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English Version

EN 1990 – EUROCODE : BASIS OF STRUCTURAL DESIGN

ANNEX A2 : Application for bridges (Normative)

EN 1990 – Eurocode : Bases de calcul des structures
Annexe A2 : Application aux ponts (normative)

EN 1990 – Eurocode : Grundlagen der Tragwerksplanung
Anhang A2 : Anwendung bei Brücken (Normativ)

FINAL PT DRAFT
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Annex A2
(normative)
Application for Bridges

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A2.1 Field of application

(1) This Annex A2 to EN 1990 gives rules and methods for establishing combinations of actions for serviceability and ultimate limit state verifications (except fatigue verifications) with the recommended design values of permanent, variable and accidental actions and ψ factors to be used in the design of road bridges, footbridges and railway bridges. It also applies to actions during execution. Methods and rules for verifications relating to some material-independent serviceability limit states are also given.

NOTE 1 Symbols, notations, Load Models and groups of loads are those used or defined in the relevant section of EN 1991-2.

NOTE 2 Symbols, notations and models of construction loads are those defined in EN 1991-1-6.

NOTE 3 Guidance may be given in the National Annex with regard to the use of Table 2.1 (design working life).

NOTE 4 Most of the combination rules defined in clauses A2.2.2 to A2.2.5 are simplifications intended to avoid needlessly complicated calculations. They may be altered as appropriate in the National Annex or for the individual project as described in A2.2.1 to A2.2.5.

NOTE 5 : This annex A2 to EN 1990 does not include rules for the determination of actions on structural bearings (forces and moments) and associated movements of bearings or give rules for the analysis of bridges involving ground-structure interaction that may depend on movements or deformations of structural bearings. For the calculation of data for procuring bearings, and also for expansion joints, see Annex E to EN 1990.

(2)The rules given in this Annex A2 to EN 1990 do not apply to :

- bridges that are not covered by EN 1991-2 (for example bridges under an airport runway, mechanically - moveable bridges, roofed bridges, bridges carrying water, etc.),
- bridges carrying both road and rail traffic, and
- other civil engineering structures carrying traffic loads (for example backfill behind a retaining wall),

A2.2 Combinations of actions

A2.2.1 General

(1) Effects of actions that cannot occur simultaneously due to physical or functional reasons should not be considered together in combinations of actions.

(2) Combinations involving actions which are outside the scope of EN 1991 (*e.g.* due to mining subsidence, particular wind effects, water, floating debris, flooding, mud slides, avalanches, fire and ice pressure) should be defined in accordance with EN 1990, 1.1(3).

NOTE 1 Combinations involving actions that are outside the scope of EN 1991 may be defined either in the National Annex or for the individual project.

NOTE 2 For seismic actions, see EN 1998.

NOTE 3 For water actions exerted by currents and debris effects, see also EN 1991-1-6.

(3) The combinations of actions given in expressions 6.9a to 6.12b should be used when verifying ultimate limit states.

NOTE Expressions 6.9a to 6.12b are not for the verification of the limit states due to fatigue. For fatigue verifications, see EN 1991 to EN 1999.

(4) The combinations of actions given in expressions 6.14a to 6.16b should be used when verifying serviceability limit states. Additional rules are given in A2.4 for verifications regarding deformations and vibrations.

(5) Where relevant, variable traffic actions should be taken into account simultaneously with each other in accordance with the relevant sections of EN 1991-2.

(6)P During execution the relevant design situations shall be taken into account.

(7)P The relevant design situations shall be taken into account where a bridge is brought into use in stages.

(8) Where relevant, particular construction loads should be taken into account simultaneously in the appropriate combination of actions.

NOTE Where control measures agreed for the individual project are taken, particular construction loads may not need to be taken into account simultaneously in the design.

(9)P For any combination of variable traffic actions with other variable actions specified in other Parts of EN 1991, any group of loads, as defined in EN 1991-2 shall be taken into account as one variable action.

(10) Snow loads and wind actions should not be considered simultaneously with loads arising from construction activity Q_{ca} (*i.e.* loads due to working personnel).

NOTE Requirements for snow loads and wind actions to be taken into account simultaneously with other construction loads (*e.g.* actions due to heavy equipment or cranes) during some transient design situations may have to be agreed for the individual project. See also EN 1991-1-3, 1-4, 1-6.

(11) Where relevant, thermal and water actions should be considered simultaneously with construction loads. Where relevant the various parameters governing water actions and components of thermal actions should be taken into account when identifying appropriate combinations with construction loads.

(12) The inclusion of prestressing actions in combinations of actions should be in accordance with A2.3.1(8) and EN 1992 to EN 1999.

(13) Uneven settlements on the structure due to soil subsidence should be classified as a permanent action, G_{set}, and included in combinations of actions for ultimate and serviceability limit state verifications of the structure. G_{set} should be represented by a set of values corresponding to differences (compared to a reference level) of settlements between individual foundations or parts of foundation, $d_{set,i}$ (*i* number of the individual foundation or part of foundation).

NOTE 1 Settlements are mainly caused by permanent loads and backfill. Variable actions may have to be taken into account for some individual projects.

NOTE 2 Settlements vary monotonically (in the same direction) with time and need be taken into account from the time they give rise to effects in the structure (*i.e.* after the structure, or a part of it, becomes statically indeterminate). In addition, in the case of a concrete structure or a structure with concrete elements, there may be an interaction between the development of settlements and creep of concrete members.

(14) Effects of uneven settlements should be taken into account if they are considered significant compared to the effects from direct actions.

(15) The differences of settlements of individual foundations or parts of foundation, $d_{set,i}$, should be taken into account as best-estimate predicted values in accordance with EN 1997 with due regard for the construction process of the structure.

NOTE Methods for the assessment of settlements are given in EN 1997

(16) Where the structure is very sensitive to uneven settlements, uncertainty in the assessment of these settlements should be taken into account.

(17) In the absence of control measures, the permanent action representing settlements should be determined as follows :

- the best-estimate predicted values $d_{set,i}$ are assigned to all individual foundations or parts of foundation,
- two individual foundations or parts of an individual foundation, selected in order to obtain the most unfavourable effect, are subject to a settlement $d_{set,i} \pm \Delta d_{set,i}$

where $\Delta d_{set,i}$ takes account of uncertainties attached to the assessment of settlements.

A2.2.2 Specific combination rules for road bridges

(1) The infrequent values of variable actions may be used for certain serviceability limit states of concrete bridges.

NOTE The National Annex may refer to the infrequent combination of actions. The expression of this combination of actions is :

$$E_d = E\{G_{k,j} \,;\, P \,;\, \psi_{1,\text{infq}} Q_{k,1} \,;\, \psi_{1,i} Q_{k,i}\} \quad j \geq 1 \,;\, i > 1 \qquad \text{(A2.1a)}$$

in which the combination of actions in brackets { } may be expressed as :

$$\sum_{j \geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,\text{infq}} Q_{\text{k},1} \text{"+"} \sum_{i>1} \psi_{1,i} Q_{\text{k,i}} \qquad \text{(A2.1b)}$$

(2) Load Model 2 (or associated group of loads gr1b) and the concentrated load Q_{fwk} (see 5.3.2.2 in EN 1991-2) on footways should not be combined with any other variable non-traffic action.

(3) Neither snow loads nor wind actions should be combined with :

– braking and acceleration forces on road bridges or the centrifugal forces or the associated group of loads gr2,
– loads on footways and cycle tracks or with the associated group of loads gr3,
– crowd loading on road bridges (Load Model 4) or the associated group of loads gr4.

NOTE The combination rules for special vehicles (see EN 1991-2, Annex A, Informative) with normal traffic (covered by LM1 and LM2) and other variable actions may be referenced as appropriate in the National Annex or agreed for the individual project.

(4) Snow loads should not be combined with Load Models 1 and 2 or with the associated groups of loads gr1a and gr1b unless otherwise specified for particular geographical areas.

NOTE Geographical areas where snow loads may have to be combined with groups of loads gr1a and gr1b in combinations of actions (*e.g.* for certain roofed bridges) may be specified in the National Annex.

(5) No wind action greater than the smaller of F_W^* and $\psi_0 F_{Wk}$ should be combined with Load Model 1 or with the associated group of loads gr1a.

NOTE For wind actions, see EN1991-1-4.

(6) Wind actions and thermal actions should not be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Depending upon the local climatic conditions a different simultaneity rule for wind and thermal actions may be defined either in the National Annex or for the individual project.

A2.2.3 Specific combination rules for footbridges

(1) The concentrated load Q_{fwk} should not be combined with any other variable actions that are not due to traffic.

(2) Wind actions and thermal actions should not be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Depending upon the local climatic conditions a different simultaneity rule for wind and thermal actions may be defined either in the National Annex or for the individual project.

(3) Snow loads should not be combined with groups of loads gr1 and gr2 for footbridges unless otherwise specified for particular geographical areas and certain types of footbridges.

NOTE Geographical areas, and certain types of footbridges (e.g. roofed bridges), where snow loads may have to be combined with groups of loads gr1 and gr2 in combinations of actions may be specified in the National Annex.

(4) For footbridges on which pedestrian and cycle traffic is fully protected from all types of bad weather, specific combinations of actions should be defined.

NOTE Such combinations of actions may be given as appropriate in the National Annex or agreed for the individual project. Combinations of actions similar to those for buildings (see Annex A1), the imposed loads being replaced by the relevant group of loads and the ψ factors for traffic actions being in accordance with Table A2.2, are recommended.

A2.2.4 Specific combination rules for railway bridges

(1) Snow loads should not be taken into account in any combination for persistent design situations nor for any transient design situation after the completion of the bridge unless otherwise specified for particular geographical areas and certain types of railway bridges.

NOTE Geographical areas, and certain types of railway bridges (e.g. roofed bridges), where snow loads may have to be taken into account in combinations of actions may be specified in the National Annex.

(2) The combinations of actions to be taken into account when 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 EN 1991-2 (6.3.4) without any dynamic factor and wind forces for checking overall stability.

(3) Wind action should not be combined with :

- groups of loads gr 13, gr 23 ;
- groups of loads gr 16, gr 17, gr 26, gr 27 and Load Model SW/2 (see EN 1991-2, 6.3.3).

(4) No wind action greater than the smaller of F_W^{**} and $\psi_0 F_{Wk}$ should be combined with traffic actions.

NOTE The National Annex may give the limits of the maximum wind speed(s) compatible with rail traffic for determining F_W^{**}.

(5) Actions due to aerodynamic effects of rail traffic (see EN 1991-2, 6.6) and wind actions should be combined together. Each action should be considered individually as a leading variable action.

(6) 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.

(7) Where groups of loads are not used for rail traffic loading, rail traffic loading should be considered as a single multi – directional variable action with individual components of rail traffic actions taken as the maximum unfavourable and minimum favourable values as appropriate.

A2.2.5 Combinations of actions for accidental (non – seismic) design situations

(1) Where an action for an accidental design situation needs to be taken into account, no other accidental action or wind action or snow load should be taken into account in the same combination.

(2) For an accidental design situation concerning impact from traffic (road or rail traffic) under the bridge, the loads due to the traffic on the bridge should be taken into account in the combinations as accompanying actions with their frequent value.

NOTE 1 For actions due to impact from traffic, see EN 1991-2 and EN 1991-1-7.

NOTE 2 Additional combinations of actions for other accidental design situations (*e.g.* combination of road or rail traffic actions with avalanche, flood or scour effects) may be agreed for the individual project.

NOTE 3 Also see (1) to table A2.1.

(3) For railway bridges, when an accidental design situation concerning actions caused by a derailed train on the bridge, rail traffic actions on any other track(s) should be taken into account as accompanying actions in the combinations with their combination value.

NOTE 1 For actions due to impact from traffic, see EN 1991-2 and EN 1991-1-7.

NOTE 2 Actions for accidental design situations due to impact from rail traffic running on the bridge including derailment actions are specified in EN1991-2, 6.7.1.

(4) Accidental design situations involving ship collisions against bridge piers should be identified

NOTE These design situations may be defined for the individual project. See EN 1991-1-7.

A2.2.6 Values of ψ factors

(1) Values of ψ factors should be specified.

NOTE 1 The ψ values may be set by the National Annex. Recommended values of ψ factors for the groups of traffic loads and the more common other actions are given in :
- Table A2.1 for road bridges,
- Table A2.2 for footbridges, and
- Table A2.3 for railway bridges, both for groups of loads and individual components of traffic actions.

Table A2.1 – Recommended values of ψ factors for road bridges

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads (see EN 1991-2, Table 4.4)	gr1a (LM1+pedestrian or cycle-track loads) [1)]	TS	0,75	0,75	0
		UDL	0,40	0,40	0
		Pedestrian+cycle-track loads [2)]	0,40	0,40	0
	gr1b (Single axle)		0	0,75	0
	gr2 (Horizontal Forces)		0	0	0
	gr3 (Pedestrian loads)		0	0	0
	gr4 (LM4 – Crowd loading))		0	0,75	0
	gr5 (LM3 – Special vehicles))		0	0	0
Wind forces	F_{Wk} - Persistent design situations - Execution		 0,6 0,8	 0,2 -	 0 0
	F_W^*		1,0	-	-
Thermal actions	T_k		0,6[3)]	0,6	0,5
Snow loads	$Q_{Sn,k}$ (during execution)		0,8	-	-
Construction loads	Q_c		1,0		1,0

1) The recommended values of ψ_0 , ψ_1 , ψ_2 for gr1a and gr1b are given for roads with traffic corresponding to adjusting factors α_{Qi} , α_{qi} , α_{qr} and β_Q equal to 1. Those relating to UDL correspond to the most common traffic scenarios, in which an accumulation of lorries can occur, but not frequently. Other values may be envisaged for other classes of routes, or of expected traffic, related to the choice of the corresponding α factors. For example, a value of ψ_2 other than zero may be envisaged for the UDL system of LM1 only, for bridges supporting a severe continuous traffic. See also EN 1998.

2) The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a "reduced" value. ψ_0 and ψ_1 factors are applicable to this value.

3) The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

NOTE 2 When the National Annex refers to the infrequent combination of actions for some serviceability limit states of concrete bridges, the National Annex may define the values of $\psi_{1,infq}$. The recommended values of $\psi_{1,infq}$ are :

- 0,80 for gr1a (LM1), gr1b (LM2), gr3 (pedestrian loads), gr4 (LM4, crowd loading) and T (thermal actions) ;
- 0,60 for F_W in persistent design situations
- 1,00 in other cases (*i.e.* the characteristic value is substituted for the infrequent value)

NOTE 3 The characteristic values of wind actions and snow loads during execution are defined in EN 1991-1-6. Where relevant, representative values of water forces (F_{wa}) may be defined for the individual project.

Table A2.2 – Recommended values of ψ factors for footbridges

Action	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads	gr1	0,40	0,40	0
	Q_{fwk}	0	0	0
	gr2	0	0	0
Wind forces	F_{Wk}	0,3	0,2	0
Thermal actions	T_k	0,6[(1)]	0,6	0,5
Snow loads	$Q_{Sn,k}$ (during execution)	0,8	-	0
Construction loads	Q_c	1,0		1,0

1) The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

NOTE 4 For footbridges, the infrequent value of variable actions is not relevant.

Table A2.3 – Recommended values of ψ factors for railway bridges

<table>
<tr><th colspan="3">Actions</th><th>ψ_0</th><th>ψ_1</th><th>ψ_2 4)</th></tr>
<tr><td rowspan="3">Individual components of traffic actions 5)</td><td colspan="2">LM 71
SW/0
SW/2
Unloaded train
HSLM</td><td>0,80
0,80
0
1,00
1,00</td><td>1)
1)
1,00
–
1)</td><td>0
0
0
–
0</td></tr>
<tr><td colspan="2">Traction and braking
Centrifugal forces
Interaction forces due to deformation under vertical traffic loads</td><td colspan="3">Individual Components of traffic action including design situations where the traffic loads are considered as a single (multi directional) leading action and not as groups of loads should use the same values as the ψ factors adopted for the associated vertical loads</td></tr>
<tr><td colspan="2">Nosing forces
Non public footpaths loads
Real trains
Traffic load surcharge horizontal earth pressure
Aerodynamic effects</td><td>1,00
0,80
0,80
0,80
0,80</td><td>0,80
0,50
0,80
1)
0,50</td><td>0
0
0
0
0</td></tr>
<tr><td rowspan="14">Main traffic actions
(Groups of loads)</td><td>gr11 (LM71 + SW/0)</td><td>Max. vertical 1 with max. longitudinal</td><td rowspan="7">0,80</td><td rowspan="7">0,80</td><td rowspan="7">0</td></tr>
<tr><td>gr12 (LM71 + SW/0)</td><td>Max. vertical 2 with max. transverse</td></tr>
<tr><td>gr13 (Braking/Traction)</td><td>Max. longitudinal</td></tr>
<tr><td>gr14 (Centrifugal/Nosing)</td><td>Max. lateral</td></tr>
<tr><td>gr15 (Unloaded train)</td><td>Lateral stability with “unloaded train”</td></tr>
<tr><td>gr16 (SW/2)</td><td>SW/2 with max. longitudinal</td></tr>
<tr><td>gr17 (SW/2)</td><td>SW/2 with max. transverse</td></tr>
<tr><td>gr21 (LM71 + SW/0)</td><td>Max. vertical 1 with max. longitudinal</td><td rowspan="6">0,80</td><td rowspan="6">0,70</td><td rowspan="6">0</td></tr>
<tr><td>gr22 (LM71 + SW/0)</td><td>Max. vertical 2 with max transverse</td></tr>
<tr><td>gr23 (Braking/Traction)</td><td>Max. longitudinal</td></tr>
<tr><td>gr24 (Centrifugal/Nosing)</td><td>Max. lateral</td></tr>
<tr><td>gr26 (SW/2)</td><td>SW/2 with max. longitudinal</td></tr>
<tr><td>gr27 (SW2)</td><td>SW/2 with max. transverse</td></tr>
<tr><td>gr31 (LM71 + SW/0)</td><td>Additional load cases</td><td>0,80</td><td>0,60</td><td>0</td></tr>
<tr><td rowspan="2">Other operating actions</td><td colspan="2">Aerodynamic effects</td><td>0,80</td><td>0,50</td><td>0</td></tr>
<tr><td colspan="2">General maintenance loading for non public footpaths</td><td>0,80</td><td>0,50</td><td>0</td></tr>
<tr><td rowspan="2">Wind forces 2)</td><td colspan="2">F_{Wk}</td><td>0,75</td><td>0,50</td><td>0</td></tr>
<tr><td colspan="2">F_W^{**}</td><td>1,00</td><td>0</td><td>0</td></tr>
<tr><td colspan="3">Table continued on next page</td><td></td><td></td><td></td></tr>
</table>

Table continued from previous page				
Thermal actions 3)	T_k	0,60	0,60	0,50
Snow loads	$Q_{Sn,k}$ (during execution)	0,8	-	0
Construction loads	Q_c	1,0		1,0

1) 0,8 if 1 track only is loaded
0,7 if 2 tracks are simultaneously loaded
0,6 if 3 or more tracks are simultaneously loaded.
2) When wind forces act simultaneously with traffic actions, the wind force $\psi_0 F_{Wk}$ should be taken as no greater than F_W^{**} (see EN 1991-1-4) See A2.2.4(4)
3) See EN 1991-1-5
4) If deformation is being considered, ψ_2 should be taken equal to 1,00 for rail traffic actions.
5) Minimum coexistent favourable vertical load with centrifugal, traction or braking individual components of rail traffic actions is 0,5LM71 etc.

NOTE 5 For specific design situations (*e.g.* calculation of bridge camber for aesthetics and drainage consideration, calculation of clearance, etc.) the requirements for the combinations of actions to be used may be defined for the individual project.

NOTE 6 For railway bridges, the infrequent value of variable actions is not relevant.

(2) For traffic actions, a unique ψ value should be applied to one group of loads as defined EN 1991-2, and taken as equal to the ψ value applicable to the leading component of the group.

(3) Where groups of loads are used for the design of railway bridges the groups of loads defined in EN 1991-2, 6.8.2, Table 6.11 should be used.

(4) Where relevant, for railway bridges, combinations of individual traffic actions (including individual components) should be taken into account.

NOTE Individual traffic actions may also have to be taken into account for example for the design of bearings, for the assessment of maximum lateral and minimum vertical traffic loading, bearing restraints, maximum overturning effects on abutments (especially for continuous bridges) etc., see Table A2.3.

A2.3 Ultimate limit states

NOTE Verification for fatigue excluded

A2.3.1 Design values of actions in persistent and transient design situations

(1) The design values of actions for ultimate limit states in the persistent and transient design situations (expressions 6.9a to 6.10b) should be in accordance with Tables A2.4(A) to (C).

NOTE The values in Tables A2.4 ((A) to (C)) may be altered in the National Annex (*e.g.* for different reliability levels see Section 2 and Annex B).

(2) In applying Tables A2.4(A) to A2.4(C) in cases when the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken according to 4.1.2(2)P.

(3) Static equilibrium (EQU, see 6.4.1 and 6.4.2(2)) for bridges should be verified using the design values of actions in Table A2.4(A).

(4) Design of structural members (STR, see 6.4.1) not involving geotechnical actions should be verified using the design values of actions in Table A2.4(B).

(5) Design of structural members (footings, piles, front walls of abutments, ballast retention walls, etc.) (STR) involving geotechnical actions and the resistance of the ground (GEO, see 6.4.1) should be verified using one only of the following three approaches supplemented, for geotechnical actions and resistances, by EN 1997 :

– Approach 1 : Applying in separate calculations design values from Table A2.4(C) and Table A2.4(B) to the geotechnical actions as well as the actions on/from the structure ;

– Approach 2 : Applying design values of actions from Table A2.4(B) to the geotechnical actions as well as the actions on/from the structure ;

– Approach 3 : Applying design values of actions from Table A2.4(C) to the geotechnical actions and, simultaneously, applying design values of actions from Table A2.4(B) to the actions on/from the structure.

NOTE The choice of approach 1, 2 or 3 is given in the National Annex.

(6) Site stability (*e.g.* the stability of a slope supporting a bridge pier) should be verified in accordance with EN 1997.

(7) Hydraulic and buoyancy failure (*e.g.* in the bottom of an excavation for a bridge foundation), if relevant, should be verified in accordance with EN 1997.

NOTE For water actions and debris effects, see EN 1991-1-6. General and local scour depths may have to be assessed for the individual project. Requirements for taking account of forces due to ice pressure on bridge piers etc. may be defined as appropriate in the National Annex or for the individual project.

(8) The γ_P values to be used for prestressing actions should be specified for the relevant representative values of these actions in accordance with EN 1990 to EN 1999.

NOTE In the cases where γ_P values are not provided in the relevant design Eurocodes, these values may be defined as appropriate in the National Annex or for the individual project. They depend, *inter alia*, on :
- the type of prestress (see the Note in 4.1.2(6))
- the classification of prestress as a direct or an indirect action (see 1.5.3.1)
- the type of structural analysis (see 1.5.6)
- the unfavourable or favourable character of the prestressing action and the leading or accompanying character of prestressing in the combination.

See also EN1991-1-6 for loading combinations during execution.

Table A2.4(A) - Design values of actions (EQU) (Set A)

Persistent and Transient Design Situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The γ values for the persistent and transient design situations may be set by the National Annex.

For persistent design situations, the recommended set of values for γ are :
$\gamma_{G,sup} = 1{,}05$
$\gamma_{G,inf} = 0{,}95^{(1)}$
$\gamma_Q = 1{,}35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1{,}45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1{,}50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable).
γ_P = recommended values defined in the relevant design Eurocode.

For transient design situations during which there is a risk of loss of static equilibrium, $Q_{k,1}$ represents the dominant destabilising variable action and $Q_{k,i}$ represents the relevant accompanying destabilising variable actions.

During execution, if the construction process is adequately controlled, the recommended set of values for γ are :
$\gamma_{G,sup} = 1{,}05$
$\gamma_{G,inf} = 0{,}95^{(1)}$
$\gamma_Q = 1{,}35$ for construction loads (0 where favourable)
$\gamma_Q = 1{,}50$ for all other variable actions, where unfavourable (0 where favourable)

(1) Where a counterweight is used, the variability of its characteristics may be taken into account, for example, by one or both of the following recommended rules :
– applying a partial factor $\gamma_{G,\text{inf}} = 0{,}8$ where the self-weight is not well defined (*e.g.* containers) ;
– by considering a variation of its project-defined location, with a value to be specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined. For steel bridges during launching, the variation of the counterweight location is often taken equal to ± 1 m.

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural elements (for example where loss of static equilibrium is prevented by stabilising systems or devices *e.g.* anchors, stays or auxiliary columns), as an alternative to two separate verifications based on Tables A2.4(A) and A2.4(B), a combined verification, based on Table A2.4(A), may be adopted with the following set of recommended values, which may be altered by the National Annex.
$\gamma_{G,sup} = 1{,}35$
$\gamma_{G,inf} = 1{,}15$
$\gamma_Q = 1{,}35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1{,}45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1{,}50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable)
$\gamma_Q = 1{,}35$ for all other variable actions, where unfavourable (0 where favourable)
provided that applying $\gamma_{G,inf} = 1{,}00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Table A2.4(B) - Design values of actions (STR/GEO) (Set B)

Persistent and Transient Design Situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying Variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and Transient Design Situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National Annex. In case of 6.10a and 6.10b, the National Annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National Annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b :

$\gamma_{G,sup} = 1,35$[1)]

$\gamma_{G,inf} = 1,00$

$\gamma_Q = 1,35$ when Q represents unfavourable actions due to road or pedestrian traffic (0 when favourable)

$\gamma_Q = 1,45$ when Q represents unfavourable actions due to rail traffic, to groups of loads 11 to 31 (except 16, 17, 26[3)] and 27[3)]), load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions (0 when favourable)

$\gamma_Q = 1,20$ when Q represents unfavourable actions due to rail traffic, to groups of loads 16 and 17 and SW/2 (0 when favourable)

$\gamma_Q = 1,50$ for other traffic actions and other variable actions [2)]

$\xi = 0,85$ (so that $\xi\gamma_{G,sup} = 0,85 \times 1,35 \cong 1,15$).

$\gamma_{Gset} = 1,20$ in case of linear elastic analysis, and 1,35 in case of non linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.

See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

γ_P = recommended values defined in the relevant design Eurocode.

1)This value covers : self-weight of structural and non structural elements, ballast, soil, ground water and free water, removable loads, etc.

2)This value covers : variable horizontal earth pressure from soil, ground water, free water and ballast, traffic load surcharge earth pressure, traffic aerodynamic actions, wind and thermal actions, etc.

3)For rail traffic actions for groups of loads 26 and 27 $\gamma_Q = 1,20$ may be applied to individual components of traffic actions associated with SW/2 and $\gamma_Q = 1,45$ may be applied to individual components of traffic actions associated with load models LM71, SW/0 and HSLM etc.

Table continued on next page

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source ; this also applies if different materials are involved. See however A2.3.1(2).

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,0 - 1,15 may be used in most common cases and may be modified in the National Annex.

NOTE 5 Where actions due to water are not covered by EN 1997 (*e.g.* flowing water), the combinations of actions to be used may be specified for the individual project.

Table A2.4(C) - Design values of actions (STR/GEO) (Set C)

Persistent and Transient Design Situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(*) Variable actions are those considered in Tables A2.1 to A2.3						

NOTE The γ values may be set by the National Annex. The recommended set of values for γ are :
$\gamma_{G,sup}$ = 1,00
$\gamma_{G,inf}$ = 1,00
γ_{Gset} = 1,00
γ_Q = 1,15 for road and pedestrian traffic actions where unfavourable (0 where favourable)
γ_Q = 1,25 for rail traffic actions where unfavourable (0 where favourable)
γ_Q = 1,30 for the variable part of horizontal earth pressure from soil, ground water, free water and ballast, for traffic load surcharge horizontal earth pressure, where unfavourable (0 where favourable)
γ_Q = 1,30 for all other variable actions where unfavourable (0 where favourable)
γ_{Gset} = 1,00 in case of linear elastic or non linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.
γ_P = recommended values defined in the relevant design Eurocode.

A2.3.2 Design values of actions in the accidental and seismic design situations

(1) The partial factors for actions for the ultimate limit states in the accidental and seismic design situations (expressions 6.11a to 6.12b) are given in Table A2.5. ψ values are given in Tables A2.1 to A2.3.

NOTE For the seismic design situation see also EN 1998.

Table A2.5 - Design values of actions for use in accidental and seismic combinations of actions

Design Situation	Permanent actions		Prestress	Accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable			Main (if any)	Others
Accidental(*) (Eq. 6.11a/b)	$G_{kj,sup}$	$G_{kj,inf}$	P	A_d	$\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$	$\psi_{2,i}\, Q_{k,i}$
Seismic(***) (Eq. 6.12a/b)	$G_{kj,sup}$	$G_{kj,inf}$	P	$A_{Ed} = \gamma_I A_{Ek}$		$\psi_{2,i}\, Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National Annex, depending on the accidental action under consideration.

(**) Variable actions are those considered in Tables A2.1 to A2.3.

(***) The National Annex may specify particular seismic design situations, especially for railway bridges.

NOTE The design values in this Table A2.5 may be altered in the National Annex. The recommended values are $\gamma = 1,0$ for all non seismic actions.

(2) Where, in special cases, one or several variable actions need to be considered simultaneously with the accidental action, their representative values should be defined.

NOTE As an example, in the case of bridges built by the cantilevered method, some construction loads may be considered as simultaneous with the action corresponding to the accidental fall of a prefabricated unit. The relevant representative values may be defined for the individual project.

(3) For execution phases during which there is a risk of loss of static equilibrium, the combination of actions should be as follows :

$$\sum_{j\geq 1} G_{kj,sup} \text{"+"} \sum_{j\geq 1} G_{kj,inf} \text{"+"} P \text{"+"} A_d \text{"+"} \psi_2 Q_{c,k} \qquad \text{(A2.2)}$$

where :

$Q_{c,k}$ is the characteristic value of construction loads as defined in EN 1991-1-6 (*i.e.* the characteristic value of the relevant combination of groups Q_{ca}, Q_{cb}, Q_{cc}, Q_{cd}, Q_{ce}, Q_{cf}).

A2.4 Serviceability and other specific limit states

A2.4.1 General

(1) For serviceability limit states the design values of actions should be taken from Table A2.6 except if differently specified in EN1991 to EN1999.

NOTE 1 γ factors for traffic and other actions for the serviceability limit state may be defined in the National Annex. The recommended design values are given in Table A2.6, with all γ factors being taken as 1,0.

Table A2.6 - Design values of actions for use in the combination of actions

Combination	Permanent actions G_d		Prestress	Variable actions Q_d	
	Unfavourable	Favourable		Leading	Others
Characteristic	$G_{kj,sup}$	$G_{kj,inf}$	P	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{kj,sup}$	$G_{kj,inf}$	P	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{kj,sup}$	$G_{kj,inf}$	P	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

NOTE 2 The National Annex may also refer to the infrequent combination of actions.

(2) The serviceability criteria should be defined in relation to the serviceability requirements in accordance with 3.4 and EN 1992 to EN 1999. Deformations should be calculated in accordance with EN 1991 to EN 1999, by using the appropriate combinations of actions according to expressions (6.14a) to (6.16b) (see Table A2.6) taking into account the serviceability requirements and the distinction between reversible and irreversible limit states.

NOTE Serviceability requirements and criteria may be defined as appropriate in the National Annex or for the individual project.

A2.4.2 Serviceability criteria regarding deformation and vibration for road bridges

(1) Where relevant, requirements and criteria should be defined for road bridges concerning :

- uplift of the bridge deck at supports,
- damage to structural bearings.

NOTE Uplift at the end of a deck can jeopardise traffic safety and damage structural and non-structural elements. Uplift may be avoided by using a higher safety level than usually accepted for serviceability limit states.

(2) Serviceability limit states during execution should be defined in accordance with EN 1990 to EN 1999

(3) Requirements and criteria should be defined for road bridges concerning deformations and vibrations, where relevant.

NOTE 1 The verification of serviceability limit states concerning deformation and vibration needs to be considered only in exceptional cases for road bridges. The frequent combination of actions is recommended for the assessment of deformation.

NOTE 2 Vibrations of road bridges may have various origins, in particular traffic actions and wind actions. For vibrations due to wind actions, see EN 1991-1-4. For vibrations due to traffic actions, comfort criteria may have to be considered. Fatigue may also have to be taken into account.

A2.4.3 Verifications concerning vibration for footbridges due to pedestrian traffic

NOTE For vibrations due to wind actions, see EN 1991-1-4.

A2.4.3.1 Design situations associated with traffic categories

(1) The design situations (see 3.2) should be selected depending on the pedestrian traffic to be admitted on the individual footbridge during its design working life.

NOTE The design situations may take into account the way the traffic will be authorised, regulated and controlled, depending on the individual project.

(2) Depending on the deck area or the part of the deck area under consideration, the presence of a group of about 8 to 15 persons walking normally should be taken into account for design situations considered as persistent design situations.

(3) Depending on the deck area or the part of the deck area under consideration, other traffic categories, associated with design situations which may be persistent, transient or accidental, should be specified when relevant, including :

- presence of streams of pedestrians (significantly more than 15 persons) ;
- occasional festive or choreographic events.

NOTE 1 These traffic categories and the relevant design situations may have to be agreed for the individual project, not only for bridges in highly populated urban areas, but also in the vicinity of railway and bus stations, schools, or any other places where crowds may congregate, or any important building with public admittance.

NOTE 2 The definition of design situations corresponding to occasional festive or choreographic events depends on the expected degree of control of them by a responsible owner or authority. No verification rule is provided in the present clause and special studies may need to be considered. Some information on the relevant design criteria may be found in appropriate literature.

A2.4.3.2 Pedestrian comfort criteria (for serviceability)

(1) The comfort criteria should be defined in terms of maximum acceptable acceleration of any part of the deck.

NOTE The criteria may be defined as appropriate in the National Annex or for the individual project.
The following accelerations (m/s^2) are the recommended maximum values for any part of the deck :

- 0,7 for vertical vibrations,
- 0,2 for horizontal vibrations in normal use,
- 0,4 for exceptional crowd conditions.

(2) A verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than :

- 5 Hz for vertical vibrations,
- 2,5 Hz for horizontal (lateral) and torsional vibrations,

NOTE The data used in the calculations, and therefore the results, are subject to very high uncertainties. When the comfort criteria are not satisfied with a significant margin, it may be necessary to make provision in the design for the possible installation of dampers in the structure after its completion. In such cases the designer should consider and identify any requirements for commissioning tests.

A2.4.4 Verifications regarding deformations and vibrations for railway bridges

A2.4.4.1 General

(1) This clause A2.4.4 gives the limits of deformation and vibration to be taken into account for the design of new railway bridges.

NOTE 1 Excessive bridge deformations can endanger traffic by creating unacceptable changes in vertical and horizontal track geometry, excessive rail stresses and vibrations in bridge structures. Excessive vibrations can lead to ballast instability and unacceptable reduction in wheel rail contact forces. Excessive deformations can also affect the loads imposed on the track/ bridge system, and create conditions which cause passenger discomfort.

NOTE 2 Deformation and vibration limits are either explicit or implicit in the bridge stiffness criteria given in A2.4.4.1(2)P.

NOTE 3 The National Annex may specify limits of deformation and vibration to be taken into account for the design of temporary railway bridges. The National Annex may give special requirements for temporary bridges depending upon the conditions in which they are used (*e.g.* special requirements for skew bridges).

(2)P Checks on bridge deformations shall be performed for traffic safety purposes for the following items :

– vertical accelerations of the deck (to avoid ballast instability and unacceptable reduction in wheel rail contact forces – see A2.4.4.2.1),
– vertical deflection of the deck throughout each span (to ensure acceptable vertical track radii and generally robust structures – see A2.4.4.2.3(3)),
– unrestrained uplift at the bearings (to avoid premature bearing failure),
– vertical deflection of the end of the deck beyond bearings (to avoid destabilising the track, limit uplift forces on rail fastening systems and limit additional rail stresses – see A2.4.4.2.3(1) and EN1991-2, 6.5.4.5.2),
– twist of the deck measured along the centre line of each track on the approaches to a bridge and across a bridge (to minimise the risk of train derailment – see A2.4.4.2.2),

NOTE A2.4.4.2.2 contains a mix of traffic safety and passenger comfort criteria that satisfy both traffic safety and passenger comfort requirements.

– rotation of the ends of each deck about a transverse axis or the relative total rotation between adjacent deck ends (to limit additional rail stresses (see EN 1991-2, 6.5.4), limit uplift forces on rail fastening systems and limit angular discontinuity at expansion devices and switch blades – see A2.4.4.2.3(2)),
– longitudinal displacement of the end of the upper surface of the deck due to longitudinal displacement and rotation of the deck end (to limit additional rail stresses and minimise disturbance to track ballast and adjacent track formation – see EN 1991-2, 6.5.4.5.2),
– horizontal transverse deflection (to ensure acceptable horizontal track radii – see A2.4.4.2.4, Table A2.8),
– horizontal rotation of a deck about a vertical axis at ends of a deck (to ensure acceptable horizontal track geometry and passenger comfort – see A2.4.4.2.4 Table A2.8),
– limits on the first natural frequency of lateral vibration of the span to avoid the occurrence of resonance between the lateral motion of vehicles on their suspension and the bridge – see A2.4.4.2.4(3).

NOTE There are other implicit stiffness criteria in the limits of bridge natural frequency given in EN 1991-2, 6.4.4 and when determining dynamic factors for Real Trains in accordance with EN 1991-2, 6.4.6.4 and EN1991-2 Annex C.

(3) Checks on bridge deformations should be performed for passenger comfort, i.e. vertical deflection of the deck to limit coach body acceleration in accordance with A2.4.4.3.

(4) The limits given in A2.4.4.2 and A2.4.4.3 take into account the mitigating effects of track maintenance (for example to overcome the effects of settlements of foundations, creep, etc.).

A2.4.4.2 Criteria for traffic safety

A2.4.4.2.1 Vertical acceleration of the deck

(1)P To ensure traffic safety, where a dynamic analysis is necessary, the verification of maximum peak deck acceleration due to rail traffic actions shall be regarded as a traffic safety requirement checked at the serviceability limit state for the prevention of track instability.

(2) The requirements for determining whether a dynamic analysis is necessary are given in EN 1991-2, 6.4.4.

(3)P Where a dynamic analysis is necessary, it shall comply with the requirements given in EN 1991-2, 6.4.6.

NOTE Generally only characteristic rail traffic actions in accordance with EN1991-2, 6.4.6.1 need to be considered.

(4)P The maximum permitted peak values of bridge deck acceleration calculated along each track shall not exceed the following design values :

i) γ_{bt} for ballasted track ;

ii) γ_{df} for direct fastened decks with track and structural elements designed for high speed traffic

for all elements supporting the track considering frequencies (including consideration of associated mode shapes) up to the greater of :

i) 30 Hz ;

ii) 1,5 times the frequency of the first mode of vibration of the element being considered including at least the first three modes.

NOTE The values and the associated frequency limits may be defined in the National Annex. The recommended values are :
γ_{bt} = 3,5 m/s^2
γ_{df} = 5 m/s^2

A2.4.4.2.2 Deck twist

(1)P The twist of the bridge deck shall be calculated taking into account the characteristic values of Load Model 71 as well as SW/0 or SW/2 as appropriate multiplied by Φ and α and Load Model HSLM including centrifugal effects all in accordance with EN1991-2, 6.

(2) The maximum twist t [mm/3m] of a track gauge s [m] of 1,435 m measured over a length of 3m (Figure A2.1) should not exceed the values given in Table A2.7 :

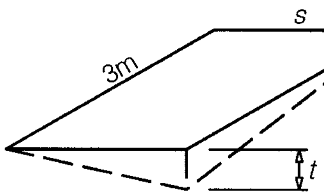


Figure A2.1 - Definition of deck twist

Table A2.7 – Limiting values of deck twist

Speed range V (km/h)	Maximum twist t (mm/3m)
$V \leq 120$	$t \leq t_1$
$120 < V \leq 200$	$t \leq t_2$
$V > 200$	$t \leq t_3$

NOTE The values for t may be defined in the National Annex.
The recommended values for the set of t are :
$t_1 = 4{,}5$
$t_2 = 3{,}0$
$t_3 = 1{,}5$
Values for track with a different gauge may be defined in the National Annex.

(3)P The total track twist due to any twist which may be present in the track when the bridge is not subject to rail traffic actions (for example in a transition curve), plus the track twist due to the total deformation of the bridge resulting from rail traffic actions, shall not exceed t_T.

NOTE The value for t_T may be defined in the National Annex. The recommended value for t_T is 7,5 mm/3m.

A2.4.4.2.3 Vertical deformation of the deck

(1) For all structure configurations loaded with the classified characteristic vertical loading in accordance with EN 1991-2, 6.3.2 (and where required classified SW/0 and SW/2 in accordance with EN 1991-2, 6.3.3) the maximum total vertical deflection measured along any track due to rail traffic actions should not exceed L/600.

NOTE Additional requirements for limiting vertical deformation for ballasted and non ballasted bridges may be specified as appropriate in the National Annex or for the individual project.

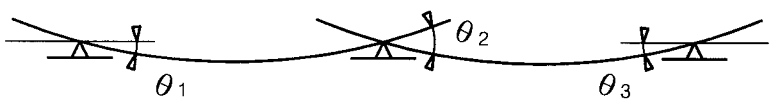


Figure A2.2 - Definition of angular rotations at the end of decks

(2) Limitations on the rotations of ballasted bridge deck ends are implicit in EN 1991-2, 6.5.4.

NOTE The requirements for non ballasted structures may be specified in the National Annex.

(3) Additional limits of angular rotations at the end of decks in the vicinity of expansion devices, switches and crossings, etc. should be specified.

NOTE The additional limits of angular rotations may be defined in the National Annex or for the individual project.

(4) Limitations on the vertical displacement of bridge deck ends beyond bearings are given in EN1991-2, 6.5.4.5.2.

A2.4.4.2.4 Transverse deformation and vibration of the deck

(1)P This condition shall be checked for characteristic combinations of Load Model 71 and SW/0 as appropriate multiplied by the dynamic factor ϕ and α (or real train with the relevant dynamic factor if appropriate), wind loads, nosing force, centrifugal forces in accordance with EN1991-2, 6 and the effect of transverse temperature differential across the bridge.

(2) The transverse deflection δ_h of the deck should be limited to ensure :
- an angular variation not greater than the values given in Table A2.8, or
- a radius of horizontal curvature less than the values in Table A2.8.

Table A2.8 - Maximum angular variation and minimum radius of curvature

Speed range V (km/h)	Maximum angular variation (radian)	Minimum radius of curvature (m)	
		Single deck	**Multi-deck bridge**
$V \leq 120$	α_1	R_1	r_4
$120 < V \leq 200$	α_2	r_2	r_5
$V > 200$	α_3	r_3	r_6

NOTE 1 The radius of curvature may be determined using :

$$R = \frac{L^2}{8\delta_h} \qquad \text{(A2.7)}$$

NOTE 2 The transverse deformation includes the deformation of the bridge deck and the substructure (including piers, piles and foundations).

NOTE 3 The values for the set of α_i and r_i may be defined in the National Annex. The recommended values are :
$\alpha_1 = 0{,}0035$; $\alpha_2 = 0{,}0020$; $\alpha_3 = 0{,}0015$;
$r_1 = 1700$; $r_2 = 6000$; $r_3 = 14000$;
$r_4 = 3500$; $r_5 = 9500$; $r_6 = 17500$

(3) The first natural frequency of lateral vibration of a span should have a minimum value of f_{h0}.

NOTE The value for f_{h0} may be defined in the National Annex. The recommended value is :
f_{h0} = 1,2 Hz

A2.4.4.2.5 Longitudinal displacement of the deck

(1) Limitations on the longitudinal displacement of the ends of decks are given in EN1991-2, 6.5.4.5.2.

NOTE Also see A2.4.4.2.3.

A2.4.4.3 Limiting values for the maximum vertical deflection for passenger comfort

A2.4.4.3.1 Comfort criteria

(1) Passenger comfort depends on the vertical acceleration b_v inside the coach during travel on the approach to, passage over and departure from the bridge.

(2) The levels of comfort and associated limiting values for the vertical acceleration should be specified.

NOTE These levels of comfort and associated limiting values may be defined for the individual project. Indicative levels of comfort are given in Table A2.9.

Table A2.9 - Indicative levels of comfort

Level of comfort	Vertical acceleration b_v (m/s^2)
Very good	1,0
Good	1,3
Acceptable	2,0

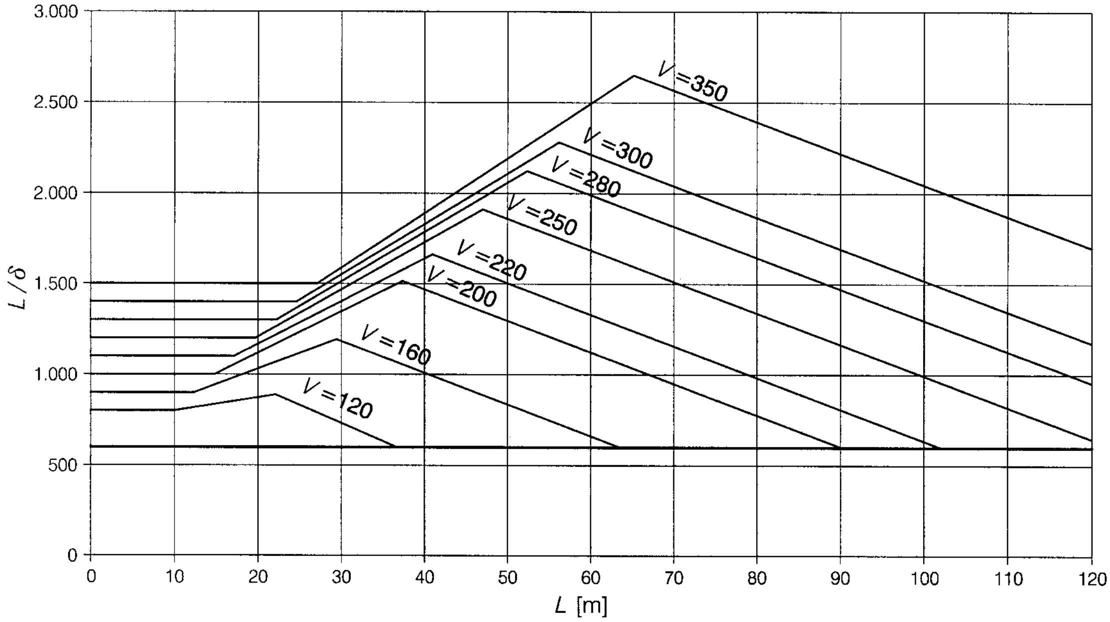
A2.4.4.3.2 Deflection criteria for checking passenger comfort

(1) To limit vertical vehicle acceleration to the values given in A2.4.4.3.1(2) values are given in this clause for the maximum permissible vertical deflection δ along the centre line of the track of railway bridges as a function of :
- the span length L [m]
- the train speed V [km/h]
- the number of spans and
- the configuration of the bridge (simply supported beam, continuous beam).

Alternatively the vertical acceleration b_v may be determined by a dynamic vehicle/bridge interaction analysis (see A2.4.4.3.3).

(2) The vertical deflections δ should be determined with Load Model 71 multiplied by the factor Φ and with the value of α taken as unity, in accordance with EN1991-2, Section 6.
For bridges with two or more tracks only one track should be loaded.

(3) For exceptional structures, *e.g.* continuous beams with widely varying span lengths or spans with wide variations in stiffness, a specific dynamic calculation should be carried out.



L/δ = 600 Limit : The factors listed in A2.4.4.3.2.(5) should not be applied to this limit.

Figure A2.3 - Maximum permissible vertical deflection δ for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of b_v = 1 m/s² in a coach for speed *V* [km/h].

(4) The limiting values of L/δ given in figure A2.3 are given for b_v = 1,0 m/s² which may be taken as providing a "very good" level of comfort.
For other levels of comfort and associated maximum permissible vertical accelerations b'_v the values of L/δ given in figure A2.3 may be divided by b'_v [m/s²].

(5) The values L/δ given in figure A2.3 are given for a succession of simply supported beams with three or more spans.
For a bridge comprising of either a single span or a succession of two simply supported beams or two continuous spans the values L/δ given in figure A2.3 should be multiplied by 0,7.
For continuous beams with three or more spans the values of L/δ given in figure A2.3 should be multiplied by 0,9.

(6) The values L/δ given in figure A2.3 are valid for span length up to 120 m. For longer spans a special analysis is necessary.

NOTE The requirements for passenger comfort for temporary bridges may be defined as relevant in the National Annex or for the individual project.

A2.4.4.3.3 Requirements for a dynamic vehicle/bridge interaction analysis for checking passenger comfort

(1) Where a vehicle/bridge dynamic interaction analysis is required the analysis should take account of the following behaviours :

i) a range of vehicle speeds up to the maximum speed specified,
ii) characteristic loading of the Real Trains specified for the individual project in accordance with EN1991-2, 6.4.6.1.1.
iii) dynamic mass interaction between vehicles in the Real Train and the structure,
iv) the damping and stiffness characteristics of the vehicle suspension,
v) sufficient vehicles to produce the maximum load effects in the longest span.
vi) a sufficient number of spans in a structure with multiple spans to develop any resonance effects in the vehicle suspension.

NOTE Any requirements for taking track roughness into account in the vehicle/bridge dynamic interaction analysis may be defined for the individual project.