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Comparative research to establish load factors for railway bridges

H.H. SNIJDER
Civil Engineer
Holland Railconsult and
Eindhoven University of
Technology
The Netherlands.

F.H. ROLF
Mechanical Engineer
Holland Railconsult
Utrecht
The Netherlands.

Frans Rolf, born in 1949, got his mechanical engineering degree at Twente University of Technology in 1974. He started his career as consultant for dynamics at the Netherlands Railways. In 1980 he became bridge engineer specialised in movable bridges. Since 1987 he was subsequently head of the bridge maintenance department, head of the steel design section and the civil works section of Netherlands Railways. Today he is senior account manager of Holland Railconsult.

Bert Snijder, born in 1959, got his civil engineering degree at Delft University of Technology in 1983. He started his career at TNO Bouw as research engineer in the field of stability and fatigue of steel structures. In 1989 he became design engineer specialised in fixed steel bridges at the Netherlands Railways. At present he is involved in the design of steel building structures for Holland Railconsult. Since 1993 he is part time professor of steel structures at Eindhoven University of Technology.

Summary

In this paper the results of a comparative research to establish load factors for railway bridges are presented. These results form the main input for section 6. part 3 of Eurocode 1 'Railway Loading' [1] for as far as partial factors are concerned. The research has been carried out by a working group of the subcommittee 'Bridges' of the UIC (Union Internationale des Chemins de fer). Aim of the research was to examine existing practices and codes. Based on these, a set of partial safety factors was proposed, to be applied to variable and permanent actions for structures carrying railway traffic.

1. Introduction

During development of European Design Codes there was need to establish safety factors for railway loading. The subcommittee 'Bridges' of UIC (Union International de Chemin de Fer) set up a working party 'Safety factors'. The results of this working party are presented in a report [2].

The rules for establishing Eurocodes say that the safety factors have to be based on probabilistic study. As very little data were available, the working party decided first to compare existing practices in the member countries with proposed Eurocodes and then tried to propose a set of partial safety factors on basis of those results.

2. Approach

It was quite clear from a brief survey of the five countries involved that just a comparison of used safety factors in the different codes would not be satisfactory.
The fact that the safety of the construction was sometimes covered by other figures than the safety factors made this impossible. Sometimes safety is implicitly available in the allowable stresses or in a higher traffic load.

The five codes (table 1) involved were the regulations used by the railways in:

Denmark	(DSB);
France	(SNCF);
Germany	(DB);
The Netherlands	(NS);
United Kingdom	(BR).

ACTION		PARTIAL SAFETY FACTORS				
		ADMINISTRATION				
		NS	SNCF	BR	DSB	DB
Permanent Action	Self Weight (Steel)	1.50	1.32	1.10	1.00	1.35
	Ballast	1.50	1.32x1.30	1.75	1.20	1.80
Variable Traffic Action	Load Model 71 *	1.50	1.35	1.40	1.30	1.35

Table 1. Fundamental partial safety factors for loads.

This leads to the conclusion that for comparison the total effect of the code on a structure should be studied. So it was agreed that comparative calculations would be required.

Six steel bridges and three concrete bridges were chosen to cover the range of spans most commonly encountered. For steel bridges three bridges with ballasted track were considered and three with non ballasted track. Only simply supported structures were chosen and only the positions of maximum bending and maximum shear of the main structure were studied.
The bridges listed in table 2 were involved in the study.

Name	Material	Type	Span	Track	Annex
SU	steel	girder	11.1	non ballasted	1 a
MU	steel	trough	29.4	,,	1 b
LU	steel	truss	59.7	,,	1 c
SB	steel	girder	11.1	ballasted	1 a
MB	steel	trough	29.4	,,	1 b
LB	steel	truss	59.7	,,	1 c
S	concrete	reinforced slab	5	,,	2 a
M	concrete	reinforced slab	15	,,	2 b
L	concrete	posttensioned boxgirder	42	,,	2 c

Table 2. Bridges studied.

3. Utilisation factors

Each bridge of table 2 was calculated by using the five sets of regulations of the administrations involved and once by using the proposed Eurocodes. So for each bridge six calculations were made and in each calculation two utilisation factors (α) were established, one for maximum bending and one for maximum shear. In case of truss bridges utilisation factors for maximum normal force were established.

$$\alpha_{code} = \frac{\text{effect of design loads according the code}}{\text{design value of resistance according the code}}$$

Only direct permanent and variable traffic action and strength criteria were considered. Aspects as stability and fatigue were neglected.

The results using the national codes were compared with those according to the proposed European codes by establishing a utilisation factor α^1 as given below:

$$\alpha^1 = \frac{\alpha_{admin}}{\alpha_{Eurocode}}$$

As the aim of the study was to establish a set of partial safety factors, the set of safety factors for Eurocode could be varied.
The calculations were carried out by using a set as given below:

γ_{G1}	=	1.35	permanent action self weight
γ_{G2}	=	1.80	permanent action ballast
γ_Q	=	1.50	variable traffic action

By varying this set the α^1 value could be influenced and a best fit between national codes and the European code could be established. Two criteria, as given below, were used for this purpose:

1 $\quad A = \dfrac{\Sigma(\alpha' - 1.0)}{n}$

2 $\quad B = \dfrac{\sqrt{\Sigma(\alpha' - 1.0)^2}}{n}$

where n = number of calculations.

In practice, the value of criterion A proved the more sensitive as can be seen from table 3 for four sets of safety factors.

ACTION		PARTIAL SAFETY FACTOR			
Permanent Action	Self Weight (steel) Ballast	1.20 1.60	1.20 1.60	1.20 1.60	1.20 1.60
Variable Traffic Action	Load Model 71	1.50	1.35	1.40	1.30
CRITERION A CRITERION B		+0.112 +0.021	+0.050 +0.017	- 0.006 +0.015	- 0.056 +0.016

Table. 3. Comparison of sensitivity of criterion A and B.

4. Load factors for steel bridges only

For the six steel bridges the calculation results are shown in table 4. It can be seen from this table that α^1 varies from 0.77 to 1.21. It is quite easy to understand that for $\alpha^1 = 1$ there is complete fit between the national code and Eurocode. This coincides with A or B = 0. As A is the more sensitive only A-values are considered for optimising.

Partial safety factors

γ_{g1} = 1,35 steel
γ_{g2} = 1,80 ballast
γ_{uic} = 1,50 live load

bridge type	sectional forces	EC3 α_{EC}	DB α_{DB}	α'	NS α_{NS}	α'	DSB α_{DSB}	α'	SNCF α_{SNC}	α'	BR α_{BR}	α'	Tot	
SU	M	0,81	0,73	0,90	0,83	1,02	0,85	1,05	0,67	0,83	0,83	1,02	0,78	0,97
	Q	0,44	0,37	0,84	0,42	0,95	0,41	0,93	0,35	0,79	0,38	0,86	0,39	0,88
MU	M	0,47	0,44	0,94	0,51	1,08	0,50	1,06	0,41	0,87	0,57	1,21	0,49	1,03
	Q	0,27	0,25	0,92	0,30	1,11	0,30	1,11	0,25	0,92	0,25	0,92	0,27	1,00
LU	N5	0,62	0,56	0,90	0,70	1,13	0,62	1,00	0,51	0,82	0,57	0,92	0,59	0,96
	N7	0,77	0,69	0,90	0,87	1,13	0,77	1,00	0,63	0,82	0,71	0,92	0,73	0,95
	N8	0,51	0,45	0,88	0,58	1,14	0,51	1,00	0,41	0,80	0,51	1,00	0,49	0,96
SB	M	0,96	0,85	0,89	0,88	0,92	0,95	0,99	0,77	0,80	1,03	1,07	0,90	0,93
	Q	0,64	0,57	0,89	0,60	0,94	0,63	0,99	0,53	0,83	0,69	1,08	0,60	0,95
MB	M	0,69	0,62	0,90	0,63	0,91	0,66	0,96	0,55	0,80	0,81	1,18	0,65	0,95
	Q	0,42	0,38	0,90	0,38	0,90	0,40	0,95	0,29	0,69	0,37	0,88	0,36	0,86
LB	N5	0,89	0,79	0,89	0,88	0,99	0,85	0,95	0,69	0,77	0,87	0,98	0,82	0,91
	N7	1,11	0,98	0,88	1,09	0,98	1,06	0,95	0,86	0,77	1,07	0,96	1,01	0,91
	N8	0,73	0,64	0,88	0,73	1,00	0,69	0,95	0,56	0,77	0,77	1,05	0,68	0,93
	Σ (x-1)/n	→		-0,11		0,01		-0,01		-0,19		0,00		-0,058
	√Σ (x-1)2/n	→		0,03		0,02		0,01		0,05		0,03		0,014

Table. 4. Resume of α-values for steel bridges.

For optimising the A-value the partial safety factors for self weight steel and ballast were fixed at 1.35 and 1.80. The safety factor for variable traffic action was varied. The value of the partial safety factor for ballast of 1.80 was made up from a self weight factor of 1.35 and a height factor of 1.33 (1.35 x 1.33 = 1.80). The self weight factor 1.35 is in line with values for permanent actions elsewhere in the Eurocodes.

The results of this analysis are shown in figure 1.
The dashed line indicates the optimum value for the safety factor for variable traffic action.

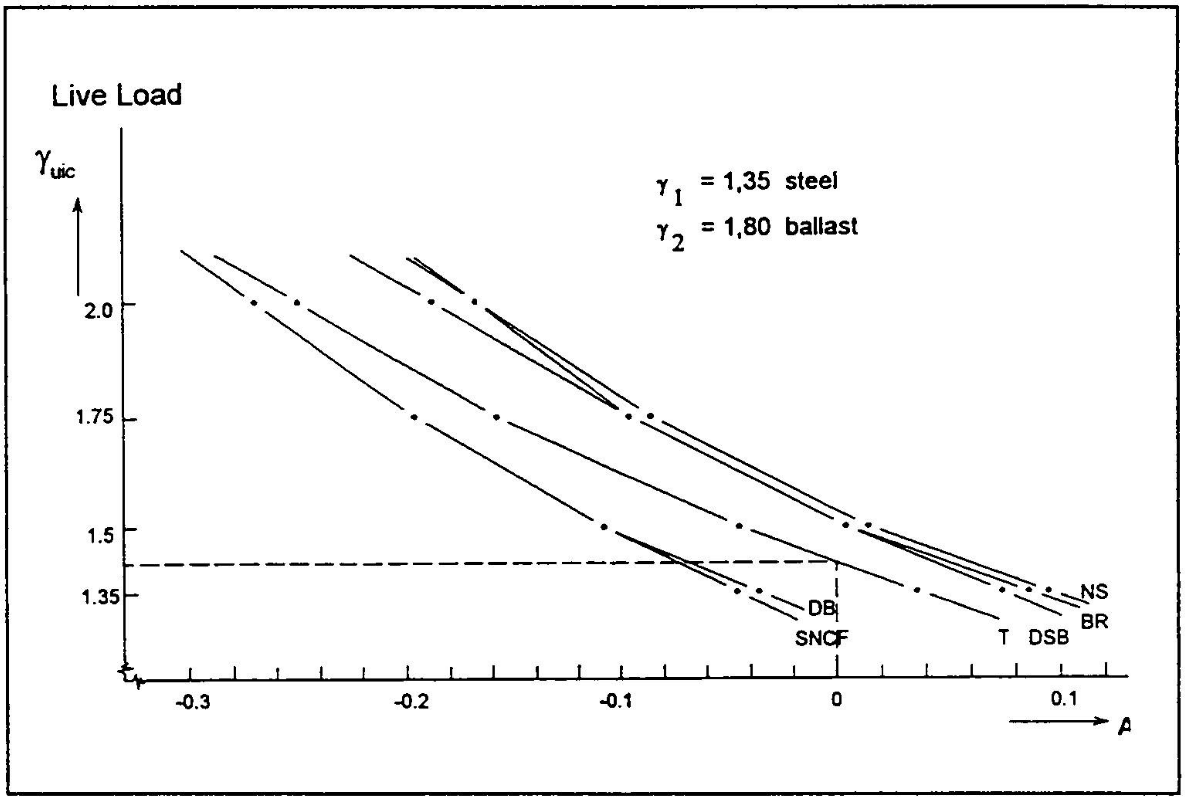

Figure. 1. Optimisation of criterion A with $\gamma_{dead\ load}$= 1.35.

It is interesting to note that the values of A derived from the codes of DB and SNCF are similar and always smaller than the values derived from the codes used by DSB, NS and BR. By T the mean values of the total amount of data are represented.

This means that in general DB and SNCF allow heavier traffic than DSB, BR and NS on the same construction. The proposed approach for this study will lead to a Eurocode that allows traffic that will lie between the two sets of administrations (DB and SNCF on the one hand DSB, BR and NS on the other).

The best fitting set found is:

γ_{G1} = 1.35
γ_{G2} = 1.80
γ_Q = 1.40

The difference between the values of γ_{G1} and γ_Q is very small. It seems unrealistic that the safety factor for self weight permanent actions and variable traffic action are so similar. Traffic actions are in general much less predictable than self weight actions. So a new optimum was investigated on basis of the following self weight factors:

γ_{G1} = 1.20 and
γ_{G2} = 1.60.

The results are shown in figure 2.

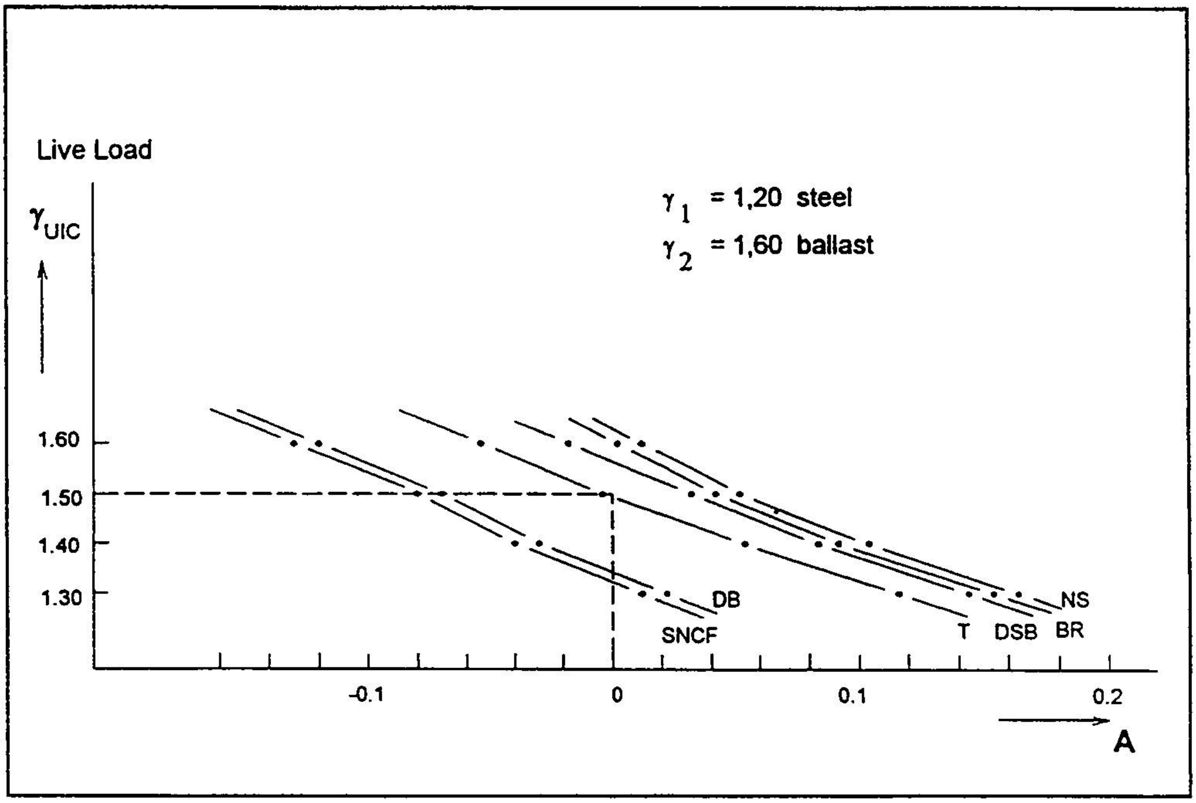

Figure. 2. Optimisation of criterion A with $\gamma_{dead\ load}$=1.20.

The optimum set showed to be:

γ_{G1} = 1.20
γ_{G2} = 1.60
γ_{Q} = 1.50

for steel bridges.

5. Load factors for steel and concrete bridges

The approach as shown above for steel bridges showed to be more complicated for concrete structures . Due to the fact that the model of failure used in the national codes are not the same, the design verification of various national codes differed very much.

So for concrete bridges new α factors had to be defined. A list of 11 different factors was the result (see table 5).

	criterion of checking	limite state	DEFINITION OF α-VALUE	according to Eurocode
BENDING				
1	concrete	SLS	$\alpha_c = \max\sigma_c/(0.45f_{ck})$ (quasi-permanent) $\alpha_c = \max\sigma_c/(0.60f_{ck})$ (infrequent)	EC2/2 4.4.1.1 (2) EC2/2 4.4.1.1 (3)
2	reinforcement steel	SLS	$\alpha_s = \max\sigma_s/(0.8f_{yk})$ (infrequent)	EC2/2 4.4.1.1 (6)
3	prestressing steel	SLS	$\alpha_p = \max\sigma_{poo}/(0.65f_{pk})$ (quasi-permanent) $\alpha_p = \max\sigma_{p0}/(0.75f_{pk})$ (infrequent)	EC2/2 4.4.1.1 (7) EC2/1 4.4.1.1 (7)
4	cross section (reinforced)	ULS	$\alpha_{cs} = M_{Sd}/M_{Rd}$	EC2/2 4.3.1
5	prestressing steel	ULS	$\alpha_p = M_{Sd}/M_{Rd}$ (M_{Rd}: $0.9f_{pk}/\gamma_s$)	EC2/1 4.2.3.3.3
6	concrete (prestressed)	ULS	$\alpha_c = \max\sigma_c/(\alpha f_{cd})$	EC2/1 4.2.1.3.3
7	crack width (reinforced)	SLS	$\alpha_{crack} = \max\sigma_s/\sigma_{s(table\ 4.11.a\ or\ b)}$	EC2/2 4.4.2.3.3
8	decompresion (prestressed)	SLS	$\alpha_{p,\ decomp} = F_{p,min}/F_{p,exist}$ $F_{p,min}$: $\varepsilon_{c1} = 0$ (infrequent)	EC2/2 4.4.2.1 (3)
9	deflection	SLS	$\alpha_f = f_{(\Phi UIC71)}/(1/1000)$	Ec1 3.4 (agreement based on a paper of discusion for PT7)
SHEAR				
10	struts	ULS	$\alpha_{c,strut} = V_{Sd(x=0)}/V_{Rd2}$	EC2/1 4.3.2.4.3 (stand.) EC2/1 4.3.2.4.4 (incl.)
11	steel + concrete	ULS	$\alpha_{s+c} = V_{Sd(x=0.5d)}/V_{Rd3}$	EC2/1 4.3.2.4.3 (stand.) EC2/1 4.3.2.4.4 (incl.)

Table. 5. Definition of α-value for concrete bridges.

As very few criteria were used in all national codes only criteria 4 and 5 were available for comparison.
A partial safety factor for prestress was introduced for the prestressed or post tensioned structures. This factor was assumed to be 1,00.
In the same way as for steel bridges, the optimum value for A could be found by varying the set of safety factors. To limit the amount of work 10 sets were investigated with γ-factors varying as follows:

γ_{G1} = 1,20; 1,35
γ_{G2} = 1,60; 1,78; 1,80
γ_{pr} = 1,00
γ_Q = 1,35; 1,40; 1,43; 1,45; 1,50

Out of these the most interesting sets are:

γ_{G1}	=	1,20	1,35
γ_{G2}	=	1,80	1,80
γ_{pr}	=	1,00	1,00
γ_Q	=	1,50	1,43

The results of these two sets are given in the tables 6 and 7.

Partial safety factors

γ_{g1} = 1,20 dead load
γ_{g2} = 1,80 ballast
γ_{uic} = 1,50 live load
γ_{pre} = 1,00 prestress

bridge type	criterium	EC α	DB α	DB α'	NS α	NS α'	DSB α	DSB α'	SNCF α	SNCF α'	BR α	BR α'	Tot (Σα)/n	(Σα/n)/α	
steel															
SU	M	0,81	0,73	0,91	0,83	1,03	0,85	1,05	0,67	0,83	0,83	1,03	0,78	0,97	
	Q	0,44	0,37	0,84	0,42	0,96	0,41	0,94	0,35	0,80	0,38	0,87	0,39	0,88	
MU	M	0,46	0,44	0,96	0,51	1,11	0,50	1,09	0,41	0,90	0,57	1,25	0,49	1,06	
	Q	0,26	0,25	0,95	0,30	1,14	0,30	1,14	0,25	0,95	0,25	0,95	0,27	1,03	
LU	N5	0,60	0,56	0,94	0,70	1,17	0,62	1,04	0,51	0,85	0,57	0,95	0,59	0,99	
	N7	0,74	0,69	0,93	0,87	1,17	0,77	1,04	0,63	0,85	0,71	0,96	0,73	0,99	
	N8	0,49	0,45	0,91	0,58	1,18	0,51	1,04	0,41	0,83	0,51	1,04	0,49	1,00	
Σ	(x-1)/n	→		-0,08		0,11		0,05		-0,14		0,01		-0,012	
√Σ	(x-1)2/n	→		0,03		0,05		0,03		0,06		0,04		0,019	
SB	M	0,95	0,85	0,89	0,88	0,92	0,95	1,00	0,77	0,81	1,03	1,08	0,90	0,94	
	Q	0,63	0,57	0,90	0,60	0,95	0,63	1,00	0,53	0,84	0,69	1,09	0,60	0,96	
MB	M	0,67	0,62	0,92	0,63	0,93	0,66	0,98	0,55	0,82	0,81	1,20	0,65	0,97	
	Q	0,41	0,38	0,92	0,38	0,92	0,40	0,97	0,29	0,70	0,37	0,90	0,36	0,88	
LB	N5	0,87	0,79	0,91	0,88	1,01	0,85	0,98	0,69	0,80	0,87	1,00	0,82	0,94	
	N7	1,08	0,98	0,91	1,09	1,01	1,06	0,98	0,86	0,80	1,07	0,99	1,01	0,94	
	N8	0,71	0,64	0,90	0,73	1,03	0,69	0,97	0,56	0,79	0,77	1,09	0,68	0,96	
Σ	(x-1)/n	→		-0,09		-0,03		-0,02		-0,21 C		0,05		-0,059	
√Σ	(x-1)2/n	→		0,03		0,02		0,01		0,08		0,04		0,020	
Σ	(x-1)/n	→		-0,09		0,04		0,02		-0,17		0,03		-0,035	•
√Σ	(x-1)2/n	→		0,02		0,03		0,01		0,05		0,03		0,014	
concrete															
S	4	0,58	0,53	0,91	0,52	0,89	0,55	0,94	0,60	1,03	0,72	1,24	0,58	1,00	
M	4	0,82	0,81	0,99	0,81	0,99	0,85	1,04	0,90	1,10	0,91	1,12	0,86	1,05	
L	5	0,77	0,86	1,12	0,85	1,11	0,83	1,08	0,76	0,99	0,78	1,02	0,82	1,06	
Σ	(x-1)/n	→		0,01		0,00		0,02		0,04		0,12		0,038	• •
√Σ	(x-1)2/n	→		0,05		0,05		0,04		0,04		0,09		0,025	D = |•-•|
total															
Σ	(x-1)/n	→		-0,07		0,03		0,02		-0,14		0,04		-0,022	
√Σ	(x-1)2/n	→		0,02		0,02		0,01		0,04		0,03		0,012	
	MEAN(steel, concrete)	Σ(x-1)/n	A											0,001	A
	MEAN(steel, concrete)	√Σ(x-1)2/n	B											0,019	

A : 0.001
C : - 0.21
D : 0.073

Table. 6. Resume of α-values for steel and concrete bridges for the first set of γ-factors.

Partial safety factors

γ_{g1}	=	1,35	dead load
γ_{g2}	=	1,80	ballast
γ_{uic}	=	1,43	live load
γ_{pre}	=	1,00	prestress

bridge type	criterium	EC	DB		NS		DSB		SNCF		BR		Tot	$(\Sigma\alpha/n)/\alpha$
		α	α	α'	α	α'	α	α'	α	α'	α	α'	(Σα)/n	
steel														
SU	M	0,77	0,73	0,94	0,83	1,07	0,85	1,10	0,67	0,87	0,83	1,07	0,78	1,01
	Q	0,42	0,37	0,88	0,42	1,00	0,41	0,97	0,35	0,83	0,38	0,90	0,39	0,92
MU	M	0,45	0,44	0,97	0,51	1,12	0,50	1,10	0,41	0,90	0,57	1,26	0,49	1,07
	Q	0,26	0,25	0,96	0,30	1,15	0,30	1,15	0,25	0,96	0,25	0,96	0,27	1,03
LU	N5	0,60	0,56	0,93	0,70	1,17	0,62	1,03	0,51	0,85	0,57	0,95	0,59	0,99
	N7	0,75	0,69	0,93	0,87	1,17	0,77	1,03	0,63	0,85	0,71	0,95	0,73	0,99
	N8	0,49	0,45	0,91	0,58	1,17	0,51	1,03	0,41	0,83	0,51	1,03	0,49	1,00
	Σ (x-1)/n	→		-0,07		0,12		0,06		-0,13		0,02		0,000
	√Σ (x-1)2/n	→		0,03		0,05		0,03		0,05		0,04		0,019
SB	M	0,93	0,85	0,92	0,88	0,95	0,95	1,02	0,77	0,83	1,03	1,11	0,90	0,96
	Q	0,62	0,57	0,92	0,60	0,97	0,63	1,02	0,53	0,86	0,69	1,12	0,60	0,98
MB	M	0,67	0,62	0,92	0,63	0,94	0,66	0,98	0,55	0,82	0,81	1,21	0,65	0,97
	Q	0,41	0,38	0,93	0,38	0,93	0,40	0,97	0,29	0,71	0,37	0,90	0,36	0,89
LB	N5	0,87	0,79	0,91	0,88	1,01	0,85	0,97	0,69	0,79	0,87	1,00	0,82	0,94
	N7	1,09	0,98	0,90	1,09	1,00	1,06	0,98	0,86	0,79	1,07	0,99	1,01	0,93
	N8	0,71	0,64	0,90	0,73	1,02	0,69	0,97	0,56	0,78	0,77	1,08	0,68	0,95
	Σ (x-1)/n	→		-0,09		-0,03		-0,01		-0,20 C		0,06		-0,054
	√Σ (x-1)2/n	→		0,03		0,02		0,01		0,08		0,04		0,019
	Σ (x-1)/n	→		-0,08		0,05		0,02		-0,17		0,04		-0,027 *
	√Σ (x-1)2/n	→		0,02		0,03		0,02		0,05		0,03		0,013
concrete														
S	4	0,57	0,53	0,93	0,52	0,92	0,55	0,97	0,60	1,06	0,72	1,27	0,58	1,03
M	4	0,83	0,81	0,98	0,81	0,98	0,85	1,02	0,90	1,08	0,91	1,10	0,86	1,03
L	5	0,80	0,86	1,08	0,85	1,07	0,83	1,04	0,76	0,95	0,78	0,98	0,82	1,02
	Σ (x-1)/n	→		0,00		-0,01		0,01		0,03		0,11		0,028 *
	√Σ (x-1)2/n	→		0,04		0,04		0,02		0,04		0,09		0,023 D
total														
	Σ (x-1)/n	→		-0,06		0,04		0,02		-0,13		0,05		-0,017
	√Σ (x-1)2/n	→		0,02		0,02		0,01		0,04		0,03		0,012
	MEAN(steel, concrete)				Σ(x-1)/n		A							0,000 A
	MEAN(steel, concrete)				√Σ(x-1)2/n		B							0,018

A : 0.000
C : - 0.20
D : 0.055

Table. 7. Resume of α-values for steel and concrete bridges for the second set of γ-factors.

As the comparing values A and B were no longer decisive for this situation two new criteria were formulated:

$$C = \frac{\Sigma(\alpha' - 1.0)}{n_{adm}}$$

the maximum deviation from zero of the national value where n_{adm} = number of calculations carried out using national code for a particular material.

$$D = \left| \frac{\Sigma(\alpha' - 1.0)}{n_c} - \frac{\Sigma(\alpha' - 1.0)}{n_s} \right|$$

the minimum deviation from zero between steel and concrete results where
n_c = number of calculations for concrete
n_s = number of calculations for steel.

Table 8 shows the different values for the criteria A, C and D as obtained for a number of sets of safety factors.

ACTION		PARTIAL SAFETY FACTORS			
		Set 1	Set 2	Set 3	Set 4
Permanent Action	Self Weight	1.20	1.35	1.35	1.35
	Ballast	1.80	1.80	1.80	1.80
	Prestress	1.00	1.00	1.00	1.00
Variable Traffic Action	Load Model 71	1.50	1.43	1.40	1.45
CRITERION A		+0.001	+0.000	-0.013	0.008
CRITERION C		-0.21	-0.20	-0.19	-0.21
CRITERION D		+0.073	+0.055	+0.052	0.056

Table. 8. Values obtained for criteria A, C and D.

Sets 2 to 4 give almost the same results. Subcommittee bridges of UIC decided to adopt set number 4 having practical figures and being on the safe side compared to the second set:

γ_{G1} = 1.35
γ_{G2} = 1.80
γ_{pr} = 1.00
γ_{Q} = 1.45

This set has also been included in part 3 of Eurocode 1 [1].

6. Conclusions

On basis of the research executed by this working party the following can be concluded.

A set of safety factors for railway loading was found which gives very good compatability with the bridge design commonly used in Western Europe.

Chosing a self-weight factor in accordance with earlier Eurocodes leads to a life-load factor which is only slightly higher than the self-weight factor.

7. Recommendations

On basis of the work done by the working party the following recommendations can be made.

A probabilistic research to justify the proposed safety factors is needed.
During this research special attention should be paid to the self-weight factor of the bridges.

Execution of more comparative calculations and exchange of the results between the railway organisations will be of great help to evaluate the draft codes during the ENV period.

More research should be done on composite bridges.

8. Acknowledgement

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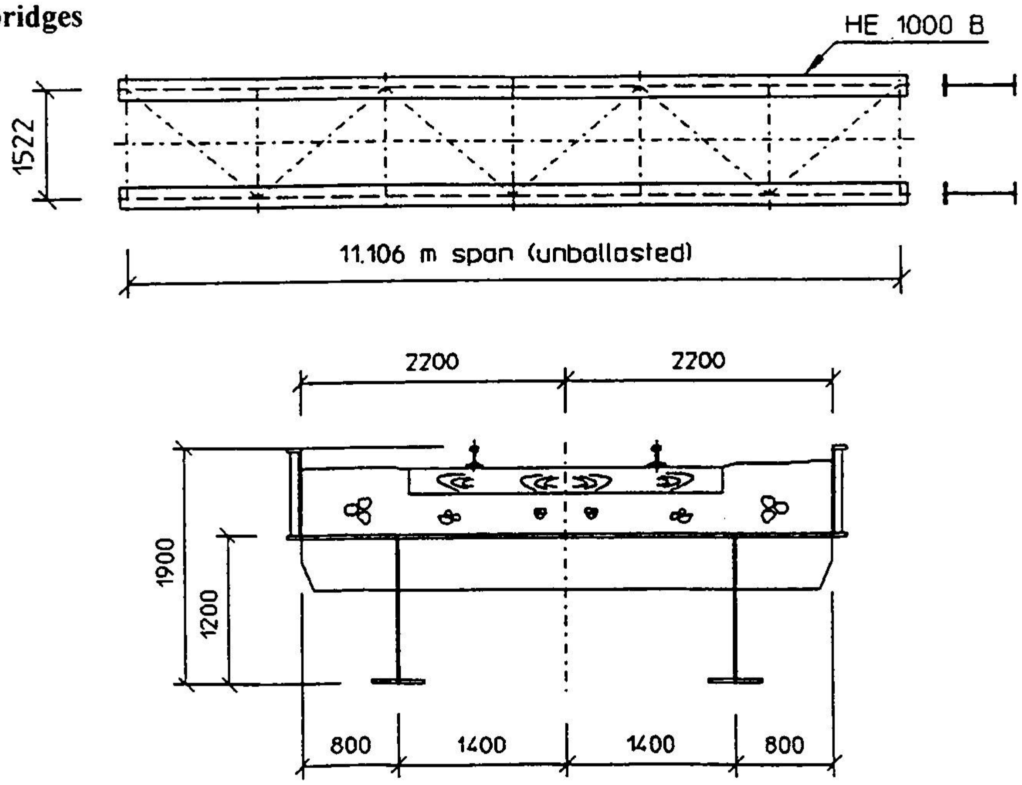
9. References

[1] ENV 1991-3, Eurocode 1: Basis of design and actions on structures, Part 3: Traffic loads on bridges, CEN/TC250/SC1

[2] UIC working party - Safety factors - Final Report, May 1994.

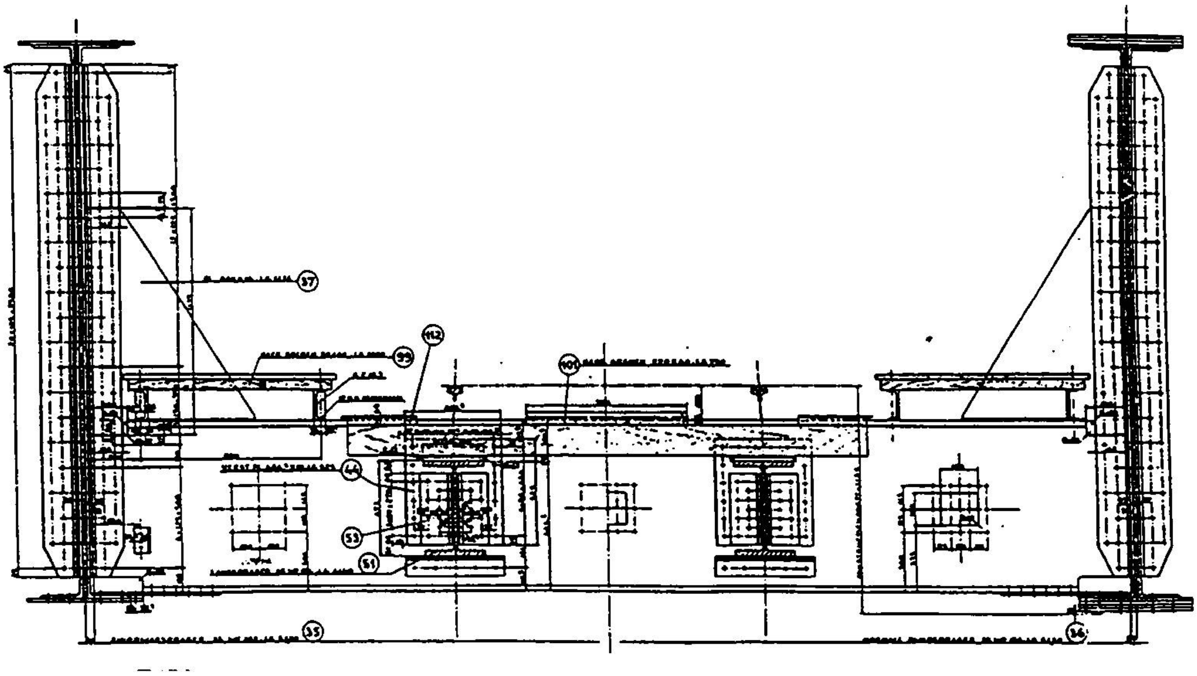
Annex 1

steel bridges



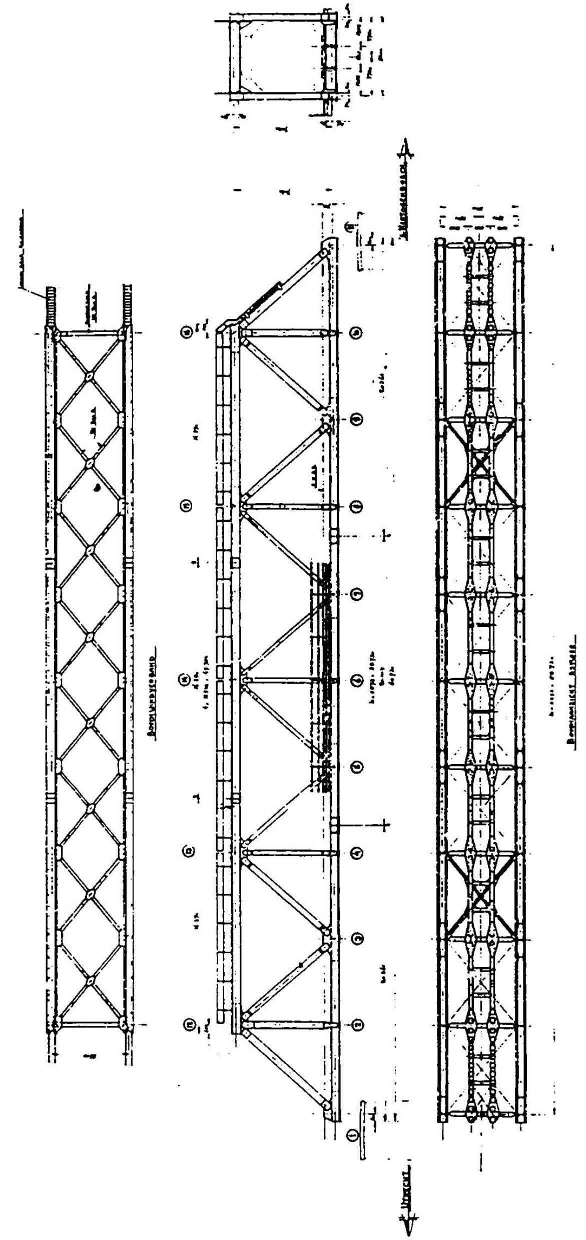
a

11106 m span typical section for a ballasted bridge



b

29.4 m span ballasted + unballasted

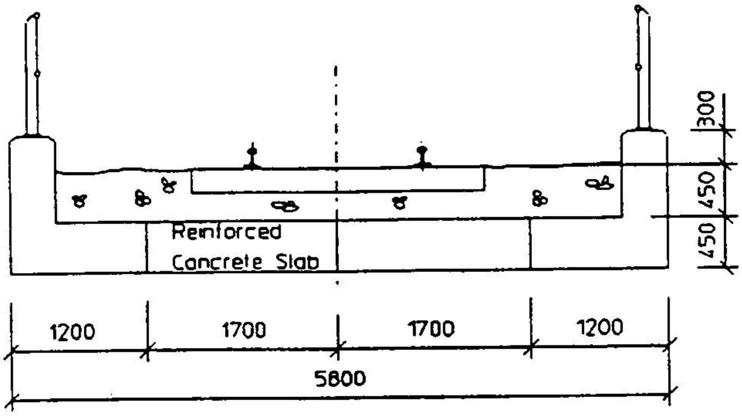


c

59.73 m span ballasted + unballasted

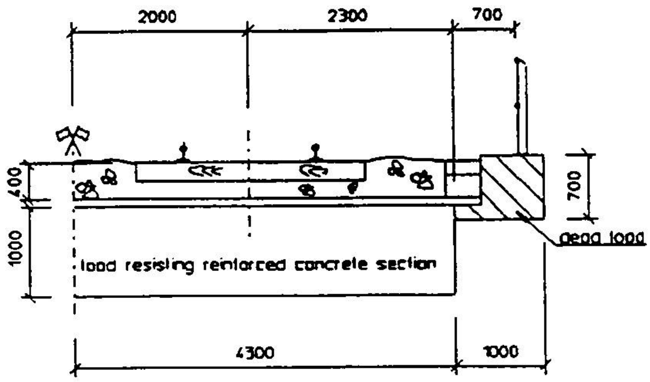
Annex 2

concrete bridges



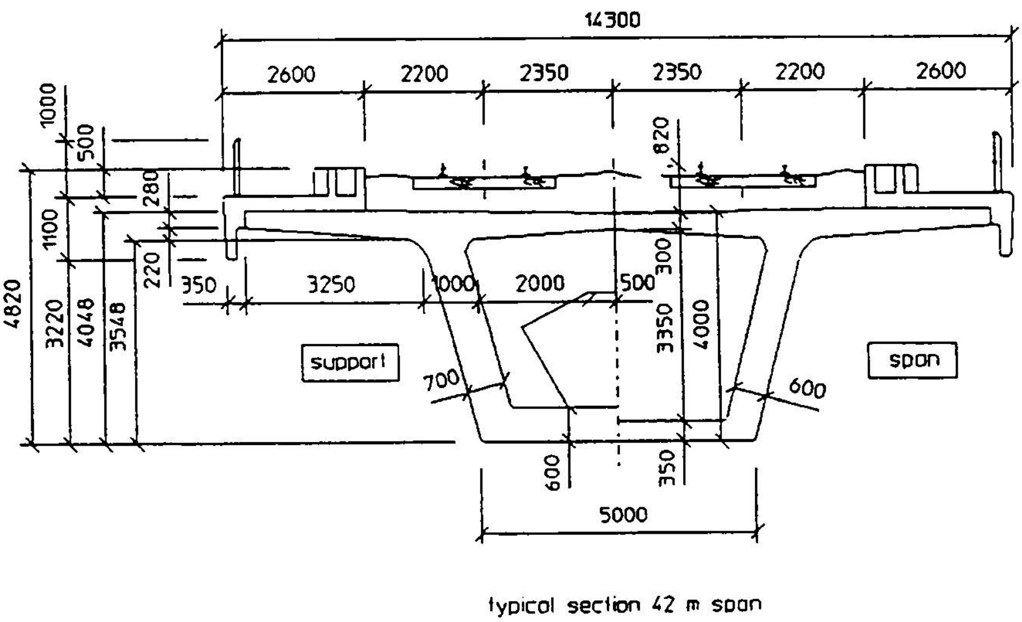
5 m span typical section

a



15 m span typical section

b



c

typical section 42 m span