The European Union

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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

EN 1995-2

November 2004

ICS 91.010.30; 91.080.20; 93.040

English version

Eurocode 5: Design of timber structures - Part 2: Bridges

Eurocode 5: Conception et calcul des structures bois -
Partie 2: Ponts

Eurocode 5: Bemessung und Konstruktion von Holzbauten
- Teil 2: Brücken

This European Standard was approved by CEN on 26 August 2004.

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

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

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

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Contents

Foreword

This European Standard EN 1995-2 has been prepared by Technical Committee CEN/TC250 "Structural Eurocodes", the Secretariat of which is held by BSI.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by May 2005, and conflicting national standards shall be withdrawn at the latest by March 2010.

This European Standard supersedes ENV 1995-2:1997.

CEN/TC250 is responsible for all Structural Eurocodes.

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

Background of the Eurocode programme

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

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

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

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

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

EN 1990:2002 Eurocode: Basis of Structural Design
EN 1991   Eurocode 1: Actions on structures
EN 1992   Eurocode 2: Design of concrete structures
EN 1993   Eurocode 3: Design of steel structures
EN 1994   Eurocode 4: Design of composite steel and concrete structures
EN 1995   Eurocode 5: Design of timber structures
EN 1996   Eurocode 6: Design of masonry structures
EN 1997   Eurocode 7: Geotechnical design

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

EN 1998   Eurocode 8: Design of structures for earthquake resistance
EN 1999   Eurocode 9: Design of aluminium structures

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

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:
– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;

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

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

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

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

National Standards implementing Eurocodes

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

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:
– values and/or classes where alternatives are given in the Eurocode;

– values to be used where a symbol only is given in the Eurocode;

– country specific data (geographical, climatic, etc.), e.g. snow map;

– the procedure to be used where alternative procedures are given in the Eurocode;

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.
[3] According to Art. 12 of the CPD the interpretative documents shall :
give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;
serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

- decisions on the application of informative annexes;

- references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

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

Additional information specific to EN 1995-2

EN 1995 describes the Principles and requirements for safety, serviceability and durability of timber bridges. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1995-2 is intended to be used, for direct application, together with EN 1995-1-1 and EN1990:2002 and relevant Parts of EN 1991.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1995-2 is used as a base document by other CEN/TCs the same values need to be taken.

National annex for EN 1995-2

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1995-2 should have a National annex containing all Nationally Determined Parameters to be used for the design of bridges to be constructed in the relevant country.

National choice is allowed in EN 1995-2 through clauses:
2.3.1.2(1)   Load-duration assignment
2.4.1   Partial factors for material properties
7.2   Limiting values for deflection
7.3.1(2)   Damping ratios

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

Section 1 General

1.1 Scope

1.1.1 Scope of EN 1990

(1)P EN 1990 applies to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners. It complies with the principles and requirements for the safety and serviceability of structures, and the basis of design and verification that are given in EN 1990:2002.

(2)P EN 1990 is only concerned with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures. Other requirements, e.g concerning thermal or sound insulation, are not considered.

(3) EN 1990 is intended to be used in conjunction with:
EN 1990:2002 Eurocode – Basis of structural design
EN 1991 "Actions on structures"
EN´s for construction products relevant to timber structures
EN 1998 "Design of structures for earthquake resistance", when timber structures are built in seismic regions

(4) EN 1990 is subdivided into various parts:
EN 1995-1   General
EN 1995-2   Bridges

(5) EN 1995-1 "General" comprises:
EN 1995-1-1 General – Common rules and rules for buildings
EN 1995-1-2 General – Structural Fire Design

1.1.2 Scope of EN 1995-2

(1) EN 1995-2 gives general design rules for the structural parts of bridges, i.e. structural members of importance for the reliability of the whole bridge or major parts of it, made of timber or other wood-based materials, either singly or compositely with concrete, steel or other materials.

(2) The following subjects are dealt with in EN 1995-2:
Section 1:   General
Section 2:   Basis of design
Section 3:   Material properties
Section 4:   Durability
Section 5:   Basis of structural analysis
Section 6:   Ultimate limit states
Section 7:   Serviceability limit states
Section 8:   Connections
Section 9:   Structural detailing and control

(3) Section 1 and Section 2 also provide additional clauses to those given in EN 1990:2002 "Eurocode: Basis of structural design".

(4) Unless specifically stated, EN 1995-1-1 applies.

1.2 Normative references

(1) The following normative documents contain provisions which, through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments

to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

European Standards:

EN 1990:2002   Eurocode – Basis of structural design
EN1990:2002/A1   Eurocode – Basis of structural design/Amendment A1 – Annex A2:
                      Application to Bridges
EN 1991-1-4   Eurocode 1: Actions on structures – Part 1-4: Wind loads
EN 1991-2   Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges
EN 1992-1-1   Eurocode 2: Design of concrete structures – Part 1-1: Common rules and
                      rules for buildings
EN 1992-2   Eurocode 2: Design of concrete structures – Part 2: Bridges
EN 1993-2   Eurocode 3: Design of steel structures – Part 2: Bridges
EN 1995-1-1   Eurocode 5: Design of timber structures – Part 1-1: General – Common
                      rules and rules for buildings
EN 10138-1   Prestressing steels – Part 1: General requirements
EN 10138-4   Prestressing steels – Part 4: Bars

1.3   Assumptions

(1) Additional requirements for execution, maintenance and control are given in section 9.

1.4   Distinction between principles and application rules

(1) See 1.4(1) of EN 1995-1-1.

1.5   Definitions

1.5.1   General

(1)P The definitions of EN 1990:2002 clause 1.5 and EN 1995-1-1 clause 1.5 apply.

1.5.2   Additional terms and definitions used in this present standard

1.5.2.1
Grooved connection
Shear connection consisting of the integral part of one member embedded in the contact face of the other member. The contacted parts are normally held together by mechanical fasteners.

NOTE: An example of a grooved connection is shown in figure 1.1.
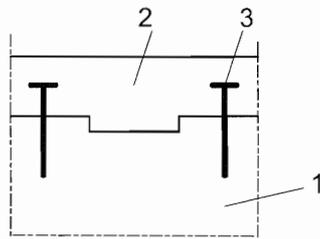
Key:
1   Timber
2   Concrete
3   Fastener

Figure 1.1 – Example of grooved connection

1.5.2.2
Laminated deck plates
Deck plates made of laminations, arranged edgewise or flatwise, held together by mechanical fasteners or gluing, see figures 1.2 and 1.3.

1.5.2.3
Stress-laminated deck plates
Laminated deck plates made of edgewise arranged laminations with surfaces either sawn or planed, held together by pre-stressing, see figure 1.2.b, c and d.

a)

b)

c)

d)
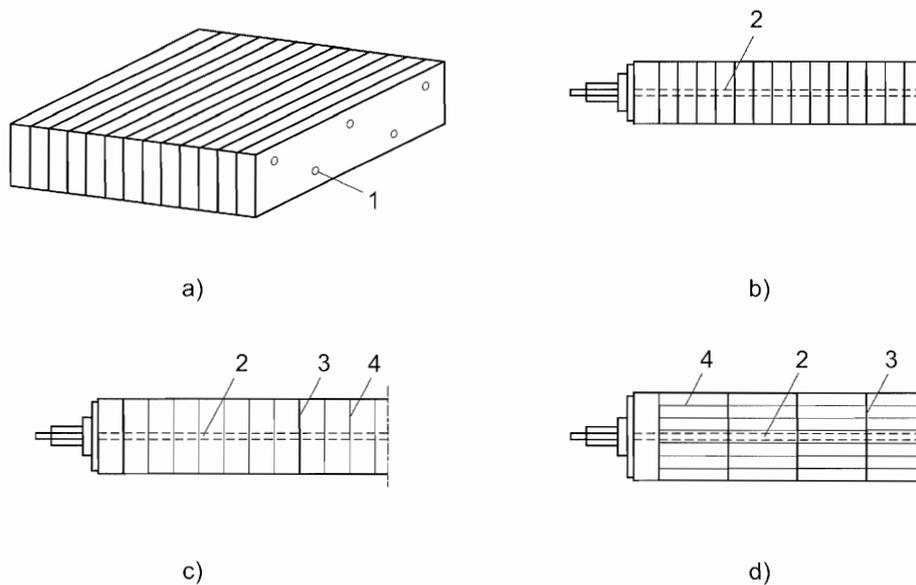
Key:
1   Nail or screw
2   Pre-stressing bar or tendon
3   Glue-line between glued laminated members
4   Glue-line between laminations in glued laminated members

Figure 1.2 – Examples of deck plates made of edgewise arranged laminations
a) nail-laminated or screw-laminated
b) pre-stressed, but not glued
c) glued and pre-stressed glued laminated beams positioned flatwise
d) glued and pre-stressed glued laminated beams positioned edgewise

1.5.2.4
Cross-laminated deck plates
Laminated deck plates made of laminations in layers of different grain direction (crosswise or at different angles). The layers are glued together or connected using mechanical fasteners, see figure 1.3.

1.5.2.5
Pre-stressing
A permanent effect due to controlled forces and/or deformations imposed on a structure.

NOTE: An example is the lateral pre-stressing of timber deck plates by means of bars or tendons, see figure 1.2 b to d.
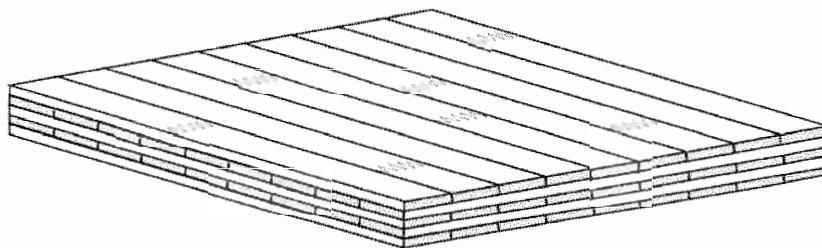

Figure 1.3 – Example of cross-laminated deck plate

1.6   Symbols used in EN 1995-2

For the purpose of EN 1995-2, the following symbols apply.

Latin upper case letters

A	Area of bridge deck
$E_{0,mean}$	Mean modulus of elasticity parallel to grain
$E_{90,mean}$	Mean modulus of elasticity perpendicular to the grain
F	Force
$F_{t,Ed}$	Design tensile force between timber and concrete
$F_{v,Ed}$	Design shear force between timber and concrete
$G_{0,mean}$	Mean shear modulus parallel to grain
$G_{90,mean}$	Mean shear modulus perpendicular to grain (rolling shear)
M	Total mass of bridge
M_{beam}	Bending moment in a beam representing a plate
$M_{max,beam}$	Maximum bending moment in a beam representing a plate
N_{obs}	Number of constant amplitude stress cycles per year
R	Ratio of stresses

Latin lower case letters

a	Distance; fatigue coefficient
$a_{hor,1}$	Horizontal acceleration from one person crossing the bridge
$a_{hor,n}$	Horizontal acceleration from several people crossing the bridge
$a_{vert,1}$	Vertical acceleration from one person crossing the bridge
$a_{vert,n}$	Vertical acceleration from several people crossing the bridge
b	Fatigue coefficient
b_{ef}	Effective width
$b_{ef,c}$	Total effective width of concrete slab
$b_{ef,1}; b_{ef,2}$	Effective width of concrete slab

b_{lam}	Width of the lamination
b_w	Width of the loaded area on the contact surface of deck plate
$b_{w,middle}$	Width of the loaded area in the middle of the deck plate
d	Diameter; outer diameter of rod; distance
h	Depth of beam; thickness of plate
$f_{c,90,d}$	Design compressive strength perpendicular to grain
$f_{fat,d}$	Design value of fatigue strength
f_k	Characteristic strength
$f_{m,d,deck}$	Design bending strength of deck plate
$f_{v,d,deck}$	Design shear strength of deck plate
$f_{m,d,lam}$	Design bending strength of laminations
$f_{v,d,lam}$	Design shear strength of laminations
f_{vert}, f_{hor}	Fundamental natural frequency of vertical and horizontal vibrations
$k_{c,90}$	Factor for compressive strength perpendicular to the grain
k_{fat}	Factor representing the reduction of strength with number of load cycles
k_{hor}	Coefficient
k_{mod}	Modification factor
k_{sys}	System strength factor
k_{vert}	Coefficient
ℓ	Span
ℓ_1	Distance
m	Mass; mass per unit length
m_{plate}	Bending moment in a plate per unit length
$m_{max,plate}$	Maximum bending moment in a plate
n	Number of loaded laminations; number of pedestrians
n_{ADT}	Expected annual average daily traffic over the lifetime of the structure
t	Time; thickness of lamination
t_L	Design service life of the structure expressed in years

Greek lower case letters

α	Expected percentage of observed heavy lorries passing over the bridge
β	Factor based on the damage consequence; angle of stress dispersion
γ_M	Partial factor for timber material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,c}$	Partial factor for concrete material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,s}$	Partial factor for steel material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,v}$	Partial factor for shear connectors, also accounting for model uncertainties and dimensional variations
$\gamma_{M,fat}$	Partial safety factor for fatigue verification of materials, also accounting for model uncertainties and dimensional variations
κ	Ratio for fatigue verification
ρ_{mean}	Mean density
μ_d	Design coefficient of friction
$\sigma_{d,max}$	Numerically largest value of design stress for fatigue loading
$\sigma_{d,min}$	Numerically smallest value of design stress for fatigue loading
$\sigma_{p,min}$	Minimum long-term residual compressive stress due to pre-stressing;
ζ	Damping ratio

Section 2 Basis of design

2.1 Basic requirements

(1)P The design of timber bridges shall be in accordance with EN 1990:2002.

2.2 Principles of limit state design

(1) See 2.2 of EN 1995-1-1.

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) Actions to be used in design of bridges may be obtained from the relevant parts of EN 1991.

Note 1: The relevant parts of EN 1991 for use in design include:
EN 1991-1-1   Densities, self-weight and imposed loads
EN 1991-1-3   Snow loads
EN 1991-1-4   Wind loads
EN 1991-1-5   Thermal actions
EN 1991-1-6   Actions during execution
EN 1991-1-7   Accidental actions due to impact and explosions
EN 1991-2   Traffic loads on bridges.

2.3.1.2 Load-duration classes

(1) Variable actions due to the passage of vehicular and pedestrian traffic should be regarded as short-term actions.

NOTE: Examples of load-duration assignments are given in note to 2.3.1 of EN 1995-1-1. The recommended load-duration assignment for actions during erection is short-term. The National choice may be given in the National annex.

(2) Initial pre-stressing forces perpendicular to the grain should be regarded as short-term actions.

2.4 Verification by the partial factor method

2.4.1 Design value of material property

NOTE: For fundamental combinations, the recommended partial factors for material properties, γ_M, are given in table 2.1. For accidental combinations, the recommended value of partial factor is $\gamma_M = 1,0$. Information on the National choice may be found in the National annex.

Table 2.1 – Recommended partial factors for material properties

1. Timber and wood-based materials
  − normal verification
    − solid timber
    − glued laminated timber
    − LVL, plywood, OSB
  − fatigue verification
	γ_M
	γ_M
	γ_M
	$\gamma_{M,fat}$
		= 1,3
		= 1,25
		= 1,2
		= 1,0

2. Connections
  − normal verification
  − fatigue verification
	γ_M
	$\gamma_{M,fat}$
		= 1,3
		= 1,0

3. Steel used in composite members	$\gamma_{M,s}$	= 1,15
4. Concrete used in composite members	$\gamma_{M,c}$	= 1,5

5. Shear connectors between timber and
   concrete in composite members
    − normal verification
    − fatigue verification
	$\gamma_{M,v}$
	$\gamma_{M,v,fat}$
		= 1,25
		= 1,0

6. Pre-stressing steel elements	$\gamma_{M,s}$	= 1,15

Section 3 Material properties

(1)P Pre-stressing steels shall comply with EN 10138-1 and EN 10138-4.

Section 4 Durability

4.1 Timber

(1) The effect of precipitation, wind and solar radiation should be taken into account.

NOTE 1: The effect of direct weathering by precipitation or solar radiation of structural timber members can be reduced by constructional preservation measures, or by using timber with sufficient natural durability, or timber preservatively treated against biological attacks.

NOTE 2: Where a partial or complete covering of the main structural elements is not practical, durability can be improved by one or more of the following measures:
– limiting standing water on timber surfaces through appropriate inclination of surfaces;

– limiting openings, slots, etc., where water may accumulate or infiltrate;

– limiting direct absorption of water (e.g. capillary absorption from concrete foundation) through use of appropriate barriers;

– limiting fissures and delaminations, especially at locations where the end grain would be exposed, by appropriate sealing and/or cover plates;

– limiting swelling and shrinking movements by ensuring an appropriate initial moisture content and by reducing in-service moisture changes through adequate surface protection

– choosing a geometry for the structure that ensures natural ventilation of all timber parts.

NOTE 3: The risk of increased moisture content near the ground, e.g. due to insufficient ventilation due to vegetation between the timber and the ground, or splashing water, can be reduced by one or more of the following measures:
– covering of the ground by course gravel or similar to limit vegetation;
– use of an increased distance between the timber parts and the ground level.

(2)P Where structural timber members are exposed to abrasion by traffic, the depth used in the design shall be the minimum permitted before replacement.

4.2 Resistance to corrosion

(1) EN 1995-1-1 clause 4.2 applies to fasteners. EN 1993-2 applies to steel parts other than fasteners.

NOTE: An example of especially corrosive conditions is a timber bridge where corrosive de-icing cannot be excluded.

(2)P The possibility of stress corrosion shall be taken into account.

(3) Steel parts encased in concrete, such as reinforcing bars and pre-stressing cables, should be protected according EN 1992-1-1 clause 4.4.1 and EN 1992-2.

(4) The effect of chemical treatment of timber, or timber with high acidic content, on the corrosion protection of fasteners should be taken into account.

4.3 Protection of timber decks from water by sealing

(1)P The elasticity of the seal layers shall be sufficient to follow the movement of the timber deck.

Section 5 Basis of structural analysis

5.1 Laminated deck plates

5.1.1 General

(1) The analysis of laminated timber deck plates should be based upon one of the following:
- the orthotropic plate theory;
- modelling the deck plate by a grid;
- a simplified method according to 5.1.3.

NOTE: In an advanced analysis, for deck plates made of softwood laminations, the relationships for the system properties should be taken from table 5.1. The Poisson ratio ν may be taken as zero.

Table 5.1 – System properties of laminated deck plates

Type of deck plate	$E_{90,mean}/E_{0,mean}$	$G_{0,mean}/E_{0,mean}$	$G_{90,mean}/G_{0,mean}$
Nail-laminated	0	0,06	0,05
Stress-laminated			
– sawn	0,015	0,06	0,08
– planed	0,020	0,06	0,10
Glued-laminated	0,030	0,06	0,15

(2) For cross-laminated deck plates, see Figure 1.3, shear deformations should be taken into account.

5.1.2 Concentrated vertical loads

(1) Loads should be considered at a reference plane in the middle of the deck plate.

(2) For concentrated loads an effective load area with respect to the middle plane of the deck plate should be assumed, see figure 5.1,
where:

b_w is the width of the loaded area on the contact surface of the pavement;

$b_{w,middle}$ is the width of the loaded area at the reference plane in the middle of the deck plate;

β is the angle of dispersion according to table 5.2.
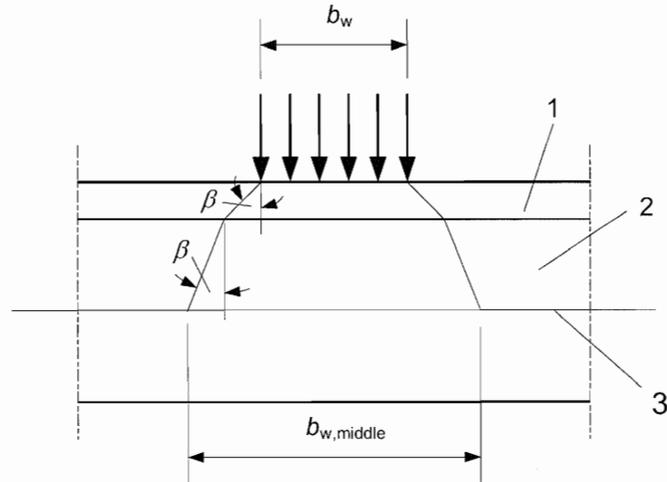
Key:
1   Pavement
2   Timber deck plate
3   Reference in middle of timber deck plate

Figure 5.1 – Dispersion of concentrated loads from contact area width b_w

Table 5.2 – Dispersion angle β of concentrated loads for various materials

Pavement (in accordance with EN 1991-2 clause 4.3.6)	45°
Boards and planks	45°
Laminated timber deck plates:	
– in the direction of the grain	45°
– perpendicular to the grain	15°
Plywood and cross-laminated deck plates	45°

5.1.3   Simplified analysis

(1) The deck plate may be replaced by one or several beams in the direction of the laminations with the effective width b_{ef} calculated as

$$b_{ef} = b_{w,middle} + a \tag{5.1}$$

where:

$b_{w,middle}$ should be calculated according to 5.1.2(2);

a should be taken from table 5.3.

Table 5.3 – Width *a* in m for determination of effective width of beam

Deck plate system	*a* m
Nail-laminated deck plate	0,1
Stress-laminated or glued laminated	0,3
Cross-laminated timber	0,5
Composite concrete/timber deck structure	0,6

5.2 Composite members

(1)P For composite action of deck plate systems, the influence of joint slip shall be taken into account.

NOTE: See clause 8.2

5.3 Timber-concrete composite members

(1) The concrete part should be designed according to EN 1992-2.

(2) The steel fasteners and the grooved connections should be designed to transmit all forces due to composite action. Friction and adhesion between wood and concrete should not be taken into account, unless a special investigation is carried out.

(3) The effective width of the concrete plate of composite timber beam/concrete deck structures should be determined as:

$$b_{ef,c} = b + b_{ef,1} + b_{ef,2} \tag{5.2}$$

where:

b is the width of the timber beam;

$b_{ef,1}, b_{ef,2}$ are the effective widths of the concrete flanges, as determined for a concrete T-section according to EN 1992-1-1, subclause 5.3.2.1.

(4)P For verification at ultimate limit state, cracks in the concrete plate shall be taken into account.

(5) The effect of concrete tension stiffening may be included. As a simple approach the stiffness of the cracked part of the concrete cross-section may be taken as 40 % of the stiffness in un-cracked condition. In such areas the need for an adequate crack distributing reinforcement should be observed.

Section 6 Ultimate limit states

6.1   Deck plates

6.1.1   System strength

(1) The relevant rules given in EN 1995-1-1 clause 6.7 apply

(2) The design bending and shear strength of the deck plate should be calculated as:

$$f_{m,d,deck} = k_{sys} f_{m,d,lam} \tag{6.1}$$

$$f_{v,d,deck} = k_{sys} f_{v,d,lam} \tag{6.2}$$

where:

$f_{m,d,lam}$ is the design bending strength of the laminations;

$f_{v,d,lam}$ is the design shear strength of the laminations;

k_{sys} is the system strength factor, see EN 1995-1-1. For decks in accordance to Fig. 1.2d EN 1995-1-1 figure 6.14 line 1 should be used.

For the calculation of k_{sys}, the number of loaded laminations should be taken as:

$$n = \frac{b_{ef}}{b_{lam}} \tag{6.3}$$

with:

b_{ef} is the effective width;;

b_{lam} is the width of the laminations.

(3) The effective width b_{ef} should be taken as (see figure 6.1):

$$b_{ef} = \frac{M_{max,beam}}{m_{max,plate}} \tag{6.4}$$

where:

$M_{max,beam}$ is the maximum bending moment in a beam representing the plate;

$m_{max,plate}$ is the maximum bending moment in the plate calculated by a plate analysis.

NOTE: In 5.1.3 a simplified method is given for the determination of the effective width.
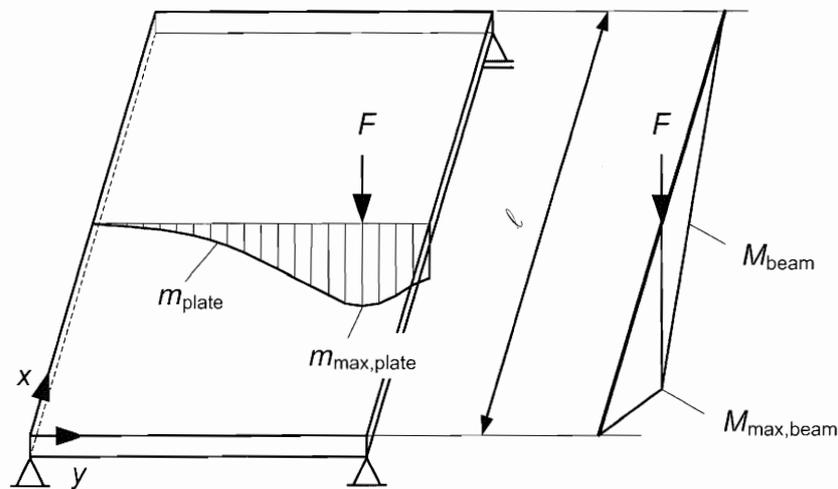
Figure 6.1 – Example of bending moment distribution in the plate for determination of effective width

6.1.2 Stress-laminated deck plates

(1)P The long-term pre-stressing forces shall be such that no inter-laminar slip occurs.

(2) The following requirement should be satisfied:

$$F_{v,Ed} \leq \mu_d \, \sigma_{p,min} \, h \tag{6.5}$$

where:

$F_{v,Ed}$ is the design shear force per unit length, caused by vertical and horizontal actions;

μ_d is the design value of coefficient of friction;

$\sigma_{p,min}$ is the minimum long-term residual compressive stress due to pre-stressing;

h is the thickness of the plate.

(3) The coefficient of friction should take into account the following:
– wood species;

– roughness of contact surface;

– treatment of the timber;

– residual stress level between laminations.

(4) Unless other values have been verified, the design static friction coefficients, μ_d, between softwood timber laminations, and between softwood timber laminations and concrete, should be taken from table 6.1. For moisture contents between 12 and 16 %, the values may be obtained by linear interpolation.

(5) In areas subjected to concentrated loads, the minimum long-term residual compressive stress, $\sigma_{p,min}$, due to pre-stressing between laminations should be not less than 0,35 N/mm^2.

(6) The long-term residual pre-stressing stress may normally be assumed to be greater than 0,35 N/mm^2, provided that:
– the initial pre-stress is at least 1,0 N/mm^2;

– the moisture content of the laminations at the time of pre-stressing is not more than 16%;

– the variation of in-service moisture content in the deck plate is limited by adequate protection, e.g. a sealing layer.

**Table 6.1 – Design values of coefficient of friction μ_d**

Lamination surface roughness	Perpendicular to grain		Parallel to grain	
	Moisture content $\leq 12\ \%$	Moisture content $\geq 16\ \%$	Moisture content $\leq 12\ \%$	Moisture content $\geq 16\ \%$
Sawn timber to sawn timber	0,30	0,45	0,23	0,35
Planed timber to planed timber	0,20	0,40	0,17	0,30
Sawn timber to planed timber	0,30	0,45	0,23	0,35
Timber to concrete	0,40	0,40	0,40	0,40

(7) The resulting pre-stressing forces should act centrally on the timber cross-section.

(8)P The compressive stress perpendicular to the grain during pre-stressing in the contact area of the anchorage plate shall be verified.

(9) The factor $k_{c,90}$ according to EN 1995-1-1 may be taken as 1,3.

(10) Not more than one butt joint should occur in any four adjacent laminations within a distance ℓ_1 given as

$$\ell_1 = \min \begin{cases} 2d \\ 30t \\ 1,2\ \text{m} \end{cases} \tag{6.6}$$

where:

d is the distance between the pre-stressing elements;

t is the thickness of the laminations in the direction of pre-stressing.

(11) In calculating the longitudinal strength of stress-laminated deck plates, the section should be reduced in proportion to the number of butt joints within a distance of 4 times the thickness of laminations in the direction of pre-stressing.
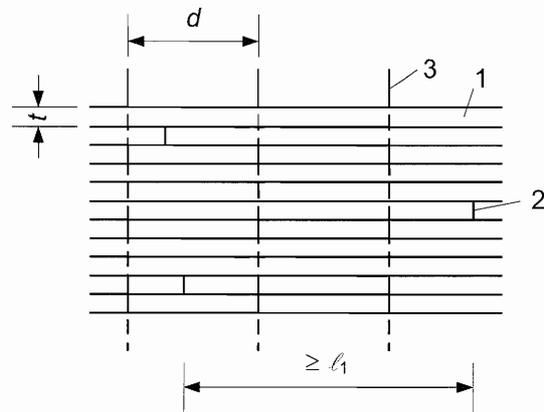
Key:
1   Lamination
2   Butt joint
3   Pre-stressing element

Figure 6.2 — Butt joints in stress-laminated deck plates

6.2   Fatigue

(1)P For structures or structural parts and connections that are subjected to frequent stress variations from traffic or wind loading, it shall be verified that no failure or major damage will occur due to fatigue.

NOTE 1: A fatigue verification is normally not required for footbridges.

NOTE 2: A simplified verification method is given in annex A (informative).

Section 7 Serviceability limit states

7.1 General

(1) In the calculations, mean values of density should be used.

7.2 Limiting values for deflections

NOTE: The range of limiting values for deflections due to traffic load only, for beams, plates or trusses with span ℓ is given in Table 7.1. The recommended values are underlined. Information on National choice may be found in the National annex.

Table 7.1 – Limiting values for deflections for beams, plates and trusses

Action	Range of limiting values
Characteristic traffic load	$\ell/400$ to $\ell/500$
Pedestrian load and low traffic load	$\ell/200$ to $\ell/400$

7.3 Vibrations

7.3.1 Vibrations caused by pedestrians

(1) For comfort criteria EN1990:2002/A1 applies.

(2) Where no other values have been verified, the damping ratio should be taken as:
- $\zeta = 0,010$ for structures without mechanical joints;

- $\zeta = 0,015$ for structures with mechanical joints.

NOTE 1: For specific structures, alternative damping ratios may be given in the National annex.

NOTE 2: A simplified method for assessing vibrations of timber bridges constructed with simply supported beams or trusses is given in Annex B.

7.3.2 Vibrations caused by wind

(1)P EN 1991-1-4 applies

Section 8 Connections

8.1 General

(1)P The following shall not be used in bridges:
- axially loaded nails;
- stapled connections;
- connections made with punched metal plate fasteners.

8.2 Timber-concrete connections in composite beams

8.2.1 Laterally loaded dowel-type fasteners

(1) The rope effect should not be used.

(2) Where there is an intermediate non-structural layer between the timber and the concrete (e.g. for formwork), see figure 8.1, the strength and stiffness parameters should be determined by a special analysis or by tests.
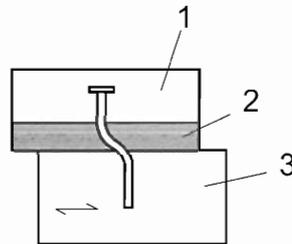
Key:
1 Concrete
2 Non-structural intermediate layer
3 Timber

Figure 8.1 – Intermediate layer between concrete and timber

8.2.2 Grooved connections

(1) For grooved connections, see figure 1.1, the shear force should be taken by direct contact pressure between the wood and the concrete cast in the groove.

(2) It should be verified that the resistance of the concrete part and the timber part of the connection is sufficient.

(3)P The concrete and timber parts shall be held together so that they can not separate.

(4) The connection should be designed for a tensile force between the timber and the concrete with a magnitude of:

$$F_{t,Ed} = 0,1 F_{v,Ed} \qquad\qquad (8.1)$$

where:

$F_{t,Ed}$ is the design tensile force between the timber and the concrete;

$F_{v,Ed}$ is the design shear force between the timber and the concrete.

Section 9 Structural detailing and control

(1)P The relevant rules given in EN 1995-1-1 Section 10 also apply to the structural parts of bridges, with the exception of clauses 10.8 and 10.9.

(2) Before attaching a seal layer on a deck plate, the deck system should be dry and the surface should satisfy the requirements of the seal layer.

Annex A (informative) Fatigue verification

A.1   General

(1) This simplified method is based on an equivalent constant amplitude fatigue loading, representing the fatigue effects of the full spectrum of loading events.

NOTE: More advanced fatigue verification for varying stress amplitude can be based on a cumulative linear damage theory (Palmgren-Miner hypothesis).

(2) The stress should be determined by an elastic analysis under the specified action.
The stresses should allow for stiff or semi-rigid connections and second order effects from deformations and distortions.

(3) A fatigue verification is required if the ratio κ given by expression (A.1) is greater than:
 – For members in compression perpendicular or parallel to grain: 0,6

 – For members in bending or tension: 0,2

 – For members in shear: 0,15

 – For joints with dowels: 0,4

 – For joints with nails: 0,1

 – Other joints: 0,15

where:

$$\kappa = \frac{\left|\sigma_{d,max} - \sigma_{d,min}\right|}{\dfrac{f_k}{\gamma_{M,fat}}} \tag{A.1}$$

$\sigma_{d,max}$ is the numerically largest design stress from the fatigue loading;

$\sigma_{d,min}$ is the numerically smallest design stress from the fatigue loading;

f_k is the relevant characteristic strength;

$\gamma_{M,fat}$ is the material partial factor for fatigue loading.

A.2   Fatigue loading

(1) A simplified fatigue load model is built up of reduced loads (effects of actions) compared to the static loading models. The load model should give the maximum and minimum stresses in the actual structural members.

(2) The fatigue loading from traffic should be obtained from the project specification in conjunction with EN 1991-2.

(3) The number of constant amplitude stress cycles per year, N_{obs}, should either be taken from table 4.5 of EN 1991-2 or, if more detailed information about the actual traffic is available, be taken as:

$$N_{obs} = 365\, n_{ADT}\, \alpha \tag{A.2}$$

where:

N_{obs} is the number of constant amplitude stress cycles per year;

n_{ADT} is the expected annual average daily traffic over the lifetime of the structure; the value of n_{ADT} should not be taken less than 1000;

α is the expected fraction of observed heavy lorries passing over the bridge, see EN 1991-2 clause 4.6 (e.g. $\alpha = 0,1$);

A.3 Fatigue verification

(1) Unless the verification model is defined below or by special investigations, the ratio κ should be limited to the value defined in the previous clause A1(3).

(2) For a constant amplitude loading the fatigue verification criterion is:

$$\sigma_{d,max} \leq f_{fat,d} \tag{A.3}$$

where:

$\sigma_{d,max}$ is the numerically largest design stress from the fatigue loading;

$f_{fat,d}$ is the design value of fatigue strength.

(3) The design fatigue strength should be taken as:

$$f_{fat,d} = k_{fat} \frac{f_k}{\gamma_{M,fat}} \tag{A.4}$$

where:

f_k is the characteristic strength for static loading;

k_{fat} is a factor representing the reduction of strength with number of load cycles.

(4) The value of k_{fat} should be taken as:

$$k_{fat} = 1 - \frac{1-R}{a(b-R)} \log(\beta N_{obs} t_L) \geq 0 \tag{A.5}$$

where:

$$R = \sigma_{d,min} / \sigma_{d,max} \qquad \text{with } -1 \leq R \leq 1; \tag{A.6}$$

$\sigma_{d,min}$ is the numerically smallest design stress from the fatigue loading;

$\sigma_{d,max}$ is the numerically largest design stress from the fatigue loading;

N_{obs} is the number of constant amplitude stress cycles as defined above;

t_L is the design service life of the structure expressed in years according to EN 1990:2002 (e.g. 100 years); β is a factor based on the damage consequence for the actual structural component;

a, b are coefficients representing the type of fatigue action according to table A.1.

The factor β should be taken as:
– Substantial consequences: $\beta = 3$

– Without substantial consequences: $\beta = 1$

26

Table A.1 – Values of coefficients a and b

	a	b
Timber members in		
– compression, perpendicular or parallel to grain	2,0	9,0
– bending and tension	9,5	1,1
– shear	6,7	1,3
Connections with		
– dowels with $d \leq 12$ mm [a]	6,0	2,0
– nails	6,9	1,2
[a]The values for dowels are mainly based on tests on 12 mm tight-fitting dowels. Significantly larger diameter dowels or non-fitting bolts may have less favourable fatigue properties.		

Annex B (informative) Vibrations caused by pedestrians

B.1   General

(1) The rules given in this annex apply to timber bridges with simply supported beams or truss systems excited by pedestrians.

NOTE: Corresponding rules will be found in future versions of EN 1991-2.

B.2   Vertical vibrations

(1) For one person crossing the bridge, the vertical acceleration $a_{vert,1}$ in m/s^2 of the bridge should be taken as:

$$a_{vert,1} = \begin{cases} \dfrac{200}{M\,\zeta} & \text{for } f_{vert} \leq 2,5 \text{ Hz} \\[2mm] \dfrac{100}{M\,\zeta} & \text{for } 2,5 \text{ Hz} < f_{vert} \leq 5,0 \text{ Hz} \end{cases} \qquad (B.1)$$

where:

M is the total mass of the bridge in kg, given by $M = m\,\ell$;

ℓ is the span of the bridge;

m is the mass per unit length (self-weight) of the bridge in kg/m;

ζ is the damping ratio;

f_{vert} is the fundamental natural frequency for vertical deformation of the bridge.

(2) For several persons crossing the bridge, the vertical acceleration $a_{vert,n}$ in m/s^2 of the bridge should be calculated as:

$$a_{vert,n} = 0,23\, a_{vert,1}\, n\, k_{vert} \qquad (B.2)$$

where:

n is the number of pedestrians;

k_{vert} is a coefficient according to figure B.1;

$a_{vert,1}$ is the vertical acceleration for one person crossing the bridge determined according to expression (B.1).

The number of pedestrians, n, should be taken as:

− $n = 13$ for a distinct group of pedestrians;

− $n = 0,6\,A$ for a continuous stream of pedestrians.

where A is the area of the bridge deck in m^2.

(3) If running persons are taken into account, the vertical acceleration $a_{vert,1}$ in m/s^2 of the bridge caused by one single person running over the bridge, should be taken as:

$$a_{vert,1} = \frac{600}{M\,\zeta} \qquad \text{for } 2,5 \text{ Hz} < f_{vert} \leq 3,5 \text{ Hz} \qquad (B.3)$$

B.3   Horizontal vibrations

(1) For one person crossing the bridge the horizontal acceleration $a_{hor,1}$ in m/s^2 of the bridge should be calculated as:

$$a_{hor,1} = \frac{50}{M\,\zeta} \qquad\qquad \text{for } 0{,}5 \text{ Hz} \leq f_{hor} \leq 2{,}5 \text{ Hz} \tag{B.4}$$

where f_{hor} is the fundamental natural frequency for horizontal deformation of the bridge.

(2) For several persons crossing the bridge, the horizontal acceleration $a_{hor,n}$ in m/s^2 of the bridge should be calculated as:

$$a_{hor,n} = 0{,}18\,a_{hor,1}\,n\,k_{hor} \tag{B.5}$$

where:

k_{hor} is a coefficient according to figure B.2.

The number of pedestrians, n, should be taken as:

- $n = 13$ for a distinct group of pedestrians;

- $n = 0{,}6\,A$ for a continuous stream of pedestrians,

where A is the area of the bridge deck in m^2.
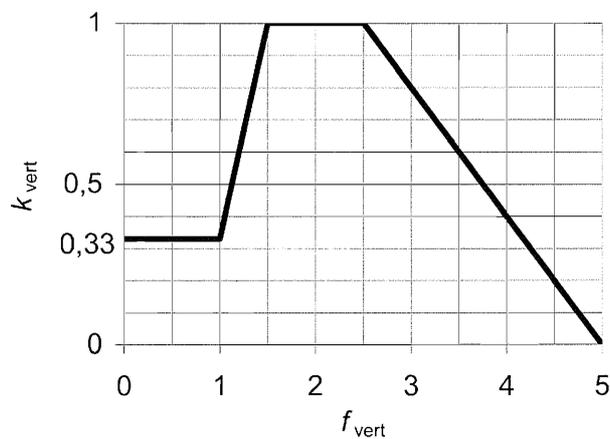
Figure B.1 – Relationship between the vertical fundamental natural frequency f_{vert} and the coefficient k_{vert}
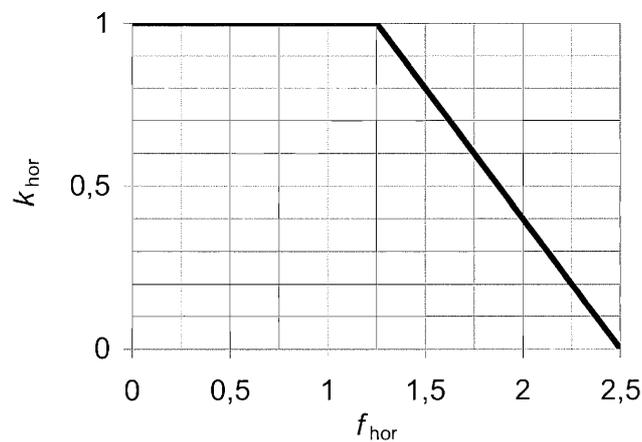
Figure B.2 – Relationship between the horizontal fundamental natural frequency f_{hor} and the coefficient k_{hor}