Actions on bridge decks and piers (EN 1991)

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CONTENTS OF THE PRESENTATION

Brief review of the structure of EN 1991

- Selfweight and imposed loads
- Wind (Example of application)
- Thermal actions
- Actions during execution
- Settlements
- Accidental actions (impact loads)

Traffic loads

- Brief review
- General Load Models
- Fatigue Load Model 3 (Example of application)

Combinations of actions

- ULS and SLS
- *Launching*
- *Seismic*

ACTIONS

It is reminded that according to EN 1991 the following should be considered:

- Selfweight and imposed loads
- Wind
- Thermal actions
- Actions during execution
- Accidental actions (impact loads)
- Traffic loads

There are also other actions described in EN 1991, such as fire and snow loads, which are considered as irrelevant for the example of bridge structure presented. Additional actions are foreseen in other EN Eurocodes, namely:

- Concrete creep and shrinkage (EN 1992)
- Settlements and earth pressures (EN 1997)
- Seismic actions (EN 1998)

PARTS AND IMPLEMENTATION OF EN 1991

Part of Eurocode 1 : Actions on structures	Title (Subject)	Issued
EN 1991-1-1	General actions – Densities, self-weight, imposed loads for buildings	April 2002
EN 1991-1-2	General actions – Actions on structures exposed to fire	November 2002
EN 1991-1-3	General actions – Snow loads	July 2003
EN 1991-1-4	General actions – Wind actions	April 2005
EN 1991-1-5	General actions – Thermal actions	November 2003
EN 1991-1-6	General actions – Actions during execution	June 2005
EN 1991-1-7	General actions – Accidental actions	July 2006
EN 1991-2	Traffic loads on bridges	September 2003
EN 1991-3	Actions induced by cranes and machinery	July 2006
EN 1991-4	Silos and tanks	May 2006

EN 1991-1-1: DENSITIES, SELF-WEIGHT, IMPOSED LOADS FOR BUILDINGS

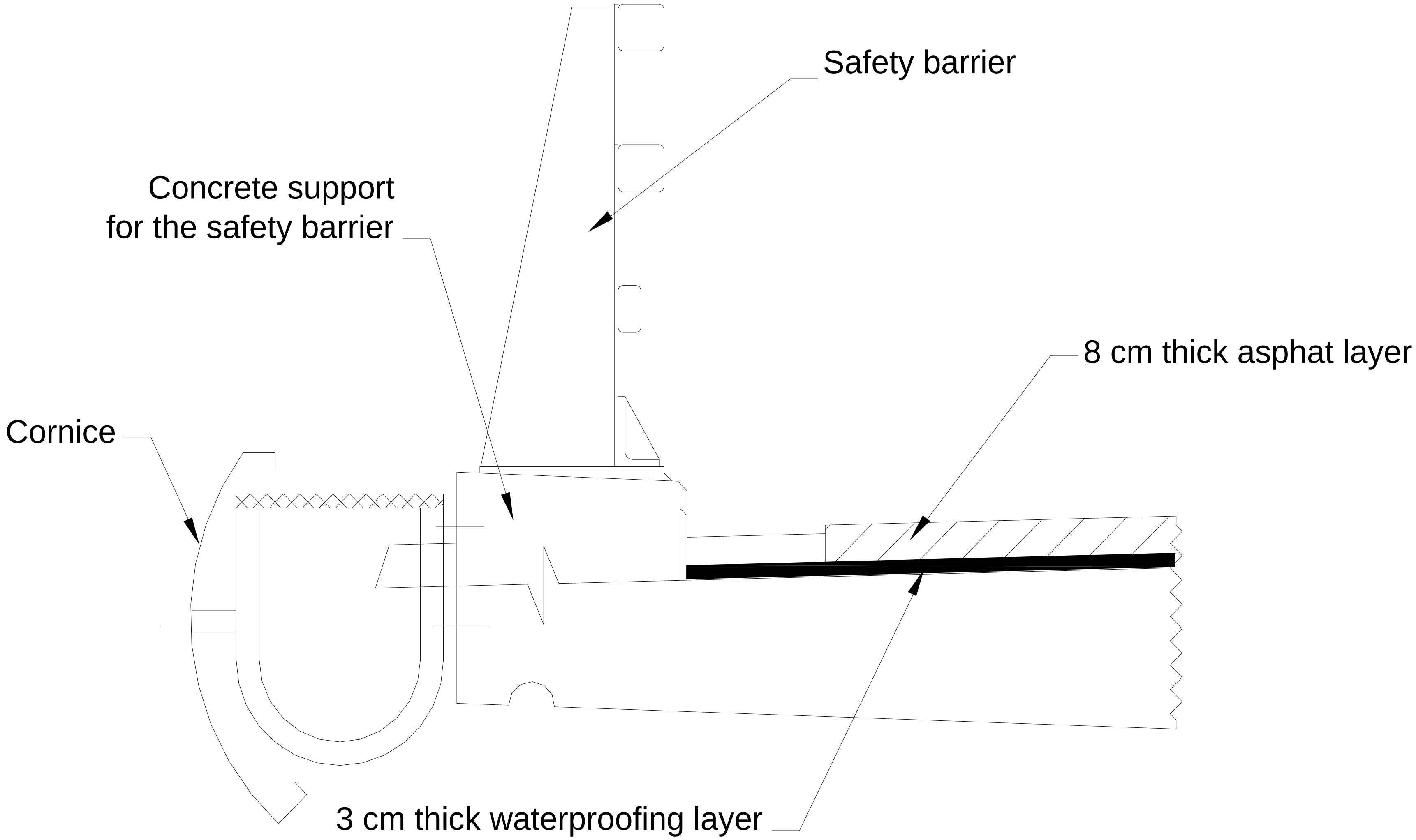
- Forward
- Section 1 – General
- Section 2 – Classification of actions
- Section 3 – Design situations
- Section 4 – Densities of construction and stored materials
- Section 5 – Self-weight of construction works
- Section 6 – Imposed loads on buildings
- Annex A (informative) – Tables for nominal density of construction materials, and nominal density and angles of repose for stored materials.
- Annex B (informative) – Vehicle barriers and parapets for car parks

ACTIONS : SELFWEIGHT

Structural parts:
The density of structural steel is taken equal to 77 kN/m^3 *[EN 1991-1-1, Table A.4].* The density of reinforced concrete is taken equal to 25 kN/m^3 *[EN 1991-1-1, Table A.1]*. The selfweight is determined based on the dimensions of the structural elements. For the longitudinal bending global analysis the selfweight of the in-span transverse cross girder is modelled by a uniformly distributed load of 1,5 kN/m applied to each main girder (about 10% of its own weight)

Non-structural parts:
The density of the waterproofing material and of the asphalt is taken as equal to 25 kN/m kN/m3 *[EN 1991-1-1, Table A.6]*.
According to *[EN 1991-1-1, 5.2.3(3)]* it is recommended that the nominal value of the waterproofing layer and the asphalt layer is multiplied by +/-20% (if the post-execution coating is taken into account in the nominal value) and by +40% / -20% (if this is not the case)

ACTIONS : SELFWEIGHT



ACTIONS : SELFWEIGHT

Non-structural parts (cont.):

The key data to evaluate the selfweight are summarized in the following table:

Item	Characteristics	Maximum multiplier	Minimum multiplier
Concrete support of the safety barrier	Area 0,5 x 0,2 m	1,0	1,0
Safety barrier	65 kg/ml	1,0	1,0
Cornice	25 kg/ml	1,0	1,0
Waterproofing layer	3 cm thick	1,2	0,8
Asphalt layer	8 cm thick	1,4	0,8

ACTIONS : SELFWEIGHT

Non-structural parts (cont.):

The values of selfweight (as uniformly distributed load per main steel girder) are summarized in the following table:

Item	q_{nom} **(kN/ml)**	q_{max} **(kN/ml)**	q_{min} **(kN/ml)**
Concrete support of the safety barrier	2,5	2,5	2,5
Safety barrier	0,638	0,638	0,638
Cornice	0,245	0,245	0,245
Waterproofing layer	4,2	5,04	3,36
Asphalt layer	11,0	15,4	8,8
TOTAL	**18,58**	**23,82**	**15,54**

EN 1991-1-4: WIND ACTIONS

- **Forward**
- **Section 1 – General**
- **Section 2 – Design situations**
- **Section 3 – Modelling of wind actions**
- **Section 4 – Wind velocity and velocity pressure**
- **Section 5 – Wind actions**
- **Section 6 – Structural factor** $c_s c_d$
- **Section 7 – Pressure and force coefficients**
- **Section 8 – Wind actions on bridges**
- **Annex A (informative) – Terrain effects**
- **Annex B (informative) – Procedure 1 for determining the structural factor** $c_s c_d$
- **Annex C (informative) – Procedure 2 for determining the structural factor** $c_s c_d$
- **Annex D (informative) –** $c_s c_d$ **values for different types of structures**
- **Annex E (informative) – Vortex shedding and aeroelastic instabilities**
- **Annex F (informative) – Dynamic characteristics of structures**

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

Courtesy of GEFYRA S.A. (Rion – Antirion Bridge, Greece)

1. Introduction

The scope of the example handled is to present the wind actions and effects usually applied on a bridge, to both deck and piers. The following cases have been handled in the written text:

- Bridge during its service life, without traffic
- Bridge during its service life, with traffic
- Bridge under construction (finished and most critical case)

Two alternative pier dimensions:

- **Squat piers of 10 m height** and rectangular cross section 2,5 m x 5,0 m
- **"High" piers of 40 m height** and circular cross section of 4 m diameter

2. Brief description of the procedure

The general expression of a wind force *Fw* acting on a structure or structural member is given by the following formula *[Eq. 5.3]*:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$$

Where:

$c_s.c_d$ is the structural factor *[6]* (= 1,0 when no dynamic response procedure is needed *[8.2(1)]*)

c_f is the force coefficient *[8.3.1, 7.6 and 7.13, 7.9.2, respectively, for the deck, the rectangular and the cylindrical pier]*

$q_p(z_e)$ is the peak velocity pressure *[4.5]* at reference height *ze*, which is usually taken as the height *z* above the ground of the C.G. of the structure subjected to the wind action

A_{ref} is the reference area of the structure *[8.3.1, 7.6, 7.9.1, respectively, for the deck, the rectangular and the cylindrical pier]*

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

2. Brief description of the procedure (continued)

Short summary of the procedure:

To determine the wind actions on bridge decks and piers, it seems convenient to follow successively the following steps (velocities → pressures→forces):

- Determine v_b (by choosing $v_{b,0}$, c_{dir}, c_{season} and c_{prob}, if relevant); q_b may also be determined at this stage
- Determine $v_m(z)$ (by choosing terrain category and reference height z to evaluate $c_r(z)$ and $c_o(z)$)
- Determine $q_p(z)$ (either by choosing directly $c_e(z)$, *where possible, either by evaluating* $I_v(z)$, after choosing $c_o(z)$)
- Determine F_W (after evaluating A_{ref} and by choosing c_f *and* $c_s.c_d$, if relevant)

2. Brief description of the procedure (continued)

The basic wind velocity *vb* is expressed by the formula *[4.1]*:

$v_b = (c_{prob}) . c_{dir} . c_{season} . v_{b,0}$

Where:

v_b is the **basic wind velocity**, defined at 10 m above ground of terrain category II

$v_{b,0}$ is the **fundamental value of the basic wind velocity**, defined as the characteristic 10 minutes mean wind velocity (irrespective of wind direction and season of the year) at 10 m above ground level in open country with low vegetation and few isolated obstacles (distant at least 20 obstacle heights)

c_{dir} is the **directional factor**, which may be an NDP; the recommended value is 1,0

c_{season} is the **season factor**, which may be an NDP; the recommended value is 1,0

2. Brief description of the procedure (continued)

In addition to that a **probability factor** c_{prob} should be used, in cases where the return period for the design defers from T = 50 years.
This is usually the case, when the construction phase is considered.
Quite often also for bridges T = 100 is considered as the duration of the design life, which should lead to c_{prob} > 1,0.
The expression of c_{prob} is given in the following formula *[4.2]*, in which the values of *K* and *n* are NDPs; the recommended values are 0,2 and 0,5, respectively:

$$c_{\text{prob}} = \left(\frac{1 - K \cdot \ln(-\ln(1 - p))}{1 - K \cdot \ln(-\ln(0{,}98))} \right)^n$$

2. Brief description of the procedure (continued)

The peak velocity pressure $q_p(z)$ at height z, includes the mean and the short-term (turbulent) fluctuations and is expressed by the formula *[4.8]*:

$$q_{\mathrm{p}}(z) = [1 + 7 \cdot I_{\mathrm{v}}(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_{\mathrm{m}}^{2}(z) = c_{\mathrm{e}}(z) \cdot q_{\mathrm{b}}$$

Where:

ρ is the **air density** (which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms; the recommended value used is **1,25 kg/m^3**

$v_m(z)$ is the **mean wind velocity** at a height z above the ground *[4.3]*

$I_v(z)$ is the **turbulence intensity** at height z, defined *[4.4(1)]* as the ratio of the standard deviation of the turbulence divided be the mean velocity, and is expressed by the following formula *[4.7]*

$c_e(z)$ is the **exposure factor** at a height z

2. Brief description of the procedure (continued)

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_o(z) \cdot \ln(z/z_0)} \qquad \text{για} \qquad z_{min} \le z \le z_{max}$$

$$I_v(z) = I_v(z_{min}) \qquad \text{για} \qquad z < z_{min}$$

Where:

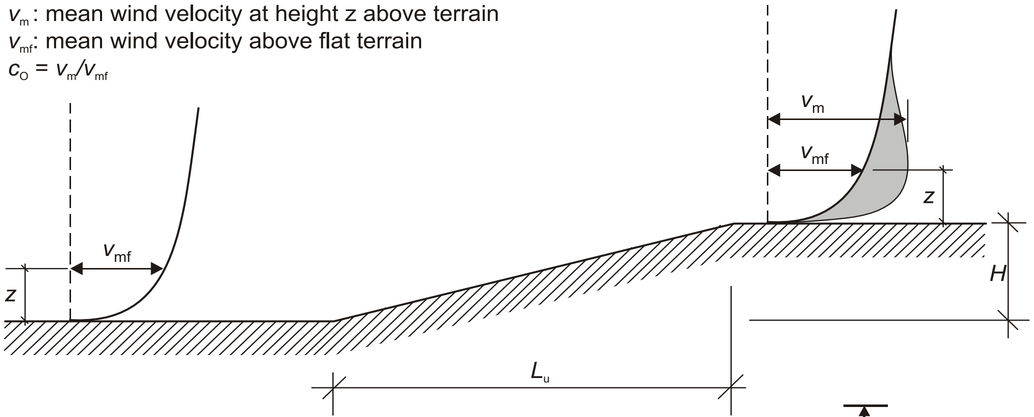
k_I is the **turbulence factor** (NDP value). The recommended value, used in the example, is 1,0

$c_o(z)$ is the **oreography factor** *[4.3.3]*

z_0 is the **roughness length** *[Table 4.1]*

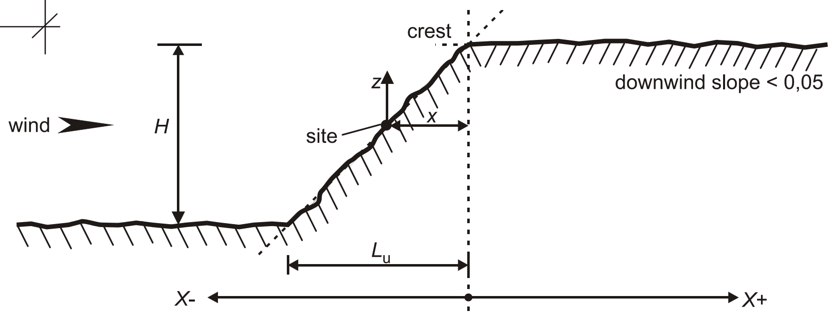
2. Brief description of the procedure (continued)

v_m : mean wind velocity at height z above terrain
v_{mf}: mean wind velocity above flat terrain
$c_O = v_m/v_{mf}$



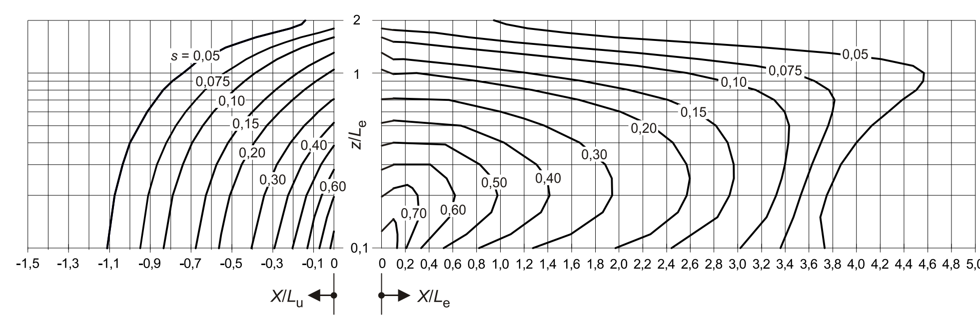
[Fig. A.1]

$$c_O = v_m/v_{mf}$$



[Fig. A.2]

$$c_O = c_O(s)$$



2. Brief description of the procedure (continued)

The mean wind velocity *vm (z)* is expressed by the formula *[4.3]*:

$v_m (z) = c_r (z) \,.\, c_o (z) \,.\, v_b$

Where:

$c_r(z)$ is the roughness **factor**, which may be an NDP, and is recommended to be determined according to the following formulas *[4.3.2]*:

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \qquad \text{for} \qquad z_{min} \le z \le z_{max}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for} \qquad z \le z_{min}$$

2. Brief description of the procedure (continued)

Where:

z_0 is the **roughness length** *[Table 4.1]*
k_r **terrain factor** depending on the roughness length and evaluated according the following formula *[4.5]*:

$$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}} \right)^{0{,}07}$$

with:

$z_{0,II}$ = 0,05 m (terrain category II, *[Table 4.1]*)
z_{min} is the minimum height defined in *[Table 4.1]*
z_{max} is to be taken as 200m

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

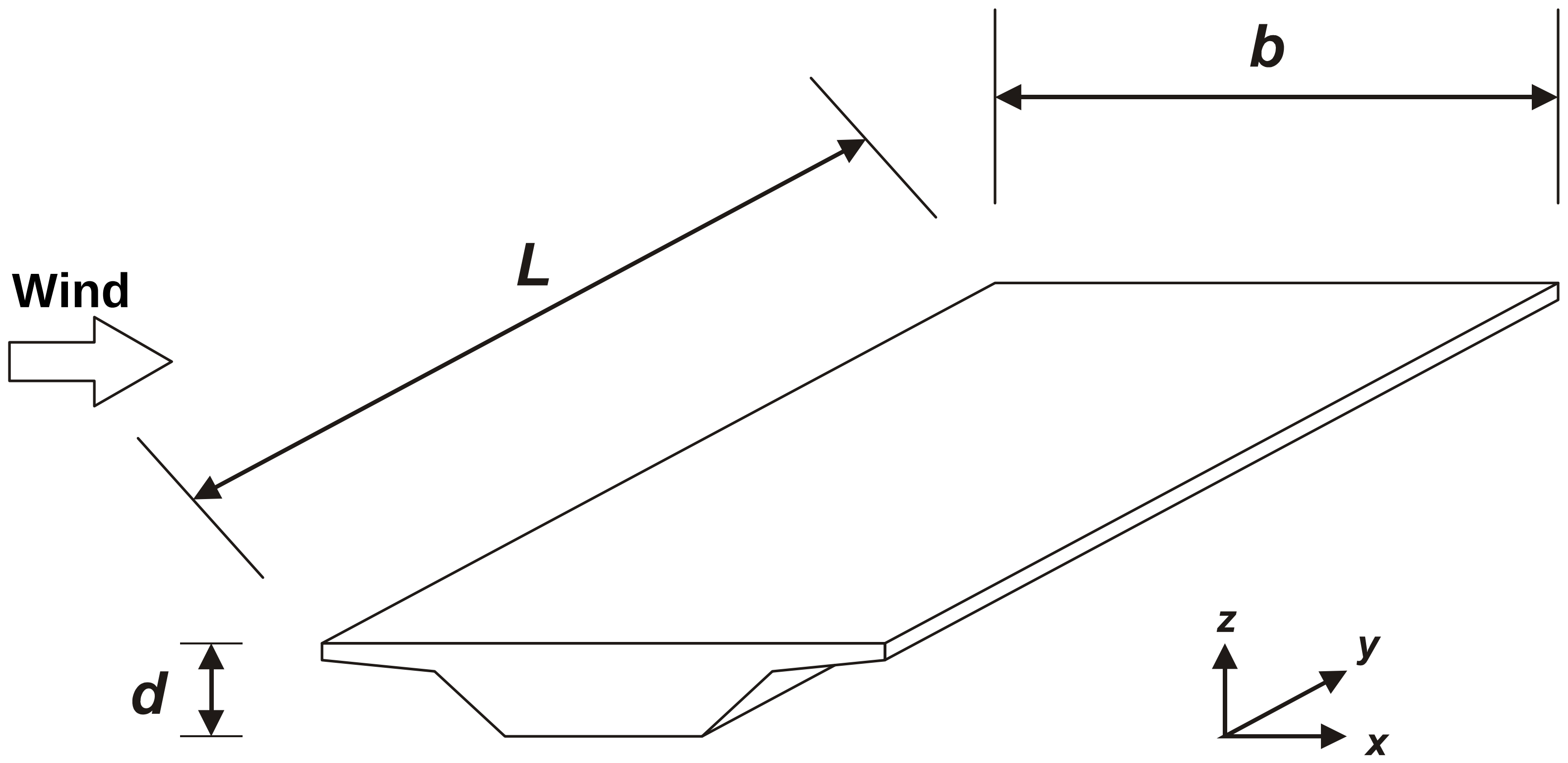


Fig. 8.2 of EN 1991-1-4 (Directions of wind actions on bridges)

3. Numerical application

3.1 Bridge during its service life, without traffic (“high” pier z = 40 m, wind transversally to the deck)

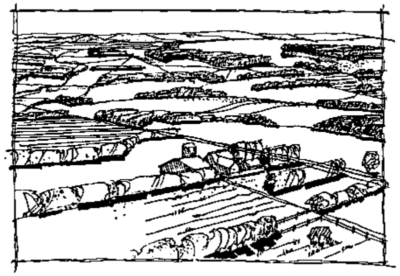
The fundamental wind velocity $v_{b,0}$ is an NDP to be determined by each Member State (given in the form of zone/isocurves maps, tables etc.). For the purpose of this example the value $v_{b,0}$ = 26 m/s (= v_b, since in this case it is considered that c_{dir} = *1,0* and c_{season} = 1,0)

The corresponding (basic velocity) pressure may also be computed, according to *[Eq. 4.10]*:

q_b = ½ x 1,25 x 26^2 = 422,5 N/m^2 (Pa)

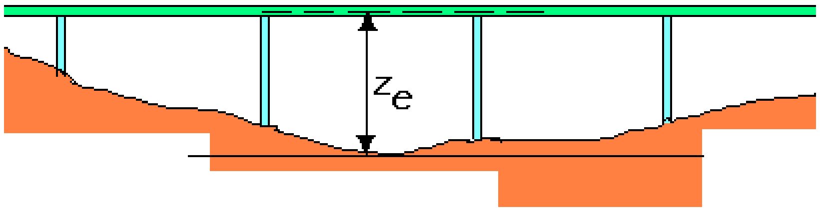
3. Numerical application (cont.)

In the present example a very flat valley will be considered with a roughness category II :
(low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights)



Concerning the reference height of the deck *ze* it may be considered more or less as equal to the mean distance *z* between the centre of the bridge deck and the soil surface *[8.3.1(6)]*

$$z_e = z$$



3. Numerical application (cont.)

For terrain category II :

Terrain category	z_0 (m)	z_{min} (m)
0	0,003	1
I	0,01	1
II	0,05	2
III	0,3	5
IV	1,0	10

thus: $$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}} \right)^{0{,}07} = 0{,}19 \cdot \left(\frac{0{,}05}{0{,}05} \right)^{0{,}07} = 0{,}19$$

and: $$c_r(40) = 0{,}19 \cdot \ln\left(\frac{40{,}00}{0{,}05} \right) = 0{,}19.\ln 800 = 0{,}19.6{,}6846 = 1{,}27$$

3. Numerical application (cont.)

For a flat valley the oreography factor $c_o(40) = 1{,}0$. Hence:

$v_m(40) = 1{,}27 \times 1{,}0 \times 26 = 33{,}02$ m/s ≈ 33 m/s

The turbulence intensity is:

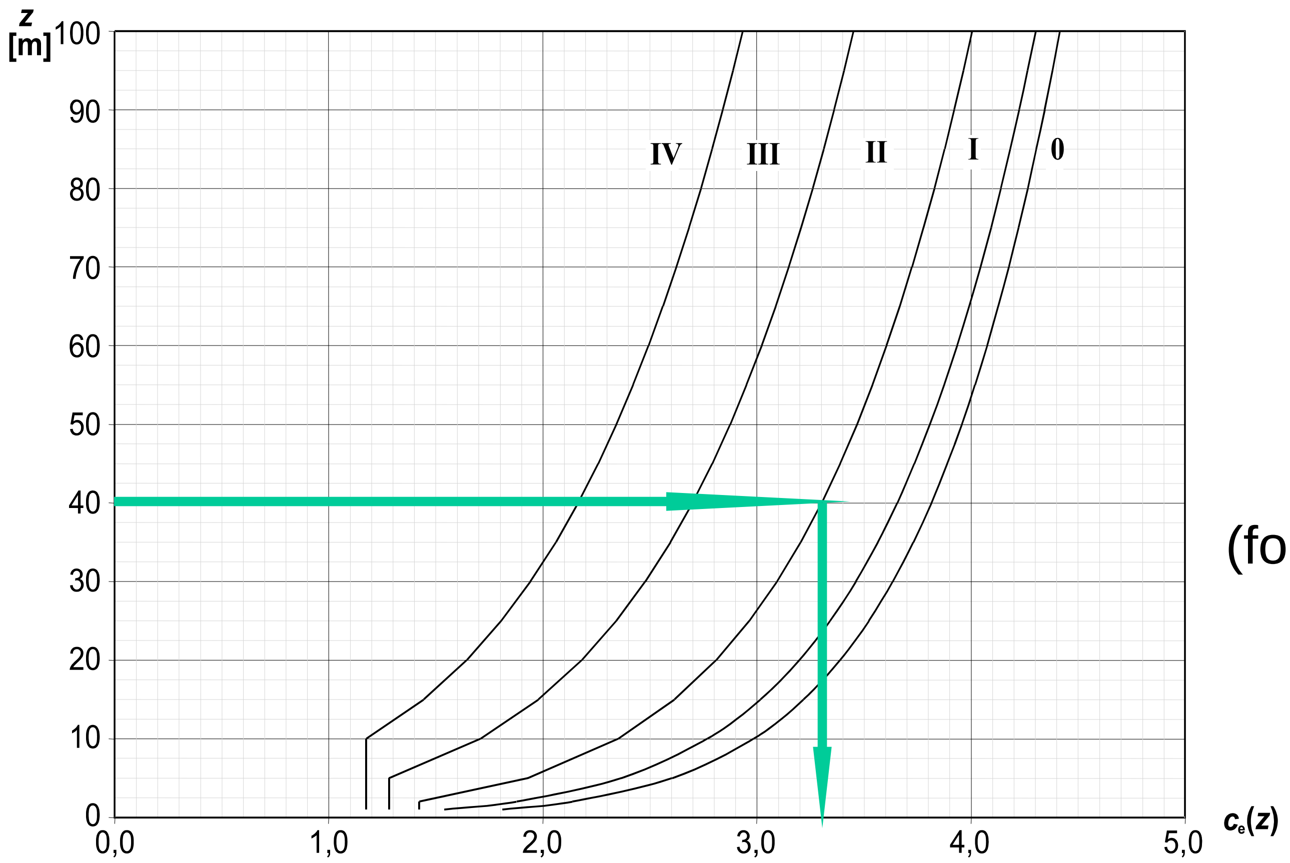
$$I_v(40) = \frac{1{,}0}{1{,}0 x \ln(40/0{,}05)} = \frac{1}{6{,}6846} = 0{,}15$$

And

$$q_p(40) = [1 + 7 \cdot 0{,}15)] x \frac{1}{2} x 1{,}25 x 33^2 = 2{,}05 x 680{,}6 = 1395{,}28$$ in N/m²

$c_e(40) = 2{,}05 \times 1{,}27^2 \times 1{,}0^2 = 2{,}05 \times 1{,}61 \times 1{,}0 = 3{,}30$
(= 1395,28 / 422,5 = $q_p(40)$ / q_b , *[Eq. 4.9]*)

3. Numerical application (cont.)



[Fig. 4.2]

Exposure coefficient $c_e(z)$ (for c_o=1,0 , k_I=1,0)

3. Numerical application (cont.)

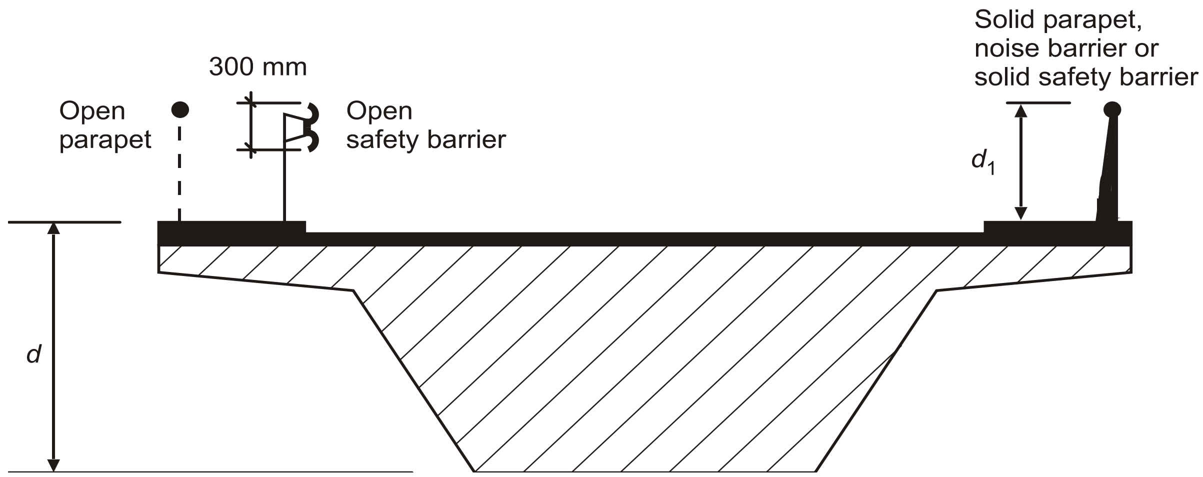
Further calculations are needed to determine the wind force on the deck *[5.3]*.

$$F_{\mathrm{w}} = c_{\mathrm{s}} c_{\mathrm{d}} \cdot c_{\mathrm{f}} \cdot q_{\mathrm{p}}(z_{\mathrm{e}}) \cdot A_{\mathrm{ref}}$$

Both the force coefficient c_f and the reference area A_{ref} of the bridge deck *[8.3.1]* depend on the width to (total) depth ratio b/d_{tot} of the deck, where d_{tot} represents the depth of the parts of the deck which are considered to be subjected to the wind pressure.

In the case of the bridge in service, without consideration of the traffic, according to *[8.3.1(4)* and *Table 8.1]*, d_{tot} is the sum of the projected (windward) depth of the structure, including the projecting solid parts, such as footway or safety barrier base, plus 0,3m for the open safety barrier BN4 in each side of the deck

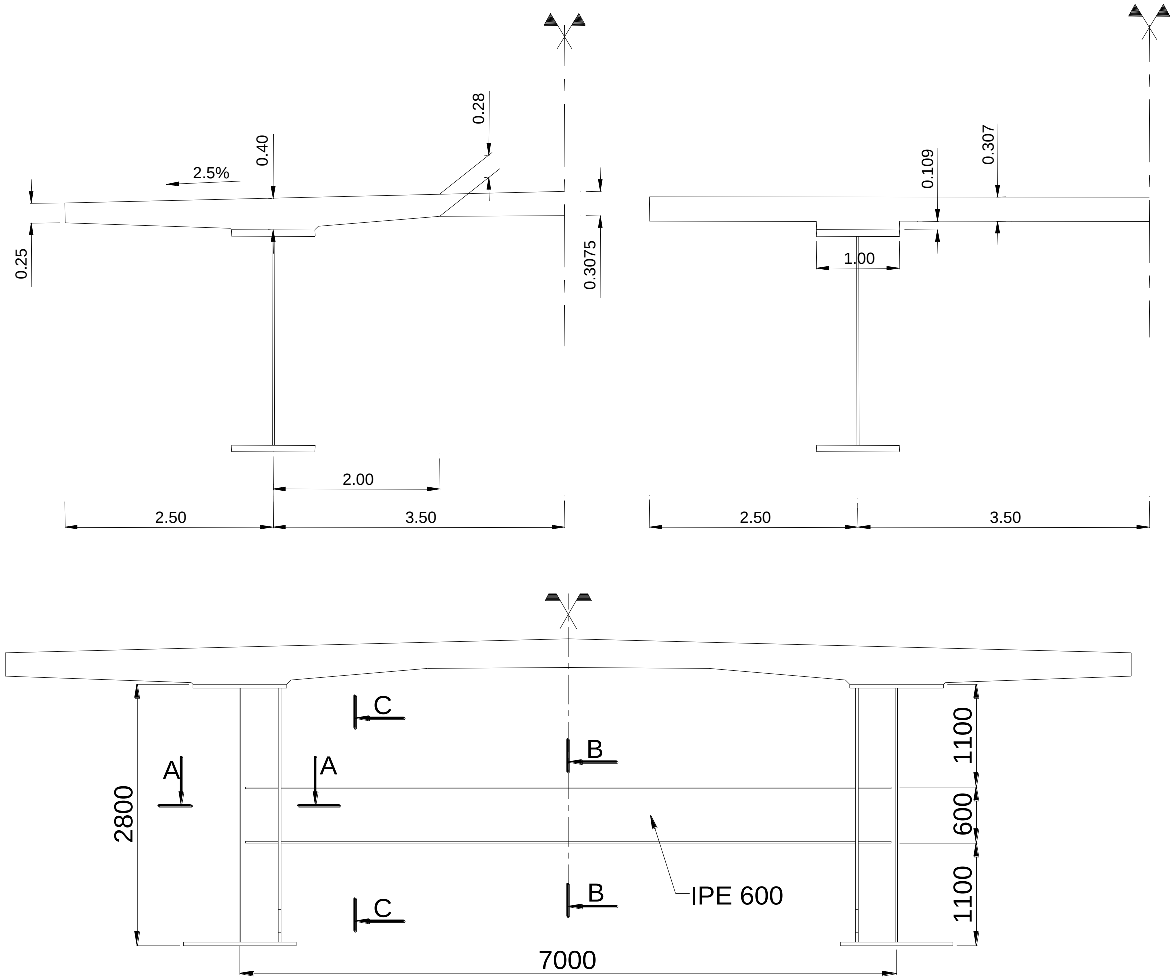
3. Numerical application (cont.)



Road restraint system	on one side	on both sides
Open parapet or open safety barrier	d + 0,3 m	d + 0,6 m
Solid parapet or solid safety barrier	$d + d_1$	$d + 2d_1$
Open parapet and open safety barrier	d + 0,6 m	d + 1,2 m

[Fig 8.5 & Table 8.1] Depth d_{tot} to be used for $A_{ref,x}$

3. Numerical application (cont.)



3. Numerical application (cont.)

Consequently:

d_{tot} = 2,800 + 0,400 – 0,025 x 2,500 + 0,200 + 2 x 0,300 = 3,1375 + 0,200 + 0,600 = 3,9375 ≈ 4,00 m

Hence:

b/d_{tot} = 12,00 / 4,00 = 3 (12,00 / 3,94 ≈ 3,05)

A_{ref} = d_{tot} . L = 4,00 x 200,00 = 800,00 m^2

$c_{fx,0}$ ≈ 1,55 *[Fig. 8.3]*

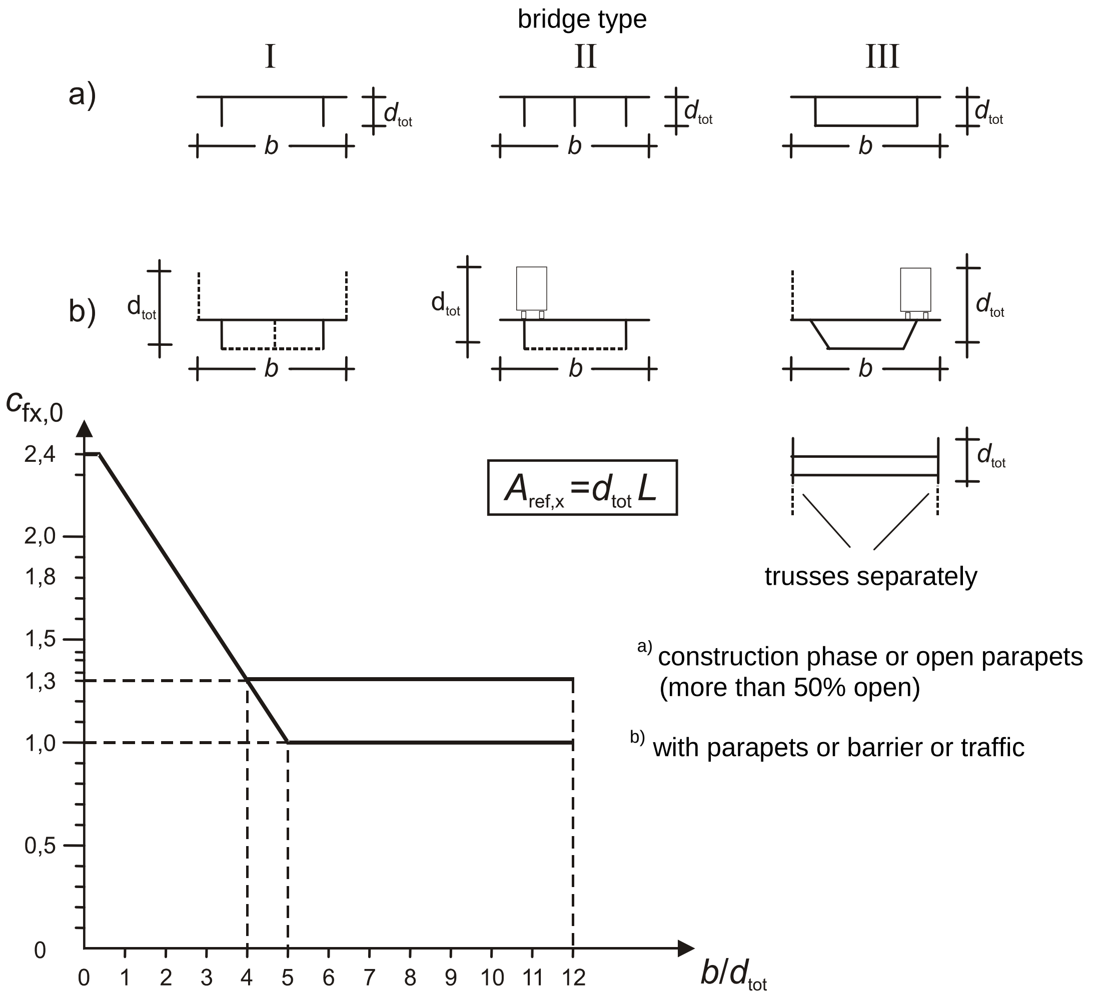
c_{fx} = $c_{fx,0}$ ≈ 1,55 *[Eq. 8.1]*

Finally:

$$F_w = 1{,}0x1{,}55x1395{,}28x800{,}00 = 2162{,}68x800{,}00 = 1730147 \text{ N} \approx 1730 \text{ kN}$$

Or "wind load" in the transverse (x-direction): w = 1730/200 ≈ 8,65 kN/m

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS



[Fig. 8.3] Force coefficient $c_{fx,0}$ for bridges

3. Numerical application (cont.)

Simplified Method *[8.3.2]*

Formula *[5.3]* is slightly modified as follows:

$$F_w = 1/2 \cdot \rho \cdot v_b{}^2 . C . A_{ref,x}$$

Where the force factor $C = c_e \, . \, c_{f,x}$ is given in *[Tab. 8.2]*

***b/d*tot**	**$z_e \leq 20$ m**	**$z_e = 50$ m**
≤ 0,5	6,7	8,3
≥ 4,0	3,6	4,5

This table is based on the following assumptions :

– terrain category II according to Table 4.1

– force coefficient $c_{f,x}$ according to 8.3.1 (1)

– c_o=1,0

– k_l=1,0

For intermediate values of b/d_{tot}, and of z_e linear interpolation may be used

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

3. Numerical application (cont.)

Simplified Method *[8.3.2]* (cont.)

By double interpolation, since 20 m < (z_e =) 40 m < 50 m and 0,5 < (b/d_{tot}) = 3,0 < 4,0 one gets $C = 5,23$

Using the interpolated value of C one gets:

$F_W = 0,5 \times 1,25 \times 262 \times 5,23 \times 800,00 = 2209,67 \times 800,00 = 1767740$ N
N ≈ 1768 kN

which is almost identical (a bit greater) than the "exact" value 1730 kN

3. Numerical application (cont.)

3.2 Bridge during its service life, with traffic ("high" pier z = 40 m, wind transversally to the deck)

The magnitude which is differentiated, compared to the case without traffic, is the reference depth *dtot* of exposure on wind action transversally to the deck. In that case:

d_{tot} = 3,1375 + 0,200 + 2,0 = 5,3375 ≈ 5,34 m

and

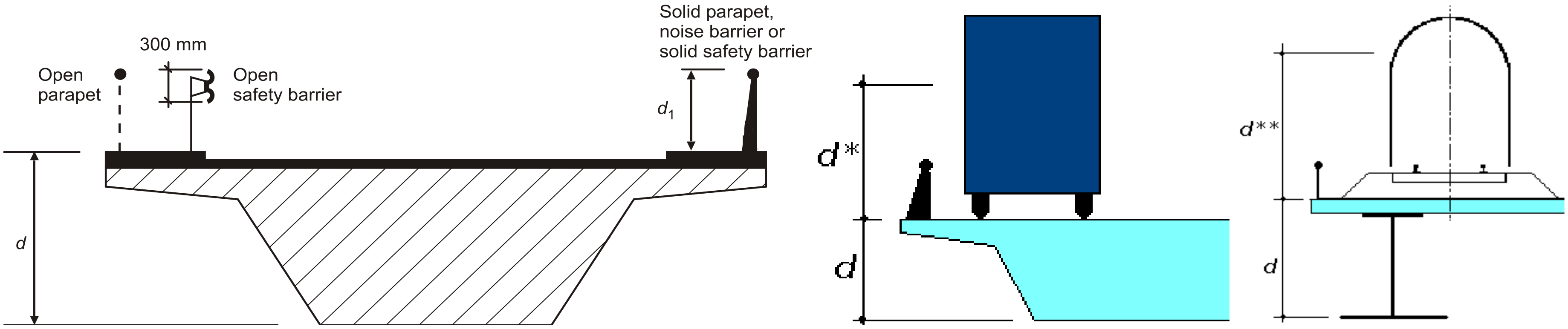
b/d_{tot} = 12,00/5,34 = 2,25, A_{ref} = 5,34 x 200,00 = 1068 m^2, $c_{fx} = c_{fx,0}$ ≈ 1,83

Hence:

$$F_w = 1{,}0x1{,}83x1395{,}28x1068{,}00 = 2553{,}36x1068{,}00 = 2726991 \text{ N} \approx 2727 \text{ kN}$$

Or "wind load" in the transverse (x-direction): w ≈ 13,64 kN/m

3. Numerical application (cont.)



Additional heights for the calculation of $A_{ref,x}$ (d* = 2 m ; d** = 4 m)

for bridges during their service life with traffic

3. Numerical application (cont.)

3.3 Bridge under construction (launched steel alone - cantilever at P2; “high” pier z = 40 m, wind transversally to the deck)

It has been agreed to use the value v_b= 50 km/h (= 50/3,6 =13,89 ≈ 14 m/s)

More generally, given that the construction phase has a limited duration and subsequently the associated return period of the actions considered is lesser than the service design life of the structure, c_{prob} may be modified accordingly. In several cases this might also be the case for c_{season} for a time period up to 3 months *[EN 1991-1-6, Table 3.1]*. In the same table the return periods for (up to) 3 months and (up to) 1 year are given, T = 5 and 10 years, respectively.

The corresponding probabilities for exceedence of the extreme event once, are p = 1/5 = 0,20 and 1/10 = 0,10, respectively

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

3. Numerical application (cont.)

Duration	Return periods (years)
≤ 3	2
≤ 3 months (but > 3 days)	5
≤ 1 year (but > 3 months)	10
> 1 year	50

Extracts from *[Table 3.1 of EN 1991-1-6]*

3. Numerical application (cont.)

In the specific case of this example one might reasonably assume 3 months for the duration of the construction, before casting the concrete slab, leading to c_{prob} = 0,85.
Nevertheless, a more conservative approach would be to assume virtual delays, thus leading to a value of c_{prob} = 0,9, as it may be seen below:

$$c_{\mathrm{prob}} = \left(\frac{1 - 0{,}2 \cdot \ln(-\ln(1 - 0{,}10))}{1 - 0{,}2 \cdot \ln(-\ln(0{,}98))} \right)^{0{,}5} =$$

$$= (1{,}45/1{,}78)^{0{,}5} = 0{,}8146^{\,0{,}5} = 0{,}902 \approx 0{,}9$$

It is to note however that the phase of launching has usually a duration that does not exceed 3 days

3. Numerical application (cont.)

The case considered is, when the steel structure pushed (without addition of a nose-girder) from one side (abutment A0) is about to reach as cantilever the pier P2. In that specific case :

L = 60,00 + 80,00 = 140,00 m and $d_{tot} = 2.\ d_{main\ beam}$ = 2 x 2,80 = 5,60 m

Hence:

b/d_{tot} = 12,00/5,60 = 2,14, A_{ref} = 5,60 x 140,00 = 784 m², $c_{fx} = c_{fx,0} \approx 1{,}9$

Consequently:

$v_m(10) = 1{,}27 \times 1{,}0 \times 14 = 17{,}78 \approx 18$ m/s

$$q_p(10) = [1 + 7x0{,}15)]x\frac{1}{2}x1{,}25x18^2 = 2{,}05x202{,}5 = 415{,}125 \approx 415 \quad \text{in N/m}^2$$

Finally: $F_w = 1{,}0x1{,}9x415x784{,}00 = 788{,}5x784{,}00 = 618184\text{N} \approx 618$ kN

Or "wind load" in the transverse (x-direction): $w \approx 4{,}4$ kN/m

3. Numerical application (cont.)

	Service life without traffic		Service life with traffic		Construction phase (steel alone – end of pushing)		Construction phase (steel alone - cantilever at P2)	
$z = z_e$ (m)	**10**	**40**	**10**	**40**	**10**	**40**	**10**	**40**
$v_{b,0}$ (m/s)	26	26	26	26	-	-	-	-
v_b (m/s)	26	26	26	26	14	14	14	14
v_m (m/s)	26	33	26	33	14	18	14	18
q_b (N/m^2)	422,5	422,5	422,5	422,5	122,5	122,5	122,5	122,5
q_m (N/m^2)	422,5	680,6	422,5	680,6	122,5	202,5	122,5	202,5
q_p (N/m^2)	980,2	1395,3	980,2	1395,3	284,2	415	284,2	415
c_e	2,32	3,30	2,32	3,30	2,32	3,30	2,32	3,30
d_{tot} (m)	4,00	4,00	5.34	5,34	5,60	5,60	5,60	5,60
L (m)	200	200	200	200	140	140	140	140
$A_{ref,x}$ (m^2)	800	800	1068	1068	1120	1120	784	784
b/d_{tot}	3,00	3,00	2,25	2,25	2,14	2,14	2,14	2,14
$c_{f,x}$	1,55	1,55	1,83	1,83	1,9	1,9	1,9	1,9
F_w (kN)	**1215**	**1730**	**1916**	**2727**	**605**	**883**	**423**	**618**
w (kN/m)	**6**	**8,65**	**9,6**	**13,64**	**3**	**4,4**	**3**	**4,4**

3. Numerical application (cont.)

Comparing the values of F_W (similarly, comparing the values of *w*) from the previous table for the bridge without and with traffic, and taking into account that **when the leading variable action on the bridge is traffic** loads, which means that the **wind is an accompanying action**, for which ψ_0 = 0,6 one gets :

Squat piers (*z*=10m) : 1215 > 0,6 x 1916 = 1149,6 (kN)

and

"High" piers (*z*=40m) : 1730 > 0,6 x 2727 = 1636 (kN)

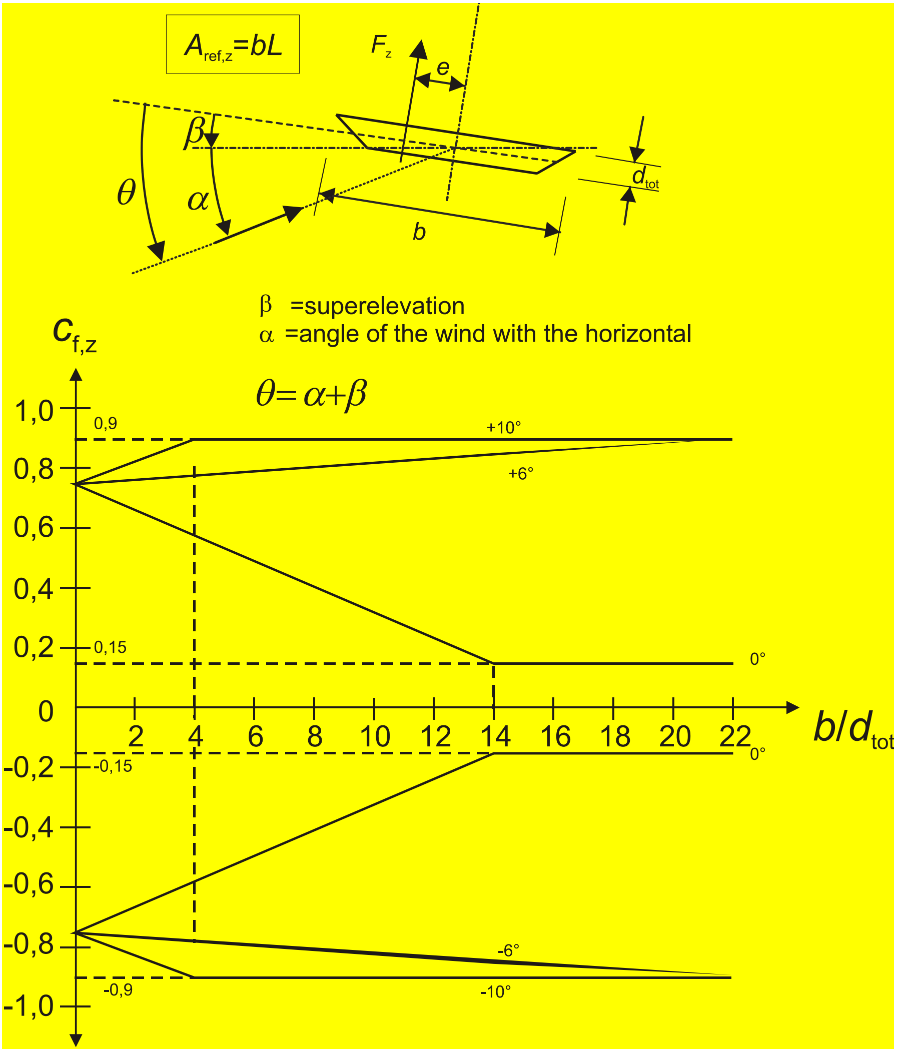
which means that, in this case, the design situation for wind without traffic is more severe than the one with traffic

3. Numerical application (cont.)

3.4 Vertical wind forces on bridge deck (z-direction)

- Use of *[8.3.3]* with recommended value for $c_{f,z} = \pm 0{,}9$, or
- Use the adjacent *[Fig. 8.6]*. The recommended value excentricity is $e = b/4$

- In the present example, both the wind angle α and the transverse slope of the bridge are taken = 0



3. Numerical application (cont.)

3.5 Wind forces along bridge deck (y-direction)

- *[8.3.4]* refers to the wind action on bridge decks in the longitudinal direction, to be taken into account, where relevant.
- The values are also left as NDPs, but it is recommended that a 25% percentage of the wind forces in x-direction is considered, in the case of plated bridges, and a 50% in the case of truss bridges.
- These two additional cases (wind action in y- and z-direction) are not treated in this example of application.

4. Wind actions on piers

"High" circular pier (4 m diameter, 40 m height)

According to *[8.4.2]* simplified rules for the evaluation of wind effects on piers may be given in the National Annexes. Otherwise the procedures described in *[7.6]*, *[7.8]* and *[7.9]*, should be applied, respectively for rectangular, regular polygonal and circular cross sections.

$$F_{\mathrm{w}} = c_{\mathrm{s}} c_{\mathrm{d}} \cdot c_{\mathrm{f}} \cdot q_{\mathrm{p}}(z_{\mathrm{e}}) \cdot A_{\mathrm{ref}}$$

The general formula *[5.3]* already used for the deck is also valid for structural elements like free standing piers. In this case $c_s\, c_d = 1{,}0$ and c_f are given by the following formula *[7.19]* of *[7.9.2]* : $c_f = c_{f,0}\, \psi_\lambda$

Where:

$c_{f,0}$ is the **force coefficient of circular sections** (finite cylinders) without free-end flow *[Fig. 7.28]*)

ψ_λ is the **end-effect factor** (for elements with free-end flow *[7.13]*)

4. Wind actions on piers (cont.)

For the use of *[Fig. 7.28]* the Reynolds number *[Eq. 7.15]* based on the peak wind velocity according to *[4.5, Eq. 4.8]* and the equivalent surface roughness *k [Tab. 7.13]* need first to be computed.

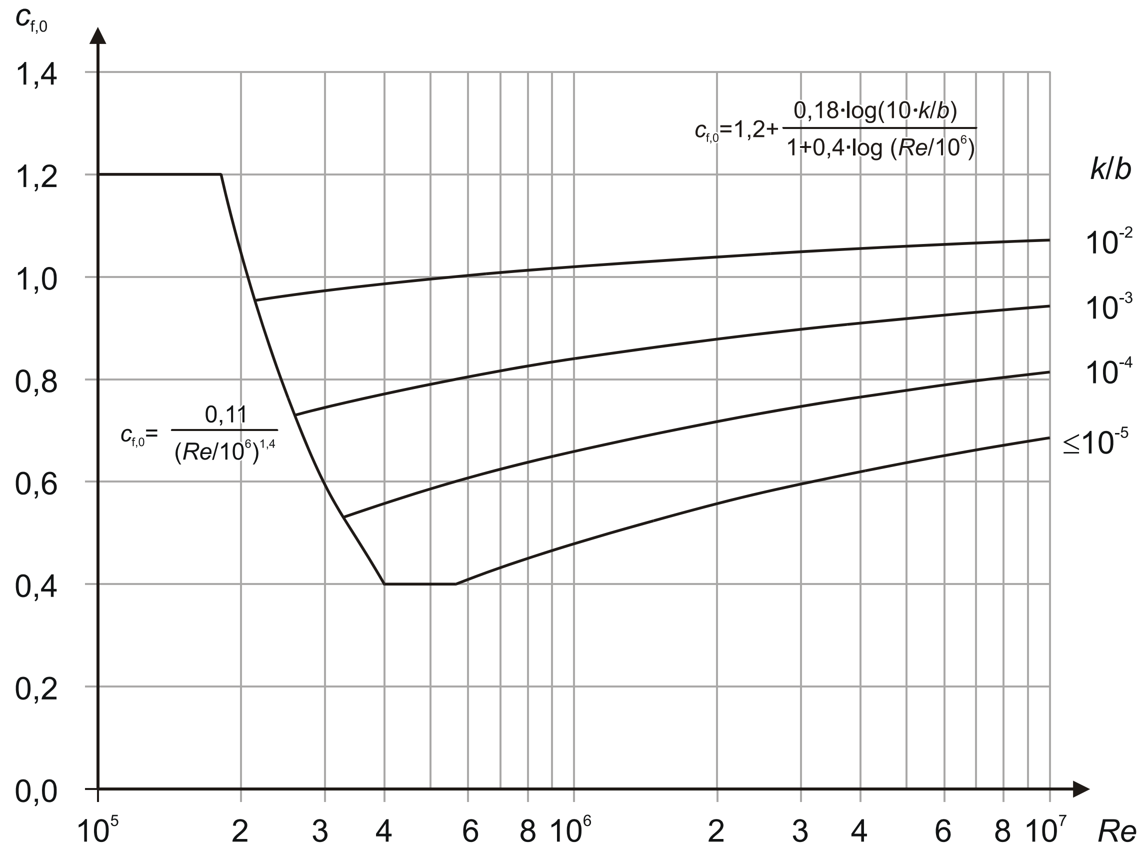
The combination of formulas [7.15] and [4.8] leads to the following expression: $v(z_e) = v_m(z_e) . \{1 + 7 . I_v(z_e)\}^{0,5}$

For z_e = 40 m one gets:
$v(40) = 33 \times \{1 + 7 \times 0{,}15\}^{0,5} = 33 \times. 2{,}05^{0,5} = 33 \times 1{,}432 = 47{,}25$ m/s
$Re = b.v(z_e)/\nu = 4{,}00 \times 47{,}25 / (15 \times 10^{-6}) = 12{,}6 \times 10^{6} = 1{,}26 \times 10^{7}$
This value is a bit further than the limiting value of *[Fig. 7.28].*

The equivalent roughness is 0,2 mm for smooth and 1,0 mm for rough concrete. Smooth concrete surface will be assumed. This leads to $k/b = 0{,}2/4000 = 5 \times 10^{-5}$. From Fig 7.28 a value greater than 0,7 is expected.

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

4. Wind actions on piers (cont.)



[Fig.7.28] Force coefficient *cf,0* for circulars cylinders without end-flow and for different equivelent roughness *k/b*

4. Wind actions on piers (cont.)

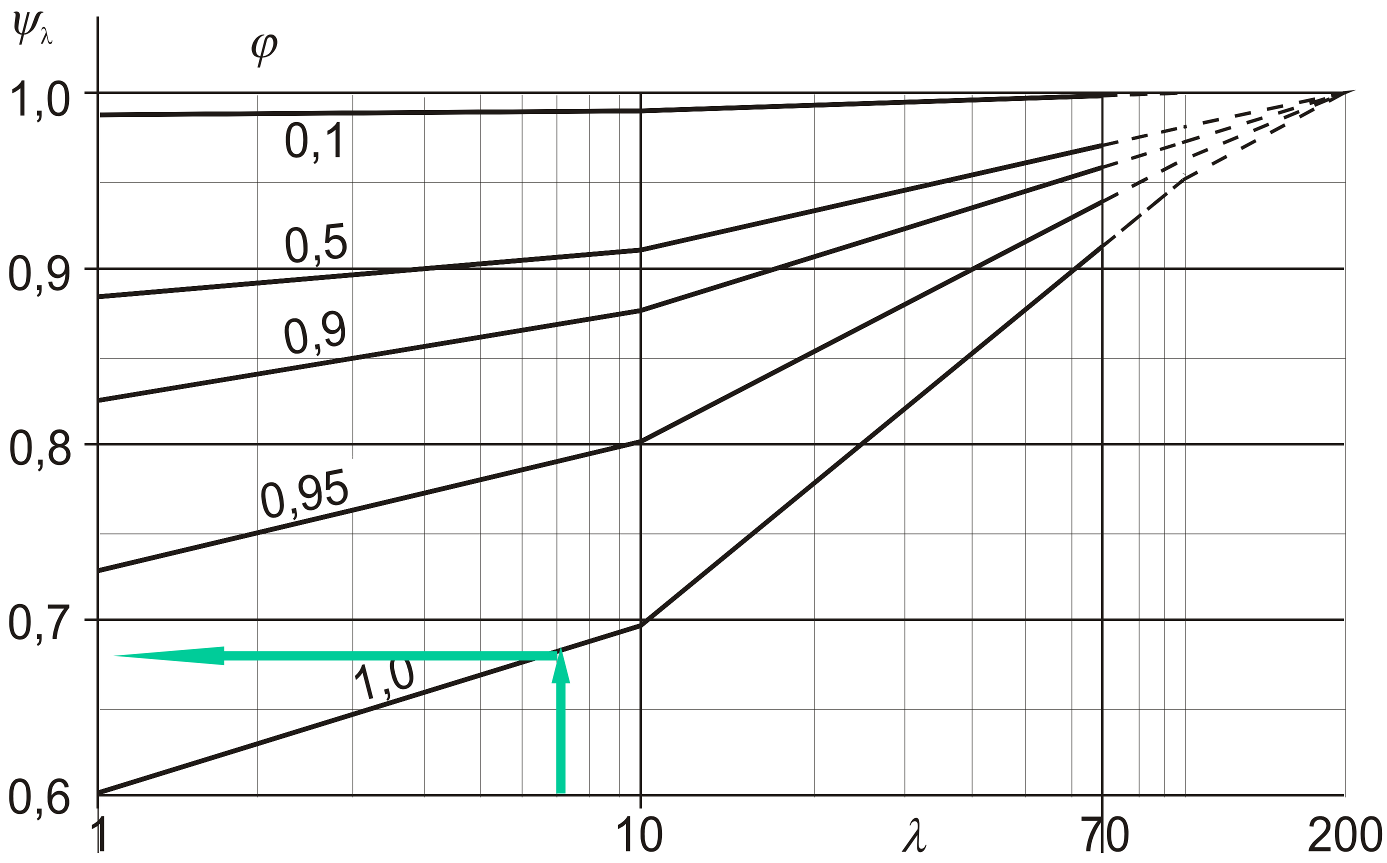
By using the relevant formula one gets:

$c_{f,0}$ = 1,2 + {0,18 . log(10 *k/b*)} / {1 + 0,4 . log (*Re*/106)} =
1,2 + {0,18 . log(10 x 5 x 10-5.)} / {1 + 0,4 . log (12,6 x 106/106)} =
1,2 - 0,594 / 1,44 = 1,2 – 0,413 = 0,787 ≈ 0,79

In the case of rough concrete one would get: $c_{f,0}$ = 0,875

Concerning the evaluation of ψ_λ one should use interpolation, while using *[Tab. 7.16]* and *[Fig. 7.36]* since 15 m < *l* = 40 m < 50 m.
For ***l* = 15 m** the effective slenderness *λ* is given as follows: ***λ*** = min { *l/b ;* 70} = min { 40,00/4,00 *;* 70} **= 10**
For ***l* = 50 m** the effective slenderness *λ* is given as follows: ***λ*** = min { 0,7 *l/b ;* 70} = min { 0,7 x 40,00/4,00 *;* 70} **= 7**
Interpolation gives *λ* = 0,786 *l* / *b* = 0,786 x 40,00 / 4,00 = 7,86

4. Wind actions on piers (cont.)



[Fig. 7.36] — Indicative values of the **end-effect factor ψ_λ** as a function of **solidity ratio φ** versus **slenderness λ**

EXAMPLE OF APPLICATION
WIND ACTIONS ON BRIDGE DECK AND PIERS

4. Wind actions on piers (cont.)

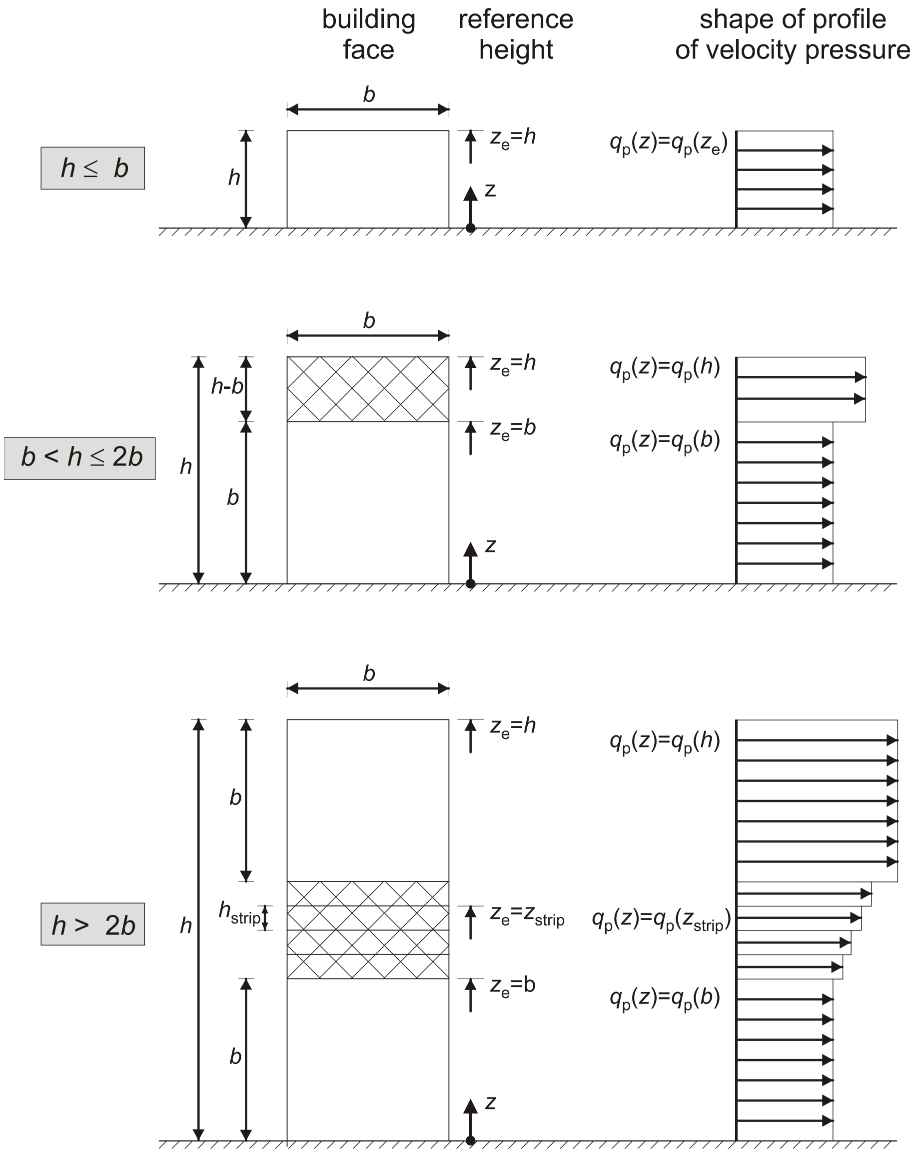
By using *[Fig. 7.36]* with φ = 1,0 one gets $\psi_\lambda \approx 0{,}685$
And: c_f = 0,79 x 1,0 x 0,685 ≈ 0,54
A_{ref} = *l. b* = 40,00 x 4,00 = 160,00 m^2
q_p (40) = 1395,3 N/m^2 (415 N/m^2 for the construction phase)

According to *[7.9.2(5)]* the reference height z_e is equal to the maximum height above the ground of the section being considered. As a conservative approach the value for z_e = 40 m may be consider, given that *[Fig. 7.4]* is not directly applicable. Nevertheless, a splitting of the pier in adjacent strips with various z_e and the associated values for *v*, q_p etc. might be considered, as a more realistic and less conservative approach

Finally: $F_w = 1{,}0 x 0{,}54 x 1295{,}3 x 160{,}00 = 753{,}46 x 160{,}00 = 120554$ N ≈ 120,5 kN

4. Wind actions on piers (cont.)

[Fig. 7.4] — Reference height, z_e, depending on h and b, and corresponding velocity pressure profile (for rectangular piers)

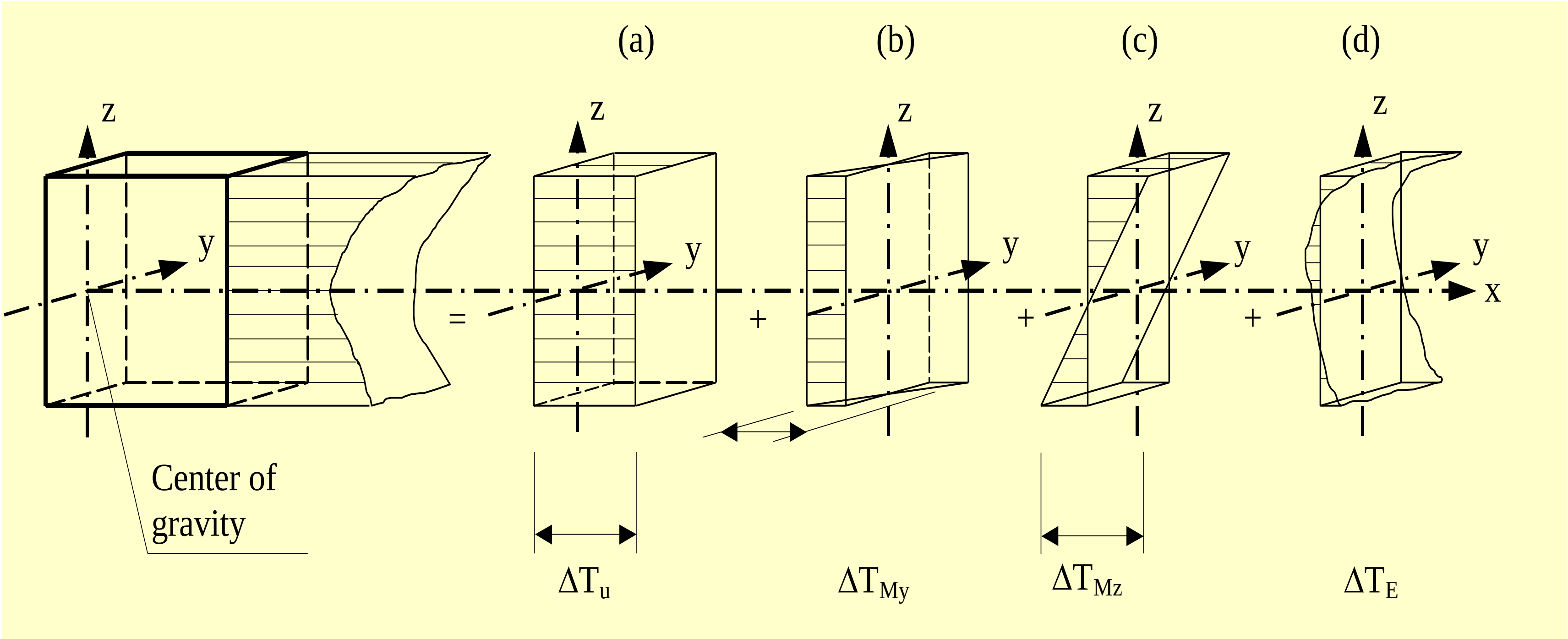


EN 1991-1-5: THERMAL ACTIONS

- **Forward**
- **Section 1 – General**
- **Section 2 – Classification of actions**
- **Section 3 – Design situations**
- **Section 4 – Representation of actions**
- **Section 5 – Temperature changes in buildings**
- **Section 6 – Temperature changes in bridges**
- **Section 7 – Temperature changes in industrial chimneys, pipelines, silos, tanks and cooling towers**
- **Annex A (normative) – Isotherms of national minimum and maximum shade air temperatures.**
- **Annex B (normative) – Temperature differences for various surfacing depths**
- **Annex C (informative) – Coefficients of linear expansion**
- **Annex D (informative) – Temperature profiles in buildings and other construction works**

ACTIONS : THERMAL ACTIONS

Diagrammatical representation of constituent components of a temperature profile *[EN 1991-1-5, Fig. 4.1]*



ACTIONS : THERMAL ACTIONS

Consideration of thermal actions on bridge decks *[EN 1991-1-5, 6.1.2]*:

- Representative values of thermal actions should be assessed by the **uniform temperature** component (ΔT_N) and the **temperature difference** components (ΔT_M).

- The **vertical temperature difference** component (ΔT_M) should generally include the non-linear component. Either Approach 1 (Vertical linear component) or Approach 2 (Vertical temperature components with non linear effects) may be used.

ACTIONS : THERMAL ACTIONS

Uniform temperature component:

This component induces a variation in length of the bridge (when the longitudinal displacements are free on supports) which is not studied for the design example.

The uniform temperature component (ΔT_N) depends on the minimum (T_{min}) and maximum (T_{max}) temperature which a bridge will achieve.

Minimum **shade air temperature** (T_{min}) and maximum shade air temperature (T_{max}) for the site are derived from isotherms.

The minimum and maximum **uniform bridge temperature** components $T_{e.min}$ and $T_{e.max}$ need to be determined.

ACTIONS : THERMAL ACTIONS- Bridge Types

Type 1 Steel deck

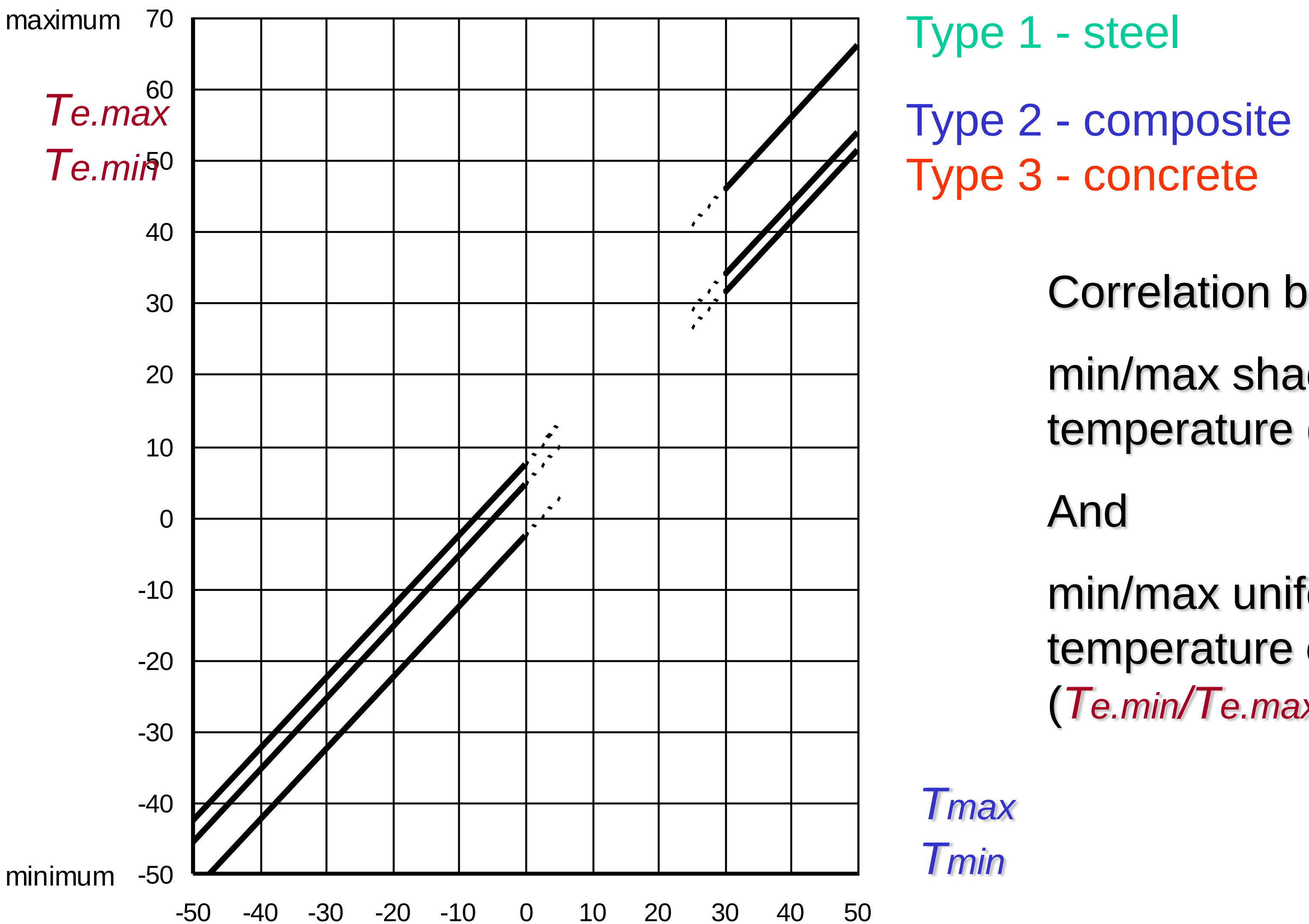
- steel box-girder
- steel truss or plate girder

Type 2 Composite deck

Type 3 Concrete deck

- concrete slab
- concrete beam
- concrete box-girder

Determination of thermal effects



Correlation between

min/max shade air temperature (*Tmin/Tmax*)

And

min/max uniform bridge temperature component (*Te.min/Te.max*)

ACTIONS : THERMAL ACTIONS
Uniform temperature component

T_0 is the initial bridge temperature at the time that the structure is restrained.

The characteristic value of the maximum contraction range of the uniform bridge temperature component, $\Delta T_{N,con}$ should be taken as : $\Delta T_{N,con} = T_0 - T_{e.min}$

The characteristic value of the maximum expansion range of the uniform bridge temperature component, $\Delta T_{N,exp}$ should be taken as : $\Delta T_{N,exp} = T_{e.max} - T_o$

The overall range of the uniform bridge temperature component is : $\Delta T_N = T_{e.max} - T_{e.min}$

ACTIONS : THERMAL ACTIONS

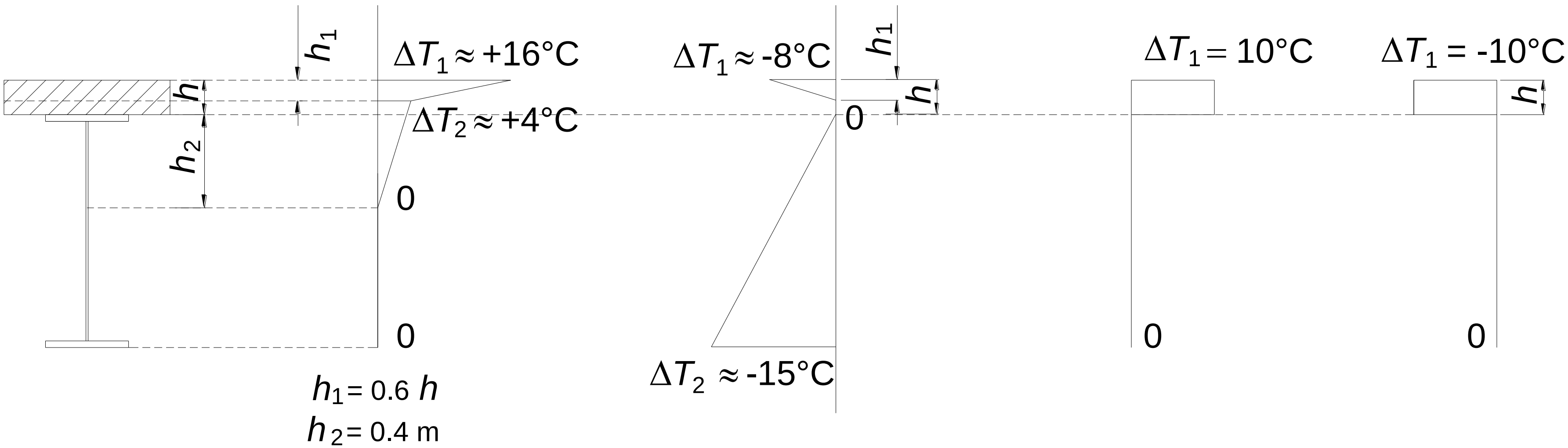
Vertical linear component (Approaches)

The National Annex of EN1991-1-5 should choose to one of the two following definitions for this thermal component in a bridge (see next figure):

- **a linear thermal gradient over the entire depth of the bridge deck *[6.1.4.1 of EN 1991-1-5]***
- **a non-linear thermal gradient which can be defined by two methods, continuous or discontinuous. The values ΔT_1 and ΔT_2 are defined according to the type of deck surfacing in Annex B to EN1991-1-5 *[6.1.4.2 and Annex B of EN 1991-1-5]***

The option adopted in this example is a variation of the second approach (simplified prcedure), i.e. the non-linear discontinuous thermal gradient with a temperature difference of +/- 10°C between the slab concrete and the structural steel. The linear temperature difference components are noted $\Delta T_{M,heat}$ (heating) and $\Delta T_{M,cool}$ (cooling).

ACTIONS : THERMAL ACTIONS
Vertical linear component (various approaches)



Approach 2 **Approach 2***

This thermal gradient is classified as a variable action (like traffic load) and is applied to composite cross-sections which are described with the short-term modular ratio.

ACTIONS : THERMAL ACTIONS
Vertical linear component (Approach 1)

Over a prescribed time period heating and cooling of a bridge deck's upper surface will result in a maximum heating (top surface warmer) and a maximum cooling (bottom surface warmer) temperature variation.

The vertical temperature difference may produce, for example, effects within a structure due to:

- **Restraint of free curvature due to the form of the structure (e.g. portal frame, continuous beams etc.);**
- **Friction at rotational bearings;**
- **The effect of vertical temperature differences should be considered by using an equivalent linear temperature difference component with $\Delta T_{M,heat}$ and $\Delta T_{M,cool}$. These values are applied between the top and the bottom of the bridge deck.**

Table 6.1: Recommended values of linear temperature difference component for different types of bridge decks for road, foot and railway bridges

Type of Deck	Top warmer than bottom	Bottom warmer than top
	$\Delta T_{M,heat}$ (°C)	$\Delta T_{M,cool}$ (°C)
Type 1: Steel deck	18	13
Type 2: Composite deck	15	18
Type 3: Concrete deck: - concrete box girder - concrete beam - concrete slab	 10 15 15	 5 8 8

NOTE 1: The values given in the table represent upper bound values of the linearly varying temperature difference component for representative sample of bridge geometries.

NOTE 2: The values given in the table are based on a depth of surfacing of 50 mm for road and railway bridges. For other depths of surfacing these values should be multiplied by the factor k_{sur}. Recommended values for the factor k_{sur} is given in Table 6.2.

Table 6.2: Recommended values of k_{sur} to account for different surfacing thickness

Road, foot and railway bridges						
Surface Thickness	Type 1		Type 2		Type 3	
	Top warmer than bottom	Bottom warmer than top	Top warmer than bottom	Bottom warmer than top	Top warmer than bottom	Bottom warmer than top
[mm]	k_{sur}	k_{sur}	k_{sur}	k_{sur}	k_{sur}	k_{sur}
unsurfaced	0,7	0,9	0,9	1,0	0,8	1,1
water-proofed [1)]	1,6	0,6	1,1	0,9	1,5	1,0
50	1,0	1,0	1,0	1,0	1,0	1,0
100	0,7	1,2	1,0	1,0	0,7	1,0
150	0,7	1,2	1,0	1,0	0,5	1,0
ballast (750 mm)	0,6	1,4	0,8	1,2	0,6	1,0

[1)] These values represent upper bound values for dark colour

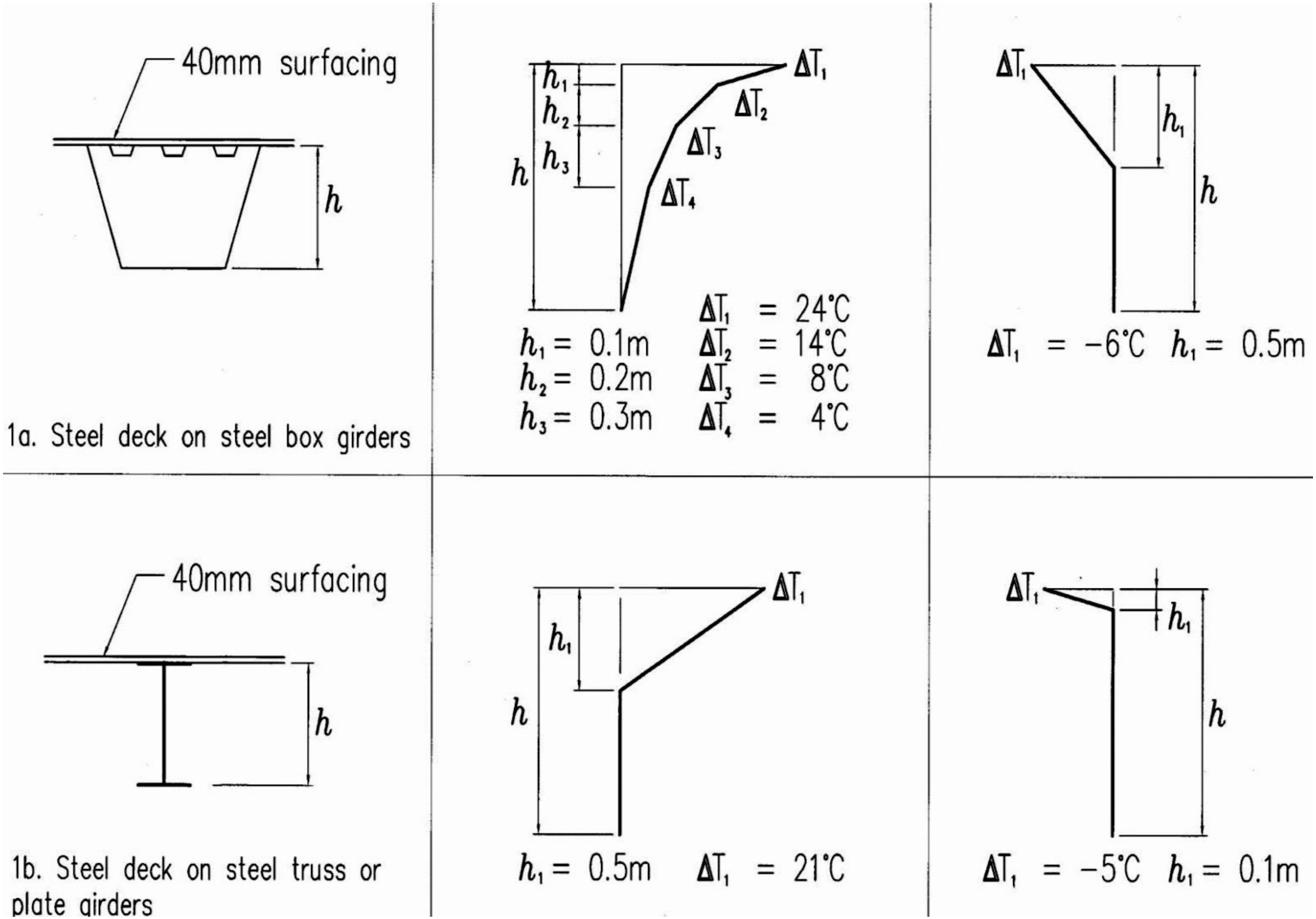
Vertical temperature components with non-linear effects (Approach 2)

The effect of the vertical temperature differences should be considered by including a non-linear temperature difference component.

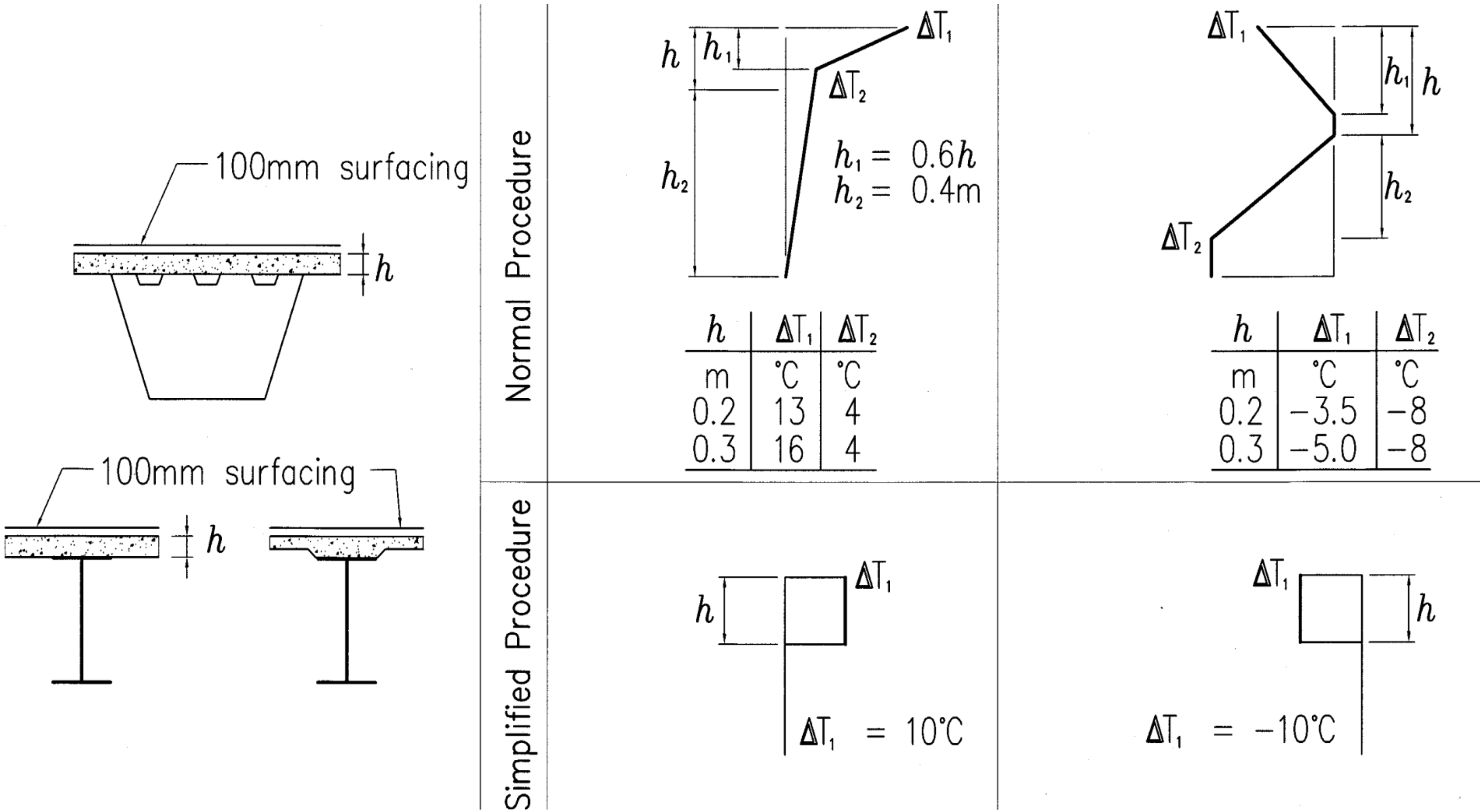
Recommended values of vertical temperature differences for bridge decks are given in next 3 Figures. In these figures *"heating"* refers to conditions such that solar radiation and other effects cause a gain in heat through the top surface of the bridge deck. Conversely, *"cooling"* refers to conditions such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

The temperature difference ΔT incorporates ΔT_M and ΔT_E together with a small part of component ΔT_N; this latter part is included in the uniform bridge temperature component.

STEEL BRIDGES



STEEL-CONCRETE COMPOSITE BRIDGES



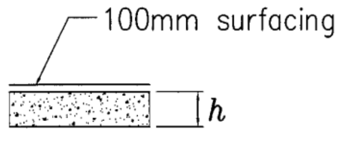
h	ΔT_1	ΔT_2
m	°C	°C
0.2	13	4
0.3	16	4

h	ΔT_1	ΔT_2
m	°C	°C
0.2	−3.5	−8
0.3	−5.0	−8

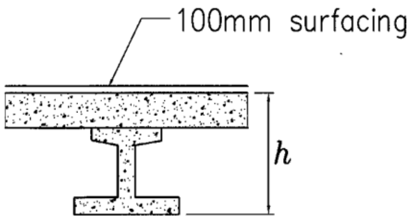
CONCRETE BRIDGES

Type of Construction	Temperature Difference (ΔT) (a) Heating	Temperature Difference (ΔT) (b) Cooling

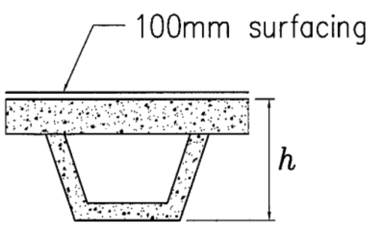
Type of Construction



3a. Concrete slab

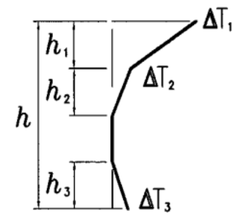


3b. Concrete beams



3c. Concrete box girder

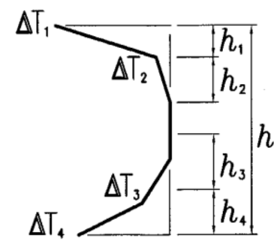
(a) Heating



$h_1 = 0.3h$ but ≤ 0.15m
$h_2 = 0.3h$ but ≥ 0.10m but ≤ 0.25m
$h_3 = 0.3h$ but $\leq (0.10$m + surfacing depth in metres)
(for thin slabs, h_3 is limited by $h - h_1 - h_2$)

h	ΔT_1	ΔT_2	ΔT_3
m		°C	
≤0.2	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
≥0.8	13.0	3.0	2.5

(b) Cooling



$h_1 = h_4 = 0.20h$ but ≤ 0.25m
$h_2 = h_3 = 0.25h$ but ≤ 0.20m

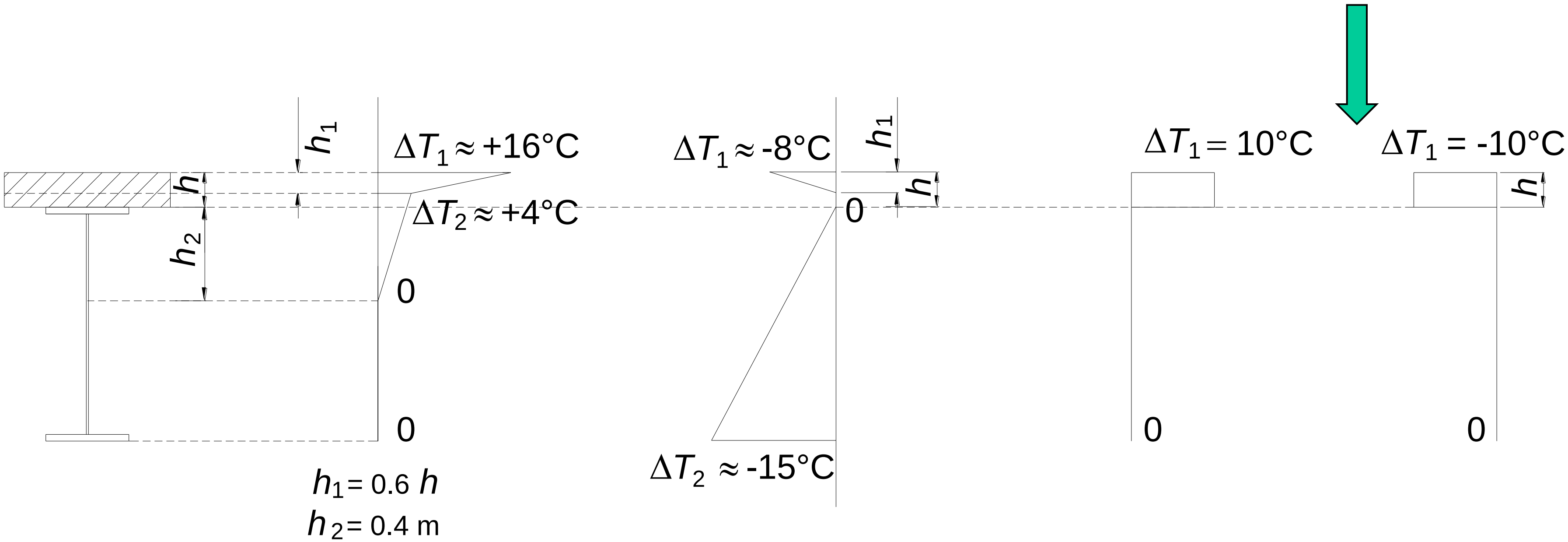
h	ΔT_1	ΔT_2	ΔT_3	ΔT_4
m		°C		
≤0.2	−2.0	−0.5	−0.5	−1.5
0.4	−4.5	−1.4	−1.0	−3.5
0.6	−6.5	−1.8	−1.5	−5.0
0.8	−7.6	−1.7	−1.5	−6.0
1.0	−8.0	−1.5	−1.5	−6.3
≥1.5	−8.4	−0.5	−1.0	−6.5

Figure 6.2c: Temperature differences for bridge decks – Type 3 : Concrete Decks

*Note: The temperature difference ΔT incorporates ΔT_M and ΔT_E (see 4.3) together with a small part of component ΔT_N ; this latter part has been included in the uniform bridge temperature component (see 6.1.3).

ACTIONS : THERMAL ACTIONS
Vertical linear component (various approaches)

Non-linear thermal gradient taken into account in the example considered



Approach 2 **Approach 2***

This thermal gradient is classified as a variable action (like traffic load) and is applied to composite cross-sections which are described with the short-term modular ratio.

ACTIONS : THERMAL ACTIONS
Additional rules

Simultaneity of uniform and temperature difference components (recommended values)

$$\Delta T_{M,heat}(or \Delta T_{M,cool}) + 0{,}35 \Delta T_{N,\exp}(or \Delta T_{N,con})$$

$$0{,}75 \Delta T_{M,heat}(or \Delta T_{M,cool}) + \Delta T_{N,\exp}(or \Delta T_{N,con})$$

Differences in the uniform temperature component between different structural elements :
- 15°C between main structural elements (e.g. tie and arch); and
- 10°C and 20°C for light and dark colour respectively between suspension/stay cables and deck (or tower).

Temperature differences between the inner and outer web walls of large concrete box girder bridges :
Recommended value 15°C

EN 1991-1-6: ACTIONS DURING EXECUTION

- **Forward**
- **Section 1 – General**
- **Section 2 – Classification of actions**
- **Section 3 – Design situations and limit states**
- **Section 4 – Representation of actions**
- **Annex A1 (normative) – Supplementary rules for buildings**
- **Annex A2 (normative) – Supplementary rules for bridges**
- **Annex B (informative) – Actions on structures during alteration, reconstruction or demolition**

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Actions during execution are classified in accordance with EN 1990, and may include

- **those actions that are not construction loads;**

  and

- **construction loads**

In the following only construction loads will be treated

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Construction Loads - Q_c

Six different sources

Q_{ca}	***Personnel and hand tools***
Q_{cb}	***Storage of movable items***
Q_{cc}	***Non-permanent equipment in position for use***
Q_{cd}	***Movable heavy machinery and equipment***
Q_{ce}	***Accumulation of waste materials***
Q_{cf}	***Loads from part of structure in a temporary state***

*Construction loads Q*c may be represented in the appropriate design situations (see EN 1990), either, as one single variable action, or where appropriate different types of construction loads may be grouped and applied as a single variable action. Single and/or a grouping of construction loads should be considered to act simultaneously with non construction loads as appropriate.

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Relate Clause In this standard	Action	Classification				Remarks	Source
		Variation in time	Classification / Origin	Spatial Variation	Nature (Static/ Dynamic)		
	Construction loads:						
4.11	Personnel and handtools	Variable	Direct	Free	Static		
4.11	Storage movable items	Variable	Direct	Free	Static / dynamic	Dynamic in case of dropped loads	EN 1991-1-1
4.11	Non permanent equipment	Variable	Direct	Fixed/ Free	Static / dynamic		EN 1991-3
4.11	Movable heavy machinery and equipment	Variable	Direct	Free	Static / dynamic		EN 1991-3, EN 1992-1
4.11	Accumulation of waste materials	Variable	Direct	Free	Static/dynamic	Can impose loads on e.g. vertical surfaces also	EN 1991-1-1
4.11	Loads from parts of structure in temporary states	Variable	Direct	Free	Static	Dynamic effects are excluded	EN 1991-1-1

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qca*

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	$\boldsymbol{Q_{ca}}$	**Working personnel, staff and visitors, possibly with hand tools or other small site equipment**
Storage of movable items	Q_{cb}	Storage of moveable items, e.g. - building and construction materials, precast elements, and - equipment
Non permanent equipment	Q_{cc}	Non permanent equipment in position for use during execution, either : - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or - during movement (e.g. travelling forms, launching girders and nose, counterweights)
Moveable heavy machinery and equipment	Q_{cd}	Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)
Loads from parts of a structure in temporary states	Q_{cf}	Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Working personnel, staff and visitors, possibly with hand tools or other site equipment

Modelled as a uniformly distributed load q_{ca} and applied as to obtain the most unfavourable effects

The recommended value is : $q_{ca,k}$ = 1,0 kN/m^2

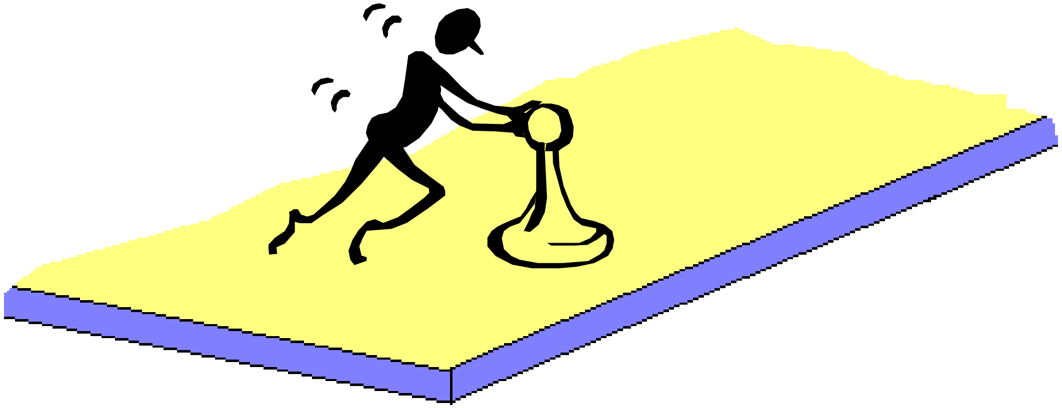
ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcb*

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	Q_{ca}	Working personnel, staff and visitors, possibly with hand tools or other small site equipment
Storage of movable items	$\mathbf{Q_{cb}}$	**Storage of moveable items, e.g.** **- building and construction materials, precast elements, and** **- equipment**
Non permanent equipment	Q_{cc}	Non permanent equipment in position for use during execution, either : - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or - during movement (e.g. travelling forms, launching girders and nose, counterweights)
Moveable heavy machinery and equipment	Q_{cd}	Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)
Loads from parts of a structure in temporary states	Q_{cf}	Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcb*

Storage of moveable items, eg. Building and construction materials, precast elements, and equipment



Modelled as a free action and represented by a uniform dead load Q_{cb} and a concentrated load F_{cb}

For bridges, the following values are recommended minimum values:

$q_{cb,k} = 0{,}2\ \text{kN/m}^2$

$F_{cb,k} = 100\ \text{kN}$

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	Q_{ca}	**Working personnel, staff and visitors, possibly with hand tools or other small site equipment**
Storage of movable items	Q_{cb}	Storage of moveable items, e.g. - building and construction materials, precast elements, and - equipment
Non permanent equipment	Q_{cc}	**Non permanent equipment in position for use during execution, either : - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or - during movement (e.g. travelling forms, launching girders and nose, counterweights)**
Moveable heavy machinery and equipment	Q_{cd}	Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)
Loads from parts of a structure in temporary states	Q_{cf}	**Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations**

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS

Representation of construction loads
Construction Loads during the casting of concrete

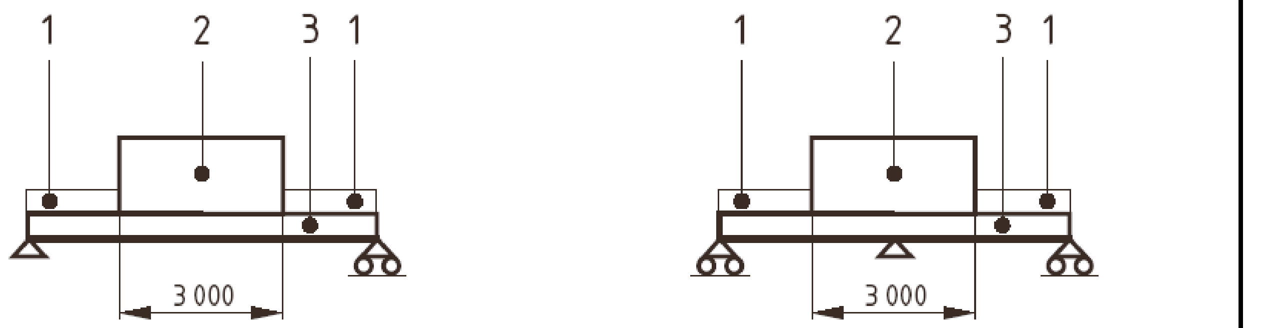
- Actions to be taken into account simultaneously during the casting of concrete may include:
- working personnel with small site equipment (Q_{ca});
- formwork and load-bearing members (Q_{cc});
- the weight of fresh concrete (which is one example of Q_{cf}), as appropriate.

ACTIONS DURING EXECUTION : casting of concrete

$\boldsymbol{Q_{ca}}$, $\boldsymbol{Q_{cc}}$ and $\boldsymbol{Q_{cf}}$ may be given in the National Annex.
Recommended values for fresh concrete ($\boldsymbol{Q_{cf}}$) may be taken from Table 4.2 and EN 1991-1-1, Table A.1. Other values may have to be defined, for example, when using self-levelling concrete or pre-cast products.

Table 4.2 : Recommended characteristic values of actions due to construction loads during casting of concrete

Action	Loaded area	Load in kN/m^2
(1)	Outside the working area	0,75 covering Q_{ca}
(2)	Inside the working area 3 m x 3 m (or the span length if less)	10 % of the self-weight of the concrete but not less than 0,75 and not more than 1,5. Includes Q_{ca} and Q_{cf}
(3)	Actual area	Self-weight of the formwork, load-bearing element (Q_{cc}) and the weight of the fresh concrete for the design thickness (Q_{cf})



ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcc*

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	Q_{ca}	Working personnel, staff and visitors, possibly with hand tools or other small site equipment
Storage of movable items	Q_{cb}	Storage of moveable items, e.g. - building and construction materials, precast elements, and - equipment
Non permanent equipment	Q_{cc}	**Non permanent equipment in position for use during execution, either :** **- static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or** **- during movement (e.g. travelling forms, launching girders and nose, counterweights)**
Moveable heavy machinery and equipment	Q_{cd}	Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)
Loads from parts of a structure in temporary states	Q_{cf}	Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcc*

Non permanent in position for use during exectuion, either: - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or – during movement (e.g. travelling forms, launching girders and nose, counterweights

Unless more accurate information is available, they may be modelled by a uniformly distributed load with a recommended minimum characteristic value of $q_{cc,k}$ = 0,5 kN/m^2

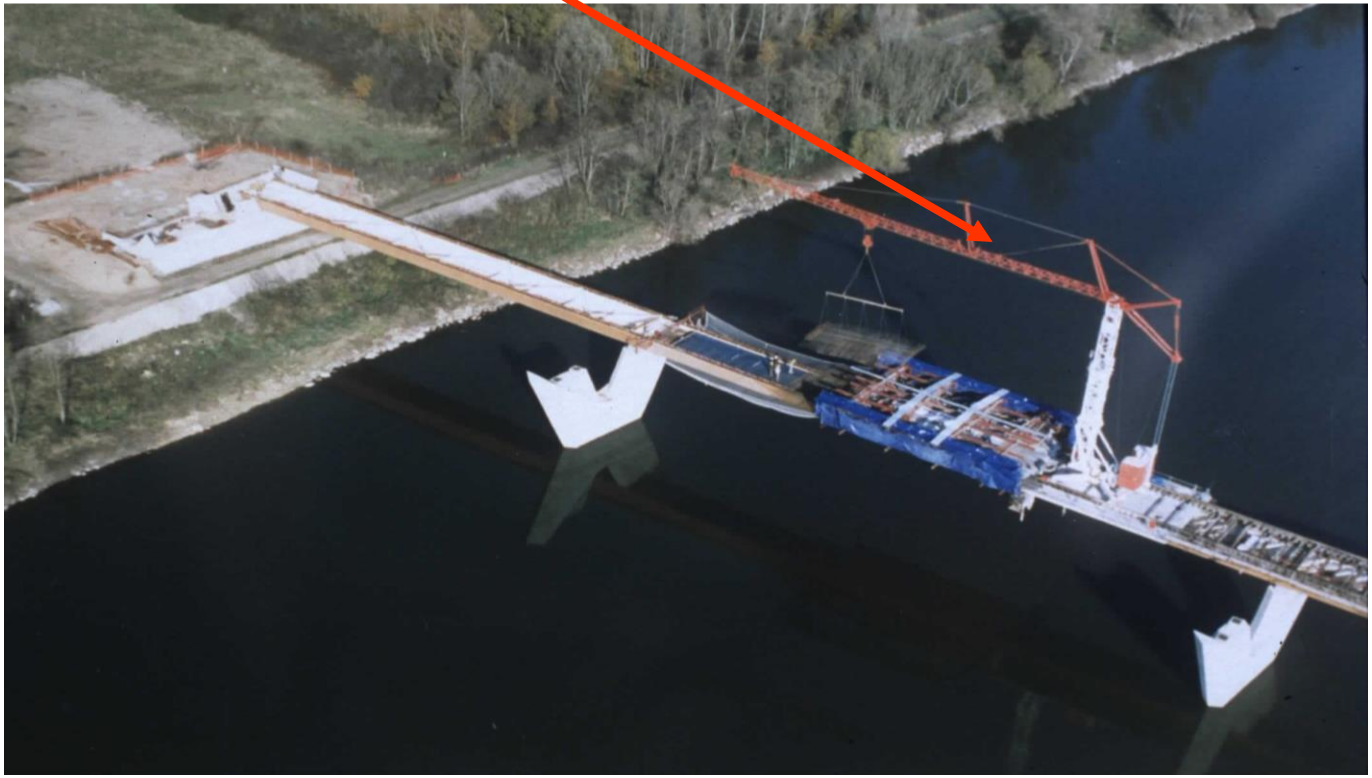
ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcd*

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	Q_{ca}	Working personnel, staff and visitors, possibly with hand tools or other small site equipment
Storage of movable items	Q_{cb}	Storage of moveable items, e.g. - building and construction materials, precast elements, and - equipment
Non permanent equipment	Q_{cc}	Non permanent equipment in position for use during execution, either : - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or - during movement (e.g. travelling forms, launching girders and nose, counterweights)
Moveable heavy machinery and equipment	$\mathbf{Q_{cd}}$	**Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)**
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)
Loads from parts of a structure in temporary states	Q_{cf}	Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qcd*

Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)



Information for the determination of actions due to vehicles when not defined in the project specification, may be found in EN 1991-2, for example

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Q_{ce} & Q_{cf}*

Representation of construction loads

Type	Symbol	Description
Personnel and handtools	Q_{ca}	Working personnel, staff and visitors, possibly with hand tools or other small site equipment
Storage of movable items	Q_{cb}	Storage of moveable items, e.g. - building and construction materials, precast elements, and - equipment
Non permanent equipment	Q_{cc}	Non permanent equipment in position for use during execution, either : - static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or - during movement (e.g. travelling forms, launching girders and nose, counterweights)
Moveable heavy machinery and equipment	Q_{cd}	Moveable heavy machinery and equipment, usually wheeled or tracked, (e.g cranes, lifts, vehicles, lifttrucks, power installations, jacks, heavy lifting devices)
Accumulation of waste materials	$\boldsymbol{Q_{ce}}$	**Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)**
Loads from parts of a structure in temporary states	$\boldsymbol{Q_{cf}}$	**Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations**

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS Q_{ce} & Q_{cf}

Accumulation of waste materials (e.g. surplus construction materials excavated soil, or demolition materials : Q_{ce}

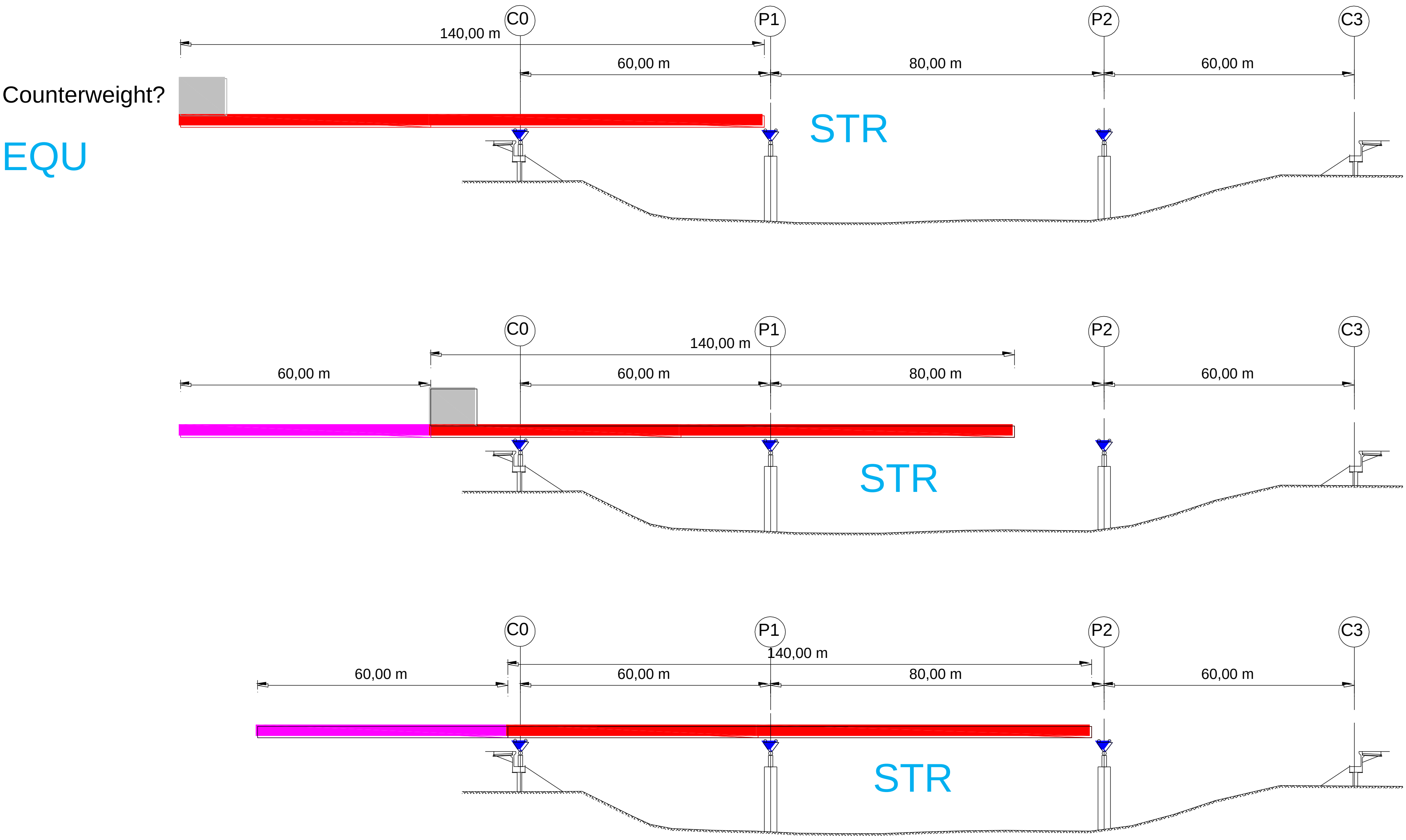
These loads are taken into account by considering possible mass effects on horizontal, inclined and vertical elements (such as walls). These loads may vary significantly, and over short time periods, depending on types of materials, climatic conditions, build-up and clearance rates.

ACTIONS DURING EXECUTION : CONSTRUCTION LOADS *Qce & Qcf*

Q_{cf} : Loads from parts of a structure in temporary states (under execution) before the final design actions take effect, such as loads from lifting operations.

Taken into account and modelled according to the planned execution sequences, including the consequences of those sequences (e.g. loads and reverse load effects due to particular processes of construction, such as assemblage).

LAUNCHING


C0
P1
P2
C3
140,00 m
60,00 m
80,00 m
60,00 m
Counterweight?
EQU
STR
C0
P1
P2
C3
140,00 m
60,00 m
60,00 m
80,00 m
60,00 m
STR
C0
P1
P2
C3
140,00 m
60,00 m
60,00 m
80,00 m
60,00 m
STR

Actions to be considered during launching

Permanent loads

Wind

Vertical temperature difference between bottom and upper part of the beam

Horizontal temperature difference

Differential deflection between the support in longitudinal direction (±10 mm)

Differential deflection between the support in longitudinal direction (±2.5 mm)

Friction forces:

-total longitudinal friction forces=10% of the vertical loads

- at every pier: the most unfavourable considering value of friction coefficient μ, considering : $\mu_{min}=0$ - $\mu_{max}=0.04$

Counterweight

If a counterweight is necessary, the variability of its characteristics should be taken into account.

For instance considering :

- $\gamma_{G,inf}$=0.8 when the weight is not well defined
- variation of its design position (for steel bridges usually ±1 m)

Design values of actions (EQU), Set A

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action	Accompanying variable actions	
	Unfavou-rable	Favoura-ble			Main	Others
Eq (6.10)	$\gamma_{Gj,\text{sup}}\ G_{kj,\text{sup}}$	$\gamma_{Gj,\text{inf}}\ G_{kj,\text{ing}}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,\ i}\ \psi_{0,i}\ Q_{k,i}$

Note 1: Recommended values of partial factors:

$\gamma_{Gj,\text{sup}}$ = 1,05 for unfavourable effects of permanent actions
$\gamma_{Gj,\text{inf}}$ = 0,95 for favourable effects of permanent actions
$\gamma_{Q,\ i}$ = 1,50 for all other variable actions in persistent design situations
$\gamma_{Q,\ i}$ = 1,35 for construction loads during execution

For favourable variable actions, $\gamma_Q = 0$.

Combined approach - EQU and STR

Note 2:

Alternative approach may be used (verification of bearing uplift of continuous bridges, and where verification of static equilibrium involves the resistance of structural members).

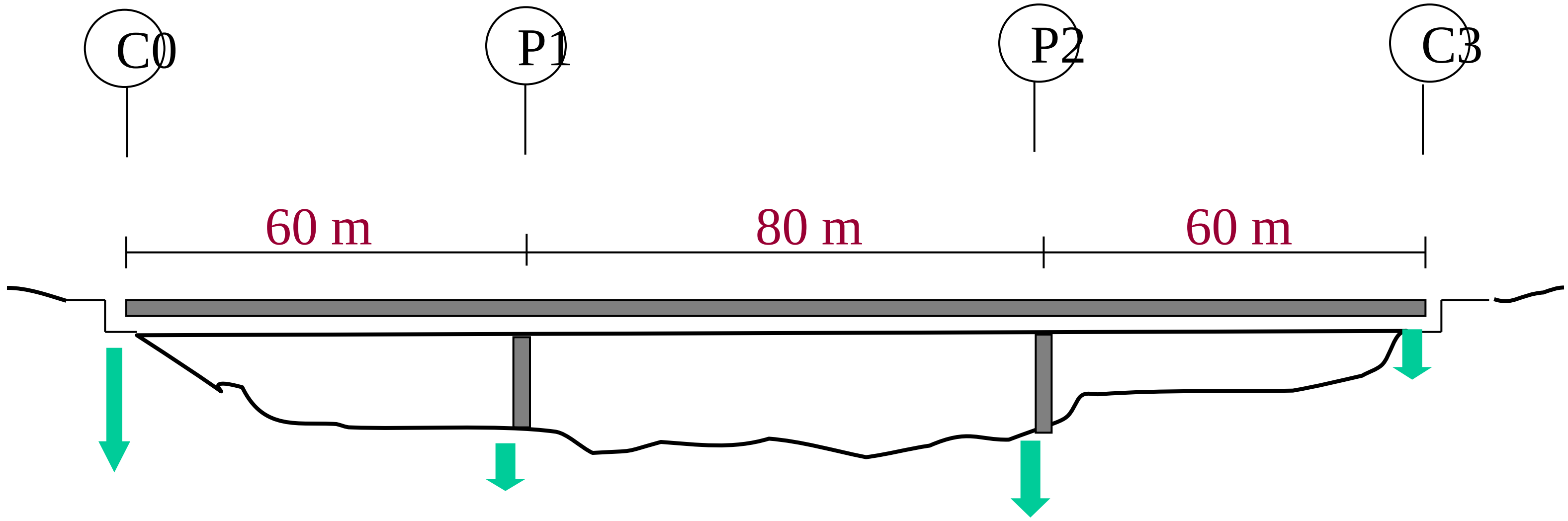
Recommended values of γ :

$\gamma_{Gj,sup}$ = 1,35, $\gamma_{Gj,inf}$ = 1,25

γ_{Q} = 1,50 for all other variable actions in persistent design situation

provided that applying $\gamma_{Gj,inf}$ = 1,00 both to the favourable and unfavourable part of permanent actions does not give a more unfavourable effect.

ACTIONS : SETTLEMENTS



dset,0 *dset,1* *dset,2* *dset,3*

Theoritically, all possible combinations should be considered, but in most cases their effects are not critical for a bridge of that type.

For the example presented the value of *dset,1* = 30 mm has been considered in P1

EN 1991-1-7: ACCIDENTAL ACTIONS

- Forward
- Section 1 – General
- Section 2 – Classification of actions
- Section 3 – Design situations
- Section 4 – Impact
- Section 5 – Internal explosions
- Annex A (informative) – Design for consequences of localised failure in buildings from an unspecified cause
- Annex B (informative) – Information on risk assessment
- Annex C (informative) – Dynamic design for impact
- Annex D (informative) – Internal explosions

ACCIDENTAL LOADS: Impact

Collisions on the bridge:

-lorries outside the regular position (footpath)

-hitting structural elements (kerbs, barriers, cables, columns, pylons)

Collisions under the bridge (EN 119-1-7):

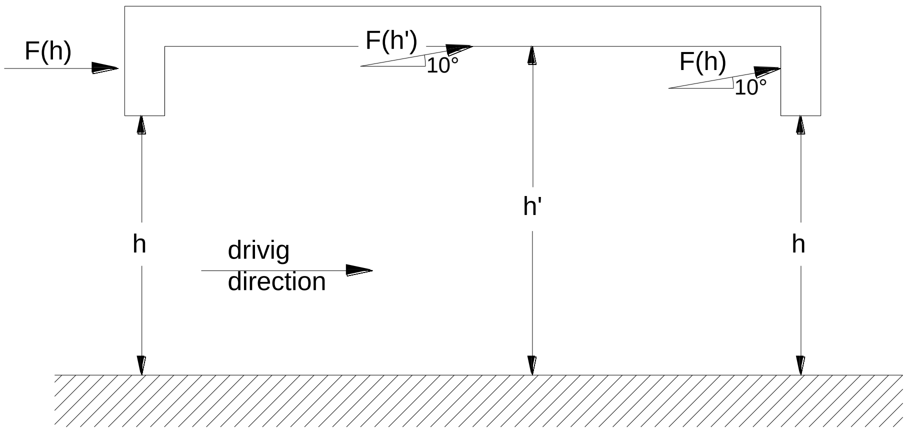
- on piers
- to the deck

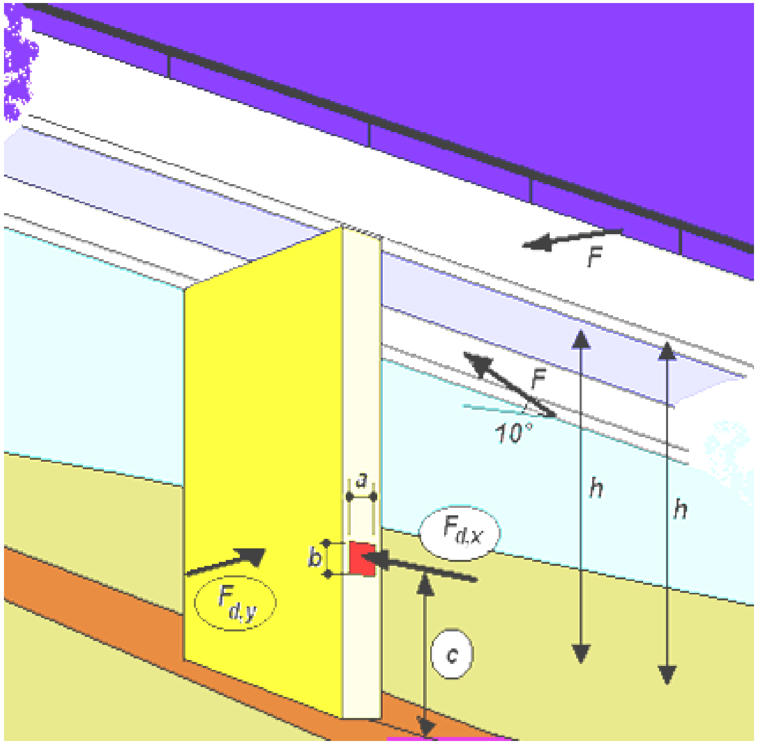
ACCIDENTAL LOADS: Impact on substructure

- Impact from road traffic
  - Type of road and vehicule
  - Distance to the road and clearance
  - Type of structures
    - Soft impact
    - Hard impact

- Impact from train traffic
  - Use of the structure
    - Class A
    - Class B
  - Line maximum speed

ACCIDENTAL LOADS: Impact on substructure



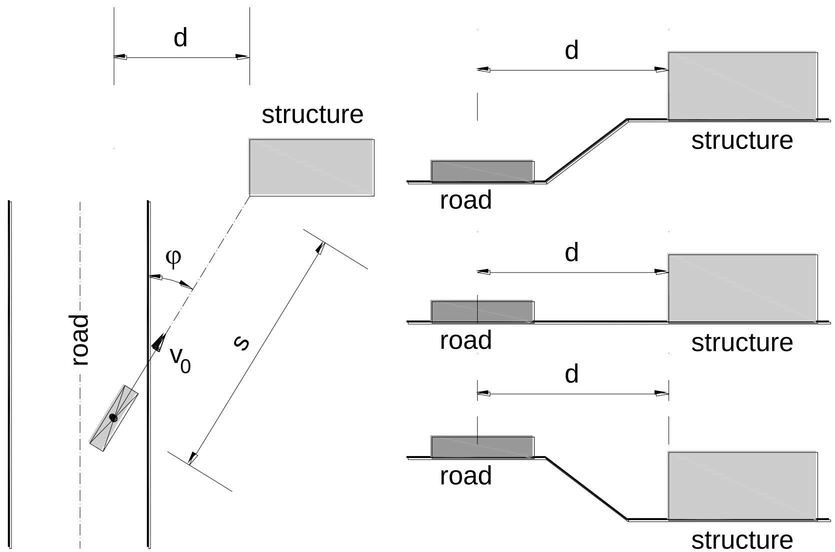


c=1.25 m for lorries

c=0.5 m for cars

Type of road	*Type of vehicle*	*Force $F_{d,x}$ [kN]*	*Force $F_{d,y}$ [kN]*
Motorway	Truck	1000	500
Country road	Truck	750	375
Urban area	Truck	500	250
Courtyards/garages	Passengers cars only	50	25
Courtyards/garages	Trucks	150	75

ACCIDENTAL LOADS: Impact on substructure



		mean value	*standard deviation*
m	mass	20 ton	12 ton
v	velocity	80 km/hr	10 km/hr
k	equivalent stiffness	300 kN/m	

Statistical parameters for input values

$$F = v_r \sqrt{km}$$

m=32 ton, *v*= 90 km/hr=25 m/s
$F = 25\ (300 \times 32)^{0.5} = 2400$ kN

$vr = (v0^2 - 2\,a\,s\,)^{0.5}$ if $a=4\ m/s^2$ $s=8$

$\varphi=15°$ $d=20\ m$

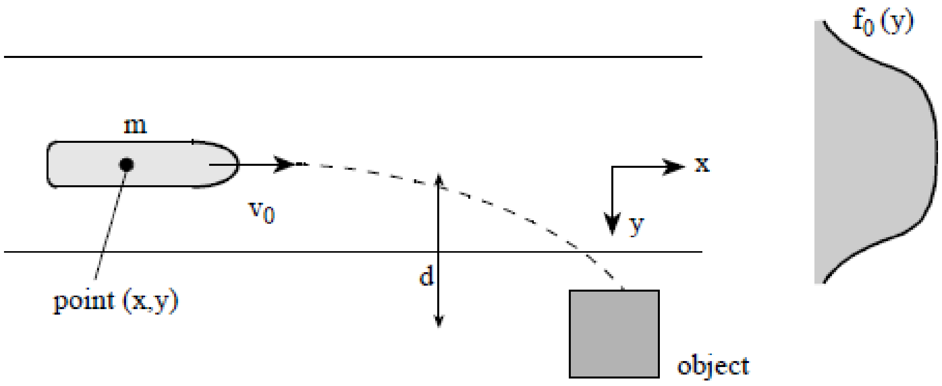
$$F = F_o \sqrt{1 - d / d_b} \quad \text{(for } d < d_b\text{).}$$

Situation sketch for impact by vehicles (top view and cross sections for upward slope, flat terrain and downward slope)

Type of road	*Type of vehicle*	*Force $F_{d,x}$ [kN]*	*Force $F_{d,y}$ [kN]*
Motorway	Truck	1000	500
Country road	Truck	750	375
Urban area	Truck	500	250
Courtyards/garages	Passengers cars only	50	25
Courtyards/garages	Trucks	150	75

ACCIDENTAL LOADS: Impact on substructure

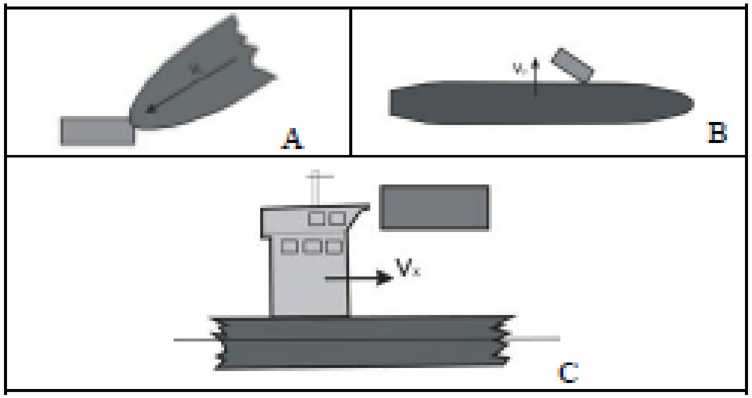
- Impact from ships
  - The type of waterway,
  - The flood conditions,
  - The type and draught of vessels
  - The type of the structures



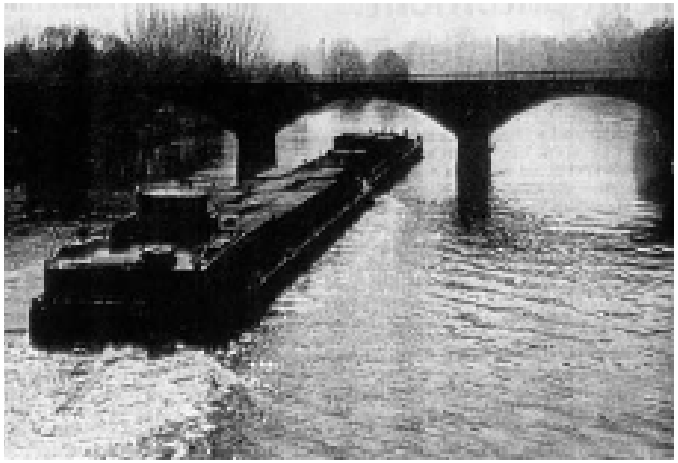
Parameters governing a ship collision model

Impact cases:

A. **bow** collision with bridge pillar,
B. **side** collision with bridge pillar,
C.deckhouse (superstructure) collision with bridge span.



ACCIDENTAL LOADS: Impact on substructure



m [ton]	*v* [m/s]	*k* [MN/m]	F_d [MN]	F_d [MN]	F_d [MN]
			Table 4.5 of EN 1991-1-7	eq (C.1) of EN 1991-1-7	eq (C.9) of EN 1991-1-7
300	3	5	2	4	5
1250	3	5	5	8	7
4500	3	5	10	14	9
20000	3	5	20	30	18

Design forces F_d for inland ships

m [ton]	*v* [m/s]	*k* [MN/m]	F_d [MN]	F_d [MN]	F_d [MN]
			Table 4.6 of EN 1991-1-7	eq(C.1) of EN 1991-1-7	eq (C.11) of EN 1991-1-7
3000	5	15	50	34	33
10000	5	30	80	87	84
40000	5	45	240	212	238
100000	5	60	460	387	460

Design forces F_d for seagoing vessels

ACCIDENTAL LOADS: Impact on superstructure

Vehicle impact on restraint system

Indicative equivalent static design forces due to impact on superstructures.

Category of traffic	*Equivalent static design force* F_{dx} [a] *[kN]*
Motorways and country national and main roads	500
Country roads in rural area	375
Roads in urban area	250
Courtyards and parking garages	75

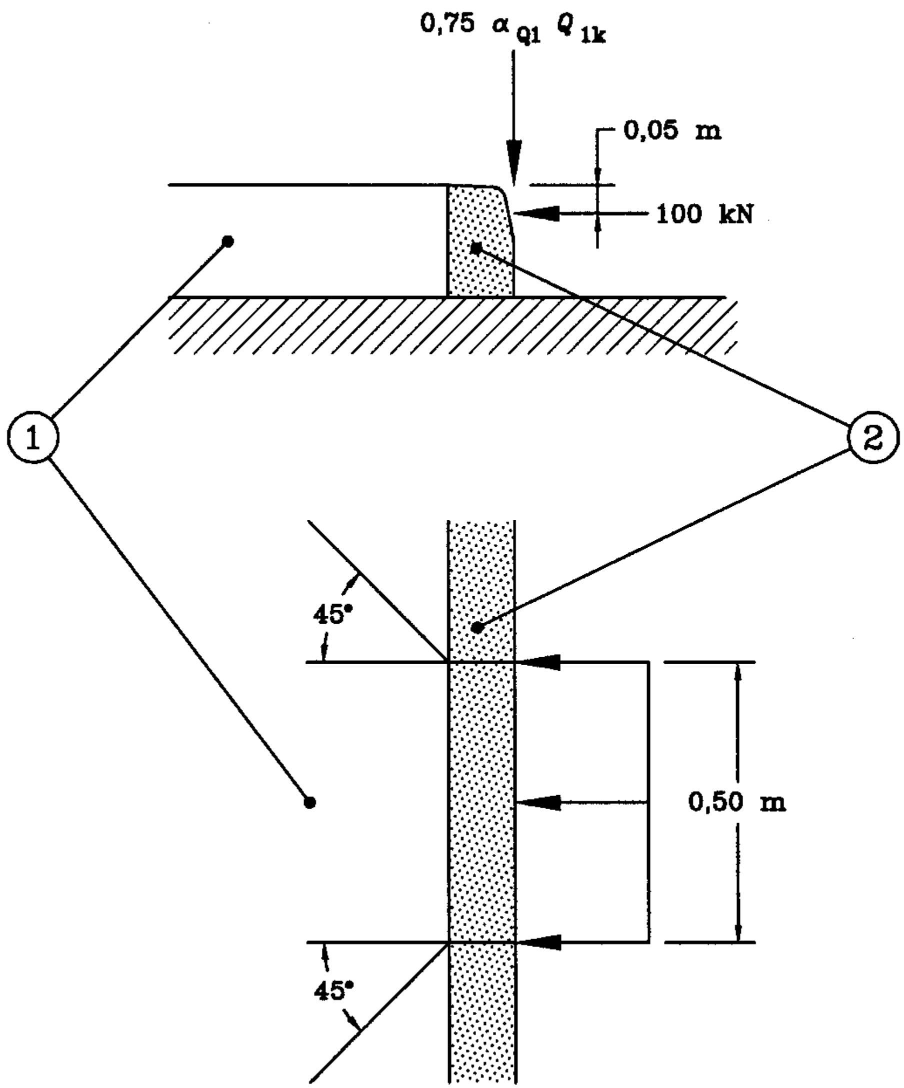
[a] x = direction of normal travel.

ACCIDENTAL LOADS: Impact on superstructure

Table 4.9 (n) – Recommended classes for the horizontal force transferred by vehicle restraint systems *(see EN 1317)*

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

ACCIDENTAL LOADS: Impact on superstructure



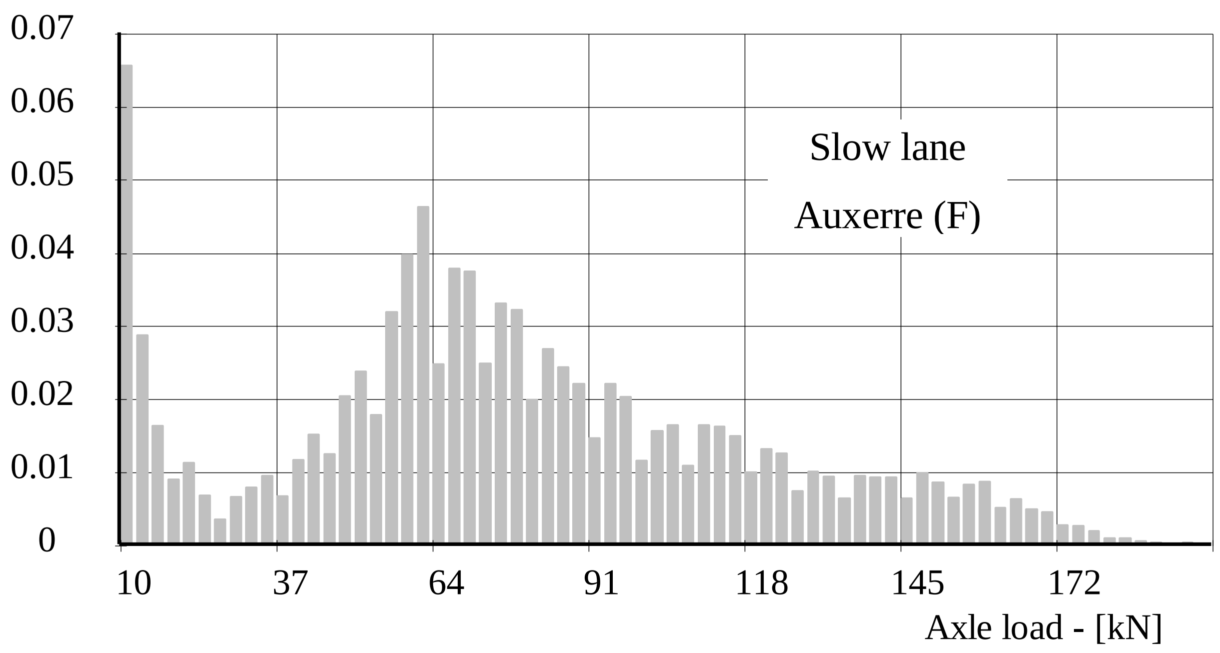
EN 1991-2: TRAFFIC LOADS ON BRIDGES

- **Forward**
- **Section 1 – General**
- **Section 2 – Classification of actions**
- **Section 3 – Design situations**
- **Section 4 – Road traffic actions and other actions specifically for road bridges**
- **Section 5 – Actions on footways, cycle tracks and footbridges**
- **Section 6 – Traffic actions and other actions specifically for railway bridges**

EN 1991-2: TRAFFIC LOADS ON BRIDGES

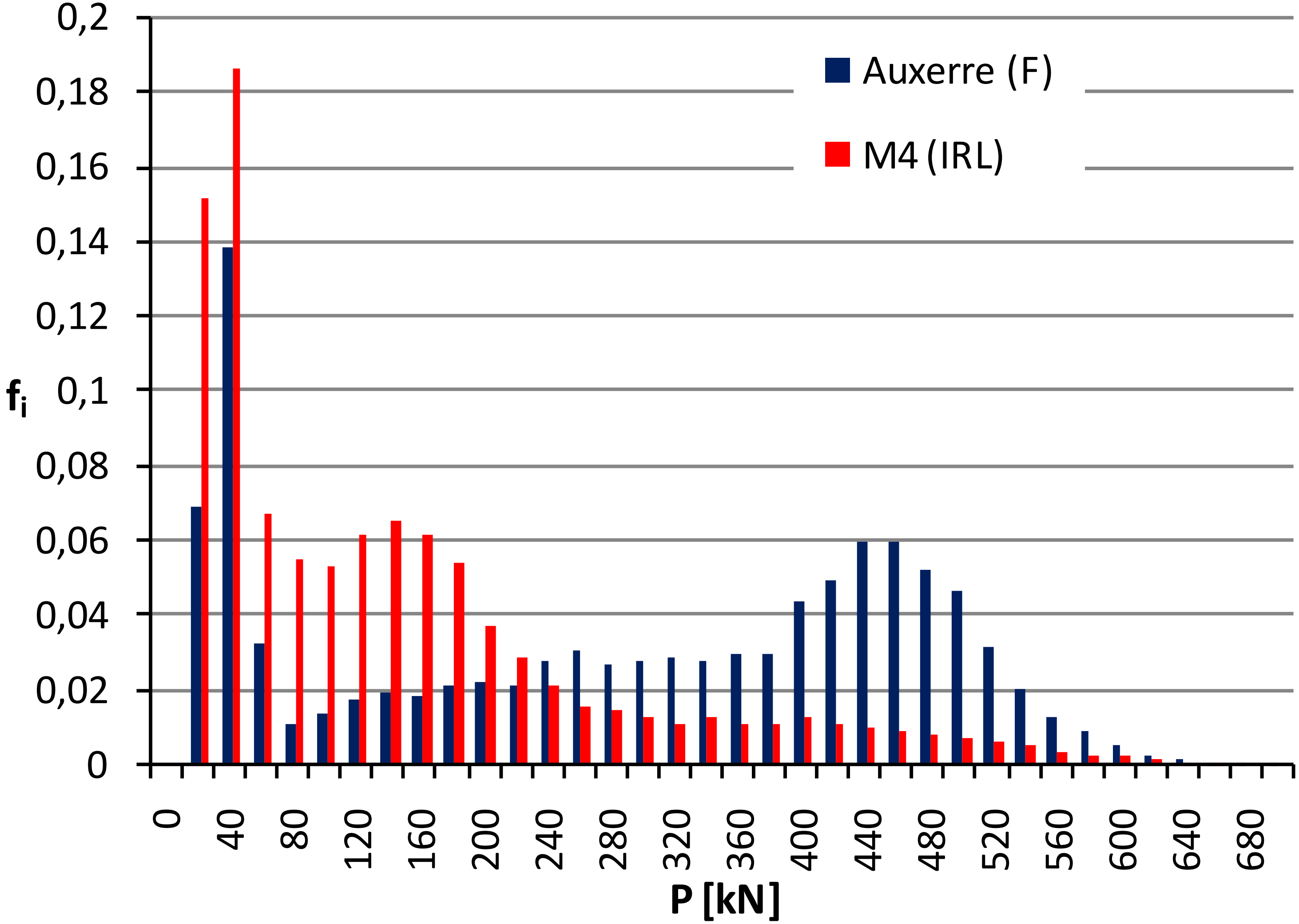
- **Annex A (informative) – Models of special vehicles for road bridges**
- **Annex B (informative) – Fatigue life assessment for road bridges assessment method based on recorded traffic**
- **Annex C (normative) – Dynamic factors 1 + φ for real trains**
- **Annex D (normative) – Basis for the fatigue assessment of railway structures**
- **Annex E (informative) – Limits of validity of load model HSLM and the selection of the critical universal train from HSLM-A**
- **Annex F (informative) – Criteria to be satisfied if a dynamic analysis is not required**
- **Annex G (informative) – Method for determining the combined response of a structure and track to variable actions**
- **Annex F (informative) – Load models for rail traffic loads in transient design situations**

Traffic measurements:



Histogram of the **axle load** frequency –
Auxerre slow lane – lorries

Traffic measurements:



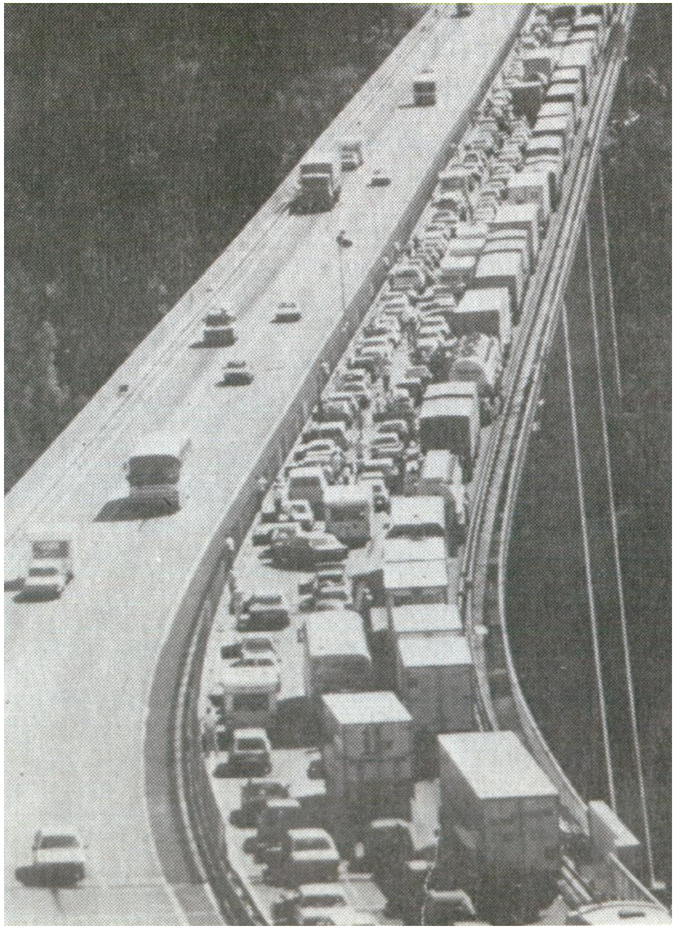
Histograms of the truck **gross weigth** – Auxerre slow lane and M4 motorway (Ireland)

Load models should:

- be easy to use
- produce main load effects correctly
- be the same for local and global verifications
- cover all possible situations (traffic scenarios)
- correspond to the target reliability levels
- include dynamic effects

EN 1991-2: TRAFFIC LOADS ON BRIDGES

Extreme traffic scenarios



Traffic jam on the Europa Bridge
(from Tschermmenegg)

ACTIONS : TRAFFIC LOADS - General organisation for road bridges

Traffic load models

- Vertical forces : LM1, LM2, LM3, LM4
- Horizontal forces : braking and acceleration, centrifugal, transverse

Groups of loads

- gr1a, gr1b, gr2, gr3, gr4, gr5
- characteristic, frequent and quasi-permanent values

Combination with actions other than traffric actions

Load Models for Road Bridges

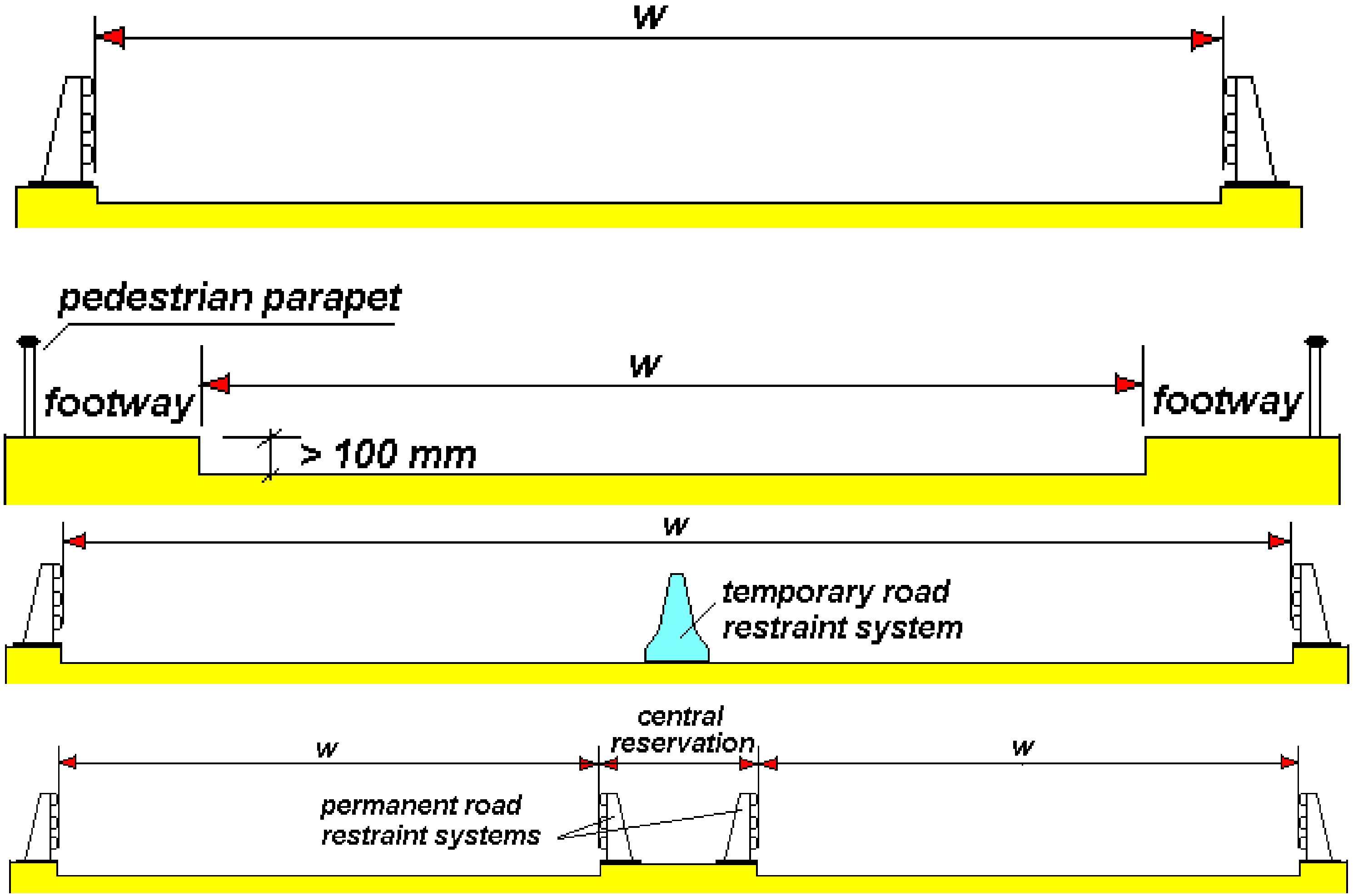
LOAD MODELS FOR LIMIT STATE VERIFICATIONS OTHER THAN FOR FATIGUE LIMIT STATES

Field of application : loaded lengths less than 200 m (maximum length taken into account for the calibration of the Eurocode) and width less than 42 m (for L>200 m they result safe-sided)

- Load Model Nr. 1 - Concentrated and distributed loads (main model)
- Load Model Nr. 2 - Single axle load
- Load Model Nr. 3 - Set of special vehicles *(Can be specified by NA)*
- Load Model Nr. 4 - Crowd loading : 5 kN/m^2

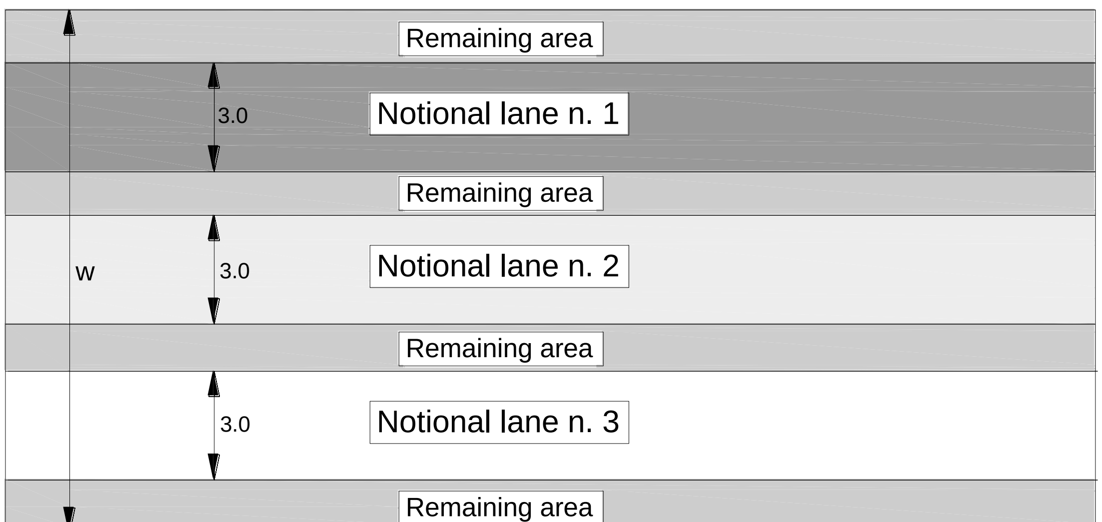
Carriageway width

Carriageway width *w* : width measured between kerbs (height more than 100 mm – recommended value) or between the inner limits of vehicle restraint systems



Division of the carriageway into notional lanes

Carriageway width	Number of notional lanes	Notional lane width	Width of the remaining area
$w < 5{,}4$ m	$n_\ell = 1$	3 m	$w - 3$ m
$5{,}4 \text{ m} \le w < 6$ m	$n_\ell = 2$	$w/2$	0
$6 \text{ m} \le w$	$n_\ell = \mathrm{int}(w/3)$	3 m	$w - 3 \times n_\ell$



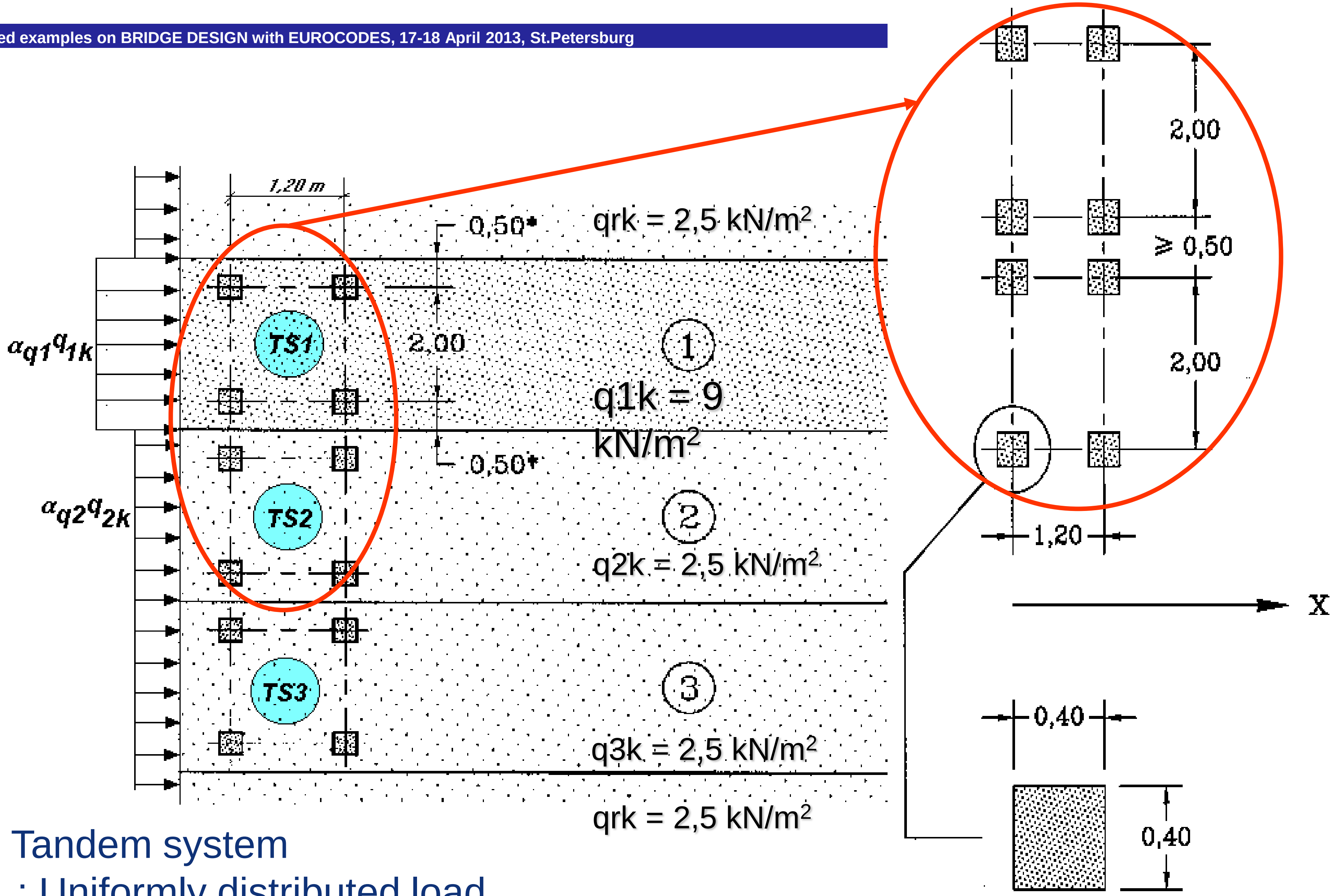
1 – Lane n° 1 (3m)

2 – Lane n° 2 (3m)

3 – Lane n° 3 (3m)

4 – Remaining area

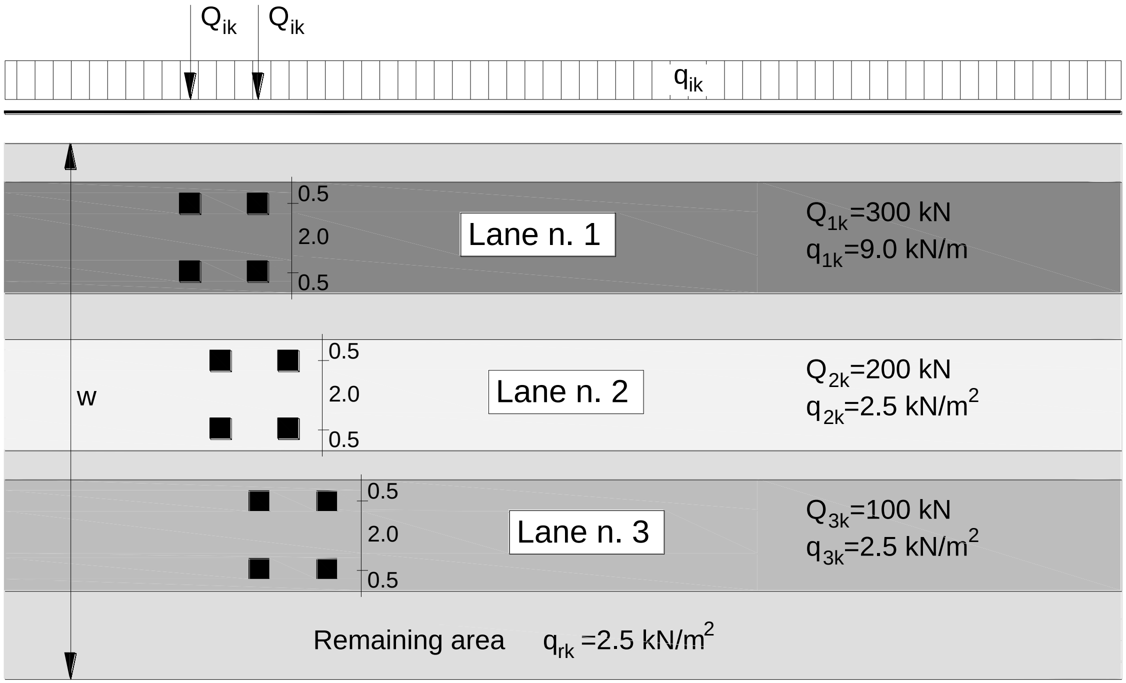
The main load model (LM1)



TS : Tandem system

UDL : Uniformly distributed load

The main load model for road bridges (LM1) : diagrammatic representation



For the determination of general effects, the tandems travel along the axis of the notional lanes

For local verifications, the heaviest tandem should be positioned to get the most unfavourable effect.

Where two tandems are located in two adjacent notional lanes, they may be brought closer, the distance between axles being not less than 0,50 m

Load model 1 : characteristic values

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

Load models for road bridges: LM1

The main load model (LM1): Concentrated and uniformly distributed loads, covers most of the effects of the traffic of lorries and cars.

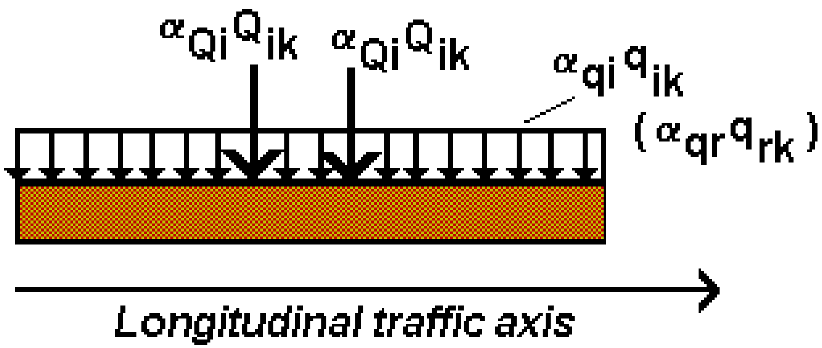
Recommended values of α_{Qi} (α_{Q1}>0.8) , α_{qi} = 1

Example of other values for α factors (NDPs) :

Q_{1k} = 300 kN

Q_{2k} = 200 kN

Q_{3k} = 100 kN



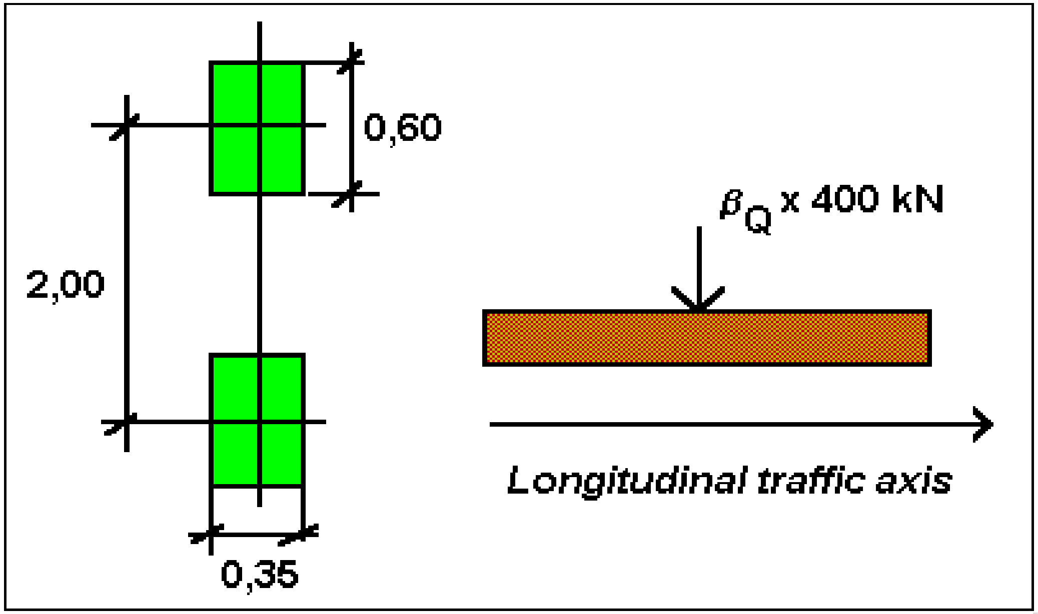
	α_{Q1}	α_{Qi} $i \geq 2$	α_{q1}	α_{qi} $i \geq 2$	α_{qr}
1st class	1	1	1	1	1
2nd class	0,9	0,8	0,7	1	1
3rd class	0,8	0,5	0,5	1	1

1st class : international heavy vehicle traffic
2nd class : « normal » heavy vehicle traffic
3rd class : « light » heavy vehicle traffic

For the example:

$\alpha_{Qi} = \alpha_{qi} = 1$

Load models for road bridges : LM2 – isolated single axle



Recommended value :

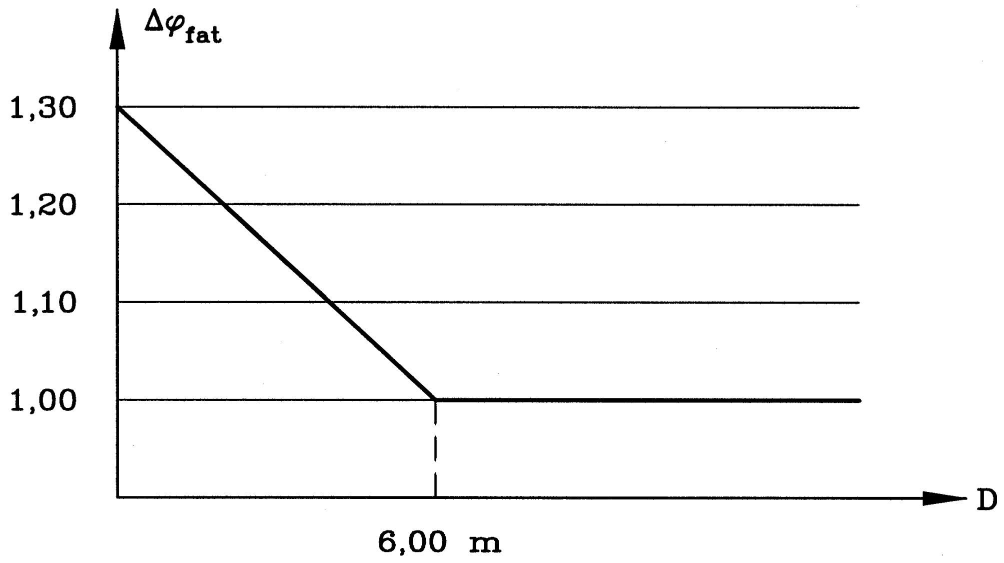
$$\beta_Q = \alpha_{Q1}$$

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

For the example : β_Q = 1

when relevant, only one wheel of 200 (kN) may be taken into account

Representation of the additional amplification factor

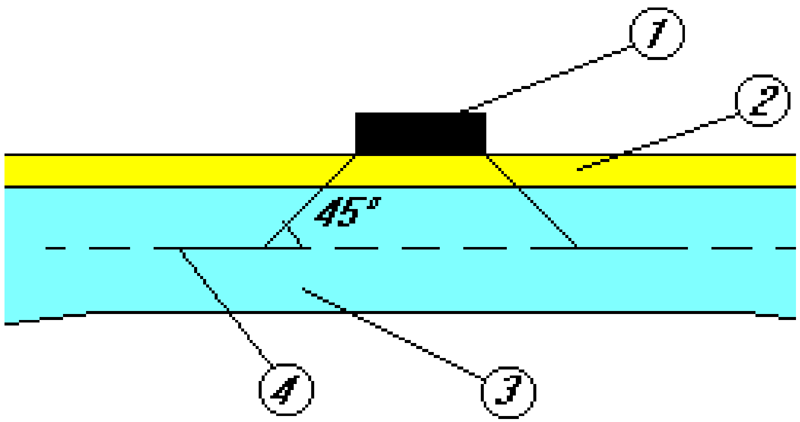


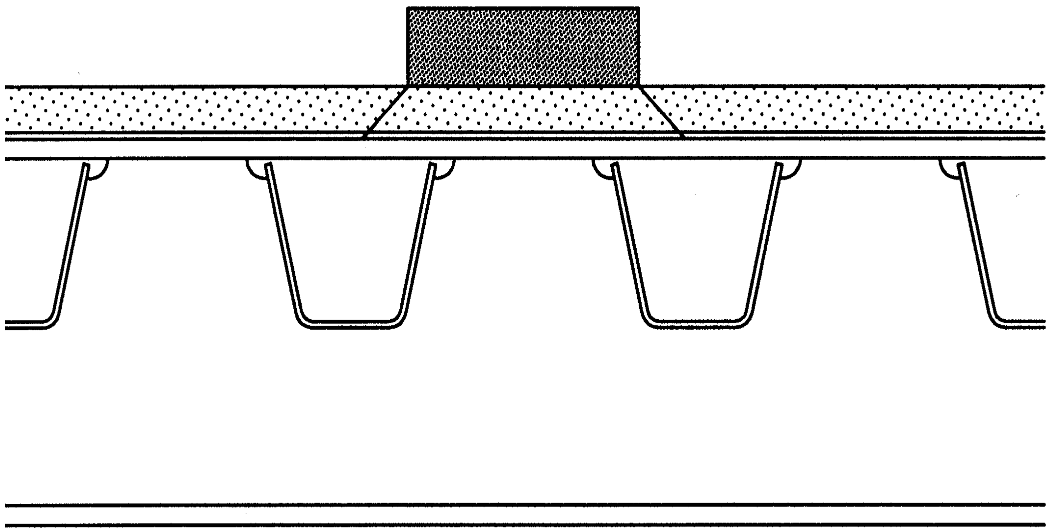
$\Delta\varphi_{fat}$ **: Additional amplification factor**

D **: Distance of the cross-section under consideration from the expansion joint**

Load models for road bridges

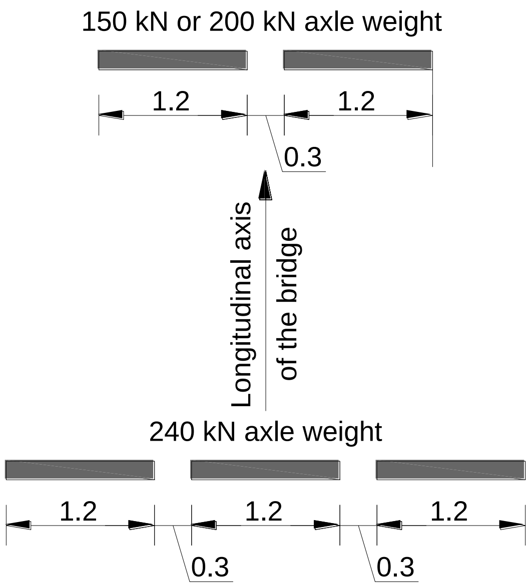
Dispersal of concentrated loads





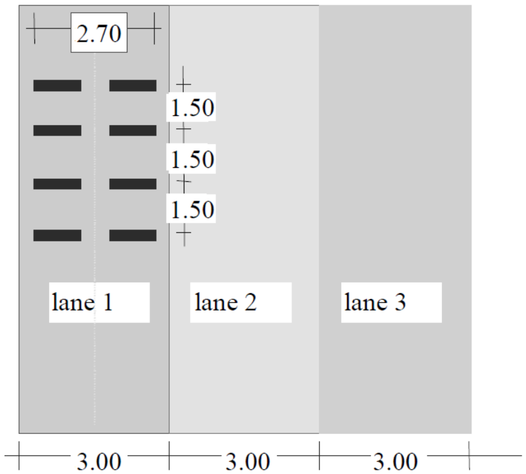
1 – Contact pressure of the wheel
2 – Surfacing
3 – Concrete slab
4 – Slab neutral axis

Load models for road bridges : LM3 – Special vehicles

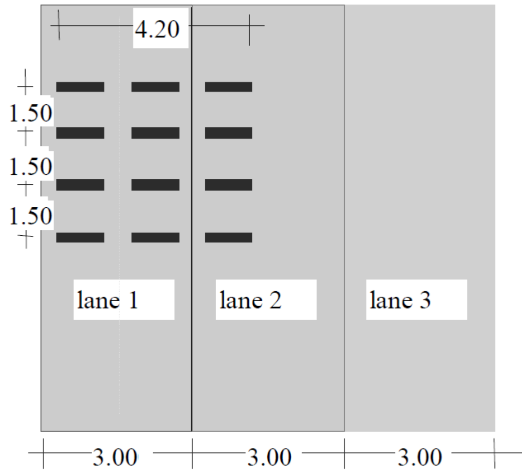


Axle lines and wheel contact areas for special vehicles

Load models for road bridges : LM3 – Special vehicles

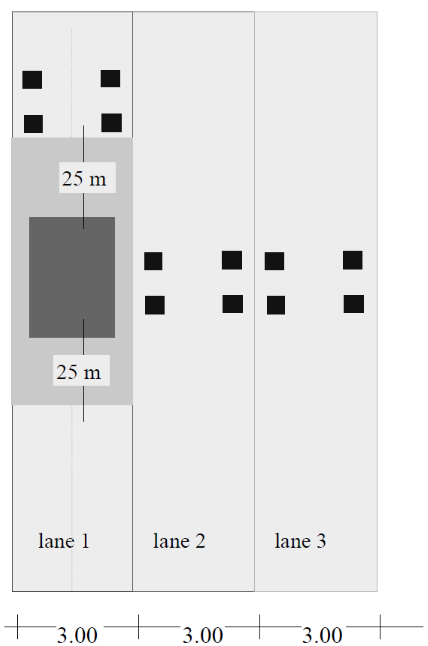


150 or 200 kN axle weight

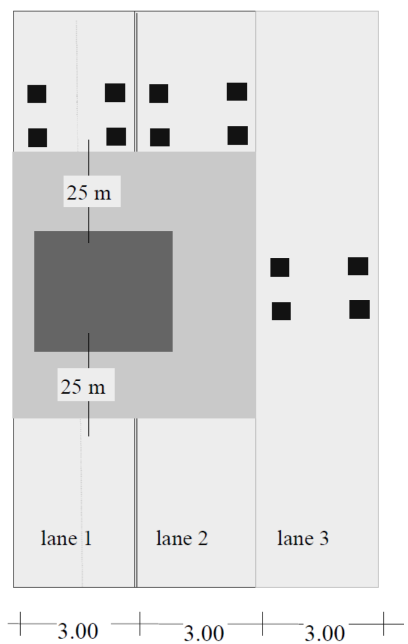


240 kN axle weight

Arrangement of special vehicle on the carriageway



150 or 200 kN axle weight



240 kN axle weight

Simultaneity of special vehicles and load model n. 1

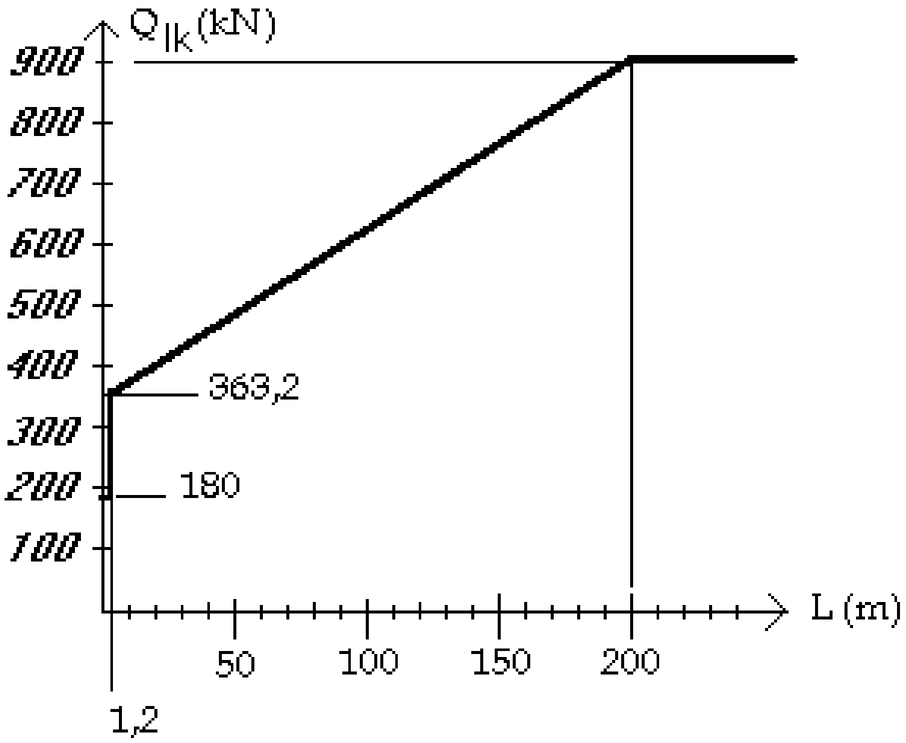
Load models for road bridges : LM4 – Crowd loading

- distributed load 5 kN/m^2 (dynamic effects included)
- combination value 3 kN/m^2 (dynamic effects included)
- to be specified per project
- for global effects
- transient design situation

Load models for road bridges

HORIZONTAL FORCES : Braking and acceleration (Lane Nr. 1)

A characteristic braking force, *Qlk*, is a longitudinal force acting at the surfacing level of the carriageway. *Qlk*, limited to 900 kN for the total width of the bridge, is calculated as a fraction of the total maximum vertical loads corresponding to Load Model 1 and applied on Lane Number 1.



$$Q_{\ell k} = 0,6\alpha_{Q1}(2Q_{1k}) + 0,10\alpha_{q1}q_{1k}w_1L$$

$$180\alpha_{Q1}kN \leq Q_{\ell k} \leq 900\,kN$$

$\alpha Q1 = \alpha q1 = 1$

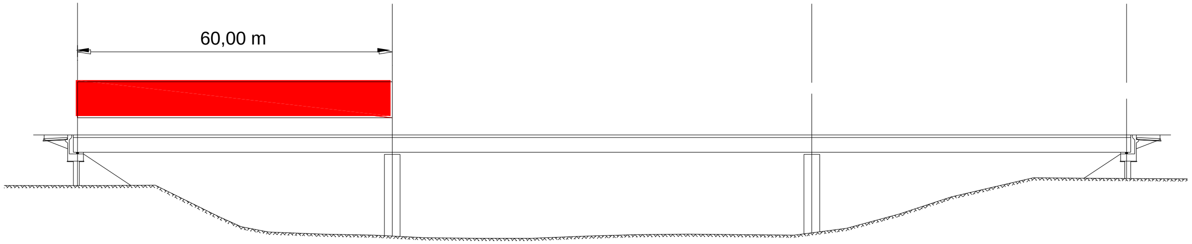
$Q1k = 180 + 2{,}7L$ for $0 \leq L \leq 1{,}2$ m

$Q1k = 360 + 2{,}7L$ for $L > 1{,}2$ m

L = length of the deck or of the part of it under consideration

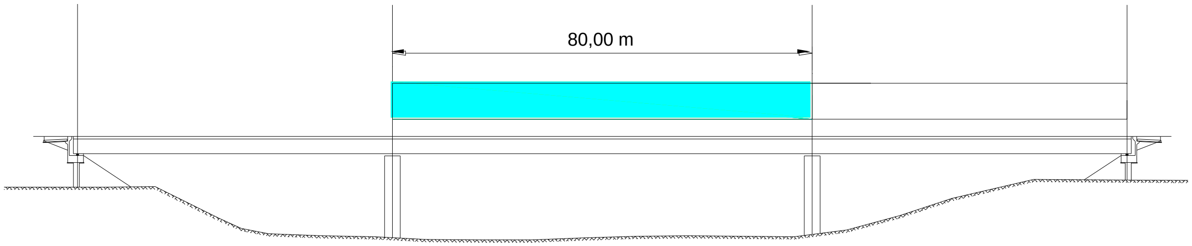
Horizontal forces (braking and acceleration)

$$Q_{\ell k} = 0{,}6(2 \times 300) + 0{,}10 \times 27{,}0 \times L \,[\mathrm{kN}]$$



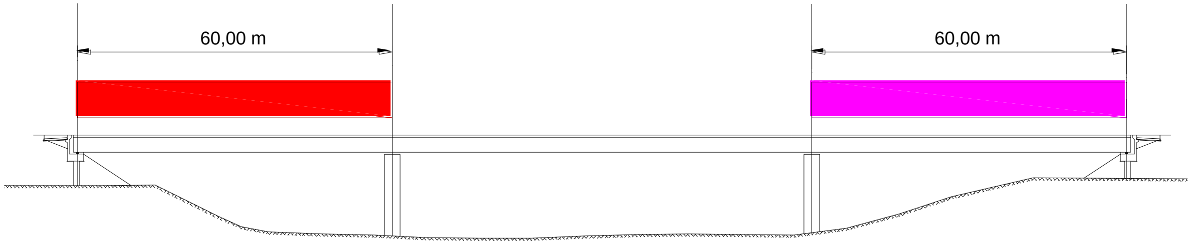
L=60 m

Q_{lk}=522 kN



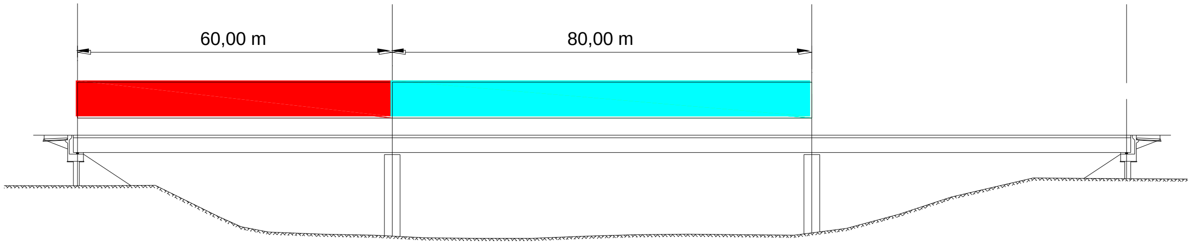
L=80 m

Q_{lk}=577 kN



L=120 m

Q_{lk}=685 kN



L=140 m

Q_{lk}=739 kN

Load models for road bridges

HORIZONTAL FORCES : Centrifugal forces

$Q_{fk} = 0{,}2Q_v \quad kN$	for *r* < 200 m
$Q_{fk} = 40Q_v / r \quad kN$	for 200 ≤ *r* < 1500 m
$Q_{fk} = 0$	for *r* > 1500 m

r : horizontal radius of curvature of the carriageway centreline [m]

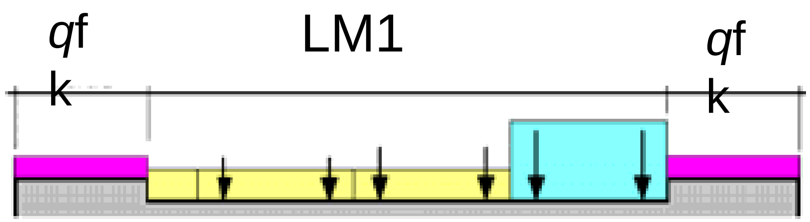
Q_v : total maximum weight of vertical concentrated loads of the tandem systems of LM1

$$\sum_i \alpha_{Qi}(2Q_{ik})$$

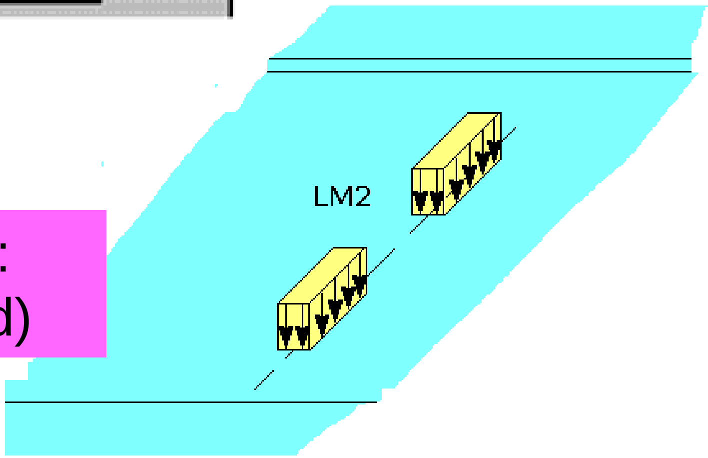
Qfk should be taken as a transverse force acting at the finished carriageway level and radially to the axis of the carriageway.

Definition of groups of loads

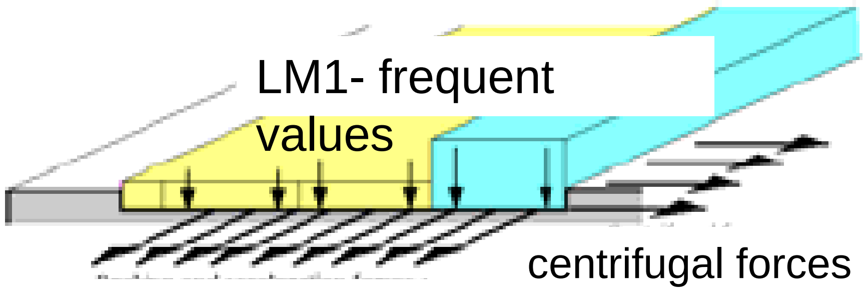
Group of loads gr1a : LM1 + *combination value* of pedestrian load on footways or cycle tracks



Group of loads gr1b : LM2 (single axle load)



Group of loads gr2 : *characteristic values* of horizontal forces, *frequent values* of LM1



Group of loads gr3 : loads on footways and cycle tracks

Group of loads gr4 : crowd loading

Group of loads gr5 : special vehicles (+ special conditions for normal traffic)

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

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

a) If specified, may be defined in the National Annex.
b) May be defined in the National Annex. Recommended value : 3 kN/m^2.
c) See 5.3.2.1-(3). One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.
d) This group is irrelevant if gr4 is considered.

Partial factors γG and γQ - EN 1990, A2, Tables A2.4(A) to (C)

Limit states	Load effects	γG	γQ
A-EQU	Unfavourable	1,05	1,50
	Favourable	0,95	0,00
B-STR/GEO	Unfavourable	1,35	1,50 [1)]
	Favourable	1,00	0,00
C- STR/GEO	Unfavourable	1,00	1,30
	Favourable	1,00	0,00

[1)] For **road traffic 1,35**, for railway traffic 1,45

ψ factors for road bridges

Action	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads (see EN 1991-2, Table 4.4)	gr 1a (LM1) TS	0,75	0,75	0
	gr 1a (LM1) UDL	0,40	0,40	0
	gr1b (single axle)	0	0,75	0
	gr2 (horizontal forces)	0	0	0
	gr3 (pedestrian loads)	0	0,4	0
	gr4 (LM4 crowd loading)	0	0	0
	gr5 (LM3 spec. vehicles)	0	1	0
Wind forces	F_w persistent (execution)	0,6 (0,8)	0,2	0
Thermal actions	T	0,6	0,6	0,5
Snow loads	S_n (during execution)	0,8	-	0
Construction loads	Q_{ca}	1	-	1

Combinations of actions in EN 1990

Ultimate limit states:

EQU – static equilibrium	(6.7)	$E_{d,dst} \leq E_{d,stb}$
STR, GEO	(6.10)	$E_d \leq R_d$
Accidental	(6.11)	
FAT - fatigue		

Serviceability limit states:

characteristic - irreversible	(6.14)	
frequent - reversible	(6.15)	$E_d \leq C_d$
quasi-permanent – long-term	(6.16)	

Combination rules for ULS

- Persistent and transient design situation – fundamental action combinations

(A) $$\sum_{j\geq 1}\gamma_{Gj}G_{kj}+\gamma_P P_k+\gamma_{Q1}Q_{k1}+\sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki} \quad (6.10)$$

or

(B) $$\sum_{j\geq 1}\gamma_{Gj}G_{kj}+\gamma_P P_k+\sum_{i\geq 1}\gamma_{Qi}\psi_{0i}Q_{ki} \quad (6.10a)$$

$$\sum_{j\geq 1}\xi_j\gamma_{Gj}G_{kj}+\gamma_P P_k+\gamma_{Q1}Q_{k1}+\sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki} \quad (6.10b)$$

- Accidental design situation

$$\sum_{j\geq 1}G_{kj}+P_k+A_d+(\psi_{11}\ or\ \psi_{21})Q_{k1}+\sum_{i>1}\psi_{2i}Q_{ki} \quad (6.11b)$$

- Seismic design situation

$$\sum_{j\geq 1}G_{kj}+P_k+A_{Ed}+\sum_{i\geq 1}\psi_{2i}Q_{ki} \quad (6.12b)$$

Combination rules for SLS

- Characteristic – permanent (irreversible) changes

$$\sum_{j\geq 1} G_{kj} + P_k + Q_{k1} + \sum_{i>1} \psi_{0i} Q_{ki} \qquad (6.14)$$

- Frequent – local effects

$$\sum_{j\geq 1} G_{kj} + P_k + \psi_{11} Q_{k1} + \sum_{i>1} \psi_{2i} Q_{ki} \qquad (6.15)$$

- Quasi-permanent – long-term effects

$$\sum_{j\geq 1} G_{kj} + P_k + \sum_{i\geq 1} \psi_{2i} Q_{ki} \qquad (6.16)$$

- Infrequent – concrete bridges

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

Design values of actions (EQU), Set A

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action	Accompanying variable actions	
	Unfavou-rable	Favoura-ble			Main	Others
Eq (6.10)	$\gamma_{Gj,sup}\ G_{kj,sup}$	$\gamma_{Gj,inf}\ G_{kj,ing}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$

Note 1: Recommended values of partial factors:

$\gamma_{Gj,sup}$ = 1,05 for unfavourable effects of permanent actions
$\gamma_{Gj,inf}$ = 0,95 for favourable effects of permanent actions
$\gamma_{Q,1}$ = 1,35 for road and pedestrian traffic actions
$\gamma_{Q,1}$ = 1,45 for rail traffic actions
$\gamma_{Q,i}$ = 1,50 for all other variable actions in persistent design situations
$\gamma_{Q,i}$ = 1,35 for construction loads during execution

For favourable variable actions, $\gamma_Q = 0$.

Combined approach - EQU and STR

Note 2:

Alternative approach may be used (verification of bearing uplift of continuous bridges, and where verification of static equilibrium involves the resistance of structural members).

Recommended values of γ:

$\gamma_{Gj,\mathrm{sup}} = 1{,}35$, $\gamma_{Gj,\mathrm{inf}} = 1{,}25$

$\gamma_Q = 1{,}35$ for road and pedestrian traffic actions

$\gamma_Q = 1{,}45$ for rail traffic actions

$\gamma_Q = 1{,}50$ for all other variable actions in persistent design situation

provided that applying $\gamma_{Gj,\mathrm{inf}} = 1{,}00$ both to the favourable and unfavourable part of permanent actions does not give a more unfavourable effect.

Design values of actions (STR/GEO), Set B

Persistent and transient design situation	Permanent actions		Pres-tress	Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable			Main (if any)	Others
Eq(6.10)	$\gamma_{Gj,sup}\ G_{kj,sup}$	$\gamma_{Gj,inf}, G_{kj,inf}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\,\psi_{0,i} Q_{k,i}$
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)	$\xi\ \gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf}, G_{kj,inf}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\,\psi_{0,i} Q_{k,i}$

$\gamma_{Gj,sup}$ = 1,35 unfavourable effects of permanent actions

$\gamma_{Gj,inf}$ = 1,00 favourable effects of permanent actions

$\gamma_{Q,1}$ = 1,35 unfavourable actions due to road or pedestrian traffic

$\gamma_{Q,1}$ = 1,45 (1,20) for specific actions due to rail traffic

$\gamma_{Q,i}$ = 1,50 for other variable actions in persistent design situations

ξ = 0,85 (- 1,00)

Design values of actions (STR/GEO), set C

Persistent and transient design situation	Permanent actions		Pres-tress	Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable			Main	Others
Eq (6.10)	$\gamma_{Gj,\mathrm{sup}}\, G_{kj,\mathrm{sup}}$	$\gamma_{Gj,\mathrm{inf}}\, G_{kj,\mathrm{inf}}$	$\gamma_P\, P$	$\gamma_{Q,1}\, Q_{k,1}$		$\gamma_{Q,i}\, \psi_{0,i} Q_{k,i}$

$\gamma_{Gj,\mathrm{sup}} = \gamma_{Gj,\mathrm{inf}}$ = 1,0 for permanent actions

$\gamma_{Q,1}$ = 1,15 for unfavourable effects of variable actions due to road and pedestrian traffic

$\gamma_{Q,1}$ = 1,25 for unfavourable effects of variable actions due to rail traffic

$\gamma_{Q,i}$ = 1,3 for variable actions due to horizontal earth pressures (soil, ground water) in persistent design situations

$\gamma_{Q,i}$ = 1,3 for all other unfavourable effects of variable actions

Design values of actions in accidental and seismic design situations

Design situation	Permanent actions		Pres-tress	Accidental or seismic action	Accompanying variable actions	
	Unfavourable	Favourable			Main	Others
Eq (6.11a/b)	$G_{kj,\text{sup}}$	$G_{kj,\text{inf}}$	P	A_d	$\psi_{1,1}$ (or $\psi_{2,1}$) Q_{k1}	$\psi_{2,i}Q_{k,i}$
Eq (6.12 a/b)	$G_{kj,\text{sup}}$	$G_{kj,\text{inf}}$	P	$A_{Ed} = \gamma_I A_{Ek}$	$\psi_{2,i}\,Q_{k,i}$	

Design values of actions in the serviceability limit states

Combination	Permanent actions		Variable actions	
Characteristic	$G_{kj,\,sup}$	$G_{kj,\,inf}$	$Q_{k,1}$	$\psi_{0,i}\,Q_{k,i}$
Frequent	$G_{kj,\,sup}$	$G_{kj,\,inf}$	$\psi_{1,1}\,Q_{k,1}$	$\psi_{2,i}\,Q_{k,i}$
Quasi-permanent	$G_{kj,\,sup}$	$G_{kj,\,inf}$	$\psi_{2,1}\,Q_{k,1}$	$\psi_{2,i}\,Q_{k,i}$

Fundamental combination of actions

Leading action, accompanying

Eq. (6.10)

$$\sum_{j\geq 1}(1{,}35\, G_{kj,\mathrm{sup}} \text{ or } 1{,}00\, G_{kj,\mathrm{inf}})\ \text{"+"}\ (1{,}00 \text{ or } 0)\times S\ \text{"+"} \begin{cases} 1{,}35\times\overbrace{\left(TS+UDL+q^{*}_{fk}\right)}^{\text{gr1a}}+1{,}5\times\begin{cases}\min\left(0{,}6\,F_{Wk},F^{*}_{w}\right)\\ or\ \ 0{,}6\,T_k\end{cases}\\ 1{,}35\ \mathrm{gr1b}\\ 1{,}35\ \mathrm{gr2}+1{,}5\times 0{,}6\,T_k\\ 1{,}35\left(\mathrm{gr3}\ or\ \mathrm{gr4}\right)+1{,}5\times 0{,}6\,T_k\\ 1{,}35\ \mathrm{gr5}\\ 1{,}5T_k+1{,}35\times\underbrace{(0{,}75TS+0{,}4UDL+0{,}4q^{*}_{fk})}_{\psi 0\mathrm{gr1a}}\\ 1{,}5F_{Wk}\\ 1{,}5Q_{Sn,k}\end{cases}$$

TS tandem system, *UDL* uniformly distributed load

The ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO.

Characteristic combination of actions (SLS)

Leading action, accompanying

$$\sum_{j\geq 1}(G_{kj,\sup} \text{ or } G_{kj,\inf}) \text{"+"} (1{,}00 \text{ or } 0) \times S \text{"+"} \begin{cases} \overbrace{\left(TS + UDL + q^{*}_{fk}\right)}^{\text{gr1a}} + \begin{cases} \min\left(0{,}6\, F_{Wk}, F^{*}_{w}\right) \\ or \;\; 0{,}6\, T_k \end{cases} \\ \text{gr1b} \\ \text{gr2} + 0{,}6\, T_k \\ \left(\text{gr3 } or \text{ gr4}\right) + 0{,}6\, T_k \\ \text{gr5} \\ T_k + \underbrace{(0{,}75 TS + 0{,}4 UDL + 0{,}4 q^{*}_{fk})}_{\psi 0\text{gr1a}} \\ F_{Wk} \\ Q_{Sn,k} \end{cases}$$

TS tandem system, *UDL* uniformly distributed load

The ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO.

Frequent combination of actions (SLS)

Leading action Accompanying action

ψ1gr1a

$$\sum_{j\geq 1}(G_{kj,\sup} \text{ or } G_{kj,\inf})\text{"+"}(1{,}00 \text{ or } 0)\times S\text{"+"}\begin{cases}\overbrace{(0{,}75TS+0{,}4UDL)}^{\psi_1 gr1a}+0{,}5\,T_k\\0{,}75\,\mathrm{gr}1\mathrm{b}\\0{,}4\,\mathrm{gr}3+0{,}5\,T_k\\0{,}75\,\mathrm{gr}4+0{,}5\,T_k\\0{,}2\,F_{Wk}\\0{,}6\,T_k\end{cases}$$

TS tandem system, *UDL* uniformly distributed load

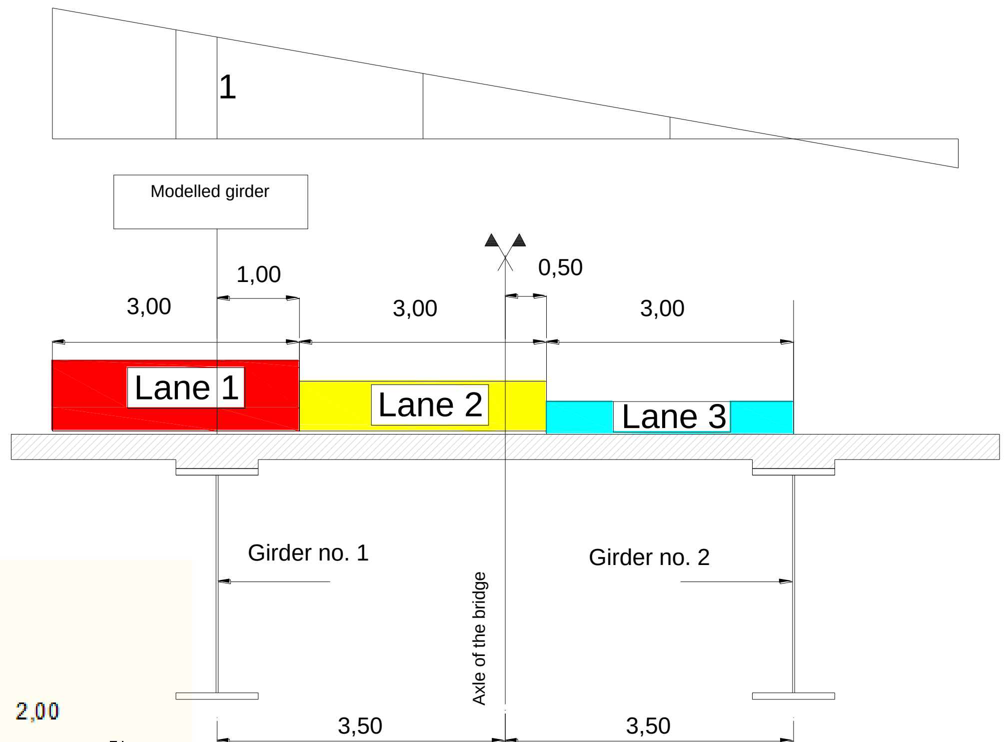
Quasi permanent-combination of actions (SLS)

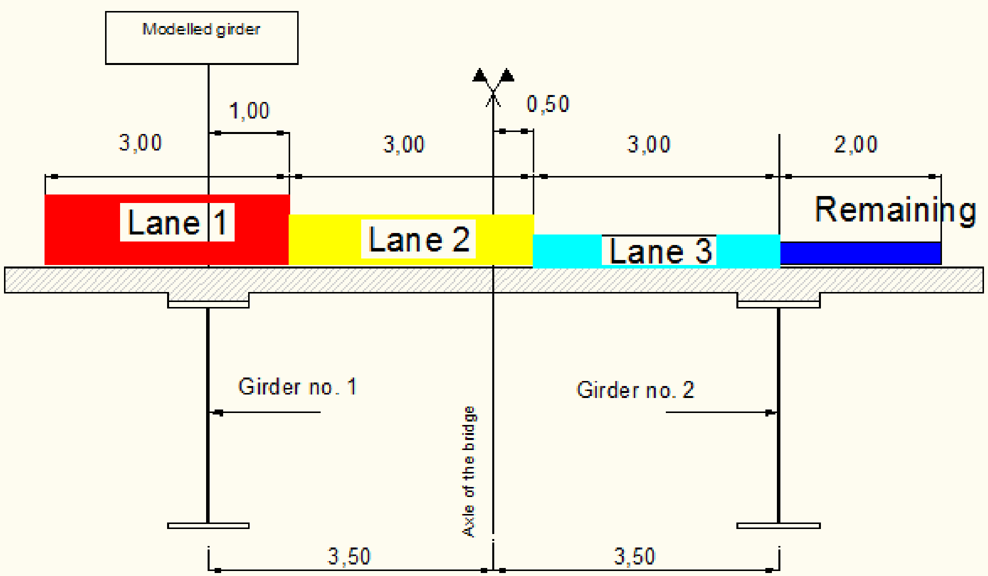
Leading action (no accompanying)

$$\sum_{j\geq 1}(G_{kj,\sup} \text{ or } G_{kj,\inf})\text{ "+" }(1{,}00 \text{ or } 0)\times S\text{ "+" }0{,}5\,T_k$$

Subdivision of the composite bridge in notional lanes

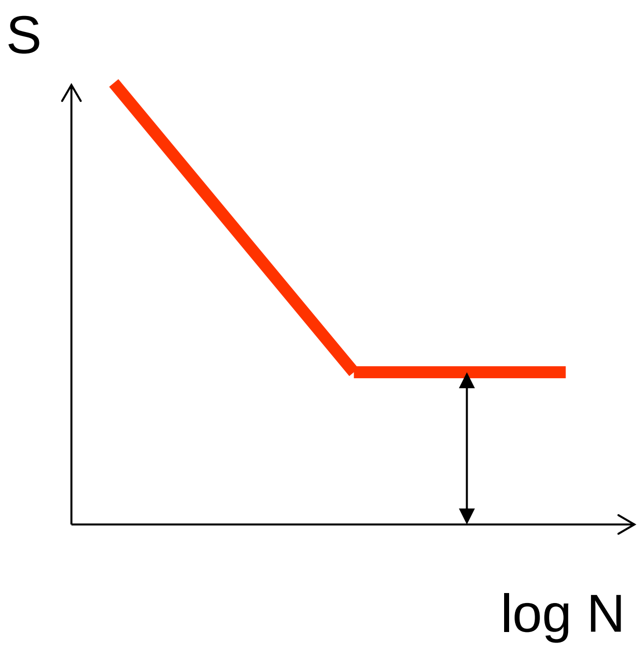
Physical lanes





Notional lanes

Fatigue



Fatigue verification

model 1 = reduced LM1 (0,70 TS + 0,30 UDL)
model 2 = frequent loads (set of typical lorries)
model 3 = N vehicles (1 type)
model 4 = N vehicle (5 types, equivalent loads)
model 5 = real traffic

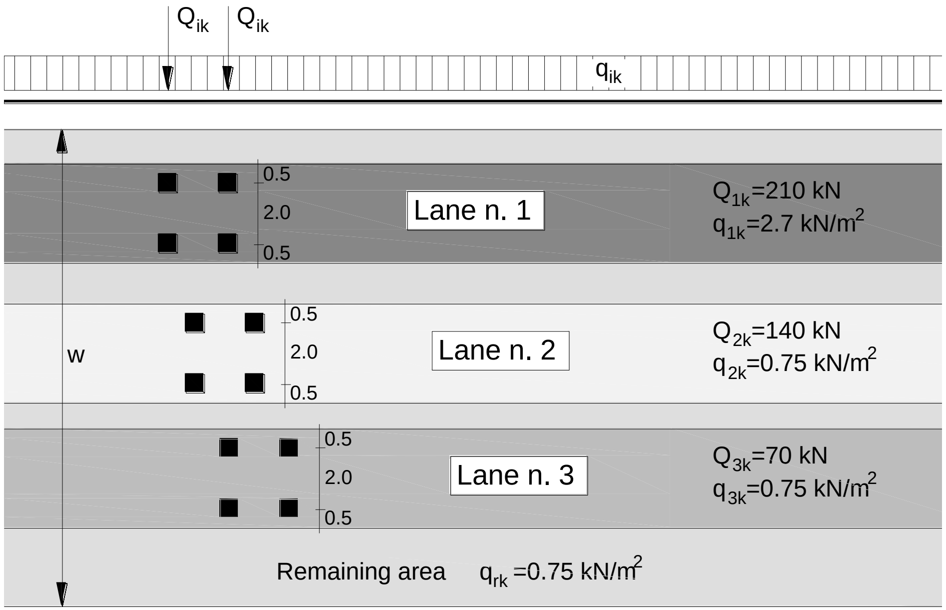
N = 0.05 – 2 million on lane 1 depending on road type

models 1-2: just check whether max stress range S < fatigue limit

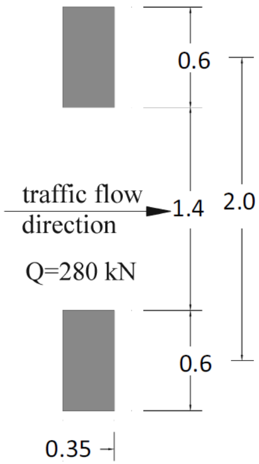
models 3-4: damage assessment

model 5 : general (additional assumptions might be necessary)

Fatigue LM 1



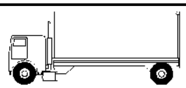
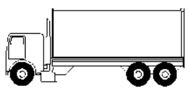
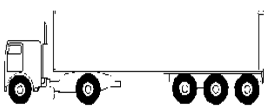
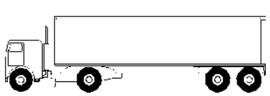
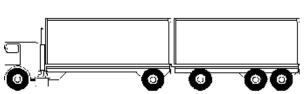
Fatigue load model n. 1

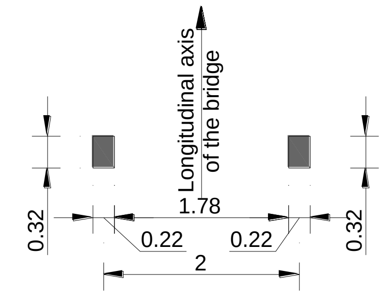
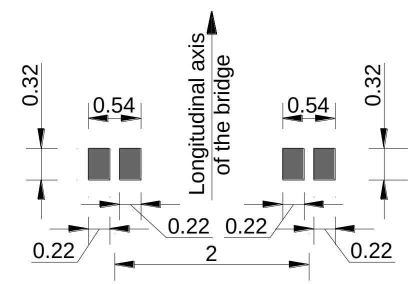
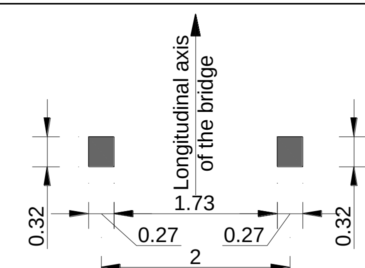


Fatigue load model n. 1 for local verifications

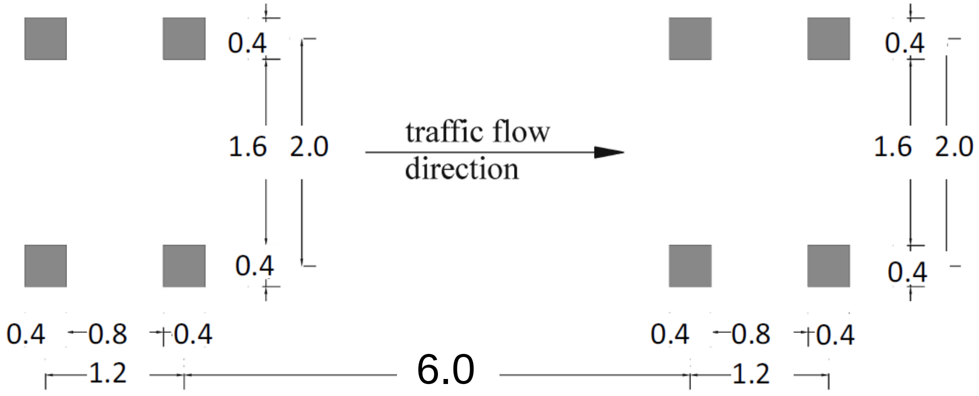
Fatigue LM 2

Fatigue load model n. 2 – frequent set of lorries

LORRY SILHOUETTE	*Interaxles* [*m*]	*Frequent axle loads* [*kN*]	*Wheel type (see table 3)*
	4.5	90 190	A B
	4.20 1.30	80 140 140	A B B
	3.20 5,20 1.30 1.30	90 180 120 120 120	A B C C C
	3.40 6.00 1.80	90 190 140 140	A B B B
	4.80 3.60 4.40 1.30	90 180 120 110 110	A B C C C

Wheel axle type	*Geometrical definition*
A	
B	
C	

Fatigue LM 3



Fatigue load model n. 3 – Axle load 120 kN

Indicative number of lorries expected per year on a slow lane

Traffic categories		*Nobs per year and per slow lane*
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	$2.0 \cdot 10^6$
2	Roads and motorways with medium flow rates of lorries	$0.5 \cdot 10^6$
3	Main roads with low flow rates of lorries	$0.125 \cdot 10^6$
4	Local roads with low flow rates of lorries	$0.05 \cdot 10^6$

Equivalent damage coefficient λ

$$\gamma_{F,fat}\Delta\sigma_{s,equ} = \gamma_{F,fat}\lambda_s\,\Delta\sigma_{s,EC} \leq \frac{\Delta\sigma_{s,Rsk}}{\gamma_{s,fat}}$$

$$\lambda_s = \varphi_{fat}\lambda_1\,\lambda_2\,\lambda_3\,\lambda_4$$

Table 3.1: Recommended values for partial factors for fatigue strength

Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1,00	1,15
Safe life	1,15	1,35

$\Delta\sigma_{s,EC} = \Delta\sigma_{max}$ induced by LM 3 - Problem: calibration of λ values

Equivalent damage coefficient λ

$$\lambda_s = \varphi_{fat} \lambda_1 \lambda_2 \lambda_3 \lambda_4$$

It is reminded the above factor are used to take into account :

φ_{fat} : the quality of surface roughness

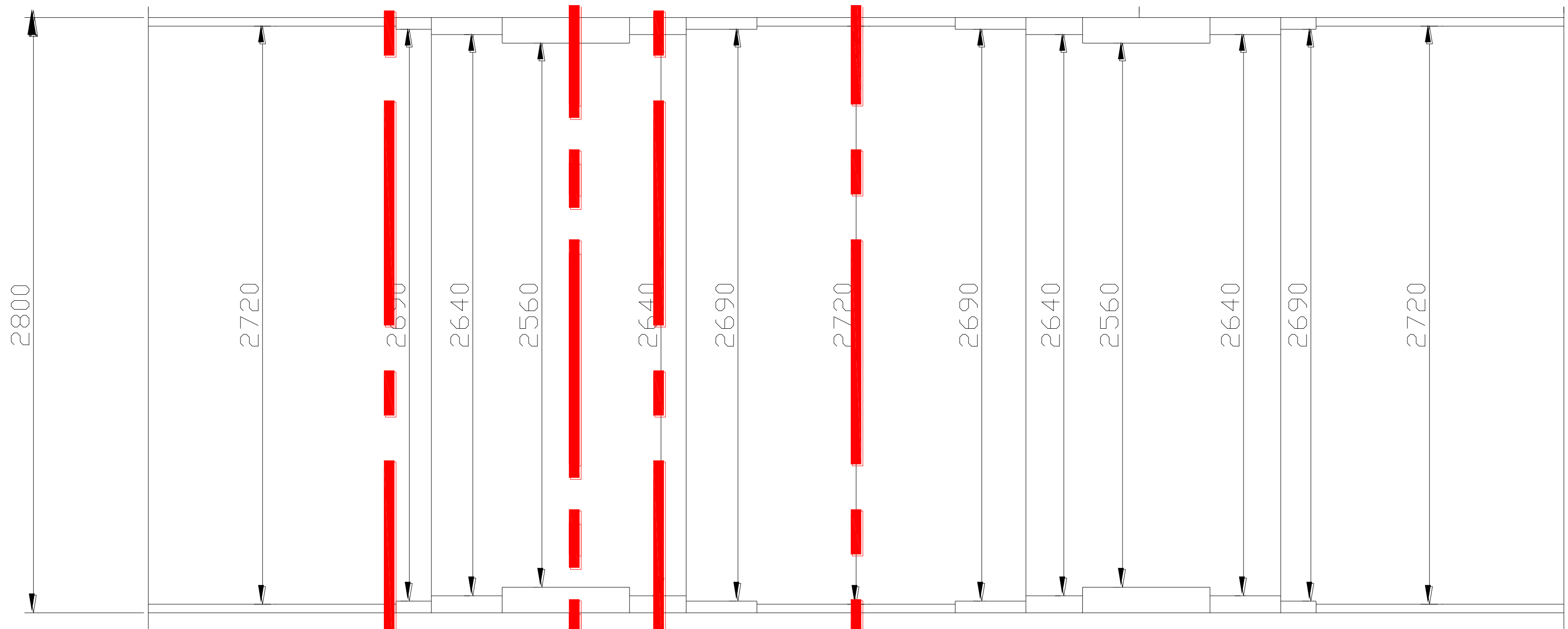
λ_1 : the damaging effect of the traffic (depends on the influence line (span) length)

λ_2 : the expected annual traffic volume

λ_3 : the design working life of the bridge (=1 for T=100 years)

λ_4 : the mult-lane effects

Cross sections taken into account for fatigue assessments



UPPER FLANGE Constant width 1000	35000 × 40	5000 × 55	10000 × 80	18000 × 120	8000 × 80	10000 × 55	28000 × 40	10000 × 55	8000 × 80	18000 × 120	10000 × 80	5000 × 55	35000 × 40
WEB	18 × 40000		26 × 36000			18 × [illegible]8000			26 × 36000			18 × 40000	
LOWER FLANGE Constant width 1200	35000 × 40	5000 × 55	10000 × 80	18000 × 120	8000 × 80	10000 × 55	28000 × 40	10000 × 55	8000 × 80	18000 × 120	10000 × 80	5000 × 55	35000 × 40

support midspan

x=35 m x=72 m

Assumptions considered

- Annual traffic flow of lorries per slow lane set to $\mathbf{0.5 \times 10^6}$, considering **a road with medium flow of lorries** according to EN1991-2 (table 4.5);

- Fatigue life equal to 100 years, consequently the **total lorry flow** per lane resulted $\mathbf{5.0 \times 10^7}$;

- According to table 3.1 of EN1993-1-9, a partial factor for fatigue strength $\gamma_{MF} = \mathbf{1.15}$ has been adopted, considering damage tolerant details and high consequences of fatigue failure;

- Stress cycles have been identified using the **reservoir counting method**, or, equivalently, the **rainflow method**;

- Fatigue damage has been assessed using the **Palmgren-Miner rule**

Damage assessment

Palmgren- Miner rule

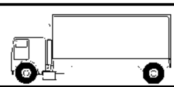
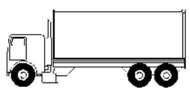
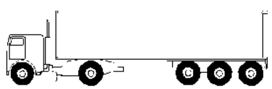
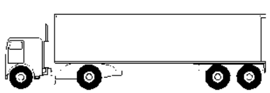
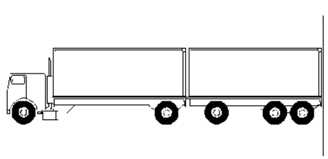
$$D = \frac{\sum_i n_i}{\sum_i N_i}$$

Table 3.1: Recommended values for partial factors for fatigue strength

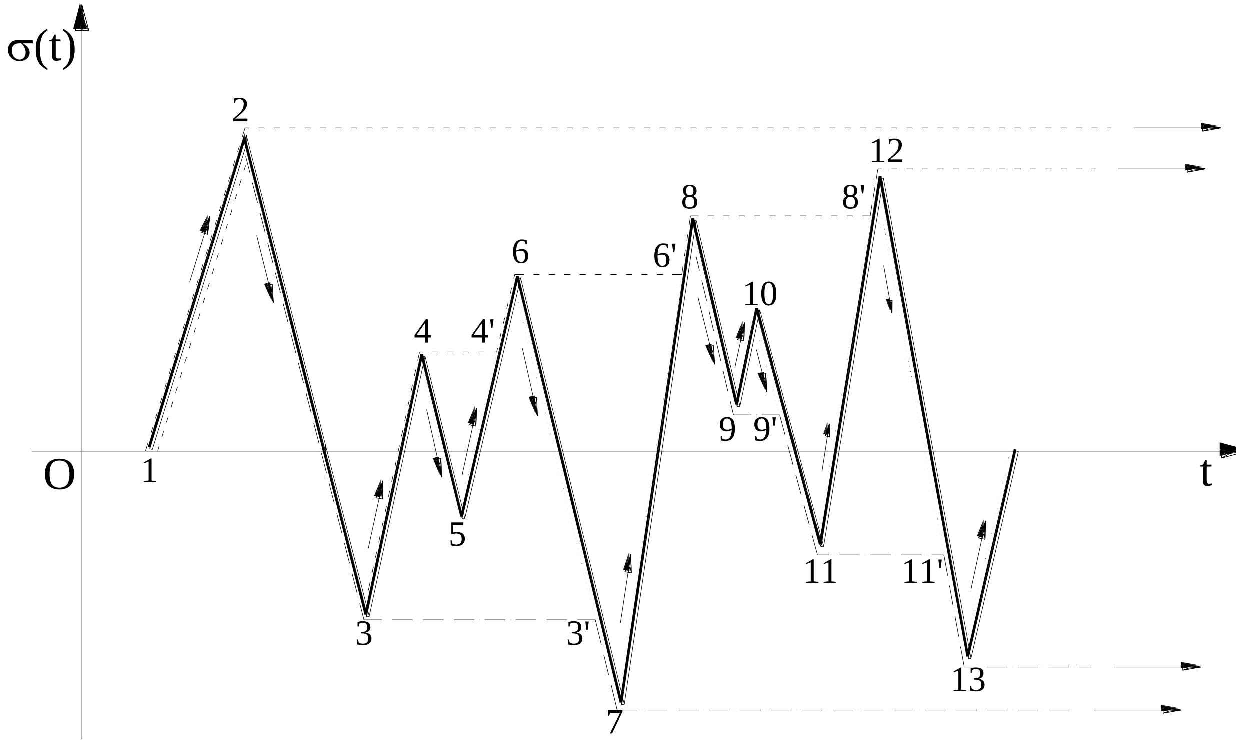
Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1,00	1,15
Safe life	1,15	1,35

Fatigue LM 4

Fatigue load model n. 4 – equivalent set of lorries

LORRY SILHOUETTE			*TRAFFIC TYPE*		
			Long distance	*Medium distance*	*Local traffic*
LORRY	*Axle spacing* [*m*]	*Equivalent Axle loads* [*kN*]	*Lorry percentage*	*Lorry percentage*	*Lorry percentage*
	4.5	70 130	20.0	40.0	80.0
	4.20 1.30	70 120 120	5.0	10.0	5.0
	3.20 5.20 1.30 1.30	70 150 90 90 90	50.0	30.0	5.0
	3.40 6.00 1.80	70 140 90 90	15.0	15.0	5.0
	4.80 3.60 4.40 1.30	70 130 90 80 80	10.0	5.0	5.0

Rainflow method



Traffic flow: 500 000 lorries per years per slow lane

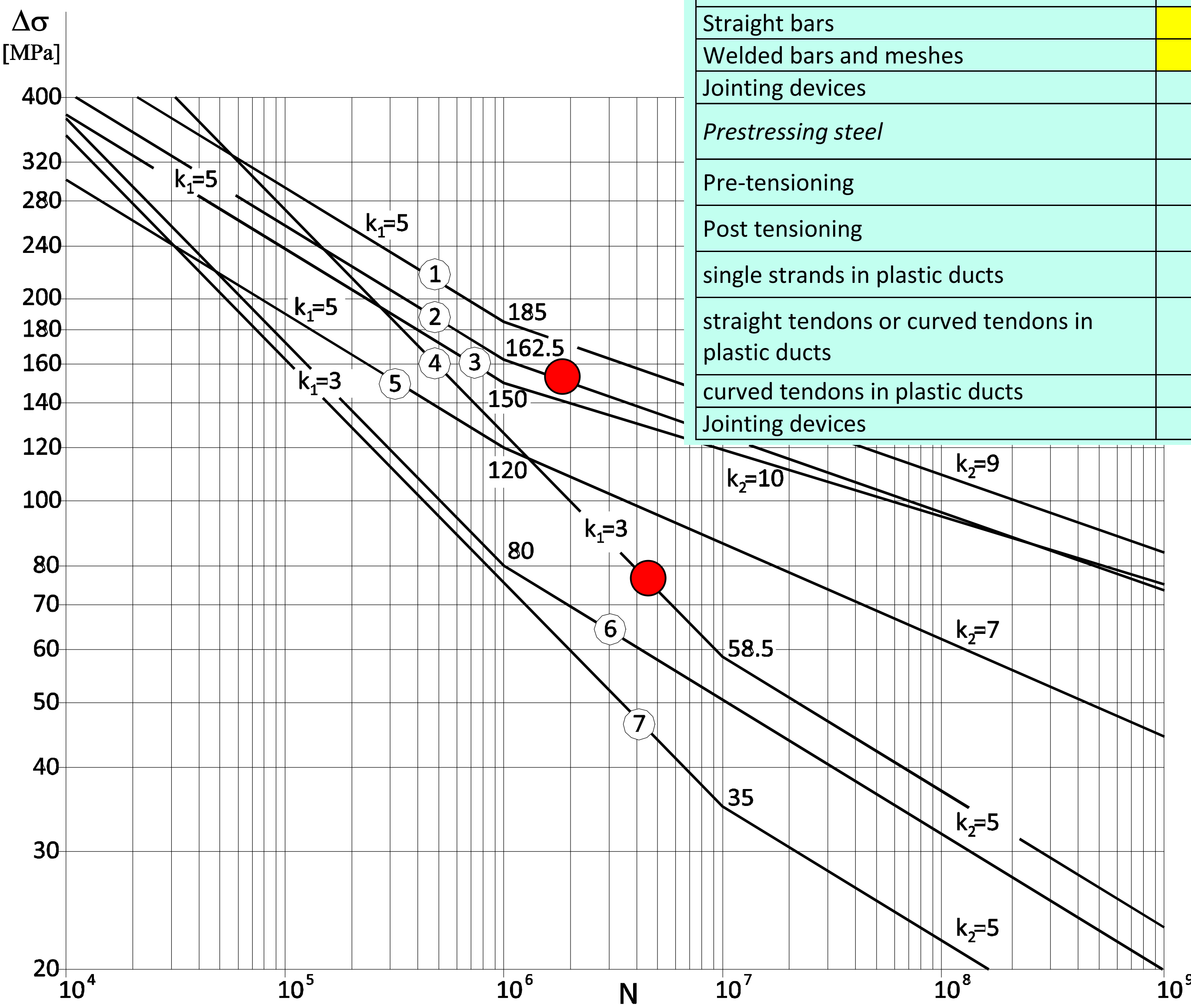
500 000 lorries per year on lane 1

500 000 lorries per year on lane 2

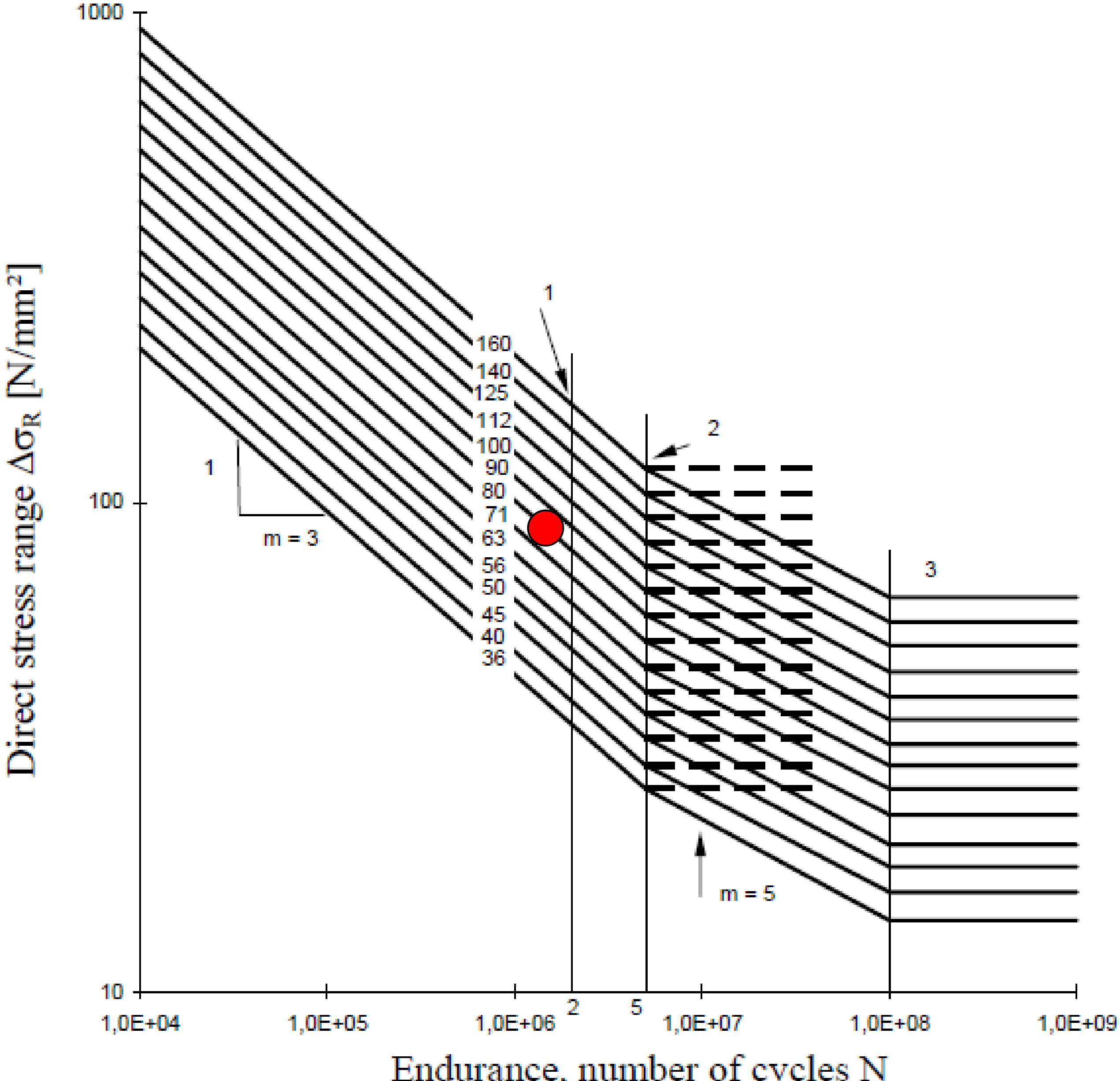
Fatigue life: 100 years

S-N curves for steel reinforcement in concrete

Steel reinforcement	*S-N curve n.*	N*	k_1	k_2	Δσ(N*) [MPa]
Straight bars	2	10^6	5	9	162.5
Welded bars and meshes	4	10^7	3	5	58.5
Jointing devices	7	10^7	3	5	35
Prestressing steel					
Pre-tensioning	1	10^6	5	9	185
Post tensioning					
single strands in plastic ducts	1	10^6	5	9	185
straight tendons or curved tendons in plastic ducts	3	10^6	5	10	150
curved tendons in plastic ducts	5	10^6	5	7	120
Jointing devices	6	10^6	3	5	80



S-N curves for steel details

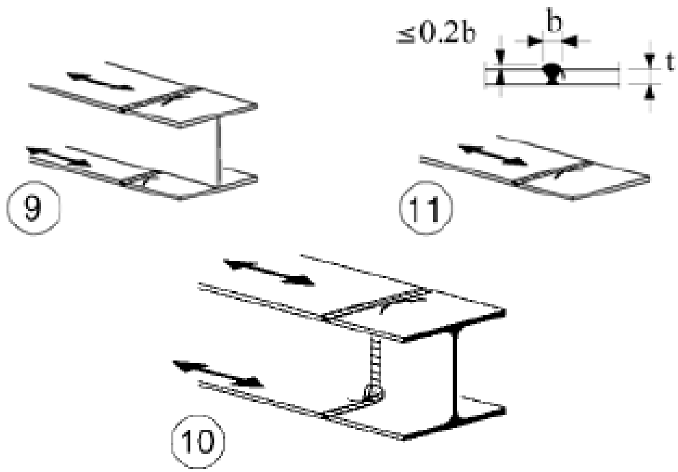


1 Detail category $\Delta\sigma_C$

2 Constant amplitude fatigue limit $\Delta\sigma_D$

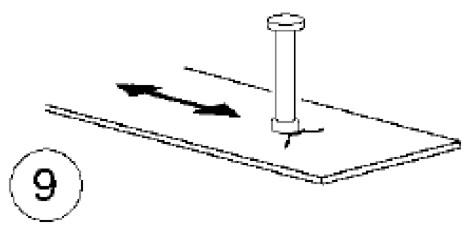
3 Cut-off limit $\Delta\sigma_L$

S-N curves for steel details

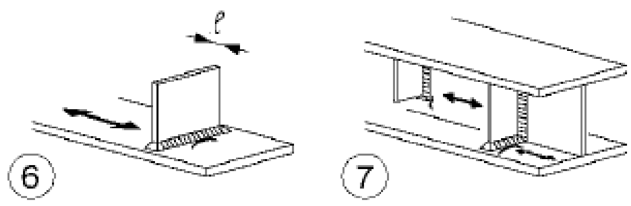
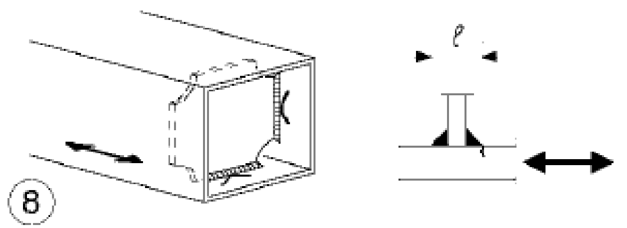
80	size effect for t>25mm: $k_s=(25/t)^{0.2}$		9) Transverse splices in welded plate girders without cope hole. 10) Full cross-section butt welds of rolled sections with cope holes. 11) Transverse splices in plates, flats, rolled sections or plate girders.	- The height of the weld convexity to be not greater than 20% of the weld width, with smooth transition to the plate surface. - Weld not ground flush - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. Detail 10: The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the plate surface.

$$\text{for t} = 40\text{ mm it results } k_s = \left(\frac{25}{40}\right)^{0.2} = 0.91$$

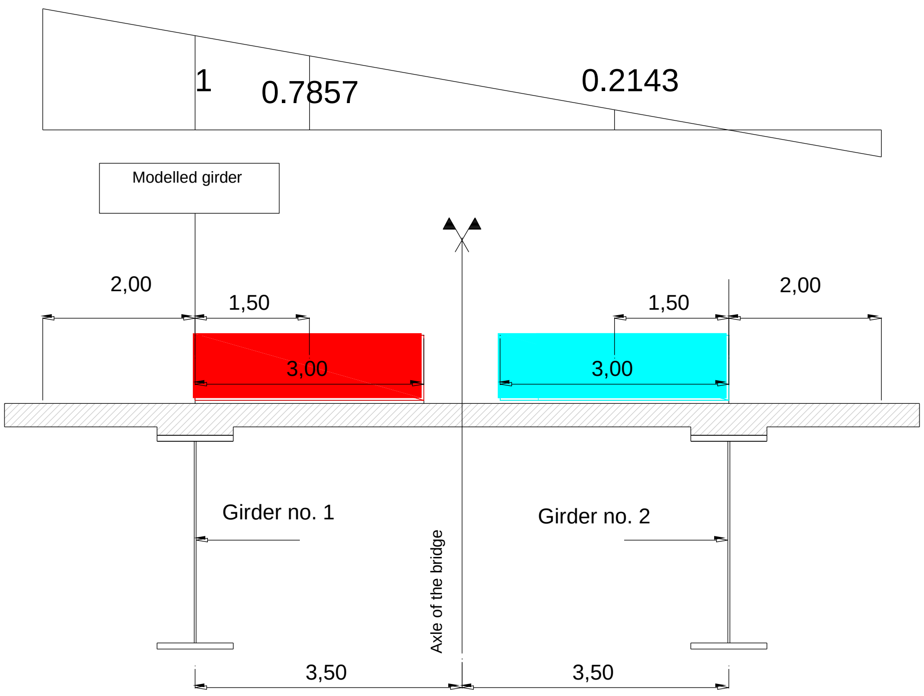
Effective detail class (t=40 mm) ΔσC=72.8

80		9) The effect of welded shear studs on base material.	

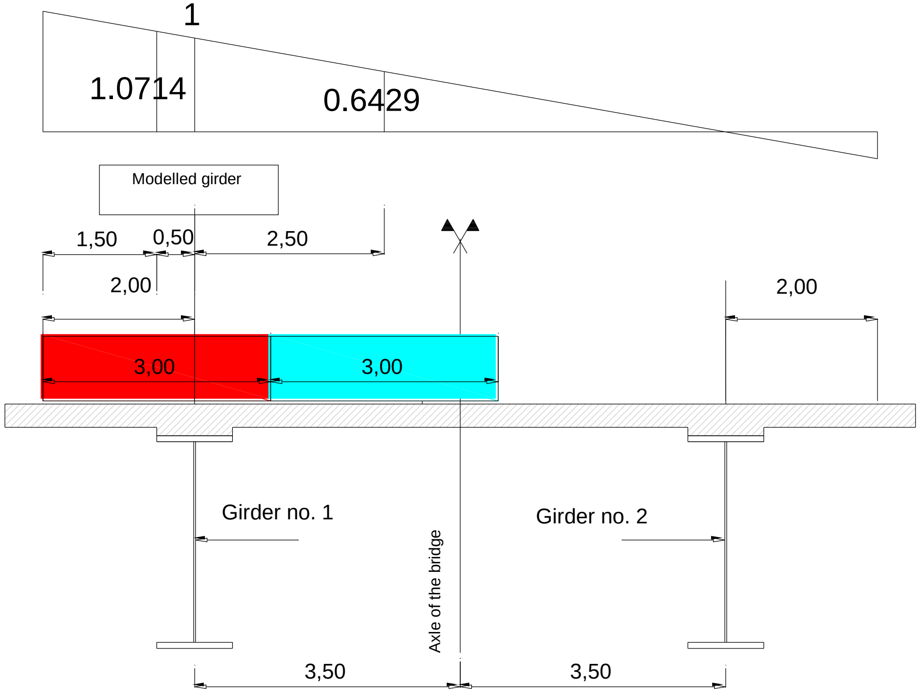
S-N curves for steel details

80	$\ell \leq 50$mm		<u>Transverse attachments:</u> 6) Welded to plate. 7) Vertical stiffeners welded to a beam or plate girder. 8) Diaphragm of box girders welded to the flange or the web. May not be possible for small hollow sections. The values are also valid for ring stiffeners.	<u>Details 6) and 7):</u> Ends of welds to be carefully ground to remove any undercut that may be present. 7) $\Delta\sigma$ to be calculated using principal stresses if the stiffener terminates in the web, see left side.
71	$50 < \ell \leq 80$mm			

Notional lanes for fatigue assessments



Case 1

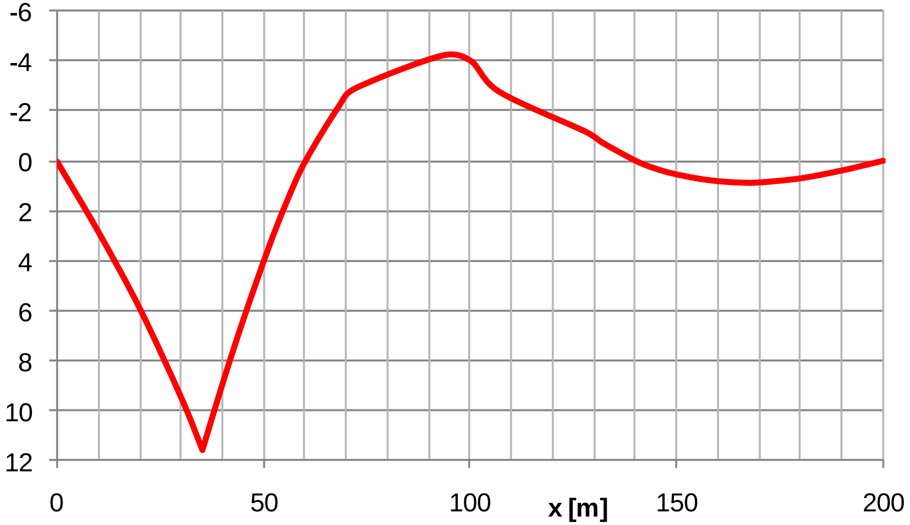


Case 2

Physical lanes (more realistic and used in this example)

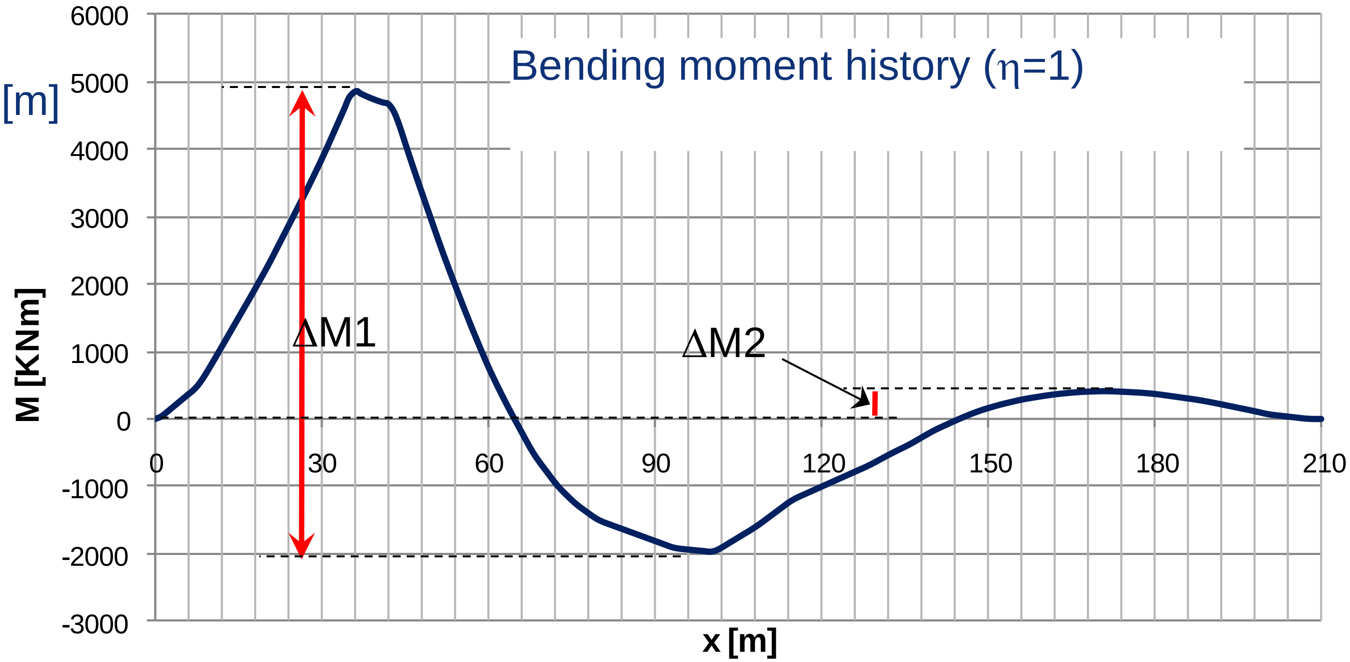
Notional lanes (very severe)

Bending moment history - section x=35 m (uncracked)

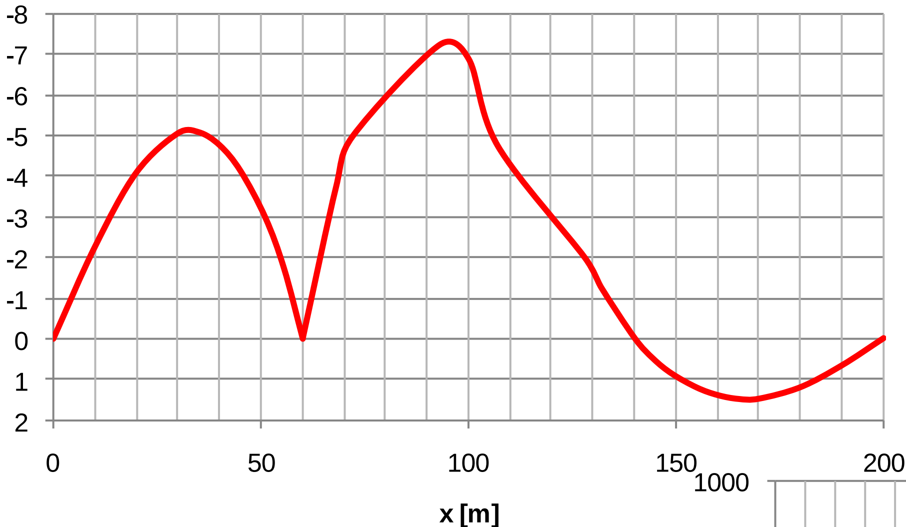


Influence line for bending moment - section x=35 m – [m]

	Case 1	Case 2
$\gamma_{Mf}\,\Delta M_1$ [kNm]	6160.4	8400.5
$\gamma_{Mf}\,\Delta M_2$ [kNm]	1680.1	5040.3
$\gamma_{Mf}\,\Delta M_3$ [kNm]	372.5	507.9
$\gamma_{Mf}\,\Delta M_4$ [kNm]	101.6	304.7
D (upper flange)	0.000E+00	0.000E+00
D (lower flange)	7.470E-01	3.522E+00
D (straight rebar)	6.444E-11	1.061E-09
D (mesh)	2.054E-04	1.042E-03

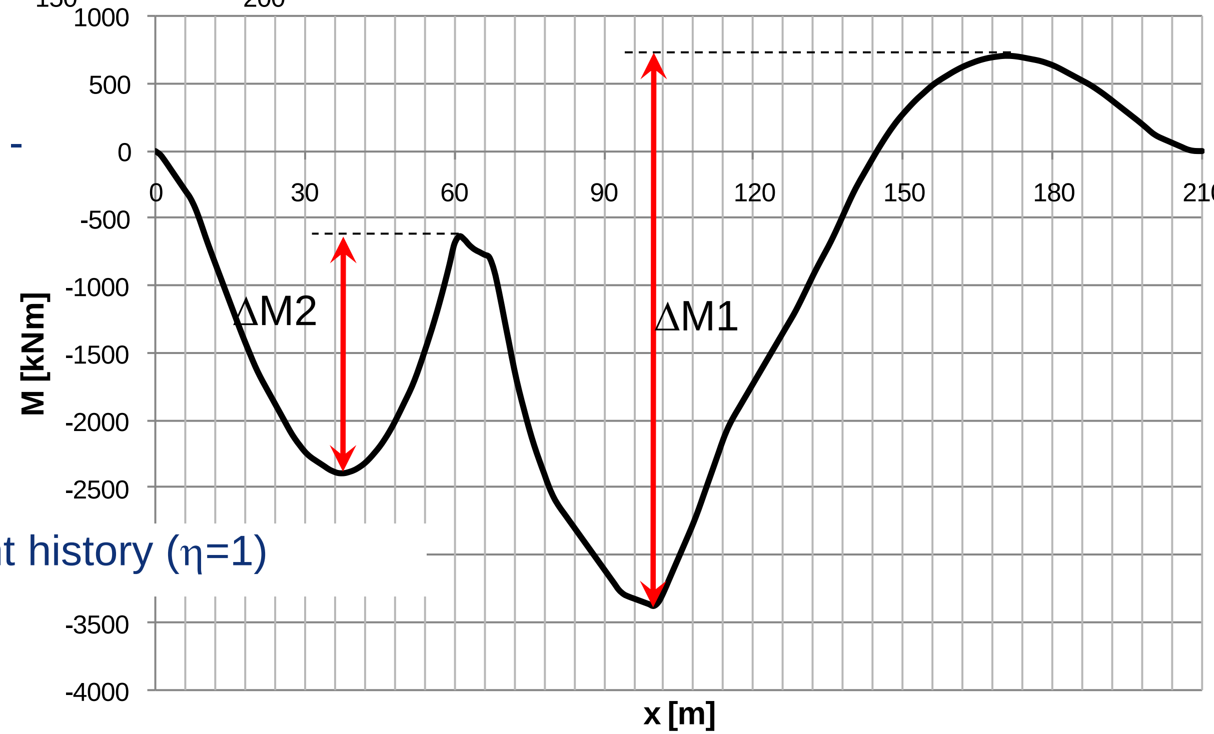


Bending moment history - section x=60 m (support) – (cracked)



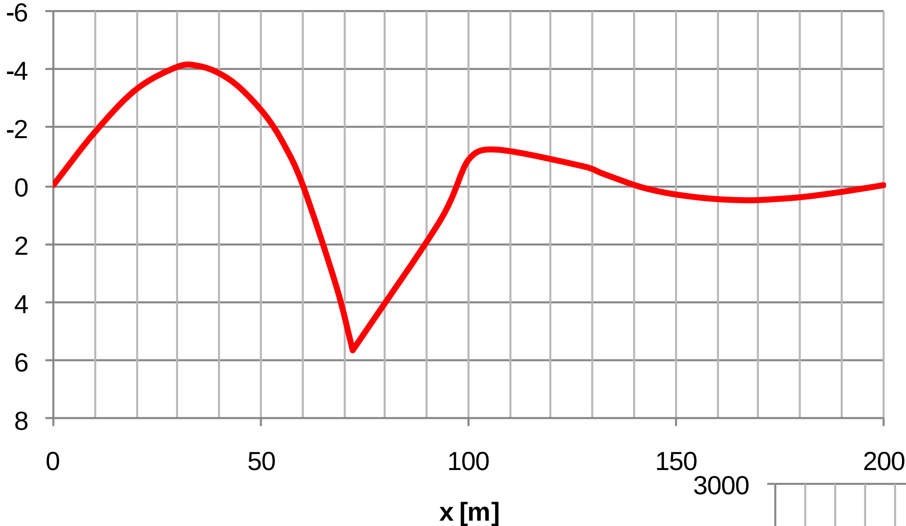
	Case 1	Case 2
$\gamma_{Mf}\,\Delta M_1$ [kNm]	3688.9	5030.3
$\gamma_{Mf}\,\Delta M_2$ [kNm]	1006.1	3018.2
$\gamma_{Mf}\,\Delta M_3$ [kNm]	1589.0	2166.9
$\gamma_{Mf}\,\Delta M_4$ [kNm]	433.4	1300.1
D (upper flange)	0.000E+00	0.000E+00
D (lower flange)	0.000E+00	0.000E+00
D (straight rebar)	1.761E-09	2.900E-08
D (mesh)	1.309E-03	6.644E-03

Influence line for bending moment - section x=60 m – [m]



Bending moment history (η=1)

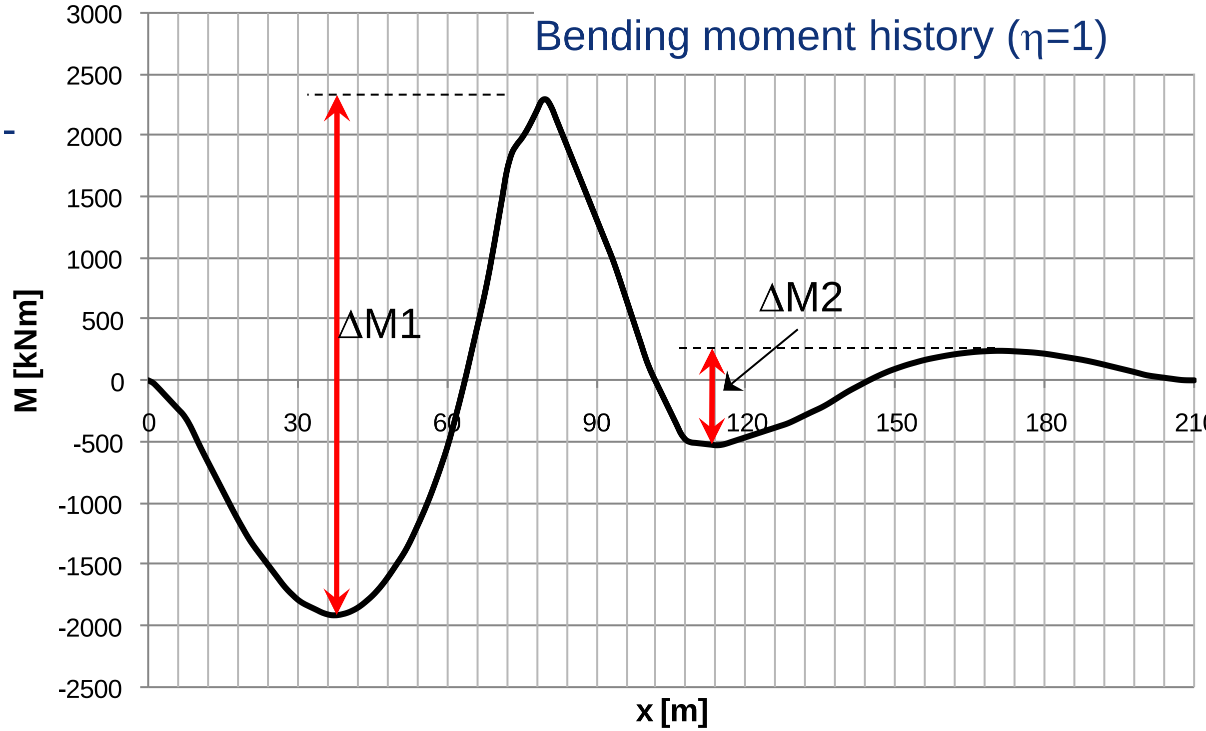
Bending moment history - section x=72 m (cracked)



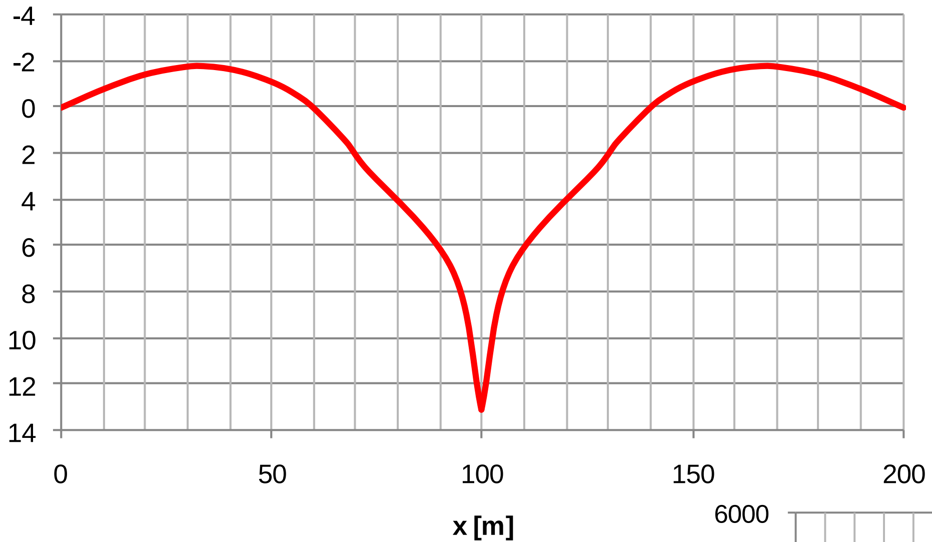
Influence line for bending moment - section x=72 m – [m]

	Case 1	Case 2
$\gamma_{Mf}\,\Delta M_1$ [kNm]	3819.0	5207.7
$\gamma_{Mf}\,\Delta M_2$ [kNm]	1041.5	3124.6
$\gamma_{Mf}\,\Delta M_3$ [kNm]	699.5	953.8
$\gamma_{Mf}\,\Delta M_4$ [kNm]	190.8	572.3
D (upper flange)	0.000E+00	0.000E+00
D (lower flange)	0.000E+00	0.000E+00
D (straight rebar)	2.664E-08	4.387E-07
D (mesh)	5.838E-03	2.962E-02

Bending moment history (η=1)

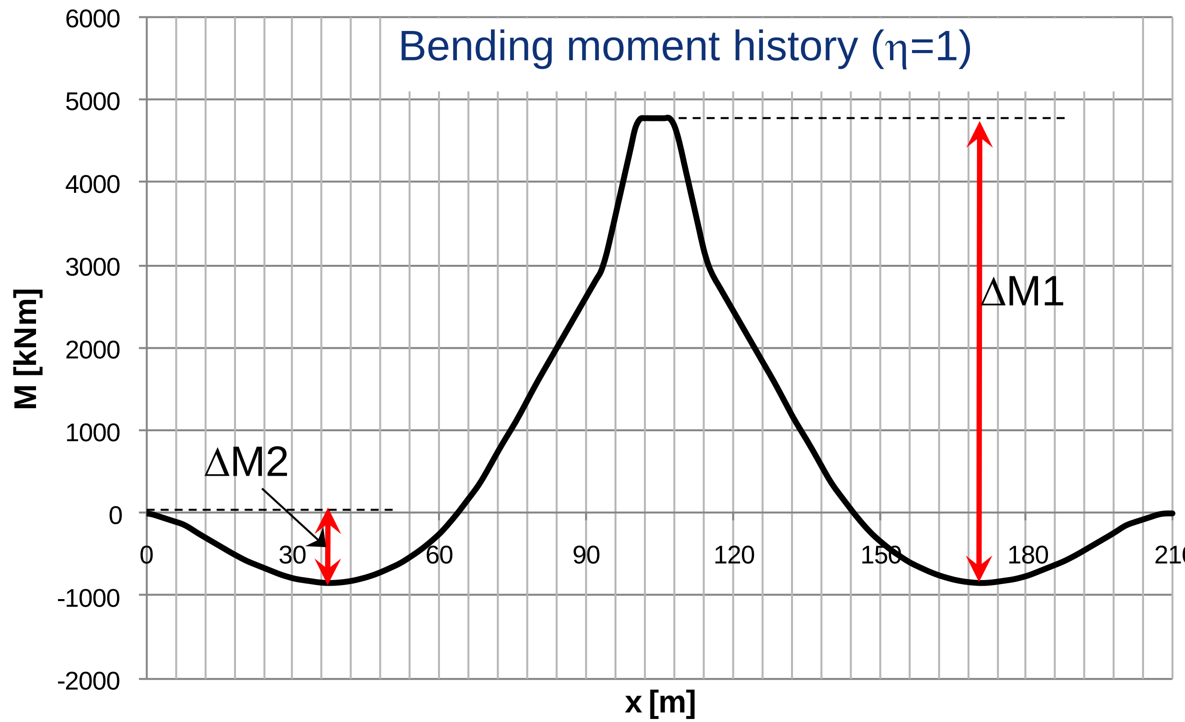


Bending moment history – midspan (uncracked)



Influence line for bending moment - midspan– [m]

	Case 1	Case 2
$\gamma_{Mf}\,\Delta M_1$ [kNm]	5086.4	6936.0
$\gamma_{Mf}\,\Delta M_2$ [kNm]	1387.2	4161.6
$\gamma_{Mf}\,\Delta M_3$ [kNm]	761.1	1037.8
$\gamma_{Mf}\,\Delta M_4$ [kNm]	207.6	622.7
D (upper flange)	0.000E+00	0.000E+00
D (lower flange)	0.000E+00	1.352E+00
D (straight rebar)	1.149E-11	1.893E-10
D (mesh)	7.884E-05	4.000E-04



THANK YOU FOR YOUR ATTENTION