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Recommendations for the design of joist-in-concrete railway bridges

International Union of Railways

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Leaflets to be classified in volume

VII - Ways and works

Amendments

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Introductory remarks:

The double lines (||) at the margin indicate amendments which became effective on the date given at the foot of the page.

The date on which the present leaflet becomes effective is indicated by the entry under "Application" at the end of the document.

Note

This Leaflet forms part of a set which also includes:

Contents

1 - General - Field of application - Regulations

1.1 - General

These rules relate to European Standards or Eurocodes, for definition of the effects of rail traffic and of the materials used.

Other standards or rules for the materials may be used if the safety factors are validated(1).

Joist-in-concrete decks consist of rolled steel beams encased in concrete.

The following calculation rules are only appropriate for decks with filler beams encased in concrete, which are constructed according to the requirements given in 1.3.

The calculations for the transverse bending and the torsion of slabs with encased filler beams should be based on the rules applicable to reinforced concrete railway bridges supplemented by the specific rules given in this document.

1.2 - Field of utilisation

Steel joist-in-concrete decks may be used for railway bridges with a span of up to roughly 30 m for simply supported beams and roughly 35 m for continuous beams.

They are of shallow depth and are well suited for spans where the construction depth is limited.

The prefabrication of small decks is possible.

1.3 - Requirements

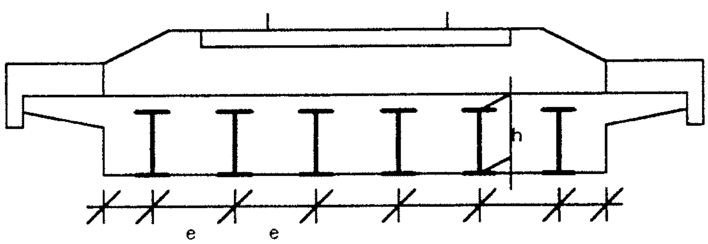

Figure 1

(1) This applies particularly to Appendix K to ENV 1994 part 2 concerning the filler beam decks. The principle rules of this appendix, which are consistent with the present rules, were worked out in concert with the UIC working group.

1.3.1 - Steel beams

The decks are constructed using plain, unstrengthened rolled beams. Specific mechanical shear connection means need not be provided. The steel must be cleaned to remove mill scale.

The web of the beam must be drilled to accommodate the transverse reinforcement and the fixing of spacers.

The clear distance between the flanges of adjacent beams must not be less than 15 cm. The distance e between the centre-lines of the beams should not exceed the following values :

$e \leq 75$ cm

$e \leq h/3 + 60$ cm

with : h = nominal beam depth in [cm]

The use of welded beams is allowed, when their dimensions are not different from the dimensions of the equivalent rolled beams.

1.3.2 - Concrete cover

The thickness of concrete above the upper flanges must be 7 cm minimum; it must not exceed one-third of beam depth, with a maximum of 15 cm.

The concrete cover must also satisfy the rule concerning the position of the neutral plastic axis given in point 5.1.2

1.3.3 - Stability of beams during construction

Spacers should be provided between the beams in order to facilitate their setting up.

Bracing is provided to prevent beams from lateral - torsional buckling during concreting. It consists of transverse bracing at supports which maintains beams in a vertical position. Additional intermediate transverse bracing and plan bracing may be needed. In order to reduce the number of braces, the concreting may be carried out in more than one stage. Appropriate checks must be made at the calculation stage to ensure the safety of such staged construction.

Welding must not be used for attachments in order to avoid any risk of fatigue.

1.3.4 - Permanent formwork

Shutter plates placed on the lower flanges of the beams and bedded on cement mortar or on a rubber strip may be used as permanent formwork. The thickness of this formwork depends on its span and the depth of the concrete. Its ability to carry the weight of the wet concrete must be verified.

The detailing and fixing of this formwork must be carefully considered in order to reduce the risk of corrosion between the plate and the flange of the beam.

The formwork does not contribute to the resistance or to the stiffness of the structure.

1.3.5 - Reinforcing steel

1.3.5.1 - Transverse reinforcement in the lower part of the slab

In order to resist torsion and transverse bending transverse reinforcement should be provided in the lower part of the slab over the whole width of the deck.

This reinforcement, which passes through the web above the root radius between the web and the lower flange, must be fully anchored in the concrete, beyond the edge beams, (anchorage by hooks) or by bolting to the webs of the beams.

1.3.5.2 - Transverse reinforcement in the upper part of the slab

For this reinforcement, which must also provided over the whole width of the deck, the cross sectional area shall be equal to at least half that of the lower transverse reinforcement, and shall, in any case, be not less than 5 Ø 10 mm diameter high bond bar per meter.

This reinforcement shall be prevented from contact with the upper flanges of the beams. Spacers or longitudinal reinforcement can be used to ensure this.

The vertical reinforcement of the free ends should be extended in order to encircle efficiently the whole slab, together with the upper and lower transverse reinforcement.

1.3.5.3 - Longitudinal upper reinforcement

Where the concrete is in tension in the upper part of the slab, reinforcement shall be provided longitudinally in order to limit crack width (see 4.4). This reinforcement can be taken into account in determining the resisting bending moment at the intermediate supports of continuous bridges (see 5.1).

1.3.5.4 - Longitudinal lower reinforcement

In order to limit the width of cracks in the concrete tension zone, longitudinal lower reinforcement may be provided if the spacing of the beams is large. This reinforcement shall be not less than 5 Ø 10 mm diameter high bond bar per metre.

1.3.6 - Skew

The effects of the skew angle of the decks must be taken into account in the calculations when the skew angle of the supports is lower than 70 gon.

1.3.7 - Beam supports

Each rolled beam should be supported individually on bearings, on bearing blocks at the piers or abutments. This facilitates the concreting and ensures a direct transfer of the deck loads to the supports.

If other support modes are considered, the transfer of the loads shall be carefully studied and justified in the calculations.

2 - Materials

2.1 - Steel of the beams

The grades and qualities of the steel for the beams shall be conforming to European Standard EN 10025 "Hot-rolled products of non alloy structural steels" and EN 10113 - parts 1 to 3 "Hot-rolled products in weldable fine grain structural steels".

The recommended[1] grades and qualities for rolled beams used in railway bridges if not spliced by welding are underlined in the following table. The recommended grades and qualities for rolled beams used in railway bridges with welded joints, are indicated in bold type in the following table.

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(1) These recommendations assume that the price differences between different grades and qualities are small and marginal with respect to the cost of the construction of railway bridges on lines open to traffic. For the same reason, the use of steel of lower quality is not recommended, when steel with higher grade is chosen.

The grades and qualities indicated in italics in the following table may be used experimentally, in particular for large spans. The bond between concrete and steel shall be checked beforehand under repeated loading, by tests or calculations [(1)] .

	European Standard EN 10025 : 1993		European Standard EN 10113-1 : 1993 EN 10113-2 : 1993 EN 10113-3 : 1993	
Nominal yield strength	**Grade**	**Quality**	**Grade**	**Quality**
240 Mpa	S 235	JR,J0		
275 Mpa	S275	J0	**S 275**	N, **M** NL, **ML**
355 Mpa	S 355	J0, **J2G3** **K2G3**	**S 355**	N, **M** NL, **ML**
420 Mpa			*S 420*	*N, M* *NL, ML*
460 Mpa			*S 460*	*N, M* *NL, ML*

The values for the yield strength to be adopted as a function of the thickness are given in the above-mentioned two European Standards.

2.2 - Reinforcing steel

Concrete reinforcement must be in accordance with the requirements of section 3.2 of ENV 1992 part 2, concrete bridges (Eurocode 2).

Only steels with high ductility B500H are to be used[(2)].

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(1) Within the scope of the studies for the elaboration of Eurocode n° 4, tests have been carried out on composite beams with encased webs. They have shown good bending performance up to the ultimate limit state, with or without shear connectors and using steel with a yield strength close to 450 N/mm².

(2) According to ENV 10080

2.3 Concrete

Concrete must be in accordance with the requirements of section 1 of chapter 3 of ENV 1992, (Eurocode 2) which are summarised as follows.

Strength class of concrete	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50
f_{ck} (N/mm^2)	20	25	30	35	40
f_{ctm}(N/mm^2)	2.2	2.6	2.9	3.2	3.5

f_{ck} : characteristic value of the concrete compressive strength measured on a cylinder

f_{ctm} : mean value of the tensile strength

According to point 7.3.1.1 of ENV 1992 (Eurocode 2), the classification of concrete involves two values, for example, C 20/25; the first value refers to the characteristic cylinder strength, (20N/mm^2 in the example), the second to the cube strength (25 N/mm^2 in the example).

For reasons of durability of the bridges, the recommended concretes for railway bridges with beams or rolled girders encased in concrete are from class C30/37.

In the case of exceptional use of lightweight concrete the complementary recommendations of Appendix 5 are applicable.

2.4 - Modular ratio

2.4.1 - The modular ratio steel-concrete for short - term loading is defined as the ratio :

$$n_i = \frac{E_a}{E_i}$$

where :

E_a = modulus of longitudinal elasticity of the beam steel,

E_i = secant modulus of elasticity of the concrete, for short- term loading.

For normal weight concrete the following value should be adopted:

$$n_i = 6$$

2.4.2 - The long term modular ratio, which takes into account the shrinkage of the concrete, is defined by the expression :

$$n_V = \frac{E_a}{E_i} (1 + K_{f1})$$

If the value of the coefficient K_{f1} is not otherwise specified, K_{f1} shall be taken as equal to 2 in the case of normal weight concrete and the long term modular ratio is then equal to 3 times the short term modular ratio :

$$\text{Thus } n_V = 18$$

2.5 - Shrinkage of concrete

Shrinkage of the concrete is not considered.

2.6 - Poisson's ratio

The effective Poisson's ratio (or average) shall be taken equal to 0 for the calculations for ultimate limit states and 0.2 for the calculations for serviceability limit states.

The same value may be used for both orthogonal directions of filler beam slabs.

2.7 - Properties of filler beam decks for dynamic calculations

2.7.1 - Dynamic calculations

The stiffness and the mass of the deck change during the life of the bridge. For each of the different parameters, which determine the dynamic behaviour of filler beam slabs, two values have to be taken into account in the calculations, the highest and the lowest value, applying the appropriate value in each situation in order to take into account the most unfavourable case.

The minimum flexural stiffness is evaluated by assuming that concrete in tension is cracked for all the cross sections under the action of the permanent loads and variable traffic loads and by taking into account the short term modulus of elasticity of the concrete (short term modular ratio given in 2.4.1).

The maximum stiffness is determined by assuming that the whole concrete in tension is uncracked.

The stiffness may be considered as constant along the beam.

The mass of the deck, as well as the rotational inertia of mass should be evaluated using both the maximum and minimum values, for the depth of ballast on the deck.

The damping of the structure may be taken as equal to 2% of the critical damping(1).

2.7.2 - Simplified calculations

In view of the application of 6.4.3 of ENV 1991-3 : 1994, the first natural frequency of the flexural vibration of the structure may be calculated using the following assumption: a constant moment of inertia determined as indicated in 3-4-2, the modular ratio corresponding to the short-term loading and the mass taken as equal to the minimum nominal value given above.

3 - Loads and other factors

3.1 - Constant loads

The constant loads comprise the following:

a) the weight of the steel beams, the permanent formwork, the reinforcement, the cross bracing, windbracing and spacers, and encasing concrete,

b) the weight of the protective layer, waterproofing, the track and its components. This weight can vary during use. For these loads, reference should be made to ENV 1991-2-1, sections, 3.4.3 et 3.4.4.

c) the imposed permanent deformations, induced settlements and/or support settlements.

Where this is not the case, the loads listed under a) should be applied to the steel beams alone, whilst the actions b) and c) should always be applied on the composite structure.

The load a) can be applied to the composite structure, after hardening of concrete, provided that the beams are supported along their whole length during concreting and the supports remain in place control the concrete has set.

The weight of the elements constituting the deck is calculated using the nominal dimensions assuming a unit mass of 7.85 ton per m^3 for the steel (beams, reinforcement, spacers,...), of 2.4 ton per m^3 for the normal weight encasing concrete, and of 2.5 ton per m^3 for the normal weight reinforced concrete (parts in cantilever...).

(1) The corresponding logarithmic decrement is equal to 0.126.

3.2 - Railway loads

The railway loads are given in section 6 of part 3 of ENV 1991-3 : 1994.

3.3 - Other variable loads

The loads due to wind action are given in the ENV 1991-2.4. The combination with the railway loads are defined in ENV 1991-3 : 1994.

The loads due to aerodynamic effects from passing trains are given in part 6 of ENV 1991-3 : 1994.

The effects due to the temperature do not need to be taken into account in designing the deck, (but they must be considered for the bearings). The effects due to snow also do not have to be considered.

3.4 - Calculation of internal forces and deformations

3.4.1 - Calculation models

The internal forces and the deformations of the structure are calculated by considering a linear elastic model.

The deck may be modelled as a thin orthotropic plate possibly skewed. The calculation of the internal forces can then be made using the tables of Guyon-Massonnet-Barès or with the help of finite element software. The utilisation of software is necessary in the case of decks with a skew angle less than 70 gon.

The modelling of the deck as single beams which may be calculated by classical theory of bending is allowable, if the transverse bending or the torsion of the structure can be taken into account by means of the transverse reinforcement. This is the case for the decks carrying only one track and with a skew angle higher than 70 gon, where the transverse lower reinforcement consists of high bond steel bars with a spacing of 300 mm, and with a diameter of 16 mm, when the span is greater than 5 m, and with a diameter of 20 mm, when the span is lower than 5 m.

3.4.2 - Calculation of internal forces and deformations under longitudinal bending

The check of the longitudinal bending should determine first the beams with the highest internal forces and then calculate the most unfavourable bending moments in the cross sections subjected to the highest internal forces.

Internal forces and deflections may be determined assuming a moment of inertia with a constant value along the structure.

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For loads applied after complete hardening of concrete, the moment of inertia I is determined from the following assumptions:

- I is equal to the mean value:

$$I = \frac{I' + I''}{2}$$

where:

I' = moment of inertia of the composite cross-section (equivalent section) subjected to sagging bending, calculated neglecting concrete in tension.

I'' = moment of inertia of the composite cross-section (equivalent section) subjected to sagging bending, calculated assuming that concrete is uncracked and participates in resistance.

- The width of the concrete associated with each steel beam is equal to their spacing e.

- The longitudinal reinforcement must not be taken into account.

- The modular ratios to be used are those indicated above.

The deflections, which are used in determining the fabrication camber must be calculated in accordance with the above-mentioned rules.

4 Serviceability limit states

4.1 - General

The serviceability limit states to be considered are given in the section 6 of ENV 1991-3 : 1994.

In addition to the deformation limit states following limit states should be checked: limitation of stresses in the materials and of crack width.

The combination of actions to be considered for the serviceability limit states are given in parts 1 and 3 of ENV 1991 : 1994.

4.2 - Deformations

To check the serviceability limit states of deformations, these must be evaluated by using the rules given above in section 3.4.

4.3 - Limitation of stresses

4.3.1 - Verification principle

This limit state is aimed at verifying by calculation of the stresses in the materials, that neither the concrete in compression, nor the structural steel suffers irreversible deformations under any possible load combination.

The maximum total stresses for each construction phase and in service shall not exceed, at any point in the structure, the limit stresses of the materials given hereafter in 4.3.3.

The stresses due to the permanent loads applied before the concrete has fully hardened must be calculated considering that only the steel beams are carrying the loads.

The stresses due to the permanent loads acting on the structure after all concrete has fully hardened must be calculated considering the resistance of the composite section with the long term modular ratio given in 2.4.2.

The stresses due to the variable loads are calculated also considering the resistance of the composite section but with the short term modular ratio given in 2.4.1.

4.3.2 - Sections to be taken into account for the stress calculations :

For the interior beams of the slab, the width of concrete contributing to the composite bending action should be taken as equal to the distance between the centre lines of the beams.

For the edge beams, this width should be taken as equal to half the distance between the centre lines of two adjacent beams plus the distance between the axis of the web and the vertical exterior face of the slab but not greater than half the distance between the centre lines of the beams.

At the cross sections subjected to a sagging bending, the stresses must be calculated by considering that the concrete in tension is cracked and does not provide resistance.

The cross sections subjected to a hogging bending, the stresses must be calculated assuming that the concrete in tension is cracked and does not provide resistance, but taking into account the tensile reinforcement situated above the rolled beams.

4.3.3 - Stress limits of materials

4.3.3.1 - Structural steel

The stress in structural steel σ_a shall not exceed f_y / 1.15.

Taking into account the presence of holes in the web for the transverse reinforcement, the maximum horizontal shear stress τ_h in the steel, calculated using the net section and assuming that all the shear force is supported by the web of the beams, shall not exceed :

$$\tau_h \leq 0.45\ f_y$$

If there are normal stresses σ at the level of these holes, the following expression must be satisfied:

$$\sigma^2 + 3.7\ \tau_h^2 \leq (\sigma_a)^2$$

Neither of these two conditions have to be verified, if the diameter of the holes is less than or equal to 50 mm and if the distance between the holes of the beams is greater than or equal to 300 mm.

4.3.3.2 - Concrete

The maximum compressive stress in concrete shall not exceed 0.6 fck.

The tensile stress in concrete is not limited.

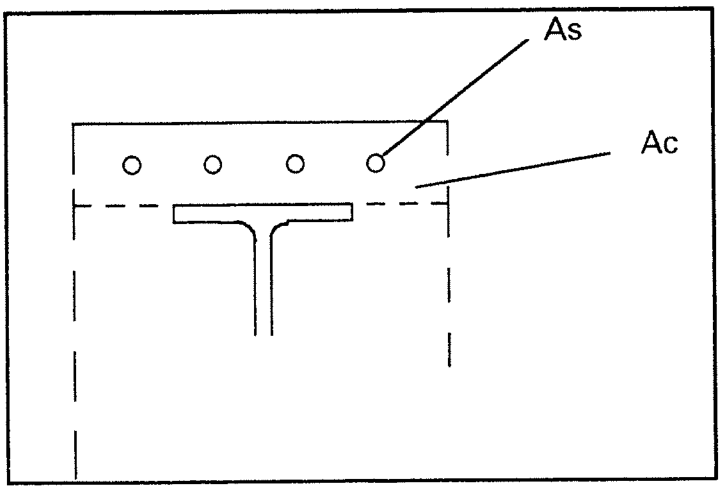
4.3.3.3 - Reinforcing steel

The stress in the longitudinal tension reinforcing steel shall not exceed fy / 1.5.

4.4 - Limitation of crack widths

In the top of the slab, where there is continuity at intermediate supports, a minimum longitudinal reinforcement is required in order to limit crack widths. The area As of the reinforcement to be provided must be determined from the requirements of ENV 1994-2 applicable to composite bridges.

In applying the requirements of ENV 1994-2, the area of the concrete Ac to be taken into account is the area of concrete in tension situated above the upper flanges of the steel beams.



$As \geq 0.65\ Ac \times f_{tc} / \sigma_s$ with:

- f_{tc}: concrete tensile strength, at the moment when cracking is assumed to occur,

- σ_s: stress in the reinforcement steel given in the following table (in Mpa), depending on the allowable value F of crack width :

ϕ_s (mm)	10	12	16	20	25
F = 0.3 mm	360	320	280	240	200
F = 0.2 mm	280	240	200	180	160

The diameter ϕ_s to be taken into account in the table is equal to $\phi_r \times 2.5 / f_{tc}$ (where ϕ_r is the diameter of the provided bar and f_{tc} is the tensile strength of the concrete in MPa.

The maximum allowable width of cracks F must be specified by the relevant authority. It must not be higher than 0.3 mm for railway bridges.

4.5 - Check of dynamic behaviour of filler beam decks used for high speed trains

The criteria given in part 6 of ENV 1991-3: 1994 for the dynamic behaviour of joist-in-concrete decks must be observed.

The deformations must be calculated in accordance with 2.7.

5 - Ultimate limit states

5.1 - Ultimate limit state of resistance

5.1.1 - Combination of loads to be taken into account

The load combinations to be taken into account in the checks at the ultimate limit states are given in Appendix G to ENV 1991-3 : 1994.

The only effects of loads, which must be taken into account at the ultimate limit state of resistance of the cross sections with joist-in-concrete decks are the bending moments. The main combination can be represented by the following expression :

$M_{Sd} = 1.35\ M_G + 1.45\ (\text{or } 1.2)\ M_Q + 0.9\ M_W$

where:

M_{Sd} = design value of the applied internal bending moment at the ultimate limit state,

M_G = bending moment resulting from permanent loads (except the settlements of the supports: in this case γ_g = 1.00),

M_Q = bending moment due to the railway traffic loads. The factor γ is equal to 1.45 for the loads due to load model UIC 71 and 1.2 for the loads due to load model SW/2,

M_W = bending moment due to wind loads $\Psi_0\ F_{wk}$ (or $\Psi_0\ F_{wn}$) given in ENV 1991-3: 1994, Appendix G.

5.1.2 - Verifications

At the ultimate limit state, the bending moments at the intermediate supports of the continuous bridges can be reduced by an amount $\Delta M = \frac{R \times a}{8}$

where:

- R is the reaction at the intermediate support,

- a = b + h where b = width of the bearing and h = resistant depth of the cross section.

The bending moments at the intermediate supports can also be reduced by imposed deformations. These imposed settlements can be introduced either by the means of an appropriate cambering of the beams before erection or by a vertical displacement at supports, after hardening of concrete.

It must be verified that at all cross sections the bending moment M_{Sd} [1] satisfies the following condition: $M_{Sd} \leq M_{Rd}$, where M_{Rd} is the design value of the resisting bending moment.

For simplification, the resisting bending moment M_{Rd} may be determined by using a birectangular stress distribution for both steel and concrete, taking into account the following stress limits for each material:

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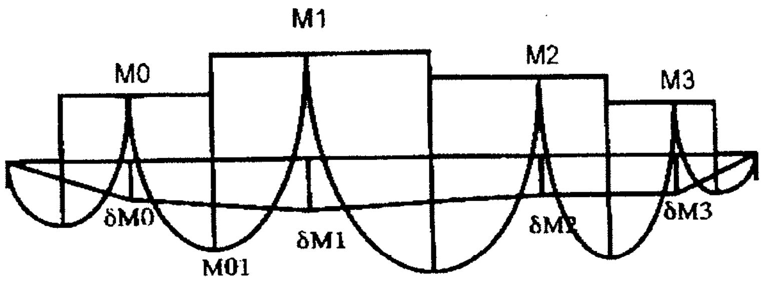
(1) For continuous bridges, the redistribution of the bending moments due to the inelastic behaviour of the materials at the ultimate limit state may be taken into account in the following conditions :

- a) the moments at supports shall not be reduced in order to take into account the width of the bearing,
- b) the cross sections of the steel beams shall be in class 1, as defined in ENV 1994-1-1 : 1992.
- c) the reduction of the moment at support δM_i shall not exceed 15%; thus it is only applicable when

$M_{Rdi} \leq M_{Sdi} \leq \frac{M_{Rdi}}{0,85}$, where M_{Rdi} is the ultimate resisting moment at the support "i",

- d) the moments within spans must be increased, so that internal forces and moments are in equilibrium with the loads.



M01 (after redistribution) = M01 + (δM0+δM1)/2

- in concrete, the limit compressive stress is equal to $\lambda.f_{ck}/1.5$ under traffic loads and $\lambda.f_{ck}/1.15$ for the load combination in situation. λ is taken equal to 0.85 for normal-weight concrete,

- in tension, the concrete strength is assumed to be equal to zero,

- in the steel used for the beams, the stress limit is f_Y / γ_a , where: $\gamma_a = 1.1$(1).

- in the reinforcement, the stress limit is $f_Y / 1.15$.

The plastic neutral axis must be located within the limits of the depth of the steel beam.

The effect of the shear force is not taken into account for the calculation of the resisting bending moment.

The resistance (in tension or in compression) of the permanent formwork (plates in fibre-cement, etc.) is not taken into account.

The formulae to be used in the calculation of M_{rd} are given in Appendix 1.

5.2 - Other ultimate limit states

The resistance and the elastic stability (lateral torsional buckling during concreting, ...) of the beams must be verified in accordance with the requirement of ENV 1993 and ENV 1994 for all situations where concrete has not completely hardened..

Where there is a risk of lateral buckling of the beams during the concreting, the following requirements must be met:

- a) calculation of the design buckling resistance moment of the single beams which are supposed to be laterally unrestrained, except at supports,

- b) evaluation of the maximum depth of wet concrete which corresponds to this resistance moment,

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(1) The value 1.1 is based on safety. It is given as indication only. The specification of the definitive value of this safety factor for each limit state is the responsibility of the relevant authority.

- c) determination of the concreting conditions:

  - c1) If the calculated depth is greater than or equal to the depth of the deck, the concreting can be executed in one stage,

  - c2) If the calculated depth is lower than the overall depth but exceeds 15 cm, concreting must be executed in several stages, the concrete depth for the first phase should not be higher than the calculated depth. The concrete of the first phase may be taken into account for the stabilisation of the beams in the next concreting phases, if concrete has hardened for more than 24 hours.

  - c3) If the calculated depth is less than 15 cm, the beams must be provided with intermediate lateral restraints (vertical bracing, plan bracing, etc.). These must be designed, to increase the resistance moment indicated in a) and to resist the destabilisation forces. The maximum depth of wet concrete must then be again evaluated.

The elastic stability (lateral buckling) of the beams must also be verified for the erection stage.

The other ultimate limit states to be considered are the limit states for static equilibrium (particularly for the operations of launching or erection of the decks). These limit states must be verified in accordance with ENV 1993 and ENV 1994.

6 - Fatigue limit state

6.1 - Beams without welds

No verification of the fatigue behaviour of the beams is required. But, when the steel is damaged, either during the erection, during flame cutting or during the drilling of holes, repairs are to be carried out and fatigue checks can be required.

6.2 - Beams with welds

All welded details must be verified in accordance with section 6 of ENV 1991-3 : 1994 using the fatigue strength categories indicated in ENV 1993 - 1 (Eurocode 3, part 2).

But welded splices of rolled beams do not need to be verified, when executed in accordance with the rules indicated in chapter 7, and when located at cross sections where bending moment under permanent loads is equal to zero. Splices must be staggered, so that at most half of the beam splices are situated in one given cross-section of the bridge deck. The longitudinal distance between the welded splices of adjacent rolled beams must be at least 1 meter.

The stress ranges must be calculated under the 71 load model as indicated in the point 4.3

6.3 - Reinforcement

The longitudinal reinforcement at the intermediate supports of continuous bridges must be verified in accordance with ENV 1992, part 2, for concrete railway bridges(1)

The stress ranges under the 71 load model are calculated as indicated in point 4.3.

7 - Execution of filler beam decks

The execution of joist-in-concrete bridges must conform with the requirements of chapters 9 "Execution" of part 1 of ENV 1993 (EC3) and 1994 (EC4), taking into account the additional requirements included in the relevant parts 2 of those Eurocodes, which relate to bridges.

7.1 - Fabrication and finishing of the beams in the factory

The rolling and finishing of the beams must be carried out in accordance with current standards.

(1) Test calculations carried out for two-span continuous bridges have shown that, in usual situations, these verifications do not lead to any increase in the area of reinforcement designed to section 4.4.

Corrosion protection is only required for the bottom flanges of the beams (soffit, upper surface, root radii and edges). It is forbidden for the rest of the beam[1].

The web/flange welds of welded beams used for filler beam decks are not visible after concreting. Therefore the methods applicable to steel railway bridges for the production and control of welds which cannot be inspected are to be applied. These particular requirements can be:

- automatic welding with guaranteed full penetration of the welds,

- 100 % checking of the welds.

7.2 - Transport and erection of the beams

The transport and erection of the beams must be carried out in such a way as to guarantee particularly the absence of any risk of lateral buckling or irreversible deformation.

7.3 - Execution of butt welded splices of rolled beams

Welded splices of rolled beams must be carried out in accordance with the requirements of Appendix 2.

All precautions normally taken to ensure that the welded joints are without defect, must be observed without exception: the welding procedure, the welders and operators must be appropriately qualified, a welding procedure has to be established. The integrity of the welds must be verified.

a) Test joint

A test joint is not necessary if the welding procedure has previously been approved for another bridge.

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(1) Even though experience shows that limited accidental application of thin paint coatings outside of the area to be protected does not harm the adhesion of the concrete to the steel, it should be avoided.

When a test joint is necessary, it must be carried out using two 1 m long beams, both of which have been ordered at the same time as the beams themselves. This allows the welding procedure to be adjusted and in particular allows the measurement of the gap reduction in the joint due to the shrinkage of the weld.

Tensile and bending test-pieces must also be taken from this test joint, after radiographic or ultrasonic inspection, so that its mechanical characteristics can be tested.

b) Preparation of the welding site

Welding must be carried out under shelter from bad weather. It must be stopped if the temperature of beams to be welded falls below + 5°C. Welding may be done in cold weather as long as the sheltered welding site is sufficiently heated and the temperature of beams to be welded is kept above + 5°C.

The edge of beam or girder joints to be welded must be prepared and bevelled: double V weld for the web and single V weld for flanges with a thickness lower than or equal to 15 mm and double V asymmetrical (2/3 of thickness over and 1/3 under) for the flanges with a thickness greater than 15 mm.

The two beams to be welded must be supported on rollers allowing the free displacement of the beam ends, in order to minimise residual stresses due to restraint of the welds. The girders can be supported in a "rocking situation", as indicated in the appendix n°2. The opening of V or double V asymmetrical welds of the upper flange may be calculated from the preliminary test or from the qualification of the welding procedure.

c) Welding procedure

Work must be carried out by two welders working on both sides of the beam simultaneously. In the case of flanges with a thickness less than or equal to 15 mm, temporary backing bars can be fixed under the top and bottom flanges to avoid welding in overhead position. These backing bars and any run-off plates provided to develop the full length of welds must not be attached to the beams by tack welds.

Welding of flanges is performed in flat or overhead position without turning over; web welding is performed in vertical position (going upwards). Welding is carried out using several runs.

Welding is generally carried out in the following order:

- welding of bottom and top flange,
- welding of web.

If beams are spliced in a « rocking situation » the order is as follows:

- welding of bottom flange
- welding of web
- welding of top flange

d) Inspection of welds

All joints must be visually examined with dye penetrant test if necessary.

At least 10% of the number of joints must be inspected by radiographic or ultrasonic test. If these tests reveal defects falling outside the tolerances of standards concerning the execution of welded joints, further examinations may be made.

7.4 - Reinforcement of filler beam decks

An example of the arrangement of reinforcement is given in Appendix 4.

7.5 - Concreting of filler beam tests

The concreting procedure must be verified by calculations as indicated in point 5.2.

Care must be taken to ensure the necessary depth of concrete and the wet concrete must be uniformly distributed around the beams. It is recommended that concreting is stopped momentarily approximately 15 cm below the level of the upper flange of the beam, in order to ensure good concreting and filling up under the upper flanges.

Other requirements relating to execution are given in ENV 1992 (part 2).

APPENDIX 1

Formulae for the mechanical characteristics of the cross sections

Calculation of the design value of the resisting bending moment of a filler beam cross section (M_{Rd})

*** Neutral axis in the web**

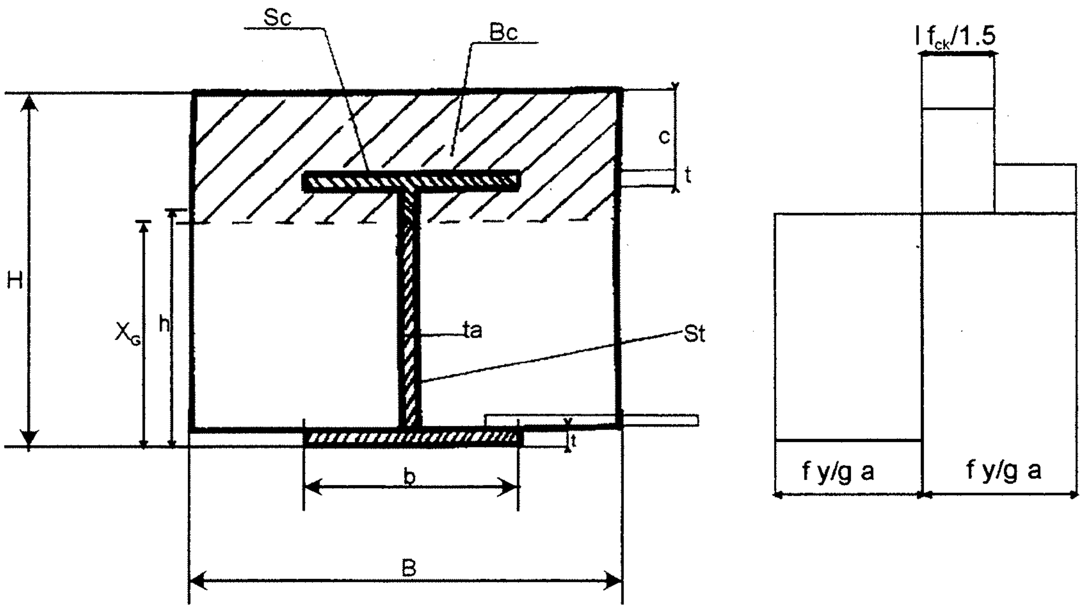


Figure 1

The ultimate moment is the maximum moment, which a cross section can resist before failure.

Calculation of M_{Rd}

Calculation of the position of the plastic neutral axis X_G :

For the equilibrium of the cross section the force resulting from the tension stresses in the beam must be equal to the sum of the forces resulting from the compression stresses in the steel and concrete.

$$F_{St} = F_{Bc} + F_{Sc}$$

$$F_{St} = \frac{f_y}{\gamma_a}\left[bt + t_a(x_G - t)\right]$$

$$F_{Sc} = \frac{f_y}{\gamma_a}\left[bt + t_a(h - t - {}_G)\right]$$

$$F_{Bc} = \frac{\lambda . f_{ck}}{1.5}\left[B\,(H - x_G) - t.b - t_a\,(h - t - x_G\right]$$

$$\frac{f_y}{\gamma_a}\left[bt - t\,t_a\right] + x_G\,\left[\frac{f_y}{\gamma_a}t_a\right] = \frac{\lambda f_{ck}}{1.5}\left[BH - tb - t_a(h-t)\right] + x_G\frac{\lambda f_{ck}}{1.5}\left[t_a - B\right] + \frac{f_y}{\gamma_a}\left[bt + t_a(h-t)\right] - x_G\,\frac{f_y}{\gamma_a}t_a$$

$$x_G = \frac{\frac{\lambda f_{ck}}{1.5}\left[BH - tb - t_a(h-t)\right] + \frac{f_y}{\gamma_a}t_a h}{\frac{\lambda f_{ck}}{1.5}\left[B - t_a\right] + 2\frac{f_y}{\gamma_a}t_a}$$

The resulting ultimate moment is the sum of the moments of these forces related to X_G :

$$M_{Rd} = F_{Sc}x_{FSc} + F_{Bc}x_{FBc} + F_{St}x_{FSt}$$

calculation of x_{FSt} : distance between the resulting F_{St} to x_G

This is the position of the centre of gravity of the tension zone of the steel.

$$x_{FSt} = \frac{\frac{t_a(x_G - t)^2}{2} + bt\,(x_G - t/2)}{t_a(x_G - t) + bt}$$

calculation of x_{FSc}: distance between the resulting F_{Sc} to x_G

This is the position of the centre of gravity of the compression zone of the concrete.

$$x_{FSc} = \frac{\frac{t_a(h - x_G - t)^2}{2} + bt\,(h - x_G - t/2)}{t_a(h - x_G - t) + bt}$$

calculation of x_{FBc}: distance between the resulting F_{Bc} to x_G

This is the position of the centre of gravity of the compression zone of the concrete.

$$x_{FBc} = \frac{B(H-h)(\frac{h}{2} - x_G + \frac{H}{2}) + t(B-b)(h - x_G - \frac{t}{2}) + (B - t_a)\frac{(h - x_G - t)^2}{2}}{B(H - x_G) - bt - t_a(h - x_G - t)}$$

The calculation method of MRd at the support is identical, but the longitudinal reinforcement in the upper part must also be taken into account.

*** Neutral axis in the upper flange of the beam**

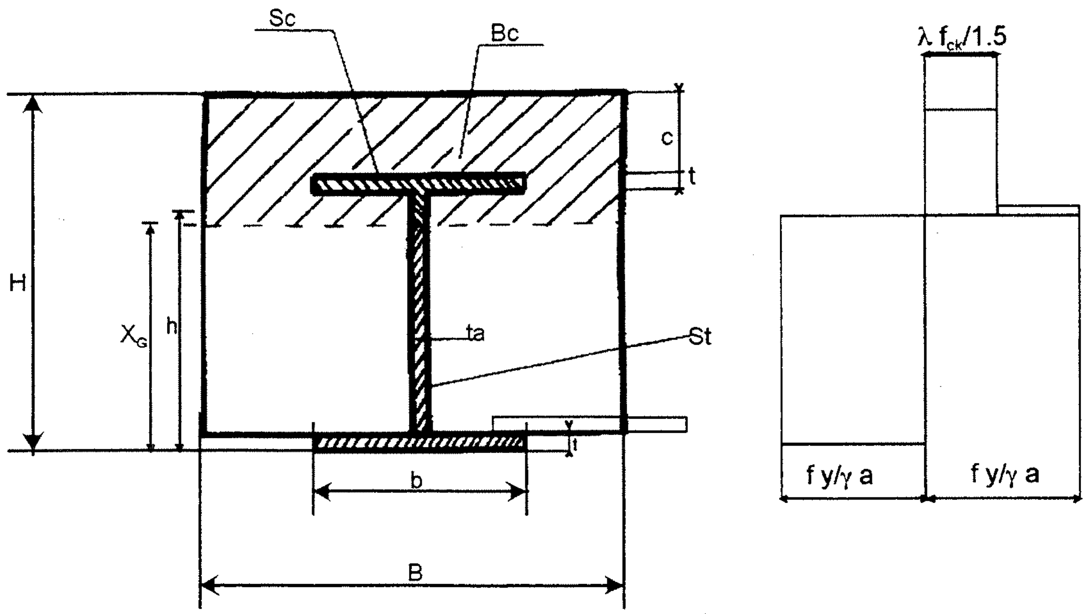


Figure 2

The ultimate moment is the maximum moment, which a cross section can resist before failure. This moment is obtained, when all the materials of the cross section become plastic.

Calculation of M_{Rd}

Calculation of the position of the plastic neutral axis x_G :

For the equilibrium of the cross section the force resulting from the tension stresses in the beam must be equal to the sum of the forces resulting from the compression stresses in the steel and concrete.

$$F_{St} = F_{Bc} + F_{Sc}$$

$$F_{St} = (f_y/\gamma_a)\ [bt + t_a(h - 2t) + b(t - h + x_G)]$$

$$F_{Sc} = (f_y/\gamma_a)\ [b(h - x_G)]$$

$$F_{Bc} = (\lambda\ f_{ck}/1.5)\ [B(H - x_G) - b(h - x_G)]$$

APPENDIX 1

$x_G[bf_y/\gamma_a]+(f_y/\gamma_a)[2bt-h(b-t_a)-2t.t_a] = -x_G(\lambda f_{ck}/1.5)[B-b]+(\lambda f_{ck}/1.5[BH-bh]-x_G (f_y/\gamma_a)b + (f_y/\gamma_a)bh$

$$x_G = \frac{(\lambda f_{ck} / 1.5)[BH - bh] + f_y / \gamma_a[2t.t_a + h(2b - t_a) - 2bt]}{(\lambda f_{ck} / 1.5)[B - b] + 2(f_y / \gamma_a)b}$$

The resulting ultimate moment is the sum of the moments of these forces related to x_G :

$$M_{Rd} = F_{Sc} x_{FSc} + F_{Bc} x_{FBc} + F_{St} x_{FSt}$$

calculation of x_{FSt}: distance between the resulting F_{St} and x_G

This is the position of the centre of gravity of the tension zone of the steel.

$$x_{FSt} = \frac{bt(x_G - t / 2) + t_a(h - 2t)[x_G - h / 2] + (b / 2)(t - h + x_G)^2}{bt + t_a(h - 2t) + b(t - h + x_G)}$$

calculation of x_{FSc}: distance between the resulting F_{Sc} and x_G

This is the position of the centre of gravity of the compression zone of the steel.

$$x_{FSc} = (h - x_G)/2$$

calculation of x_{FBc}: distance between the resulting F_{Bc} and x_G

This is the position of the centre of gravity of the compression zone of the concrete.

$$x_{FBc} = \frac{B(H - h)[(1 / 2)(H + h) - x_G] + (1 / 2)(B - b)(h - x_G)^2}{B(H - h) + (B - b)(h - x_G)}$$

The calculation method of MRd at the support is identical, but the longitudinal reinforcement in the upper part must also be taken into account.

Execution of the welded splices of rolled beams

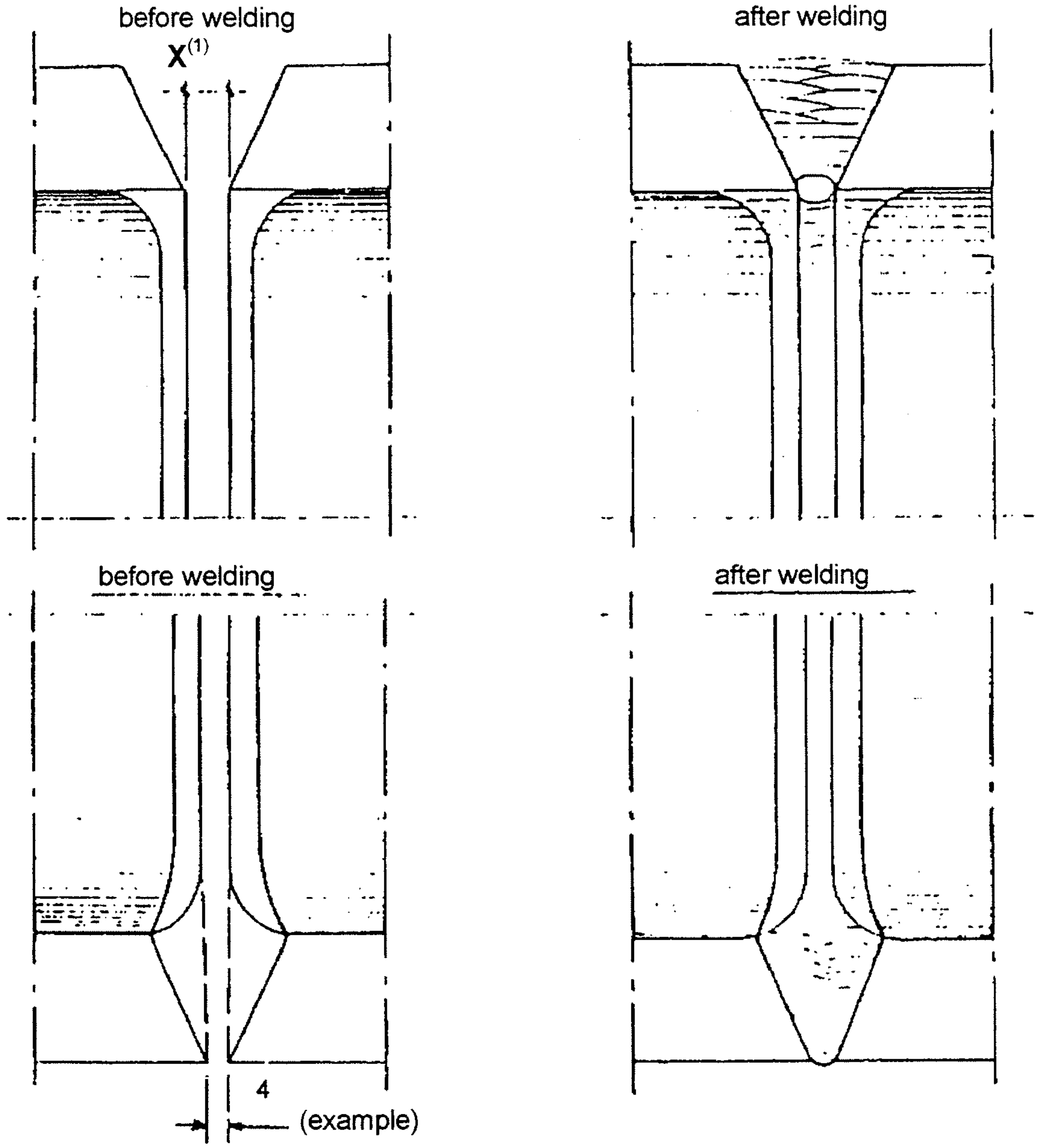


Figure 1 - Weld preparation - Welding

(1) The value of x shall be determined when the test weld is made and checked when the first beam is welded.

APPENDIX 2

Beams supported on rollers

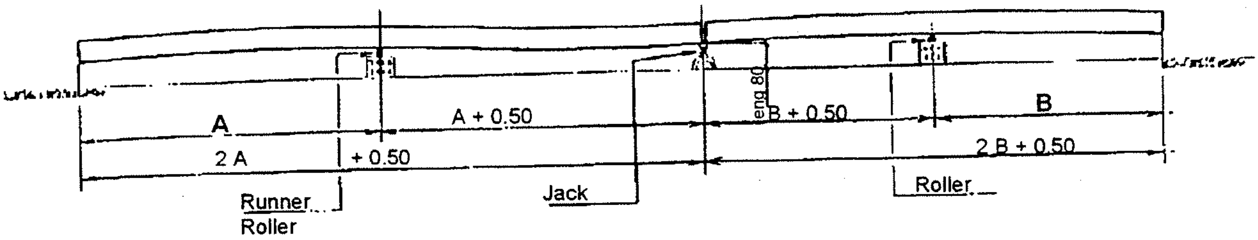


Figure 2

Schematic drawing of welded joint

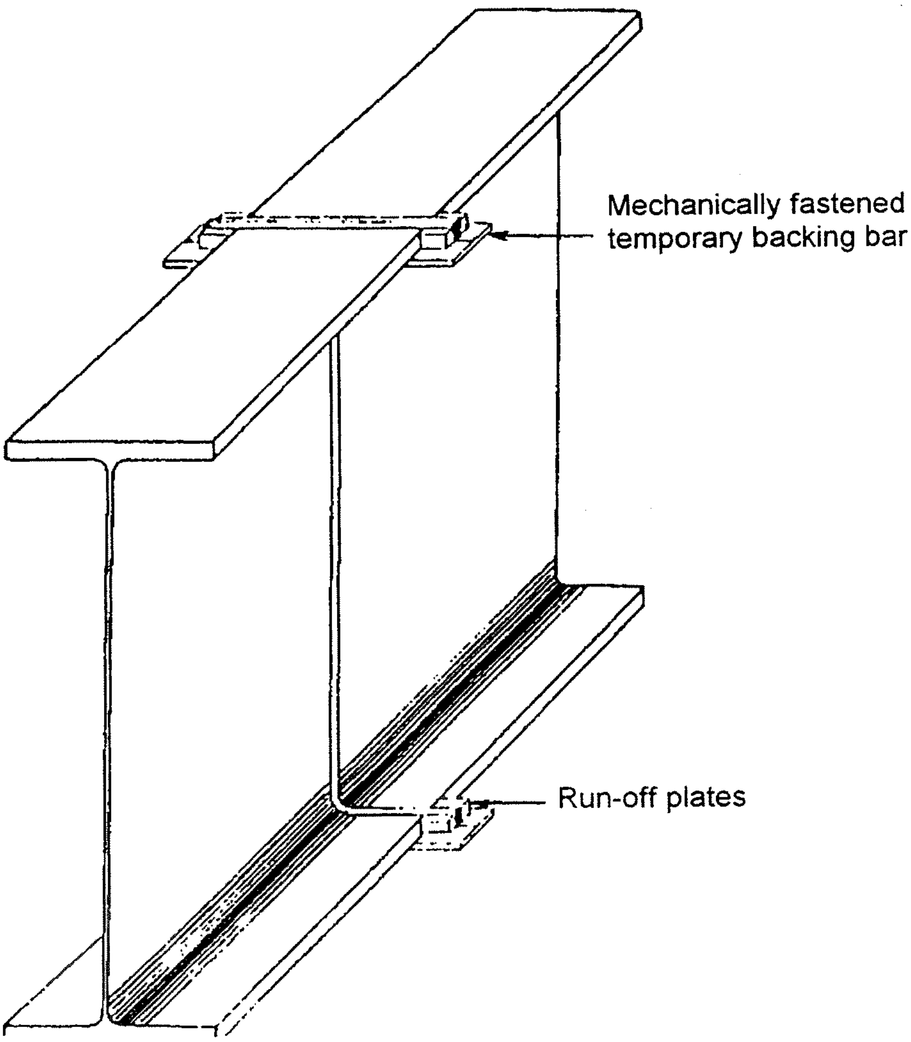


Figure 3

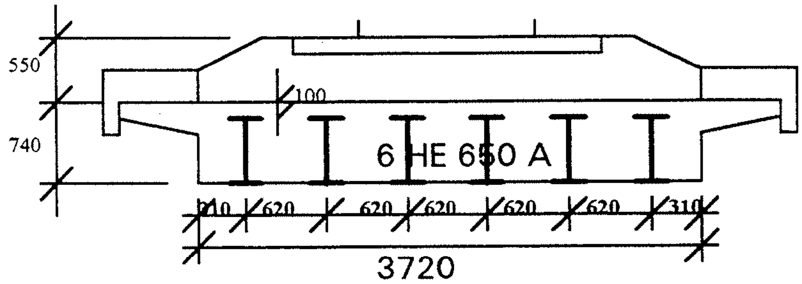
Calculation examples:

3.1 one span bridge of 11 m
3.2 two span continuous bridge of 17 m each span

3.1 - Railway bridge with one simply supported span of 11 m

3.1.1 - General - Cross sectional resistance of the deck

Cross section



Materials:

Beams: steel S235
nominal yield strength: $f_y = 235\ N/mm^2$

Concrete: C 30/37
characteristic compressive cylinder strength: $f_{ck} = 30\ N/mm^2$

Characteristics of the beams:
Weight: 1.90 kN/m
Moment of inertia (strong axis): 175 200 cm^4
Section modulus (strong axis): 5 474 cm^3

Characteristics of the composite section composed of **one** beam and the concrete with a width equal to the spacing of the beams (620 mm):

Modular ratio:	$n_v = 18$	$n_i = 6$
Moment of inertia, assuming that the concrete in tension is cracked:	233 200 cm^4	293 800 cm^4
Moment of inertia, assuming that the concrete is uncracked:	279 600 cm^4	482 000 cm^4
Section modulus, related to the lower fibre of the lower flange, assuming that the concrete in tension is cracked:	5 860 cm^3	6 280 cm^3
Section modulus, related to the upper fibre of the concrete, Ie with the assumption that the concrete in tension is cracked (M>0):	123 000 cm^3	64 800 cm^3

3.1.2 - Actions and internal forces

a) Constant loads

Self weight:

beams: 6 × 1.90 =	11.4	kN/m
encasing concrete: 3.72 × 0.714 × 24 =	63.4	
	------	
total:	75.1	kN/m

Additional constant loads:

Protective layer and waterproofing	3.2	kN/m
Ballast : 0,55 × 4 × 20 kN/m^3	44.0	
Rails	1.2	
Cantilever part of slab	7.0	
Facing plate and parapet	8.0	
Cable trough	6.0	

Total	69.4	kN/m

NOTE: Pending the publication of part 2.1 of ENV 1991, the additional loads are evaluated in this example with a unique characteristic value, although variations which are by no means negligible can occur.

Bending moment at mid-span for one beam:

The loads are assumed to be uniformly distributed on the 6 beams:

Selfweight:

$$M_{G1} = \frac{1}{6} \times \frac{75.1 \times (11)^2}{8} = 189.3 \text{ kNm}$$

Additional constant loads:

$$M_{G2} = \frac{1}{6} \times \frac{69.4 \times (11)^2}{8} = 174.9 \text{ kNm}$$

b) Traffic loads

- Loads model 71 with α = 1 and dynamic factor Φ_2 (according to ENV 1991-3: 1994). Load model SW/0 produces smaller internal forces than LM 71

  $\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82$ with L_Φ = length of the span = 11 m; Φ_2 = 1.28

- Loads model SW/2: 150 kN/m, with dynamic factor: Φ_2 = 1.28

Bending moment at mid span for one beam:

Loads are assumed to be uniformly distributed over the 6 beams.

$$M_{Q71} = \frac{1}{6} \times 1.28 \times 2187 = 467 kNm$$

$$M_{QSW2} = \frac{1}{6} \times 1.28 \times \frac{150 \times 11^2}{8} = 484 kNm$$

For simplicity live loads on the side paths are disregarded.

c) criteria to validate dynamic factor Φ:

The natural frequency of the deck without traffic load is:

$n_0 = \frac{17.75}{\sqrt{\delta_0}}$ with δ_0 = deflection (in mm) at mid span under constant loads:

$\delta_0 = \frac{5}{384} \times \frac{g.L^4}{E.I}$ with:

g = permanent loads = $\frac{1}{6}(75.1 + 69.4) = 24.08$ kN/m

L = span = 11 m

E = modulus of elasticity = 210 x 10^6 kN/m^2

I = average moment of inertia = $\frac{I'+I'}{2}$ for $n_i = 6$; I = 387 900 cm^4

thus: $\delta_0 = \frac{5}{384} \times \frac{24.18 \times (11)^4}{210.10^6 \times 387900.10^{-8}} = 0.00566 = 5.66$ mm

$$n_0 = \frac{17.75}{\sqrt{5.66}} = 7.46 H_Z > n_{0MIN} = \frac{80}{11} = 7.27 H_Z \quad \text{et} \quad n_0 < n_{0max} = 15.7\ H_z$$

3.1.3 - Verification of the serviceability limit state

a) Limitation of stresses:

Maximum stress in the lower flange of the beams:

$$\sigma_a = \frac{M_{G1}}{W_{beam}} + \frac{M_{G2}}{W_{anv}} + \frac{M_{QSW2}}{W_{ani}} = 35 + 30 + 77 = 142 N/mm^2 < \frac{f_y}{1.15} = \frac{235}{1.15} = 204 N/mm^2$$

Maximum compression stress in concrete:

$$\sigma_c = \frac{M_{G2}}{W_{cn_v}} + \frac{M_{QSW2}}{W_{cn_i}} = 1.4 + 7.5 = 8.9 N/mm^2 < 0.6.f_{ck} = 0.6 \times 30 = 18 N/mm^2$$

b) Limitation of vertical deformation under the 71 load model

Maximum deflection due to load model 71

$$\delta(m) = \Phi_2 = \frac{1}{6} \times \frac{1846 \times (L^4)}{10^3 \times EI} \text{ with:}$$

L (m) = span

E (kN/m^2) = modulus of elasticity

I (m^4) = moment of inertia; in this case, $\frac{I'+I''}{2} = \frac{293800 + 482000}{2} = 387900 cm^4$

thus δ = 0.007 m

$\frac{\delta}{L} = \frac{0.007}{11} = 0.00064 = \frac{1}{1554} < 2 \times \frac{1}{1200}$ = limiting value for a single span bridge, with a speed V < 280 km/h and a very high ride quality (as specified in ENV 1991-3:1994)

c) Limitation of the end rotation

$$\theta \approx \frac{4\delta}{L} = 2.55\ 10^{-3} \quad < \quad 6.5\ 10^{-3} \text{ radians}$$

3.1.4 - Verification of the ultimate limit state

a) Design value of the resisting bending moment MRd:

$$\frac{f_y}{\gamma_a} = \frac{235}{1.1} = 214 \text{ N/mm}^2 \qquad \frac{\lambda \times f_{ck}}{\gamma_c} = \frac{0.85 \times 30}{1.5} = 17 \text{ N/mm}^2$$

Thus $M_{pl.Rd}$ = 1595 kNm

b) Design value of the applied bending moment M_{Sd}:

For the main loads combination:

$$M_{Sd71} = 1.35M_G + 1.45M_{Q71} = 1.35 \times (189 + 175) + 1.45 \times 467 = 1169 kNm < M_{pl.Rd} = 1595 kNm$$

$$M_{SdSW2} = 1.35M_G + 1.2M_{QSW2} = 1.35 \times (189 + 175) + 1.2 \times 484 = 1072 kNm < M_{pl.Rd} = 1595 kNm$$

Note: For simplicity wind loads are disregarded.

3.1.5 - Conclusions and comments

The span of 11 m has been chosen because it just satisfies the natural frequency criteria. On condition that dynamic behaviour check calculations are carried out, this deck cross section could be used for greater spans depending upon the relevant requirements that must be complied with, or by using a higher strength steel.

3.1.5.1 - Allowable span for the ULS criteria:

M_{sd} = 1169 kNm for 11 m

$M_{pl.Rd}$ = 1595 kNm

or: $L_{allowable} \approx \sqrt{\frac{1595}{1169}} \times 11 = 12.85m$ for steel S235 and

$L_{allowable} = 12.85 \times \frac{355}{255} \times 0.9 = 16.1m$ for steel S355; the reduction factor 0.9 approximately takes into account the participation of concrete within $M_{pl.Rd}$

3.1.5.2 - Allowable span for the SLS criteria

a) Limitation of stresses:

σ_a = 142 N/mm2 for L=11m

$\frac{f_y}{1.15} = 204 N/mm^2$ for steel S255 and = 308.7 N/mm2 for steel S355:

$L_{allowable} = 11 \times \sqrt{\frac{204}{142}} = 13.2m$ for S255 and $L_{allowable} = 11 \times \sqrt{\frac{308.7}{142}} = 16.2m$ for S355

b) Limitation of the deflection under load model 71:

δ = 7 mm for 11 m

$\delta(L) \cong 7 \times \frac{L^4}{11^4} \Rightarrow$ L < 15.1 m in order to comply with the condition: $\frac{\delta}{L} < \frac{1}{600}$

c) Limitation of the end rotation:

θ (L=11 m) = 2.55 10^{-3} radian θ (L) < 6.5 10^{-3} radian $\Rightarrow$ L < 13.9 m

3.1.5.3 - Span allowing the use of Φ without dynamic calculation:

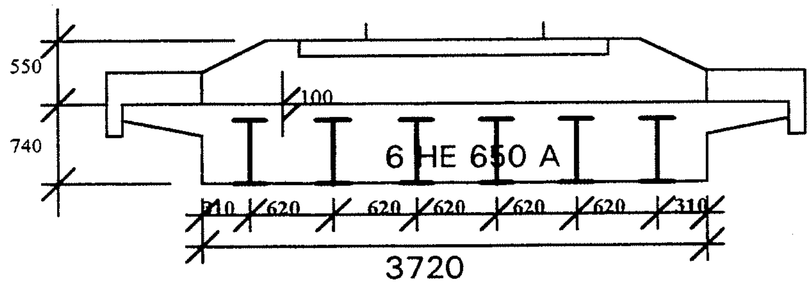
$n_{0min.} = \frac{80}{L}$ with $n_0 = \frac{17.75}{\sqrt{\delta_0}}$ where $\delta_0 \approx 5.66 \times \frac{L^4}{11^4}$ assuming that neither the constant load, nor the moment of inertia increase.

The result is L = 11.28 m.

3.2 - Railway bridge with two continuous spans of 17 m each

3.2.1 - General - Cross section of the deck

3.2.1.1 - Cross section



3.2.1.2 - Materials:

Beams: Steel S355
Nominal yield strength: fy = 345 N/mm2

Concrete: C 30/37
Characteristic compressive cylinder strength: fck = 30 N/mm2

Reinforcement: Steel Fe E 500 high grip
Section area: 5 HA 25 mm diameter high bond bar per meter at the Intermediate support, equal to 15.2 cm² per beam; this is higher than the Minimum area, that is required in order to limit the crack opening to 0.2 mm (see § 4.4 of the rules).

3.2.1.3 - Characteristics of the beams HEA 650:

Weight: 1.90 kN/m
Moment of inertia (strong axis): 175 200 cm^4
Bending modulus (strong axis): 5 474 cm^3

3.2.1.4 - Characteristics of the composite section composed of one beam and the concrete with a width equal to the spacing of the beams (620 mm):

Modular ratio:	n_v **= 18**	n_i **= 6**
Moment of inertia, assuming that the concrete in tension is cracked, moment > 0:	233 200 cm^4	293 800 cm^4
Moment of inertia, assuming that the concrete in tension contributes to the strength, moment > 0:	279 600 cm^4	482 000 cm^4
Section modulus, related to the lower fibre of the lower flange, with the assumption that the concrete in tension is cracked (M>0):	5 860 cm^3	6 280 cm^3
Section modulus, related to the upper fibre of the concrete, le with the assumption that the concrete in tension is cracked (M>0):	123 000 cm^3	64 800 cm^3
Section modulus, related to the upper fibre of the upper flange, with the assumption that the concrete in tension is cracked (M<0):	5 713 cm3	5762 cm3
Section modulus, related to the upper reinforcement, with the assumption that the concrete in tension is cracked (M<0):	4 320 cm3	4 957 cm3
Section modulus, related to the lower fibre of the concrete, with the assumption that the concrete in tension is cracked (M<0):	156 280 cm3	71 290 cm3

3.2.2 - Actions and internal forces

3.2.2.1 - Constant loads

Self weight:

beams: 6 × 1.90 ==	11.4 kN/m
encasing concrete: 3.72 × 0.714 × 24 =	63.4
	------
total:	75.1 kN/m

Additional permanent loads:

protective layer and waterproofing	3.2 kN/m
ballast : 0.55 × 4 × 20 kN/m^3	44.0
rails	1.2
cantilever part of slab	7.0
facing plate and parapet	8.0
cable trough	6.0
	-------
total	69.4 kN/m

NOTA: Pending the publication of the part 2.1 of the ENV 1991, the additional loads are evaluated in this example with a unique characteristic value, although they can vary quite appreciably.

The loads are assumed to be uniformly distributed over the 6 beams:

Bending moment at mid-span for one beam:

Selfweight:

$$M_{G1} = \frac{75.1}{6} \times 20.23 = 253.21 \text{ kN.m}$$

Additional constant loads:

$$M_{G2} = \frac{69.4}{6} \times 20.23 = 234.0 \text{ kN.m}$$

Maximum bending moment on the intermediate support for one beam:

Selfweight:

$$M_{G1} = \frac{75.1}{6} \times -36.125 = -452.16 \text{ kN.m}$$

Additional constant loads:

$$M_{G2} = \frac{69.4}{6} \times -36.125 = -417.85 \text{ kN.m}$$

3.2.2.2 - Traffic loads

- Load model 71 with $\alpha = 1$ and dynamic factor Φ_2 (according to ENV 1991-3:1994)

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82$$ with L_Φ = average length of the span $\times$ 1.2 = 20.4 m;

$\Phi_2 = 1.154$

- Load model SW/2: 150 kN/m, with dynamic factor: $\Phi_2 = 1.154$

- Load model SW/0 is not taken into account.

3.2.2.3 - Criteria to validate the dynamic factor Φ:

The natural frequency of the deck without traffic load is:

$n_0 = \frac{17.75}{\sqrt{\delta_0}}$ with δ_0 = maximum deflection (in mm) under constant loads:

$\delta_0 = 4510 \times \frac{g}{E \times I}$ with:

g = constant loads = $\frac{1}{6}(75.1 + 69.4) = 24.08$ kN/m

E = modulus of elasticity = 210. 10^6 kN/m^2

I = average moment of inertia = $\frac{I' + I'}{2}$ for $n_i = 6$; I = 387 900 cm^4

thus: $d_0 = 4510 \frac{24.08}{210 \times 10^6 \times 38.79 \times 10^{-4}} = 0.01333 = 13.33$ mm

$n_0 = \frac{17.75}{\sqrt{13.33}} = 4.85 H_Z > n_{0MIN} = \frac{80}{17} = 4.706 H_Z$ and $n_0 < n_{0max} = 11.4$ Hz

Bending moment at mid span for one beam:

$M_{Q71} = \frac{1}{6} \times 1.154 \times 3542.2 = 681.3 kNm$

$M_{QSW2} = \frac{1}{6} \times 1.154 \times 4118.24 = 792.1 kNm$

Bending moment at the intermediate support for one beam:

$M_{Q71} = \frac{1}{6} \times 1.154 \times -3649.9 = -702 kNm$

$M_{SW2} = \frac{1}{6} \times 1.154 \times -4686 = -901.3 kNm$

Note: For simplicity live loads on the side paths are disregarded.

3.2.3 - Verification of the serviceability limit state

3.2.3.1 - Limitation of stresses:

a) At mid span

Maximum stress in the lower flange of the beams:

$$\sigma_a = \frac{M_{G1}}{W_{beam}} + \frac{M_{G2}}{W_{anv}} + \frac{M_{QSW2}}{W_{ani}} = 46.2 + 39.9 + 126.1 = 212.2 N/mm^2$$

$$< \frac{f_y}{1.15} = \frac{345}{1.15} = 300 N/mm^2$$

Maximum compression stress in the concrete:

$$\sigma_c = \frac{M_{G2}}{W_{cn_v}} + \frac{M_{QSW2}}{W_{cn_i}} = 1.9 + 12.2 = 14.1 N/mm^2 \quad < \quad 0.6.f_{ck} = 0.6 \times 30 = 18 N/mm^2$$

b) At intermediate support

Maximum stress in the upper flange of the beams:

$$\sigma_a = \frac{M_{G1}}{W_{beam}} + \frac{M_{G2}}{W_{anv}} + \frac{M_{QSW2}}{W_{ani}} = 82.6 + 73.1 + 156.4 = 312.1 N/mm^2 \quad > 300\ N/mm^2$$

The calculated stress is higher than the limiting value of 12.1 N/mm².

In order to reduce the moments at support, it is possible either to carry out an imposed settlement at support of the steel beams, or of the concreted deck; it is also possible to increase the cross section of the reinforcing bars.

The value of the necessary imposed settlement at support of the steel beams alone, would be:

$$d = \frac{2 \times s \times l^2}{3 \times h \times E} = \frac{2 \times 12.1 \times 17000^2}{3 \times 640 \times 210000} = 17.3 mm$$

The supplementary moments introduced by the imposed settlement are:

- at support: $MG1 = \frac{2 \times \sigma \times I}{h} = \frac{2 \times 12.1 \times 1.752.10^{\circ}}{640} = 66.25 kN.m$

- at midspan: $M_{G1} = 0.4 \times 66.25 = 26.5 kN.m$

The stress in the lower flange at midspan would be increased by 0.4×12.1 N/mm² thus 4.84 N/mm². this stress value would be allowable.

Maximum compression stress in concrete:

$$\sigma_C = \frac{M_{G2}}{W_{cn_V}} + \frac{M_{QSW2}}{W_{cn_i}} = 2.7 + 12.6 = 15.3 N/mm^2 < 18\ N/mm^2$$

Maximum stress in reinforcing bars:

$$\sigma_C = \frac{M_{G2}}{W_{cn_V}} + \frac{M_{QSW2}}{W_{cn_i}} = 86.4 + 181.8 = 268.2 N/mm^2 < 500/1.15 = 434.8\ N/mm^2$$

3.2.3.2 - Criteria of concrete cracking at intermediate support

See point 3.2.1.2 above

3.2.3.3 - Limitation of the vertical deformation

Maximum deflection due to load model 71

$$\delta(m) = \Phi_2 \frac{1}{6} \times \frac{1256 \times (L^4)}{10^3 \times EI} \quad \text{with:}$$

L (m) = span

E (kN/m^2) = modulus of elasticity

I (m^4) = moment of inertia; in this case, $I = \frac{I' + I''}{2} = \frac{293800 + 482000}{2} = 387900 cm^4$

The result is δ = 0.0025 m

$\frac{\delta}{L} = \frac{0.0025}{17} = 0.00015 = \frac{1}{6666} < 1.5 \times \frac{1}{1500}$ = limiting value for a one span bridge, with a speed V < 280 km/h and a very high ride quality(as specified in ENV 1991-3:1994)

3.2.3.4 - Criteria for the end rotation under loads model 71

$$\theta \cong \frac{4\delta}{L} = 0.6\ 10^{-3} < 6.5\ 10^{-3} \text{ radian}$$

3.2.4 - Verification of the ultimate limit state

3.2.4.1 - Design value of the resisting bending moment M_{Rd}:

Design stresses:

steel of the beams $\frac{f_y}{\gamma_a} = \frac{345}{1.1} = 313.6 \text{ N/mm}^2$ Concrete: $\frac{\lambda \times f_{ck}}{\gamma_c} = \frac{0.85 \times 30}{1.5} = 17 \text{ N/mm}^2$

reinforcement: $\frac{f_y}{1.15} = 434.8 \text{ N/mm}^2$

a) At mid span

$M_{pl.Rd} = 2266$ kNm

b) At intermediate support

$M_{pl.Rd} =$ - 1919 kNm

3.2.4.2 - Bending moments

For the main load combination:

a) At mid span

$M_{Sd71} = 1.35M_G + 1.45M_{Q71} = 1.35 \times (253.21 + 26.5 + 234) + 1.45 \times 681.3 = 1681.3\text{kNm} < M_{pl.Rd} = 2266\text{kNm}$

$M_{SdSW2} = 1.35M_G + 1.2M_{QSW2} = 1.35 \times (253.21 + 26.5 + 234) + 1.2 \times 792.1 = 1644.1\text{kNm} < M_{pl.Rd} = 2266\text{kNm}$

b) At support

$M_{Sd71} = 1.35M_G + 1.45M_{Q71} = 1{,}35 \times (-452.16 + 66.25 - 417.85) + 1.45 \times (-702) = -2103.0\text{kNm} < M_{pl.Rd} = -1982.4\text{kNm}$

$M_{SdSW2} = 1.35M_G + 1.2M_{QSW2} = 1.35 \times (-452.16 + 66.25 - 417.85) + 1.2 \times (-901.3) = -2166.7\text{kNm} < M_{pl.Rd} = -1982.4\text{kNm}$

Note: For simplicity wind loads are disregarded.

3.2.4.3 - Reduction of the moment at support due to the presence of the bearings (b=0.25 m)

a = 0.25 + 0.74 = 0.99 m

The maximum support reaction R at the ultimate limit state is equal to:

$$R = \frac{10 \times M}{l} = \frac{10 \times 2166}{17} = 1274 kN$$

$$\Delta M = \frac{R \times a}{8} = \frac{1274 \times 0.99}{8} = 157.7 kN\ m$$

The reduced moment is equal to: 2166 - 157.7 = 2008.3 *kNm*. It is a little higher (1.3 %) than the resistant moment (= 1982.4 *kN m*).

3.2.4.4 - Redistribution of the moment at support

The lower flange of the beam is in class 1

At support, $|M_{Sd}| > |M_{pl.Rd}|$

We reduce the moment at support by 15%. M_{Sd} = 0.85 × 2166.7 = 1841.7 < 1982.4 *kNm*. The maximum moment at midspan is increased by 0.4 × 0.15 × 2166.7 = 130 *kN m* the maximum calculated moment is: 168.3 + 130 = 1811.3 *kN m* < 2266 *kN m*.
The design is therefore verified at the ultimate limit state.

3.2.5 - Conclusions and comments

The redistribution of the moment at support avoids the need for imposed settlements of the supports of the beams or of the concreted deck.

Nevertheless it is recommended to carry out an imposed settlement of the steel beams, which requires only a modified line of the fabrication camber used.

Reinforcement of filler beam deck: example

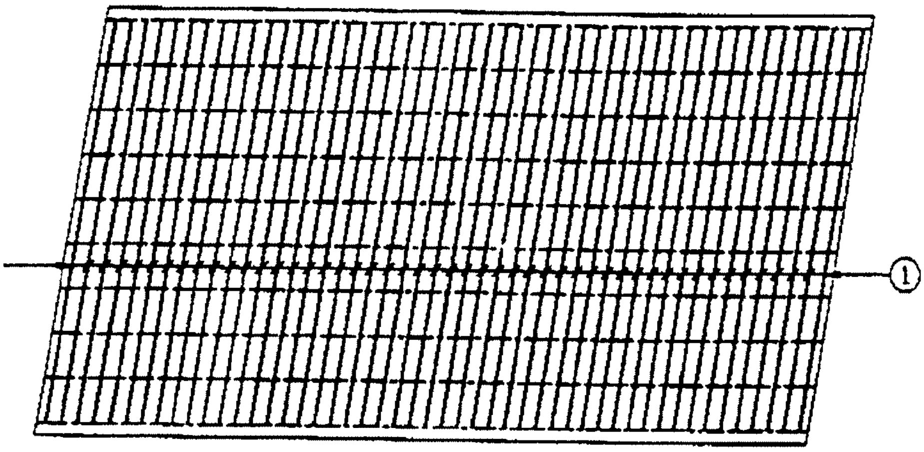


Figure 1 : transverse reinforcement parallel to the support lines

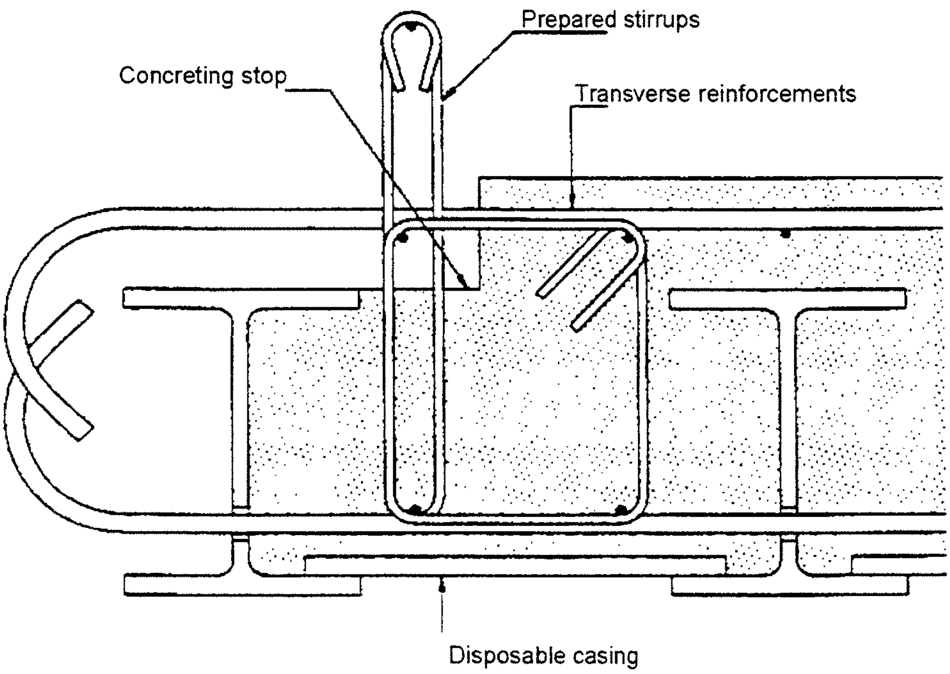


Figure 2 : Edge of the deck before construction of the face plates

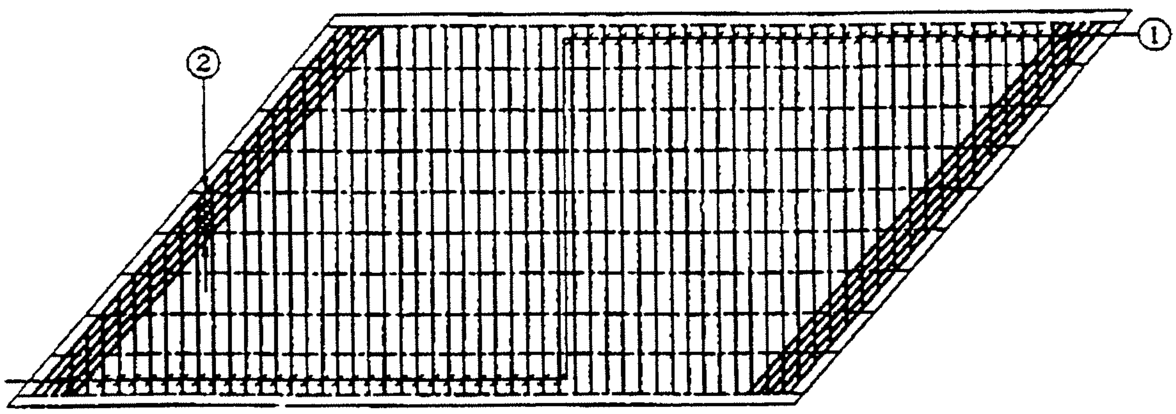


Figure 3: **Transverse reinforcement composed of bars perpendicular to the beams (bar no.1) and with local strengthening at the ends, orientated according to skew (bars no. 2)**

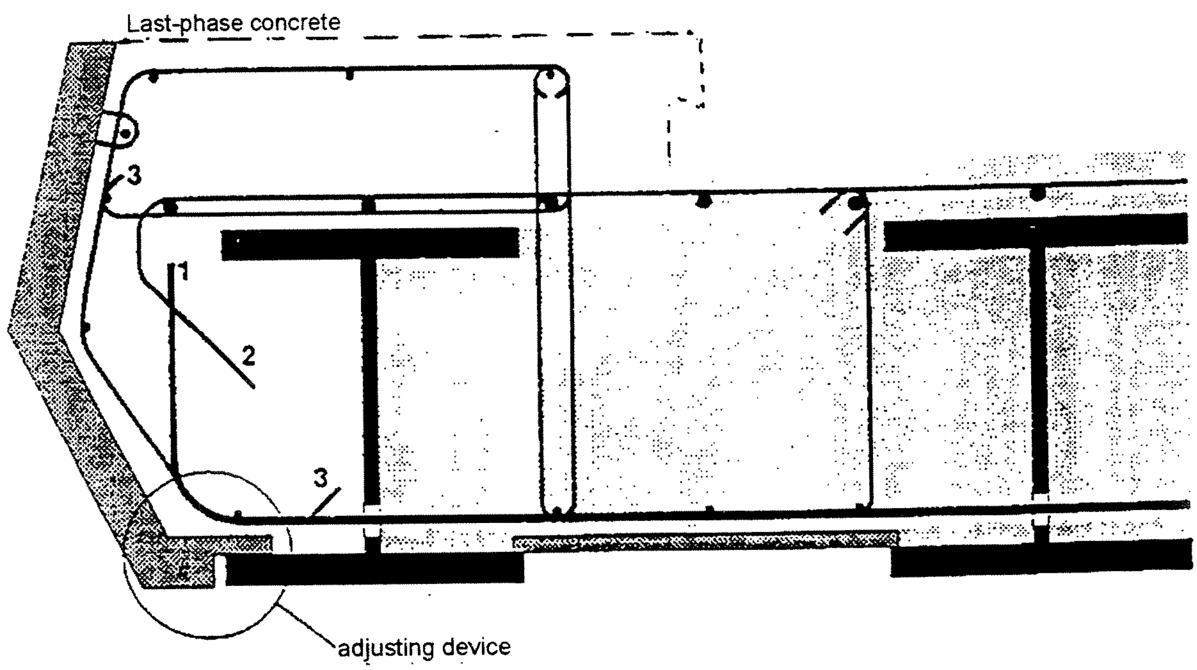


Figure 4: **Principles for the reinforcement details at the free edges and for the prefabricated concrete face plates.**

Complementary recommendations if lightweight concrete is used

5.1 - Modular ratio steel/concrete

5.1.1 - The short term ratio is, unless otherwise specified by the relevant authority, equal to :

$$n_i = 6\left(\frac{2.5}{\rho_s}\right)^{3/2} = 23.717\ (\rho_s)^{-3/2}$$

ρ_s is the dry unit mass of the lightweight concrete expressed in t/m^3.

5.1.2 - The long term ratio, which takes into account the shrinkage of concrete, is defined, unless otherwise specified as (using formula of 2.4.2 with $K_{f1} = 1$) :

$$n_v = 2\ n_i$$

5.2 Constant loads

For the calculation of the weight of the elements of the structure, the unit mass of the light weight encased concrete must be prescribed by the relevant authority.

5.3 - Limiting stress of the concrete

Factor λ for the calculation of the compressive stress in the concrete $\lambda\ f_{ck}/1.5$ in operating mode and $\lambda\ f_{ck}/1.5$ for load combinations involving accidental loads is equal to 0.75 for the light weight concrete, unless otherwise specified by the relevant authority.

Application

With effect from 1 January 1997.

All UIC Members.

Record References

Headings under which the question has been studied:

- Point 4 - Updating of Leaflet773.
(Sub-Committee 7 J « Bridges » : Paris, January 1996).

Point 11 - f) Design rules for joist-in-concrete bridges.
Approval of Leaflet 773.
(Infrastructure Commission: Killarney, April 1996).

- Point 4.1 - Amendment to the text of Leaflet 773.
(Technical Sub-Group for « Bridges » : Paris, March 1997).

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