EU-Russia Regulatory Dialogue: Construction Sector Subgroup

Seminar ' Bridge Design with Eurocodes'

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DG Joint Research Centre
DG Enterprise and Industry

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Federal Highway Agency, Ministry of Transport

European Committee for Standardization
TC250 Structural Eurocodes

Railway Bridges

Basis of Design of railway bridges, some important points

The European High Speed Railway Network with

examples of Steel and Composite Railway Bridges

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Retired, ex Head of Bridges at SBB
(Swiss Federal Railways)

Actions on structures – Traffic loads on bridges

EN 1991-2 – CONTENTS (continued)

Actions on structures – Traffic loads on bridges

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

Basis of structural design – Application for bridges

Section A2.1 Field of application

Section A2.2 Combinations of actions

- A2.2.1 General
  - A2.2.2 …for road bridges
  - A2.2.3 …for footbridges
  - **A2.2.4 …for railway bridges**
  - A2.2.5

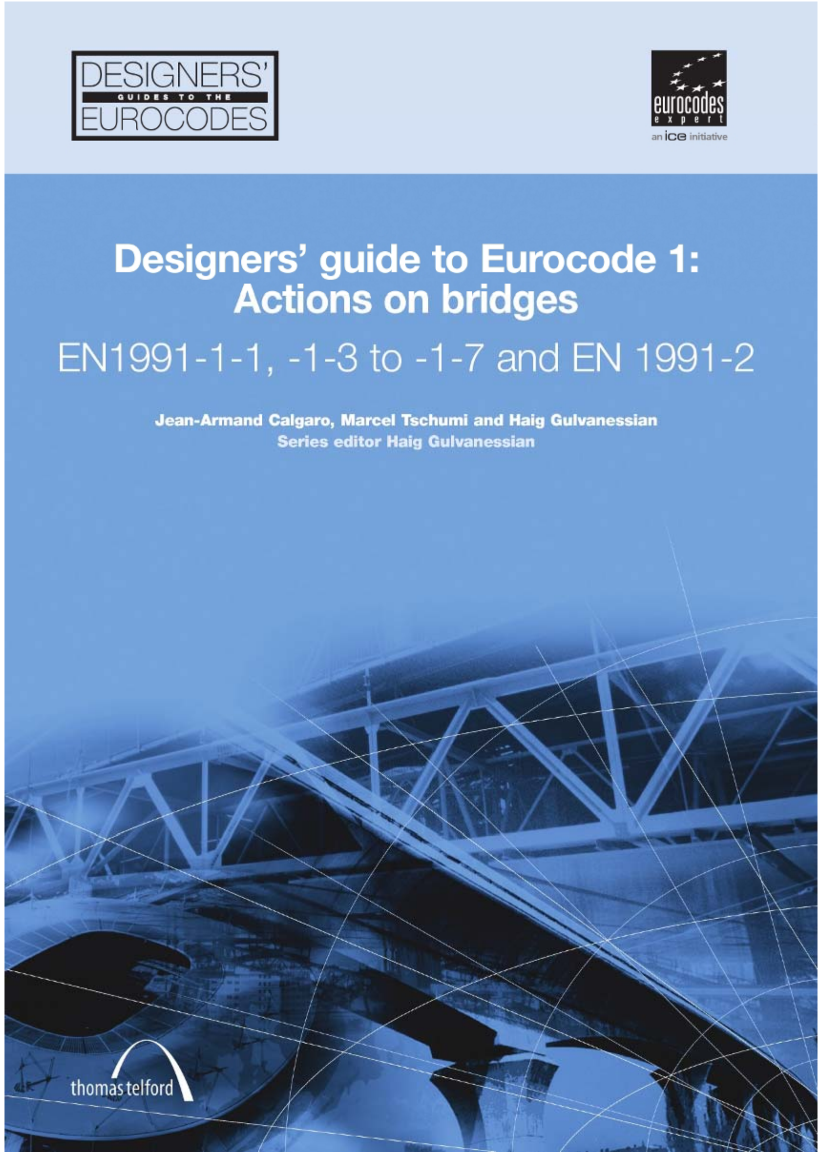
Section A2.3 Ultimate limit states

Section A2.4 Serviceability limit states

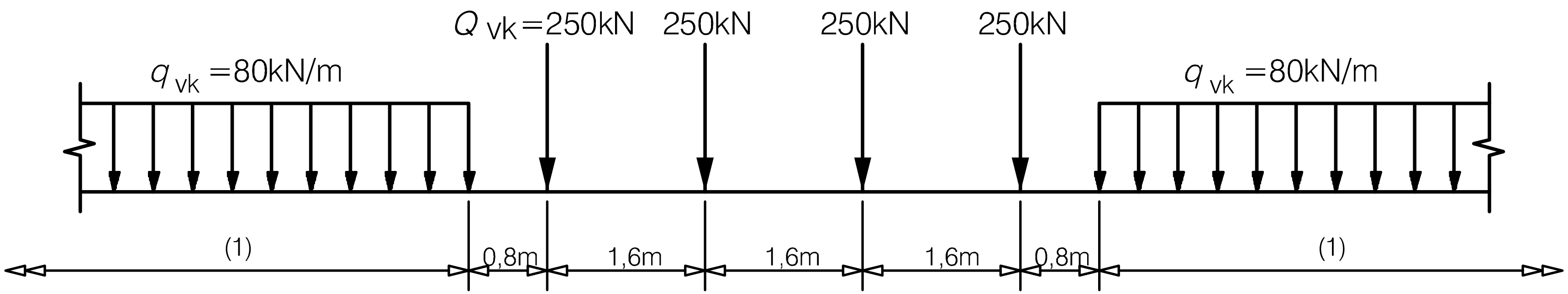
- A2.4.1 General
- A2.4.2 …serviceability criteria for road bridges
- A2.4.3 …serviceability criteria for footbridges
- **A2.4.4 serviceability criteria for railway bridges**

Designers' guides to Eurocodes, by Telford

European Commission



Load Model 71, also for HSL!



The characteristic values given in this figure of EN 1991-2 shall be multiplied by a factor *α* on lines carrying rail traffic which is heavier or lighter than normal rail traffic. When multiplied by the factor *α*, the loads are called "classified vertical loads". This factor *α* shall be one of the following: 0,75 - 0,83 - 0,91 - 1,00 - 1,10 - 1,21 - 1,33 – 1,46.

The value 1,33 is normally recommended on lines for freight traffic and international lines (UIC CODE 702, 2003). (for ULS)

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

centrifugal forces

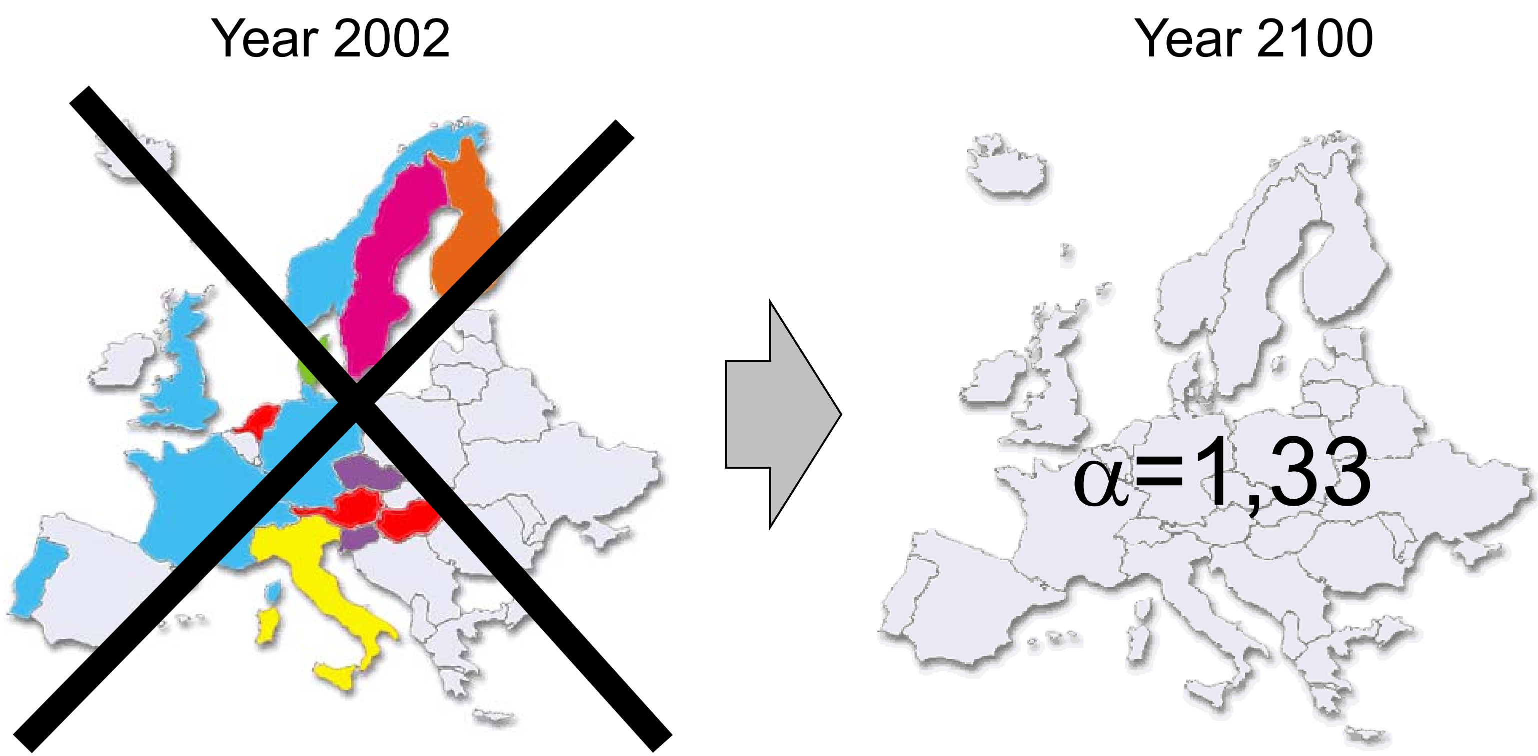
nosing force

traction and braking forces

load model SW/0 for continuous span bridges

Vision of future European Network

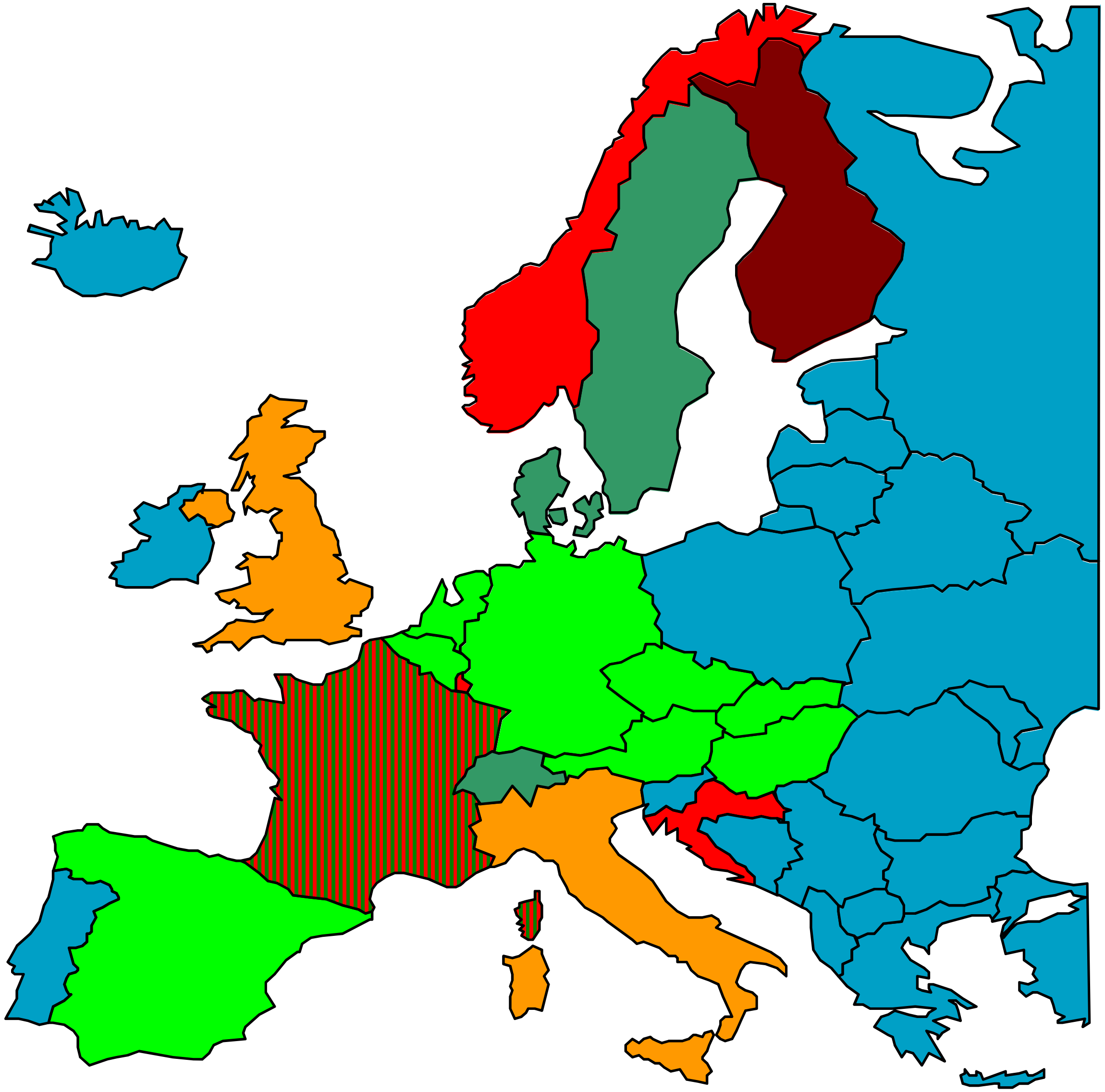
The freedom for the choice of the factor α could provoke a non homogeneous railway network in Europe! Therefore in UIC Leaflet 702 (2003) α = 1,33 is generally recommended for all new bridges constructed for the international freight network, unfortunately not compulsory!



Factor alpha, situation 2011

EN 1991-2
factor α

- α= 1,46
- α= 1,33
- α= 1,21
- α= 1,10
- α= 1,00
- α= 1,00/1,33
- α= n.n.



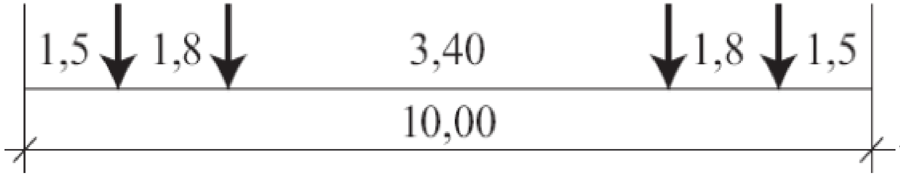
Ultimate Limit States (ULS):

For **new bridges** it should absolutely be adopted
$\alpha = 1{,}33$.

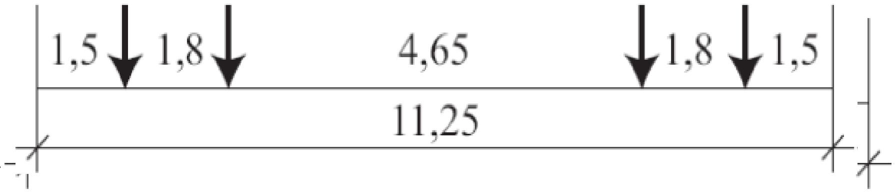
Due to UIC CODE 700		Mass per axle A	B	C	D	E
Mass per m = p		16t	18t	20t	22,5t	25t
1	5 t/m	A	B1			
2	6,4 t/m		B2	C2 (~1920)	D2 (~1970)	
3	7,2 t/m			C3	D3	
4	8 t/m			C4	D4	E4 (2003)
5	8,8 t/m					E5

UIC track classes

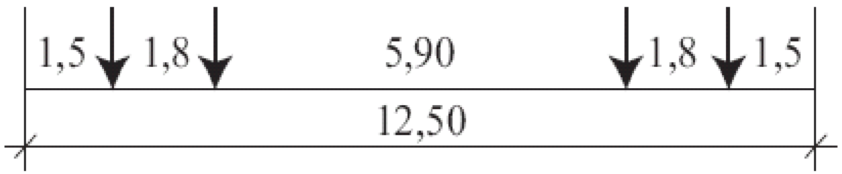
Indefinite number of wagons for a track line:



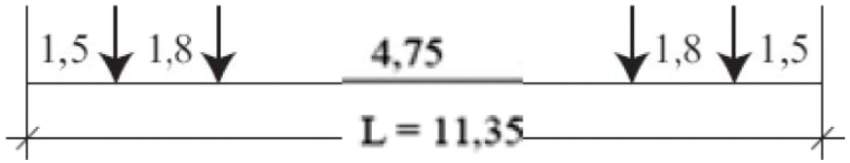
C4 Q= 20 t
q = 8 t/m



D4 Q= 22.5 t
q = 8 t/m

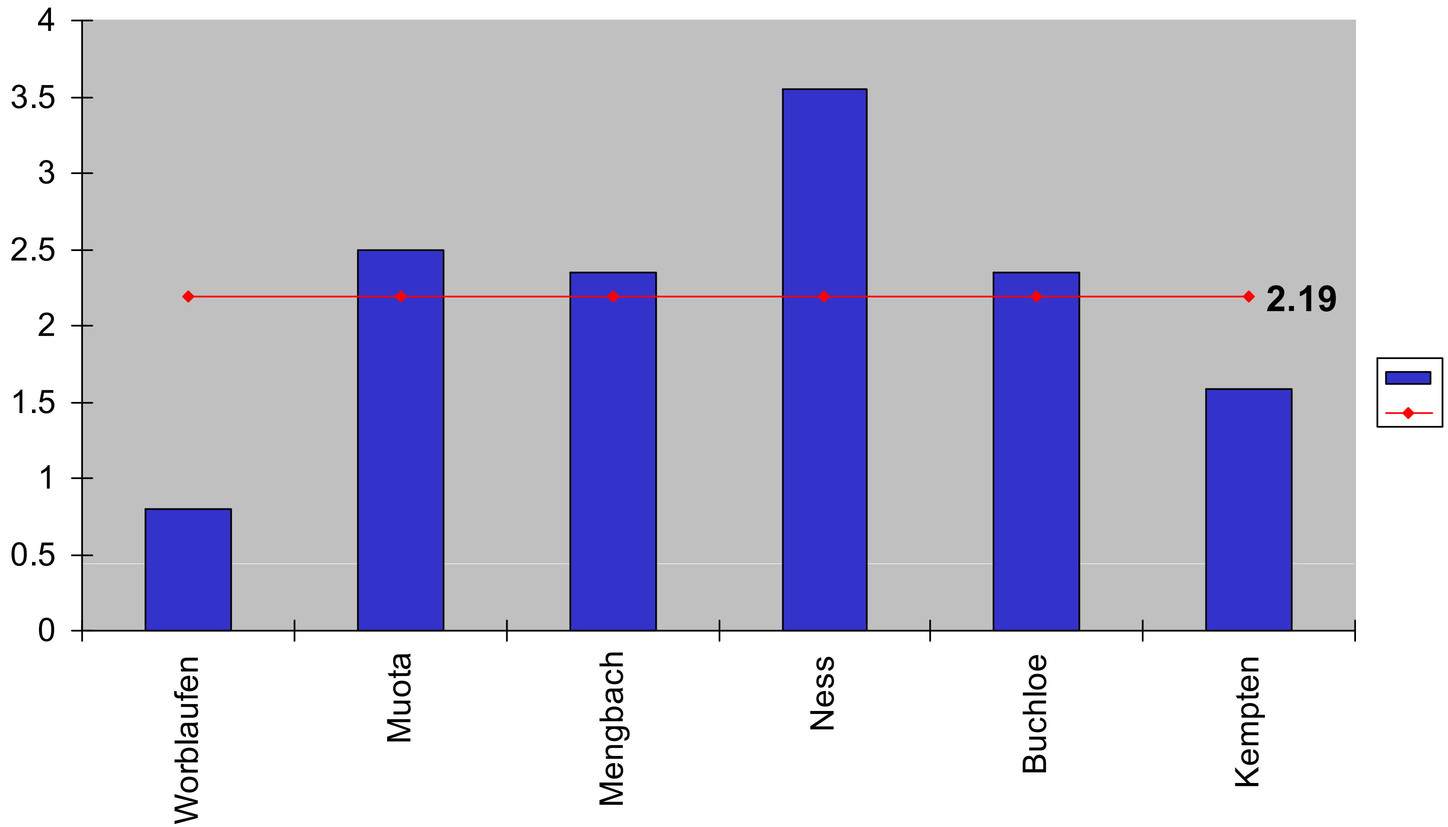


E4 Q= 25 t
q = 8 t/m



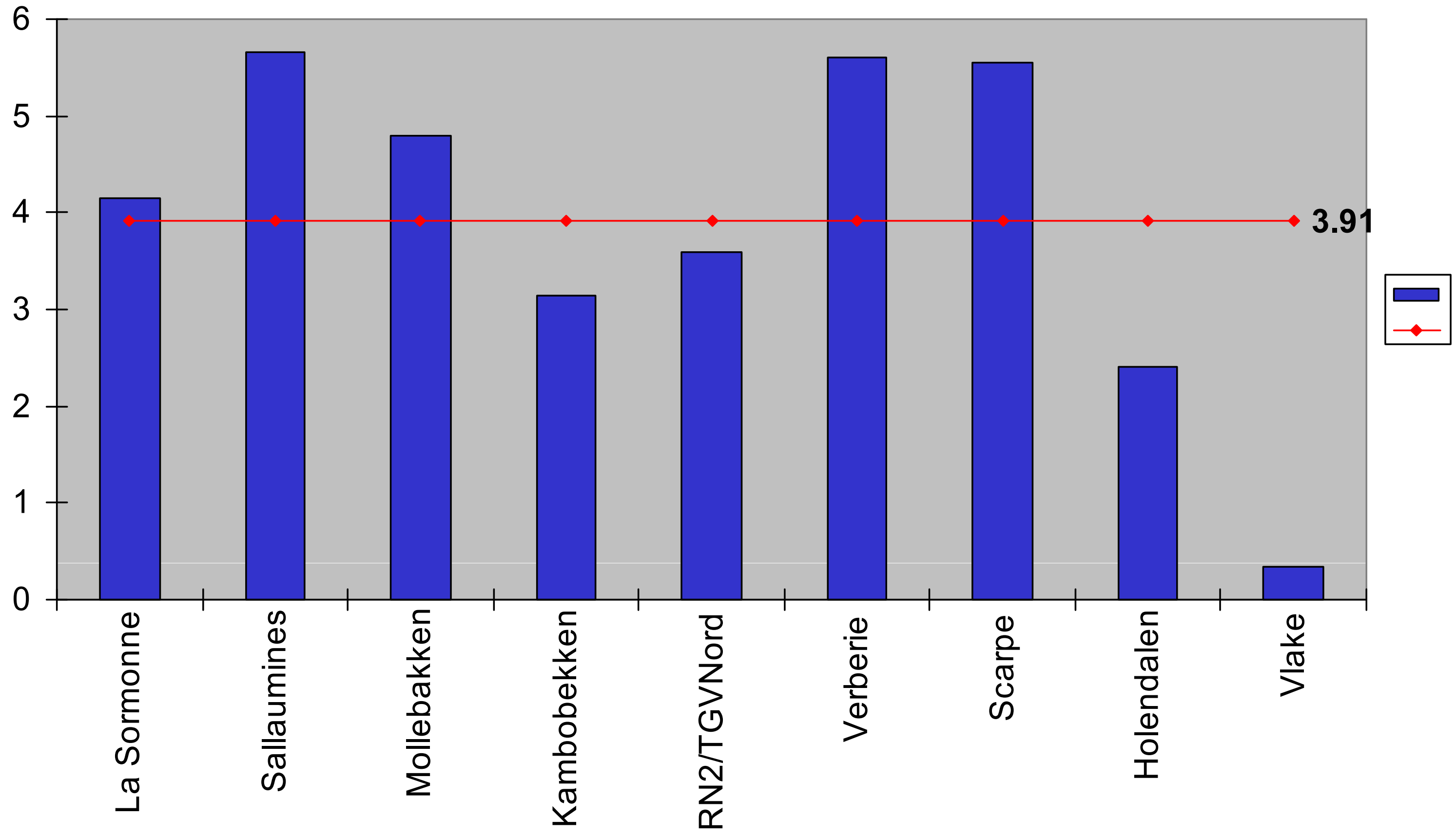
E5 Q= 25 t
q = 8,8 t/m

Increase of costs in % due to α = 1,33, related to those calculated with α = 1,0 / bridges built with traffic interference (ERRI D 192/RP 4, 1996):



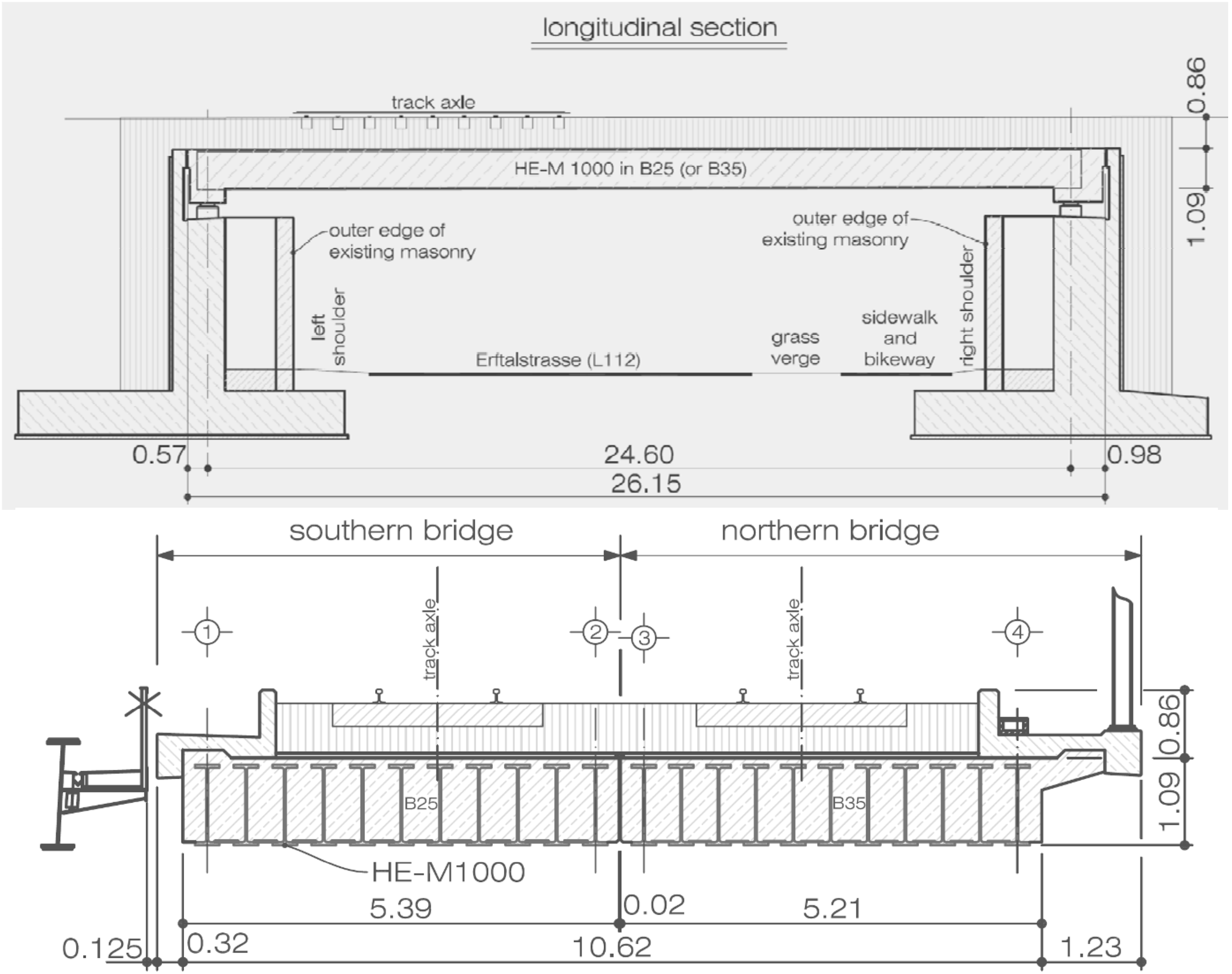
Heavier loads do not significantly influence the costs of bridges!

Increase of costs in % due to α = 1,33, related to those calculated with α = 1,0 / bridges built without traffic interference, (ERRI D 192/RP 4, 1996):



EÜ Erfttalstrasse, ABS 4/S 13, line Köln - Aachen, km 21,223, (D)

EÜ Erfttalstrasse, ABS 4/S 13, line Köln - Aachen, km 21,223, (D)



Heavier loads do not significantly influence the costs of bridges!

EX.DETAILS: DB - EÜ Erfttalstrasse, Köln - Aachen, km 21,223

The span of this simply supported bridge with embedded steel girders is *l* = 24,6 m. 22 steel girders HE 1000M were used. Due to a report of DB, the deflection of this bridge under the vertical load ΦLM71 is 19,1 mm, what correspond to the value *l* /1288. The required stiffness of this bridge was only determined by a dynamic study.

At my opinion this is too weak, I will explain that later, when I speak about permissible deflections, where for this case, to avoid excessive track maintenance, we should have l/2600.

Now how this bridge could have been stiffer, without more construction height than with the existing steel girders, same height to avoid costs for constructing a lower road below the bridge, taking into consideration the required clearance.

In the tables of ARCELOR, we find the following possible steel girders which practically fulfil this condition, namely the profiles HL 1100 R and HL 1000M x 642.

Result of my calculations: A 100% higher stiff bridge gives only 10% more investment costs. This is an interesting linear extrapolation of the results mentioned above (α = 1,33 => Δ investment costs = 2 to 4%)!

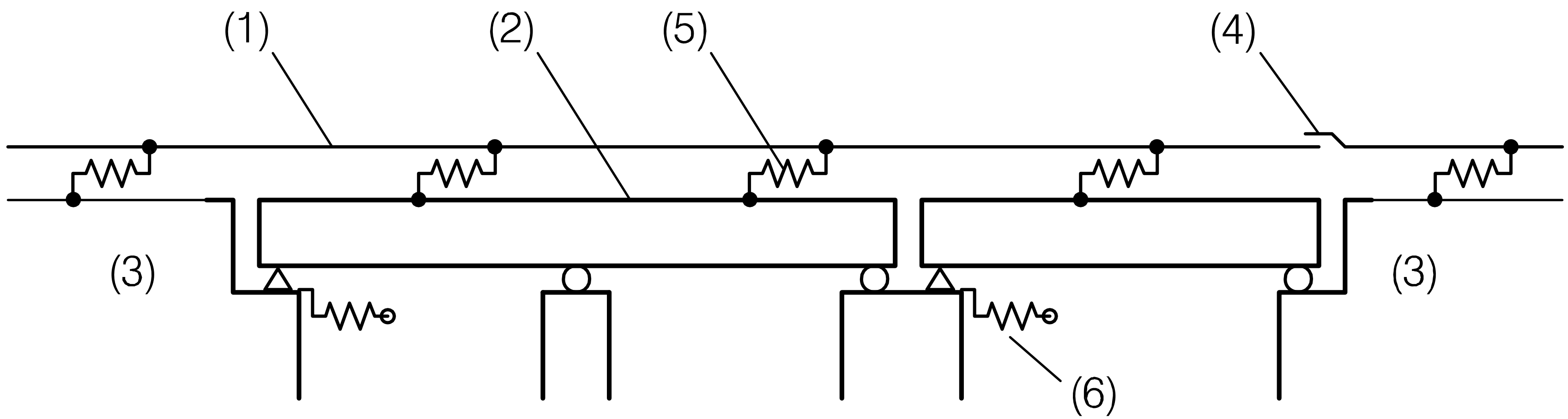
Serviceability Limit States (SLS)
Interaction track – bridge:

Theoretically this is a Seviceability Limit State (SLS) for the bridge and an Ultimate Limit State (ULS) for the rail. But as the given permissible rail stresses and deformations were obtained by deterministic design methods, calibrated on the existing practice, the calculations for interaction have to be done – in contradiction to EN1991-2, where there is a mistake - always with

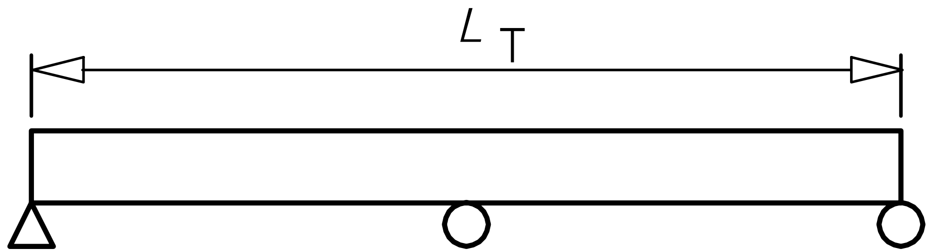
$$\alpha = 1{,}00!!$$

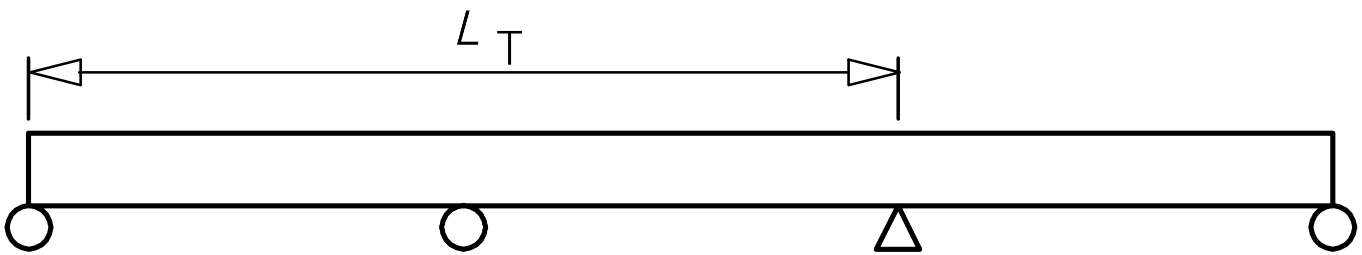
Relative displacements of the track and of the bridge, caused by the combination of the effects of thermal variations, train braking and traction forces, as well as deflection of the deck under vertical traffic loads (LM 71), lead to the track/bridge phenomenon that results in additional stresses to the bridge and the track.

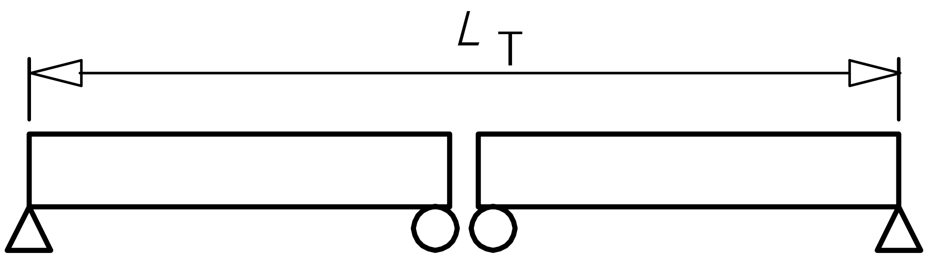
Take LM 71 with α = 1.00 !

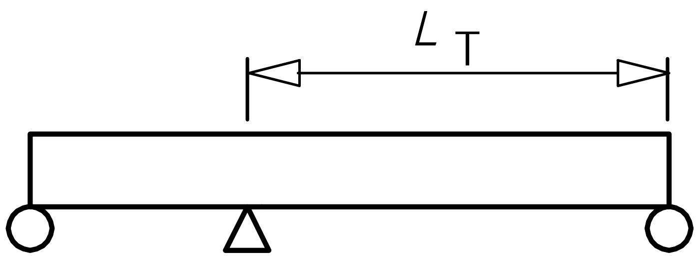


Examples of expansion lengths

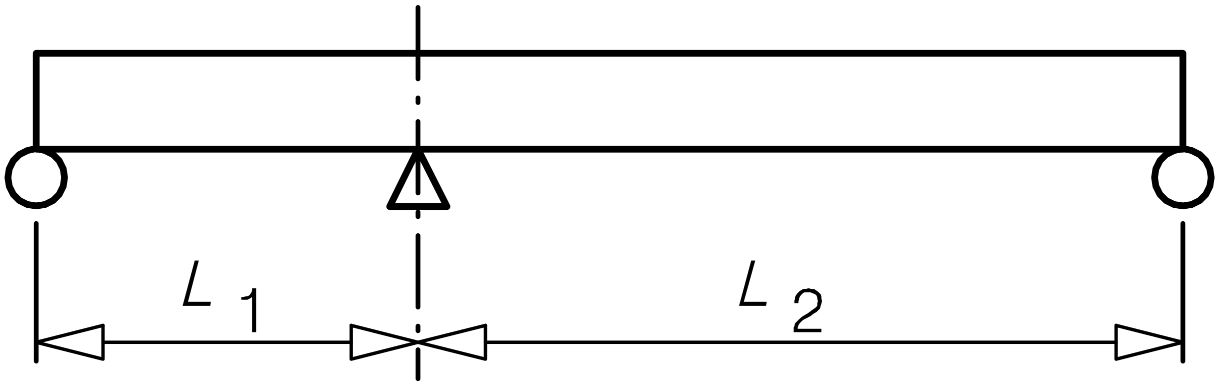








Avoid where ever possible expansion devices!



Remark:The decks corresponding to *L*1 or to *L*2 may have additional supports.

***L*1max. or *L*2 max. without expansion joints:**

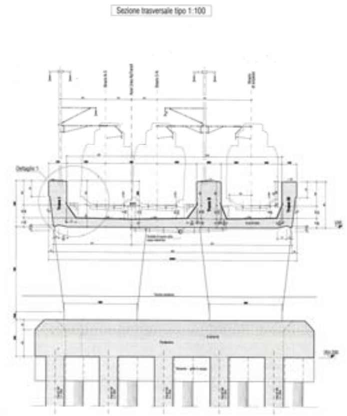
- **90 m (concrete, composite)**
- **60 m (steel),**

but:

***L*1 + *L*2 = 180 m/ 120 m with fixed bearing in the middle !!!!!!**

AlpTransit Gotthard, Bridge over the river Brenno near Biasca, CH

Practical example:
Remark:
Prestressed bridge, but the result would be the same for a composite bridge



How can we avoid expansion joints in the rails to get long welded rails (LWR) over a bridge more than 90 m long?

Fix point on an abutment:

L_T =37 + 42,5 + 29,5 m = 109 m > 90 m => LWR not poss.

With a fix point on a pier => LWR possible:

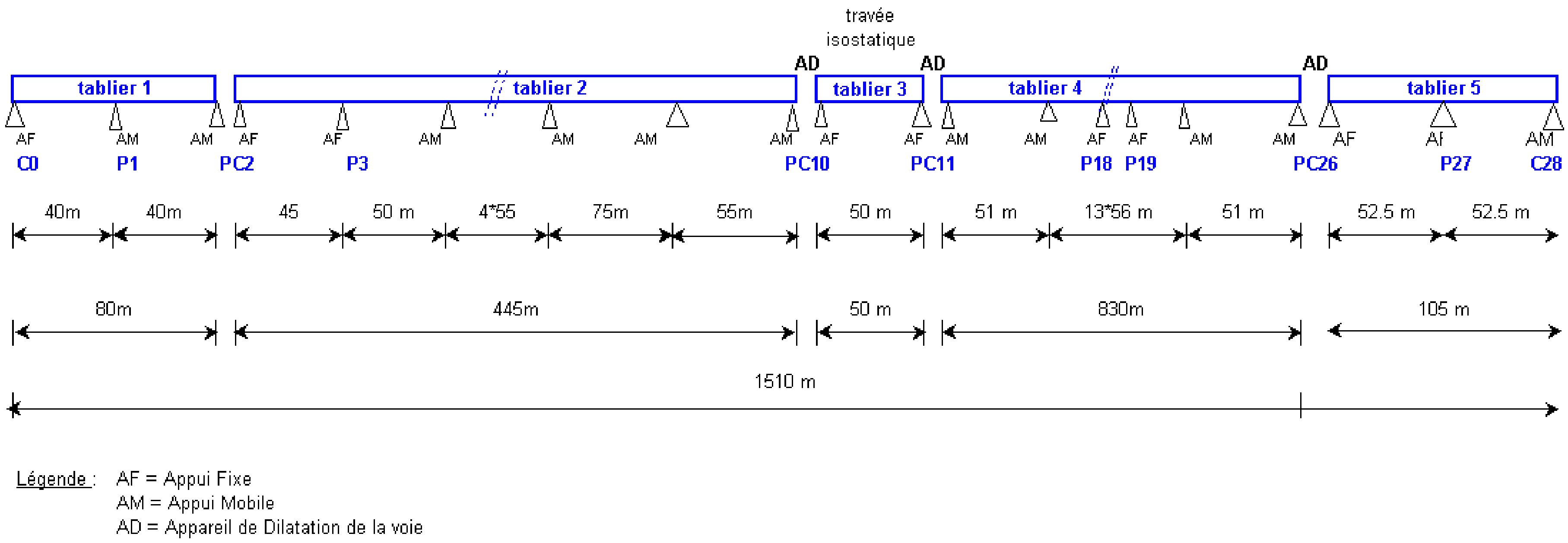
L_{T1}= 37 + 42,5 = 79,5 m < 90 m L_{T2}=29,5 m < 90 m

With fix points on two piers => LWR poss., chosen solution):

L_{Tmax} = 42,5/2 + 37 m = 79,5 m < 90 m

Viaduc de la Moselle, interaction track - bridge

Viaduc de la Moselle, interaction rail



Longitudinal system of a composite bridge with a length of 1510 m
Usual expansion devices SNCF for L_T < 450 m

FATIGUE: choices for α and λ

For new bridges, even if taking α = 1,33 for ULS design, fatigue assessments are done with the load model LM 71 and α = 1,00.

The calculation of the damage equivalent factors for fatigue λ should be done with the heavy traffic mix, that means waggons with 25t (250kN) axles, in accordance with Annex D of EN 1991-2.

Alternatively, if the standard traffic mix represents the actual traffic more closely than the heavy traffic mix, the standard traffic mix could be used, but with the calculated λ values enhanced by a factor 1,1 to allow for the influence of 250 kN axle loads. (Swiss National Annex)

General:

It cannot be stressed often enough that railway bridges must be designed and constructed in a fatigue-resistant way. For having optimal Life Cycle Costs (LCC) and for reaching the intended design life of minimum 100 years, all important structural members shall be designed for fatigue!

Rules for steel bridges:

Constructional details have to be chosen and found which give the maximum possible fatigue detail categories $\Delta\sigma c$, due to EN 1993-1-9:

Composite girders:	**detail category 71**
Welded plate girders:	**detail category 71**
Truss bridges:	**detail category 71 at sites where fatigue is a risk / detail category 36 at sites where fatigue is no risk.**

Constructional details, fatigue, (F)

European Commission

bad example (2004!)
(French but not SNCF)

good example (SNCF)

- **Dynamic enhancement for real trains**

  $1 + \varphi = 1 + \varphi' + (½)\ \varphi''$

- **Dynamic enhancement for fatigue calculations**

  $\varphi = 1 + ½(\varphi' + (½)\varphi'')$

- **Dynamic coefficient Φ_2 Φ_3**

  (determinant length L_Φ Table 6.2)

- **Dynamic enhancement for dynamic studies**

$$\varphi'_{dyn} = \max \left| y_{dyn} / y_{stat} \right| - 1$$

Permissible deflections (rules in Swiss Codes) (page 237 in book TELFORD)

In EN 1990, Annex A2 only minimum conditions for bridge deformations are given. The rule does not take into account track maintenance. A simplified rule for permissible deflections is given below for trains and speeds up to 200km/h, to avoid the need for excessive track maintenance. In addition, this simplified rule has the advantage, that no dynamic analysis is necessary for speeds less than 200km/h. For all classified lines with α >1,0, that means also if α = 1.33 is adopted for ULS, the following permissible values for deflections are recommended, always calculated with LM71 "+" SW/O, multiplied by Φ, and with α = 1.0:

V<80 km/h $\qquad \delta_{stat} \leq l / 800$*

*Note: Due to what is said in see A.2.4.4.2.3 [2], namely that the maximum total deflection measured along any track due to rail traffic actions should not exceed L/600, please note that 600 multiplied with 1,33 gives approximately 800.

$80 \leq V \leq 200$ km/h $\qquad \delta_{stat} \leq l / (15V – 400)$**

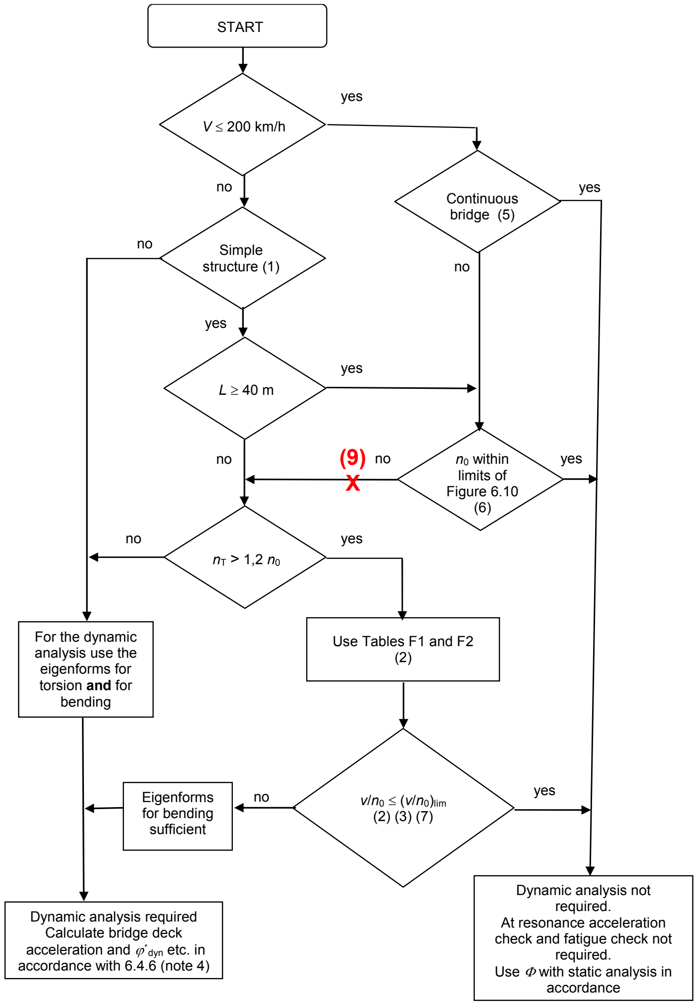
** Note: The upper limit l/2600 for 200 km/h is the permissible deflection which DB has taken during many years for designing bridges for high speed lines in Germany, with satisfactory results. It is also the formula which you can find in the Swiss Codes (SIA 260).

V > 200 km/h $\qquad$ value determined by dynamic study, but min. $\delta_{stat} \leq l / 2600$

Modified flow chart in Figure 6.9 of EN 1991-2

European Commission

Flow chart for determining whether a dynamic analysis is required.



(9) If the permissible deformations given just before are respected, taking into account less track maintenance, no dynamic study is necessary for speeds ≤ 200 km/h.

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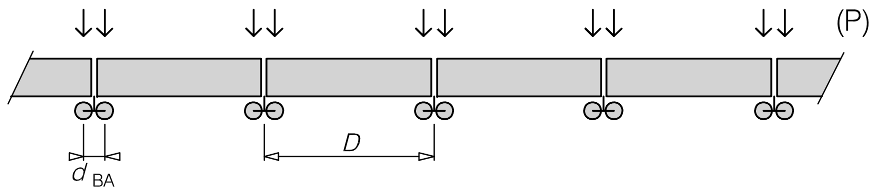
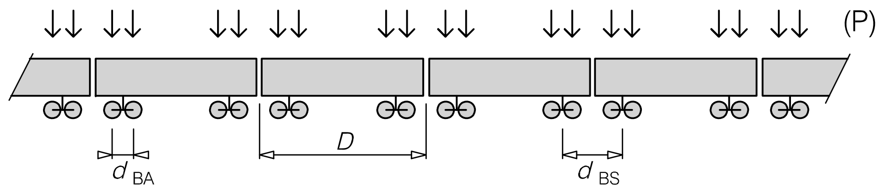
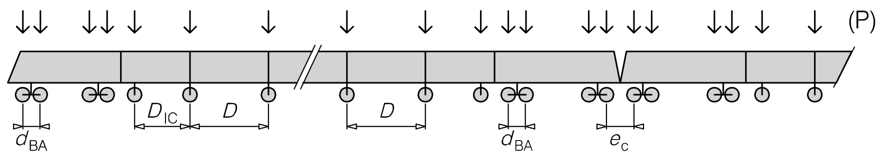
Rolling stock for high speeds (STI)



Articulated trains

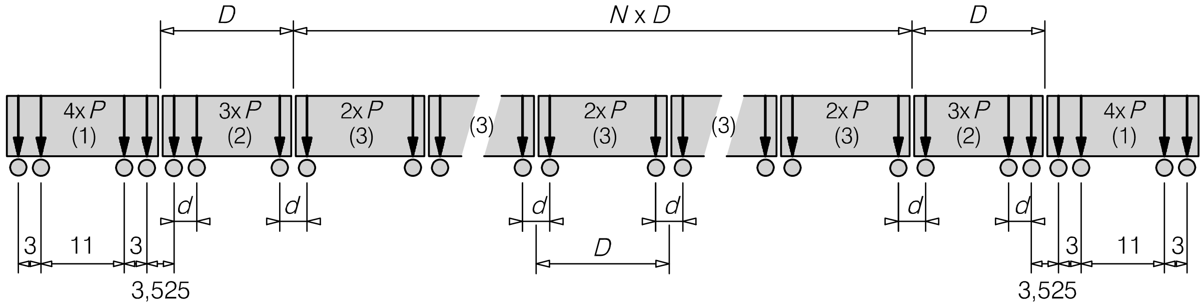


Conventional trains



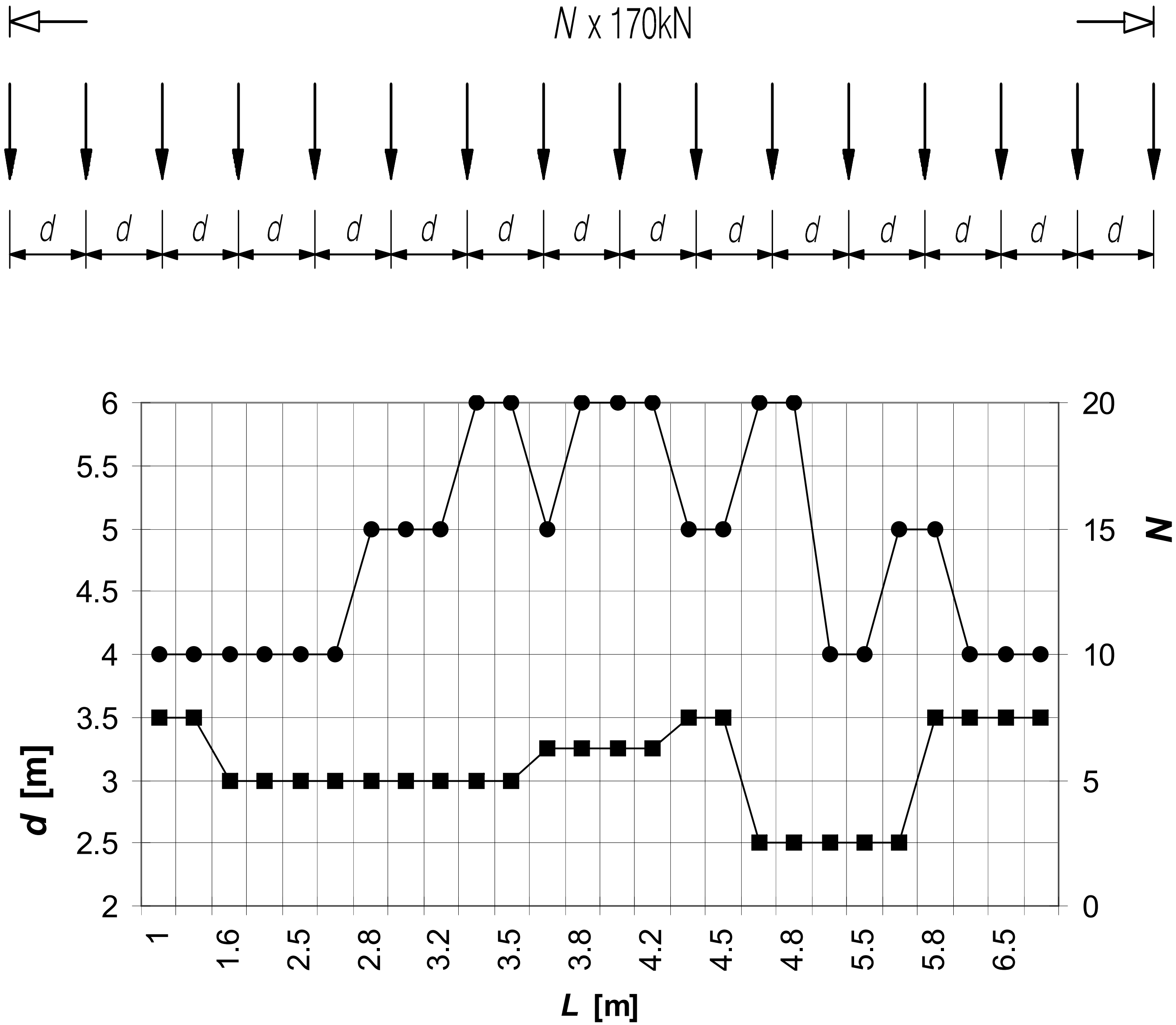
Regular trains

Models HSLM-A for int. lines



Universal Train	Number of intermediate coaches N	Coach length D [m]	Bogie axle spacing d [m]	Point force P [kN]
A1	18	18	2,0	170
A2	17	19	3,5	200
A3	16	20	2,0	180
A4	15	21	3,0	190
A5	14	22	2,0	170
A6	13	23	2,0	180
A7	13	24	2,0	190
A8	12	25	2,5	190
A9	11	26	2,0	210
A10	11	27	2,0	210

Models HSLM-B for int. lines



Application of HSLM-A and HSLM-B

Structural configuration	Span	
	$L < 7m$	$L \geq 7m$
Simply supported span	HSLM-B	HSLM-A 1 Train determined with the help of Annex E
Continuous structure or Complex structure	HSLM-A All Trains A1 to A10	HSLM-A All Trains A1 to A10

Determination of the critical Universal Train HSLM-A (EN1991-2, Annex E)

L = 15 m, simple supported bridge

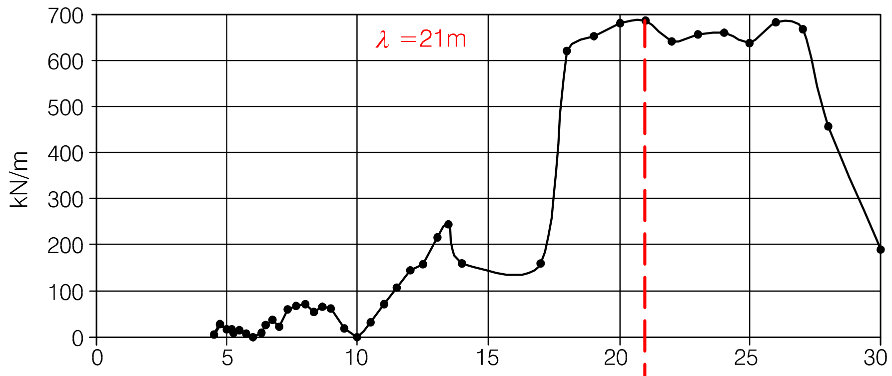
fo = 6 Hz

ζ = 1%

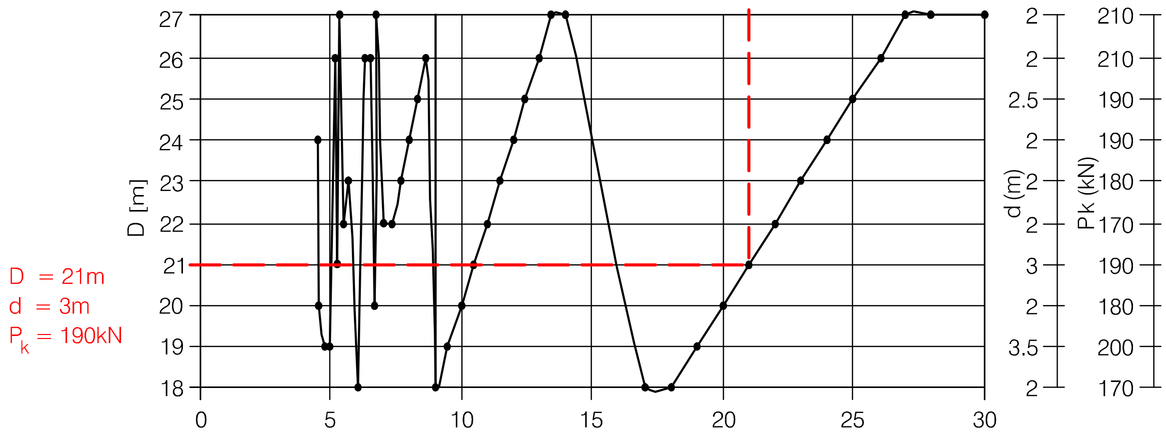
v max = 420 x 1,2 = 500 km/h (Maximum Design Speed)

so that λmax = v max/ fo = 500/3,6/6 = 23 m.

aggressiveness curve (E.7) →



Critical wavelength of excitation λc (E.18) →



Supplementary design checks for V > 200km/h

- **Max. peak deck along each track (EN1990:2002/A1, A2.4.4.2.1(4)P):**
  γ_{bt} = 0,35g (3,43 m/s^2) (ballasted track)

- **Verification of whether the calculated load effects from high speed trains are greater than those of normal rail traffic**

$$\left(1+\varphi'_{dyn}+\varphi''/2\right)\times\begin{pmatrix} HSLM \\ \text{or} \\ RT \end{pmatrix} \quad \text{or} \quad \Phi\times\text{(LM71"+"SW/0)}$$

- **Verification of fatigue where dynamic analysis is required**

- **Verification of twist**

- **Maximum vertical deflection for passenger comfort (EN1990:2002/A1, A2.4.4.2.3(1))**
  not necessary if you take permissible deflections recommended before

European HS Network Situation as at 12.2008

European Commission

v ≥ 250 km/h

v ≥ 250 km/h planned

180 ≤ v < 250 km/h

Other lines

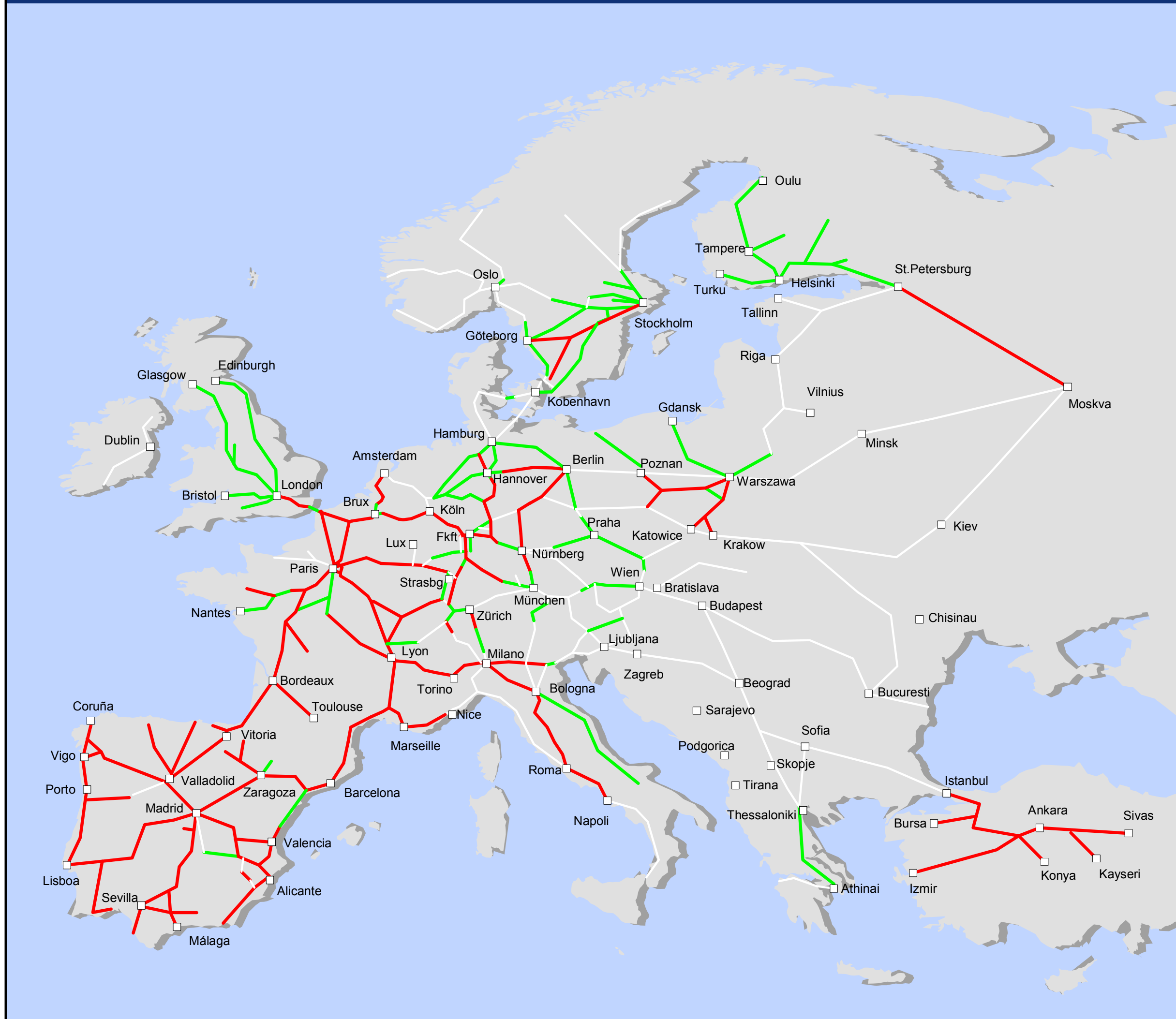
Information given by the Railways

UIC - High-Speed
Updated 14.12.2008

European HS Network Forecasting 2025

European HS Network

Forecasting 2025



v ≥ 250 km/h

v ≥ 250 km/h Planned

180 ≤ v < 250 km/h

Other lines

Information given by the Railways

UIC - High-Speed
Updated 14.12.2008
OG/IB

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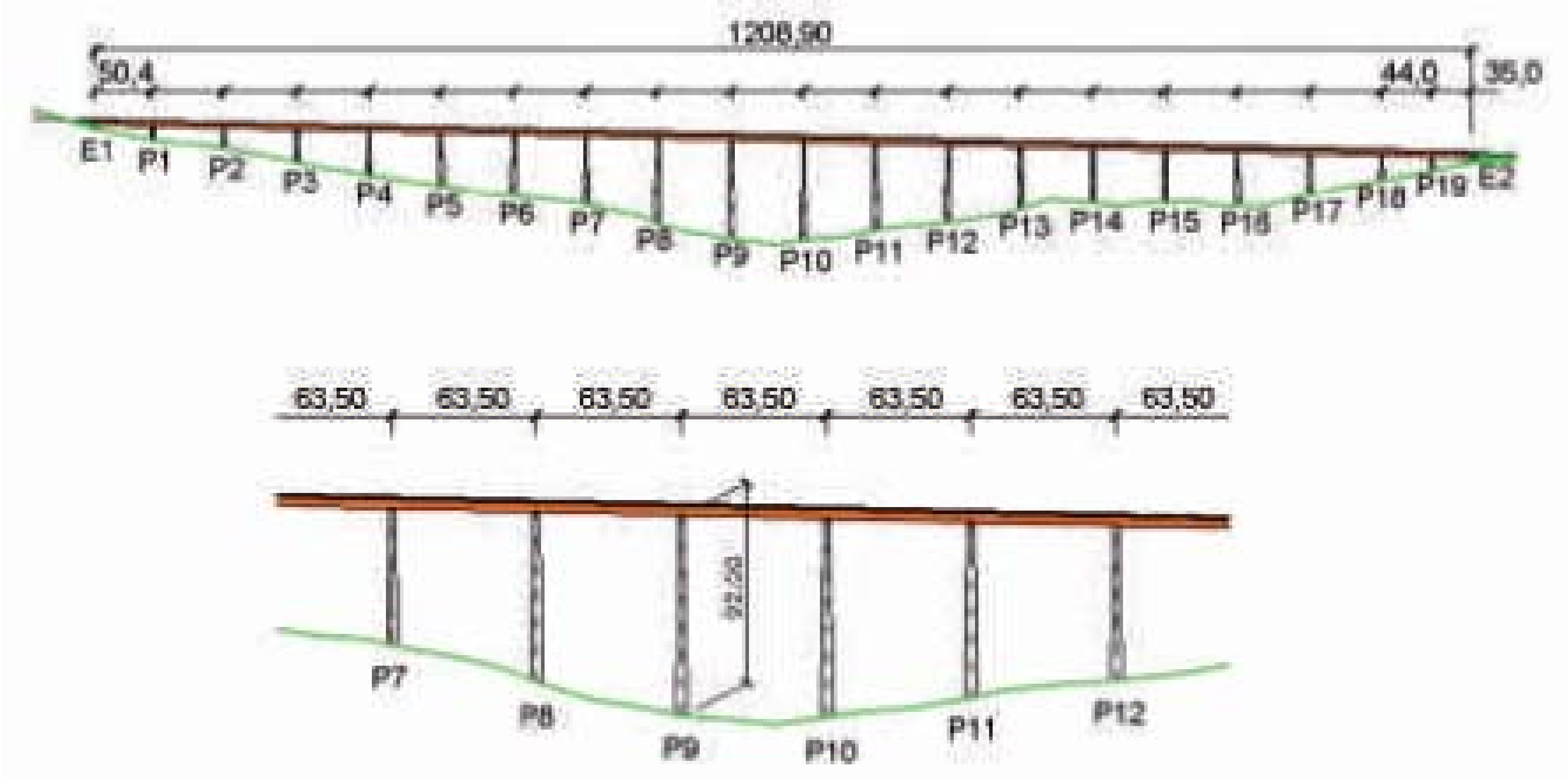
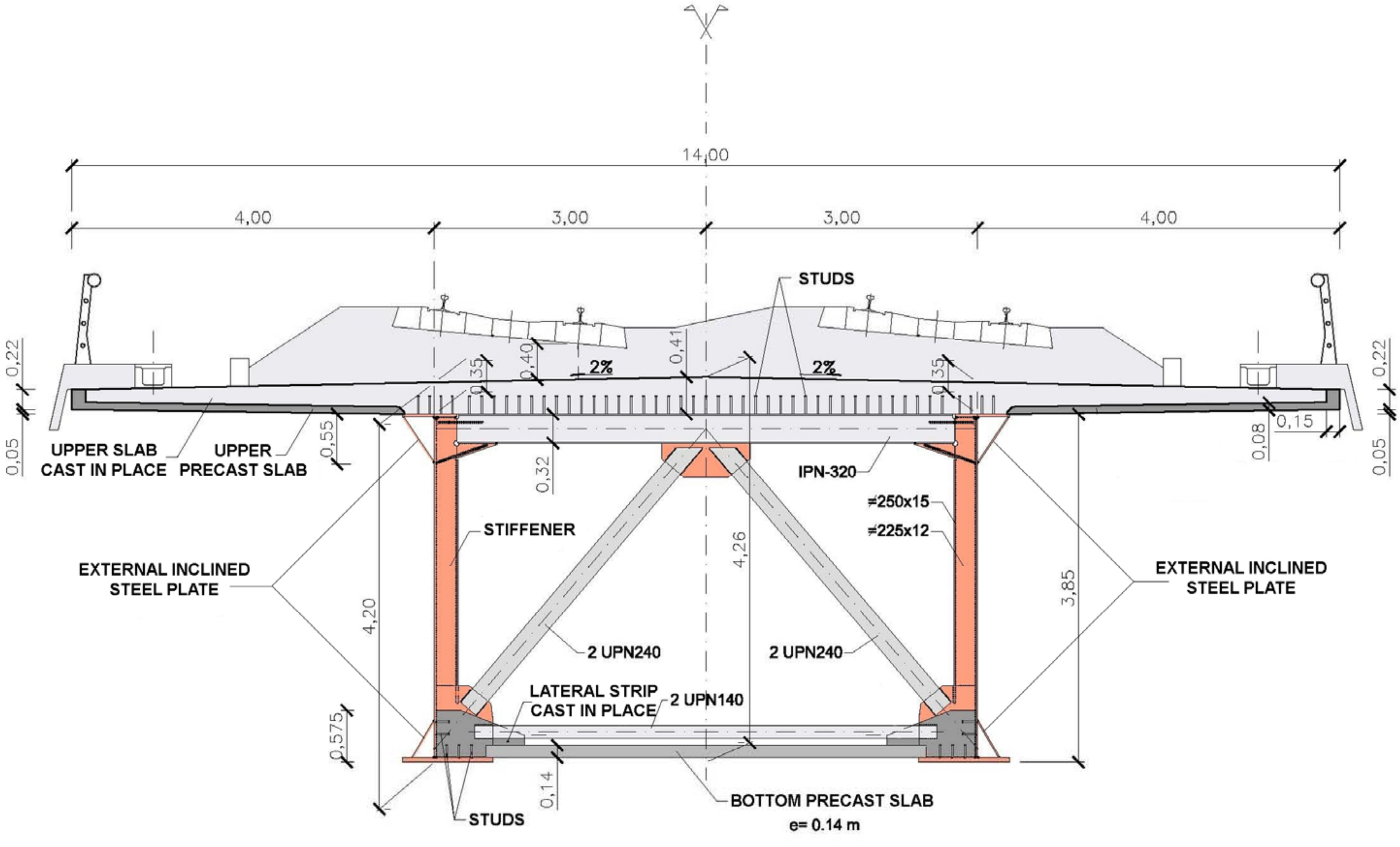
General view of the Arroyo Las Piedras viaduct , 1208.9 m, 2005, (Spain)

Elevation view of the Arroyo Las Piedras viaduct [m]

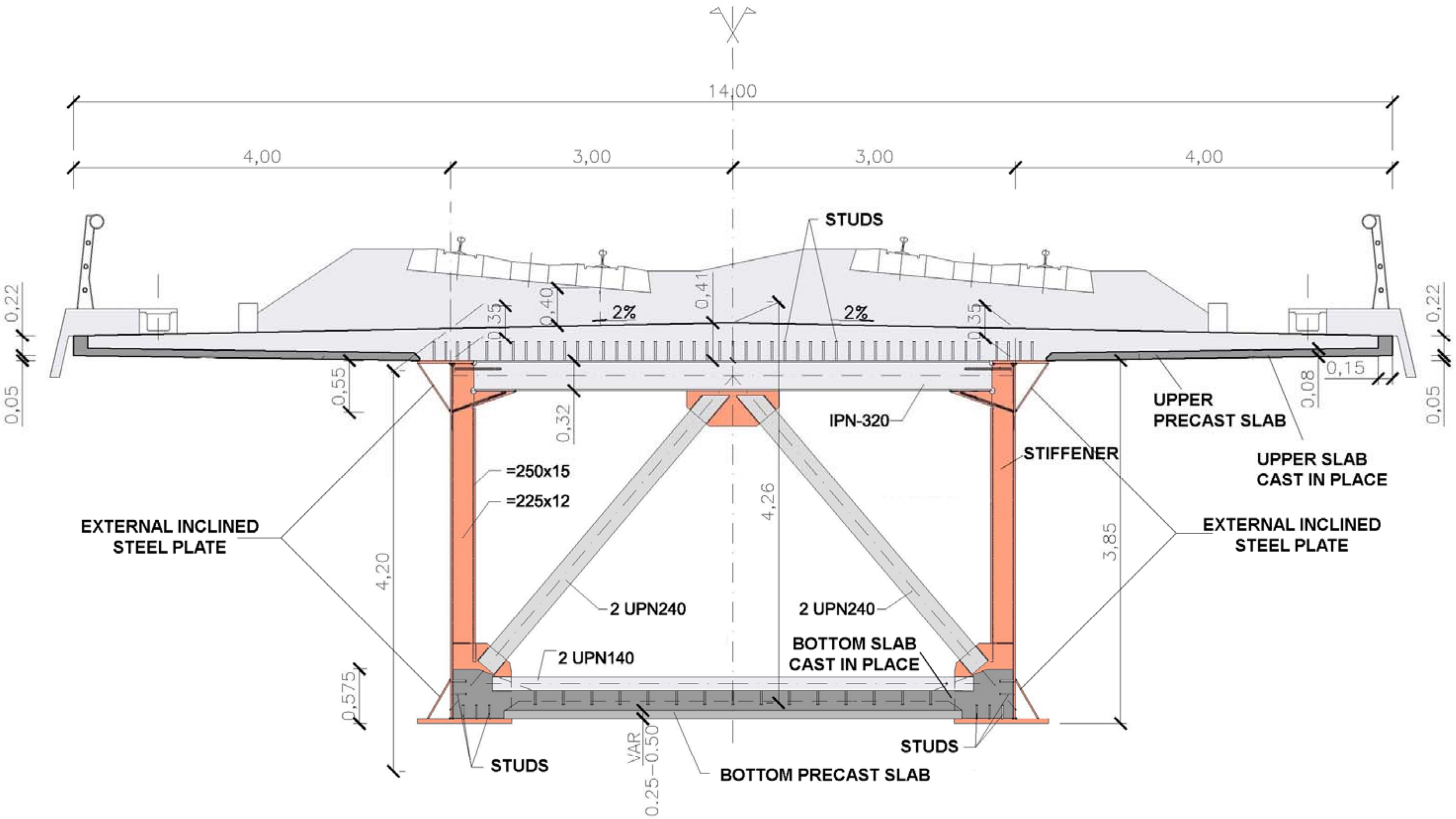


Shock absorbers of the Arroyo Las Piedras viaduct

Mid-span cross section of the Arroyo Las Piedras viaduct



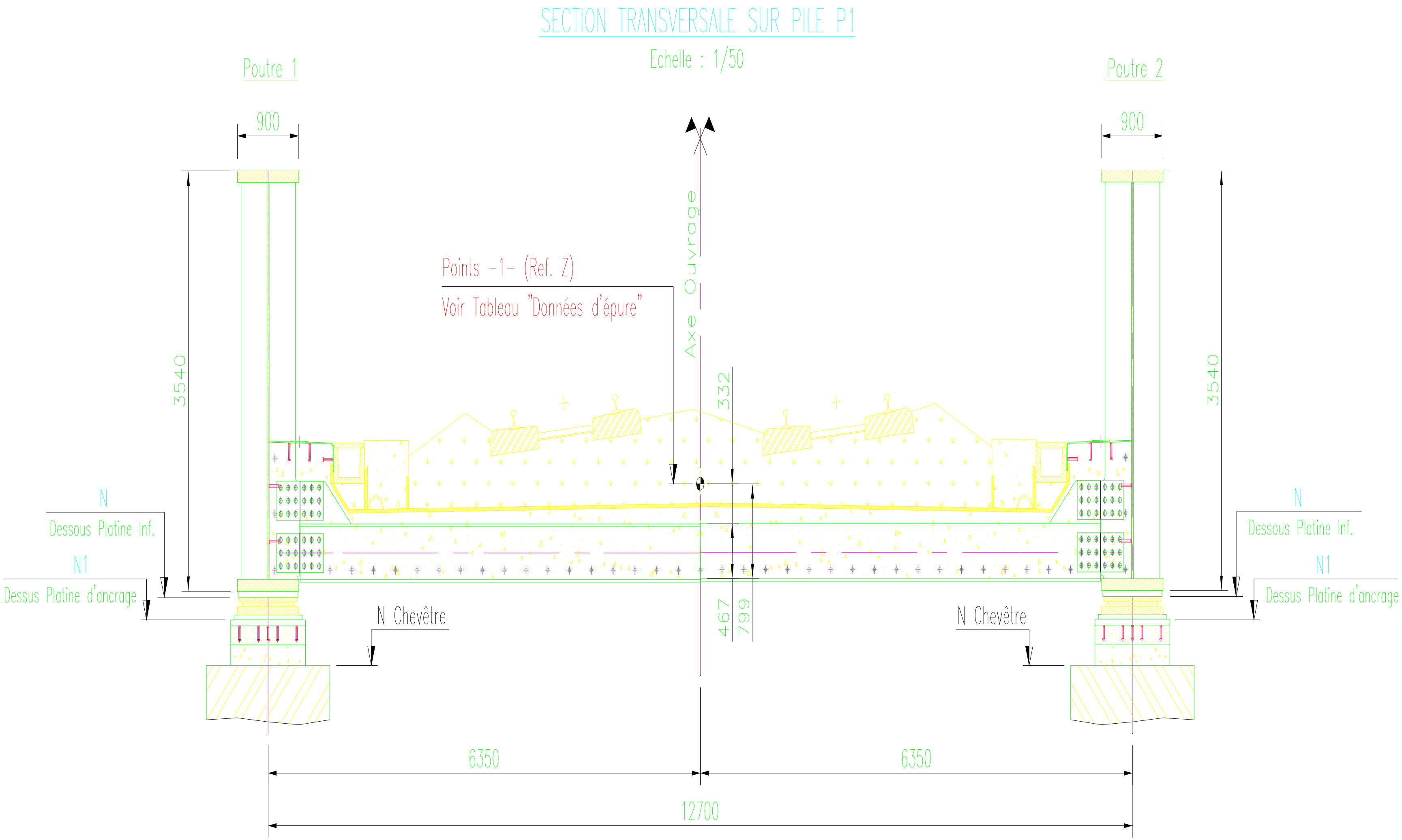
Hogging cross section of the Arroyo Las Piedras viaduct



Half through bridges with two lateral main girders (welded plates), France

Crossing over A104 at Pomponne
Deckslab; embedded cross girders

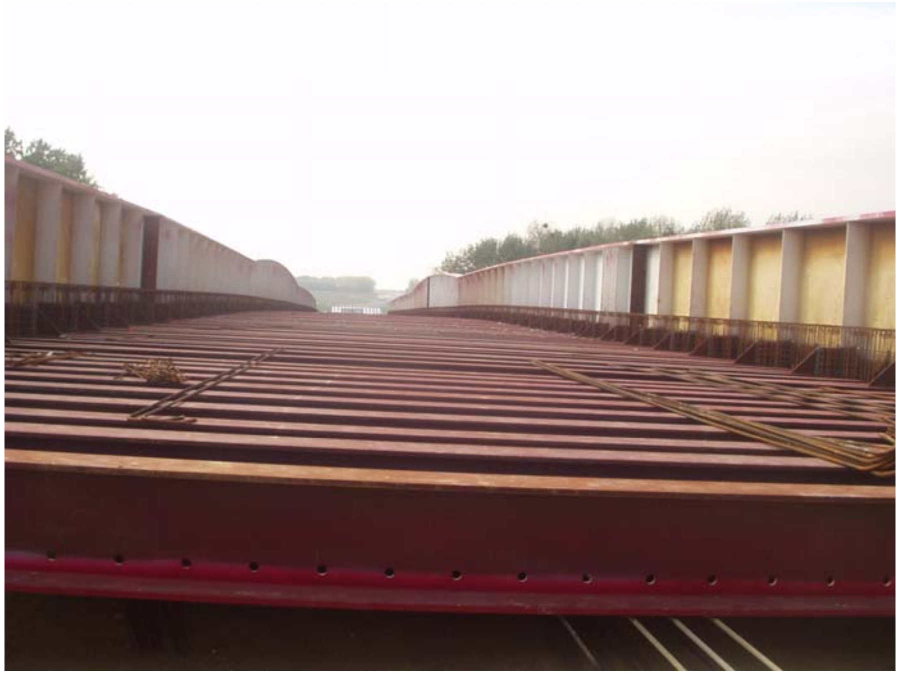
Crossing over A104 at Pomponne (77) (F)



Half through bridges with two lateral main girders (welded plates), France

Viaduct crossing the A4
(département de l'Aisne)

Viaduct crossing the A4 (département de l'Aisne)



Deck plate: embedded cross girders

Concrete deck over two welded steel plate main girders (France)

Viaduct crossing A31 near Lesmésnils

Viaduct crossing A31 near Lesménsils

Viaduc de Mornas, LGV Méditerranée, span 121,4 m, built 1999, F

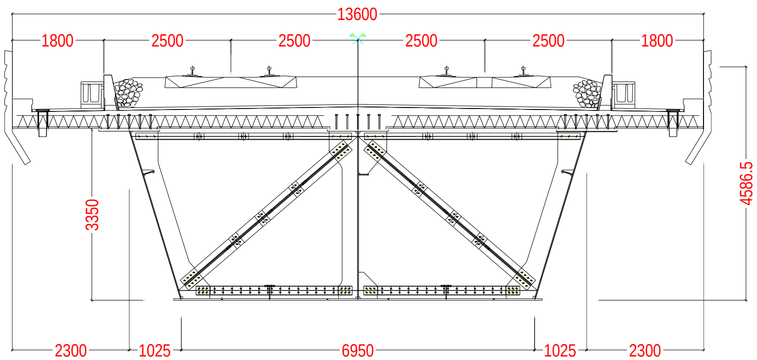
Viaduc de la Garde-Adhémar, LGV Méditerranée, 2 spans of 115.4 m, total length 325 m, built in 2000, F

Viaduc du Péage de l’A7 à Bonpas (TGV Méd.,1998, span 124 m), F

Sesia Viaduct,Torino-Milano High Speed Railway line, 2003, (I)

7 x 46 m = 322 m

Sesia viaduct



M5 twin parallel girder bridge, HSRL Vienna - Salzburg, 1994, (A)

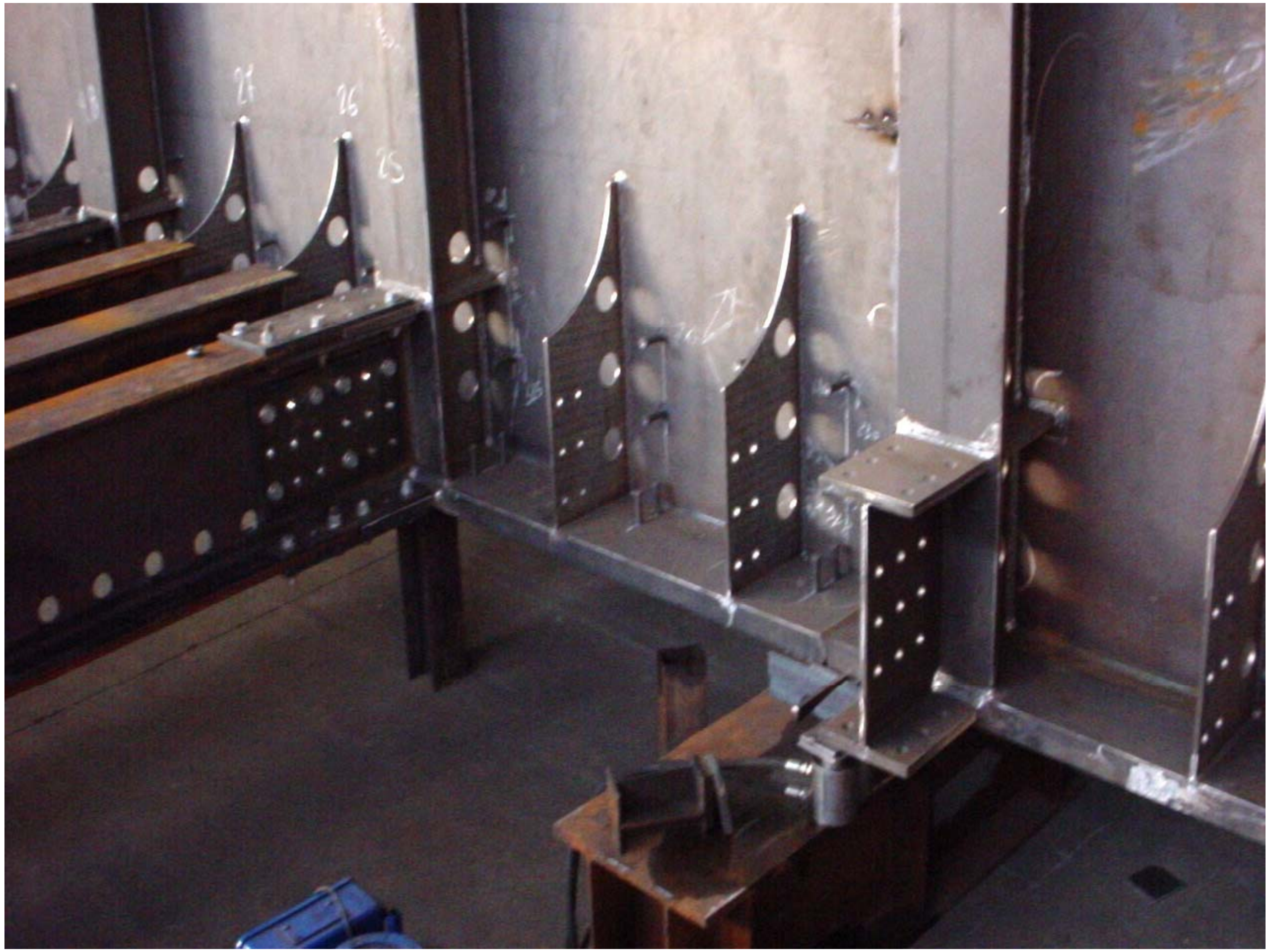
Risk scenario to avoid, yesterday and tomorrow:

<= Collapse of railway bridge over the river Birs in Münchenstein, Switzerland, the 14th June 1891, by buckling of a diagonal in the middle of the bridge under an overloaded train, 73 persons were killed, 131 persons more or less injured.=> Tetmajers law.

Stewarton collapse, 27th January 2009, bridge in wrought iron

Bridge collapse beneath a train of 100 ton tank wagons travelling at 60 mph. Centre and east side girders failed in shear due to very severe corrosion of the webs which had been concealed against inspection by timber boards retaining the ballast

Risk scenario to avoid tomorrow:



3012: collapses due to fatigue cracks in bad details of welded constructions executed today???