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Translation

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Recommendations for the fatigue design of railway bridges in reinforced and prestressed-concrete

Recommandations pour le dimensionnement des ponts-rails en béton armé et précontraint à la fatigue
Empfehlung zur Ermüdungsbemessung von Eisenbahnbrücken aus Stahl- und Spannbeton

UNION INTERNATIONALE DES CHEMINS DE FER
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1st edition, July 1970	First issue published under the title: "Recommendations for determining permissible stresses in reinforced concrete railway bridges" and 1 Amendment
2nd edition, July 1984	with the current title
3rd edition, February 2005	Adaptation to the Eurocodes Retyped in FrameMaker New layout

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

Contents

Summary

The aim of this Leaflet is to provide recommendations for fatigue verification of the elements of railway bridges in common cases. It is valid for high-cycle fatigue and applies to new bridges but may also be used for verification of existing bridges, providing the actual state of those bridges is taken into account (steel corrosion, etc.).

The limit-state design concept is used for obtaining adequate durability during the life of the structure, which also guarantees adequate safety. The leaflet defines the actions and the combination of actions applied on the structure. It also determines the internal forces and stresses based on these actions.

Methods and, in some possible cases, simplified methods allow the fatigue verification of the reinforcement steel and concrete for reinforced and prestressed structures. For those methods, the values of the applied factors and definitions are given in tables or formulas, and where necessary, references are specified. Finally, recommendations for reinforcement detailing are listed.

1 - Introduction

The last issue of *UIC Leaflet 774-1* published in July 1984 included a chapter (Chapter 8) on fatigue verification.

The former ERRI Specialists' Committees D 183 and then D 216 participated in the drafting of new rules concerning the fatigue of concrete structures, notably as regards the determination of the λ correction factors.

The present leaflet is the result of that work and therefore conforms to the Eurocode standards, which regulate bridge calculations in Europe. This leaflet was drawn up to replace the 2nd issue of *UIC Leaflet 774-1*.

The aim of this Leaflet is to provide recommendations for fatigue verification of the elements of railway bridges in common cases.

This Leaflet is valid for high-cycle fatigue.

This Leaflet applies to new bridges but may also be used for verification of existing bridges, providing the actual state of those bridges is taken into account (e.g. reduction of steel section as a result of corrosion).

2 - Basic prescriptions - Notations - Units

2.1 - Capital letters

A = area

D = diameter

G = permanent action

N = number of cycles

P = prestressing

Q = variable action

R = strength

S = internal force

T = temperature

V = shear force

2.2 - Small letters

f = strength of a material

2.3 - Small Greek letters

γ = safety factor

ζ = reduction factor

η = increase factor

λ = correction factor

σ = normal stress

τ = shear stress

ξ = ratio of bond strength

2.4 - Capital Greek letters

Δ	=	difference
Θ	=	angle of inclination of concrete compression struts
$\varnothing$	=	diameter
Φ	=	dynamic factor

2.5 - General indices

c	=	concrete
d	=	design
fat	=	fatigue
k	=	characteristic
p	=	prestressing
s	=	steel

3 - General

3.1 - Principles

When subjected to a high number of load cycles, the reinforced and prestressed-concrete elements of bridges may sustain progressive damage to the reinforcement or the concrete resulting in structural failure, even though the stress levels applied are lower than the static strength. This phenomenon is called fatigue.

The resistance of railway bridges to fatigue shall be verified. The aim of performing a fatigue-safety verification is to demonstrate that the fatigue-effects of rail traffic loads will not impair the safety of the railway bridge during its intended service life.

The fatigue-safety verification shall be performed on the bearing capacity of members and separately for concrete and steel.

To a large extent, fatigue damage can be avoided by following the rules for detailing reinforcement as set down in point 6 - page 14. Clearly, observance of the latter does not dispense with fatigue verification.

In general, fatigue can be expected to prevail in:

- reinforced-concrete and prestressed-concrete structural elements for which the ratio of variable to permanent internal forces is high;

- mechanical or welded joints in prestressed-concrete reinforcements and in joints achieved by coupling prestressed reinforcements, particularly in areas subjected to varying stress ranges.

Fatigue verification is in principle not necessary for bridge elements for which the dynamic coefficient is not necessary, e.g., bridges with sufficient earth cover or an adequate layer of other damping material, piers and abutments which are not rigidly connected to the superstructure (except the slabs and walls of hollow abutments and ballast-retaining walls) and foundations.

3.2 - Actions and combinations of actions

3.2.1 - Fatigue actions

Fatigue actions are mainly caused by traffic loads.

The internal forces and stresses involved shall be calculated using UIC Load Model 71 - as defined in *UIC Leaflet 702* (class k_o), including the dynamic factor Φ in accordance with *UIC Leaflet 776-1* (see Bibliography - page 24).

For certain elements of the bridge, it may be necessary to take into account the internal forces due to changes in wind speed and/or direction.

3.2.2 - Other actions to be taken into account for stress calculations

These are:

- permanent actions (including settlement of supports, where applicable),
- prestressing,
- temperature.

3.2.3 - Combination(s) of actions

The following combinations of actions shall be taken into account:

- for the simplified verifications (see points 4.3 - page 10 and 5.2.1 - page 12):

$$S_{d,fat} = S\left(\sum_{j \geq 1} G_{kj} + P_k + 0{,}8Q_{k1} + 0{,}6T_k\right) \quad \textbf{(1)}$$

- for the general verifications (see points 4.4 - page 10 and 5.2.2 - page 13):

$$S_{d,fat} = S\left(\sum_{j \geq 1} G_{kj} + P_k + Q_{k1} + 0{,}8T_k\right) \quad \textbf{(2)}$$

where: G_{kj}, P_k, Q_{k1} and T_k are the characteristic values (respectively) of the permanent actions (including any settlement of supports), of the prestressing, of the variable fatigue actions and of the temperature.

NB : in the absence of a more detailed method of verification for coupled joints, the smallest characteristic value of the prestressing force acting in the couplers shall be reduced by applying a coefficient of 0,85.

3.3 - Determining the internal forces and stresses

- Internal forces and stresses shall be determined based on the assumption of cracked cross sections and ignoring the tensile strength of concrete but satisfying compatibility of strains, plane sections remaining plane.

- Internal forces and stresses may be determined using linear elastic models throughout the structural elements considered. In cracked zones, a reduced stiffness can be taken into account.

- The effect of the different bond behaviour of prestressing and reinforcing steel shall be taken into account by increasing the stress in the reinforcing steel by calculating with the following factor, assuming a perfect bond, according to *prEN 1992-1*, January 2001 (see Bibliography - page 24):

$$\eta = \frac{A_s + A_p}{A_s + A_p\sqrt{\xi \varnothing_s / \varnothing_p}} \qquad \textbf{(3)}$$

where:

A_s : section of reinforcing steel

A_p : section of prestressing steel

$\varnothing_s$: largest diameter of rebar

$\varnothing_p$: equivalent diameter of prestressing steel

$\varnothing_p = 1,6\sqrt{A_p}$ for multi-strand cables

$\varnothing_p = 1,75\ \varnothing_{wire}$ for single 7-wire strands

$\varnothing_p = 1,20\ \varnothing_{wire}$ for single 3-wire strands

ξ : ratio of bond strength between bonded-prestressing and bonded-reinforcing steel (Table 1 below)

Table 1 : Ratio of bond strength between bonded-prestressing and bonded-reinforcing steel

Prestressing steel	ξ	
	Pre-tensioned	**Post-tensioned**
Smooth bars and wires	-	0,3
Strands	0,6	0,5
Indented wires	0,7	0,6
Ribbed bars	0,8	0,7

- When action effects due to shear are calculated using the variable strut inclination method, the inclination of the compression struts Θ_{fat} may be taken according to equation (4):

$$\tan\Theta_{fat} = \sqrt{\tan\Theta} \leq 1,0 \qquad \textbf{(4)}$$

where:

Θ is the angle of concrete compression struts to the beam axis assumed in the design for shear at the ultimate limit-state.

- For prestressed-concrete structures with *bonded* tendons, stresses can only be calculated if the true prestressing force is known. If the moment due to fatigue action is smaller than the decompression moment of the cross section, the stress range remains small. In the case of partially prestressed elements with bonded tendons, the stresses in the tendons increase significantly when the decompression moment is exceeded.

- Where prestressing is by means of unbonded tendons, the stresses will have to be calculated iteratively.

4 - Fatigue verification of the reinforcement steel in reinforced concrete and prestressed-concrete

4.1 - Principles

The fatigue-safety of the reinforcement steel in reinforced concrete and prestressed-concrete shall be deemed proven if the following condition is satisfied:

$$S_{d,fat} = \Delta\sigma_s(Q_{fat}) \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{sfat}} = R_{d,fat} \qquad \textbf{(5)}$$

where:

$S_{d,fat}$:	fatigue-effect design value
$\Delta\sigma_s(Q_{fat})$:	stress range in steel under the effect of fatigue actions
$\Delta\sigma_{Rsk}(N^*)$:	stress range under N* cycles derived from the S-N curves
$R_{d,fat}$:	fatigue-strength design value
γ_{sfat} :	1,15 (partial safety factor for steel)

Fatigue-safety shall in principle be verified using the simplified method given in point 4.3 - page 10. A general verification according to point 4.4 - page 10 may be performed if fatigue-safety is not proven by the simplified verification.

The following provisions for fatigue verification apply to all kinds of reinforcement.

4.2 - Fatigue-strength

Common types of reinforcement and prestressing are divided into fatigue categories according to their fatigue-strength $\Delta\sigma_{Rsk}$, expressed in MPa, for a number N* of load cycles (Table 2).

Table 2 : Fatigue categories for reinforcing and prestressing steel

Type of reinforcement for reinforced concrete	N*	Stress exponent		$\Delta\sigma_{Rsk}$ at N* cycles
		k_1	k_2	
Straight and bent bars a)	10^6	5	9	162,5
Welded bars including flash-butt welds b)	10^7	3	5	58,5
Splicing devices b)	10^7	3	5	35

a) Values of $\Delta\sigma_{Rsk}$ are those of the appropriate straight bar. Values for a bar of diameter $\varnothing$ with a bent diameter $D < 25\varnothing$ should be obtained by multiplying the straight bar values by a reduction factor $\zeta = 0,35 + 0,026 \cdot D/\varnothing$. For stirrups, ζ should be taken as 0,9.

b) Values to be fixed by the responsible railway authority.

S-N curve of prestressing steel used for	N*	Stress exponent		$\Delta\sigma_{Rsk}$ at N* cycles
		k_1	k_2	
Pre-tensioning	10^6	5	9	185
Post-tensioning:				
- Single strands in plastic ducts	10^6	5	9	185
- Straight tendons or curved tendons in plastic ducts	10^6	5	10	150
- Curved tendons in steel ducts	10^6	3	7	120
- Splicing devices a)	10^6	3	5	80

a) Values to be fixed by the responsible railway authority.

- The S-N curves according to Table 2 generally follow the equation:

$$(\Delta\sigma_{Rsk})^m = N$$

where $m = k_1$ for $N < N^*$ and $m = k_2$ for $N > N^*$.

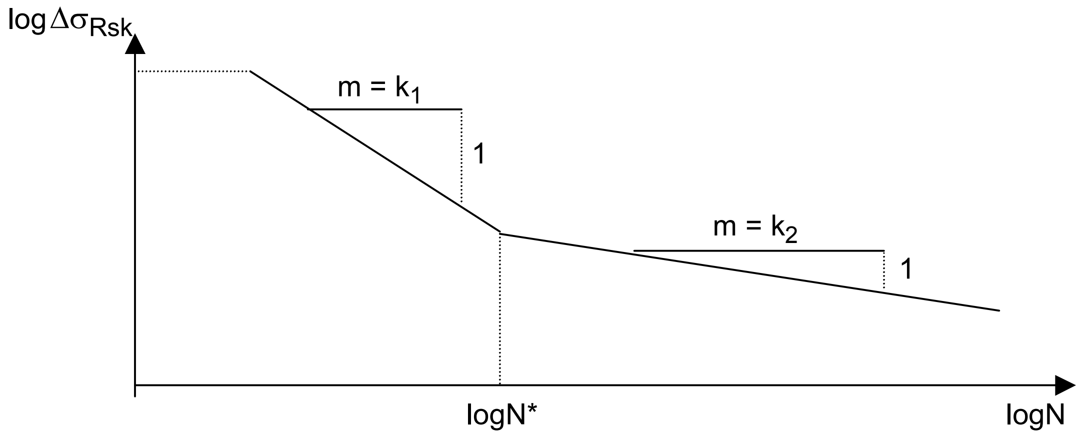

Fig. 1 - Shape of the characteristic fatigue-strength curve (S-N curves) for steel

- Special attention should be given to fatigue-effects in a corrosive environment such as that identified by Exposure Classes 4 and 5 in *prEN 1992-1, Table 4.1*.

- For concrete members where the corrosion process has started, the stress range $\Delta\sigma_{Rsk}$ can be determined by reducing the stress exponent k_2 for straight and bent bars to $k_2 = 5$.

4.3 - Simplified verification

Simplified verification consists of proving that the stress ranges in the element studied remain short of the fatigue limit for the entire service life of the structure:

$$\Delta\sigma_s(Q_{fat}) \leq \sqrt[m]{\frac{N^*}{N_D}} \cdot \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{sfat}} \qquad \textbf{(6)}$$

in which expression $m = k_1$ and N^* are given in Table 2 - page 9 and N_D is the number of cycles for which the fatigue limit is defined. For all types of reinforcement $N_D = 5 \cdot 10^6$ load cycles. The stress range $\Delta\sigma_s(Q_{fat})$ shall be determined under the **combination of actions (1)** given in point 3.2.3 - page 6.

It should be noted that this simplified formula is based on theories that deviate slightly from those defined in Eurocode, though they can be considered acceptable in view of the degree of accuracy of the verifications.

4.4 - General verification

If fatigue-safety has not been proven using the simplified method, the general method below may be used to demonstrate that the equivalent stress range does not exceed the design strength:

$$\Delta\sigma_{s,equ} \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_f \cdot \gamma_{sd} \cdot \gamma_{sfat}} \qquad \textbf{(7)}$$

The equivalent stress range $\Delta\sigma_{s,equ}$ is the product of the correction factor λ_s and the stress range generated by the fatigue load Q_{fat}:

$$\Delta\sigma_{s,equ} = \lambda_s \cdot \Delta\sigma_s(Q_{fat}) \qquad \textbf{(8)}$$

The stress range $\Delta\sigma_s(Q_{fat})$ is the stress range under the action of UIC Load Model 71 (including the dynamic increment) in the least favourable position under the influence of the **combination of actions (2)** in point 3.2.3 - page 6.

The correction factor λ_s adjusts the fatigue-effect of UIC Load Model 71 to that of the actual fatigue load. This correction factor depends on the statics model of the structure, the span length of the element under consideration, the annual traffic volume, the service life and the number of tracks; it is given by the following formula:

$$\lambda_s = \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \qquad \textbf{(9)}$$

where:

$\lambda_{s,1}$: correction factor for the span length of the structural element and for the traffic type

$\lambda_{s,2}$: correction factor for the annual traffic volume

$\lambda_{s,3}$: correction factor for the expected service life

$\lambda_{s,4}$: correction factor for the number of tracks.

The values of the λ_s factors are given in *ENV 1992-2, Chapter 7 Appendix F106* (see Bibliography - page 24).

The partial safety factor γ_f on the loads and γ_{sd} on the stresses is 1 according to *ENV 1992-2*. In the case of complex static systems where the determination of internal forces and stresses is more uncertain, a partial safety factor γ_{sd} of 1,1 may be used.

NB : γ_f = 1 because the UIC 71 train is the reference point for the current Eurocodes. Attention is drawn to the development of railway loads and the preparation of a new load model (UIC 2000) which is more stringent than the current model.

5 - Fatigue verification of concrete

5.1 - Principles

For concrete in compression, the fatigue-strength is a function not only of the stress range but also of the level of stress.

The verification shall be performed for the fibres of concrete in compression and for the compression struts of the elements subjected to shear force.

In the latter case, the fatigue-strength f_{cd} of concrete shall be reduced by applying the factor ν from the equation *(6.8) supplied by prEN 1992-1*.

5.2 - Verification of concrete in compression

5.2.1 - Simplified verification

- The fatigue-strength of concrete in compression shall be assumed sufficient if:

$$\frac{\sigma_{c,\,max}}{f_{cd}} \leq 0{,}5 + 0{,}45 \cdot \frac{\sigma_{c,\,min}}{f_{cd}} \leq 0{,}9 \qquad \textbf{(10)}$$

  where:

  $\sigma_{c,\,max}$: maximum compressive stress on a fibre under the combination of actions (1) from point 3.2.3 - page 6

  $\sigma_{c,\,min}$: minimum compressive stress on the same fibre, under the same combination of actions (if $\sigma_{c,\,min}$ is tensile stress, a check shall be performed to verify that $\sigma_{c,\,max}/f_{cd} \leq 0,5$)

  $f_{cd} = f_{ck}/\gamma_c$, where $\gamma_c = 1{,}5$

  and f_{ck} : characteristic compressive strength at 28 days.

- In practice, it may be sufficient to verify the following condition:

$$\sigma_{c,\,max} \leq 0,5\ f_{cd} \qquad \textbf{(11)}$$

- The increase in compressive strength with the age of the concrete during the time t_0 (before the cyclic loading) may be taken into account by applying *point 4.3.7.4 of ENV 1992-2*.

5.2.2 - General verification

In cases where the above conditions are not met, a more detailed verification may be necessary.

This should be done in accordance with the set of rules in force.

5.3 - Verification of concrete in shear

In members without shear reinforcement, a fatigue verification for concrete under shear need not be performed if either of the conditions below is satisfied. Otherwise, a more detailed fatigue verification may be necessary:

$$\text{for } \frac{\tau_{min}}{\tau_{max}} \geq 0 : \qquad \left|\frac{\tau_{max}}{\tau_{Rd}}\right| \leq 0{,}5 + 0{,}45 \cdot \left|\frac{\tau_{min}}{\tau_{Rd}}\right| \leq 0{,}9 \qquad \textbf{(12)}$$

$$\text{for } \frac{\tau_{min}}{\tau_{max}} < 0 : \qquad \left|\frac{\tau_{max}}{\tau_{Rd}}\right| \leq 0{,}5 - \left|\frac{\tau_{min}}{\tau_{Rd}}\right| \qquad \textbf{(13)}$$

where:

τ_{max} = maximum nominal shear stress under a combination of actions (1) from point 3.2.3 - page 6

τ_{min} = minimum nominal shear stress in the same section and under the same combination of actions

τ_{Rd} = $\frac{V_{Rd,ct}}{b_w \cdot d}$ with the design shear resistance ($V_{Rd,ct}$ according to *prEN 1992-1, equation (6.2)*)

where:

bw : width of the investigated cross section

d : static height of the investigated cross section

In the case of punching shear, the maximum and minimum shear stresses for the calculation shall satisfy equations (12) and (13).

6 - Fatigue resistant detailing

The following rules for detailing of reinforcement are to be observed:

- Where possible, connections between reinforcements, anchorages for and couplers between prestressing elements should be located in areas where the stress ranges are low.

- Welding of prestressing steel and ducts is not allowed.

- Welding of the reinforcement (including the use of welded wire mesh) should be avoided where possible, as welding reduces fatigue-strength significantly.

- If load-bearing welds in the reinforcement are unavoidable, only butt welds may be used.

- Tack welding to reinforcing steel, prestressing steel and ducts is prohibited.

- Where reinforcing rebars are joined mechanically or by welding, fatigue-strength of the connections is to be verified by testing beforehand.

- Shear reinforcement must enclose the main reinforcement, and care shall be taken that the concrete cover over the shear stirrups meets at least the minimum specified.

- Radii of curvature of reinforcement may under no circumstances be smaller than the minimum values specified in standards.

- Where prestressing is carried out using unbonded tendons, the anchorage and any couplers shall be the determining factors.

The process of placing the reinforcement must be considered carefully and the reinforcement must be so placed as to facilitate the pouring and working of the concrete.

7 - Correction factors

7.1 - Reinforcing and prestressing steel

The damage-equivalent stress range for reinforcing and prestressing steel shall be calculated according to equation (14):

$$\Delta\sigma_{s,equ} = \lambda_s \cdot \Delta\sigma_{s,71} \quad \textbf{(14)}$$

where:

$\Delta\sigma_{s,71}$	steel stress range due to load model 71 (being placed in the most unfavourable position) under the infrequent combination of actions, which includes the dynamic factor according to *ENV 1991-3* (see Bibliography - page 24).
λ_s	correction factor to calculate the damage-equivalent stress range from the stress range caused by $\Delta\sigma_{s,71}$. The values given in Table 3 - page 17 are based on $\psi'_1 = 1$.

The correction factor λ_s takes account of the span, annual traffic volume, service life and multiple tracks. It may be calculated from the following formula:

$$\lambda_s = \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \quad \textbf{(15)}$$

where:

$\lambda s,1$	factor to take account of the span of the member and the traffic mix
$\lambda_{s,2}$	factor to take account of the annual traffic volume
$\lambda_{s,3}$	factor to take account of the service life
$\lambda_{s,4}$	factor for multiple tracks.

The factor $\lambda_{s,1}$ is a function of the span of the member and the traffic mix. The values of $\lambda_{s,1}$ for standard traffic mix and heavy traffic mix as defined in *ENV 1991-3, Tables F.1 and F.2* may be taken from Table 3.

For other combinations of train types the factor $\lambda_{s,1}$ may be calculated from methods given in relevant documents (see for instance, explanatory document entitled: "Fatigue Design for Concrete Railway Bridges in Eurocode 2, Part 2, Loading, Resistance, Verification Formats").

The $\lambda_{s,2}$ value denotes the influence of annual traffic volume and may be calculated from equation (16):

$$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 \cdot 10^6}} \tag{16}$$

where:

Vol	volume of traffic (tonnes per year and track)
k_2	slope of the S-N curve.

The $\lambda_{s,3}$ value denotes the influence of the service life and may be calculated from equation (17):

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}} \tag{17}$$

where:

N_{years}	design working life of the bridge (years)
k_2	slope of the S-N curve

The $\lambda_{s,4}$ value denotes the effect of loading from more than one track. The effect of loading from two tracks may be calculated from equation (18):

$$\lambda_{s,4} = \sqrt[k_2]{n + (1-n) \cdot s_1^{k_2} + (1-n) s_2^{k_2}}$$
$$s_1 = \frac{\Delta\sigma_1}{\Delta\sigma_{1-2}} \quad s_2 = \frac{\Delta\sigma_2}{\Delta\sigma_{1-2}} \quad n = \frac{N_c}{N_T} \tag{18}$$

where:

n	proportion of traffic crossing simultaneously the bridge
N_c	number of trains crossing simultaneously the bridge
N_T	total number of trains running on one track
$\Delta\sigma_1$, $\Delta\sigma_2$	stress range due to load model 71 on one track
$\Delta\sigma_{1-2}$	stress range due to load model 71 on two tracks
k_2	slope of the S-N curve.

If only compression stresses occur under traffic loads on a track, set the corresponding value $s_j = 0$.

Table 3 : $\lambda_{s,1}$ values for single and continuous beams

a) Simply-supported beams

Type	S-N curve			Span [m]	Traffic mix	
	k_1	k_2	N*		Standard	Heavy
[1]	5	9	10^6	≤ 2	0,90	0,95
				≥ 20	0,65	0,70
[2]	3	7	10^6	≤ 2	1,00	1,05
				≥ 20	0,70	0,70
[3]	3	5	10^6	≤ 2	1,25	1,35
				≥ 20	0,75	0,75
[4]	3	5	10^7	≤ 2	0,80	0,85
				≥ 20	0,40	0,40

b) Continuous beams (intermediate span, central section)

Type	S-N curve			Span [m]	Traffic mix	
	k_1	k_2	N*		Standard	Heavy
[1]	5	9	10^6	≤ 2	0,95	1,05
				≥ 20	0,50	0,55
[2]	3	7	10^6	≤ 2	1,00	1,15
				≥ 20	0,55	0,55
[3]	3	5	10^6	≤ 2	1,25	1,40
				≥ 20	0,55	0,55
[4]	3	5	10^7	≤ 2	0,75	0,90
				≥ 20	0,35	0,30

Table 3 : $\lambda_{s,1}$ values for single and continuous beams

c) Continuous beams (end span section)

Type	**S-N curve**			**Span [m]**	**Traffic mix**	
	k_1	**k_2**	**N***		**Standard**	**Heavy**
[1]	5	9	10^6	≤ 2	0,90	1,00
				≥ 20	0,65	0,65
[2]	3	7	10^6	≤ 2	1,05	1,15
				≥ 20	0,65	0,65
[3]	3	5	10^6	≤ 2	1,30	1,45
				≥ 20	0,65	0,70
[4]	3	5	10^7	≤ 2	0,80	0,90
				≥ 20	0,35	0,35

d) Continuous beams (intermediate support section)

Type	**S-N curve**			**Span [m]**	**Traffic mix**	
	k_1	**k_2**	**N***		**Standard**	**Heavy**
[1]	5	9	10^6	≤ 2	0,85	0,85
				≥ 20	0,70	0,75
[2]	3	7	10^6	≤ 2	0,90	0,95
				≥ 20	0,70	0,75
[3]	3	5	10^6	≤ 2	1,10	1,10
				≥ 20	0,75	0,80
[4]	3	5	10^7	≤ 2	0,70	0,70
				≥ 20	0,35	0,40

Definition of Types in Table 3

[1] Reinforcing steel, pre-tensioning (all), post-tensioning (strands in plastics and straight tendons in steel ducts).

[2] Post-tensioning (curved tendons in steel ducts).

[3] Couplers (prestressing steel).

[4] Splicing devices (reinforcing steel), welded bars including tack welding and butt joints (see *ENV 1992-1, point 5.2*).

Values of $\lambda_{s,1}$ for spans L between 2 m and 20 m may be obtained from the following equation:

$$\lambda_{s,1}(L) = \lambda_{s,1}(2) + [\lambda_{s,1}(20) - \lambda_{s,1}(2)](\log L - 0{,}3)$$

For the definition of end span section, intermediate support section and intermediate span, central section, see figure 2:

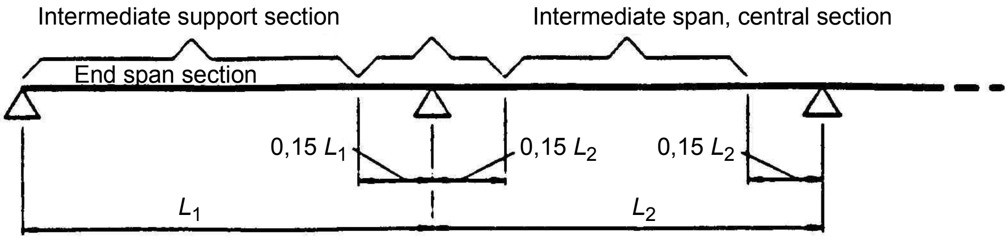

Fig. 2 - Areas for span sections

7.2 - Concrete subjected to compression

For concrete subjected to compression, adequate fatigue resistance may be assumed if the following expression is satisfied:

$$14\frac{1 - S_{cd,\,max,\,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \qquad \textbf{(19)}$$

where:

$$R_{equ} = \frac{S_{cd,\,min,\,equ}}{S_{cd,\,max,\,equ}}\ ;\ S_{cd,\,min,\,equ} = S_d\frac{\sigma_{cd,\,min,\,equ}}{f_{cd,\,fat}}\ ;\ S_{cd,\,max,\,equ} = S_d\frac{\sigma_{cd,\,min,\,equ}}{f_{cd,\,fat}}$$

$\sigma_{cd,\,max,\,equ}$ and $\sigma_{cd,\,min,\,equ}$ are the upper and lower stresses of the damage-equivalent stress range with a number of cycles N = 10^6.

The upper and lower stresses of the damage-equivalent stress range should be calculated according to equation (20):

$$\begin{aligned}\sigma_{cd,\,max,\,equ} &= \sigma_{c,\,perm} + \lambda_c(\sigma_{c,\,max,\,71} - \sigma_{c,\,perm})\\ \sigma_{cd,\,min,\,equ} &= \sigma_{c,\,perm} - \lambda_c(\sigma_{c,\,perm} - \sigma_{c,\,min,\,71})\end{aligned} \qquad \textbf{(20)}$$

where:

$\sigma_{c,\,perm}$:	Compressive concrete stress under the infrequent combination of actions without load model 71.
$\sigma_{c,\,max,\,71}$; $\sigma_{c,\,min,\,71}$:	Maximum or minimum compressive stress under the infrequent combination of actions, which includes the dynamic factor Φ_2 according to *ENV 1991-3*.
λ_c :	Correction factor to calculate the upper and lower stresses of the damage-equivalent stress range from the stresses caused by load model 71.

The values given in Table 4 - page 22 are based on $\psi'_1 = 1$.

The correction factor λ_c takes account of the permanent stress, the span, annual traffic volume, service life and multiple tracks. It may calculated from the following formula:

$$\lambda_c = \lambda_{c,0} \cdot \lambda_{c,1} \cdot \lambda_{c,2} \cdot \lambda_{c,3} \cdot \lambda_{c,4} \qquad \textbf{(21)}$$

where:

$\lambda_{c,0}$	factor to take account of the permanent stress
$\lambda_{c,1}$	factor to take account of the span of the member and the traffic mix
$\lambda_{c,2}$	factor to take account of the annual traffic volume
$\lambda_{c,3}$	factor to take account of the service life
$\lambda_{c,4}$	factor for multiple tracks.

The $\lambda_{c,0}$ value denotes the influence of the permanent stress and may be calculated from equation (22):

$$\lambda_{c,0} = 0{,}94 + 0{,}2\frac{\sigma_{c,perm}}{f_{cd,\,fat}} \geq 1{,}0 \qquad \textbf{(22)}$$

For the precompressed tensile zone in prestressed-concrete members, the value $\lambda_{c,0}$ may be taken equal to 1,0.

The factor $\lambda_{c,1}$ is a function of the span of the member and the traffic mix. The values of $\lambda_{c,1}$ for standard traffic mix and heavy traffic mix as defined in *ENV 1991-3, Tables F.1 and F.2* may be taken from Table 4.

The $\lambda_{c,2}$ value denotes the influence of the annual traffic volume and may be calculated from equation (23):

$$\lambda_{c,2} = 1 + \frac{1}{8}\log\left[\frac{Vol}{25 \cdot 10^6}\right] \qquad \textbf{(23)}$$

where:

Vol Volume of traffic (tonnes per year and track).

The $\lambda_{c,3}$ value denotes the influence of the service life and may be calculated from equation (24):

$$\lambda_{c,3} = 1 + \frac{1}{8}\log\left[\frac{N_{years}}{100}\right] \qquad \textbf{(24)}$$

where:

N_{years} Service life of the bridge (years).

The $\lambda_{c,4}$ value denotes the effect of loading from more than one track. The effect of loading from two tracks may be calculated from equation (25):

$$\lambda_{c,4} = 1 + \frac{1}{8}\log n \geq 0{,}54 \qquad \text{for } a \leq 0{,}8 \qquad \textbf{(25)}$$

$$\lambda_{c,4} = 1{,}0 \qquad \text{for } a > 0{,}8$$

$$a = \frac{\max\{\Delta\sigma_{c,1}, \Delta\sigma_{c,2}\}}{\Delta\sigma_{c,1\text{-}2}} \;;\; n = \frac{N_c}{N_T} \qquad \textbf{(26)}$$

where:

n	Proportion of traffic crossing the bridge
N_c	Number of trains crossing the bridge
N_T	Total number of trains running on one track
$\Delta\sigma_{c,1}, \Delta\sigma_{c,2}$:	Compressive stress range caused by load model 71 on one track
$\Delta\sigma_{c,1\text{-}2}$:	Compressive stress range caused by load model 71 on two tracks.

Table 4 : $\lambda_{s,1}$ values for simply-supported and continuous beams

a) simply-supported beams

Zone of cross section	**Span [m]**	**Traffic mix**	
		Standard	**Heavy**
Compressed zone	≤ 2	0,70	0,70
	≥ 20	0,75	0,75
Precompressed tensile zone	≤ 2	0,95	1,00
	≥ 20	0,90	0,90

b) Continuous beams (intermediate span, central section)

Zone of cross section	**Span [m]**	**Traffic mix**	
		Standard	**Heavy**
Compressed zone	≤ 2	0,75	0,90
	≥ 20	0,55	0,55
Precompressed tensile zone	≤ 2	1,05	1,15
	≥ 20	0,65	0,70

c) *Continuous beams (end-span section)*

Zone of cross section	**Span [m]**	**Traffic mix**	
		Standard	**Heavy**
Compressed zone	≤ 2	0,75	0,80
	≥ 20	0,70	0,70
Precompressed tensile zone	≤ 2	1,10	1,20
	≥ 20	0,70	0,70

Table 4 : $\lambda_{s,1}$ values for simply-supported and continuous beams

d) Continuous beams (intermediate support section)

Zone of cross section	**Span [m]**	**Traffic mix**	
		Standard	**Heavy**
Compressed zone	≤ 2	0,70	0,75
	≥ 20	0,85	0,85
Precompressed tensile zone	≤ 2	1,10	1,15
	≥ 20	0,80	0,85

Values of $\lambda_{c,1}$ for spans between 2 m and 20 m may be obtained from the following equation:

$$\lambda_{c,1}(L) = \lambda_{c,1}(2m) + [\lambda_{c,1}(20m) - \lambda_{c,1}(2m)](\log L - 0,3)$$

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