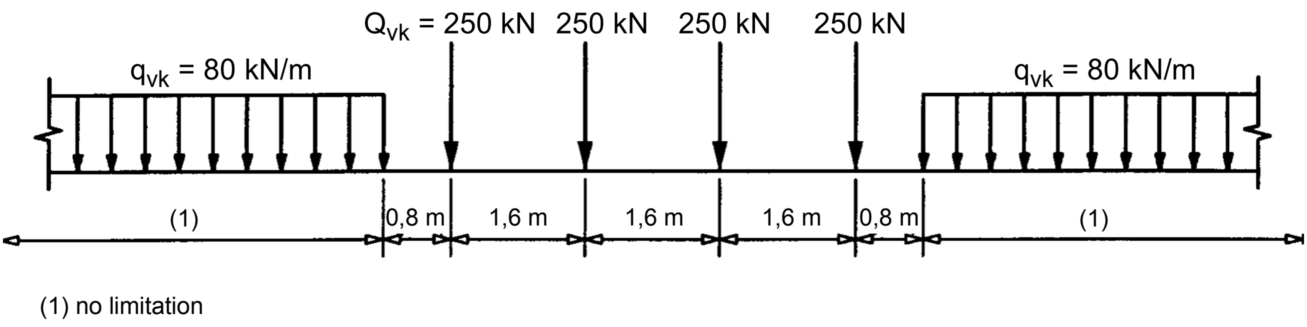


Fig. 1 - Load Model 71 and characteristic values for vertical loads

The characteristic values given in Fig. 1 should be multiplied by a factor α, on lines carrying rail traffic which is heavier or lighter than normal rail traffic. When multiplied by the factor α the loads are called "classified vertical loads". This factor α should be one of the following:

0,75 - 0,83 - 0,91 - 1,00 - 1,10 - 1,21 - 1,33 - 1,46

NB : The commissioning party should specify the value of α to be used. On international lines, it is recommended that $\alpha \geq 1$. For lines carrying traffic with 25t axles, the commissioning party should consider specifying α = 1,1.

The actions listed below should be multiplied by the same factor α:

- equivalent vertical loading for earthworks and earth pressure effects,
- centrifugal forces,
- nosing force (multiplied by α for $\alpha \geq 1$ only),
- traction and braking forces,
- combined response of structure and track to variable actions,
- derailment actions for Accidental Design Situations,
- Load Model SW/0 for continuous span bridges.

For checking limits of deflection, classified vertical loads and other actions enhanced by α should be used (except for passenger comfort where α should be taken as unity).

2.3.3 - Load Models SW/0 and SW/2

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

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

The load arrangement should be taken as shown in Fig. 2, with the characteristic values of the vertical loads according to Table 1.

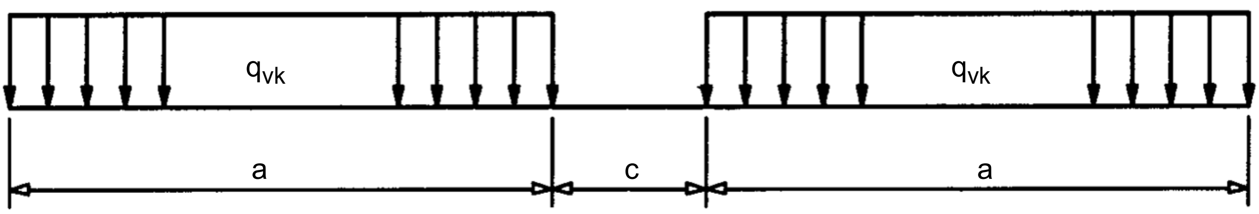


Fig. 2 - Load Models SW/0 and SW/2

Table 1 : Characteristic values for vertical loads for Load Models SW/0 and SW/2

Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15,0	5,3
SW/2	150	25,0	7,0

The lines or section of line over which heavy rail traffic may operate where Load Model SW/2 should be taken into account, should be designated by the railway operator.

2.3.4 - Load Model "unloaded train"

For some specific verifications, a particular load model is used, called "unloaded train". The Load Model "unloaded train" consists of a vertical uniformly distributed load with a characteristic value of 10,0 kN/m.

2.3.5 - Eccentricity and transverse distribution of vertical loads (Load Models 71 and SW/0)

The effect of lateral displacement of vertical loads should be considered by taking the ratio of wheel loads on all axles as up to 1,25:1,00 on any one track.

NB : The above criteria may be used to determine the eccentricity of loading with respect to the centreline of the track. Also see point 2.8.1 for requirements relating to the position of tracks.

2.3.6 - Transverse and longitudinal distribution of vertical loads

The transverse and longitudinal distribution of actions on bridges with ballasted track is given in *UIC Leaflet 774-2* (see Bibliography - page 46).

2.3.7 - Equivalent vertical loading for earthworks and earth pressure effects

For global effects, the equivalent characteristic vertical loading due to rail traffic actions for earthworks under or adjacent to the track may be taken as the appropriate load model (LM71, or classified vertical load where required, and SW/2 where required) uniformly distributed over a width of 3,00 m at a level 0,70 m below the running surface of the track.

No dynamic factor or increment needs to be applied to the above uniformly distributed load.

For the design of local elements close to a track (e.g. ballast retention walls), a special calculation should be carried out taking into account the maximum local vertical, longitudinal and transverse loading on the element due to rail traffic actions.

2.3.8 - General maintenance loading for non-public footpaths

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

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

For the design of local elements, a concentrated load $Q_k = 2{,}0$ kN acting alone should be taken into account and applied on a square surface with a 200 mm side.

2.3.9 - Loading for platforms

The loading for station platforms on bridges should be in accordance with the requirements of the railway operators.

2.3.10 - Loads on parapets and safety barriers

The horizontal loading for pedestrian parapets and vehicle parapets should be in accordance with the relevant national and international requirements for pedestrian load effects and load effects from constraining vehicular traffic.

2.4 - Dynamic effects

2.4.1 - Introduction

A static analysis should be carried out with the load models (LM71 and where required Load Models SW/0 and SW/2). The results should be multiplied by the dynamic factor Φ defined in point 2.4.2.2 - page 16 (and if required multiplied by α).

The criteria for determining whether a dynamic analysis is required are given in *UIC Leaflet 776-2*.

2.4.2 - Dynamic factor Φ (Φ_2, Φ_3)

2.4.2.1 - Field of application

The dynamic factor Φ takes account of the dynamic magnification of stresses and vibration effects in the structure but does not take account of resonance effects.

The natural frequency of the structure should be within the frequency limits given in *UIC Leaflet 776-2, Fig. 11*. Where the criteria specified in *UIC Leaflet 776-2* are not satisfied, there is a risk that resonance or excessive vibration of the bridge may occur (with a possibility of excessive deck accelerations leading to ballast instability, etc. and excessive deflections and stresses, etc.). For such cases, a dynamic analysis should be carried out to calculate impact and resonance effects.

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

2.4.2.2 - Definition of the dynamic factor Φ

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

Generally, the dynamic factor Φ is taken as either Φ_2 or Φ_3 according to the quality of track maintenance as follows:

1. for carefully maintained track:

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

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

2. for track with standard maintenance:

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

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

where:

L_Φ "determinant" length (length associated with Φ in [m] defined in Table 2 - page 17).

NB : The dynamic factors were established for simply supported girders. The length L_Φ allows these factors to be used for other structural members with different support conditions.

If no dynamic factor is specified, Φ_3 should be used.

The dynamic factor Φ should not be used with:

- the loading due to Real Trains,
- the Load Model "unloaded train" (see point 2.3.4 - page 14)

2.4.2.3 - Determinant length L_Φ

The determinant lengths L_Φ to be used are given in Table 2.

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

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

Table 2 : Determinant lengths L_{Φ}

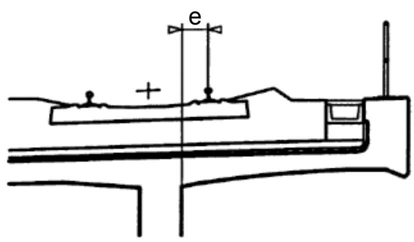
Case	Structural element	Determinant length L_{Φ}
Steel deck plate: closed deck with ballast bed (orthotropic deck plate) (for local and transverse stresses)		
1.	Deck with cross girders and continuous longitudinal ribs:	
1.1	Deck plate (for both directions)	3 times cross girder spacing
1.2	Continuous longitudinal ribs (including small cantilevers up to 0,50 m) [a]	3 times cross girder spacing
1.3	Cross girders	Twice the length of the cross girder
1.4	End cross girders	3,6 m [b]
2.	Deck plate with cross girders only:	
2.1	Deck plate (for both directions)	Twice cross girder spacing + 3 m
2.2	Cross girders	Twice cross girder spacing + 3 m
2.3	End cross girders	3,6 m [b]
Steel grillage: open deck without ballast bed [b] (for local and transverse stresses)		
3.1	Rail bearers: - as an element of a continuous grillage - simply supported	 3 times cross girder spacing Cross girder spacing + 3 m
3.2	Cantilever of rail bearer [a]	3,6 m [b]
3.3	Cross girders (as part of cross girder/continuous rail bearer grillage)	Twice the length of the cross girder
3.4	End cross girders	3,6 m [b]
Concrete deck slab with ballast bed: (for local and transverse stresses)		
4.1	Deck slab as part of box girder or upper flange of main beam: - spanning transversely to the main girders - spanning in the longitudinal direction - cross girders - transverse cantilevers supporting railway loading	 3 times span of deck plate 3 times span of deck plate Twice the length of the cross girder  *Fig. 3 - Transverse cantilever supporting railway loading* - e $\leq$ 0,5 m: 3 times the distance between the webs e > 0,5 m: [a]

Table 2 : Determinant lengths L_Φ

Case	Structural element	Determinant length L_Φ
4.2	Deck slab continuous (in main girder direction) over cross girders	Twice the cross girder spacing
4.3	Deck slab for half through and trough bridges:	
	- spanning perpendicular to the main girders - spanning in the longitudinal direction	Twice span of deck slab + 3 m Twice span of deck slab
4.4	Deck slabs spanning transversely between longitudinal steel beams in filler beam decks	Twice the determinant length in the longitudinal direction
4.5	Longitudinal cantilevers of deck slab	- $e \leq 0{,}5$ m: 3,6 [b] - $e > 0{,}5$ m: [a]
4.6	End cross girders or trimmer beams/trimmer girders	3,6 [b]
NB :	For cases 1.1 to 4.6 inclusive, L_Φ is subject to a maximum of the determinant length of the main girders.	
Main girders		
5.1	Simply supported girders and slabs (including steel beams embedded in concrete)	Span in main girder direction
5.2	Girders and slabs continuous over n spans with: $L_m = 1/n(L_1 + L_2 + .. + L_n)$	$L_\Phi = k \times L_m$, but not less than max L_i (i = 1, ..., n) n = 2 / 3 / 4 / ≥ 5 k = 1,2 / 1,3 / 1,4 / 1,5
5.3	Portal frames and closed frames or boxes:	
	- single-span	Consider as three-span continuous beam (use 5.2, with vertical and horizontal lengths of members of the frame or box).
	- multi-span	Consider as multi-span continuous beam (use 5.2, with lengths of end vertical members and horizontal members)
5.4	Single arch, archrib, stiffened girders of bowstrings	Half-span
5.5	Series of arches with solid spandrels retaining fill	Twice the clear opening
5.6	Suspension bars (in conjunction with stiffening girders)	4 times the longitudinal spacing of the suspension bars
Structural supports		
6.	Columns, trestles, bearings, uplift bearings, tension anchors and for the calculation of contact pressures under bearings	Determinant length of the supported members

a. In general all cantilevers greater than 0,50 m supporting rail traffic actions need a special study in accordance with 6.4.6 and with the loading agreed with the relevant authority specified in the National Annex.

b. It is recommended to apply Φ_3.

2.4.2.4 - Reduced dynamic effects

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

$$\text{red } \Phi_{2,3} = \Phi_{2,3} - \frac{h - 1,00}{10} \geq 1,0$$

where:

h is the height of cover including the ballast from the top of the deck to the top of the sleeper, (for arch bridges, from the crown of the extrados) [m].

The effects of rail traffic actions on columns with a slenderness (buckling length/radius of gyration) < 30, abutments, foundations, retaining walls and ground pressures may be calculated without taking into account dynamic effects.

2.5 - Horizontal forces - Characteristic values

2.5.1 - Centrifugal forces

Where the track on a bridge is curved over the whole or part of the length of the bridge, the centrifugal force and the track cant should be taken into account.

The centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1,80 m above the running surface. For some traffic types, e.g. double stacked containers, the individual project should specify an increased value of h_t.

The centrifugal force should always be combined with the vertical traffic load. The centrifugal force should not be multiplied by the dynamic factor Φ_2 or Φ_3.

NB : When considering the vertical effects of centrifugal loading, the vertical load effect of centrifugal loading less any reduction due to cant is enhanced by the relevant dynamic factor.

The characteristic value of the centrifugal force shall be determined according to the following equations:

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

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

where:

Q_{tk}, q_{tk} characteristic values of the centrifugal forces [kN, kN/m],

Q_{vk}, q_{vk} characteristic values of the vertical loads specified in point 2.3 - page 12 (excluding any enhancement for dynamic effects) for Load Models 71, SW/0, SW/2 and "unloaded train". For Load Model HSLM, the characteristic value of centrifugal force should be determined using Load Model 71,

f	reduction factor (see below),
v	maximum speed [m/s],
V	maximum speed [km/h],
g	acceleration due to gravity [9,81 m/s^2],
r	radius of curvature [m].

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

The calculations should be based on the Maximum Line Speed at the Site specified for the individual project. In the case of Load Model SW/2, a maximum speed of 80 km/h may be assumed.

In addition, for bridges located in a curve, the case of the loading specified in point 2.3.2 - page 12 and, if applicable, point 2.3.3 - page 13 should also be considered without centrifugal force.

For Load Model 71 (and where required Load Model SW/0) and a Maximum Line Speed at the Site higher than 120 km/h, the following cases should be considered:

Case a: Load Model 71 (and where required Load Model SW/0) with its dynamic factor and the centrifugal force for V=120 km/h with f = 1.

Case b: Load Model 71 reduced (f x Q_{vk}, f x q_{vk}) (and where required f x Load Model SW/0) with its dynamic factor and the centrifugal force for the maximum speed V specified, with a value for the reduction factor f.

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

$$f = \left[1 - \frac{V - 120}{1\,000}\left(\frac{814}{V} + 1{,}75\right)\left(1 - \sqrt{\frac{2{,}88}{L_f}}\right)\right]$$

subject to a minimum value of 0,35 where:

L_f	is the influence length of the loaded part of curved track on the bridge, which is most unfavourable for the design of the structural element under consideration [m],
V	is the maximum speed.

f = 1	for either	$V \leq 120$ km/h	or	$L_f \leq 2{,}88$ m
f < 1	for	120 km/h $< V \leq$ 300 km/h	and	$L_f > 2{,}88$ m
$f_{(V)} = f_{(300)}$	for	$V > 300$ km/h	and	$L_f > 2{,}88$ m

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

For LM71 and SW/0, centrifugal forces should be determined using classified vertical loads (see point 2.3.2 - page 12) in accordance with the load cases given in Table 3 - page 21.

Table 3 : Load cases for centrifugal force corresponding to values of α and maximum line speed at site

Value of α	Maximum line speed at site [km/h]	Centrifugal force based on: [a]				Associated vertical traffic action base on: [b]
		V [km/h]	α	f		
α < 1	> 120	V	1 [c]	f	1 [c] x f x (LM71 "+" SW/0) for case b	Φ x 1[c] x 1 x (LM71 "+" SW/0)
		120	α	1	α x 1 x (LM71 "+" SW/0) for case a	Φ x α x 1 x (LM71 "+" SW/0)
		0	-	-	-	
	≤ 120	V	α	1	α x 1 x (LM71 "+" SW/0)	
		0	-	-	-	
α = 1	> 120	V	1	f	1 x f x (LM71 "+" SW/0) for case b	Φ x 1 x 1 x (LM71 "+" SW/0)
		120	1	1	1 x 1 x (LM71 "+" SW/0) for case a	Φ x 1 x 1 x (LM71 "+" SW/0)
		0	-	-	-	
	≤ 120	V	1	1	1 x 1 x (LM71 "+" SW/0)	
		0	-	-	-	
α > 1	> 120 [d]	V	1	f	1 x f x (LM71 "+" SW/0) for case b	Φ x 1 x 1 x (LM71 "+" SW/0)
		120	α	1	α x 1 x (LM71 "+" SW/0) for case a	Φ x α x 1 x (LM71 "+" SW/0)
		0	-	-	-	
	≤ 120	V	α	1	α x 1 x (LM71 "+" SW/0)	
		0	-	-	-	

a. Vertical load effect of centrifugal loading less any reduction due to cant should be enhanced by the relevant dynamic factor.
b. 0,5 x (LM71"+"SW/0) instead of (LM71"+"SW/0) where vertical traffic actions favourable.
c. α = 1 to avoid double counting the reduction in mass of train with f.
d. Valid for heavy freight traffic limited to a maximum speed of 120 km/h.

where:	V	maximum speed [km/h]
	f	reduction factor
	α	factor for classified vertical loads in accordance with point 2.3.2 - page 12
	LM71 "+" SW/0	Load Model 71 and, if relevant, Load Model SW/0.

The criteria in this point are not valid for heavy freight traffic with a maximum permitted vehicle speed exceeding 120 km/h. For heavy freight traffic with a speed exceeding 120 km/h, additional requirements should be specified.

2.5.2 - Nosing force

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

The characteristic value of the nosing force should be taken as $Q_{sk} = 100$ kN. It should not be multiplied by the factor Φ (see point 2.4.2 - page 15) or by the factor f in point 2.5.1 - page 19.

The characteristic value of the nosing force should be multiplied by the factor α in accordance with point 2.3.2 - page 12 for values of $\alpha \geq 1$.

The nosing force should always be combined with a vertical traffic load.

2.5.3 - Actions due to traction and braking

Traction and braking forces act at the top of the rails in the longitudinal direction of the track. They should be considered as uniformly distributed over the corresponding influence length $L_{a,b}$ for traction and braking effects for the structural element considered. The direction of the traction and braking forces should take account of the permitted direction(s) of travel on each track.

The characteristic values of traction and braking forces should be taken as follows:

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

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

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

The characteristic values of traction and braking forces should not be multiplied by the factor Φ (see point 2.4.2.2 - page 16) or by the factor f in point 2.5.1 - page 19.

NB : For Load Models SW/0 and SW/2, traction and braking forces need only to be applied to those parts of the structure which are loaded according to Fig. 2 and Table 1.

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

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

The traction and braking forces for Load Models 71 and SW/0 should be multiplied by the factor α in accordance with the requirements of point 2.3.2.

For loaded lengths greater than 300 m, additional requirements should be specified by the commissioning party for taking into account the effects of braking.

For lines carrying special traffic (e.g. restricted to high speed passenger traffic), the traction and braking forces may be taken as equal to 25% of the sum of the axle-loads (Real Train) acting on the influence length of the action effect of the structural element considered, with a maximum value of 1 000 kN for Q_{lak} and 6 000 kN for Q_{lbk} where specified by the commissioning party.

Traction and braking forces should be combined with the corresponding vertical loads.

When the track is continuous at one or both ends of the bridge, only a proportion of the traction or braking force is transferred through the deck to the bearings, the remainder of the force being transmitted through the track where it is resisted behind the abutments. The proportion of the force transferred through the deck to the bearings should be determined by taking into account the combined response of the structure and track in accordance with *UIC Leaflet 774-3*.

In the case of a bridge carrying two or more tracks, the braking forces on one track should be considered with the traction forces on one other track.

Where two or more tracks have the same permitted direction of travel, either traction on two tracks or braking on two tracks should be taken into account.

NB : For bridges carrying two or more tracks with the same permitted direction of travel, the commissioning party may specify alternative requirements for the application of traction and braking forces.

2.6 - Other actions for railway bridges

The following actions should also be considered in the design of the structure:

- effects due to inclined decks or inclined bearing surfaces,
- longitudinal anchorage forces from stressing or destressing rails in accordance with any requirements specified for the individual project,
- longitudinal forces due to the accidental breakage of rails in accordance with any requirements specified for the individual project,
- aerodynamic and slipstream effects caused by passing trains on structures adjacent to the track as defined in *UIC Leaflet 779-1* or as specified by the commissioning party.
- load effects from catenaries and other overhead line equipment attached to the structure,
- load effects from other railway infrastructure and equipment.

The relevant national and international requirements should be applied for other actions listed in point 1.3.2 - page 6 and which are not defined in point 2 - page 11.

2.7 - Derailment

Railway structures should be designed in such a way that, in the event of a derailment, the resulting damage to the bridge (in particular overturning or the collapse of the structure as a whole) is limited to a minimum.

2.7.1 - Derailment actions from rail traffic on a railway bridge

Derailment of rail traffic on a railway bridge should be considered as an accidental design situation.

Two design situations should be considered:

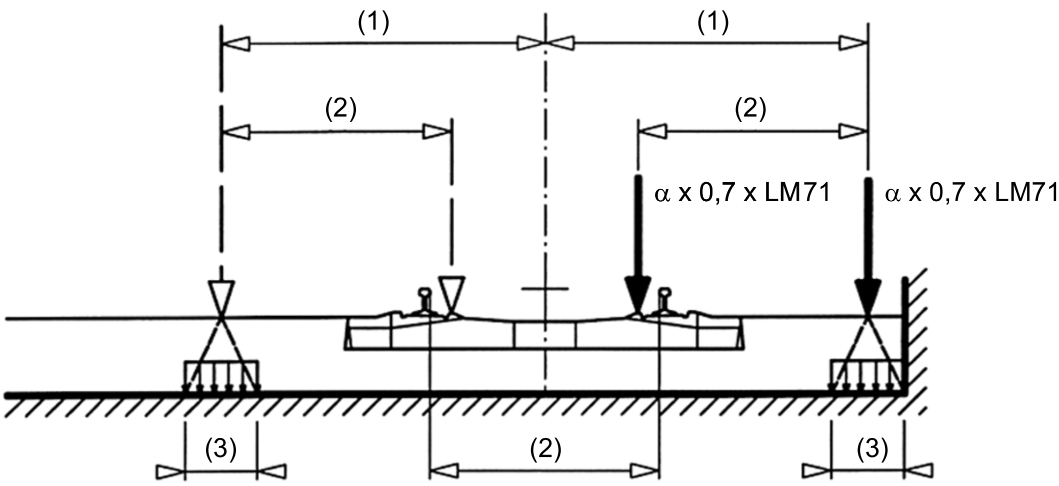
- *Design situation I:* Derailment of railway vehicles, with the derailed vehicles remaining in the track area on the bridge deck with vehicles retained by the adjacent rail or an upstand wall.

- *Design situation II:* Derailment of railway vehicles, with the derailed vehicles balanced on the edge of the bridge and loading the edge of the superstructure (excluding non-structural elements such as walkways).

NB : The commissioning party may specify additional requirements.

For design situation I, collapse of a major part of the structure should be avoided. Local damage, however, may be tolerated. The parts of the structure concerned should be designed for the following design loads in the accidental design situation:

α x 1,4 x LM71 (both point loads and uniformly distributed loading, Q_{A1d} and q_{A1d} excluding dynamic factor) parallel to the track in the most unfavourable position inside an area of width 1,5 times the track gauge on either side of the centre-line of the track.



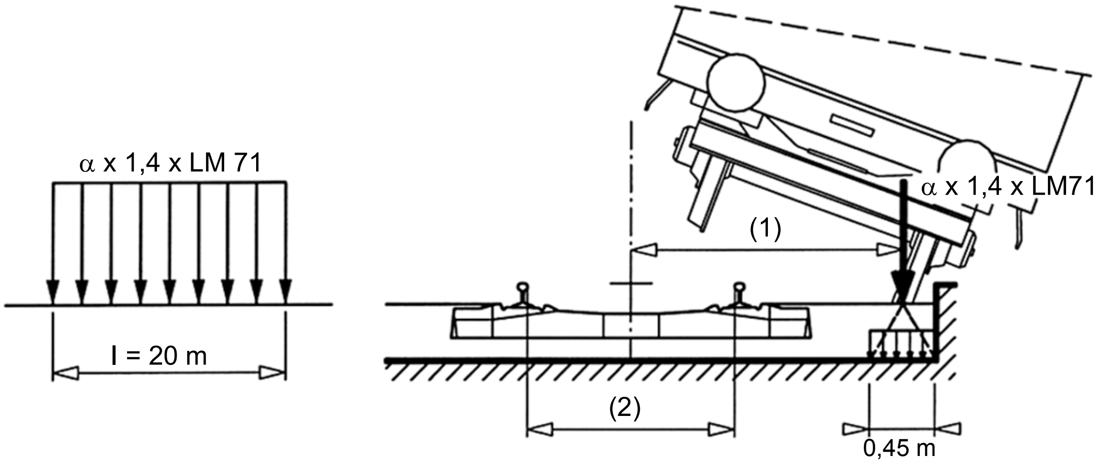
(1) Max. 1,5 s or less if against wall

(2) Track gauge s

(3) For ballasted decks, the point forces may be assumed to be distributed on a square of side 450 mm at the top of the deck

Fig. 4 - Design situation I - Equivalent load Q_{A1d} and q_{A1d}

For design situation II, the bridge should not overturn or collapse. For the determination of overall stability, a maximum total length of 20 m of $q_{A2d} = \alpha \times 1{,}4 \times LM71$ (excluding dynamic factor) should be taken as a uniformly distributed vertical line load acting on the edge of the structure under consideration.



(1) Load acting on edge of structure
(2) Track gauge s

Fig. 5 - Design situation II - Equivalent load q_{A2d}

NB : The above-mentioned equivalent load is only to be considered for determining the ultimate strength or the stability of the structure as a whole. Minor structural elements need not be designed for this load.

Design situations I and II should be examined separately. A combination of these loads need not be considered.

For design situations I and II, other rail traffic actions should be neglected for the track subjected to derailment actions.

For structural elements which are situated above the level of the rails, measures to mitigate the consequences of a derailment should be in accordance with the requirements specified by the commissioning party.

2.7.2 - Derailment under or adjacent to a structure and other actions for other accidental design situations

When a derailment occurs, there is a risk of collision between derailed vehicles and structures over or adjacent to the track. The recommendations for collision loading and other design recommendations are given in *UIC Leaflet 777-2*.

Other actions for other accidental design situations should be taken into account in accordance with the requirements specified by the commissioning party.

2.8 - Application of traffic loads on railway bridges

2.8.1 - General

The bridge should be designed for the required number and position(s) of the tracks in accordance with the track positions and tolerances specified for the individual project.

Each structure should also be designed for the greatest number of tracks geometrically and structurally possible in the least favourable position, irrespective of the position of the intended tracks taking into account the minimum spacing of tracks and structural gauge clearance requirements specified for the individual project.

The effects of all actions should be determined with the traffic loads and forces placed in the most unfavourable positions. Traffic actions which produce a relieving effect should be neglected.

For the determination of the most adverse load effects from the application of Load Model 71:

- any number of lengths of the uniformly distributed load q_{vk} should be applied to a track and up to four of the individual concentrated loads Q_{vk} should be applied once per track,

- for structures carrying two tracks, Load Model 71 should be applied to one or both tracks,

- for structures carrying three or more tracks, Load Model 71 should be applied to one or two tracks, or 0,75 times Load Model 71 to three or more of the tracks.

For the determination of the most adverse load effects from the application of Load Model SW/0:

- the loading defined in Fig. 2 - page 13 and Table 1 - page 14 should be applied once to a track,

- for structures carrying two tracks, Load Model SW/0 should be applied to one or both tracks,

- for structures carrying three or more tracks, Load Model SW/0 should be applied to one or two tracks, or 0,75 times Load Model SW/0 to three or more of the tracks.

For the determination of the most adverse load effects from the application of Load Model SW/2:

- the loading defined in Fig. 2 and Table 1 should be applied once to a track,

- for structures carrying more than one track, Load Model SW/2 should be applied to one track only with Load Model 71 or Load Model SW/0 applied to one other track as specified above.

For the determination of the most adverse load effects from the application of Load Model "unloaded train":

- any number of lengths of the uniformly distributed load q_{vk} should be applied to a track,

- generally Load Model "unloaded train" should only be considered in the design of structures carrying one track.

All continuous beam structures designed for Load Model 71 should be checked additionally for Load Model SW/0.

Where a dynamic analysis is required in accordance with *UIC Leaflet 776-2*, all bridges should also be designed for the loading from Real Trains and Load Model HSLM where required by *UIC Leaflet 776-2*. The determination of the most adverse load effects from Real Trains and the application of Load Model HSLM should be in accordance with *UIC Leaflet 776-2*.

For the verification of deformations and vibrations, the vertical loading to be applied should be in accordance with *UIC Leaflet 776-2*.

2.8.2 - Groups of loads - Characteristic values of the multicomponent action

The simultaneity of the loading defined in points 2.3 - page 12 to 2.5 - page 19 and point 2.7 - page 24 may be taken into account by considering the groups of loads defined in Table 4 - page 28. Each of these groups of loads, which are mutually exclusive, should be considered as defining a single variable characteristic action for combination with non-traffic loads. Each Group of Loads should be applied as a single variable action.

In some cases, it is necessary to consider other appropriate combinations of unfavourable individual traffic actions (see point 3.4 - page 31).

The factors given in the Table 4 should be applied to the characteristic values of the different actions considered in each group.

Where groups of loads are not taken into account, rail traffic actions shall be combined in accordance with Appendix A, Table 2, paragraph 2.2 - page 37.

Table 4 : Assessment of Groups of Loads for rail traffic
(characteristic values of the multicomponent actions)

Number of tracks on structure			Groups of loads			Vertical forces			Horizontal forces			Comment
						2.3.2/2.3.3	2.3.3	2.3.4	2.5.3	2.5.1	2.5.2	
1	2	≥ 3	Number of tracks loaded	Load group	Loaded track	LM 71[a] SW/0 [ab] HSLM [cd]	SW/2 [ae]	Unloaded train	Traction braking [a]	Centri-fugal force [a]	Nosing force [a]	
			1	gr11	T_1	1			1[f]	0,5 [f]	0,5 [f]	Max. vertical 1 with max. longitudinal
			1	gr12	T_1	1			0,5 [f]	1[f]	1[f]	Max. vertical 2 with max. transverse
			1	gr13	T_1	1[g]			1	0,5 [f]	0,5 [f]	Max. longitudinal
			1	gr14	T_1	1[g]			0,5 [f]	1	1	Max. lateral
			1	gr15	T_1			1		1[f]	1[f]	Lateral stability with "unloaded train"
			1	gr16	T_1		1		1[f]	0,5 [f]	0,5 [f]	SW/2 with max. longitudinal
			1	gr17	T_1		1		0,5 [f]	1[f]	1[f]	SW/2 with max. transverse
			2	gr21	T_1 T_2	1 1			1[f] 1[f]	0,5 [f] 0,5 [f]	0,5 [f] 0,5 [f]	Max. vertical 1 with max. longitudinal
			2	gr22	T_1 T_2	1 1			0,5 [f] 0,5 [f]	1[f] 1[f]	1[f] 1[f]	Max. vertical 2 with max. transverse
			2	gr23	T_1 T_2	1[g] 1[g]			1 1	0,5 [f] 0,5 [f]	0,5 [f] 0,5 [f]	Max. longitudinal
			2	gr24	T_1 T_2	1[g] 1[g]			0,5 [f] 0,5 [f]	1 1	1 1	Max. lateral
			2	gr26	T_1 T_2	 1	1 		1[f] 1[f]	0,5 [f] 0,5 [f]	0,5 [f] 0,5 [f]	SW/2 with max. longitudinal
			2	gr27	T_1 T_2	 1	1 		0,5 [f] 0,5 [f]	1[f] 1[f]	1[f] 1[f]	SW/2 with max. transverse
			≥ 3	gr31	T_i	0,75			0,75 [f]	0,75 [f]	0,75 [f]	Additional load case

a. All relevant factors (α, Φ, f,...) shall be taken into account.
b. SW/0 shall only be taken into account for continuous span bridges.
c. HSLM and Real Trains where required in accordance with UIC Leaflet 776-2.
d. If a dynamic analysis is required in accordance with UIC Leaflet 776-2.
e. SW/2 needs to be taken into account only if it is stipulated for the line.
f. In favourable cases these non-dominant values shall be taken equal to zero.
g. Factor may be reduced to 0,5 if favourable effect. It cannot be zero.

Dominant component action as appropriate.

To be considered in designing a structure supporting one track (Load Groups 11-17).

To be considered in designing a structure supporting two tracks (Load Groups 11-27 except 15). Each of the two tracks should be considered as either T_1 (Track one) or T_2 (Track 2).

To be considered in designing a structure supporting three or more tracks (Load Groups 11 to 31 except 15). Any one track should be taken as T_1, any other track as T_2 with all other tracks unloaded. In addition the Load Group 31 has to be considered as an additional load case where all unfavourable lengths of track T_i are loaded.

2.8.3 - Groups of loads - Other representative values of the multicomponent actions

2.8.3.1 - Frequent values of the multicomponent actions

Where groups of loads are taken into account, the same rule as in point 2.8.2 - page 27 is applicable by applying the factors given in Table 4 - page 28 for each group of loads, to the frequent values of the relevant actions considered in each group of loads.

Where groups of loads are not used rail traffic actions should be combined in accordance with Table 2, point 2.2 - page 37.

2.8.3.2 - Quasi-permanent values of the multicomponent actions

Quasi-permanent traffic actions should be taken as zero.

2.8.4 - Traffic loads for transient design situations

Traffic loads for transient design situations should be defined for the individual project.

3 - Load combinations and appropriate partial factors

3.1 - General

Generally, the design of railway bridges should be verified using the partial factor method. When using the partial factor method, it should be verified that in all relevant design situations no relevant limit state is exceeded in accordance with relevant international and national requirements.

The design of railway bridges should take into account the design situations given in point 1.2.1 - page 4 for which the design should satisfy the relevant limit state requirements given in point 1.2.5 - page 5.

For each design situation considered and relevant limit state, the individual actions for the critical load cases should be combined to produce the most adverse effects. However, actions that cannot occur simultaneously, for example due to physical reasons, should not be considered simultaneously.

3.2 - Ultimate limit state

For the ultimate limit state when considering the equilibrium of the structure, it should be verified that:

$$E_{d,dst} \leq E_{d,stb}$$

where: $E_{d,dst}$ design value of the effect of destabilising actions,

$E_{d,stb}$ design value of the effect of stabilising actions.

When considering the ultimate limit state associated with rupture or collapse of the structure or failure of the ground, etc., it should be verified that:

$$E_d \leq R_d$$

where: E_d design value of the effect of actions, e.g. internal force, moment, etc. representing the total adverse action effect,

R_d design value of the corresponding resistance of the structure.

For each critical load case, the design values of the effects of actions (E_d) should be determined by combining the values of actions that are considered to occur simultaneously.

In addition to the above, the relevant international and national requirements should be satisfied.

3.3 - Serviceability limit state

For the serviceability limit state, it should be verified that :

$$E_d \leq C_d$$

where: C_d design value of the relevant serviceability criterion,

E_d design value of the effects of actions corresponding to the serviceability criteria.

For each critical load case, the design values of the effects of actions (E_d) should be determined by combining the values of actions that are considered to occur simultaneously. Generally, the partial factor for each action may be taken as unity.

Where appropriate, characteristic, frequent and quasi permanent combinations of actions should be taken into account.

In addition to the above, the relevant international and national requirements should be satisfied.

3.4 - Combinations of actions

Actions should be combined in accordance with the requirements of the relevant international and national requirements with design values determined using appropriate partial factors. To avoid undue conservatism, an additional factor y may be used to take account *inter alia* that maximum values of an action do not occur simultaneously.

Generally for railway bridges:

- requirements for taking wind and snow loading into account with construction loading should be in accordance with the relevant international or national requirements;

- requirements for taking snow loading into account for persistent and transient Design Situations should be in accordance with the relevant international or national requirements;

- the combinations of actions to be taken into account when rail traffic actions and wind actions act simultaneously should include:

  • vertical rail traffic actions including dynamic factor, horizontal rail traffic actions and wind forces with each action being considered as the leading action of the combination of actions one at a time;

  • vertical rail traffic actions excluding dynamic factor, lateral rail traffic actions from the "unloaded train" defined in point 2.3.4 - page 14 and wind forces for checking overall stability;

- wind action should not be combined with:

  • groups of loads gr 13, gr 23 (maximum longitudinal effect);

  • groups of loads gr 16, gr 17, gr 26, gr 27 and the individual traffic action Load Model SW/2 (groups of loads containing SW/2) (see point 2.8.2 - page 27);

- no wind action greater than the smaller of F_W^{**} and $\psi_0 F_{wk}$ should be combined with traffic actions. The commissioning party may specify the maximum wind speed compatible with rail traffic for determining F_W^{**};

- actions due to aerodynamic effects of rail traffic and wind actions should be combined together. Each action should be considered individually as a leading variable action;

- if a structural member is not directly exposed to wind, the action q_{ik} due to aerodynamic effects should be determined for train speeds enhanced by the speed of the wind;

- where groups of loads are used to represent the combined load effects of rail traffic actions, the combinations of rail traffic actions defined in the groups of loads given in point 2.8.2 should be used;

- the groups of loads technique is intended to be a simplified approach describing common critical combinations of rail traffic load effects (also see Appendix C - page 41). In some situations, individual traffic actions should be considered where the group of loads technique is not conservative. For example, for the design of bearings and bearing restraints, for the assessment of maximum lateral and minimum vertical traffic loading, determining maximum overturning effects on abutments (especially for continuous bridges), etc.;

- where groups of loads are not used for rail traffic loading, rail traffic loading should be considered as a single multidirectional variable action with individual components of rail traffic actions taken as the maximum unfavourable and minimum favourable values as appropriate;

- requirements for combining actions for accidental design situations and seismic design situations should be in accordance with the relevant international or national requirements (generally only one accidental action is taken into account at any one time and excluding wind actions or snow loading. For combinations including derailment loading rail traffic actions should be taken into account as accompanying actions in the combinations with their combination value);

- the minimum coexistent favourable vertical load with centrifugal, traction or braking individual components of rail traffic actions is 0,50 LM71;

- where groups of loads are used, a unique ψ value should be applied to one of the groups of loads as defined in Tables 1 to 3 of Appendix A - page 35 with ψ taken as equal to the ψ value applicable to the leading component of the group (see Appendix D - page 45);

- in applying Tables 1 to 3 of Appendix A in cases where the limit state is very sensitive to variations in magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken into account with appropriate combinations of favourable and unfavourable actions;

- for the design of structural members subject to geotechnical actions and for other geotechnical design situations, the combinations of loading and design philosophy should be in accordance with the relevant national and international requirements;

- for bridges carrying both rail traffic and road traffic, the combination of actions to be considered should be in accordance with the requirements of the relevant authorities.

3.5 - Recommended design values, partial factors and ψ factors

The validity of the recommendations of the present point is limited to the design of railway bridges in accordance with the requirements of the Eurocodes. When using other design codes, appropriate combination of loading and appropriate factors should be used with the loading specified in this leaflet.

Design values for the load effects of loads to be taken into account in the design of railway bridges are obtained by taking appropriate combinations of actions with appropriate partial factors and ψ factors.

For persistent and transient design situations two approaches are given in *Eurocode EN 1990* (including Annex A2) for evaluating the total design effect of actions, either the approach defined in *equation 6.10 of EN 1990* or an alternative approach in *equations 6.10a and 6.10b of EN 1990:*

$$E_d = \sum_{j \geq 1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$ *(EN 1990, equation 6.10)*

or the less favourable of:

$$E_d = \sum_{j \geq 1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} \psi_{0,1} Q_{k,1} \text{ "+" } \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$ *(EN 1990, equation 6.10a)*

or:

$$E_d = \sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$ *(EN 1990, equation 6.10b)*

where:

	" + "	means "to be combined with",
	Σ	means "the combined effect of",
	ξ	is a reduction factor for unfavourable permanent actions.

Generally, the approach described in Equation 6.10 should be used unless specified otherwise by the commissioning party or relevant authority.

The partial factors and ψ factors given in Tables 1 to 3 of Appendix A - page 35 may be used in conjunction with the *Eurocodes EN 1990* and *EN 1991-2*.

For accidental design situations, the following expression is given in *EN1990* for evaluating the total design effect of actions:

$$E_d = \sum_{j \geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_d \text{ "+" } (\psi_{1,1} \text{ "or" } \psi_{2,1})\, Q_{k,1} \text{ "+" } \sum_{i > 1} \gamma_{Q,i} \psi_{2,i} Q_{k,i}$$ *(EN 1990, equation 6.11b)*

with the choice between $\psi_{1,1}$ or $\psi_{2,1}$ related to the relevant accidental design situation in accordance with the requirements of the railway operators and of the commissioning party.

NB : For example, any requirement to take LM71, etc. into account on a second track when loading corresponding derailment actions from rail traffic on the bridge is being considered.

See also *EN 1990* for the general format of the expressions for combining the effects of actions for seismic design situations.

The combination of actions for the serviceability limit states are defined in the following expressions given in *EN 1990* where all partial factors γ have been taken equal to unity:

For the characteristic combination:

$$E_d = \sum_{j \geq 1} G_{k,j} \;"+"\; P \;"+"\; Q_{k,1} \;"+"\; \sum_{i > 1} \psi_{0,i} Q_{k,i} \qquad \textit{(EN 1990, equation 6.14b)}$$

For the frequent combination:

$$E_d = \sum_{j \geq 1} G_{k,j} \;"+"\; P \;"+"\; \psi_{1,1} Q_{k,1} \;"+"\; \sum_{i > 1} \psi_{2,i} Q_{k,i} \qquad \textit{(EN 1990, equation 6.15b)}$$

For the quasi-permanent combination:

$$E_d = \sum_{j \geq 1} G_{k,j} \;"+"\; P \;"+"\; \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \qquad \textit{(EN 1990, equation 6.16b)}$$

For design situations and combinations of actions, see Tables 1 to 3 of Appendix A - page 35.

3.6 - Fatigue

Requirements for the fatigue loading of railway bridges and taking fatigue into account in the design should be in accordance with the requirements of international and national requirements.

Appendix A - Design situations and combinations of actions

General notes:

- The values hereafter are intended to be used only in conjunction with *Eurocodes EN 1990* (including Annex A2) and *EN 1991-2* using Equation 6.10 in *EN 1990*, etc. Alternative approaches using Equations 6.10 a and 6.10b using ξ are not covered although a similar table may be developed to cover this alternative approach.
- The format of the table is based on *EN 1990* (including Annex A2).
- Components of rail traffic actions are introduced as a single variable action in the combination of load effects defined in the groups of loads in Table 2 - page 37.
- The groups of loads do not cover all critical combinations for all structural elements. In some situations it is necessary to consider individual rail traffic actions (see point 3.4 - page 31).
- Where individual rail traffic actions are considered appropriate combinations of unfavourable vertical, centrifugal, nosing and traction and braking load should be taken into account.
- γ values of unity are explicitly shown for serviceability limit states.
- Only one accidental loading to be considered in a combination at any one time.
- For accidental design situations, the values of ψ to use for accompanying variable actions depend on the accident scenario being considered. The commissioning party should specify the design requirements.
- For requirements relating to construction, see relevant national and international requirements
- Key:

L	Leading variable action
A	Accompanying action
M	Main accompanying variable action
O	Other accompanying variable action

Table 1 : Permanent actions

Action			**Design situation and limit state**											
			Persistent and transient								**Accidental**		**Seismic**	
			Ultimate		**Fatigue**			**Serviceability**			**Ultimate**		**Ultimate**	
			Resistance	Static equilibrium (10)				Characteristic	Frequent	Quasi-permanent	Resistance	Static equilibrium (10)	Resistance	Static equilibrium (10)
1 Permanent actions			-	-	-			-	-	-	-	-	-	-
			$\gamma_{Gj}(\gamma_P)$	$\gamma_{Gj}\cdot(\gamma_p)$	γ_{Ff}	γ_σ	γ_k	$\gamma_{Gj}\cdot(\gamma_p)$	$\gamma_{Gj}\cdot(\gamma_p)$	$\gamma_{Gj}\cdot(\gamma_p)$	$\gamma_{GAj}(\gamma_{PA})$	$\gamma_{GAj}(\gamma_{PA})$	$\gamma_{Gj}(\gamma_P)$	$\gamma_{Gj}(\gamma_P)$
1.1 Direct actions														
Self-weight (5) (6) (8) (9)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,35 1	1,1 or 1,15 0,9 or 0,85 (3)	See National Requirements			1 1			1 1	1 1	1 1	1 1
Horizontal earth pressure (6) (8) (9) (12)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,5 1	1,1 or 1,15 0,9 or 0,85 (3)				1 1			1 1	1 0,9	1 1	1 0,9
Ballast (1) (6) (8)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,35 x 1,33 1 x 0,75	(1,1 or 1,15) x 1,33 (0,9 or 0,85) x 0,75 (3)				1,33 0,75			1,33 0,75	1,33 0,75	1,33 0,75	1,33 0,75
Movable loads (6) (7) (8)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,35 1	1,1 or 1,15 0,9 or 0,85 (3)				1 1			1 1	1 1	1 1	1 1
1.2 Indirect actions														
Settlement (9) (11) (18)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,5 0	1,35 0	See National Requirements			1 0			1 0	1 0	1 0	1 0
Differential settlement (9) (11) (18)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,5 0	1,35 0				1 0			1 0	1 0	1 0	1 0
Shrinkage and creep	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	1,5 0	- -				1 -			1 -	- -	1 -	- -
Prestress (13) (19)	Unfavourable Favourable	$\gamma_{G, sup}$ $\gamma_{G, inf}$	Use recommended values in relevant National Requirements											

see specific notes - page 38

Table 2 : Variable actions

Action	Design situation and limit state																		
	Persistent and transient													**Accidental**				**Seismic**	
	Ultime				**Fatigue**			**Serviceability (16)**						**Ultime**				**Ultime**	
	Resistance		Static equilibrium (10)					Characteristic		Frequent		Quasi-permanent (15)		Resistance		Static equilibrium (10)		Resistance	Static equilibrium (10)
2 Variable actions (30)	L	A	L	A	-			L	O	L	O	L	O	M	O	M	O	O	O
	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	γ_{Ff}	γ_{σ}	γ_k	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	$\gamma_{Q1} \cdot \psi_{11}$	$\gamma_{Qi} \cdot \psi_{2i}$	$\gamma_{Q1} \cdot \psi_{21}$	$\gamma_{Qi} \cdot \psi_{2i}$	$\gamma_{QA1} \cdot \psi_{11}$ or (4) $\gamma_{QA1} \cdot \psi_{21}$	$\gamma_{QAi} \cdot \psi_{2i}$	$\gamma_{QA1} \cdot \psi_{11}$ or (4) $\gamma_{QA1} \cdot \psi_{21}$	$\gamma_{QAi} \cdot \psi_{2i}$	$\gamma_{Qi} \cdot \psi_{2i}$	$\gamma_{Qi} \cdot \psi_{2i}$
2.1 Traffic actions: Load groups (18)																			
Load groups 11-14 (LM71, SW/0)	1,45	1,45 x 0,8	1,45	1,45 x 0,8	See National Requirements			1	1 x 0,8	1 x 0,8	0	0	0	See National Requirements				See National Requirements	
Load groups 15 (unloaded train)	-	-	-	1,0 x 1,0				-	-	-	-	-	-						
Load groups 16-17 (SW/2)	1,2	-	1,2	-				1	-	1 x 0,8	-	0	-						
Load groups 21-24 (LM71, SW/0)	1,45	1,45 x 0,8	1,45	1,45 x 0,8				1	1 x 0,8	1 x 0,7	0	0	0						
Load groups 26-27 (LM71, SW/2) (14)	1,45/ 1,20	-	1,45/ 1,20	-				1	1 x 0,8	1 x 0,7	0	0	0						
Load groups 31 (LM71, SW/0)	1,45	1,45 x 0,8	1,45	1,45 x 0,8				1	1 x 0,8	1 x 0,6	0	0	0						
2.2 Traffic actions: Individual actions, etc. (2) (18)																			
LM71, SW/0	1,45	1,45 x 0,8	1,45	1,45 x 0,8	See National Requirements			1	1 x 0,8	1 x 0,8 (13)	0	0	0	See National Requirements				See National Requirements	
Unloaded train	-	-	-	1,0 x 1,0				-	-	-	-	-	-						
SW/2	1,2	-	1,2	-				1	-	1 x 0,8	0	0	0						
Load model HSLM (17)	1,45	1,45 x 0,8	1,45	1,45 x 0,8				1	1 x 0,8	1 x 0,8	0	0	0						
Load effects from Real Trains	1,45	1,45 x 0,8	1,45	1,45 x 0,8				1	1 x 0,8	1 x 0,8	0	0	0						
Fatigue traffic actions	-	-	-	-				-	-	-	-	-	-						
Traffic load surcharge horizontal earth pressure	1,45	1,45 x 0,8	1,45	1,45 x 0,8				1	1 x 0,8	1 x 0,8	0	0	0						
Aerodynamic actions (20)	1,5	1,5 x 0,8	1,5	1,5 x 0,8				1	1 x 0,8	1 x 0,5	0	0	0						
2.3 Other variable actions																			
General loading on non-public footpaths	1,5	1,5 x 0,8	1,5	1,5 x 0,8	See National Requirements			1	1 x 0,8	1 x 0,5	0	0	0	See National Requirements				See National Requirements	
Other operating actions	1,5	1,5 x 0,8	1,5	1,5 x 0,8				1	1 x 0,8	1 x 0,5	0	0	0						
Natural actions - Wind F_{wk} or F_{wn} (20) (22)	1,5	1,5 x 0,6	1,5	1,5 x 0,6				1	1 x 0,6	1 x 0,5	0	0	0						
- Wind F_w^{**} (20)	1,5	1,5 x 1,0	1,5	1,5 x 1,0				1	1 x 1	0	0	0	0						
- Thermal (21)	1,5	1,5 x 0,6	1,5	1,5 x 0,6				1	1 x 0,6	1 x 0,6	1 x 0,5	1 x 0,5	1 x 0,5						
- Hydraulic	1,5	1,5 x 1,0	1,5	1,5 x 1,0				1	1 x 1	1 x 1	1 x 1	1 x 1	1 x 1						
- Snow and ice	See National Requirements																		

Table 3 : Accidental and seismic actions

Action	Design situation and limit state																
	Persistent and transient													**Accidental**		**Seismic**	
	Ultimate				**Fatigue**			**Serviceability**						**Ultimate**		**Ultimate**	
	Resistance		Static equilibrium (10)					Characteristic		Frequent		Quasi-permanent		Resistance	Static equilibrium (10)	Resistance	Static equilibrium (10)
3 Accidental and seismic actions	L	O	L	O	-			L	O	L	O	L	O	L	L	L	L
	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	γ_{Ff}	γ_{σ}	γ_k	γ_{Q1}	$\gamma_{Qi} \cdot \psi_{0i}$	$\gamma_{Q1} \cdot \psi_{11}$	$\gamma_{Qi} \cdot \psi_{2i}$	$\gamma_{Q1} \cdot \psi_{21}$	$\gamma_{Qi} \cdot \psi_{2i}$	γ_A	γ_A	γ_I	γ_I
3.1 Accidental actions																	
Derailment loading (see point 2.7.1 - page 24)	Not applicable													1	1		
Other accidental rail loading (see point 2.7.2 - page 25)	Not applicable													1	1		
Other accidental actions on rail carrying structures including impact from road traffic, ship impact, etc. See point 2.7.2 and *EN 1991-1-7*	Not applicable													1	1		
3.2 Seismic actions																	
Seismic actions	Not applicable															1	1

(1) The factors 1,33/ 0,75 are allowances for variation in ballast depth. See point 1.4 - page 9 and *EN 1991-1-1*. The density of ballast should also be in accordance with *EN 1991-1-1*.

(2) For associated traction and braking, centrifugal forces, interaction forces due to deflection under traffic vertical loads, etc. the ψ values are to be taken as the ψ factors specified for the associated vertical loads.

(3) The factors $\gamma_{G,sup}/\gamma_{G,inf} = 1{,}1/0{,}9$ should be increased to 1,15/0,85 where loss of equilibrium could result in multiple fatalities. Where verification of static equilibrium involves the resistance of structural members (for example where loss of equilibrium is prevented by holding down ties) an additional check should be carried out considering structural resistance at the ultimate limit state, etc.

(4) Depending upon accidental design situation. See point 3.5 - page 33 and *National Annex*. The main variable action should be taken with its frequent value.

(5) Structural and non-structural elements including soil, ground water and free water.

(6) In this verification, the characteristic values of all permanent actions from one source are multiplied by 1,35 if the total resulting action effect is unfavourable and by 1,0 if the total resulting action is favourable. See also *EN 1990*.

(7 $\gamma_{G,inf} = 0$ should also be considered where the effect is favourable.

(8) In cases where 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 *EN 1990*

(9) All soil actions including lateral earth pressure effects, settlement and actions of ground water should be calculated in accordance with *EN 1997*.

(10) General equilibrium of earthworks is not included in this table. See *EN 1997*.

(11) Settlement predictions to be a best estimate prediction in accordance with *EN 1990*.

(12) Horizontal earth pressure from soil, ground water, free water and ballast. See *EN 1990* and *EN 1997*.

(13) 0,8/0,7/0,6 for 1, 2 or 3 (or more) tracks.

(14) SW/2 applied to any one track. LM71 or SW/0 applied to other track. Take $\gamma_{Q1} = 1{,}45$ for contribution from LM71, and $\gamma_{Q1} = 1{,}2$ for contribution from SW/2.

(15) For quasi-permanent traffic actions, ψ_{2i} taken as 0,0. For special cases such as terminal tracks and freight sidings, ψ_{2i} should be taken as 0,8.

(16) If deflection is being considered, see point 2.8.1 - page 26 and *UIC Leaflet 776-2*. ψ_2 should be taken as 1,0.

(17) Generally HSLM is applied to one track only with/without "LM71+SW/0" applied to other track(s). See *UIC Leaflet 776-2*.

(18) Favourable values of traffic actions and settlement/ differential settlement should be taken as zero (except for consideration of rail traffic actions where vertical effect are favourable and horizontal effects unfavourable then 0,5 times the vertical effects of LM71+SW/0 should be taken as coexistent with full horizontal rail traffic actions. Or both vertical and horizontal rail traffic actions taken as zero).

(19) See design Eurocodes for values of γ for imposed deformations.

(20) Where lightweight flexible members susceptible to fatigue damage from vibration arising from aerodynamic or wind loading special studies are required.

(21) See *EN 1991-1-5*

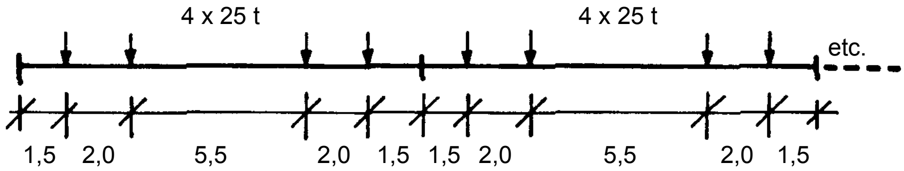
(22) Whenever wind action is required to be considered with traffic, the wind action $\psi_0 F_{Wk}$ or $\psi_0 F_{Wn}$ should be taken as no greater than F_W^{**} , see *EN 1991-2.4*.
F_W^{**} should be calculated with the maximum wind speed compatible with railway traffic as specified by the commissioning party.

Appendix B - Determination of Load Models

Table 4 : Different service trains, used for determining the UIC Load Model 71

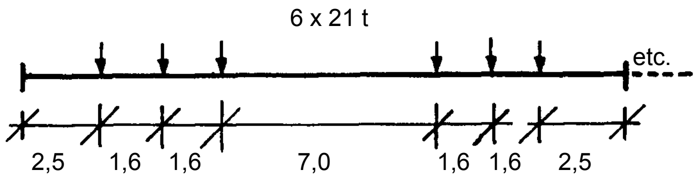
Wagons for V = 120 km/h

1



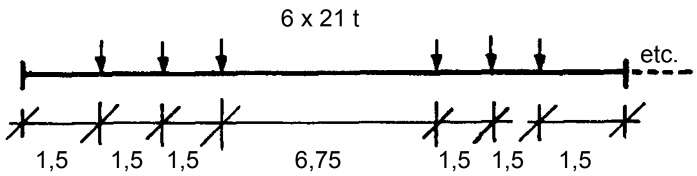
2 CC locomotives for V = 120 km/h

2



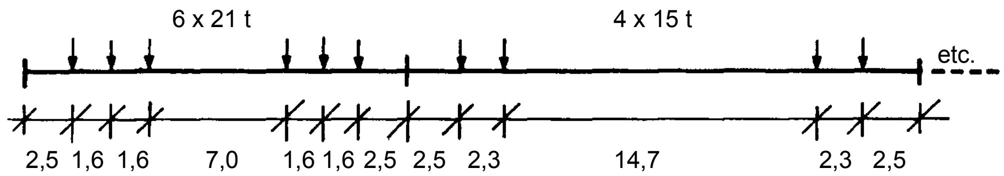
Wagons for V = 120 km/h

3



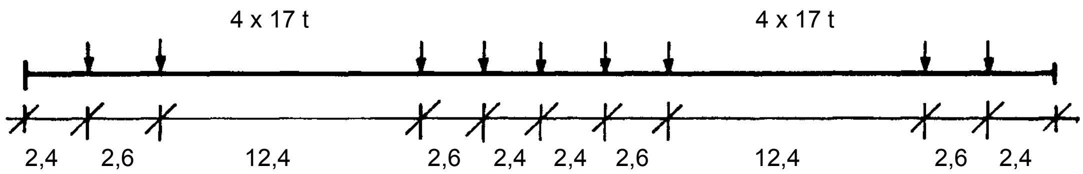
Passenger trains for V = 250 km/h

4



Turbotrain for V = 300 km/h

5



Special vehicles for V = 80 km/h

6

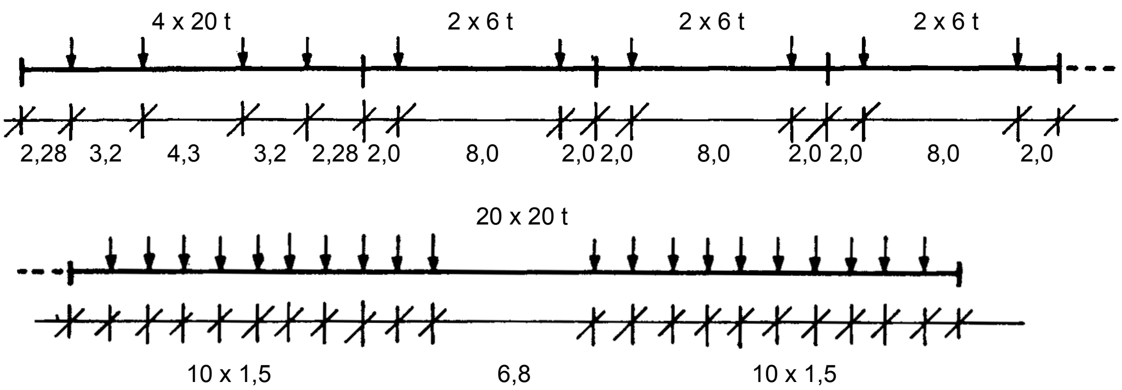
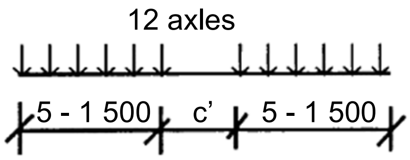
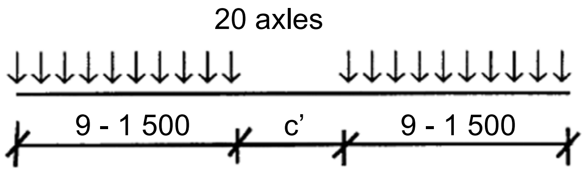
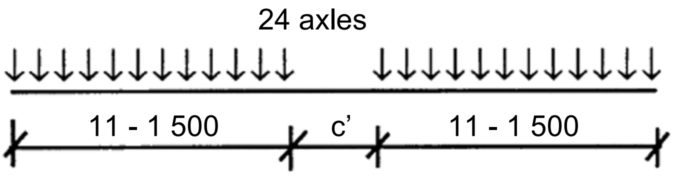
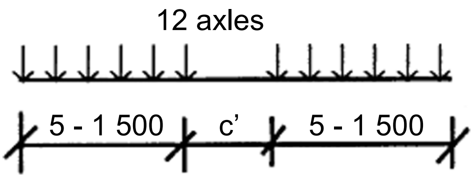
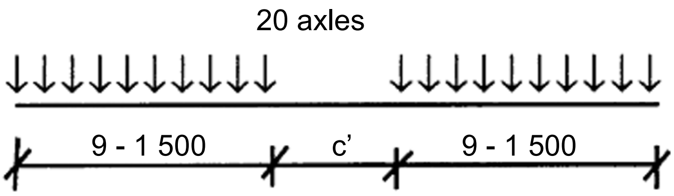
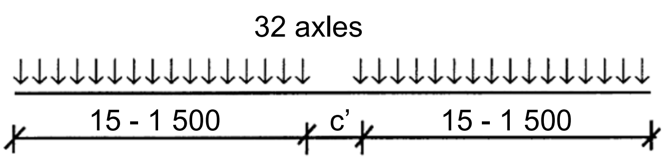


Table 5 : Allocation of heavy wagons to load classifications

Load classifi-cations	Diagram of heavy wagons	Axle-loads (in tonnes)	c' (m)	No.
SW/0		20 22,5	≥ 3,0 ≥ 6,0	4 5
		20	≥ 6,8	6
		19	≥ 9,0	7
SW/2		17 19	≥ 3,0 ≥ 6,0	1 2
		17	≥ 5,0	3
SW/2		22,5	≥ 8,5	10

Appendix C - Dynamic factors for Real Trains

ORE Specialists' Committee D23 provided the basis for determining the dynamic factors. Its work was supplemented by model tests and theoretical studies, especially in those areas which were not covered by line tests. The accuracy of the results of the theoretical studies was confirmed by tests (*ORE Report D 128/RP 3* - see Bibliography page 46).

The laws were deduced from the behaviour of a simply supported beam. They cover most of the effects in continuous girders and other structures; where this is not the case, they are taken into account by the values given for L_Φ .

When service trains pass over a bridge, the resulting oscillations increase the load by a quantity φ made up of two components as follows:

φ' is the proportion applicable for a track in perfect geometrical condition

φ'' is the proportion representing the effects of vertical track irregularities

$$\varphi = \varphi' + \varphi''$$

The value φ' is given by the following formula:

$$\varphi' = \frac{K}{1 - K + K^4} \quad (1)$$

in which:

$$K = \frac{V}{2 \bullet n_o \bullet L} \quad (2)$$

The following formula was established on the basis of theoretical studies to take account of track irregularities:

$$\varphi'' = \frac{a}{100} \bullet \left[56 \bullet e^{\frac{L^2}{100}} + 50 \bullet \left(\frac{n_o L}{80} - 1 \right) \bullet e^{\frac{L^2}{400}} \right] \quad (3)$$

In these formulae:

v speed in m/s

L in the case of a main beam with 2 bearings: span in m;

in other cases, the value L_Φ in Table 2 - page 17 should be used instead of L in the calculation. This also applies to the assessment of old bridges if service trains are used as live loads

n_o	natural frequency of the unloaded bridge (S^{-1})
e	base of natural logarithms (2,71828 ...)
$a = \frac{v}{22}$	for speeds up to 22 m/s (approx. 80 km/h)
a = 1,0	for speeds above 22 m/s.

The term φ' in formula (1) covers about 95% of the values studied, giving a statistical confidence limit of 95% (approximately mean value plus two standard deviations).

The term φ'' in formula (3) has been fixed by assuming a vertical dip in the track of 2 mm over a length of 1 m or 6 mm over a length of 3 m, and an unsprung mass of 2 t per axle.

The formulae given represent upper bounds which may, however, be exceeded by at the most 30% in particular cases, such as very high speed trains or long wheelbase vehicles, while only half these values are reached in the case of special vehicles with closely spaced axles.

Generally speaking, these effects are not predominant - but they should be taken into account when calculating bridges for the acceptance of actual trains. It is particularly important to take this fact into account for short span bridges.

The dynamic factors for the UIC loading are calculated from the increase in loads φ for the chosen service trains, so that the loads in the UIC loading multiplied by Φ (total load) cover the loads of actual trains multiplied by (1 + φ) with sufficient safety.

The values $\varphi = \varphi' + \varphi''$ have been calculated for bridges with high and low natural frequencies, taking the most unfavourable values. The upper and lower limits of the used frequencies are shown in *UIC Leaflet 776-2, Fig. 11*.

The limit of validity for φ' is the lower limit of natural frequency. For all other cases, φ' should be determined by a dynamic analysis in accordance with *UIC Leaflet 776-2*.

The limit of validity for φ'' is the upper limit of natural frequency. For all other cases, φ'' may be determined by a dynamic analysis taking into account mass interaction between the unsprung axle masses of the train and the bridge in accordance with *UIC Leaflet 776-2*.

The values of $\varphi' + \varphi''$ should be determined using upper and lower limiting values of n_o, unless it is being made for a particular bridge of known first natural frequency.

The upper limit of n_o is given by:

$$n_o = 94,76\ L_\Phi^{-0,748}$$

and the lower limit is given by:

$$n_o = \frac{80}{L_\Phi} \qquad \text{for } 4\ \text{m} \le L_\Phi \le 20\ \text{m}$$

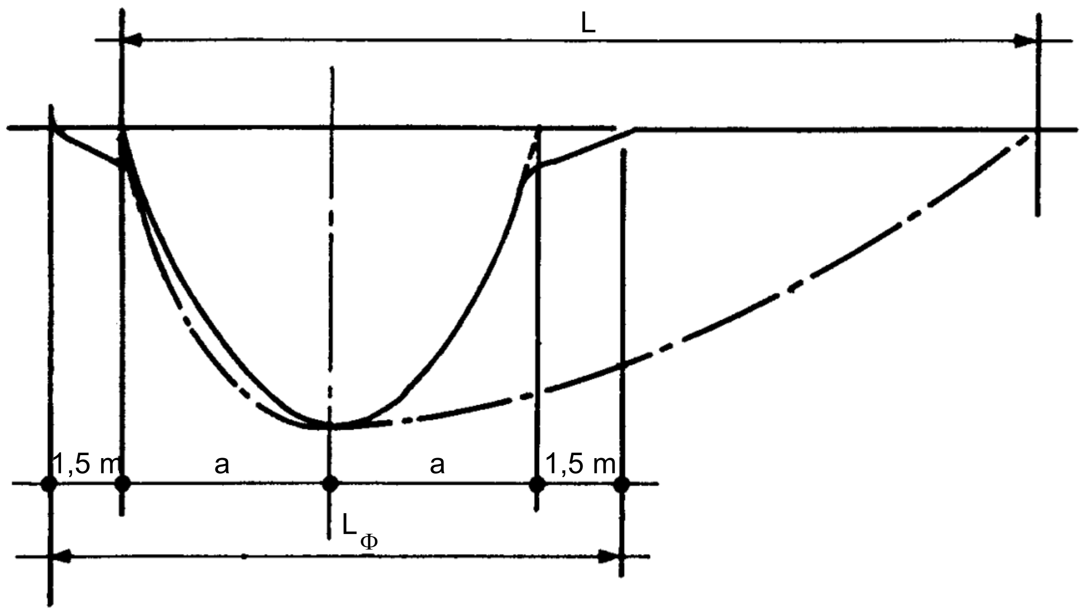
$$n_o = 23{,}58\ L_\Phi^{-0,592} \qquad \text{for } 20\ \text{m} \le L_\Phi \le 100\ \text{m}$$

Damping was taken to correspond to logarithmic decrements from 0,0 to 1,0.

Service trains have been divided into six representative types for which standard speeds have been set. These six types of service trains are given in Appendix B - page 39. The maximum loadings in relation to span were obtained for three of the six standard trains. However, the effects of all six standard trains should be taken into account for checking purposes.

The values of L_Φ were based on the influence line for the deflection of the member to which the calculations refer. In the case of asymmetrical influence lines, the following formula is applied:

$$L_\Phi = 2.\ (a + 1{,}5) \quad [\text{m}]$$



The definition of $L_\Phi = 2 \cdot (a + 1{,}5)$ is based on the assumption that a structure with a symmetrical influence line and the same maximum value will produce the same dynamic effect. This follows from the fact that the dynamic effects depend on the slope of the influence line at the bearing.

To allow for the effect of distribution of the load by the rails, the value is increased by $2 \times 1{,}50 = 3{,}00$ m.

In assessing existing bridges, formula (1) to (3) can be used to determine dynamic factors.

When assessing the strength of old lattice girder bridges, account must be taken of the fact that secondary vibrations occur in flexible diagonals (formed of flats) which result in stress increases at the extreme fibres. To allow for this, it is recommended that a stress of 5 N/mm^2 for speeds of V < 50 km/h and a stress of 10 N/mm^2 for higher speeds be added to the stresses calculated for the live load and the dynamic effect.

For special trains with a large number of axles and a total weight of more than 400 t, a dynamic increment φ of 0,15 to 0,10 may be added if more accurate calculations are not carried out and if such trains travel at speeds of 40 km/h or less.

The dynamic factors $1 + \varphi$ are also used for fatigue damage calculations.

The static load due to a Real Train at v [m/s] should be multiplied by:

either,	$1 + \varphi = 1 + \varphi' + \varphi''$	for track with standard maintenance
or,	$1 + \varphi = 1 + \varphi' + 0{,}5\ \varphi''$	for carefully maintained track.

Appendix D - Description of groups of loads

As stated in point 2.8.2 - page 27, the simultaneity of the loading systems described in points 2.3 - page 12 to 2.5 - page 19 is taken into account by considering the groups of loads defined in Table 4 - page 28. Each of these groups of loads, which are mutually exclusive, should be considered as defining a single **characteristic** action for combination with non-traffic loads.

That means:

1. A group of loads is a multi component traffic action with a characteristic value defined in Table 4.

2. In each group of loads, one component is considered as dominant, other components as accompanying. For the assessment of the characteristic value of this group of loads, the dominant component action is taken into account with its **full** characteristic value, the other accompanying component actions with generally reduced values.

3. For defining **other** representative values of the multicomponent action (group of loads) defined in Table 4, all values given to the different components in a group have to be multiplied by the same value of factor ψ (ψ_0, ψ_1 or ψ_2, depending on the representative value to be obtained). This representative value will, when necessary, be taken into account with other actions in the considered combinations (values of ψ to be considered for groups of loads are given in Tables 1 to 3 of Appendix A - page 35).

4. All values given to the different components in a group are multiplied by the same value of partial factor γ_Q for verification at ULS.

5. The values of ψ and γ_Q to be used correspond to the values to be used for the component considered as dominant in the group when the dominant component is considered alone.

6. If two components are designated as dominant in the same group, for simplification purpose, it is the most unfavourable of the two values of ψ (and/or of γ_Q) which should be used for the whole group (if these are not identical).

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Printed by the International Union of Railways (UIC)
16, rue Jean Rey 75015 Paris - France, August 2006
Dépôt Légal August 2006

ISBN 2-7461-0902-6 (French version)
ISBN 2-7461-0903-4 (German version)
ISBN 2-7461-0904-2 (English version)