COMBRI DESIGN MANUAL

Part I: Application of Eurocode rules

A project carried out with a financial grant from the Research Fund for Coal and Steel (RFCS) of the European Community.
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Part I: Application of Eurocode rules
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The present document and others related to the research project COMBRI+RFS-CR-03018 “Competitive Steel and Composite Bridges by Improved Steel Plated Structures” and the successive dissemination project RFS2-CT-2007-00031 “Valorisation of Knowledge for Competitive Steel and Composite Bridges”, which have been co-funded by the Research Fund for Coal and Steel (RFCS) of the European Community, can be accessed for free on the following project partners’ web sites:

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France:    www.cticm.com
Germany:   www.uni-stuttgart.de/ke, www.stb.rwth-aachen.de
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Cover pictures (from left to right):
Valley bridge Haseltal near Suhl, Germany, 2006 (© KE)
Valley bridge Dambachtal near Suhl, Germany, 2005 (© KE)
Viaduct over Dordogne river near Souillac, France, 2000 (© Sétra)
Preface

This design manual is an outcome of the research project RFS-CR-03018 “Competitive Steel and Composite Bridges by Improved Steel Plated Structures - COMBRI” [7] and the successive dissemination project RFS2-CT-2007-00031 “Valorisation of Knowledge for Competitive Steel and Composite Bridges - COMBRI+” which have been funded by the Research Fund for Coal and Steel (RFCS) of the European Community. Within the RFCS research project essential knowledge has been acquired to enhance the competitiveness of steel and composite bridges and this has been incorporated in the design manual at hand which has been also presented in the frame of several seminars and workshops. The manual is subdivided into two parts to provide the reader with clearly arranged and concise documents:

▶ Part I: Application of Eurocode rules

In the research project the different national background of each partner how to apply and interprete Eurocode rules was brought together and a European melting pot of background information and general knowledge has been created. In order to maintain this valuable information two composite bridge structures - a twin-girder and a box-girder bridge - are covered in this part of the COMBRI Design Manual on the basis of worked examples for which the knowledge is written down in a descriptive manner. The examples include references to current Eurocode rules.

▶ Part II: State-of-the-Art and Conceptual Design of Steel and Composite Bridges

The national state-of-the-art in bridge design can be different so that firstly bridge types of the project partners’ countries - Belgium, France, Germany, Spain and Sweden - are introduced. They reflect the current practice in those countries and common bridge types as well as unusual bridges intended to solve special problems and some solutions being part of development projects are presented in Part II of the COMBRI Design Manual [8]. Also, improvements which can be provided to the design of steel and composite bridges are discussed and the possibilities and restrictions given by the current Eurocode rules are highlighted.

Moreover, the features of software **EBPlate** [13] developed in the research project to determine the elastic critical buckling stresses are presented in its contributive application for bridge design.

Finally, the authors of this design manual gratefully acknowledge the support and financial grant of the Research Fund for Coal and Steel (RFCS) of the European Community.

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5 Summary

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Small Latin letters

\( a \)  
Length of a web plate between adjacent vertical stiffeners (possible lower indices: p, w)

\( b_b \)  
Width of the bottom flange

\( b_{bf} \)  
Width of the bottom flange of the beam

\( b_{eff} \)  
Effective width of the concrete slab

\( b_i \)  
Actual geometric width of the slab associated to the main girder

\( b_{gf} \)  
Width of the top flange of the beam

\( b_p \)  
Width of the panel

\( b_{slab} \)  
Thickness of the concrete slab

\( b_{slab} \)  
Width of each subpanel of the bottom flange

\( b_0 \)  
Center-to-center distance between the outside stud rows

\( b_1 \)  
Distance between webs of stiffener

\( b_2 \)  
Width of stiffener flange

\( b_3 \)  
Width of each stiffener web

\( c_{bf} \)  
Part of the bottom flange subject to compression

\( c_w \)  
Part of the web subject to compression

\( c_{lr} \)  
Distance between the centroid of each lower longitudinal reinforcing steel layer and the near free external surface

\( c_{ur} \)  
Distance between the centroid of each upper longitudinal reinforcing steel layer and the near free external surface

\( e \)  
Thickness of the concrete slab

\( f_{cd} \)  
Design value of concrete compressive strength

\( f_{ck} \)  
Characteristic compressive cylinder strength at 28 days

\( f_{ck,0.05} \)  
5% fractile of the characteristic axial tensile strength

\( f_{ck,0.95} \)  
95% fractile of the characteristic axial tensile strength

\( f_{cm} \)  
Mean value of axial tensile strength

\( f_{cm} \)  
Mean value of concrete cylinder strength at 28 days

\( f_{sk} \)  
Yield strength of the reinforcing steel bars

\( f_u \)  
Ultimate stress of the structural steel

\( f_y \)  
Yield strength of the structural steel (possible lower indices: w, tf, tf1, tf2, tst, p)

\( f_{yd} \)  
Design yield strength (possible lower indices: w, tf, tf1, tf2, tst, p)

\( h \)  
Height of the beam

\( h_{seff} \)  
Effective elastic neutral axis (ENA) of the structural steel part only

\( h_{eff} \)  
Effective elastic neutral axis of the composite cross-section
$h_w$  Height of the web of the beam

$h_{w,\text{eff}}$  Effective height of the web in compression of the beam

$h_{w1,2}$  Distribution of the effective height of the web in compression of the beam

$h_{st}$  Height of stiffener

$k_{\sigma}$  Plate buckling coefficient for normal stress (possible lower indices: c, p, w, pw, pbf, LT, op)

$k_{\tau}$  Plate buckling coefficient for shear stress

$k_{\tau,\text{st}}$  Plate buckling coefficient for shear stress for plate with longitudinal stiffeners

$m_q$  Torque loads due to distributed loads

$n_{lr}$  Total number of steel reinforcement in lower layer

$n, n_0$  Structural steel / concrete modular ratio for short-term loading

$n_L$  Structural steel / concrete modular ratio for long-term loading

$n_{st}$  Number of stiffeners (equally spaced) of the bottom flange

$n_{ur}$  Total number of steel reinforcement on in upper layer

$p$  Perimeter of the concrete slab

$q$  Eccentric distributed traffic loads

$q_{\text{pk}}$  Characteristic value of the uniformly distributed load due to pedestrian and cycle traffic

$q_{\text{min}}$  Minimum value of the line load due to bridge equipment

$q_{\text{max}}$  Maximum value of the line load due to bridge equipment

$q_{\text{nom}}$  Nominal value of the line load due to bridge equipment (safety devices, pavement, …)

$s_{lr}$  Spacing in lower layer of steel reinforcement of the concrete slab

$s_{ur}$  Spacing in upper layer of steel reinforcement of the concrete slab

$t$  Plate thickness (possible lower indices: tf, tf1, tf2, p, w, st)

$t$  Time

$t_0$  Age of the concrete put in place at each construction phase

$t_f$  Thickness of the flange of the beam (possible lower indices: 1, 2)

$t_{\text{ini}}$  Time at traffic opening

$t_{\text{slab}}$  Thickness of the concrete slab

$t_{st}$  Thickness of stiffener

$t_{st,eq}$  Equivalent thickness of stiffener web

$t_w$  Thickness of the web of the beam

$w$  Width of all traffic lanes

$x$  Abscissa of bridge

$z$  Position of the centre of gravity of the cross-section

$z_{\text{na}}$  Position of the centre of gravity of the composite cross-section

$z_{\text{pl}}$  Position of the plastic neutral axis (PNA) of the composite cross-section
Capital Latin Letters

\( A_a \)  Cross-sectional area of the structural steel section

\( A_{abf} \)  Cross-sectional area of the bottom flange

\( A_a, \text{eff} \)  Effective cross-sectional area of the steel structural part only

\( A_{a2f} \)  Cross-sectional area of the top flange (possible lower indices: 1, 2)

\( A_{aw} \)  Cross-sectional area of the web

\( A_c \)  Total cross-sectional area of the concrete slab; Gross area of the bottom flange with stiffeners neglecting parts supported by adjacent plate elements

\( A_{c, \text{eff,loc}} \)  Effective local area of the bottom flange with stiffeners

\( A_{c2r} \)  Cross-sectional area of the concrete located under the lower layer of steel reinforcement of the slab

\( A_{c3r} \)  Cross-sectional area of the concrete located between the lower and upper layer of steel reinforcement of the slab

\( A_{c4r} \)  Cross-sectional area of the concrete located above the upper layer of steel reinforcement of the slab

\( A_{\text{eff}} \)  Effective cross-sectional area of the composite cross-section

\( A_{slr} \)  Cross-sectional area of one steel reinforcement of the lower layer of the slab

\( A_{trw} \)  Cross-sectional area of the web stiffener

\( A_{sur} \)  Cross-sectional area of one steel reinforcement of the upper layer of the slab

\( A_{tot} \)  Gross cross-sectional area of the composite cross-section

\( A_{slr} \)  Cross-sectional area of the total steel reinforcement of the lower layer of the slab

\( A_{sur} \)  Cross-sectional area of the total steel reinforcement of the upper layer of the slab

\( E_a \)  Elasticity modulus of structural steel

\( E_{cm} \)  Elasticity modulus of concrete

\( E_s \)  Elasticity modulus of reinforcing steel

\( G_k \)  Characteristic (nominal) value of the effect of permanent actions

\( G_{k, \text{inf}} \)  Characteristic value of a favourable permanent action (nominal value of self-weight and minimum value of non-structural equipments) taking account of construction phases

\( G_{k, \text{sup}} \)  Characteristic value of an unfavourable permanent action (nominal value of self-weight and maximum value of non-structural equipments) taking account of construction phases

\( I \)  Second moment of area

\( I_{a, \text{eff}} \)  Effective second moment of area of the structural steel part

\( I_{\text{eff}} \)  Effective second moment of area of the cross-section

\( I_t \)  The St. Venant torsional stiffness

\( I_{tot} \)  Gross second moment of area of the composite bridge

\( I_A \)  Second moment of area (around a horizontal axis \( \Delta \) located at the steel/concrete interface)

\( L_e \)  Equivalent span length in the considered cross-section

\( L_i \)  Length of the span i
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<th>Description</th>
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<td>$M_{Ed}$</td>
<td>Design bending moment acting on the composite cross-section</td>
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<td>Design value of the plastic resistance moment of a cross-section consisting of the flanges only</td>
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<td>Design bending moment</td>
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<td>$Q_{k1,i}$</td>
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<td>$S$</td>
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</tr>
<tr>
<td>$V_{pl,a,Rd}$</td>
<td>Design value of the plastic shear resistance of the structural steel cross-section</td>
</tr>
</tbody>
</table>
Notations

Small Greek letters

\( \alpha \)  
Factor; angle; compressed height percentage

\( \alpha_{qi} \)  
Adjustment factor on concentrated load TS of LM1 on lanes i (i = 1, 2, …)

\( \alpha_{qi} \)  
Adjustment factor on uniformly distributed load UDL of LM1 on lanes i (i =1, 2,…)

\( \alpha_{qr} \)  
Adjustment factor on load model LM1 on the remaining area

\( \beta \)  
Reduction factor for shear lag effect

\( \gamma_c \)  
Partial safety factor for resistance of concrete

\( \gamma_M \)  
Partial safety factor for resistance of structural steel

\( \gamma_{M0} \)  
Partial safety factor for resistance of structural steel (Yielding, local instability)

\( \gamma_{M1} \)  
Partial safety factor for resistance of structural steel (Resistance of members to instability)

\( \gamma_{M2} \)  
Partial safety factor for resistance of structural steel (Resistance of joints)

\( \gamma_{M,ser} \)  
Partial safety factor for resistance of structural steel at Serviceability Limit State (SLS)

\( \gamma_S \)  
Partial safety factor for resistance of reinforcing steel

\( \varepsilon \)  
Strain; factor \( \sqrt{ \frac{235N}{mm^2}} \) (possible lower indices: tf, tf1, tf2, p, w, st)

\( \varepsilon_{ca} \)  
Autogenous shrinkage strain

\( \varepsilon_d \)  
Drying shrinkage strain

\( \varepsilon_s \)  
Total shrinkage strain

\( \eta \)  
Coefficient of the yield strength of structural steel

\( \eta_1 : \eta_3 \)  
Ratio between applied stress and yield strength in a structural steel cross-section

\( \bar{\eta}_1 : \bar{\eta}_3 \)  
Ratio between applied force and resistance in a structural steel cross-section

\( \theta_{wa} \)  
Incline angle of the web with reference to the vertical direction

\( \kappa \)  
Factor to take into account shear lag

\( \bar{\lambda} \)  
Reduced slenderness (possible lower indices: c, p, w, pw, pbf, LT, op)

\( \mu \)  
Moment of area

\( \nu \)  
Poisson’s ratio

\( \sigma_{abfu} \)  
Stress at the upper edge of the bottom flange (possible lower indices: eff)

\( \sigma_{abl} \)  
Stress at the lower edge of the bottom flange (possible lower indices: eff)

\( \sigma_{at} \)  
Stress at the lower edge of the top flange (possible lower indices: eff)

\( \sigma_{aufu} \)  
Stress at the upper edge of the top flange (possible lower indices: eff)

\( \sigma_c \)  
Longitudinal upper fibre tensile stress in the concrete slab

\( \sigma_{cr} \)  
Elastic critical plate buckling stress

\( \sigma_E \)  
Elastic critical Euler’s stress

\( \sigma_{Ed} \)  
Design value of a direct stress in a cross-section

\( \sigma_{atr} \)  
Stress in the lower reinforcement of the concrete slab (possible lower indices: eff)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{\text{tiss}}$</td>
<td>Stress in the upper reinforcement of the concrete slab (possible lower indices: eff)</td>
</tr>
<tr>
<td>$\sigma_{\text{sup.reinf}}$</td>
<td>Maximal ULS Stress in the upper reinforcement of the concrete slab in the cracked behaviour (sagging moment)</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Reduction factor (( \leq 1.0 )) for effective area of a structural steel cross-section</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>Reduction factor for effective width</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Reinforcement ratio in a concrete cross-section</td>
</tr>
<tr>
<td>$\tau_{\text{cr}}$</td>
<td>Elastic critical shear buckling stress</td>
</tr>
<tr>
<td>$\tau_{\text{Ed}}$</td>
<td>Design value of a shear stress in a cross-section</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Creep function</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Diameter of the steel reinforcement of the concrete slab</td>
</tr>
<tr>
<td>$\phi_r$</td>
<td>Diameter of the upper steel reinforcement of the concrete slab</td>
</tr>
<tr>
<td>$\phi_{ur}$</td>
<td>Diameter of the upper steel reinforcement of the concrete slab</td>
</tr>
<tr>
<td>$\chi$</td>
<td>Reduction factor (( \leq 1 )) for instability (possible lower indices: c, p, w)</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Stress ratio between opposite edges of a structural steel plate (possible lower indices: w)</td>
</tr>
<tr>
<td>$\psi_L$</td>
<td>Creep multiplier for modular ratio</td>
</tr>
<tr>
<td>$\psi_0$</td>
<td>Factor for the combination value of a variable action</td>
</tr>
<tr>
<td>$\psi_1$</td>
<td>Factor for the frequent value of a variable action</td>
</tr>
<tr>
<td>$\psi_2$</td>
<td>Factor for the quasi-permanent value of a variable action</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>Area bordered by the mid-planes of the internal elements of the box cross-section</td>
</tr>
</tbody>
</table>
1 Introduction and scope

1.1 Introduction

In the COMBRI research project [7], in which this design manual has its origin, the different national background of each partner how to apply and interpret Eurocode rules was brought together and a European melting pot of background information and general knowledge has been created. In order to facilitate the implementation of Eurocodes EN 1993-1-5, EN 1993-2 and EN 1994-2 with regard to plate buckling verifications, it was decided to cover two steel-concrete composite bridges - a twin-girder and a box-girder bridge - in order to present the knowledge with the help of worked examples and in a very descriptive manner. As the examples focus in detail on the application and interpretation of Eurocode rules which are related to plate buckling verifications, the overall view on bridge design cannot be covered. In this context, Figure 1-1 shows how many standards can be involved in the design of a composite bridge. Here, we are mainly dealing with EN 1993-1-5 “Plated Structural Elements”, EN 1993-2 “Steel bridges” and EN 1994-2 “Composite bridges”.

![Figure 1-1: Eurocodes to be used in a composite bridge design.](image)

In some parts, this design manual introduces general assumptions e.g. on actions without aiming to present the theoretical background or the modelling in detail. In addition to that, it is assumed that the reader is familiar with general design and modelling issues of bridges because this design manual gives a detailed view on plate buckling topics but it can of course not cover all other topics related to the verification of the design. For further information to the aforementioned topics, the reader is referred to e.g. [2], [4], [6], [32], [33], [34], [35], [36], [37], [39], [41]. However, this summary of references does not attempt to provide a comprehensive overview on available literature so that much more excellent work may exist which is not mentioned here.
The design of steel and composite bridges according to the Eurocodes

For the design of steel and composite bridges the following Eurocodes are mandatory [3]:

- EN 1991-1-3  Eurocode 1: Actions on structures – Part 1-3: General actions, Snow loads [16]
- EN 1991-1-5  Eurocode 1: Actions on structures – Part 1-5: General actions, Thermal actions [18]
- EN 1997-1  Eurocode 7: Geotechnical design – Part 1: General rules [27]

Throughout the document, references to the Eurocodes used herein is given.
1.2 Structure of the document

In the following, the worked examples are presented in a double-sided layout with comments, background information and interpretation issues on the left-hand side and the example calculation on the right. All relevant references to current Eurocode rules are provided. As mentioned above, the examples are a twin-girder and a box-girder bridge which allows to look basically at a design without and with longitudinal stiffeners.

In Chapter 2 the deck of the twin-girder and the box-girder bridge is described and the global analysis of both bridges is introduced. For this purpose, an overview on the bridge geometry, material distribution and construction sequences is given firstly. Secondly, a general section follows in which common data such as material properties and actions as well as combinations thereof are given. Last but not least, the global analysis is presented for both bridges and the relevant results - internal forces and moments - are summarised for the verifications. Based on that, Chapters 3 and 4 look at the verifications during the final stage and the execution stage. Here, each chapter is subdivided into a part dealing with the verifications of the twin-girder bridge or the box-girder bridge.
2 Description of the deck and global analysis

2.1 Twin-girder bridge

2.1.1 Longitudinal elevation

The bridge is a symmetrical twin-girder composite structure with three spans of 50 m, 60 m and 50 m (i.e. a total length between abutments of 160 m). This is a theoretical example for which a few geometrical simplifications have been made:

- Straight horizontal alignment
- Flat top face of the deck
- Straight bridge
- Constant height of 2400 mm for the structural steel main girders

![Figure 2-1: Elevation of the twin-girder bridge.]

2.1.2 Transverse cross-section

The bridge carries a two-lane traffic road with two 3.5 m wide lanes, hard shoulders of 2 m width on each side and a standard safety barrier (see Figure 2-2).

The transverse cross-section of the concrete slab and of the non-structural equipment is symmetrical with reference to the axis of the bridge. The slab thickness varies from 0.4 m above the main girders to 0.25 m at its free edges, but it has been modelled as a 0.325 m deep rectangular cross-section.

The total slab width is 12 m. The centre-to-centre spacing between main girders is 7 m and the slab cantilever on each side is 2.5 m.

2.1.3 Structural steel distribution

The structural steel distribution for a main girder is presented in Figure 2-4.

Each main girder has a constant depth of 2400 mm and the variations in thickness of the upper and lower flanges face towards the inside of the girder. The lower flange is 1000 mm wide whereas the upper flange is 800 mm wide.
The two main girders have transverse bracings on abutments and on internal supports as well as every 8.333 m in side spans (C0-P1 and P2-C3) and every 7.5 m in the central span (P1-P2). Figure 2-3 illustrates the geometry adopted for this transverse cross-bracing on supports. In order to justify the shear resistance of the internal support sections, vertical stiffeners are added at 1.5 m and at about 4 m from the internal supports.

The optimisation of the stiffening will be discussed in Design Manual Part II [8], according to the results of COMBRI research project [7].
**Description of the deck and global analysis - Twin-girder bridge**

**Figure 2-4: Structural steel distribution for a main girder of the twin-girder bridge.**
EN 1994-2, 5.4.2.4, Stages and sequence of construction

(1) Appropriate analysis shall be made to cover the effects of staged construction including where necessary separate effects of actions applied to structural steel and to wholly or partially composite members.

(2) The effects of sequence of construction may be neglected in analysis for ultimate limit states other than fatigue, for composite members where all cross-sections are in Class 1 or 2 and in which no allowance for lateral-torsional buckling is necessary.

EN 1994-2, 6.6.5.2(3)

(3) In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations. Wherever possible, deformation should not be imposed on a shear connection until the concrete has reached a cylinder strength of at least 20 N/mm².
2.1.4 Construction phases

The assumptions regarding the construction phases are important for all the verifications during erection of the structural steel structure of the deck and during concreting. They are also necessary to determine the values of steel/concrete modular ratios (see Paragraph 2.3.3.3). Finally, the calculation of internal forces and moments in the deck should take construction phases into account (EN 1994-2, 5.4.2.4).

The following construction phases have been adopted:

- Erection of the structural steel structure by launching (see Section 4.1)
- On-site pouring of the concrete slab segments by casting them in a selected order:
  
  The total length of 160 m has been broken down into 16 identical 10-m-long concreting segments. They are poured in the order indicated in Figure 2-5. The start of pouring the first slab segment is the time origin \( t = 0 \). Its definition is necessary for determining the respective ages of the concrete slab segments during the construction phasing.

  The time taken to pour each slab segment is assessed to three working days. The first day is devoted to the concreting, the second day to its hardening and the third day to moving the mobile formwork. The slab is thus completed within 48 days (EN 1994-2, 6.6.5.2(3)).

- Installation of non-structural equipments:

  It is assumed to be completed within 32 days, so that the deck is fully constructed at the date \( t = 48 + 32 = 80 \) days.

Given these choices, Table 2-1 shows the ages of the various slab segments and the mean value of the age \( t_0 \) for all the concrete put in place at each construction phase.

![Figure 2-5: Order for concreting the slab segments of the twin-girder bridge.](image-url)
Table 2-1: Age of concrete slab segments at the end of the construction phasing of the twin-girder bridge.

<table>
<thead>
<tr>
<th>Segment</th>
<th>Mean age (days) of the concrete at the time of segment 1</th>
<th>Mean age (days) of the concrete at the end of construction phasing</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>13.5</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

Notes: The age is typically calculated from the beginning of the construction phase to the time when the segment is placed on the bridge. The ages provided are typical values and may vary depending on the specific project conditions.
2.2 **Box-girder bridge**

2.2.1 **Longitudinal elevation**

The bridge is a symmetrical box-girder composite structure with five spans of 90 m, 3 x 120 m and 90 m (i.e. a total length between abutments of 540 m). This is a theoretical example for which a few geometrical simplifications have been made:

- Straight horizontal alignment
- Flat top face of the deck
- Straight bridge
- Constant height of 4000 mm for the structural steel box-girder

![Figure 2-6: Elevation of the box-girder bridge.](image)

2.2.2 **Transverse cross-section**

The bridge carries a four-lane traffic road. Each lane is 3.50 meter wide and the two outside ones are bordered by a 2.06 meter wide safety lane. Standard safety barriers are located outside the traffic lanes and at the middle of the slab width (see Figure 2-7).

The transverse cross-section of the concrete slab and of the non-structural equipment is symmetrical with reference to the axis of the bridge. The 21.50 m wide slab has been modelled with a theoretical constant thickness of 0.325 m. The centre-to-centre spacing between the webs in the upper part is 12.00 m and the slab cantilever on each side is 4.75 m.

![Figure 2-7: Cross-section with traffic data of the box-girder bridge.](image)
The concrete slab is connected to an open box section with the following features:

- total depth: 4.00 m
- centre-to-centre distance between the webs in the upper part: 12.00 m
- centre-to-centre distance between the webs in the lower part: 6.50 m
- width of upper flanges: 1.50 m
- width of lower flange: 6.70 m

### 2.2.3 Structural steel distribution

The structural steel distribution is presented in Figure 2-10.

The box-girder has a constant depth of 4000 mm and the variations in thickness of the upper and lower flanges face towards the inside of the box-girder. The bottom flange is 6700 mm wide whereas the upper flanges are 1500 mm wide. An additional upper flange is necessary around the intermediate supports. It is located below the main upper flange, so that the total height of the box-girder is always equal to 4000 mm. The width of this additional upper flange is 1400 mm.

An additional steel rolled I-girder, which is located along the longitudinal bridge axis of symmetry, has been connected to the concrete slab. It helps during the concreting phases of the slab and participates in the resistance of the composite cross-section as additional section for the upper steel flanges.

The box section has transverse frames on abutments and on internal supports as well as every 4.0 m in side and central spans. Figure 2-8 illustrates the adopted geometry of this transverse cross-bracing on supports.

In order to justify the shear resistance in the web panels adjacent to an intermediate support, transverse frames are added at 2.5 m from the internal supports.

![Figure 2-8: Transverse cross-bracings on supports of the box-girder bridge.](image)

Figure 2-9 illustrates the dimensions of the bottom flange longitudinal trapezoidal stiffeners. The thickness is 15 mm for the webs and the flange of the stiffeners. They are continuous along the whole bridge, whereas web longitudinal stiffeners are only used for the panels surrounding the intermediate supports. The web longitudinal stiffeners have the same dimensions as the bottom flange longitudinal stiffeners; they are located at mid-depth of the webs. They have been added to justify the shear resistance of the web.

The stiffening design has been made following the recommendations of the COMBRI research project [7], leading to dimensions which are bigger than the classical ones.
EN 1994-2, 5.4.2.4, Stages and sequence of construction

(1)P Appropriate analysis shall be made to cover the effects of staged construction including where necessary separate effects of actions applied to structural steel and to wholly or partially composite members.

(2) The effects of sequence of construction may be neglected in analysis for ultimate limit states other than fatigue, for composite members where all cross-sections are in Class 1 or 2 and in which no allowance for lateral-torsional buckling is necessary.

EN 1994-2, 6.6.5.2(3)

(3) In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations. Wherever possible, deformation should not be imposed on a shear connection until the concrete has reached a cylinder strength of at least 20 N/mm².
Figure 2-9: Detail of a bottom flange longitudinal stiffener of the box-girder bridge.

**NOTE1:** On the intermediate supports, an additional upper flange of 1400 mm x 90 mm is welded to the main upper flange.

**NOTE2:** Different thickness ratios could be obtained by using S355 with quality M or ML around the intermediate supports. Indeed, according to EN 10025-3, by using steel S355 N/NL the main upper flange thickness is limited to 100 mm to keep a yield strength equal to 315 MPa, whereas according to EN 10025-4 by using S355 M/ML the yield strength is equal to 320 MPa for up to 120 mm thick plates. So a design in steel S355 M/ML allows a main 120 mm thick upper flange and an additional 70 mm thick upper flange at intermediate supports. The choice of plate thickness should also fulfill the requirements of EN 1993-1-10, Table 2.1.

**NOTE3:** An alternative design with a single upper flange made of S 460 is studied in Design Manual Part II, Chapter 3 [8].

### 2.2.4 Construction phases

The assumptions regarding the construction phases are important for all verifications during erection of the structural steel structure of the deck and during concreting. They are also necessary to determine the values of steel/concrete modular ratios (see Paragraph 2.3.3.3). Finally the calculation of internal forces and moments in the deck should take construction phases into account (EN 1994-2, 5.4.2.4).

The following construction phasing has been adopted:

- Erection of the structural steel structure by launching (see Section 4.2);
- On-site pouring of the concrete slab segments by casting them in a selected order:
  
  The total length of 540 m has been broken down into 45 identical 12-m-long concreting segments. They are poured in the order indicated in Figure 2-11. The start of pouring the first slab segment is the time origin \( t = 0 \). Its definition is necessary to determine the respective ages of the concrete slab segments during the construction phasing.

  The time taken to pour each slab segment is assessed to three working days. The first day is devoted to the concreting, the second day to its hardening and the third day to move the mobile formwork. The slab is thus completed within 135 days (EN 1994-2, 6.6.5.2(3)).

- Installation of non-structural equipments:
  
  It is assumed to be completed within 35 days, so that the deck is fully constructed at the date \( t = 135 + 35 = 170 \) days.

Given these choices Table 2-2 shows the ages of the various slab segments and the mean value of the age \( t_0 \) for all the concrete put in place at each construction phase. For simplification reasons, no allowance has been made to non-working days.
Figure 2-10: Structural steel distribution for a main girder of the box-girder bridge.
Table 2-2: Age of concrete slab segments at the end of the construction phasing of the box-girder bridge.

<table>
<thead>
<tr>
<th>Loading</th>
<th>segment 1</th>
<th>segment 2</th>
<th>segment 3</th>
<th>segment 4</th>
<th>segment 5</th>
<th>segment 6</th>
<th>segment 7</th>
<th>segment 8</th>
<th>...</th>
<th>segment 39</th>
<th>segment 40</th>
<th>segment 41</th>
<th>segment 42</th>
<th>segment 43</th>
<th>segment 44</th>
<th>segment 45</th>
<th>Mean age of the concrete at the time t</th>
</tr>
</thead>
<tbody>
<tr>
<td>pouring of the segment 1</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>pouring of the segment 2</td>
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<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>pouring of the segment 3</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>pouring of the segment 4</td>
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<td>9</td>
<td>6</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>pouring of the segment 5</td>
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<td>12</td>
<td>9</td>
<td>6</td>
<td>3</td>
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<td></td>
<td>7.5</td>
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</tr>
<tr>
<td>pouring of the segment 6</td>
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<td>15</td>
<td>12</td>
<td>9</td>
<td>6</td>
<td>3</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>pouring of the segment 7</td>
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<td>18</td>
<td>15</td>
<td>12</td>
<td>9</td>
<td>6</td>
<td>3</td>
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<td></td>
<td></td>
<td>10.5</td>
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</tr>
<tr>
<td>pouring of the segment 8</td>
<td>21</td>
<td>21</td>
<td>18</td>
<td>15</td>
<td>12</td>
<td>9</td>
<td>6</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>12</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>pouring of the segment 40</td>
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<td>117</td>
<td>114</td>
<td>111</td>
<td>108</td>
<td>105</td>
<td>102</td>
<td>99</td>
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2.3 General common data

2.3.1 Reinforcement of the concrete slab

2.3.1.1 Description of the slab reinforcement

This report does not deal with the transverse reinforcement. Only longitudinal reinforcement is described.

For the example dealt with herein, the cross-sections have been classified between span regions and intermediate support regions for calculation of the longitudinal reinforcing steel. The lengths of these regions are illustrated in Figure 2-12 and Figure 2-13.

- Span regions:
  High bond bars with diameter $\phi = 16$ mm and spacing $s = 130$ mm in upper and lower layers (i.e. in total $\rho_s = 0.96\%$ of the concrete section)

- Intermediate support regions:
  High bond bars with diameter $\phi = 20$ mm and spacing $s = 130$ mm in upper layer; high bond bars with diameter $\phi = 16$ mm and spacing $s = 130$ mm in lower layer (i.e. in total $\rho_s = 1.22\%$ of the concrete section)

![Figure 2-12: Location of mid-span and support sections for longitudinal reinforcing steel of the twin-girder bridge.](image)

![Figure 2-13: Location of mid-span and support sections for longitudinal reinforcing steel of the box-girder bridge.](image)

2.3.1.2 Modelling the slab to calculate the general longitudinal bending

For simplification reasons the actual slab cross-section of a half-deck (see Figure 2-15) is modelled by a rectangular area with the actual width (i.e. 6 m). The height $e$ of this rectangle is calculated so that the actual and modelled sections have the same area. This gives $e = 32.5$ cm.
The mechanical properties of the whole transverse cross-section of the slab are:

**Twin-girder bridge**
- Area: $A_b = 3.9 \, \text{m}^2$
- Second moment of area (around a horizontal axis $\Delta$ located at the steel/concrete interface):
  $$I_\Delta = 0.137 \, \text{m}^4$$
- Perimeter: $p = 24.65 \, \text{m}$

![Figure 2-14: Modelling the concrete slab for the longitudinal global bending (twin-girder bridge).](image)

**Box-girder bridge**
- Area: $A_b = 21.5 \cdot 0.325 = 6.99 \, \text{m}^2$
- Second moment of area (around a horizontal axis $\Delta$ located at the steel/concrete interface):
  $$I_\Delta = 21.5 \cdot 0.325^3 / 12 + A_b \cdot (0.325 / 2)^2 = 0.246 \, \text{m}^4$$
- Perimeter: $p = (21.5 + 0.325) \cdot 2 = 43.65 \, \text{m}$

![Figure 2-15: Modelling the concrete slab for the longitudinal global bending (box-girder bridge).](image)
EN 1993-1-1, 3.2.6, Design values of material coefficients

(1) The material coefficients to be adopted in calculations for the structural steels covered by this Eurocode Part should be taken as follows:

- modulus of elasticity \( E = 210000 \text{ N/mm}^2 \)
- shear modulus \( G = \frac{E}{2(1+\nu)} = 81000 \text{ N/mm}^2 \)
- Poisson’s ratio in elastic stage \( \nu = 0.3 \)
- coefficient of linear thermal expansion \( \alpha = 12.10^{-6} \text{ per } K \) (for \( T \leq 100 ^\circ\text{C} \))

NOTE: For calculating the structural effects of unequal temperatures in composite concrete-steel structures according to EN 1994 the coefficient of linear thermal expansion is taken as \( \alpha = 10.10^{-6} \text{ per } K \).

EN 1993-1-10 and EN 10164

EN 1992-1-1, Table 3.1, Strength and deformation characteristics for concrete

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<th>Analytical relation / Explanation</th>
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</table>
Similarly, to model the reinforcing bars, each longitudinal reinforcement layer is replaced by a single point-shaped bar with the same area and located in the plane of the main steel web. The reinforcing steel areas are introduced into the numerical model as percentages of the total area of the concrete slab:

Table 2-3: Areas of the steel reinforcement.

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<tr>
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<th>( \rho_s (%) )</th>
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<tr>
<td>Mid-span cross-sections</td>
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<tr>
<td>top layer</td>
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<tr>
<td>bottom layer</td>
<td>0.48</td>
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<tr>
<td>Support cross-sections</td>
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<tr>
<td>top layer</td>
<td>0.74</td>
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<tr>
<td>bottom layer</td>
<td>0.48</td>
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</table>

The centroid of each longitudinal reinforcing steel layer has been assumed to be located at 60 mm away from the closest horizontal face of the concrete slab. This value takes account of the concrete cover and the fact that the transverse reinforcing bars are placed outside the longitudinal reinforcing bars (on the side of the slab free surface).

2.3.2 Material properties

2.3.2.1 Structural steel

Steel grade S355 is considered for this bridge. The subgrades (also called quality) N or NL have been adopted (depending on the plate thickness). The corresponding structural steel mechanical properties are given in EN 10025-3.

Table 2-4: Decrease of \( f_y \) and \( f_u \) according to the plate thickness \( t \).

<table>
<thead>
<tr>
<th>( t ) [mm]</th>
<th>( \leq 16 )</th>
<th>( &gt; 16 )</th>
<th>( \leq 40 )</th>
<th>( &gt; 40 )</th>
<th>( \leq 63 )</th>
<th>( &gt; 63 )</th>
<th>( \leq 80 )</th>
<th>( &gt; 80 )</th>
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<td>335</td>
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The structural steel has a modulus of elasticity \( E_a = 210 \, 000 \) MPa (EN 1993-1-1, 3.2.6). In order to avoid any lamellar tearing, the steel has a though-thickness ductility quality Z15 for the main web (when a transverse bracing is welded to it) according to EN 1993-1-10 and EN 10164.

2.3.2.2 Concrete

Normal concrete C35/45 is used for the reinforced slab. The main mechanical properties are as follows (EN 1992-1-1, 3.1.2, Table 3.1):

- Characteristic compressive cylinder strength at 28 days: \( f_{ck} = 35 \) MPa
- Mean value of axial tensile strength: \( f_{cm} = -3.2 \) MPa
- 5% fractile of the characteristic axial tensile strength: \( f_{ck,0.05} = -2.2 \) MPa
- 95% fractile of the characteristic axial tensile strength: \( f_{ck,0.95} = -4.2 \) MPa
- Mean value of concrete cylinder strength at 28 days: \( f_{cm} = f_{ck} + 8 = 43 \) MPa
- Modulus of elasticity: \( E_{cm} = 22 \, 000 \, (f_{cm} / 10)0.3 = 34 \, 077 \) MPa
<table>
<thead>
<tr>
<th><strong>EN 1992-1-1, 3.2 and Annex C, Reinforcing steel</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EN 1994-2, 3.2(2), Reinforcing steel</strong></td>
</tr>
<tr>
<td>(2) For composite structures, the design value of the modulus of elasticity $E_s$ may be taken as equal to the value for structural steel given in EN 1993-1-1, 3.2.6.</td>
</tr>
<tr>
<td><strong>EN 1992-1-1, 2.4.2.4, Partial safety factors for materials</strong></td>
</tr>
<tr>
<td><strong>EN 1993-2, 6.1 and Table 6.2, General</strong></td>
</tr>
<tr>
<td><strong>EN 1993-2, 7.3(1), Limitations for stress</strong></td>
</tr>
</tbody>
</table>
2.3.2.3 Reinforcement

The reinforcing steel bars used in this project are class B high bond bars with a yield strength $f_{yk} = 500$ MPa (EN 1992-1-1, 3.2 and Annex C).

In EN 1992-1-1 the elasticity modulus of reinforcing steel is $E_s = 200,000$ MPa. However, in order to simplify the modulus used for the structural steel, EN 1994-2 allows the use of $E_s = E_a = 210,000$ MPa which will be done in this project (EN 1994-2, 3.2(2)).

2.3.2.4 Partial safety factors for materials

For Ultimate Limit State (ULS) see Table 2-5.

Table 2-5: Partial safety factors for materials (ULS).

<table>
<thead>
<tr>
<th>Design situation</th>
<th>$\gamma_C$ (concrete)</th>
<th>$\gamma_S$ (reinforcing steel)</th>
<th>$\gamma_M$ (structural steel)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Persistent</td>
<td>1.5</td>
<td>1.15</td>
<td>$\gamma_{M0} = 1.0$(Yielding, local instability)</td>
</tr>
<tr>
<td>Transient</td>
<td>1.5</td>
<td>1.15</td>
<td>$\gamma_{M1} = 1.1$ (Resistance of members to instability)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\gamma_{M2} = 1.25$ (Resistance of joints)</td>
</tr>
<tr>
<td>Reference</td>
<td>EN 1992-1-1, 2.4.2.4.</td>
<td>EN 1993-2, 6.1 and Table 6.1</td>
<td></td>
</tr>
</tbody>
</table>

For Serviceability Limit State (SLS) see Table 2-6.

Table 2-6: Partial safety factors for materials (SLS).

<table>
<thead>
<tr>
<th>$\gamma_C$ (concrete)</th>
<th>$\gamma_S$ (reinforcing steel)</th>
<th>$\gamma_{M,ser}$ (structural steel)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>EN 1992-1-1, 2.4.2.4.</td>
<td>EN 1993-2, 7.3 (1)</td>
<td></td>
</tr>
</tbody>
</table>

2.3.3 Actions

In order to simplify the calculations, only six different load cases have been defined:

1. Self-weight of the structural steel
2. Self-weight of the reinforced concrete slab (with construction phasing, so in fact 16 load cases for the twin-girder bridges and 45 load cases for the box-girder bridge)
3. Self-weight of the non structural bridge equipments
4. Shrinkage
5. Creep
6. Traffic load LM1

Specifications to apply these loads on the bridge are explained below.
### EN 1991-1-1, Table A.4, Construction materials-metals

<table>
<thead>
<tr>
<th>Materials</th>
<th>Density $\gamma$ [kN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>aluminium</td>
<td>27,0</td>
</tr>
<tr>
<td>brass</td>
<td>83,0 to 85,0</td>
</tr>
<tr>
<td>bronze</td>
<td>83,0 to 85,0</td>
</tr>
<tr>
<td>copper</td>
<td>87,0 to 89,0</td>
</tr>
<tr>
<td>iron, cast</td>
<td>71,0 to 72,5</td>
</tr>
<tr>
<td>iron, wrought</td>
<td>76,0</td>
</tr>
<tr>
<td>lead</td>
<td>112,0 to 114,0</td>
</tr>
<tr>
<td>steel</td>
<td>77,0 to 78,5</td>
</tr>
<tr>
<td>zinc</td>
<td>71,0 to 72,0</td>
</tr>
</tbody>
</table>

### EN 1991-1-1, Table A.1, Construction materials-concrete and mortar

<table>
<thead>
<tr>
<th>Materials</th>
<th>Density $\gamma$ [kN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete (see EN 206)</td>
<td></td>
</tr>
<tr>
<td>lightweight</td>
<td></td>
</tr>
<tr>
<td>density class LC 1,0</td>
<td>9,0 to 10,0 $^{(1)}$</td>
</tr>
<tr>
<td>density class LC 1,2</td>
<td>10,0 to 12,0 $^{(2)}$</td>
</tr>
<tr>
<td>density class LC 1,4</td>
<td>12,0 to 14,0 $^{(2)}$</td>
</tr>
<tr>
<td>density class LC 1,6</td>
<td>14,0 to 16,0 $^{(2)}$</td>
</tr>
<tr>
<td>density class LC 1,8</td>
<td>16,0 to 18,0 $^{(2)}$</td>
</tr>
<tr>
<td>density class LC 2,0 normal</td>
<td>18,0 to 20,0 $^{(2)}$</td>
</tr>
<tr>
<td>weight heavy weight</td>
<td>24,0 $^{(2)}$</td>
</tr>
<tr>
<td>mortar</td>
<td></td>
</tr>
<tr>
<td>cement mortar</td>
<td>19,0 to 23,0</td>
</tr>
<tr>
<td>gypsum mortar</td>
<td>12,0 to 18,0</td>
</tr>
<tr>
<td>lime-cement mortar</td>
<td>18,0 to 20,0</td>
</tr>
<tr>
<td>lime mortar</td>
<td>12,0 to 18,0</td>
</tr>
</tbody>
</table>

1) Increase by 1kN/m$^3$ for normal percentage of reinforcing and pre-stressing steel
2) Increase by 1kN/m$^3$ for unhardened concrete

NOTE: See Section 4
2.3.3.1 Permanent loads

Distinction is made for permanent loads between the self-weights of the structural steel girders, of reinforced concrete slab and of non-structural equipments.

2.3.3.1.1 Self-weight

The density of the structural steel is taken as equal to 77 kN/m³ (EN 1991-1-1, Table A-4). For calculation of the internal forces and moments and the stresses for the longitudinal bending global analysis, the self-weights are modelled as follows:

**Twin-girder bridge**

The self-weight of the in-span located transverse cross girders is modelled by a vertical uniformly distributed load of 1300 N/m applied on each main girder (about 12% of the weight of this main girder).

**Box-girder bridge**

The self-weight of the in-span located transverse frames is modelled by a vertical uniformly distributed load of 8000 N/m for the total width of the bridge (about 12.2% of the weight of the full box-girder).

The density of the reinforced concrete is taken as equal to 25 kN/m³ (EN 1991-1-1, Table A-1).

2.3.3.1.2 Non-structural equipment

The nominal value of the waterproofing layer is multiplied by ±20% and the nominal value of the asphalt layer by +40% / -20% for all spans (EN 1991-1-1, 5.2.3).

**Table 2-7: Loads of the non-structural equipment (twin-girder bridge).**

<table>
<thead>
<tr>
<th>Item</th>
<th>Characteristics</th>
<th>Maximum multiplier</th>
<th>Minimum multiplier</th>
<th>( q_{\text{nom}} ) (kN/m)</th>
<th>( q_{\text{max}} ) (kN/m)</th>
<th>( q_{\text{min}} ) (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterproofing layer</td>
<td>3 cm thick, 25 kN/m³</td>
<td>1.2</td>
<td>0.8</td>
<td>4.2</td>
<td>5.04</td>
<td>3.36</td>
</tr>
<tr>
<td>Asphalt</td>
<td>8 cm thick, 25 kN/m³</td>
<td>1.4</td>
<td>0.8</td>
<td>11</td>
<td>15.4</td>
<td>8.8</td>
</tr>
<tr>
<td>Concrete support for the safety barrier</td>
<td>area 0.5 x 0.2 m, 25 kN/m³</td>
<td>1</td>
<td>1</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Safety barriers</td>
<td>65 kg/m</td>
<td>1</td>
<td>1</td>
<td>0.638</td>
<td>0.638</td>
<td>0.638</td>
</tr>
<tr>
<td>Cornice</td>
<td>25 kg/m</td>
<td>1</td>
<td>1</td>
<td>0.245</td>
<td>0.245</td>
<td>0.245</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>18.58</strong></td>
<td><strong>23.82</strong></td>
<td><strong>15.54</strong></td>
</tr>
</tbody>
</table>

**Table 2-8: Loads of the non-structural equipment (box-girder bridge).**

<table>
<thead>
<tr>
<th>Item</th>
<th>Characteristics</th>
<th>Maximum multiplier</th>
<th>Minimum multiplier</th>
<th>( q_{\text{nom}} ) (kN/m)</th>
<th>( q_{\text{max}} ) (kN/m)</th>
<th>( q_{\text{min}} ) (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterproofing layer</td>
<td>3 cm thick, 25 kN/m³</td>
<td>1.2</td>
<td>0.8</td>
<td>7.66</td>
<td>9.19</td>
<td>6.13</td>
</tr>
<tr>
<td>Asphalt</td>
<td>8 cm thick, 25 kN/m³</td>
<td>1.4</td>
<td>0.8</td>
<td>20.22</td>
<td>28.31</td>
<td>16.18</td>
</tr>
<tr>
<td>Safety barrier ground girder</td>
<td>area 0.5 x 0.2 m, 25 kN/m³</td>
<td>1</td>
<td>1</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Safety barrier</td>
<td>65 kg/m</td>
<td>1</td>
<td>1</td>
<td>0.64</td>
<td>0.64</td>
<td>0.64</td>
</tr>
<tr>
<td>Cornice</td>
<td>25 kg/m</td>
<td>1</td>
<td>1</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>31.26</strong></td>
<td><strong>40.88</strong></td>
<td><strong>25.68</strong></td>
</tr>
</tbody>
</table>
EN 1991-1-1, 5.2.3, Additional provisions specific for bridges

(1) The upper and lower characteristic values of densities for non structural parts, such as ballast on railway bridges, or fill above buried structures such as culverts, should be taken into account if the material is expected to consolidate, become saturated or otherwise change its properties, during use.

*NOTE: Suitable values may be given in the National Annex.*

(2) The nominal depth of ballast on railway bridges should be specified. To determine the upper and lower characteristic values of the depth of ballast on railway bridges a deviation from the nominal depth of ±30 % should be taken into account.

*NOTE: A suitable value may be given in the National Annex*

(3) To determine the upper and lower characteristic values of self-weight of waterproofing, surfacing and other coatings for bridges, where the variability of their thickness may be high, a deviation of the total thickness from the nominal or other specified values should be taken into account. Unless otherwise specified, this deviation should be taken equal to ± 20 % if a post-execution coating is included in the nominal value, and to + 40 % and – 20 % if such a coating is not included.

*NOTE: Suitable specifications may be given in the National Annex.*

(4) For the self-weight of cables, pipes and service ducts, the upper and lower characteristic values should be taken into account. Unless otherwise specified, a deviation from the mean value of the self-weight of ± 20 % should be taken into account.

*NOTE: Suitable specifications may be given in the National Annex. See also EN 1990, 4.1.2(4)*

(5) For the self-weight of other non structural elements such as:

- hand rails, safety barriers, parapets, kerbs and other bridge furniture,
- joints/fasteners,
- void formers,

the characteristic values should be taken equal to the nominal values unless otherwise specified.

*NOTE: Suitable specifications may be given in the National annex. An allowance for voids filling with water may be made depending on the project.*
Figure 2-16 details the non-structural bridge equipment used for the example.

![Bridge Equipment Diagram](image)

**Figure 2-16: Non-structural bridge equipment details.**

### 2.3.3.2 Concrete shrinkage

According to Eurocode 4, three different concrete shrinkage deformations should be considered in the design. In order to simplify the analysis and because thermal shrinkage at early age is part of national choices, it has been decided not to consider it in the calculations. Only autogenous and drying shrinkage deformations ($\varepsilon_{cs} = \varepsilon_{ca} + \varepsilon_{cd}$ with notations from EN 1992-1-1, 3.1.4(6)) have been taken into account.

Two values of the total deformation $\varepsilon_{cs}$ have been then calculated:

**Shrinkage deformation for persistent design situation at traffic opening**

(for the twin-girder bridge date $t_{ini} = 80$ days, for the box-girder bridge date $t_{ini} = 170$ days)

Table 2-9 summarises the values of autogenous and drying shrinkage deformation for persistent design situation at traffic opening for the twin-girder bridge and the box-girder bridge respectively.

<table>
<thead>
<tr>
<th></th>
<th>twin-girder bridge</th>
<th>box-girder bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Autogenous shrinkage</strong></td>
<td>4.88E-05</td>
<td>5.44E-05</td>
</tr>
<tr>
<td><strong>Drying shrinkage</strong></td>
<td>1.36E-05</td>
<td>2.23E-05</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>6.20E-05</td>
<td>7.67E-05</td>
</tr>
</tbody>
</table>

**Shrinkage deformation for persistent design situation at infinite time**

Table 2-10 summarises the values of autogenous and drying shrinkage deformation for persistent design situation at infinite time for the twin-girder bridge and the box-girder bridge.
EN 1994-2, 5.4.2.2 (2), Creep and shrinkage

(2) Except for members with both flanges composite, the effects of creep may be taken into account by using modular ratios \( n_L \) for the concrete. The modular ratios depending on the type of loading (subscript \( L \)) are given by:

\[
n_L = n_0 \left( 1 + \psi_L / \varphi_c \right)
\]  

(5.6)

where:

- \( n_0 \) is the modular ratio \( E_a / E_{cm} \) for short-term loading;
- \( E_{cm} \) is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1, Table 3.1 or Table 11.3.1;
- \( \varphi_c \) is the creep coefficient \( \varphi \left( t, t_0 \right) \) according to EN 1992-1-1, 3.1.4 or 11.3.3, depending on the age \( t \) of concrete at the moment considered and the age \( t_0 \) at loading;
- \( \psi_L \) is the creep multiplier depending on the type of loading, which be taken as 1.1 for permanent loads, 0.55 for primary and secondary effects of shrinkage and 1.5 for prestressing by imposed deformations.
Table 2-10: Shrinkage at infinite time.

<table>
<thead>
<tr>
<th></th>
<th>( t = \text{infinite} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Autogenous shrinkage</td>
<td>6.25E-05</td>
</tr>
<tr>
<td>Drying shrinkage</td>
<td>1.77E-04</td>
</tr>
<tr>
<td>Total</td>
<td>2.40E-04</td>
</tr>
</tbody>
</table>

Finally for calculating the internal forces and moments for the persistent design situation at traffic opening, a shrinkage deformation of \( 6.2 \cdot 10^{-5} \) (for the twin-girder-bridge) and \( 7.67 \cdot 10^{-5} \) (for the box-girder-bridge) is applied to each slab segment following the concreting order. For the persistent design situation at infinite time, a shrinkage deformation of \( 2.4 \cdot 10^{-4} \) (for the 2 bridges) is applied to the whole slab after finishing all concreting phases.

2.3.3.3 Creep – Modular ratios

2.3.3.3.1 Modular ratio for short-term loading

\[
n_b = \frac{E_f}{E_{cm}} = \frac{210000}{22000 \left( \frac{t_0}{10} \right)^{0.3}} = 6.1625
\]

2.3.3.3.2 Modular ratio for long-term loading

For a given long-term loading \( L \) applied to the bridge when the mean age of concrete is equal to \( t_0 \), the modular ratio is defined by the following equation for the calculations of the bridge at infinite time (EN 1994-2, 5.4.2.2(2)):

\[
n_L = n_b \left( 1 + \psi_L \phi \left( \infty, t_0 \right) \right)
\]

Table 2-11 and Table 2-12 summarise the intermediate values for the calculation of the creep factor \( \phi \left( \infty, t_0 \right) \) and the modular ratio values \( n_L \) used in the design of the twin-girder bridge and the box-girder bridge. See Table 2-1 and Table 2-2 for further details on the ages \( t_0 \).

Table 2-11: Modular ratio for long-term loading (twin-girder bridge).

<table>
<thead>
<tr>
<th>Load case</th>
<th>( \psi_L )</th>
<th>( t_0 ) (days)</th>
<th>( \phi \left( \infty, t_0 \right) )</th>
<th>( n_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concreting</td>
<td>1.1</td>
<td>24</td>
<td>1.484</td>
<td>16.22</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>0.55</td>
<td>1</td>
<td>2.683</td>
<td>15.25</td>
</tr>
<tr>
<td>Non structural bridge equipments</td>
<td>1.1</td>
<td>57.5</td>
<td>1.256</td>
<td>14.68</td>
</tr>
</tbody>
</table>

Table 2-12: Modular ratio for long-term loading (box-girder bridge).

<table>
<thead>
<tr>
<th>Load case</th>
<th>( \psi_L )</th>
<th>( t_0 ) (days)</th>
<th>( \phi \left( \infty, t_0 \right) )</th>
<th>( n_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concreting</td>
<td>1.1</td>
<td>67.5</td>
<td>1.215</td>
<td>14.40</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>0.55</td>
<td>1</td>
<td>2.674</td>
<td>15.23</td>
</tr>
<tr>
<td>Non structural bridge equipments</td>
<td>1.1</td>
<td>104</td>
<td>1.118</td>
<td>13.74</td>
</tr>
</tbody>
</table>
EN 1991-2, 4.2.3, Divisions of the carriageway into notional lanes

(1) The carriageway width, \( w \), should be measured between kerbs or between the inner limits of vehicle restraint systems, and should not include the distance between fixed vehicle restraint systems or kerbs of a central reservation nor the widths of these vehicle restraint systems.

NOTE: The National Annex may define the minimum value of the height of the kerbs to be taken into account. The recommended minimum value of this height is 100 mm.

(2) The width \( w_l \) of notional lanes on a carriageway and the greatest possible whole (integer) number \( n_l \) of such lanes on this carriageway are defined in Table 4.1.

### Table 4.1: Number and width of notional lanes

<table>
<thead>
<tr>
<th>Carriageway width ( w )</th>
<th>Number of notional lanes ( n_l )</th>
<th>Width of a notional lane ( \ell_l )</th>
<th>Width of the remaining area ( w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w &lt; 5.4 ) m</td>
<td>( n_l = 1 )</td>
<td>3 m</td>
<td>( w - 3 ) m</td>
</tr>
<tr>
<td>( 5.4 \leq w &lt; 6 ) m</td>
<td>( n_l = 2 )</td>
<td>( w )</td>
<td>0</td>
</tr>
<tr>
<td>( 6 \leq w ) m</td>
<td>( n_l = \text{Int} \left( \frac{w}{3} \right) )</td>
<td>3 m</td>
<td>( w - 3 \times n_l )</td>
</tr>
</tbody>
</table>

NOTE: For example, for a carriageway width equal to 11 m, \( n_l = \text{Int} \left( \frac{11}{3} \right) = 3 \), and the width of the remaining area is \( 11 - 3 \times 3 = 2 \) m.

(3) For variable carriageway widths, the number of notional lanes should be defined in accordance with the principles used for Table 4.1.

NOTE: For example, the number of notional lanes will be:

- 1 where \( w < 5.4 \) m
- 2 where \( 5.4 \leq w < 9 \) m
- 3 where \( 9 \leq w < 12 \) m, etc.

(4) Where the carriageway on a bridge deck is physically divided into two parts separated by a central reservation, then:

(a) each part, including all hard shoulders or strips, should be separately divided into notional lanes if the parts are separated by a permanent road restraint system;

(b) the whole carriageway, central reservation included, should be divided into notional lanes if the parts are separated by a temporary road restraint system.

NOTE: The rules given in 4.2.3(4) may be adjusted for the individual project, allowing for envisaged future modifications of the traffic lanes on the deck, e.g. for repair.
The hyperstatic effects (also called “secondary effects” in Eurocode 4) of creep are negligible compared to the other action effects. So they have not been considered in the calculations (and the corresponding modular ratio is not mentioned here).

2.3.3.4 Traffic loads

2.3.3.4.1 Adjustment coefficients

The definition of the LM1 vertical loads (consisting of the tandem system TS and the uniformly distributed load UDL) includes a series of adjustment coefficients $\alpha_{Q_i}$, $\alpha_{q_i}$ and $\alpha_{Q_r}$. These coefficients are given in the National Annex of each country. Only minimum recommended values are given in EN 1991-2. Here, the following values have been assumed (coming from the French National Annex to EN 1991-2, for a highway or motorway traffic category) (EN 1991-2, 4.3.2 (3)):

<table>
<thead>
<tr>
<th>Lane no.</th>
<th>$\alpha_{Q_i}$ (for TS)</th>
<th>$\alpha_{q_i}$ (for UDL)</th>
<th>$\alpha_{Q_r}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.9</td>
<td>0.7</td>
<td>/</td>
</tr>
<tr>
<td>2 or more</td>
<td>0.8</td>
<td>1.0</td>
<td>/</td>
</tr>
<tr>
<td>Remaining area</td>
<td>/</td>
<td>/</td>
<td>1</td>
</tr>
</tbody>
</table>

### Table 2-13: Adjustment coefficients for LM1.

**2.3.3.4.2 Transverse positioning of LM1**

UDL and TS are positioned longitudinally and transversally on the deck so as to achieve the most unfavourable effect for the studied main girder (girder no. 1 in Figure 2-17) and for the box-girder.

**Twin-girder bridge**

A straight transverse influence line is used (see Figure 2-20 and Figure 2-22) with the assumption that a vertical load introduced in the web plane of a main girder is entirely resisted by this girder. The unfavourable parts of each longitudinal influence line are then loaded according to the transverse distribution of the traffic vertical loads UDL and TS between the two main girders.

![Figure 2-17: Traffic lanes positioning for calculating the girder no.1.](image)
The pavement width between the internal vertical faces of the concrete longitudinal supports of the safety barriers reaches $w = 11$ m, centered on the deck axis. Three traffic lanes each 3 m wide and a 2 m wide remaining area can be placed within this width. The traffic lanes are thus arranged in the most unfavourable way for the studied girder no. 1 according to the diagram in Figure 2-20 (EN 1991-2, 4.2.3).

**Box-girder bridge**

A horizontal straight transverse influence line is used (see Figure 2-21 and Figure 2-23). It corresponds to the assumption that sufficient stiffening is provided to prevent the deformation of the cross-sections. The eccentric traffic loads $Q$ (TS) and $q$ (UDL) are modeled by using centered loads with the same values $Q$ and $q$, and torque loads ($M_Q$ for the concentrated one and $m_q$ for the distributed one, see Figure 2-18). Torque has been studied and its calculation is explained in Paragraph 2.3.3.5.

![Figure 2-18: Calculation of the box-girder for eccentric concentrated load.](image)

The pavement width between internal vertical faces of the concrete longitudinal supports of the external safety barriers reaches 20.22 m, centered on the deck axis. The safety barrier, which separates the two ways of the road in the middle of the deck, has not been considered when applying the traffic load model 1 on this 20.22 m wide pavement.

Six traffic lanes each 3.00 m wide and a 2.22 m wide remaining area can be placed within this 20.22 m wide pavement. The transverse positioning of traffic lanes doesn’t matter for studying the effect of the centered vertical loads $Q$ and $q$, as each load is resisted equally by the two webs. This latter assumption corresponds to the use of the horizontal transverse influence line in Figure 2-21 and Figure 2-23, with the 0.5 imposed value at each main web location. On the contrary this transverse positioning of the traffic lanes influences the results of the torque global analysis. Figure 2-19 shows the most unfavourable lane distribution when studying torque.

![Figure 2-19: Traffic lanes positioning for calculating the box-girder.](image)
EN 1991-2, 4.3.2(1) (a), Load Model 1

(1) Load Model 1 consists of two partial systems:

(a) Double-axle concentrated loads (tandem system: TS), each axle having the following weight:

\[ \alpha_q Q_k \]

(4.1)

where:
\[ \alpha_q \] are adjustment factors.

- No more than one tandem system should be taken into account per notional lane.
- Only complete tandem systems should be taken into account.
- For the assessment of general effects, each tandem system should be assumed to travel centrally along the axes of notional lanes (see (5) below for local verifications and Figure 4.2b).
- Each axle of the tandem system should be taken into account with two identical wheels, the load per wheel being therefore equal to \(0.5 \alpha_q Q_k\).
- The contact surface of each wheel should be taken as square and of side 0.40 m (see Figure 4.2b).

(b) Uniformly distributed loads (UDL system), having the following weight per square metre of notional lane:

\[ \alpha_q q_k \]

(4.2)

where:
\[ \alpha_q \] are adjustment factors.

The uniformly distributed loads should be applied only in the unfavourable parts of the influence surface, longitudinally and transversally.

**NOTE:** LM1 is intended to cover flowing, congested or traffic jam situations with a high percentage of heavy lorries. In general, when used with the basic values, it covers the effects of a special vehicle of 600 kN as defined in Annex A.

(4) The characteristic values of \(k Q\) and \(k q\), dynamic amplification included, should be taken from Tab 4.2.

**Table 4.2: Load model 1: characteristic values**

<table>
<thead>
<tr>
<th>Location</th>
<th>Tandem system TS</th>
<th>UDL system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axle loads (Q_{ik}) (kN)</td>
<td>(q_{ik}) (or (q_{ik}')) (kN/m²)</td>
</tr>
<tr>
<td>Lane Number 1</td>
<td>300</td>
<td>9</td>
</tr>
<tr>
<td>Lane Number 2</td>
<td>200</td>
<td>2.5</td>
</tr>
<tr>
<td>Lane Number 3</td>
<td>100</td>
<td>2.5</td>
</tr>
<tr>
<td>Other lanes</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>Remaining area (q_{ik}')</td>
<td>0</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Tandem System TS

Each axle of the tandem TS has to be centered in its traffic lane. The vertical load magnitudes per axle are given in EN 1991-2, Table 4.2. Figure 2-20 indicates the transverse position of the three tandems considered with respect to the main structural steel girders (EN 1991-2, 4.3.2(1) (a)).

**Figure 2-20: Tandem TS loading on the deck for the twin-girder bridge.**

**Figure 2-21: Tandem TS loading on the deck for the box-girder bridge.**

Each traffic lane can only support a single tandem TS in the longitudinal direction. The three used tandem TS (one per lane) could not be necessarily located in the same transverse cross-section.
Uniformly Distributed Load UDL

For the twin-girder-bridge, the traffic lanes are loaded with UDL up to the axis of girder no. 2 (see Figure 2-22) i.e. the positive part of the transverse influence line. For the box-girder bridge, given the transverse influence line, the whole pavement width is loaded with UDL. The vertical load magnitudes of UDL are given in EN 1991-2, Table 4.2 (EN 1991-2, 4.3.2(1) (b)).

![Figure 2-22: UDL tranverse distribution on the bridge deck for the twin-girder bridge.](image)

**NOTE:** that if lane no. 3 extended beyond the axis of main girder no. 2 it would only be partly loaded in the positive zone of the transverse influence line.

![Figure 2-23: UDL tranverse distribution on the bridge deck for the box-girder bridge.](image)

### 2.3.3.4.3 Longitudinal positioning of LM1

The software used to calculate the internal forces and moments automatically moves along the bridge the part of the traffic loads TS and UDL which transversally comes into the modelled girder no. 1 (the most loaded one according to the analysis in the Paragraph 2.3.3.4.2, see Figure 2-30). It directly gives the envelopes of bending moments and shear forces for:

- the characteristic value of LM1: \(1.0 \cdot \text{UDL} + 1.0 \cdot \text{TS}\)
- the frequent value of LM1: \(0.4 \cdot \text{UDL} + 0.75 \cdot \text{TS}\)
EN 1994-2, 5.4.2.2(11), Creep and shrinkage

(11) The St. Venant torsional stiffness of box-girders should be calculated for a transformed cross-section in which the concrete slab thickness is reduced by the modular ratio \( n_{0G} = G_a/G_c \) where \( G_a \) and \( G_c \) are the elastic shear moduli of structural steel and concrete respectively. The effects of creep should be taken into account in accordance with (2) with the modular ratio
\[
 n_{LG} = n_{0G} \left( 1 + \psi_L \varphi_L \right).
\]

EN 1994-2, 5.4.2.3(6), Effects of cracking of concrete

(6) The torsional stiffness of box-girders should be calculated for a transformed cross-section. In areas where the concrete slab is assumed to be cracked due to bending and where membrane shear stresses are so large that shear reinforcement is required, the calculation should be performed considering a slab thickness reduced to one half, unless the effect of cracking is considered in a more precise way.
2.3.3.5 Torque

All the loads applied on the box-girder bridge are transversally symmetrical, except the traffic loads. So the torque in the sections will be only produced by LM1. Considering the vertical loads applied on the left hand side of the longitudinal axis of the deck:

- for TS, the torque moment (for one axle per tandem) is equal to:
  \[ 270 \text{ kN} \times 8.61 \text{ m} + 160 \text{ kN} \times 5.61 \text{ m} + 80 \text{ kN} \times 2.61 \text{ m} = 3431 \text{ kN.m} \]
- for UDL, the linear torque moment is equal to:
  \[ 18.9 \text{ kN/m} \times 8.61 \text{ m} + 17.8 \text{ kN/m} \times 3.55 \text{ m} = 226 \text{ kN.m/m}. \]

As already mentioned in Paragraph 2.3.3.4.2 for the longitudinal bending, the software automatically moves along the bridge the traffic loads TS and UDL to calculate the most unfavourable torsional moment in each section. The influence of the thickness variations on the location of the shear centre has been neglected. So every section of the bridge is assumed having the same shear centre.

The St. Venant torsional stiffness of each box section has been calculated by using the following formula:

\[ I_t = \frac{4\Omega^2}{\int l/e} \]

\( \Omega \) represents the area bordered by the mid-planes of the internal elements of the box cross-section.

For the concrete element, the thickness \( e \) is divided by the modular ratio \( n_{0G} \) for short-term loading according to EN 1994-2, 5.4.2.2 (11). If the section is located in a cracked zone of the global bending analysis, the slab thickness is reduced to one half to take the effect of cracking into account (EN 1994-2 5.4.2.3 (6)).

For the section located in the middle of the bridge \((x = 270 \text{ m})\), it gives:

\[ \Omega = \frac{6.5 + 12}{2} = 4.15 = 38.4 \text{ m}^2 \]

\[ n_{0G} = n_0 \frac{1 + \nu}{1 + \nu} = 6.1625 \frac{1.3}{1.2} = 5.69 \]

lower flange: \( \frac{l}{e} = \frac{6.5}{0.025} = 260 \)

each web: \( \frac{l}{e} = \frac{4.8}{0.018} = 266 \)

each upper composite flange: \( \frac{l}{e} = \frac{0.75}{0.05 + 0.325/5.69} = 7.0 \)

upper flange with only concrete: \( \frac{l}{e} = \frac{10.5}{0.325/5.69} = 184 \)

\[ \sum \frac{l}{e} = 990 \]

Thus, this section gives: \( I_t = \frac{4\Omega^2}{\int l/e} = 5.96 \text{ m}^4 \).
EN 1990, 4.1.2, Characteristic values of actions

EN 1990, 6.4.3.2(3), Combinations of actions for persistent or transient design situations (fundamental combinations)

(3) The combination of actions in brackets { }, in (6.9b) may either be expressed as:

\[
\sum_{j=1}^{n} \gamma_{G,j} G_{k,j} + " \gamma_P P" + " \gamma_Q Q_{k,j} + " \sum_{i=1}^{n} \gamma_{Q,i} Q_{0,i,j}
\]

or, alternatively for STR and GEO limit states, the less favourable of the two following expressions:

\[
\sum_{j=1}^{n} \gamma_{G,j} G_{k,j} + " \gamma_P P" + " \gamma_Q Q_{k,j} + " \sum_{i=1}^{n} \gamma_{Q,i} Q_{0,i,j}
\]

\[
\sum_{j=1}^{n} \xi \gamma_{G,j} G_{k,j} + " \gamma_P P" + " \gamma_Q Q_{k,j} + " \sum_{i=1}^{n} \gamma_{Q,i} Q_{0,i,j}
\]

where:

" + " implies "to be combined with"

\( \sum \) implies "the combined effect of"

\( \xi \) is a reduction factor for unfavourable permanent actions \( G \)

EN 1990, Annex 2, Table A2.4(B), Design values of actions (STR/GEO) (Set B)

<table>
<thead>
<tr>
<th>Permanent and Transient Design Situation</th>
<th>Permanent actions (Unfavourable)</th>
<th>Permanent actions (Favourable)</th>
<th>Dead loads (Unfavourable)</th>
<th>Dead loads (Favourable)</th>
<th>Leading variable action (%)</th>
<th>Accompanying variable action (%)</th>
<th>Permanent actions (Unfavourable)</th>
<th>Permanent actions (Favourable)</th>
<th>Permanent actions (Unfavourable)</th>
<th>Permanent actions (Favourable)</th>
<th>Leading variable action (%)</th>
<th>Accompanying variable action (%)</th>
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<tbody>
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<td>0.005%</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The classes between 6.10e or 6.10f and 6.10g will be in the National Annex. In case of 6.10f and 6.10g, the National Annex may include addition not included in the permanent actions only.

NOTE 2 The values for \( \gamma \) values may be set by the National Annex. The following values for \( \gamma \) are recommended when using expressions 6.10, or 6.10a and 6.10b:

\( \gamma = 1.35 \)

\( \gamma = 1.00 \)

\( \gamma = 1.50 \) when \( G \) represents unfavourable actions due to wind or traffic load (only when favourable)

\( \gamma = 1.15 \) when \( G \) represents unfavourable actions due to wind and traffic load (only when favourable)

\( \gamma = 1.10 \) when \( G \) represents unfavourable actions due to wind or traffic load (only when favourable)

\( \gamma = 1.05 \) when \( G \) represents unfavourable actions due to wind and traffic load (only when favourable)

\( \gamma = 1.00 \) when \( G \) represents unfavourable actions due to wind, traffic, or other variable actions

\( \gamma = 0.85 \) for \( G \) in and building collapse analysis

\( \gamma = 0.80 \) in case of non-linear analysis, for design situations where actions due to earth movements may have unfavourable effects. For design situations where actions due to earth movements may have favourable effects, these actions are not to be taken into account.

See also EN 1991 to EN 1999 for values to be used for unassigned deformations.

See relevant notes and comments in the relevant design clauses.

1 This value covers: self-weight of structural elements, bulwark, guardrail, guardwall and footwater, removable loads, etc.

2 This value covers: variable horizontal earth pressure from soil, earth fill, earth embankment, earth pressure, interaction earth pressure, traffic loading, earth pressure on underground structures, wind and thermal actions, etc.

3 For road traffic actions, the values of 5 and 7 may be applied to individual components of traffic actions associated with SDE and LD, and LD1 and LD2, respectively.

4 The characteristic values of actions from one source are multiplied by \( \gamma \) to the total arising action effect is unfavourable and \( \gamma \) if the total arising action effect is favourable. For example, all actions originating from the soil weight of the structure may be considered as coming from one source, but this applies if different materials are involved, see Annex A2.3.5.

5 For particular ventilations, the values for \( \gamma \) and \( \gamma \) may be subdivided into \( \gamma \) and \( \gamma \) and the model uncertainty factor \( \gamma \). A value of \( \gamma \) in the range 1.0 to 1.15 may be used in most common cases and may be modified in the National Annexes.

NOTE 1 Where actions due to water are not covered by EN 1997 (i.e., flowing water), the combination of actions to be used may be specified by the individual project.
2.3.3.6 Thermal gradient

To simplify the study, the thermal gradient is not addressed in this report. Taken the thermal gradient into account would have had an influence on:

- the value of the stresses at Serviceability Limit State (SLS) due to primary effects of the gradient – but this Design Manual focuses on the ULS buckling phenomena.
- the amplitude of the concrete stress envelope which would have been wider, and consequently the cracked zones of the global analysis, too.

2.3.4 Combinations of actions

2.3.4.1 Design situations

The bridge should be verified for the following design situations:

- Transient design situations:
  - for the structural steel alone under its self-weight (launching steps),
  - during and after concreting of each slab segment (16 or 45 situations for the example following the bridge),
- Permanent design situations:
  - at traffic opening,
  - at infinite time.

The transient design situations, i.e. launching of the structural steel part, are studied in Chapter 4.

Both permanent design situations will be included in the global analysis (through the use of two sets of modular ratios). The verification of the bridge will then be performed once, considering the final envelopes of internal forces and moments.

2.3.4.2 General remarks

The notations used are those of Eurocodes:

- $G_{k,\text{sup}}$: characteristic value of an *unfavourable* permanent action (nominal value of self-weight and maximum value of non-structural equipments) taking account of construction phases
- $G_{k,\text{inf}}$: characteristic value of a *favourable* permanent action (nominal value of self-weight and minimum value of non-structural equipments) taking account of construction phases
- $S$: envelope of characteristic values of internal forces and moments (or deformations) due to concrete shrinkage
- $UDL_k$: envelope of characteristic values of internal forces and moments (or deformations) due to the vertical uniformly distributed loads from Load Model no. 1 in EN 1991-2
- $TS_k$: envelope of characteristic values of internal forces and moments (or deformations) due to the vertical concentrated loads from Load Model no. 1 in EN 1991-2

An envelope calculation with $G_{k,\text{sup}}$ and $G_{k,\text{inf}}$ is necessary for the permanent loads, only because of the variability of the deck surfacing load. The nominal value of the self-weight is considered (EN 1990, 4.1.2).

The combinations of actions indicated below have been established using EN 1990 and its normative Annex A2 "Application for bridges”.

The shrinkage is not taken into account if its effect is favourable.
EN 1990 Annex 2, Table A.2.1, Recommended values of $\psi$ factors for road bridges

<table>
<thead>
<tr>
<th>Action</th>
<th>Symbol</th>
<th>$\psi_0$</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic loads (see EN 1991-2, Table 4.4)</td>
<td>gr1a (LM1+pedestrian or cycle-track loads) (^1)</td>
<td>TS</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UDL</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>gr1b (Single axle)</td>
<td></td>
<td>0</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>gr2 (Horizontal Forces)</td>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>gr3 (Pedestrian loads)</td>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>gr4 (LM4 – Crowd loading))</td>
<td></td>
<td>0</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>gr5 (LM3 – Special vehicles))</td>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind forces $F_{gr}$</td>
<td>Persistent design situations</td>
<td>0.6</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Execution</td>
<td>0.8</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Thermal actions $T_h$</td>
<td>0.6 (^2)</td>
<td>0.6</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Snow loads $Q_{w,e}$ (during execution)</td>
<td>0.8</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Construction loads $Q_c$</td>
<td>1.0</td>
<td>-</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

1) The recommended values of $\psi_0$, $\psi_1$, $\psi_2$ for gr1a and gr1b are given for roads with traffic corresponding to adjusting factors $\alpha_Q$, $\beta_Q$, $\beta_{QR}$ and $\beta_G$ equal to 1. Those relating to UDL correspond to the most common traffic scenarios, in which an accumulation of lorries can occur, but not frequently. Other values may be envisaged for other classes of routes, or of expected traffic, related to the choice of the corresponding $\alpha$ factors. For example, a value of $\psi_2$ other than zero may be envisaged for the UDL system of LM1 only, for bridges supporting a severe continuous traffic. See also EN 1998.

2) The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a "reduced" value. $\psi_0$ and $\psi_1$ factors are applicable to this value.

3) The recommended $\psi_0$ value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

EN 1990, Annex 2, A2.4.1, General

(1) For serviceability limit states the design values of actions should be taken from Table A2.6 except if differently specified in EN 1991 to EN 1999.

**NOTE 1:** $\gamma$ factors for traffic and other actions for the serviceability limit state may be defined in the National Annex. The recommended design values are given in Table A2.6, with all $\gamma$ factors being taken as 1.0.

Table A.2.6: Design values of actions for use in the combination of actions

<table>
<thead>
<tr>
<th>Combination</th>
<th>Permanent actions $G_4$</th>
<th>Prestress</th>
<th>Variable actions $Q_4$</th>
<th>Unfavourable</th>
<th>Favourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic</td>
<td>$G_{k,sUP}$</td>
<td>$G_{k,inf}$</td>
<td>$P$</td>
<td>$Q_{k,1}$</td>
<td>$\psi_0, Q_{k,1}$</td>
</tr>
<tr>
<td>Frequent</td>
<td>$G_{k,sUP}$</td>
<td>$G_{k,inf}$</td>
<td>$P$</td>
<td>$\psi_1, Q_{k,1}$</td>
<td>$\psi_0, Q_{k,1}$</td>
</tr>
<tr>
<td>Quasi-permanent</td>
<td>$G_{k,sUP}$</td>
<td>$G_{k,inf}$</td>
<td>$P$</td>
<td>$\psi_1, Q_{k,1}$</td>
<td>$\psi_0, Q_{k,1}$</td>
</tr>
</tbody>
</table>

**NOTE 2:** The National Annex may also refer to the infrequent combination of actions.

(2) The serviceability criteria should be defined in relation to the serviceability requirements in accordance with 3.4 and EN 1992 to EN 1999. Deformations should be calculated in accordance with EN 1991 to EN 1999, by using the appropriate combinations of actions according to expressions (6.14a) to (6.16b) (see Table A2.6) taking into account the serviceability requirements and the distinction between reversible and irreversible limit states.

**NOTE:** Serviceability requirements and criteria may be defined as appropriate in the National Annex or for the individual project.
2.3.4.3 **ULS combinations other than fatigue**

\[ 1.35 G_{k,\text{sup}} \text{ (or } G_{k,\text{inf}}) + (1.0 \text{ or } 0.0) S + 1.35 \{ UDL_k + TS_k \} \]

The above-mentioned combination of actions corresponds to Equation (6.10) in EN 1990, 6.4.3.2. Equations (6.10 a) and (6.10 b) have not been used. The \( \gamma \) values have been taken from Table A.2.4 (B) of Annex A2 to EN 1990. The \( \psi_0 \) factors used for defining the combination value of a variable action have been taken from Table A.2.1 of Annex A2 to EN 1990.

2.3.4.4 **SLS combinations**

According to A2.4.1 of Annex A2 to EN 1990, the following combinations should be considered:

- **Characteristic SLS combination:**
  \[ G_{k,\text{sup}} \text{ (or } G_{k,\text{inf}}) + (1.0 \text{ or } 0.0) S + UDL_k + TS_k \]

- **Frequent SLS combination:**
  \[ G_{k,\text{sup}} \text{ (or } G_{k,\text{inf}}) + (1.0 \text{ or } 0.0) S + 0.4 \cdot UDL_k + 0.75 \cdot TS_k \]

- **Quasi-permanent SLS combination:**
  \[ G_{k,\text{sup}} \text{ (or } G_{k,\text{inf}}) + (1.0 \text{ or } 0.0) S \]
EN 1994-2, 5.4.2.3(2), Effects of cracking of concrete

(2) The following method may be used for the determination of the effects of cracking in composite beams with concrete flanges. First the envelope of the internal forces and moments for the characteristic combinations, see EN 1990, 6.5.3, including long-term effects should be calculated using the flexural stiffness $E_a I_1$ of the un-cracked sections. This is defined as “un-cracked analysis”.

In regions where the extreme fibre tensile stress in the concrete due to the envelope of global effects exceeds twice the strength $f_{ctm}$ or $f_{lctm}$, see EN 1992-1-1, Table 3.1 or Table 11.3.1, the stiffness should be reduced to $E_a I_2$, see 1.5.2.12. This distribution of stiffness may be used for ultimate limit states and for serviceability limit states. A new distribution of internal forces and moments, and deformation if appropriate, is then determined by re-analysis. This is defined as “cracked analysis”.
2.4 Global analysis

2.4.1 General

The global analysis is the calculation of the whole bridge to determine the internal forces and moments and the corresponding stresses in all its cross-sections. This is calculated by respecting the defined construction phases and by considering two particular dates in the bridge life – at traffic opening (short term situation) and at infinite time (long-term situation).

According to Eurocode 4, the global analysis of a two-girder bridge is a first order linear elastic analysis, taking into account the construction phases and the cracking of concrete around intermediate supports.

2.4.1.1 Concrete cracking

For the example, it has been chosen to calculate the cracked lengths around internal supports instead of using the simplified “15%-method”. This is achieved by two successive global analysis (EN 1994-2, 5.4.2.3(2)):

- In a first global analysis - called „uncracked analysis“ - the concrete is considered as uncracked for calculating the cross-sectional properties of all the cross-sections in the modelled main girder;
- In a given cross-section if the longitudinal upper fibre tensile stress $\sigma_c$ in the concrete slab is higher than $-2f_{ctm} (= -6.4$ MPa in the example) under characteristic SLS combination of actions, then the concrete of this cross-section should be considered as cracked in the second global analysis. This criterion thus defines cracked zones on both sides of the intermediate supports;
- In a second global analysis - called „cracked analysis“ - the concrete slab stiffness in the cracked zones is reduced to the stiffness of its reinforcing steel. The internal forces and moments - as well as the corresponding stress distributions - of this cracked analysis should be used to verify all the transverse cross-sections of the deck.

See also the chart in Figure 2-28.

2.4.1.2 Shear lag in the concrete slab

The shear lag in the concrete slab is taken into account by reducing the actual slab width to an “effective” width. It thus influences the cross sectional properties of the cross-sections which are used in the global analysis (EN 1994-2, 5.4.1.2).

See also Paragraph 2.4.2.2 for the calculation of effective widths in this project.

2.4.2 Internal forces and moments – Stresses

2.4.2.1 Numerical model

2.4.2.1.1 Twin-girder bridge

To analyse the global longitudinal bending, the deck is modelled as a continuous line of bar elements which corresponds to the neutral fiber of the modelled main girder and which is simply supported on piers and abutments. With respect to a fixed reference (which can be attached e.g. to the final longitudinal profile of the pavement) this neutral fiber changes throughout the calculation according to the cross sectional properties (areas and second moments of area) allocated to the bar elements in the model. This is due to the different modular ratios to be considered and to the fact that a given cross-
EN 1994-2, 5.4.1.2(5), Effective width of flanges for shear lag

At mid-span or an internal support, the total effective width \( b_{\text{eff}} \), see Figure 5.1, may be determined as:

\[
b_{\text{eff}} = b_0 + \sum b_{ei}
\]  

(5.3)

where:

- \( b_0 \) is the distance between the centres of the outstand shear connectors;
- \( b_{ei} \) is the value of the effective width of the concrete flange on each side of the web and taken as \( L_{ei}/8 \) (but not greater than the geometric width \( b_i \)). The value \( b_i \) should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge \( b_i \) is the distance to the free edge. The length \( L_e \) should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, \( L_e \) may be assumed to be as shown in Figure 5.1.
section could be composite or not, with a cracked concrete or not, following the phases of the global analysis. In addition to the cross-sections on internal and end supports and at mid-spans, some particular cross-sections are worthy of being at the bar element ends:

- at the quarter and three-quarters of each span (to define the effective widths of the slab to calculate the stress distribution, see also Paragraph 6.2.2),
- at the ends of every slab concreting segment,
- at the thickness changes in the structural steel distribution.
- in order to have a good precision of the cracked zone length, the bar element length is limited to 1.5 m in the central span and to 1.25 m in the end spans.

Every load case is introduced into the numerical model with the corresponding mechanical properties of the cross-sections.

### 2.4.2.1.2 Box-girder-bridge

**Global analysis of the bending moment**

The box-girder is modelled as a twin-girder bridge whose bottom flange width is half of the bottom flange of the box-section. A half box-girder is studied (i.e. the equivalent of one girder for a twin-girder bridge).

Major differences are the traffic lane positioning and the transverse influence line, already described in Paragraph 2.3.3.4.

To analyse the global longitudinal bending, the deck is modelled as a continuous line of bar elements which corresponds to the neutral fiber of the modelled half box-girder and which is simply supported on piles and abutments. With respect to a fixed reference (which can be attached, for example, to the final longitudinal profile of the pavement) this neutral fiber changes throughout the calculation according to the mechanical properties (areas and second moments of area) allocated to the bar elements in the model. This is due to the different modular ratios to be considered and to the fact that a given cross-section could be composite or not, with a cracked concrete or not, following the phases of the global analysis.

In addition to the cross-sections on internal and end supports and at mid-spans, some peculiar cross-sections are worthy of being at the bar element ends:

- at the quarter and three-quarters of each span (to define the effective widths of the slab to calculate the stress distribution, see also Paragraph 2.4.2.2),
- at the ends of every slab concreting segment,
- at the thickness changes in the structural steel distribution.

Every load case is introduced into the numerical model with the corresponding mechanical properties of the cross-sections.

**Global analysis of the torque**

The first model is a 2D model and only represents a half deck. The torque study requires a 3D model with the definition of the torsional stiffness all along the bridge and of the whole box cross-section of the deck.

### 2.4.2.2 Effective width

#### 2.4.2.2.1 Twin-girder bridge

In a given cross-section of one of the main girder, the effective width of the concrete slab is the sum of 3 terms (see Figure 2-24):

\[ b_{\text{eff}} = b_0 + \beta_1 b_{c1} + \beta_2 b_{c2} \]  

(EN 1994-2, 5.4.1.2 (5))
EN 1994-2, 5.4.1.2(6), Effective width of flanges for shear lag

The effective width at an end support may be determined as:

\[ b_{\text{eff}} = b_0 + \sum \beta_i b_{ei} \]  

with:

\[ \beta_i = (0.55 + 0.025 \frac{L_e}{b_{ei}}) \leq 1.0 \]  

where:

- \( b_{ei} \) is the effective width, see (5), of the end span at mid-span and \( L_e \) is the equivalent span of the end span according to Figure 5.1.

![Figure 5.1: Equivalent spans for effective width of concrete flange](image)

**Key:**

1. \( L_e = 0.85 L_1 \) for \( b_{\text{eff,1}} \)
2. \( L_e = 0.25(L_1 + L_2) \) for \( b_{\text{eff,2}} \)
3. \( L_e = 0.70 L_2 \) for \( b_{\text{eff,1}} \)
4. \( L_e = 2 L_3 \) for \( b_{\text{eff,2}} \)
with

\[ b_0 = (650\,\text{mm for the example}), \text{the center-to-center distance between the outside stud rows;} \]

\[ b_i = \min \left\{ \frac{L_e}{8} ; b_i \right\} \text{where } L_e \text{ is the equivalent span length in the considered cross-section and} \]

where \( b_i \) is the actual geometric width of the slab associated to the main girder;

\[ \beta_1 = \beta_2 = 1 \text{ except for the cross-sections on end supports C0 and C3 where} \]

\[ \beta = 0.55 + 0.025 \cdot \frac{L_e}{b_e} < 1.0 \text{ with } b_e \text{ taken as equal to the effective width at mid-end span} \]

(EN 1994-2, 5.4.1.2 (6)).

\[ L_{e1} = 0.85 \cdot L_1 = 0.85 \cdot 50 = 42.5 \, \text{m for the cross-sections located in the end spans C0-P1} \]

and P2-C3 and for the cross-sections located on end supports C0 and C3 (EN 1994-2, Figure 5.1);

\[ L_{e2} = 0.7 \cdot L_2 = 0.7 \cdot 60 = 42 \, \text{m for the cross-sections located in the central span P1-P2}; \]

\[ L_{e3} = 0.25 \cdot (L_1 + L_2) = 0.25 \cdot (50+60) = 27.5 \, \text{m for the cross-sections located on internal supports P1 and P2}. \]

As \( L_e/8 \) is always greater than \( b_i \) for the example it is deduced that the effective width is equal to the actual width except for the cross-sections on end supports C0 and C3 where the factor \( \beta_1 \) has an impact:

\[ \beta_1 = 0.55 + 0.025 \cdot \frac{L_e}{b_e} = 0.55 + 0.025 \cdot 42.5/3.175 = 0.88 < 1.0, \]

\[ \beta_2 = 0.55 + 0.025 \cdot \frac{L_e}{b_e} = 0.55 + 0.025 \cdot 42.5/2.175 = 1.04 \text{ but as } \beta_2 > 1 \text{ } \beta_2 = 1 \text{ is retained} \]

The slab width will therefore vary linearly from 5.634 m on end support C0 to 6.0 m for the abscissa 0.25\( \cdot L_1 = 12.5 \) m in the span C0-P1 (EN 1994-2, Figure 5.1). Afterwards it will be constant and equal to 6.0 m up to the abscissa 2\( \cdot L_1 + L_2 - 0.25 \cdot L_1 = 147.5 \) m and then it will vary linearly from 6.0 m to 5.634 m on end support C3.
EN 1994-2, 5.4.1.2(4), Effective width of flanges for shear lag

When elastic global analysis is used, a constant effective width may be assumed over the whole of each span. This value may be taken as the value \( b_{\text{eff,1}} \) at mid-span for a span supported at both ends, or the value \( b_{\text{eff,2}} \) at the support for a cantilever.

EN 1994-2, 5.4.1.2(5), Effective width of flanges for shear lag

At mid-span or an internal support, the total effective width \( b_{\text{eff}} \), see Figure 5.1, may be determined as:

\[
b_{\text{eff}} = b_0 + \sum b_{\text{ei}}
\]

(5.3)

where:

- \( b_0 \) is the distance between the centres of the outstand shear connectors;
- \( b_{\text{ei}} \) is the value of the effective width of the concrete flange on each side of the web and taken as \( L_\text{e}/8 \) (but not greater than the geometric width \( b_i \)). The value \( b_i \) should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge \( b_i \) is the distance to the free edge. The length \( L_\text{e} \) should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, \( L_\text{e} \) may be assumed to be as shown in Figure 5.1.

EN 1994-2, 5.4.1.2(6), Effective width of flanges for shear lag

The effective width at an end support may be determined as:

\[
b_{\text{eff}} = b_0 + \sum \beta_i b_{\text{ei}}
\]

(5.4)

with:

\[
\beta_i = (0.55 + 0.025 \frac{L_\text{e}}{b_{\text{ei}}}) \leq 1.0
\]

(5.5)

where:

- \( b_{\text{ei}} \) is the effective width, see (5), of the end span at mid-span and \( L_\text{e} \) is the equivalent span of the end span according to Figure 5.1.

![Figure 5.1: Equivalent spans for effective width of concrete flange](image)

Key:
1. \( L_\text{e} = 0.85 L_1 \) for \( b_{\text{eff,1}} \)
2. \( L_\text{e} = 0.25(L_1 + L_2) \) for \( b_{\text{eff,2}} \)
3. \( L_\text{e} = 0.70 L_2 \) for \( b_{\text{eff,1}} \)
4. \( L_\text{e} = 2 L_3 \) for \( b_{\text{eff,2}} \)
This variable effective width is always taken into account to calculate the longitudinal stress distribution.

To calculate the internal forces and moments with a linear elastic global analysis, constant widths have been used for each span by considering the values at mid-span (EN 1994-2, 5.4.1.2(4)). For the example this means that the calculation can be performed with the actual slab width over the entire bridge length.

\[ b_{\text{eff}} = 6 \text{ m} \]

### 2.4.2.2.2 Box-girder bridge

#### Effective width of the concrete slab

In a given cross-section of one of the main girder, the effective width of the concrete slab is the sum of 3 terms (see Figure 2-25):

\[ b_{\text{eff}} = b_0 + \beta_1 b_{e1} + \beta_2 b_{e2} \quad (\text{EN 1994-2, 5.4.1.2 (5)}) \]

with:

- \( b_0 \) (= 1250 mm for the example), the center-to-center distance between the outside stud rows;
- \( b_{e1} = \min \{L_e/8 ; b_i\} \) where \( L_e \) is the equivalent span length in the considered cross-section and where \( b_i \) is the actual geometric width of the slab associated to the main girder;
- \( \beta_1 = \beta_2 = 1 \) except for the cross-sections on end supports C0 and C5 where \( \beta_i = 0.55 + 0.025 \cdot L_e/b_{ei} < 1.0 \) with \( b_{ei} \) taken as equal to the effective width at mid-end span (EN 1994-2, 5.4.1.2 (6)).

![Figure 2-25: Effective slab width for a main girder in a given cross-section of the box-girder bridge.](image)

The equivalent spans are:

\[ L_{e1} = 0.85 \cdot L_1 = 0.85 \cdot 90 = 76.5 \text{ m} \] for the cross-sections located in the end spans C0-P1 and P4-C5 and for the cross-sections located on end supports C0 and C5 (EN 1994-2, Figure 5.1);

\[ L_{e2} = 0.7 \cdot L_2 = 0.7 \cdot 120 = 84 \text{ m} \] for the cross-sections located in the central spans P1-P2, P2-P3 and P3-P4;

\[ L_{e3} = 0.25 \cdot (L_1 + L_2) = 0.25 \cdot (90+120) = 52.5 \text{ m} \] for the cross-sections located on internal supports P1 and P4.

\[ L_{e4} = 0.25 \cdot (L_2 + L_3) = 0.25 \cdot (120+120) = 60 \text{ m} \] for the cross-sections located on internal supports P2 and P3.
EN 1993-1-5, 3.3 (1), Shear lag at the ultimate limit states

(1) At the ultimate limit states shear lag effects may be determined as follows:

a) elastic shear lag effects as determined for serviceability and fatigue limit states,

b) combined effects of shear lag and of plate buckling,

c) elastic-plastic shear lag effects allowing for limited plastic strains.

NOTE1: The National Annex may choose the method to be applied. Unless specified otherwise in EN 1993-2 to EN 1993-6, the method in NOTE: 3 is recommended.

NOTE2: The combined effects of plate buckling and shear lag may be taken into account by using $A_{\text{eff}}$ as given by:

$$A_{\text{eff}} = A_{c, \text{eff}} \beta_{\text{ult}}$$

where $A_{c, \text{eff}}$ is the effective area of the compression flange due to plate buckling (see 4.4 and 4.5);

$\beta_{\text{ult}}$ is the effective width factor for the effect of shear lag at the ultimate limit state, which may be taken as $\beta$ determined from Table 3.1 with $a_0$ replaced by:

$$\alpha^* = \frac{A_{c, \text{eff}}}{b_0 t_f}$$

$t_f$ is the flange thickness.

NOTE3: Elastic-plastic shear lag effects allowing for limited plastic strains may be taken into account using $A_{\text{eff}}$ as follows:

$$A_{\text{eff}} = A_{c, \text{eff}} \beta x \geq A_{c, \text{eff}} \beta$$

where $\beta$ and $\kappa$ are taken from Table 3.1.

The expressions in NOTE: 2 and NOTE: 3 may also be applied for flanges in tension in which case $A_{c, \text{eff}}$ should be replaced by the gross area of the tension flange.

<table>
<thead>
<tr>
<th>Table 3.1: Effective width factor $\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\kappa$</td>
</tr>
<tr>
<td>$\kappa \leq 0.02$</td>
</tr>
<tr>
<td>$0.02 &lt; \kappa \leq 0.70$</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>$\kappa &gt; 0.70$</td>
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<tr>
<td></td>
</tr>
<tr>
<td>all $\kappa$</td>
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<tr>
<td>all $\kappa$</td>
</tr>
</tbody>
</table>

$\kappa = a_0 b_0 / L_e$ with $\alpha_a = \sqrt{1 + \frac{A_{\text{eff}}}{b_0 t}}$

in which $A_{\text{eff}}$ is the area of all longitudinal stiffeners within the width $b_0$ and other symbols are as defined in Figure 3.1 and Figure 3.2.
As \( L_e/8 \) is always greater than \( b_i \) for the example it is deduced that the effective width is equal to the actual width except for the cross-sections at end supports C0 and C5 where the factor \( \beta_i \) has an impact:

\[
\begin{align*}
\beta_1 &= 0.55 + 0.025 \cdot \frac{L_{e1}}{b_{e1}} = 0.55 + 0.025 \cdot \frac{76.5}{5.375} = 0.906 < 1.0, \\
\beta_2 &= 0.55 + 0.025 \cdot \frac{L_{e1}}{b_{e2}} = 0.55 + 0.025 \cdot \frac{76.5}{4.125} = 1.01 > 1.0 \text{ then } \beta_2 = 1
\end{align*}
\]

The slab width will therefore vary linearly from 10.24 m at end support C0 to 10.75 m for the abscissa \( 0.25 \cdot L_1 = 22.5 \text{ m} \) in the span C0-P1 (EN 1994-2, Figure 5.1). Afterwards it will be constant and equal to 10.75 m up to the abscissa \( 2 \cdot L_1 + 3 \cdot L_2 - 0.25 \cdot L_1 = 517.5 \text{ m} \) and then it will vary linearly from 10.75 m to 10.24 m at end support C5.

This variable effective width is always taken into account to calculate the longitudinal stress distribution.

To calculate the internal forces and moments with a linear elastic global analysis, constant widths have been used for each span by considering the values at mid-span (EN 1994-2, 5.4.1.2(4)). For the example this means that the calculation can be performed with the actual slab width over the entire bridge length.

\[ \Rightarrow \quad b_{\text{eff}} = 10.75 \text{ m} \]

**Effective width of the box-girder bottom flange**

**Global analysis**

For the box-girder bridge global analysis, shear lag is taken into account by an effective width of the steel bottom flange each side of the web which is equal to the smallest of the values between the actual total half width of the steel bottom flange and \( L/8 \) (on each side of the web), where \( L \) is the span length.

In this design example, given the fairly large span lengths, the shear lag effect does not reduce the width at all for the bottom plate.

A bottom flange with a half-width \( b_0 = 3250 \text{ mm} \) gives:

\- for the end spans, \( b_{\text{eff}} = \min (b_0 ; \frac{L_1}{8}) = b_0 \text{ with } L_1 = 90 \text{ m}, \)
\- for the central spans, \( b_{\text{eff}} = \min (b_0 ; \frac{L_2}{8}) = b_0 \text{ with } L_2 = 120 \text{ m}. \)

**Section analysis**

Distinction is made between the shear lag effects for calculating stresses at SLS and at fatigue ULS on one hand, and the shear lag effects for calculating stresses at ULS on the other hand.

ULS stresses are calculated with gross mechanical characteristics (without considering shear lag in the bottom flange and reductions for buckling). Shear lag in the bottom flange is nevertheless described below.

ULS stresses:

At ULS three methods of calculating the effective width for shear lag are proposed in EN 1993-1-5, 3.3, to be chosen by the National Annex. The method recommended in NOTE: 3 of EN 1993-1-5, 3.3(1), is adopted here. The shear lag is then taken into account at ULS via the reduction factor \( \beta^x \). Factors \( \beta \) and \( \kappa \) are given in EN 1993-1-5 Table 3.1.

This method gives values of \( \beta^x \) nearly equal to 1 (higher than 0.97 in every section).

SLS stresses:

Shear lag is taken into account at SLS via the reduction factor \( \beta \), which values are around 0.7 for cross-sections located near the intermediate supports. The corresponding SLS stresses have not been systematically calculated as they do not govern the design of the cross-sections and they are not the objective of this Design Manual.
### 2.4.2.3 Determination of the cracked zones around internal supports

Firstly, a global uncracked analysis is performed for the example. The internal forces and moments as well as the longitudinal stresses \( \sigma \) in the concrete slab are calculated by considering the concrete participation in the bending stiffness of all the cross-sections. Figure 2-26 and Figure 2-27 show the stresses thus obtained under SLS characteristic combination of actions as well as the zones where this stress exceeds \( 2f_{cm} \) in the upper fibre of the concrete slab.

![Figure 2-26: Cracked zones of the twin-girder bridge used in the global analysis.](image-url)

![Figure 2-27: Cracked zones of the box-girder bridge used in the global analysis.](image-url)
EN 1994-2, 5.4.2.2(8)

(8) In regions where the concrete slab is assumed to be cracked, the primary effects due to shrinkage may be neglected in the calculation of secondary effects.
The observed discontinuities in these envelope curves correspond to the end cross-sections of the concreting slab segments and to the cross-sections in which the thicknesses of the structural steel change. Although the bending moment is equal to zero in the cross-sections at the deck ends, the corresponding tensile stresses are not because their values include the self-balancing stresses due to the shrinkage (called “primary effects” or “isostatic effects” in EN 1994-2).

In practical terms, this gives:

**Twin-girder bridge**

- a cracked zone around P1 which starts at the abscissa x = 47.5 m (i.e. 5.0% for the cracked length in the left end span) and which ends at the abscissa x = 53.0 m (i.e. 5.0% for the cracked length in the central span);
- a cracked zone around P2 which starts at the abscissa x = 109.1 m (i.e. 1.5% for the cracked length in the central span) and which ends at the abscissa x = 112 m (i.e. 4.0% for the cracked length in the right end span).

**Box-girder bridge**

- a cracked zone around P1 which starts at the abscissa x = 83.1 m (i.e. 7.7% for the cracked length in the left end span) and which ends at the abscissa x = 98.1 m (i.e. 6.7% for the cracked length in the central span);
- a cracked zone around P2 which starts at the abscissa x = 183.5 m (i.e. 22.1% for the cracked length in the central span) and which ends at the abscissa x = 217.8 m (i.e. 6.5% for the cracked length in the central span);
- a cracked zone around P3 which starts at the abscissa x = 304.1 m (i.e. 21.6% for the cracked length in the central span) and which ends at the abscissa x = 338.8 m (i.e. 7.3% for the cracked length in the central span);
- a cracked zone around P3 which starts at the abscissa x = 444.4 m (i.e. 4.7% for the cracked length in the central span) and which ends at the abscissa x = 457.8 m (i.e. 8.7% for the cracked length in the right end span).

Most of these cracked zones are smaller than 15% of the span lengths, as it would have been directly considered by using the alternative simplified method of EN 1994-2.

They are asymmetric due to the choice made for the order for concreting the slab segments (see Figure 2-5 and Figure 2-11).

### 2.4.2.4 Shrinkage and cracked zones

During the second step of the global analysis, the cracked zones modify the introduction of the concrete shrinkage in the numerical model.

In fact the isostatic (or “primary”) effect of shrinkage \( N_b = E_{cm} \varepsilon_c A_b \) which is applied to the center of gravity of the concrete slab is no longer applied in the cross-sections located in the cracked zones around internal supports (EN 1994-2, 5.4.2.2 (8)).

The “hyperstatic” or “secondary” effect of shrinkage is finally considered as the difference between the internal forces and moments calculated in the continuous girder by the elastic linear analysis under the action of the isostatic effects of shrinkage, and the isostatic effects themselves (see Figure 2-29).

### 2.4.2.5 Organisation of the global analysis calculations

Figure 2-28 shows the sequence considered for the longitudinal bending calculations in the example. This especially includes the changes of cross sectional properties of the cross-sections following the successive introduction of the load cases into the model with respect to the adopted construction phases.
Figure 2.28: Global analysis organisation chart.

- Geometrical Data
  - Spans, transverse cross-sections, slab, structural steel distribution, construction phases
  - Calculation of effective widths (for the global analysis and for the stress calculation)

- Permanent design situation without variable actions
  - at infinite time
  - \( n_{\mu} = 15.25 \)

- Geometrical definition of the model
  - Composite mechanical properties for all the cross-sections

- Geometrical definition of the model
  - Mechanical properties calculated with the structural steel section only

- Geometrical definition of the model
  - Composite mechanical properties for all the cross-sections

- Concrete of slab segment

- Composite mechanical properties for the bar elements in the slab segment

- Construction end

- Self weight + (1 or 0) Shrinkage + Bridge equipment

- Envelope of the two considered permanent design situations

- Traffic load UDL 25 (LM1) applied to calculate the internal forces and moments under SLS characteristic combination of actions

- Determination of the eroded zones on each internal support

- New definition of the model geometry and of the eroded composite mechanical properties (reinforcing steel + structural steel) in the eroded zones

- Permanent design situation without variable actions at traffic opening
  - \( n_{\mu} = 0.10 \)

- Geometrical definition of the model
  - Mechanical properties calculated with the structural steel section only

- Concrete of slab segment

- Composite mechanical properties for the bar elements in the slab segment

- Early age shrinkage applied to slab segment: \( \varepsilon_{sh} = 0.2 \times 10^{-6} \)

- Non-structural bridge equipment

- Self weight + (1 or 0) Shrinkage + Bridge equipment

- Traffic loads (UDL, TS, etc.)

- Combinations – SLS and ULS final envelopes for internal forces and moments
2.4.2.6 Results

2.4.2.6.1 Twin-girder bridge

Figure 2-29 to Figure 2-32 present a few results of internal forces and moments coming from the global analysis of the twin-girder bridge.

Figure 2-29: Isostatic and hyperstatic bending moments due to the long-term concrete shrinkage for the twin-girder bridge.

Figure 2-30: Bending moments under the uniformly distributed load and tandem traffic load (frequent and characteristic LM1) for the twin-girder bridge.
Figure 2-31: Bending moments under the fundamental ULS and characteristic SLS combinations of actions for the twin-girder bridge.

Figure 2-32: Shear forces under the fundamental ULS and characteristic SLS combinations of actions for the twin-girder bridge.
2.4.2.6.2 Box-girder bridge

Figure 2-33 to Figure 2-37 present a few results of internal forces and moments coming from the global analysis of the box-girder bridge.

![Graph showing isostatic and hyperstatic bending moments due to the long-term concrete shrinkage for the box-girder bridge.]

Figure 2-33: Isostatic and hyperstatic bending moments due to the long-term concrete shrinkage for the box-girder bridge.

![Graph showing bending moments under the uniformly distributed load and tandem traffic load (frequent and characteristic LM1) for the box-girder bridge.]

Figure 2-34: Bending moments under the uniformly distributed load and tandem traffic load (frequent and characteristic LM1) for the box-girder bridge.
Figure 2-35: Torque under characteristic LM1 for the box-girder bridge.

Figure 2-36: Bending moments under the fundamental ULS and characteristic SLS combinations of actions for the box-girder bridge.
Figure 2-37: Shear forces under the fundamental ULS and characteristic SLS combinations of actions for the box-girder bridge.
3 Cross-section verifications

3.1 Twin-girder bridge

3.1.1 General

According to the location of the vertical stiffeners of the twin-girder bridge (see Figure 3-1) and the shape of the bending moment and shear diagrams at ULS (see Figure 2-31 and Figure 2-32), the different critical sections to check are displayed in Figure 3-2:

- At the end support C0, see Section 3.1.2
- At mid-span C0-P1, see Section 3.1.3
- At mid-span P1-P2, see Section 3.1.4
- At the internal support P2, see Section 3.1.5

Figure 3-1: Position of vertical stiffeners of the twin-girder bridge.

For each critical section, the verification is done on a panel which is located between two vertical stiffeners. For the internal support P2, the three subpanels shown in Figure 3-3 must be checked.

Figure 3-2: Checked sections of the twin-girder bridge.
3.1.2 Check of cross-section at the end support C0

3.1.2.1 Geometry
At end support C0 at ULS the concrete slab is in compression over its whole height. Its contribution is therefore taken into account in the cross-section resistance.
**General properties of the twin-girder bridge in cross-section C0**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L₁</td>
<td>50 m</td>
</tr>
<tr>
<td>a</td>
<td>8.333 m</td>
</tr>
<tr>
<td>h</td>
<td>2400 mm</td>
</tr>
<tr>
<td>t₀</td>
<td>19 mm</td>
</tr>
<tr>
<td>b₀</td>
<td>800 mm</td>
</tr>
<tr>
<td>b₀f</td>
<td>1000 mm</td>
</tr>
<tr>
<td>tₐ</td>
<td>40 mm</td>
</tr>
<tr>
<td>hₐ</td>
<td>h - 2tₐ = 2.32 m</td>
</tr>
<tr>
<td>e</td>
<td>32.5 cm</td>
</tr>
<tr>
<td>φw₀</td>
<td>16 mm</td>
</tr>
<tr>
<td>φf₀</td>
<td>16 mm</td>
</tr>
<tr>
<td>s₀w</td>
<td>130 mm</td>
</tr>
<tr>
<td>s₀f</td>
<td>130 mm</td>
</tr>
<tr>
<td>c₀w</td>
<td>60 mm</td>
</tr>
<tr>
<td>c₀f</td>
<td>60 mm</td>
</tr>
<tr>
<td>b₀f</td>
<td>6 m</td>
</tr>
<tr>
<td>n₀w</td>
<td>( \frac{b₀f}{s₀w} = 46.154 )</td>
</tr>
<tr>
<td>n₀f</td>
<td>( \frac{b₀f}{s₀f} = 46.154 )</td>
</tr>
</tbody>
</table>

**Main areas of the different parts of the composite section**

\[
A_{w₀f} = t₀ b₀f = 0.032 m²
\]
\[
A_{w₀w} = t₀ h₀ = 0.044 m²
\]
\[
A_{w₀f} = t₀f b₀f = 0.04 m²
\]
\[
A_s = A_{w₀f} + A_{w₀w} + A_{w₀f} = 0.107 m²
\]
\[
A_{w₀w} = \frac{\pi d₀²}{4} = 2.011 cm²
\]
\[
A_{w₀f} = n₀w A_{w₀w} = 92.816 cm²
\]
\[
A_{s₀f} = \frac{\pi d₀²}{4} = 2.011 cm²
\]
\[
A_{s₀w} = n₀f A_{s₀w} = 92.816 cm²
\]
\[
A_{s₀f} = c₀w b₀f = 0.36 m²
\]
\[
A_{s₀w} = (e - c₀w - c₀f)b₀f = 1.23 m²
\]
\[
A_{s₀w} = e b₀f = A_{w₀w} + A_{s₀w} + A_{s₀f} = 1.95 m²
\]

(see notation and Figure 3-4)

### 3.1.2.2 Material properties

**Structural steel**

\[
f_{yw} = 345 \text{ N/mm}² \text{ because } 16 \text{ mm} < t₀ = 19 \text{ mm} \leq 40 \text{ mm (see Table 2-4)}
\]
\[
\varepsilon_w = \sqrt{\frac{235 \text{ N/mm}²}{f_{yw}}} = 0.825
\]
\[
f_{ysf} = 345 \text{ N/mm}² \text{ because } 16 \text{ mm} < t₀ = 40 \text{ mm} \leq 40 \text{ mm (see Table 2-4)}
\]
\[
\varepsilon_f = \sqrt{\frac{235 \text{ N/mm}²}{f_{ysf}}} = 0.825
\]
\[
f_{ydw} = \frac{f_{yw}}{γ_M₀} = 345 \text{ N/mm}²
\]
\[
f_{ysf} = \frac{f_{ysf}}{γ_M₀} = 345 \text{ N/mm}²
\]
\[
E_{sf} = 210000 \text{ N/mm}²
\]


EN 1994-2, 5.5.2(1), Classification of composite sections without concrete encasement

A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class 1 if the spacing of connectors is in accordance with 6.6.5.5.

EN 1994-2, 6.6
Concrete

\[ f_{ck} = 35 \text{ N/mm}^2 \]

\[ f_{cd} = \frac{f_{ck}}{\gamma_c} = 23.333 \text{ N/mm}^2 \]

\[ E_{cm} = 34077 \text{ N/mm}^2 \]

\[ n = \frac{E_{cm}}{E_{cm}} = \frac{210000}{34077} = 6.163 \]

Reinforcement

\[ f_{sk} = 500 \text{ N/mm}^2 \]

\[ f_{sf} = \frac{f_{sk}}{\gamma_s} = 434.734 \text{ N/mm}^2 \]

\[ E_s = E_a = 210000 \text{ N/mm}^2 \]

3.1.2.3 Internal forces and moments

The internal forces and moments in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{Ed} = 26.156 \text{ MNm} \quad \text{(at the end of the panel a = 8.333 m: x = 8.333 m)} \]

\[ V_{Ed} = 3.977 \text{ MN} \quad \text{(at the support C0: x = 0 m)} \]

3.1.2.4 Determination of the cross-section class

- Bottom flange is in tension: no buckling problem
- Top flange is composite and connected to the slab following the recommendations of EN 1994-2, 6.6: Class 1
- To classify the steel web, the position of the Plastic Neutral Axis (PNA) is determined as:
  - Design plastic resistance of the concrete in compression:
    \[ N_c = 0.85A_c \frac{f_{ck}}{\gamma_c} = 38.675 \text{ MN} \]
  - Design plastic resistance of the structural steel top flange:
    \[ N_{atf} = A_{atf} \frac{f_{sf}}{\gamma_{M0}} = 11.04 \text{ MN} \]
  - Design plastic resistance of the structural steel web:
    \[ N_{aw} = A_{aw} \frac{f_{sw}}{\gamma_{M0}} = 15.208 \text{ MN} \]
  - Design plastic resistance of the structural steel bottom flange:
    \[ N_{abf} = A_{abf} \frac{f_{sf}}{\gamma_{M0}} = 13.8 \text{ MN} \]
  - Design plastic resistance of the structural steel:
    \[ N_a = N_{atf} + N_{aw} + N_{abf} = 40.048 \text{ MN} \]
Relations to find the location of the Plastic Neutral Axis (PNA) under positive moment $M_{pl,Rd}$

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{abf} \geq N_{aw} + N_{af} + N_c$</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>$N_{abf} + N_{aw} \geq N_c$ and $N_{abf} &lt; N_{aw} + N_{af} + N_c$</td>
<td>PNA in the web</td>
</tr>
<tr>
<td>$N_a \geq N_{cur} + N_{clur}$ and $N_a &lt; N_c$</td>
<td>PNA in the top flange</td>
</tr>
<tr>
<td>$N_a + N_{sl} \geq N_{cur}$ and $N_a + N_{sl} &lt; N_{cur} + N_{clur}$</td>
<td>PNA in the slab between reinforcements</td>
</tr>
<tr>
<td>$N_a + N_{sl} &lt; N_{cur}$</td>
<td>PNA in the slab above upper reinforcements</td>
</tr>
</tbody>
</table>

EN 1994-2, 6.2.1.2(1), Plastic resistance moment $M_{pl,Rd}$ of a composite cross-section

(1) The following assumptions should be made in the calculation of $M_{pl,Rd}$:

a) there is full interaction between structural steel, reinforcement, and concrete;

b) the effective area of the structural steel member is stressed to its design yield strength $f_{yd}$ in tension or compression;

c) the effective areas of longitudinal reinforcement in tension and in compression are stressed to their design yield strength $f_{sd}$ in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected;

d) the effective area of concrete in compression resists a stress of $0.85 f_{cd}$, constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete, where $f_{cd}$ is the design cylinder compressive strength of concrete.

Typical plastic stress distributions are shown in Figure 6.2.

**Figure 6.2**: Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending
Design plastic resistance of the concrete in compression located above upper reinforcements:

\[ N_{cuw} = 0.85 A_{cuw} \frac{f_{ck}}{\gamma_c} = 7.14 \text{ MN} \]

Design plastic resistance of the concrete in compression located between reinforcements:

\[ N_{clur} = 0.85 A_{clur} \frac{f_{ck}}{\gamma_c} = 24.395 \text{ MN} \]

Design plastic resistance of the concrete in compression located under lower reinforcements:

\[ N_{clr} = 0.85 A_{clr} \frac{f_{ck}}{\gamma_c} = 7.14 \text{ MN} \]

Design plastic resistance of the lower steel reinforcements:

\[ N_{sl} = A_{slr} \frac{f_{ck}}{\gamma_s} = 4.035 \text{ MN} \]

Design plastic resistance of the upper steel reinforcements:

\[ N_{su} = A_{sur} \frac{f_{ck}}{\gamma_s} = 4.035 \text{ MN} \]

Location of the Plastic Neutral Axis (PNA):

\[ N_a = 40.048 \text{ MN} \geq N_c = 38.675 \text{ MN} \]

and \( N_{ad} + N_{aw} = 29.008 \text{ MN} < N_{ad} + N_c = 49.715 \text{ MN} \).

Thus, the PNA is deduced to be located in the top flange at a distance \( z_{pl} \) from the extreme lower fibre of the bottom flange. Writing the force equilibrium around the PNA:

\[ z_{pl} = h - e - \frac{N_u}{0.85 b_{eff} f_{cd}} = 2.398 \text{ m} \]

As the PNA is located in the top flange, the whole web is in tension and therefore in Class 1.

Conclusion: the cross-section at the external support C0 and C3 is in class 1 and is checked by a plastic section analysis.

### 3.1.2.5 Plastic section analysis

#### 3.1.2.5.1 Bending resistance check

The design plastic resistance moment is calculated from the position of the PNA:

\[ M_{pl,Rd} = 57.597 \text{ MNm} \]

Reinforcement in compression in the concrete slab is neglected according EN 1994-2, 6.2.1.2(1).

\[ M_{Ed} = 26.156 \text{ MNm} \leq M_{pl,Rd} = 57.597 \text{ MNm} \]

⇒ Bending resistance is verified!
EN 1993-1-5, 5.1(2)

Plates with $h_w/t$ greater than $\frac{n}{72} \varepsilon$ for an unstiffened web, or $\frac{31}{\eta} \varepsilon \sqrt{k_c}$ for a stiffened web, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports, where $\varepsilon = \sqrt{\frac{235}{f_y[N/mm^2]}}$.

EN 1994-2-5, 6.2.2, Resistance to vertical shear

EN 1993-1-5, 6.2.6, Shear

(2) In the absence of torsion the design plastic shear resistance is given by:

$$V_{pl,Rd} = \frac{A_y f_y}{\sqrt{3} \gamma_{M0}}$$

where $A_y$ is the shear area.

(3) The shear area $A_v$ may be taken as follows:

d) for welded I, H and box sections, load parallel to web: $A_v = \eta \sum (h_w t_w)$

where $h_w$ is the depth of the web; $t_w$ is the web thickness.

EN 1993-1-5, 5.2(1), Design Resistance

For unstiffened or stiffened webs the design resistance for shear should be taken as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_y}{\sqrt{3} \gamma_{M1}} h_w t_w$$

(5.1)

in which the contribution from the web is given by:

$$V_{bw,Rd} = \sum f_y h_w t_w \sqrt{3} \gamma_{M1}$$

(5.2)

and the contribution from the flanges $V_{bf,Rd}$ is according to 5.4.

Comments on the assessment of $k_c$: $k_c$ is the critical coefficient giving the critical shear stress of the plate through the relation:

$$\tau_{cr} = k_c \sigma_c$$

with:

$$\sigma_c = \frac{\pi^2 E}{12 (1-\nu^2)} \frac{t^2}{b^2}$$

$k_c$ can be assessed by several ways, assuming the plate supported and free to rotate at its four edges:

- using Kloppel und Sheer charts
- using EBP Plate software
- using the Annex A of EN 1993-1-5 as follows
Cross-section verifications - Twin-girder bridge at the end support C0

3.1.2.5.2 Shear resistance check

The web should be checked in terms of shear buckling if:

- \( \frac{h_w}{t_w} > \frac{72}{\eta} \varepsilon_y \) for an unstiffened web
- \( \frac{h_w}{t_w} > \frac{31}{\eta} \varepsilon_y \sqrt{k_i} \) for a stiffened web

In this example, the web is stiffened by intermediate vertical stiffeners.

Comments: The stiffening considered here above is provided by intermediate stiffeners. Longitudinal stiffeners are not considered. The web is stiffened at supports.

A beam stiffened by vertical stiffeners only at its supports should be considered as having an unstiffened web to apply this previous criterion.

The vertical stiffeners at the bracing transverse frames which border the webpanel adjacent to the support C0 and located in span C0-P1, are assumed to be rigid (to be checked by using Section 9 of EN1993-1-5). They are equally spaced by \( a = 8.333 \) m.

- \( k_{tiw} = 0 \) because there is no longitudinal stiffeners
- \( a/h_w = 3.592 \geq 1 \)
- \( k_i = 5.34 + 4 \left( \frac{h_w}{a} \right)^2 + k_{tiw} = 5.65 \)
- \( \frac{h_w}{t_w} = 122.105 > \frac{31}{\eta} \varepsilon_y \sqrt{k_i} = 50.679 \) then the web should be checked in terms of shear buckling.

The maximum design shear resistance is given by \( V_{rd} = \min (V_{b,rd}, V_{pl,a,rd}) \)

where \( V_{b,rd} = V_{bw,rd} + V_{bf,rd} = \frac{\chi_y f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} + b_f \frac{f_{yw} f_f}{c \gamma_{M1}} \left( 1 - \left( \frac{M_{Ed}}{M_{f,rd}} \right)^2 \right) \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = 9.578 \) MN

\( V_{pl,a,rd} = \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M0}} = 10.536 \) MN

where \( \eta = 1.2 \) for steel grades up to and including S460
EN 1993-1-5, Annex A3, Shear buckling coefficients

(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient $k_c$ can be obtained as follows:

$$k_c = 5.34 + 4 \left( \frac{h_w}{a} \right)^2 + k_{rst} \text{ when } a/h_w \geq 1$$

$$k_c = 4 + 5.34 \left( \frac{h_w}{a} \right)^2 + k_{rst} \text{ when } a/h_w < 1$$

where $k_{rst} = 9 \left( \frac{h_w}{a} \right)^2 I_{sl} I_{w}^{\frac{1}{2}} t^3 h_w$ but not less than $k_{rst} = \frac{2.1}{t} I_{sl} I_{w}^{\frac{2}{3}} h_w$

$a$ is the distance between transverse stiffeners (see Figure 5.3);

$I_{sl}$ is the second moment of area of the longitudinal stiffener about the $z$-axis, see Figure 5.3 (b). For webs with two or more longitudinal stiffeners, not necessarily equally spaced, $I_{sl}$ is the sum of the stiffness of the individual stiffeners.

Figure 5.3: Web with transverse and longitudinal stiffeners

EN 1993-1-5, 5.3(3)

(3) The slenderness parameter $\lambda_w$ may be taken as follows (for transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both):

$$\lambda_w = \frac{h_w}{37.4t_w e_w \sqrt{k_c}}$$

EN 1993-1-5, 5.3(1)

(1) For webs with transverse stiffeners at supports only and for webs with either intermediate transverse stiffeners or longitudinal stiffeners or both, the factor $\chi_w$ for the contribution of the web to the shear buckling resistance should be obtained from Table 5.1.

Table 5.1: Contribution from the web $\chi_w$ to shear buckling resistance

<table>
<thead>
<tr>
<th>$\lambda_w$</th>
<th>Rigid end post</th>
<th>Non-rigid end post</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tilde{\lambda}_w &lt; 0.83/\eta$</td>
<td>$\eta$</td>
<td>$\eta$</td>
</tr>
<tr>
<td>$0.83/\eta \leq \lambda_w &lt; 1.08$</td>
<td>$0.83/\tilde{\lambda}_w$</td>
<td>$0.83/\tilde{\lambda}_w$</td>
</tr>
<tr>
<td>$\lambda_w \geq 1.08$</td>
<td>$1.37/(0.7 + \tilde{\lambda}_w)$</td>
<td>$0.83/\tilde{\lambda}_w$</td>
</tr>
</tbody>
</table>
Contribution of the web $V_{bw,Rd}$

$$V_{bw,Rd} = \frac{\chi_{w} f_{w} h_{w} t_{w}}{\sqrt{3} \gamma_{M1}}$$

$$\chi_{w} = \frac{h_{w}}{37.4 t_{w} c_{w} \sqrt{k_{r}}} = 1.664 \geq 1.08$$

$$\Rightarrow \chi_{w} = \frac{1.37}{(0.7 + \chi_{w})} = 0.579$$

$$V_{bw,Rd} = \frac{\chi_{w} f_{w} h_{w} t_{w}}{\sqrt{3} \gamma_{M1}} = 4.625 \text{ MN}$$

Contribution from the flanges $V_{bf,Rd}$

$$V_{bf,Rd} = \frac{b_{f} t_{f}^{2} f_{yf}}{c \gamma_{M1}} \left[ 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^{2} \right]$$

$b_{f}$ and $t_{f}$ are taken for the flange which provides the least axial resistance,

$b_{f}$ being taken as not larger than $15 c_{f} t_{f}$ on each side of the web,

The lower flange of the cross-section is a structural steel section whereas its upper flange is a composite section (structural steel + concrete slab + eventually reinforcing steel bars). The formulae for calculating $V_{bf,Rd}$ should be used with the lower steel flange properties.

---

**EN 1993-1-5, 5.4(1), Contribution from flanges**

(1) When the flange resistance is not completely utilized in resisting the bending moment ($M_{Ed} \leq M_{f,Rd}$), the contribution from the flanges should be obtained as follows:

$$V_{bf,Rd} = \frac{b_{f} t_{f}^{2} f_{yf}}{c \gamma_{M1}} \left[ 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^{2} \right] \quad (5.8)$$

$b_{f}$ and $t_{f}$ are taken for the flange which provides the least axial resistance,

$b_{f}$ being taken as not larger than $15 c_{f} t_{f}$ on each side of the web,

$M_{f,Rd} = \frac{M_{f,k}}{\gamma_{M0}}$ is the moment of resistance of the cross-section consisting of the area of the effective composite flanges only,

$$c = a \left( 0.25 + \frac{1.6 b_{f} t_{f}^{2} f_{yf}}{t h_{w}^{2} f_{yw}} \right)$$
EN 1993-1-5, 7.1(1), Interaction between shear force, bending moment and axial force

(1) Provided that \( \eta_h \) (see below) does not exceed 0.5, the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If \( \eta_h \) is more than 0.5 the combined effects of bending and shear in the web of an I or box-girder should satisfy:

\[
\eta_h + \left[ 1 - \frac{M_{f,Rd}}{M_{Pl,Rd}} \right] \left[ 2\eta_h - 1 \right]^2 \leq 1 \quad \text{for} \quad \eta_h \geq \frac{M_{f,Rd}}{M_{Pl,Rd}}
\]  

(7.1)

where \( M_{f,Rd} \) is the design plastic moment of resistance of the section consisting of the effective area of the flanges;

\( M_{Pl,Rd} \) is the design plastic resistance of the cross-section consisting of the effective area of the flanges and the fully effective web irrespective of its section class.

\[
\eta_h = \frac{M_{rd}}{M_{Pl,Rd}} \quad \text{and} \quad \eta_v = \frac{V_{rd}}{V_{fw,Rd}}
\]

In addition the requirements in sections 4.6 and 5.5 should be met.

Action effects should include global second order effects of members where relevant.

---

Relations to find the location of the Plastic Neutral Axis (PNA) under positive moment \( M_{Pl,Rd} \)

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_{abf} \geq N_{aw} + N_{af} + N_c )</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>( N_{abf} + N_{aw} \geq N_{af} + N_c ) and ( N_{abf} &lt; N_{aw} + N_{af} + N_c )</td>
<td>PNA in the web</td>
</tr>
<tr>
<td>( N_e \geq N_c ) and ( N_{abf} + N_{aw} &lt; N_{af} + N_c )</td>
<td>PNA in the top flange</td>
</tr>
<tr>
<td>( N_e \geq N_{cu} + N_{cl} ) and ( N_e &lt; N_c )</td>
<td>PNA in the slab under lower reinforcements</td>
</tr>
<tr>
<td>( N_e + N_{cu} \geq N_{cl} ) and ( N_e + N_{sl} &lt; N_{cu} + N_{cl} )</td>
<td>PNA in the slab between reinforcements</td>
</tr>
<tr>
<td>( N_e + N_{sl} &lt; N_{cu} )</td>
<td>PNA in the slab above upper reinforcements</td>
</tr>
</tbody>
</table>

---

EN 1994-2, 6.2.2.5(2)

(2) For the calculation of \( M_{f,Rd} \) in EN 1993-1-5, 7.1(1) the design plastic resistance to bending of the effective composite section excluding the steel web should be used.
The design plastic bending resistance $M_{f,Rd}$ of the cross-section consisting of the flanges only (structural steel flange + concrete slab + eventually reinforcing steel bars) should be first calculated. $M_{f,Rd}$ is calculated as $M_{pl,Rd}$ neglecting the web contribution.

To calculate $M_{f,Rd}$, the position of the Plastic Neutral Axis (PNA) is determined (by using the same definition than in Paragraph 3.1.2.4) as:

\[ N_{abf} + N_{af} + N_{sl} = 28.875 \text{ MN} \geq N_{cur} = 7.14 \text{ MN} \]

and

\[ N_{ubf} + N_{uf} + N_{ul} = 28.875 \text{ MN} < N_{cur} + N_{clur} = 31.535 \text{ MN} \]

Thus the PNA is deduced to be located in the concrete slab between reinforcements at a distance $z_{pl}$ from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA:

\[ x_{pl} = h + e \frac{N_{abf} + N_{af} + N_{sl}}{0.85 b_{eff} f_{cd}} = 2.482 \text{ m} \]

The design plastic resistance moment of the flanges only is calculated from the position of the PNA: $M_{f,Rd} = 38.704 \text{ MNm}$.

\[ c = a \left( 0.25 + \frac{1.6 b_{f} t_{f}^{2} f_{sf}}{h_{w} f_{yw}} \right) = 2.25 \text{ m} \]

\[ V_{bf,Rd} = \frac{b_{f} t_{f}^{2} f_{sf}}{c y_{M1}^{2}} \left( 1 - \frac{M_{Ed}}{M_{f,Rd}} \right) = 0.121 \text{ MN} \]

The contribution of the flanges $V_{bf,Rd}$ is negligible compared to contribution from the web. Then the contribution of the flanges may be neglected.

\[ V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 4.625 + 0.121 = 4.746 \text{ MN} \leq \frac{h_{w} f_{yw}}{\sqrt{3 y_{M1}}} = 9.578 \text{ MN} \]

\[ V_{Rd} = \min (V_{b,Rd}; V_{pl,a,Rd}) = \min (4.746; 10.536) = 4.746 \text{ MN} \]

**Cross section verification**

The verification should be performed as follows:

\[ V_{Ed} = 3.977 \text{ MN} \leq V_{Rd} = 4.746 \text{ MN} \]

\[ \eta_{b} = \frac{V_{Ed}}{V_{Rd}} = 0.838 \leq 1 \]

⇒ **Shear resistance is verified!**

**3.1.2.5.3 M-V-interaction**

\[ V_{Ed} = 3.977 \text{ MN} \geq 0.5 V_{Rd} = 2.373 \text{ MN} \]

Therefore the M-V-interaction should be checked.

\[ \frac{M_{Ed}}{M_{f,Rd}} = 0.676 \leq 1; \quad \frac{V_{Ed}}{V_{bw,Rd}} = 0.86 \leq 1 \]

$M_{Ed} < M_{f,Rd}$ so that according to EN 1993-1-5, 7.1(1), there is no interaction. It means that the flanges are enough to resist alone the bending moment so that the entire web can be used for the resistance to the shear force.

Then flanges of the steel girder take the bending moment and web of the steel beam takes shear force.

⇒ **There is no interaction.**
Further explanations on the determination of the cross-section class
See Paragraph 3.1.2.4, page 85f.

Further explanations on the bending resistance check
See Paragraph 3.1.2.5.1, page 87f.

Further explanations on the shear resistance check
See Paragraph 3.1.2.5.2, page 89ff.
3.1.3 Check of cross-section at mid-span C0-P1

3.1.3.1 Geometry
The geometry is the same as for the check of cross-section at external support C0 (see Paragraph 3.1.2.1.)

At mid-span C0-P1 at ULS the concrete slab is in compression over its whole height. Its contribution is therefore taken into account in the cross-section resistance.

The effective width of the slab is the same as for the check of cross-section at external support C0 (see Paragraph 3.1.2.1).

\[ b_{\text{eff}} = 6 \text{ m} \]

3.1.3.2 Material properties
See Paragraph 3.1.2.2.

3.1.3.3 Internal forces and moments
The bending moment and shear force in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{\text{Ed}} = 39.314 \text{ MNm} \text{ (at a distance of 25 m of the external support C0: } x = 25 \text{ m)} \]
\[ V_{\text{Ed}} = 1.952 \text{ MN} \text{ (at a distance of 20 m of the external support C0: } x = 20 \text{ m)} \]

The check is realised on the third panel located at mid-span CO-P1 (see Figure 3-2). In a safe way, maximum values of internal forces acting on this panel are used to check it.

3.1.3.4 Determination of the cross-section class
The Class of section is the same as for the check of cross-section at external support C0 (see Paragraph 3.1.2.1.)

Conclusion: the cross-section at mid-span C0-P1 and P2-C3 is in class 1 and is checked by a plastic section analysis.

3.1.3.5 Plastic section analysis

3.1.3.5.1 Bending resistance check
As the geometry of the section at mid-span C0-P1 is the same as in C0, \( M_{\text{pl, Rd}} \) does not change:

\[ M_{\text{pl, Rd}} = 57.597 \text{ MNm} \]

Reinforcement in compression in the concrete slab is neglected according to EN 1994-2, 6.2.1.2(1).

\[ M_{\text{Ed}} = 39.314 \text{ MNm} \leq M_{\text{pl, Rd}} = 57.597 \text{ MNm} \]

\[ \Rightarrow \text{ Bending resistance is verified!} \]

3.1.3.5.2 Shear resistance check
As the geometry of the section at mid-span C0-P1 is the same as in C0, \( V_{bw, Rd} \) does not change:

\[ V_{bw, Rd} = 4.625 \text{ MN} \]

On the contrary, as \( V_{bf, Rd} \) is a function of \( M_{\text{Ed}} \) and because \( M_{\text{Ed}} \) changes:
Further explanations on the M-V-interaction
See Paragraph 3.1.2.5.3, page 93.
Cross-section verifications - Twin-girder bridge at mid-span P1-P2

\[ M_{f,Rd} = 38.704 \text{ MN} \quad \Rightarrow \quad \frac{M_{Ed}}{M_{f,Rd}} = 1.016 \geq 1 \Rightarrow V_{bf,Rd} = 0 \text{ MN} \leq \frac{\eta f_{yw} h w t w}{\sqrt{3} \gamma_{M1}} = 9.578 \text{ MN} \]

Then:

\[ V_{Rd} = V_{bw,Rd} = 4.625 \text{ MN} \]

Cross-section verification

The verification should be performed as follows:

\[ V_{Ed} = 1.952 \text{ MN} \leq V_{Rd} = \min(4.625; 10.536) = 4.625 \text{ MN} \text{ is verified.} \]

\[ \eta_h = \frac{V_{Ed}}{V_{Rd}} = 0.422 \leq 1 \]

⇒ Shear resistance is verified!

3.1.3.5.3 M-V-interaction

\[ V_{Ed} = 1.952 \text{ MN} \leq 0.5 \cdot V_{Rd} = 2.318 \text{ MN} \]

⇒ There is no need to check the M-V-interaction.

3.1.4 Check of cross-section at mid-span P1-P2

3.1.4.1 Geometry

At mid-span P1-P2 at ULS the concrete slab is almost in compression over its whole height. Its contribution is therefore taken into account in the cross-section resistance. The values in **bold** are the values which only change compared to the check of the cross-section at external support C0, see Section 3.1.2.

![Figure 3-6: Cross-section at mid-span P1-P2.](#)
### General properties of the twin-girder bridge in cross-section P1-P2

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_2 = 60$ m</td>
<td></td>
</tr>
<tr>
<td>$a = 7.5$ m</td>
<td></td>
</tr>
<tr>
<td>$h = 2400$ mm</td>
<td></td>
</tr>
<tr>
<td>$t_w = 19$ mm</td>
<td></td>
</tr>
<tr>
<td>$b_{tf} = 800$ mm</td>
<td></td>
</tr>
<tr>
<td>$b_{bf} = 1000$ mm</td>
<td></td>
</tr>
<tr>
<td>$t_f = 35$ mm</td>
<td></td>
</tr>
<tr>
<td>$h_u = h - 2t_f = 2.33$ m</td>
<td></td>
</tr>
<tr>
<td>$e = 32.5$ cm</td>
<td></td>
</tr>
<tr>
<td>$\phi_u = 16$ mm</td>
<td></td>
</tr>
<tr>
<td>$\phi_w = 16$ mm</td>
<td></td>
</tr>
<tr>
<td>$s_{ur} = 130$ mm</td>
<td></td>
</tr>
<tr>
<td>$s_{lr} = 130$ mm</td>
<td></td>
</tr>
<tr>
<td>$c_u = 60$ mm</td>
<td></td>
</tr>
<tr>
<td>$c_w = 60$ mm</td>
<td></td>
</tr>
<tr>
<td>$b_{eff} = 6$ m</td>
<td></td>
</tr>
<tr>
<td>$n_{ur} = \frac{b_{eff}}{s_{ur}} = 46.154$</td>
<td></td>
</tr>
<tr>
<td>$n_{lr} = \frac{b_{eff}}{s_{lr}} = 46.154$</td>
<td></td>
</tr>
</tbody>
</table>

### Principle areas of the different part of the composite section

<table>
<thead>
<tr>
<th>Area</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{rsf} = t_f b_{tf}$</td>
<td>$0.028$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_{sw} = t_w h_u$</td>
<td>$0.044$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_{idy} = t_{ij} b_{ij}$</td>
<td>$0.035$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_a = A_{sf} + A_{sw} + A_{idy}$</td>
<td>$0.107$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_{sur} = \pi d_i^2 / 4$</td>
<td>$2.011$ cm²</td>
<td></td>
</tr>
<tr>
<td>$A_{sur} = n_u A_{sur}$</td>
<td>$92.816$ cm²</td>
<td></td>
</tr>
<tr>
<td>$A_{slr} = \pi d_{er}^2 / 4$</td>
<td>$2.011$ cm²</td>
<td></td>
</tr>
<tr>
<td>$A_{slr} = n_l A_{slr}$</td>
<td>$92.816$ cm²</td>
<td></td>
</tr>
<tr>
<td>$A_u = c_u b_{eff}$</td>
<td>$0.36$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_{ur} = (e - c_u - c_w) b_{eff}$</td>
<td>$1.23$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_{slr} = c_w b_{eff}$</td>
<td>$0.36$ m²</td>
<td></td>
</tr>
<tr>
<td>$A_c = e b_{eff} = A_u + A_{ur} + A_{slr}$</td>
<td>$1.95$ cm²</td>
<td></td>
</tr>
</tbody>
</table>

(see notation and Figure 3-6)

### 3.1.4.2 Material properties

**Structural steel**

$f_{yw} = 345$ N/mm² because $16$ mm $< t_f = 19$ mm $\leq 40$ mm (see Table 2-4)

$$\varepsilon_u = \sqrt{\frac{235 N/mm^2}{f_{yw}}} = 0.825$$

$f_{sf} = 345$ N/mm² because $16$ mm $< t_f = 35$ mm $\leq 40$ mm (see Table 2-4)

$$\varepsilon_f = \sqrt{\frac{235 N/mm^2}{f_{sf}}} = 0.825$$

$f_{ydw} = 345$ N/mm²

$f_{ydf} = 345$ N/mm²

$E_a = 210000$ N/mm²
Further explanations on the determination of the cross-section class
See Paragraph 3.1.2.4, page 91f.
Concrete

See Paragraph 3.1.2.2.

Reinforcement

See Paragraph 3.1.2.2.

3.1.4.3 Internal forces and moments

The internal forces and moments in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{Ed} = 30.17 \text{ MNm} \text{ (at mid-span of the second span L2 panel: } x = 80 \text{ m)} \]

\[ V_{Ed} = 2.152 \text{ MN} \text{ (at a distance } a = 7.5 \text{ m of the mid-span of the second span L2: } x = 87.5 \text{ m)} \]

3.1.4.4 Determination of the cross-section class

- Bottom flange is in tension: Class 1
- Top flange is composite and connected to the slab following the recommendations of EN 1994-2, 6.6: Class 1
- To classify the steel web, the position of the Plastic Neutral Axis (PNA) is determined as:
  - Design plastic resistance of the concrete in compression located above upper reinforcements of the slab:
    \[ N_{cuw} = 0.85.A_{cuw} \frac{f_{ck}}{\gamma_c} = 7.14 \text{ MN} \]
  - Design plastic resistance of the concrete in compression located between upper and lower reinforcements of the slab:
    \[ N_{cluw} = 0.85.A_{cluw} \frac{f_{ck}}{\gamma_c} = 24.395 \text{ MN} \]
  - Design plastic resistance of the concrete in compression located under lower reinforcements of the slab:
    \[ N_{clw} = 0.85.A_{clw} \frac{f_{ck}}{\gamma_c} = 7.4 \text{ MN} \]
  - Design plastic resistance of the concrete in compression:
    \[ N_c = N_{cuw} + N_{cluw} + N_{clw} = 38.675 \text{ MN} \]
  - Design plastic resistance of the total upper reinforcements:
    \[ N_{sur} = A_{sur} f_{yd} = 4.035 \text{ MN} \]
  - Design plastic resistance of the total lower reinforcements:
    \[ N_{slr} = A_{slr} f_{yd} = 4.035 \text{ MN} \]
  - Design plastic resistance of the structural steel top flange:
    \[ N_{af} = A_{af} \frac{f_{yf}}{\gamma_{M0}} = 9.66 \text{ MN} \]
  - Design plastic resistance of the structural steel web:
    \[ N_{aw} = A_{aw} \frac{f_{yw}}{\gamma_{M0}} = 15.273 \text{ MN} \]
Further explanations on the bending resistance check
See Paragraph 3.1.2.5.1, page 87f.

Further explanations on the shear resistance check
See Paragraph 3.1.2.5.2, page 89ff.

Relations to find the location of the Plastic Neutral Axis (PNA) under positive moment $M_{Pl,Rd}$

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{abf} \geq N_{aw} + N_{af} + N_{c}$</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>$N_{abf} + N_{aw} \geq N_{af} + N_{c}$ and $N_{abf} &lt; N_{aw} + N_{af} + N_{c}$</td>
<td>PNA in the web</td>
</tr>
<tr>
<td>$N_a \geq N_{cur}$ and $N_{abf} + N_{aw} &lt; N_{af} + N_{c}$</td>
<td>PNA in the top flange</td>
</tr>
<tr>
<td>$N_a \geq N_{cur} + N_{clur}$ and $N_a &lt; N_{c}$</td>
<td>PNA in the slab under lower reinforcements</td>
</tr>
<tr>
<td>$N_a + N_{sl} \geq N_{cur}$ and $N_a + N_{sl} &lt; N_{cur} + N_{clur}$</td>
<td>PNA in the slab between reinforcements</td>
</tr>
<tr>
<td>$N_a + N_{sl} + N_{su} &lt; N_{cur}$</td>
<td>PNA in the slab above upper reinforcements</td>
</tr>
</tbody>
</table>
Cross-section verifications - Twin-girder bridge at mid-span P1-P2

- Design plastic resistance of the structural steel bottom flange:
  \[ N_{abf} = A_{abf} \frac{f_{sf}}{\gamma_{M0}} = 12.075 \text{ MN} \]

- Design plastic resistance of the structural steel:
  \[ N_a = N_{a\text{sf}} + N_{aw} + N_{abf} = 37.008 \text{ MN} \]

- Location of the Plastic Neutral Axis (PNA):
  \[ N_a = 37.008 \text{ MN} \geq N_{\text{cur}} + N_{\text{clur}} = 31.535 \text{ MN} \]
  and \[ N_a = 37.008 \text{ MN} < N_c = 38.675 \text{ MN} \]
  Thus, the PNA is deduced to be located in the slab in the lower reinforcements at a distance \( z_{pl} \) from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA:
  \[ z_{pl} = h + e - \frac{N_a}{0.85b_{eff}f_{ed}} = 2.414 \text{ m} \]

As the PNA is located in the slab below lower reinforcement, the whole web is in tension and therefore in Class 1.

**Conclusion:** the cross-section at the mid-span P1-P2 is in class 1 and is checked by a plastic section analysis.

### 3.1.4.5 Plastic section analysis

#### 3.1.4.5.1 Bending resistance check

The design plastic resistance moment is calculated from the position of the PNA:

\[ M_{pl,Rd} = 53.532 \text{ MNm} \]

Reinforcement in compression in the concrete slab is neglected according EN 1994-2, 6.2.1.2(1).

\[ M_{Ed} = 30.17 \text{ MNm} \leq M_{pl,Rd} = 53.532 \text{ MNm} \]

\[ \Rightarrow \quad \text{Bending resistance is verified!} \]

#### 3.1.4.5.2 Shear resistance check

The web should be checked in terms of shear buckling if:

- \[ \frac{h_w}{t_w} > \frac{72}{\eta} \varepsilon_u \] for unstiffened web
- \[ \frac{h_w}{t_w} > \frac{31}{\eta} \varepsilon_u \sqrt{k_i} \] for stiffened web

In this case, the web is stiffened by vertical stiffeners.

\[ \frac{h_w}{t_w} = 122.632 > \frac{31}{\eta} \varepsilon_u \sqrt{k_i} = 51.019 \] then the web should be checked in terms of shear buckling.

The maximum design shear resistance is given by:

\[ V_{Rd} = \min \left( V_{b,Rd} ; V_{pl,a,Rd} \right) \]
where \( V_{h,Rd} = V_{bw,Rd} + V_{bf,Rd} = \frac{x_n f_{yw} h_w t_w}{\sqrt{3 \gamma_{M1}}} + b_f t_f f_{sf} \left( 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) \leq \eta f_{yw} h_w t_w = 9.62 \text{ MN} \)

\( V_{pl,a,Rd} = \eta f_{yw} h_w t_w = 10.582 \text{ MN} \)

where \( \eta = 1.2 \) for steel grades up to and including S460

**Contribution of the web** \( V_{bw,Rd} \)

\( V_{bw,Rd} = \frac{x_n f_{yw} h_w t_w}{\sqrt{3 \gamma_{M1}}} \)

The vertical stiffeners at the bracing transverse frames which border the webpanel located in span P1–P2, are assumed to be rigid (to be checked by using Section 9 of EN 1993-1-5). They are equally spaced by \( a = 8.333 \text{ m} \).

\( k_{vw} = 0 \) because there is no longitudinal stiffeners

\( a/h_w = 3.219 \geq 1 \)

\( k_i = 5.34 + 4 \left( \frac{h_u}{a} \right)^2 + k_{vw} = 5.726 \)

\( \bar{x}_w = \frac{h_w}{37.4 t_c e_u \sqrt{k_i}} = 1.66 \geq 1.08 \)

\[ \Rightarrow \bar{x}_w = \frac{1.37}{0.7 + \bar{x}_w} = 0.58 \]

\( V_{bw,Rd} = \frac{x_n f_{yw} h_w t_w}{\sqrt{3 \gamma_{M1}}} = 4.653 \text{ MN} \)

**Contribution from the flanges** \( V_{bf,Rd} \)

\( b_f t_f \) are taken for the flange which provides the least axial resistance,

\( b_f \) being taken as not larger than \( 15 \alpha t_f \) on each side of the web,

The lower flange of the cross-section is a structural steel section whereas its upper flange is a composite section (structural steel + concrete). The formulae for calculating \( V_{bf,Rd} \) should be used with the lower steel flange properties.

The design plastic bending resistance \( M_{f,Rd} \) of the cross-section consisting of the flanges only should be first calculated. \( M_{f,Rd} \) is calculated as \( M_{p,Rd} \) but neglecting the web contribution.

To calculate \( M_{f,Rd} \), the position of the Plastic Neutral Axis (PNA) is determined (by using the same definition than in Paragraph 3.1.2.4) as:

\( N_{adv} + N_{aft} + N_{sl} = 25.77 \text{ MN} \geq N_{cur} = 7.14 \text{ MN} \)

and \( N_{adv} + N_{aft} + N_{sl} = 25.77 \text{ MN} < N_{cur} + N_{clus} = 31.535 \text{ MN} \)

Thus, the PNA is deduced to be located in the concrete slab between reinforcements at a distance \( z_{pl} \) from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA:
Further explanations on the M-V-interaction

See Paragraph 3.1.2.5.3, page 93.
Cross-section verifications - Twin-girder bridge at the internal support P2

\[ \bar{z}_{pl} = h + e - \frac{N_{ref} + N_{wef} + N_d}{0.85 b_{eff} f_{cd}} = 2.508 \text{ m} \]

The design plastic resistance moment of the flanges only is calculated from the position of the PNA: \( M_{f,Rd} = 34.281 \text{ MNm} \)

\[ c = 0.25 + \frac{1.6 b_j t_j^2 f_{sf}}{t_h^2 f_{yw}} = 1.989 \text{ m} \]

\[ V_{bf,Rd} = \frac{b_j t_j^2 f_{sf}}{c' \gamma_M 1} \left( 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) = 0.035 \text{ MN} \]

The contribution \( V_{bf,Rd} \) of the flanges is negligible.

\[ V_{b,Rd} = V_{bw,Rd} + V_{f,Rd} = 4.667 + 0.035 = 4.688 \text{ MN} \]

\[ V_{Rd} = \min (V_{b,Rd}; V_{pl,a,Rd}) = \min (4.688; 10.582) = 4.688 \text{ MN} \]

Cross-section verification

The verification should be performed as follows:

\[ V_{Ed} = 2.152 \text{ MN} \leq V_{Rd} = 4.688 \text{ MN} \]

\[ \eta_3 = \frac{V_{Ed}}{V_{Rd}} = 0.459 \leq 1 \]

\[ \Rightarrow \quad \text{Shear resistance is verified!} \]

3.1.4.5.3 M-V-interaction

\[ V_{Ed} = 2.152 \text{ MN} \leq 0.5 V_{Rd} = 2.344 \text{ MN} \]

\[ \Rightarrow \quad \text{There is no need to check the M-V-interaction.} \]

3.1.5 Check of cross-section at the internal support P2

Two vertical stiffeners are added to reduce the length \( a \) of each webpanel located at each side of the internal support P2. According to Figure 3-3, at each side of the internal support P2, these two vertical stiffeners divide the panel in three parts: denoted subpanel 1, 2 and 3.

**NOTE:** Here a subpanel denotes a longitudinally unstiffened webpanel which is only bordered by flanges and transverse stiffeners on each side.

3.1.5.1 Subpanel 1 - Geometry

At internal support P2 at ULS the concrete slab is in tension over its whole height. Its contribution is therefore neglected in the cross-section resistance.
General properties of the twin-girder bridge in cross-section P2

- $L_1 = L_3 = 50$ m, $L_2 = 60$ m
- $a = 1.5$ m
- $h = 2400$ mm
- $t_w = 19$ mm
- $b_{gf} = 800$ mm
- $b_{bf} = 1000$ mm
- $t_f = 95$ mm
- $h_w = h - 2t_f = 2.21$ m
- $e = 32.5$ cm
- $\phi_w = 20$ mm
- $\phi_f = 16$ mm
- $s_{aw} = 130$ mm
- $s_{slr} = 130$ mm
- $c_{aw} = 60$ mm
- $c_{slr} = 60$ mm
- $b_{eff} = 6$ m
- $\frac{b_{eff}}{s_{aw}} = n_{aw} = 46.154$
- $\frac{b_{eff}}{s_{slr}} = n_{slr} = 46.154$

Main areas of the different parts of the composite section

- $A_{ugf} = t_g b_{gf} = 0.076$ m$^2$
- $A_{uw} = t_w h_w = 0.042$ m$^2$
- $A_{ubf} = t_f b_{bf} = 0.095$ m$^2$
- $A_u = A_{ugf} + A_{uw} + A_{ubf} = 0.213$ m$^2$
- $A_{aw} = \frac{\pi d_w^2}{4} = 3.142$ cm$^2$
- $A_{aw} = n_{aw} A_{aw} = 144.997$ cm$^2$
- $A_{bf} = \frac{\pi d_{bf}^2}{4} = 2.011$ cm$^2$
- $A_{bf} = n_{bf} A_{bf} = 92.816$ cm$^2$
- $A_{aw} = c_{aw} b_{eff} = 0.36$ m$^2$
- $A_{bf} = c_{bf} b_{eff} = 0.36$ m$^2$
- $A_v = e b_{eff} = A_{aw} + A_{bf} + A_{slr} = 1.95$ cm$^2$

(see notation and Figure 3-7)
EN 1993-1-1, Table 5.2 (sheet 2 of 3), Maximum width-to-thickness ratios for compression parts

**Outstand flanges**

<table>
<thead>
<tr>
<th>Class</th>
<th>Part subject to compression</th>
<th>Part subject to bending and compression</th>
<th>Tip in compression</th>
<th>Tip in tension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress distribution in parts (compression positive)</td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>1</td>
<td>c/t ≤ 9 ε</td>
<td>c/t ≤ ( \frac{9 \varepsilon}{\alpha} )</td>
<td>c/t ≤ ( \frac{9 \varepsilon}{\alpha \sqrt{\alpha}} )</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>c/t ≤ 10 ε</td>
<td>c/t ≤ ( \frac{10 \varepsilon}{\alpha} )</td>
<td>c/t ≤ ( \frac{10 \varepsilon}{\alpha \sqrt{\alpha}} )</td>
<td></td>
</tr>
<tr>
<td>Stress distribution in parts (compression positive)</td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>3</td>
<td>c/t ≤ 14 ε</td>
<td>c/t ≤ ( 21 \varepsilon \sqrt{k_e} ) for ( k_e ) see EN 1993-1-5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ \varepsilon = \sqrt{\frac{235}{f_y}} \]

| \( f_y \) | 235 | 275 | 355 | 420 | 460 |
| \( \varepsilon \) | 1.00 | 0.92 | 0.81 | 0.75 | 0.71 |
3.1.5.2 Subpanel 1 - Material properties

Structural steel

\[ f_{yw} = 345 \text{ N/mm}^2 \text{ because } 16 \text{ mm} < t_f = 19 \text{ mm} \leq 40 \text{ mm} \text{ (see Table 2-4)} \]

\[ \varepsilon_{yw} = \sqrt{\frac{235 \text{ N/mm}^2}{f_{yw}}} = 0.825 \]

\[ f_{sf} = 315 \text{ N/mm}^2 \text{ because } 80 \text{ mm} < t_f = 95 \text{ mm} \leq 100 \text{ mm} \text{ (see Table 2-4)} \]

\[ \varepsilon_{sf} = \sqrt{\frac{235 \text{ N/mm}^2}{f_{sf}}} = 0.825 \]

\[ f_{ydw} = \frac{f_{yw}}{\gamma_{M0}} = 345 \text{ N/mm}^2 \]

\[ f_{ydf} = \frac{f_{sf}}{\gamma_{M0}} = 315 \text{ N/mm}^2 \]

\[ E_a = 210000 \text{ N/mm}^2 \]

Concrete

See Paragraph 3.1.2.2.

Reinforcement

See Paragraph 3.1.2.2.

3.1.5.3 Subpanel 1 - Internal forces and moments

The internal forces and moments in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{Ed} = 65.44 \text{ MNm} \text{ (at the internal support P2: } x = 110 \text{ m)} \]

\[ V_{Ed} = 6.087 \text{ MN} \text{ (at the internal support P2: } x = 110 \text{ m)} \]

The maximum value of bending moment and shear force is in the internal support P2 (see Figure 2-31 and Figure 2-32). Figure 2-31 and Figure 2-32 are not perfectly symmetric.

3.1.5.4 Subpanel 1 - Determination of the cross-section class

- The top flange is in tension: Class 1
- The bottom flange is in compression:
  \[ \frac{c_{sf}}{t_f} = \frac{b_{sf} - t_w}{2t_f} = 5.163 \leq 9\varepsilon = 7.774 \text{ therefore in Class 1} \]
- The web is in tension in its upper part and in compression in its lower part. To classify the steel web, the position of the Plastic Neutral Axis (PNA) is determined as follows:
  - Design plastic resistance of total reinforcements of the slab:
    \[ N_{su} + N_{sl} = (A_{sur} + A_{sub}) \frac{f_{sk}}{\gamma_s} = 10.339 \text{ MN} \]
  - Design plastic resistance of the structural steel top flange:
Relations to find the location of the Plastic Neutral Axis (PNA) under negative moment $M_{PL,Rd}$

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{abf} \geq N_{aw} + N_{af} + N_{sl} + N_{su}$</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>$N_{abf} + N_{aw} \geq N_{af} + N_{sl} + N_{su}$ and $N_{abf} &lt; N_{aw} + N_{af} + N_{sl} + N_{su}$</td>
<td>PNA in the web</td>
</tr>
<tr>
<td>$N_s \geq N_a + N_{su}$ and $N_{abf} + N_{aw} &lt; N_{af} + N_{sl} + N_{su}$</td>
<td>PNA in the top flange</td>
</tr>
<tr>
<td>$N_{sl} + N_{su} &gt; N_a$</td>
<td>PNA in the slab</td>
</tr>
</tbody>
</table>

EN 1993-1-1, Table 5.2 (sheet 1 of 3), Maximum width-to-thickness ratios for compression parts

<table>
<thead>
<tr>
<th>Class</th>
<th>Part subject to bending</th>
<th>Part subject to compression</th>
<th>Part subject to bending and compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress distribution in parts (compression positive)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>$c/t \leq 72\varepsilon$</td>
<td>$c/t \leq 33\varepsilon$</td>
<td>when $\alpha &gt; 0.5$: $c/t \leq \frac{396\varepsilon}{13\alpha - 1}$, when $\alpha \leq 0.5$: $c/t \leq \frac{36\varepsilon}{\alpha}$</td>
</tr>
<tr>
<td>2</td>
<td>$c/t \leq 83\varepsilon$</td>
<td>$c/t \leq 38\varepsilon$</td>
<td>when $\alpha &gt; 0.5$: $c/t \leq \frac{456\varepsilon}{13\alpha - 1}$, when $\alpha \leq 0.5$: $c/t \leq \frac{41.5\varepsilon}{\alpha}$</td>
</tr>
<tr>
<td>Stress distribution in parts (compression positive)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$c/t \leq 124\varepsilon$</td>
<td>$c/t \leq 42\varepsilon$</td>
<td>when $\psi &gt; -1$: $c/t \leq \frac{42\varepsilon}{0.67 + 0.33\psi}$, when $\psi \leq -1$: $c/t \leq 62(1 - \psi)\sqrt{(-\psi)}$</td>
</tr>
</tbody>
</table>

$\varepsilon = \sqrt{235/f_y}$

<table>
<thead>
<tr>
<th>$f_y$</th>
<th>235</th>
<th>275</th>
<th>355</th>
<th>420</th>
<th>460</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon$</td>
<td>1.00</td>
<td>0.92</td>
<td>0.81</td>
<td>0.75</td>
<td>0.71</td>
</tr>
</tbody>
</table>

*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$
Cross-section verifications - Twin-girder bridge at the internal support P2

\[ N_{\text{af}} = A_{\text{af}} \frac{f_{sf}}{\gamma_{M0}} = 23.94 \text{ MN} \]

- Design plastic resistance of the structural steel web assumed to be entirely in compression:
  \[ N_{\text{aw}} = A_{\text{aw}} \frac{f_{sw}}{\gamma_{M0}} = 14.487 \text{ MN} \]

- Design plastic resistance of the structural steel bottom flange:
  \[ N_{\text{abf}} = A_{\text{abf}} \frac{f_{sf}}{\gamma_{M0}} = 29.925 \text{ MN} \]

- Design plastic resistance of the structural steel:
  \[ N_a = N_{\text{af}} + N_{\text{aw}} + N_{\text{abf}} = 68.352 \text{ MN} \]

- Location of the Plastic Neutral Axis (PNA):
  \[ N_{\text{abf}} + N_{\text{aw}} = 44.412 \text{ MN} \geq N_{\text{af}} + N_{\text{sl}} + N_{\text{su}} = 34.279 \text{ MN} \]
  and \[ N_{\text{abf}} = 29.925 \text{ MN} < N_{\text{aw}} + N_{\text{af}} + N_{\text{sl}} + N_{\text{su}} = 48.766 \text{ MN} \]
  Thus, the PNA is deduced to be located in the steel web at a distance \( z_p \) from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA:
  \[ z_p = \frac{2hb_{\text{f}} f_{sf} + N_{\text{su}} + N_{\text{sl}} - N_a}{2b_{\text{f}} f_{sf}} = 1.532 \text{ m} \]

More than half the web height is in compression:
\[ \alpha = \frac{(z_p - t_f)}{h_w} = 0.65 > 0.5 \]

Therefore the limiting slenderness between Class 2 and Class 3 is given by:
\[ c_w = \frac{h_w}{t_w} = 116.316 \gg \frac{456\varepsilon_w}{13\alpha - 1} = 50.492 \]

The steel web is at least in Class 3 and reasoning is now based on the elastic stress distribution at ULS given by the global analysis which taken into account the history of the construction (erection phasing: see Paragraph 2.1.4):
\[ \sigma_{\text{abfu}} = -276.93 \text{ N/mm}^2 \]
\[ \sigma_{\text{af}} = 265.58 \text{ N/mm}^2 \]

And the elastic stress distribution at ULS:
\[ \psi_w = \frac{\sigma_{\text{af}}}{\sigma_{\text{abfu}}} = \frac{-266.71}{265.58} = -1.043 \leq 1 \]

Therefore the limiting slenderness between Class 3 and Class 4 is given by:
\[ c_w = \frac{h_w}{t_w} = 116.316 > 62\varepsilon(1 - \psi_w)\sqrt{-\psi_w} = 106.737 \]

It is deduced that the steel web is in Class 4.

**Conclusion:** the cross-section at the internal support P1 and P2 is in Class 4 and is checked by a elastic section analysis.
EN 1993-1-5, 4.6(3)

(3) The plate buckling verification of the panel should be carried out for the stress resultants at a distance 0.4a or 0.5b, whichever is the smallest, from the panel end where the stresses are the greater. In this case the gross sectional resistance needs to be checked at the end of the panel.

EN 1993-1-5, 4.4, Plate elements without longitudinal stiffeners

(1) The effective areas of flat compression elements should be obtained using Table 4.1 for internal elements and Table 4.2 for outstand elements. The effective area of the compression zone of a plate with the gross cross-sectional area Ac should be obtained from:

\[
A_{c,\text{eff}} = \rho A_c
\]  
(4.1)

where \( \rho \) is the reduction factor for plate buckling.

(2) The reduction factor \( \rho \) may be taken as follows:

- **internal compression elements:**
  \[
  \rho = 1,0 \quad \text{for } \lambda_p \leq 0,673
  \]
  \[
  \rho = \frac{\lambda_p - 0,055(3 + \psi)}{\lambda_p^2} \leq 1 \quad \text{for } \lambda_p > 0,673, \text{ where } (3 + \psi) \geq 0
  \]

- **outstand compression elements:**
  \[
  \rho = 1,0 \quad \text{for } \lambda_p \leq 0,748
  \]
  \[
  \rho = \frac{\lambda_p - 0,188}{\lambda_p^2} \leq 1 \quad \text{for } \lambda_p > 0,748
  \]

where \( \lambda_p = \sqrt{\frac{f_s}{\sigma_{e}} = \frac{\bar{b} / t}{28.4 \varepsilon \sqrt{k_e}}} \)

\( \psi \) is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

\( \bar{b} \) is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

- \( b_w \) for webs;
- \( b \) for internal flange elements (except RHS);
- \( b - 3 \, t \) for flanges of RHS;
- \( c \) for outstand flanges;
- \( h \) for equal-leg angles;
- \( h \) for unequal-leg angles;

\( k_e \) is the buckling factor corresponding to the stress ratio \( \psi \) and boundary conditions. For long plates \( k_e \) is given in Table 4.1 or Table 4.2 as appropriate;
3.1.5.5 Subpanel 1 - Elastic section analysis

3.1.5.5.1 Bending resistance check

The section is in Class 4 so that its effective cross-section under bending moment has to be calculated according to section EN 1993-1-5, 4.4.

**Bottom flange in compression**

\[ k_{bpf} = 0.43 \] (see Table 4.2 of EN 1993-1-5, 4.4: outstand elements)

\[ \bar{\chi}_{bpf} = \frac{b}{t} = \frac{b_{bpf} - t_{w}}{28,4\sqrt{k_{\sigma}}} \frac{2t_{w}}{28,4\sqrt{k_{\sigma}}} = 0.321 \leq 0.748 \]

\[ \Rightarrow \quad \rho_{bpf} = 1 \text{ there is no reduction of the width of the bottom flange. The full bottom flange is effective.} \]

**Web in bending**

Stresses at the end of the web are given by global analysis:

\[ \sigma_{abf} = -276.93 \text{ MPa} \]
\[ \sigma_{abfu} = 265.58 \text{ MPa} \]
\[ \psi_{w} = \frac{\sigma_{abf}}{\sigma_{abfu}} = -1.043 \leq 1 \]

\[ k_{cw} = 5.98(1 - \psi_{w})^{2} = 24.953 \] (see Table 4.1 of EN 1993-1-5, 4.4: internal compression elements)

\[ \bar{\lambda}_{pw} = \frac{b}{t} = \frac{h_{w}}{28,4\sqrt{k_{\sigma}}} = \frac{t_{w}}{28,4\sqrt{k_{\sigma}}} = 0.993 > 0.673 \]

\[ \Rightarrow \quad \rho_{w} = \frac{\bar{\lambda}_{pw} - 0.055(3 + \psi_{w})}{\bar{\lambda}_{pw}^{2}} = 0.898; \text{ there is a reduction of the height of the steel web} \]

Then the effective height of the web in compression can be calculated:

\[ h_{wef} = \frac{\rho_{w}h_{w}}{(1 - \psi_{w})} = 0.971 \text{ m} \]

And this effective height of the web can be distributed as show in Table 4.1 of EN 1993-1-5:

\[ h_{wef1} = 0.4h_{wef} = 0.388 \text{ m} \]
\[ h_{wef2} = 0.6h_{wef} = 0.583 \text{ m} \]

**Final mechanical properties of the effective structural steel twin-girder section**

(only flanges and web)

The effective cross-sectional properties can be calculated:

\[ A_{wef} = A_{abf} + A_{abf} + (h_{wef} + h_{w} - h_{w} + t_{f})t_{w} = 0.21 \text{ m}^{2} \]
### EN 1993-1-5, 4.4, Table 4.2, Outstand compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective width $b_{\text{eff}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram" /></td>
<td>$1 &gt; \psi &gt; 0$: $b_{\text{eff}} = b_c$</td>
</tr>
<tr>
<td><img src="image2" alt="Diagram" /></td>
<td>$\psi &lt; 0$: $b_{\text{eff}} = b_c (1 - \psi)$</td>
</tr>
<tr>
<td><img src="image3" alt="Diagram" /></td>
<td>$\psi = \alpha_2/\alpha_1$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi$</th>
<th>$1$</th>
<th>$0$</th>
<th>$-1$</th>
<th>$1 &gt; \psi &gt; -1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_\psi$</td>
<td>0.43</td>
<td>0.57</td>
<td>0.85</td>
<td>$0.57 - 0.21\psi + 0.07\psi^2$</td>
</tr>
</tbody>
</table>

### EN 1993-1-5, 4.4, Table 4.1, Internal compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective width $b_{\text{eff}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image4" alt="Diagram" /></td>
<td>$1 &gt; \psi &gt; 0$: $b_{\text{eff}} = b_c$</td>
</tr>
<tr>
<td><img src="image5" alt="Diagram" /></td>
<td>$\psi &lt; 0$: $b_{\text{eff}} = b_c (1 - \psi)$</td>
</tr>
<tr>
<td><img src="image6" alt="Diagram" /></td>
<td>$\psi = \alpha_2/\alpha_1$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi$</th>
<th>$1$</th>
<th>$1 &gt; \psi &gt; 0$</th>
<th>$0$</th>
<th>$0 &gt; \psi &gt; -1$</th>
<th>$-1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_\psi$</td>
<td>0.43</td>
<td>0.578 / ($\psi + 0.34$)</td>
<td>1.79</td>
<td>$1.7 - 5\psi - 17.1\psi^2$</td>
<td>23.8</td>
</tr>
</tbody>
</table>

### EN 1993-1-5, 4.4, Table 4.1, Internal compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective width $b_{\text{eff}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image7" alt="Diagram" /></td>
<td>$1 &gt; \psi &gt; 0$: $b_{\text{eff}} = b_c$</td>
</tr>
<tr>
<td><img src="image8" alt="Diagram" /></td>
<td>$\psi &lt; 0$: $b_{\text{eff}} = b_c (1 - \psi)$</td>
</tr>
<tr>
<td><img src="image9" alt="Diagram" /></td>
<td>$\psi = \alpha_2/\alpha_1$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi$</th>
<th>$1$</th>
<th>$1 &gt; \psi &gt; 0$</th>
<th>$0$</th>
<th>$0 &gt; \psi &gt; -1$</th>
<th>$-1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_\psi$</td>
<td>4.0</td>
<td>$8.2 / (1.05 + \psi$)</td>
<td>7.81</td>
<td>$7.81 - 6.29\psi + 9.78\psi^2$</td>
<td>23.9</td>
</tr>
</tbody>
</table>
The effective elastic neutral axis (ENA) must be determined from the extreme lower fiber of the bottom flange:

\[ h_{u,\text{eff}} = \frac{A_{u,\text{eff}}}{A_{u,\text{eff}}} = 1.106 \text{ m} \]

And it can be deduced, the effective second moment of area of the cross-section:

\[ I_{u,\text{eff}} = \frac{b_{g}f_{f}^{3}}{12} + A_{u,\text{eff}}\left( h_{u,\text{eff}} - \frac{f_{f}}{2} \right)^{2} + \frac{b_{g}f_{f}^{3}}{12} + A_{u,\text{eff}}\left( h_{u,\text{eff}} - \frac{f_{f}}{2} - h_{u,\text{eff}} \right)^{2} \]

\[ + \frac{f_{f}^{3}h_{w,\text{eff}}}{12} + h_{w,\text{eff}}t_{w}\left( h_{u,\text{eff}} - \frac{h_{w,\text{eff}}}{2} - f_{f} \right)^{2} \]

\[ + \left( h_{w,\text{eff}} + h_{u} - f_{f} \right)t_{w}\left( h - h_{w,\text{eff}} + h_{u} - f_{f} + 3f_{f} - h_{u,\text{eff}} \right)^{2} = 0.241 \text{ m}^{4} \]

**Final mechanical properties of the effective composite twin-girder section (structural steel and reinforcements)**

The effective cross-sectional properties can be calculated:

\[ A_{\text{eff}} = A_{\text{sur}} + A_{\text{hub}} + A_{\text{u,eff}} + A_{\text{alb}} + \left( h_{\text{u,eff}} + h_{y} + f_{f} \right)t_{w} = 0.233 \text{ m}^{2} \]

The effective elastic neutral axis (ENA) must be determined from the extreme lower fiber of the bottom flange:

\[ h_{\text{eff}} = \frac{A_{\text{alb}}f_{f}}{2} + A_{\text{u,eff}}\left( h - \frac{f_{f}}{2} \right) + A_{\text{hub}}\left( h + c_{u} \right) + A_{\text{sur}}\left( h + e - c_{u} \right) \]

\[ + h_{w,\text{eff}}t_{w}\left( f_{f} + \frac{h_{w,\text{eff}}}{2} \right) + h_{w,\text{eff}}t_{w}\left( h_{w,\text{eff}} + h_{u} - f_{f} \right)t_{w}\left( h - f_{f} - \frac{h_{w,\text{eff}} + h_{u} - f_{f}}{2} \right) = 1.257 \text{ m} \]

And it can be deduced, the effective second moment of area of the cross-section:

\[ I_{\text{eff}} = \frac{b_{g}f_{f}^{3}}{12} + A_{\text{alb}}\left( h_{\text{eff}} - \frac{f_{f}}{2} \right)^{2} + A_{\text{u,eff}}\left( h - \frac{f_{f}}{2} - h_{\text{eff}} \right)^{2} \]

\[ + \frac{f_{f}^{3}h_{w,\text{eff}}}{12} + h_{w,\text{eff}}t_{w}\left( h_{\text{eff}} - \frac{h_{w,\text{eff}}}{2} - f_{f} \right)^{2} \]

\[ + \left( h_{w,\text{eff}} + h_{u} - f_{f} \right)t_{w}\left( h - h_{w,\text{eff}} + h_{u} - f_{f} + 3f_{f} - h_{\text{eff}} \right)^{2} \]

\[ + A_{\text{hub}}\left( h + c_{u} - h_{\text{eff}} \right)^{2} + A_{\text{sur}}\left( h + e - c_{u} - h_{\text{eff}} \right)^{2} = 0.288 \text{ m}^{4} \]

The stress in the upper reinforcement of the concrete slab is given by the global analysis:

\[ \sigma_{\text{sur}} = -185.85 \text{ MPa} \]
EN1993-1-1, 6.2.1(9)

(9) Where all the compression parts of a cross-section are Class 3, its resistance should be based on an elastic distribution of strains across the cross-section. Compressive stresses should be limited to the yield strength at the extreme fibres.

NOTE: The extreme fibres may be assumed at the midplane of the flanges for ULS checks. For fatigue see EN 1993-1-9.
Final mechanical properties of the gross composite twin-girder cross section (structural steel and reinforcement)

The cross-sectional properties can be calculated:

\[ A = A_{\text{tsur}} + A_{\text{tslr}} + A_{\text{atf}} + A_{\text{abf}} + A_{\text{aw}} = 0.237 \text{ m}^2 \]

The elastic neutral axis (ENA) must be determined from the extreme lower fiber of the bottom flange:

\[
h_A = \frac{A_{\text{atf}} t_f}{2} + A_{\text{abf}} \left( h - \frac{t_f}{2} \right) + A_{\text{aw}} \left( h + c_e \right) + A_{\text{tsur}} \left( h + e - c_w \right)
\]

\[ h_A = 1.247 \text{ m} \]

And it can be deduced, the effective second moment of area of the cross-section:

\[
I = \frac{h_A^3}{12} + A_{\text{abf}} \left( h_A - \frac{t_f}{2} \right)^2 + A_{\text{atf}} \left( h - \frac{t_f}{2} - h_A \right)^2 + \frac{t_f h_A^3}{12} + A_{\text{aw}} \left( h_A - h \right)^2
\]

\[ + A_{\text{tslr}} \left( h + c_e - h_A \right)^2 + A_{\text{tsur}} \left( h + e - c_w - h_A \right)^2 = 0.29 \text{ m}^4 \]

The bending moment taken by the composite twin-girder section (structural steel and reinforcements):

\[ M_{c,\text{Ed}} = \frac{\sigma_{\text{tsur}} I}{h + e - c_w - h_A} = -38.224 \text{ MNm} \]

The bending moment taken by the structural steel twin-girder section (only flanges and web):

\[ M_{a,\text{Ed}} = M_{\text{Ed}} - M_{c,\text{Ed}} = -65.44 + 38.224 = -27.216 \text{ MNm} \]

Then, the stress on each level of the cross-section can be easily determined:

\[
\sigma_{\text{atf}} = \frac{-M_{a,\text{Ed}} h_{\text{atf}}}{I_{\text{atf}}} + \frac{-M_{c,\text{Ed}} h_{\text{atf}}}{I_{\text{eff}}} = 291.511 \text{ N/mm}^2 \leq f_{\text{ydff}} = 315 \text{ N/mm}^2
\]

\[
\sigma_{\text{abf}} = \frac{-M_{a,\text{Ed}} (h_{\text{abf}} - t_f)}{I_{\text{abf}}} + \frac{-M_{c,\text{Ed}} (h_{\text{abf}} - t_f)}{I_{\text{eff}}} \leq \min \left( f_{\text{ydff}}; f_{\text{ysub}} \right) = 315 \text{ N/mm}^2
\]

\[
\sigma_{\text{aw}} = \frac{M_{a,\text{Ed}} (h - t_f - h_{\text{aw}})}{I_{\text{aw}}} + \frac{M_{c,\text{Ed}} (h - t_f - h_{\text{aw}})}{I_{\text{eff}}} = -274.462 \text{ N/mm}^2
\]

\[
\sigma_{\text{tslr}} = \frac{M_{a,\text{Ed}} (h - h_{\text{tslr}})}{I_{\text{tslr}}} + \frac{M_{c,\text{Ed}} (h - h_{\text{tslr}})}{I_{\text{eff}}} = -297.788 \text{ N/mm}^2 \leq f_{\text{ydff}} = 315 \text{ N/mm}^2
\]

\[
\sigma_{\text{tsur}} = \frac{M_{a,\text{Ed}} (h + c_e - h_{\text{tsur}})}{I_{\text{tsur}}} = -159.674 \text{ N/mm}^2 \leq f_{\text{ydff}} = 434.783 \text{ N/mm}^2
\]

\[
\sigma_{\text{atf}} = \frac{M_{c,\text{Ed}} (h + e - c_w - h_{\text{atf}})}{I_{\text{atf}}} = -186.873 \text{ N/mm}^2 \leq f_{\text{ydff}} = 434.783 \text{ N/mm}^2
\]

The bending moment resistance is governed by the resistance of the top flange:

\[
\eta_1 = \frac{\sigma_{\text{atf}}}{f_{\text{ydff}}} = 0.945 \leq 1
\]

The cross-section at subpanel 1 on the internal support P2 is therefore checked for bending at ULS.
Further explanations on the shear resistance check
See Paragraph 3.1.2.5.2, page 89ff.
The verifications are here performed with the stresses in the extreme fibres of the structural steel flanges. Remember that the use of the stresses in the mid-plan of the flanges is also allowable.

⇒ Bending resistance is verified!

3.1.5.5.2 Shear resistance check

The web should be checked in terms of shear buckling if:

- \( \frac{h_w}{t_w} > \frac{72}{\eta} \varepsilon_u \) for unstiffened web
- \( \frac{h_w}{t_w} > \frac{31}{\eta} \varepsilon_u \sqrt{k_i} \) for stiffened web

In this case, the web is stiffened by vertical stiffeners.

\[
\frac{h_w}{t_w} = 116.316 > \frac{31}{\eta} \varepsilon_u \sqrt{k_i} = 84.188 \text{ then the web should be checked in terms of shear buckling.}
\]

The maximum design shear resistance is given by

\[
V_{ld} = \min \left( V_{b,Rd}; V_{pl,a,Rd} \right)
\]

where

\[
V_{b,Rd} = V_{bw,Rd} + V_{sf,Rd} = \frac{X_u f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} + \frac{b_1 t_w^2 f_{yd}}{c M_{f,Rd}} \left( 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right) \right) \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = 9.124 \text{ MN}
\]

\[
V_{pl,a,Rd} = \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M0}} = 10.037 \text{ MN}
\]

where \( \eta = 1.2 \) for steel grades up to and including S460

Contribution of the web \( V_{bw,Rd} \)

\[
V_{bw,Rd} = \frac{X_u f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}
\]

The vertical stiffeners at the bracing transverse frames which border the webpanel adjacent to the support P1 and located in span P1-P2, are assumed to be rigid (to be checked by using Section 9 of EN 1993-1-5). They are equally spaced by \( a = 7.5 \text{ m or } 8.33 \text{ m, depending on the span. Near the support P2, the first subpanel has a length } a = 1.5 \text{ m.}

Figure 3-8: Design Plastic resistance moment \( M_{f,Rd} \) of the flanges only at internal support support P2.
Cross-section verifications - Twin-girder bridge at the internal support P2

$k_{vu} = 0$ because there is no longitudinal stiffeners

$a/h_u = 0.679 \leq 1$

\[ k_t = 4 + 5.34 \left( \frac{h_u}{a} \right)^2 + k_{vu} = 15.592 \]

\[ \frac{0.83}{\eta} = 0.692 \leq \frac{\overline{h_u}}{37.4 t_e c_u \sqrt{k_t}} = 0.954 < 1.08 \]

\[ \Rightarrow \chi_u = \frac{0.83}{\overline{h_u}} = 0.87 \]

\[ V_{bw, Rd} = \frac{\chi_u f_{su} h_u t_u}{\sqrt{3} \gamma_{M1}} = 6.613 \text{ MN} \]

**Contribution from the flanges** $V_{bf, Rd}$

\[ V_{bf, Rd} = \frac{b_f t_f f_{sf}}{c_G M} \left( 1 - \left( \frac{M_{Ed}}{M_{f, Rd}} \right)^{\frac{1}{2}} \right) \]

$b_f$ and $t_f$ are taken for the flange which provides the least axial resistance,

$b_f$ being taken as not larger than $15 \cdot c \cdot t_f$ on each side of the web,

The lower flange of the cross-section is a structural steel section whereas its upper flange is a composite section (structural steel + reinforcing steel). The formulae for calculating $V_{bf, Rd}$ should be used with the lower steel flange properties.

The design plastic bending resistance $M_{f, Rd}$ of the cross-section consisting of the flanges and the reinforcing steel should be first calculated. $M_{f, Rd}$ is calculated as $M_{pl, Rd}$ neglecting the web contribution.

To calculate $M_{f, Rd}$, the position of the Plastic Neutral Axis (PNA) is determined as:

- **Design plastic resistance of total reinforcements of the slab:**
  \[ N_{su} + N_{sl} = (A_{su} + A_{sl}) \frac{f_{sl}}{\gamma_s} = 10.339 \text{ MN} \]

- **Design plastic resistance of the structural steel top flange:**
  \[ N_{aaf} = A_{aaf} \frac{f_{af}}{\gamma_{M0}} = 23.94 \text{ MN} \]

- **Design plastic resistance of the structural steel bottom flange:**
  \[ N_{abf} = A_{abf} \frac{f_{af}}{\gamma_{M0}} = 29.925 \text{ MN} \]

- **Location of the Plastic Neutral Axis (PNA):**
  \[ N_{abf} + N_{aaf} = 53.865 \text{ MN} \geq N_{su} + N_{sl} = 10.339 \text{ MN} \]
  and $N_{abf} = 29.925 \text{ MN} < N_{aaf} + N_{su} + N_{sl} = 34.279 \text{ MN}$

Thus, the PNA is deduced to be located in the top flange at a distance $z_{pl}$ from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA deduced:

\[ z_{pl} = \frac{2hb_pf_{sf}N_{su} + N_{sl} - N_{abf} - N_{aaf}}{2b_f f_{sf}} = 2.314 \text{ m} \]
The design plastic resistance moment of the flanges only is calculated from the position of the PNA: $M_{f,Rd} = 71.569 \text{ MNm}$
EN 1993-1-5, 9.3.5, Welds
(1) The web to flange welds may be designed for the nominal shear flow \( V_{Ed} / h_w \) if \( V_{Ed} \) does not exceed \( \chi_{w} f_{yw} h_w t / (\sqrt{3} \gamma_{M1}) \). For larger values \( V_{Ed} \) the weld between flanges and webs should be designed for the shear flow \( \eta f_{yw} t / (\sqrt{3} \gamma_{M1}) \).

(2) In all other cases welds should be designed to transfer forces along and across welds making up.

EN 1993-1-5, 9.3, Shear

EN 1993-1-5, 5.5, Verification
(1) The verification should be performed as follows:
\[
\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leq 1
\]
where \( V_{Ed} \) is the design shear force including shear from torque.

EN 1994-2, 6.2.2.4(1)
(1) Where the vertical shear force \( V_{Ed} \) exceeds half the shear resistance \( V_{f,Rd} \) given by \( V_{pl,Rd} \) in 6.2.2.2 or \( V_{b,Rd} \) in 6.2.2.3, whichever is the smaller, allowance should be made for its effect on the resistance moment.

Further explanations on the M-V-interaction
See Paragraph 3.1.2.5.3, page 93.

EN 1994-2, 6.2.2.4(3)
(3) For cross-sections in Class 3 and 4, EN 1993-1-5, 7.1 is applicable using the calculated stresses of the composite section.

EN 1993-1-5, 7.1(2)
(2) The criterion given in (1) should be verified at all sections other than those located at a distance less than \( h_w/2 \) from a support with vertical stiffeners.
\[ c = a \left( 0.25 + \frac{1.6b_{f}r_{f} f_{y} f_{w}}{h_{w} h_{w}} \right) = 0.545 \text{ m} \]

\[ V_{bf,Rd} = \frac{b_{f}r_{f} f_{y} f_{w}}{c'y_{M1}} \left( 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^{2} \right) = 0.621 \text{ MN} \]

In this case, the contribution of the flanges \( V_{bf,Rd} \) is not negligible compared to contribution from the web and represents 8.6 % of the design shear buckling resistance.

\[ V_{b,Rd} = V_{bu,Rd} + V_{bf,Rd} = 6.613 + 0.621 = 7.234 \text{ MN} \leq \frac{\eta f_{w} h_{w} t_{w}}{\gamma_{M1}^{3}} = 9.124 \text{ MN} \]

\[ V_{Rd} = \min (V_{b,Rd}; V_{pl.a,Rd}) = \min(7.234; 10.037) = 7.234 \text{ MN} \]

The following checks should also be performed:

- The web to flange weld should be designed for the shear stress per unit length of \( \frac{\eta f_{w} h_{w} t_{w}}{\gamma_{M1}^{3}} \);
- The tranverse stiffeners along the webpanel edges (and possibly the longitudinal stiffeners) should act as rigid end post;

The flanges are not completely used for resisting to bending moment (i.e. \( M_{Ed} \leq M_{f,Rd} \) which is verified in the example: \( M_{Ed} = 65.44 \text{ MNm} \leq M_{f,Rd} = 71.569 \text{ MNm} \)

**Cross-section verification**

The verification should be performed as follows:

\[ V_{Ed} = 6.087 \text{ MN} \leq V_{Rd} = \min (7.234; 10.037) = 7.234 \text{ MN} \]

\[ \eta_{s} = \frac{V_{Ed}}{V}_{Rd} = 0.841 \leq 1 \]

\[ \Rightarrow \quad \text{Shear resistance is verified!} \]

### 3.1.5.5.3 M-V-interaction

\[ V_{Ed} = 6.087 \text{ MN} \geq 0.5 \times V_{Rd} = 3.617 \text{ MN} \]

Therefore the M-V-interaction should be checked.

\[ \frac{M_{Ed}}{M_{f,Rd}} = 0.914 \leq 1 \]

\[ \frac{V_{Ed}}{V_{bw,Rd}} = 0.893 \leq 1 \]

\( M_{Ed} < M_{f,Rd} \) so that according to EN 1993-1-5, 7.1 (1), there is no interaction. It means that the flanges are enough to resist alone the bending moment so that the entire web can be used for the resistance to the shear force.

Then flanges of the steel girder take the bending moment and web of the steel beam takes shear force.

\[ \Rightarrow \quad \text{There is no interaction.} \]
Further explanations on the determination of the cross-section class
See Paragraph 3.1.5.4, page 117ff.

Further explanations on the bending resistance check
See Paragraph 3.1.2.5.1, page 87f. and Paragraph 3.1.5.5.1, page 121ff.
3.1.5.6 Subpanel 2 - Geometry

Follow the same procedure as for subpanel 1 (see Paragraph 3.1.5.1)

At internal support P2 at ULS the concrete slab is in tension over its whole height. Its contribution is therefore neglected in the cross-section resistance.

The geometry of the cross-section of the subpanel 2 is totally the same as the geometry of the cross-section of the subpanel 1. Only the length of the panel change (a = 2.5 m)

3.1.5.7 Subpanel 2 - Material properties

See Paragraph 3.1.5.2.

The material properties of the cross-section of the subpanel 2 are identical to the material properties of the cross-section of the subpanel 1.

3.1.5.8 Subpanel 2 - Internal forces and moments

The internal forces and moments in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{Ed} = 58.222 \text{ MNm (at the internal support P2: } x = 111.5 \text{ m)} \]
\[ V_{Ed} = 5.843 \text{ MN (at the internal support P2: } x = 111.5 \text{ m)} \]

3.1.5.9 Subpanel 2 - Determination of the cross-section class

See Paragraph 3.1.5.4.

As the geometry of the cross-section does not change compared to subpanel 1, the cross-section class is the same for subpanel 2.

3.1.5.10 Subpanel 2 - Elastic section analysis

3.1.5.10.1 Bending resistance check

The plate buckling verification of the panel should be carried out for the stress resultants at a distance 0.4·a or 0.5·b: \( \min\{0.4\cdot a ; 0.5\cdot b\} = \min\{1 ; 1.105\} = 1 \text{ m} \)

Then the value of the bending moment becomes: \( M_{Ed} (\min\{0.4\cdot a ; 0.5\cdot b\}) = 53.659 \text{ MNm} \)

The stress on each level of the cross-section can be easily determined:

\[
\sigma_{\text{ab_{eff}}} = \frac{-M_{a,Ed} h_{a,eff}}{I_{u,eff}} + \frac{-M_{c,Ed} h_{c,eff}}{I_{u,eff}} = 259.181 \text{ N/mm}^2 \leq f_{s_{eff}} = 315 \text{ N/mm}^2
\]
\[
\sigma_{\text{ab_{eff}}} = \frac{-M_{a,Ed} (h_{a,eff} - t_f)}{I_{u,eff}} + \frac{-M_{c,Ed} (h_{c,eff} - t_f)}{I_{u,eff}} = 238.478 \text{ N/mm}^2
\]
\[
\leq \min\{f_{y_{eff}} ; f_{y_{du}}\} = 315 \text{ N/mm}^2
\]

\[
\sigma_{\text{st_{eff}}} = \frac{M_{a,Ed} (h - t_f - h_{a,eff})}{I_{u,eff}} + \frac{M_{c,Ed} (h - t_f - h_{c,eff})}{I_{u,eff}} = -243.132 \text{ N/mm}^2
\]
\[
\leq \min\{f_{y_{eff}} ; f_{y_{du}}\} = 315 \text{ N/mm}^2
\]

\[
\sigma_{\text{st_{eff}}} = \frac{M_{a,Ed} (h - h_{a,eff})}{I_{u,eff}} + \frac{M_{c,Ed} (h - h_{c,eff})}{I_{u,eff}} = -263.835 \text{ N/mm}^2 \leq f_{s_{eff}} = 315 \text{ N/mm}^2
\]
Further explanations on the shear resistance check
See Paragraph 3.1.2.5.2, page 89ff.
The bending moment resistance is governed by the resistance of the top flange:

\[ \eta_h = \frac{\sigma_{\text{bend}}}{f_{\text{eff}}} = 0.838 \leq 1 \]

\[ \Rightarrow \text{ Bending resistance is verified!} \]

### 3.1.5.10.2 Shear resistance check

The web should be checked in terms of shear buckling if:

\[ \frac{h_w}{t_w} = 116.316 > \frac{31}{\eta} \varepsilon_w \sqrt{\bar{k}_w} = 62.035 \text{ so that the check is necessary.} \]

The maximum design shear resistance is given by:

\[ V_{Rd} = \min (V_{b,Rd} + V_{pl,a,Rd}) \]

where \[ V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \eta f_w t_w f_{pl,a} \sqrt{3\gamma_{M1}} = 9.124 \text{ MN} \]

\[ V_{pl,a,Rd} = 10.037 \text{ MN} \]

where \( \eta = 1.2 \) for steel grades up to and including S460

#### Contribution of the web \( V_{bw,Rd} \)

\[ V_{bw,Rd} = \frac{\chi_w f_w h_w f_{pl,a}}{\sqrt{3\gamma_{M1}}} \]

The vertical stiffeners at the bracing transverse frames which border the webpanel adjacent to the support P2 and located in span P1-P2, are assumed to be rigid (to be checked by using Section 9 of EN 1993-1-5). They are equally spaced by \( a = 7.5 \text{ m} \). Near the support P2, the second subpanel has a length \( a = 2.5 \text{ m} \).

\( k_{zt} = 0 \) because there is no longitudinal stiffeners

\[ a/h_w = 1.131 \geq 1 \]

\[ k_z = 5.34 + 4\left(\frac{h_w}{a}\right)^2 + k_{zt} = 8.466 \]

\[ \bar{\chi}_w = \frac{h_w}{37.4 t_w e_w \sqrt{k_z}} = 1.295 \geq 1.08 \]

\[ \Rightarrow \chi_w = \frac{1.37}{(0.7 + \bar{\chi}_w)} = 0.687 \]

\[ V_{bw,Rd} = 5.221 \text{ MN} \]
Further explanations on the M-V-interaction

See Paragraph 3.1.2.5.3, page 93 and Paragraph 3.1.5.5.3, page 131.
Contribution from the flanges $V_{bf,Rd}$

The design plastic resistance moment of the flanges only is calculated from the position of the PNA (see Paragraph 3.1.5.5.2): $M_{f,Rd} = 71.569$ MNm

\[
c = a \left(0.25 + \frac{1.6b_{f} t_{f} f_{sf}}{b_{w} t_{w} f_{yw}}\right) = 0.909 \text{ m}
\]

\[
V_{bf,Rd} = \frac{b_{f} t_{f} f_{sf}}{c' \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^{2}\right) = 0.769 \text{ MN}
\]

In this case, the contribution of the flanges $V_{bf,Rd}$ is not negligible compared to the contribution from the web and represents 12.8% of the design shear buckling resistance.

\[
V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 5.221 + 0.769 = 5.99 \text{ MN} \leq \frac{f_{yw} h_{w} t_{w}}{\sqrt{3} \gamma_{M1}} = 9.124 \text{ MN}
\]

\[
V_{Ed} = \min (V_{b,Rd}; V_{pl,a,Rd}) = \min (5.99; 9.124) = 5.99 \text{ MN}
\]

The flanges are not completely used to resist the bending moment (i.e. $M_{Ed} \leq M_{f,Rd}$ which is verified in the example: $M_{Ed} = 58.222$ MNm $\leq M_{f,Rd} = 71.569$ MNm)

Cross-section verification

The verification should be performed as follows:

\[
V_{Ed} = 5.843 \text{ MN} \leq V_{RD} = \min (5.99; 10.037) = 5.99 \text{ MN}
\]

\[
\eta_{3} = \frac{V_{Ed}}{V_{rd}} = 0.975 \leq 1
\]

⇒ Shear resistance is verified!

3.1.5.10.3 M-V-interaction

\[
V_{Ed} = 5.843 \text{ MN} \geq 0.5 V_{RD} = 2.995 \text{ MN}
\]

Therefore the M-V-interaction should be checked.

\[
\frac{M_{Ed}}{M_{f,Rd}} = 0.814 \leq 1 ; \quad \frac{V_{Ed}}{V_{bw,Rd}} = 0.975 \leq 1
\]

$M_{Ed} < M_{f,Rd}$ so that according to EN 1993-1-5, 7.1 (1), there is no interaction. It means that the flanges are enough to resist alone the bending moment so that the entire web can be used for the resistance to the shear force.

Then flanges of the steel girder take the bending moment and web of the steel beam takes shear force.

⇒ There is no interaction.

3.1.5.11 Subpanel 3 - Geometry

Follow the same procedure as for sup-panel 1 (see Paragraph 3.1.5.1)

At internal support P2 at ULS the concrete slab is in tension over its whole height. Its contribution is therefore neglected in the cross-section resistance.

The geometry of the cross-section of the subpanel 3 is totally the same as the geometry of the cross-section of the subpanel 1. Only the length of the panel change $(a = 4.333 \text{ m})$
Further explanations on the determination of the cross-section class
See Paragraph 3.1.5.4, page 117ff.

Further explanations on the bending resistance check
See Paragraph 3.1.2.5.1, page 87f. and Paragraph 3.1.5.1, page 121ff.
3.1.5.12 Subpanel 3 - Material properties

See Paragraph 3.1.5.2.

The material properties of the cross-section of the subpanel 3 are identical to the material properties of
the cross-section of the subpanel 1.

3.1.5.13 Subpanel 3 - Internal forces and moments

The internal forces and moments in this cross-section are (see Figure 2-31 and Figure 2-32):

\[ M_{Ed} = 47.188 \text{ MNm (at the internal support P2: } x = 114 \text{ m)} \]
\[ V_{Ed} = 5.435 \text{ MN (at the internal support P2: } x = 114 \text{ m)} \]

3.1.5.14 Subpanel 3 - Determination of the cross-section class

See Paragraph 3.1.5.4.

As the geometry of the cross-section does not change compared to subpanel 1, the cross-section class is
the same for subpanel 3.

3.1.5.15 Subpanel 3 - Elastic section analysis

3.1.5.15.1 Bending resistance check

The plate buckling verification of the panel should be carried out for the stress resultants at a distance
0.4·a or 0.5·b: \( \min(0.4·a ; 0.5·b) = \min(1.733 ; 1.105) = 1.105 \text{ m} \)

Then the value of the bending moment becomes: \( M_{Ed} (\min(0.4a ; 0.5b)) = 42.707 \text{ MNm} \)

The stress on each level of the cross-section can be easily determinated:

\[
\sigma_{abeff} = \frac{-M_{a,Ed}h_{a,eff} + -M_{c,Ed}h_{c,eff}}{I_{eff}} = 209.739 \text{ N/mm}^2 \leq f_{y,eff} = 315 \text{ N/mm}^2
\]
\[
\sigma_{abeff} = \frac{-M_{a,Ed}(h_{a,eff} - t_f) + -M_{c,Ed}(h_{c,eff} - t_f)}{I_{eff}} = 193.052 \text{ N/mm}^2
\]
\[
\leq \min(f_{y,eff} ; f_{y,eff}) = 315 \text{ N/mm}^2
\]
\[
\sigma_{abeff} = \frac{M_{a,Ed}(h - t_f - h_{a,eff}) + M_{c,Ed}(h - t_f - h_{c,eff})}{I_{eff}} = -195.141 \text{ N/mm}^2
\]
\[
\leq \min(f_{y,eff} ; f_{y,eff}) = 315 \text{ N/mm}^2
\]
\[
\sigma_{abeff} = \frac{M_{a,Ed}(h - h_{a,eff}) + M_{c,Ed}(h - h_{c,eff})}{I_{eff}} = -211.828 \text{ N/mm}^2 \leq f_{y,eff} = 315 \text{ N/mm}^2
\]
\[
\sigma_{abeff} = \frac{M_{c,Ed}(h + c_e - h_{eff})}{I_{eff}} = -123.815 \text{ N/mm}^2 \leq f_{y,eff} = 434.783 \text{ N/mm}^2
\]
\[
\sigma_{abeff} = \frac{M_{c,Ed}(h + e - c_w - h_{eff})}{I_{eff}} = -144.905 \text{ N/mm}^2 \leq f_{y,eff} = 434.783 \text{ N/mm}^2
\]

The bending moment resistance is governed by the resistance of the top flange:

\[ \eta_1 = \frac{\sigma_{abeff}}{f_{y,eff}} = 0.672 \leq 1 \]

\[ \Rightarrow \text{ Bending resistance is verified!} \]
Further explanations on the shear resistance check

See Paragraph 3.1.2.5.2, page 89ff.
3.1.5.15.2 Shear resistance check

The web should be checked in terms of shear buckling if:

\[
\frac{h_u}{t_w} = 116.316 > \frac{31}{\eta} \varepsilon_{u} \sqrt{k_t} = 53.856
\]

then the web should be checked in terms of shear buckling.

The maximum design shear resistance is given by

\[
V_{Rd} = \min (V_{b,Rd}; V_{pl,a,Rd})
\]

where

\[
V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_u t_w}{\sqrt{3} \gamma_{M1}} = 9.124 \text{ MN}
\]

\[
V_{pl,a,Rd} = \frac{\eta f_{yw} h_u t_w}{\sqrt{3} \gamma_{M0}} = 10.037 \text{ MN}
\]

where \(\eta = 1.2\) for steel grades up to and including S460

**Contribution of the web** \(V_{bw,Rd}\)

\(k_{uw} = 0\) because there is no longitudinal stiffeners

\[
a/h_u = 1.961 \geq 1
\]

\[
k_t = 5.34 + 4 \left(\frac{h_u}{a}\right)^2 = 6.381
\]

\[
\bar{\lambda}_u = \frac{h_u}{37.4 \varepsilon_{u} \sqrt{k_t}} = 1.492 \geq 1.08
\]

\[
\Rightarrow \chi_u = \frac{1.37}{0.7 + \bar{\lambda}_u} = 0.625
\]

\[
V_{bw,Rd} = 4.753 \text{ MN}
\]

**Contribution from the flanges** \(V_{bf,Rd}\)

The design plastic resistance moment of the flanges only is calculated from the position of the PNA (see Paragraph 3.1.5.5.2): \(M_{f,Rd} = 71.569 \text{ MNm}\)

\[
c = a \left(0.25 + \frac{1.6b_t f_{yf} f_{sf}}{h_u f_{yw}}\right) = 1.576 \text{ m}
\]

\[
V_{bf,Rd} = \frac{b_t f_{yf} f_{sf}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) = 0.742 \text{ MN}
\]

The design plastic resistance moment of the flanges only is calculated from the position of the PNA: \(M_{f,Rd} = 71.569 \text{ MNm}\)

In this case, the contribution of the flanges \(V_{bf,Rd}\) is not negligible compared to contribution from the web and represent 13.5 % of the design shear buckling resistance.

\[
V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 4.753 + 0.742 = 5.494 \text{ MN} \leq \frac{\eta f_{yw} h_u t_w}{\sqrt{3} \gamma_{M1}} = 9.124 \text{ MN}
\]

\[
V_{Rd} = \min (V_{b,Rd}; V_{pl,a,Rd}) = \min(5.494; 10.037) = 5.494 \text{ MN}
\]
Further explanations on the M-V-interaction
See Paragraph 3.1.2.5.3, page 93 and Paragraph 3.1.5.5.3, page 131.
The flanges are not completely used for resisting to bending moment (i.e. \( M_{Ed} \leq M_{f,Rd} \) which is verified in the example: \( M_{Ed} = 47.188 \text{ MNm} \leq M_{f,Rd} = 71.569 \text{ MNm} \)

**Cross-section verification**

The verification should be performed as follows:

\[
V_{Ed} = 5.435 \text{ MN} \leq V_{Rd} = min (5.494; 10.037) = 5.494 \text{ MN}
\]

\[
\eta_3 = \frac{V_{Ed}}{V_{Rd}} = 0.989 \leq 1
\]

Therefore the cross-section at support P2 is checked under shear force.

**3.1.5.15.3 M-V-interaction**

\[
V_{Ed} = 5.435 \text{ MN} \geq 0.5 \cdot V_{Rd} = 2.747 \text{ MN}
\]

Therefore the M-V-interaction should be checked.

\[
\frac{M_{Ed}}{M_{f,Rd}} = 0.659 \leq 1; \quad \frac{V_{Ed}}{V_{bw,Rd}} = 1.106 \leq 1; \quad \frac{V_{Ed}}{V_{p,Rd}} = 0.989 \leq 1
\]

\( M_{Ed} < M_{f,Rd} \) so that according to EN 1993-1-5, 7.1 (1), there is no interaction. It means that the flanges are enough to resist alone the bending moment so that the entire web can be used for the resistance to the shear force.

Thus, the flanges of the steel girder take the bending moment and web of the steel beam takes shear force.

\( \Rightarrow \) There is no interaction.

**3.2 Box-girder bridge**

**3.2.1 General**

According to the shape of the bending moment and shear force diagrams at ULS (see Figure 2-36 and Figure 2-37), the two different critical sections to check are displayed in Figure 3-9:

- At mid-span P1-P2, see Section 3.2.2
- At the internal support P3, see Section 3.2.3

![Figure 3-9: Checked sections of the box-girder bridge.](image)

For each critical section, the verification is done on a panel which is located between two vertical stiffeners.
### 3.2.2 Check of cross-section at mid-span P1-P2

#### 3.2.2.1 Geometry

At mid-span P1-P2, at ULS the concrete slab is almost in compression over its whole height. Its contribution is therefore taken into account in the cross-section resistance.

![Cross-section of the box-girder bridge at mid-span P1-P2](image)

**Figure 3-10: Cross-section of the box-girder bridge at mid-span P1-P2.**

<table>
<thead>
<tr>
<th>General properties of the box-girder bridge in cross-section P1-P2</th>
<th>Main areas of the different parts of the composite section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_1 = L_2 = 120 \text{ m} )</td>
<td>( A_{uf} = t_f b_f = 0.075 \text{ m}^2 )</td>
</tr>
<tr>
<td>( a = 4 \text{ m} )</td>
<td>( A_{uw} = t_u h_u = 0.086 \text{ m}^2 )</td>
</tr>
<tr>
<td>( h = 4 \text{ m} )</td>
<td>( A_{st.w} = 2 h_{st.w} t_{st,w} + b_{slab} t_{slab} = 184.451 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( h_u = \frac{(h - t_{sf} - t_p)}{\cos(\theta_u)} = 4.763 \text{ m} )</td>
<td>( A_{uf} = t_f b_f = 0.163 \text{ m}^2 )</td>
</tr>
<tr>
<td>( t_u = 18 \text{ mm} )</td>
<td>( A_{ur} = \frac{\pi d_{ur}^2}{4} = 2.011 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( b_p = 6500 \text{ mm} )</td>
<td>( A_{sur} = n_{ur} A_{ur} = 332.525 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( t_p = 25 \text{ mm} )</td>
<td>( A_{lr} = \frac{\pi d_{lr}^2}{4} = 2.011 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( t_{slab} = 32.5 \text{ cm} )</td>
<td>( A_{slr} = n_{lr} A_{lr} = 332.525 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( \phi_u = 16 \text{ mm} )</td>
<td>( A_s = t_{slab} b_{slab} = 6.987 \text{ m}^2 )</td>
</tr>
<tr>
<td>( \phi_u = 16 \text{ mm} )</td>
<td>(see notation and Figure 3-10)</td>
</tr>
<tr>
<td>( s_{ur} = 130 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>( s_{lr} = 130 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>( c_{ur} = 60 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>( c_{lr} = 60 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>( b_{slab} = 21.5 \text{ m} )</td>
<td></td>
</tr>
</tbody>
</table>
3.2.2.2 Material properties

Structural steel

\[ f_y(t_w) = 345 \text{ N/mm}^2 \text{ because } 16 \text{ mm} < t_w = 18 \text{ mm} \leq 40 \text{ mm} \text{ (see Table 2-4)} \]

\[ \epsilon(t_w) = \frac{\sqrt{235 N/\text{mm}^2}}{f_y(t_w)} = 0.825 \]

\[ f_y(t_p) = 345 \text{ N/mm}^2 \text{ because } 16 \text{ mm} < t_p = 25 \text{ mm} \leq 40 \text{ mm} \text{ (see Table 2-4)} \]

\[ \epsilon(t_p) = \frac{\sqrt{235 N/\text{mm}^2}}{f_y(t_p)} = 0.825 \]

\[ f_y(t_f) = 335 \text{ N/mm}^2 \text{ because } 40 \text{ mm} < t_f = 50 \text{ mm} \leq 63 \text{ mm} \text{ (see Table 2-4)} \]

\[ \epsilon(t_f) = \frac{\sqrt{235 N/\text{mm}^2}}{f_y(t_f)} = 0.838 \]

\[ f_y(t_{st,w}) = 315 \text{ N/mm}^2 \text{ because } t_{st,w} = 15 \text{ mm} \leq 16 \text{ mm} \text{ (see Table 2-4)} \]

\[ \epsilon(t_{st,w}) = \frac{\sqrt{235 N/\text{mm}^2}}{f_y(t_{st,w})} = 0.814 \]

\[ f_{yd}(t_w) = \frac{f_y(t_w)}{\gamma_{M0}} = 345 \text{ N/mm}^2, \quad f_{yd}(t_p) = \frac{f_y(t_p)}{\gamma_{M0}} = 345 \text{ N/mm}^2, \]

\[ f_{yd}(t_f) = \frac{f_y(t_f)}{\gamma_{M0}} = 335 \text{ N/mm}^2, \quad f_{yd}(t_{st,w}) = \frac{f_y(t_{st,w})}{\gamma_{M0}} = 355 \text{ N/mm}^2 \]

\[ E_a = 210000 \text{ N/mm}^2 \]

Concrete

\[ f_{ck} = 35 \text{ N/mm}^2 \]

\[ f_{cd} = \frac{f_{ck}}{\gamma_c} = 23.333 \text{ N/mm}^2 \]

\[ E_{cm} = 34077 \text{ N/mm}^2 \]

\[ n = \frac{E_a}{E_{cm}} = \frac{210000}{34077} = 6.163 \]

Reinforcement

\[ f_{sk} = 500 \text{ N/mm}^2 \]

\[ f_{sd} = \frac{f_{sk}}{\gamma_s} = 434.734 \text{ N/mm}^2 \]

\[ E_s = E_a = 210000 \text{ N/mm}^2 \]

3.2.2.3 Internal forces and moments

The internal forces and moments are obtained from the design model at ULS based on the cracked global analysis (see Section 2.4.2.6.2) and, considering the construction steps, they are as follows for the whole box section (see Figure 2-36 and Figure 2-37):
Further explanations on the determination of the cross-section class

See Paragraph 3.1.5.4, page 117ff.
Cross-section verifications - Box-girder bridge at mid-span P1-P2

\[ M_{Ed} = 2 \cdot 150.411 \, \text{MNm} = 300.822 \, \text{MNm} \text{ for the whole cross-section} \]

\[ V_{Ed} = 2 \cdot 2.697 \, \text{MN} = 5.394 \, \text{MN} \]

i.e. \[ V_{Ed, \text{proj}} = \frac{V_{Ed}}{2 \cos(\theta_w)} = 3.273 \, \text{MN} \text{ in each steel web by taking its inclination into account} \]

where \[ \theta_w = a \tan(\frac{12 - b_p}{2h_p}) = 0.602 = 34.509^\circ \]

### 3.2.2.4 Reduction due to shear lag effect

Verification if shear lag effect has to be taken into account:

Bridge span: \[ L_1 = 120 \, \text{m} \text{ and } L_2 = 120 \, \text{m} \]

Effective length: \[ L_e = 0.7 \, L_2 = 84 \, \text{m} \]

Considered width: \[ b_0 = b_p/2 = 3.25 \, \text{m} \]

\[ b_0 < L_e/50 \text{ requirement not fulfilled! Shear lag effect has to be taken into account.} \]

Shear lag parameters:

\[ \alpha_0 := \sqrt{\frac{A_{sl}}{2 b_0 t_p}} = 1.297 \]

\[ \kappa := \alpha_0 b_0 \frac{L_e}{L_e} = 0.05 \]

\[ \beta_{ult} := \begin{cases} 1 & \text{if } \kappa \leq 0.02 \\ 1 + \frac{1}{1 + 6 \left( \kappa - \frac{1}{2500 \cdot \kappa} \right) + 1.6 \cdot \kappa^2} & \text{if } 0.02 < \kappa \leq 0.7 \\ \frac{1}{8.6 \cdot \kappa} & \text{otherwise} \end{cases} \]

\[ \beta_{ult} = 0.795 \]

\[ \beta_{ult}^\kappa = 0.989 \]

### 3.2.2.5 Determination of the cross-section class

The web is in tension in its upper part and in tension in its lower part. As the upper flange is perfectly connected to the slab, it is a Class 1 element. To classify the steel web, the position of the Plastic Neutral Axis (PNA) is determined as follows:

- Design plastic resistance of the bottom flange
  \[ N_{af,bf} = \left( n_d \left[ t_{af} f_{yd} (t_{af,w})(b_2 + 2b_1) + t_p f_{yd} (t_p)(b_1 + b_{ab}) \right] + t_p f_{yd} (t_p)(0.2m + b_{ab}) \right) \beta_{ult}^{\kappa} = 95.967 \, \text{MN} \]

- Design plastic resistance of the 2 webs
  \[ N_{af,w} = 2 \left[ (h - t_{af} - t_p) t_{af,bf} (t_{af}) \right] = 59.158 \, \text{MN} \]
Relations to find the location of the Plastic Neutral Axis (PNA) under positive moment $M_{Pl,rd}$

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{abf} \geq N_{aw} + N_{atf} + N_c$</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>$N_{abf} + N_{aw} \geq N_{atf} + N_c$ and $N_{abf} &lt; N_{aw} + N_{atf} + N_c$</td>
<td>PNA in the web</td>
</tr>
<tr>
<td>$N_c \geq N_{atf}$ and $N_{abf} + N_{aw} &lt; N_{atf} + N_c$</td>
<td>PNA in the top flange</td>
</tr>
<tr>
<td>$N_c &gt; N_{abf} + N_{aw} + N_{atf}$</td>
<td>PNA in the slab</td>
</tr>
</tbody>
</table>

Further explanations on the bending resistance check
See Paragraph 3.1.2.5.1, page 87f.

Further explanations on the shear resistance check
See Paragraph 3.1.2.5.2, page 89ff.
• Design plastic resistance of the 2 structural steel top flange
  \[ N_{u, sf} = 2b_{sf}t_{sf}f_{yd}(t_{sf}) = 59.158 \text{ MN} \]

• Design plastic resistance of the concrete slab in compression
  \[ N_{u, sf} = 0.85t_{slab}b_{slab}f_{cd} = 138.585 \text{ MN} \]

• Location of the Plastic Neutral Axis (PNA)
  \[ N_{abh} + N_{aw} + N_{afl} = 206.484 \text{ MN} \geq N_c = 138.585 \text{ MN} \]
  and \[ N_{abh} + N_{aw} = 156.234 \text{ MN} < N_c + N_{afl} = 188.835 \text{ MN} \]
  Thus, the PNA is deduced to be located in the top flange at a distance \( z_{pl} \) from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA gives:

  \[ z_{pl} = \frac{4hb_{sf}f_{yd}(t_{sf}) + N_c - N_{abh} - N_{aw} - N_{afl}}{4b_{sf}f_{yd}(t_{sf})} = 3.967 \text{ m} \]
  Thus, the whole web is in tension.

Conclusion: the cross-section at mid-span P1-P2 is in Class 1 and is checked by a plastic section analysis.

### 3.2.2.6 Bending resistance verification

The design plastic resistance moment is calculated from the position of the PNA (see Paragraph 3.2.2.5):

\[ M_{pl,Rd} = N_{u,hf}\left(z_{pl} - \frac{t_p}{2}\right) + N_{u,aw}\left(t_{pl} - \frac{(h - t_{sf} + t_p)}{2}\right) + N_c\left(h + \frac{t_{slab}}{2} - z_{pl}\right) + \frac{(h - z_{pl})^2}{2}b_{sf}f_{yd}(t_{sf}) + \frac{(h - t_f - z_{pl})^2}{2}b_{sf}f_{yd}(t_{sf}) \]

\[ = 524.044 \text{ MNm} \]

Reinforcement in compression in the concrete slab is neglected according EN 1994-2, 6.2.1.2(1).

\[ M_{Ed} = 300.822 \text{ MNm} < M_{pl,Rd} = 524.044 \text{ MNm} \]

Bending resistance is verified!

### 3.2.2.7 Shear resistance verification

#### 3.2.2.7.1 Shear in the box-girder webs

The box-girder web is transversally stiffened on both sides at mid-span P1-P2 (\( a_w = 4 \text{ m} \)).

Stiffened webpanel

To evaluate the shear buckling coefficient of the stiffened webpanel, the second moment of area of the longitudinal stiffener must be calculated according to EN 1993-1-5, Figure 5.3:

\[ 15\varepsilon(t_w)t_w = 0.223 \text{ m} \leq \frac{h_{w,w}}{2} = 0.25 \text{ m} \]

The elastic neutral axis of the web stiffener with the width 15\( \varepsilon(t_w)t_w \) on both sides of the stiffener is:
The second moment of area of the web stiffener is:

\[
I_{\text{st}} = I_{d,w} = b_{2,w} t_{d,w} \left( h_{d,w} - z_{st,w} \right)^2 + 2 \left[ \frac{t_{d,w} h_{d,w}^3}{12} + t_{d,w} h_{d,w} \left( h_{d,w} - z_{st,w} \right)^2 \right] + 4 \cdot 15 \varepsilon(t_w) t_w z_{st,w}^2
\]

\[
= 9.829 \cdot 10^{-4} \text{ m}^4
\]

According to EN 1993-1-5, Annex 3(2), as there is only one stiffener in the web and the aspect ratio is \( \alpha_w = \frac{a_w}{h_w} = 0.84 \leq 3 \), the shear buckling coefficient is:

\[
k_{r,w} = 4.1 + \frac{6.3 + 0.18 \frac{I_{d,w}}{t_w h_w}}{\alpha_w^2} + 2.2 \sqrt{\frac{I_{d,w}}{t_w h_w}} = 29.287
\]

The transverse stiffeners of the bracings frames bordering the webpanel close to mid-span P1-P2 are assumed to be rigid.

\[
\frac{h_w}{t_w} = \frac{4763}{18} = 264.617 > \frac{31}{\eta} \varepsilon(t_w) \sqrt{k_{r,w}} = 115.383
\]

Thus, the stiffened webpanel must be checked against shear buckling.

The reduced slenderness of the stiffened webpanel:

\[
\bar{\lambda}_w = \frac{h_w}{37.4 \sqrt{\varepsilon(t_w) \sqrt{k_{r,w}}}} = 1.584
\]

**NOTE:** Alternatively, the reduced slenderness can be calculated by another way which gives the same results.

The elastic critical shear buckling stress is given by:

\[
\tau_{cr} = k_{r,w} \sigma_E = 79.384 \text{ MPa}
\]

with \( \sigma_E = \frac{\pi^2 E_I t_w^2}{12(1-\nu^2) h_w^2} = 2.711 \text{ MPa} \)

\[
\Rightarrow \quad \bar{\lambda}_w = \sqrt{\frac{f_y(t_w)}{\tau_{cr} \lambda}} = 1.584
\]

**Web subpanels**

It is possible that one or both of the two web subpanels are more critical than the stiffened webpanel. Thus, the two web subpanels must be also checked. As the longitudinal stiffeners is located at the middle of the height of the web, the two webs subpanels have the same width and the same reduced slenderness.

According to EN 1993-1-5, Annex 3(1), as the aspect ratio is \( \alpha_w = \frac{a_w}{b_{sp}} = \frac{4}{2.016} = 1.984 \geq 1 \), the shear buckling coefficient is:

\[
k_{r,w,sp} = 5.34 + 4 \left( \frac{b_{sp}}{a_w} \right)^2 = 6.356
\]
EN 1993-1-1, 6.2.6, Shear

(1) The design value of the shear force $V_{Ed}$ at each cross-section should satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1,0$$

where $V_{c,Rd}$ is the design shear resistance. For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in (2). For elastic design $V_{c,Rd}$ is the design elastic shear resistance calculated using (4) and (5).

(2) In the absence of torsion the design plastic shear resistance is given by:

$$V_{pl,Rd} = A_v \left( f_v / \sqrt{3} \right) \gamma_{M0}$$

where $A_v$ is the shear area.

EN 1993-1-5, 5.5, Verification

(1) The verification should be performed as follows:

$$\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leq 1$$

where $V_{Ed}$ is the design shear force including shear from torque.
Cross-section verifications - Box-girder bridge at mid-span P1-P2

\[
\frac{b_{w,sp}}{t_w} = \frac{2016}{18} = 112.015 > \frac{31}{\eta} e(t_w) \frac{1}{k_{r,w}} = 53.754
\]

Thus, the web subpanels must be checked against shear buckling.

The reduced slenderness of the web subpanel:

\[
\bar{x}_{w,sp} = \frac{b_{w,sp}}{37.4t_w e(t_w) \frac{1}{k_{r,w,sp}}} = 1.439
\]

Shear resistance verification

Thus, it is the stiffened webpanel which is critical: \( \bar{x}_w = \max(\bar{x}_w, \bar{x}_{w,sp}) = 1.584 \)

As the webpanel close to mid-span P1-P2 is assumed to be rigid and \( 1.08 \leq \bar{x}_w \), the reduction factor is:

\[
\chi_w = \frac{1.37}{0.7 + \bar{x}_w} = 0.6
\]

The maximum design value of the shear resistance is given by

\[
V_{b,Rd} = \min (V_{b,Rd}; V_{p,a,Rd}) \text{ with } V_{b,Rd} = V_{bw,Rd}
\]

neglecting the flange contribution to the resistance:

\[
V_{bw,Rd} = \min \left( \frac{\chi_w f_y(t_w)h_w t_w}{\sqrt{3} \gamma_{M1}}, \frac{\eta f_y(t_w)h_w t_w}{\sqrt{3} \gamma_{M1}} \right) = 9.312 \text{ MN}
\]

\[
V_{p,a,Rd} = \frac{\eta f_y(t_w)h_w t_w}{\sqrt{3} \gamma_{M0}} = 20.493 \text{ MN}
\]

so that

\[
\left( \eta, \frac{V_{bw,Rd}}{V_{Rd}} = \frac{3.273}{9.312} = 0.351 < 1 \right)
\]

Shear resistance is verified!

Addition of torsional effect

The maximum torque on the box-girder bridge at mid-span P1-P2 is equal to \( M_T = 1.5 \times 8.774 \text{ MNm} = 13.161 \text{ MNm} \) (see Figure 2-35).

The area inside the median line of the cross-section of the box-girder bridge is:

\[
S = \frac{1}{2} (b_i + b_p) \left( h + \frac{t_{slab}}{2} \right) = (12 + 6.5) \left( 4 + \frac{0.325}{2} \right) = 38.503 \text{ m}^2
\]

The shear stress in the web is given by the Brecht formula:

\[
\tau_{Ed,T,web} = \frac{M_T}{2S_{tw}} = 9.495 \text{ MPa}
\]

The shear force in the web due to torque is:

\[
V_{k,T,web} = \tau_{Ed,T,web} f_w h_w = 0.814 \text{ MN}
\]

Thus, the verification of shear including torsional effects gives:
Further explanations on the M-V-interaction

See Paragraph 3.1.2.5.3, page 93.
3.2.2.8 Interaction between bending moment and shear force

\[ \eta_s = \frac{V_{Ed}}{V_{b,Ed,Rd}} = 0.439 < 0.5 \]

⇒ There is no need to check the M-V-interaction.

3.2.3 Check of cross-section at the internal support P3

3.2.3.1 Geometry

As the concrete slab is in tension around the internal support P3, the strength is not taken into account for checking the cross-section. Only the longitudinal slab reinforcement is considered.

![Cross-section of the box-girder bridge at the internal support P3](image)

Figure 3-11: Cross-section of the box-girder bridge at the internal support P3.

<table>
<thead>
<tr>
<th>General properties of the box-girder bridge in cross-section P3</th>
<th>Main areas of the different parts of the composite section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_1 = L_2 = 120 \text{ m} )</td>
<td>( A_{diff,1} = t_{fl,1}b_{fl,1} = 0.15 \text{ m}^2 )</td>
</tr>
<tr>
<td>( a = 2.5 \text{ m} )</td>
<td>( A_{diff,2} = t_{fl,2}b_{fl,2} = 0.126 \text{ m}^2 )</td>
</tr>
<tr>
<td>( h = 4 \text{ m} )</td>
<td>( A_{ww} = t_wh_w = 0.122 \text{ m}^2 )</td>
</tr>
<tr>
<td>( h_w = \frac{(h - t_{fl,1} - t_{fl,2} - t_p)}{\cos(\theta_w)} = 4.533 \text{ m} )</td>
<td>( A_{ww} = 2h_{ww}t_{fl,ww} + b_{ww}t_{fl,ww} = 184.451 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( t_w = 27 \text{ mm} )</td>
<td>( A_{diff} = t_p b_p = 0.488 \text{ m}^2 )</td>
</tr>
<tr>
<td>( b_p = 6500 \text{ mm} )</td>
<td>( A_{ww} = \frac{\pi d_w^2}{4} = 3.142 \text{ cm}^2 )</td>
</tr>
<tr>
<td>( t_p = 75 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>( t_{slab} = 32.5 \text{ cm} )</td>
<td></td>
</tr>
</tbody>
</table>
General properties of the box-girder bridge in cross-section P3

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi_w )</td>
<td>20 mm</td>
</tr>
<tr>
<td>( \phi_h )</td>
<td>16 mm</td>
</tr>
<tr>
<td>( s_w )</td>
<td>130 mm</td>
</tr>
<tr>
<td>( s_h )</td>
<td>130 mm</td>
</tr>
<tr>
<td>( c_w )</td>
<td>60 mm</td>
</tr>
<tr>
<td>( c_h )</td>
<td>60 mm</td>
</tr>
<tr>
<td>( b_{slab} )</td>
<td>21.5 m</td>
</tr>
</tbody>
</table>

Main areas of the different parts of the composite section

<table>
<thead>
<tr>
<th>Area</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_{shw} = n_w A_{sur} )</td>
<td>( A_{sh} = \frac{\pi d_t^2}{4} )</td>
<td>519.571 cm²</td>
</tr>
<tr>
<td>( A_{sh} = n_{pl} A_{slr} )</td>
<td>( A_{sl} = n_{pl} A_{slr} )</td>
<td>332.525 cm²</td>
</tr>
<tr>
<td>( A_c = t_{slab} b_{slab} )</td>
<td>( b_{slab} )</td>
<td>6.987 m²</td>
</tr>
</tbody>
</table>

(see notation and Figure 3-11)

### 3.2.3.2 Material properties

#### Structural steel

\[
\varepsilon(t_{w}) = \sqrt{\frac{235 N/\text{mm}^2}{f_y(t_{w})}} = 0.825
\]

\[
f_y(t_{p}) = 325 N/\text{mm}^2 \text{ because } 63\text{ mm} < t_{p} = 75\text{ mm} \leq 80\text{ mm} \text{ (see Table 2-4)}
\]

\[
\varepsilon(t_{p}) = \sqrt{\frac{235 N/\text{mm}^2}{f_y(t_{p})}} = 0.85
\]

\[
f_y(t_{f,1}) = 315 N/\text{mm}^2 \text{ because } 80\text{ mm} < t_{f,1} = 100\text{ mm} \leq 100\text{ mm} \text{ (see Table 2-4)}
\]

\[
\varepsilon(t_{f,1}) = \sqrt{\frac{235 N/\text{mm}^2}{f_y(t_{f,1})}} = 0.864
\]

\[
f_y(t_{f,2}) = 315 N/\text{mm}^2 \text{ because } 80\text{ mm} < t_{f,2} = 90\text{ mm} \leq 100\text{ mm} \text{ (see Table 2-4)}
\]

\[
\varepsilon(t_{f,2}) = \sqrt{\frac{235 N/\text{mm}^2}{f_y(t_{f,2})}} = 0.864
\]

\[
f_y(t_{st,w}) = 355 N/\text{mm}^2 \text{ because } t_{st,w} = 15\text{ mm} \leq 16\text{ mm} \text{ (see Table 2-4)}
\]

\[
\varepsilon(t_{st,w}) = \sqrt{\frac{235 N/\text{mm}^2}{f_y(t_{st,w})}} = 0.814
\]

\[
f_y(t_{w}) = \frac{f_y(t_{w})}{\gamma_{M0}} = 345 N/\text{mm}^2, \quad f_y(t_{p}) = \frac{f_y(t_{p})}{\gamma_{M0}} = 325 N/\text{mm}^2, \quad f_y(t_{f,1}) = \frac{f_y(t_{f,1})}{\gamma_{M0}} = 315 N/\text{mm}^2
\]

\[
f_y(t_{f,2}) = \frac{f_y(t_{f,2})}{\gamma_{M0}} = 315 N/\text{mm}^2, \quad f_y(t_{st,w}) = \frac{f_y(t_{st,w})}{\gamma_{M0}} = 355 N/\text{mm}^2
\]

\[
E_a = 210000 N/\text{mm}^2
\]

#### Concrete

See Paragraph 3.2.2.2

#### Reinforcement

See Paragraph 3.2.2.2
3.2.3.3 Internal forces and moments

The internal forces and moments are obtained from the design model at ULS based on the cracked global analysis (see Section 2.4.2.6.2) and, considering the construction steps, they are as follows for the whole box section (see Figure 2-36 and Figure 2-37):

\[ M_{Ed} = 2 \cdot 369.889 \text{ MNm} = -739.778 \text{ MNm} \text{ for the cross-section} \]

\[ V_{Ed} = 2 \cdot 16.617 \text{ MN} = 33.234 \text{ MN} \text{ for the whole cross-section} \]

i.e. \[ \frac{V_{Ed, proj}}{2} = \frac{V_{Ed}}{2 \cos(\theta_w)} = 20.165 \text{ MN in each steel web by taking its inclination into account} \]

where \[ \theta_w = \alpha \tan \left( \frac{12 - b_p}{2h} \right) = 0.602 = 34.509^\circ \]

The maximal ULS stress in the upper reinforcement in cracked behaviour (hogging moment) given by the global analysis is:

\[ \sigma_{\text{sup. re inf}} = -144.598 \text{ MPa} \]

The bending moment \( M_{c, Ed} \) applied to the composite box section (structural steel part + reinforcement) is:

\[ M_{c, Ed} = \frac{\sigma_{\text{sup. re inf}} I_{\text{tot}}}{h + t_{\text{slab}} - c_{ur} - z_{na}} = -321.654 \text{ MNm} \]

The bending moment \( M_a \) applied to the structural steel is:

\[ M_{a, Ed} = M_{Ed} - M_{c, Ed} = -739.778 \text{ MNm} - (-321.654 \text{ MNm}) = -418.124 \text{ MNm} \]

Thus, the bending moment \( M_{Ed} \) is the sum of the moment \( M_{a, Ed} = -418.124 \text{ MNm} \) applied to the box section (structural steel part only) as long as it behaves as a pure structural steel structure (before the concreting step of the slab segment which includes the studied box section) and of the bending moment \( M_{c, Ed} = -321.654 \text{ MNm} \) applied to the composite box section (structural steel part + reinforcement).

3.2.3.4 Mechanical properties of the gross cross-section

The mechanical properties of the composite box section (structural steel part and reinforcement) are:

- **Area**
  \[ A_{\text{tot}} := A_{\text{sur}} + A_{\text{slr}} + 2 \cdot \left( b_{\text{tf.1}} \cdot t_{\text{tf.1}} + b_{\text{tf.2}} \cdot t_{\text{tf.2}} \right) + 2 \cdot \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - t_p \right) \cdot t_{\text{w.h}} + A_{\text{st.w}} \]...
  \[ + n_{\text{st}} \left( b_{2} + 2 \cdot b_3 \right) + t_p \left( b_1 + b_{\text{sub}} \right) \]...
  \[ - \left( b_{\text{sub}} + 0.2m \right) \cdot t_p \]
  \[ A_{\text{tot}} = 1.532 \text{ m}^2 \]

- **First moment of area**
  \[ S_{\text{na}} := A_{\text{sur}} \left( h + t_{\text{slab}} - c_{ur} \right) + A_{\text{slr}} \left( h + c_{lr} \right) \]...
  \[ + 2 \cdot b_{\text{tf.1}} \cdot t_{\text{tf.1}} \left( h - \frac{t_{\text{tf.1}}}{2} \right) + b_{\text{tf.2}} \cdot t_{\text{tf.2}} \left( h - \frac{t_{\text{tf.1}}}{2} - \frac{t_{\text{tf.2}}}{2} \right) \]...
  \[ + 2 \cdot \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - t_p \right) \cdot t_{\text{w.h}} \cdot \frac{h - t_{\text{tf.1}} - t_{\text{tf.2}} - t_p}{2} + A_{\text{st.w}} \cdot \frac{h - t_{\text{tf.1}} - t_{\text{tf.2}} + t_p}{2} \]...
  \[ + n_{\text{st}} \left[ t_{\text{st}} \left( b_{2} + 2 \cdot b_3 \right) + t_p \left( b_1 + b_{\text{sub}} \right) \right] \]...
  \[ + \left( b_{\text{sub}} + 0.2m \right) \cdot t_p \cdot z_{\text{sl.1}} \]
  \[ S_{\text{na}} = 3.081 \cdot \text{m}^3 \]
Cross-section verifications - Box-girder bridge at the internal support P3

- Distance between the center of gravity and the lower face of the bottom flange
  \[ z_{na} := \frac{S_{na}}{A_{tot}} = 2.011 \text{ m} \]

- Second moment of area
  \[ I_{tot} := A_{sur} \left( h + t_{slab} - c_{ur} - z_{na} \right)^2 + A_{tslr} \left( h + c_{lr} - z_{na} \right)^2 \]
  \[ + 2 \left( \frac{b_{tf.1} \cdot t_{tf.1}}{12} \right) \left( h - \frac{t_{bf.1} \cdot t_{bf.1}}{2} - z_{na} \right)^2 \]
  \[ + 2 \left( \frac{b_{tf.2} \cdot t_{tf.2}}{12} \right) \left( h - t_{bf.1} - \frac{t_{bf.2} \cdot t_{bf.2}}{2} - z_{na} \right)^2 \]
  \[ + 2 \left( \frac{t_{w.h} \cdot \left( h - t_{tf.1} - t_{tf.2} - t_p \right)^3}{12} \right) + t_{w.h} \left( h - t_{bf.1} - t_{bf.2} - t_p \right) \left( h - t_{bf.1} - t_{bf.2} - t_p \right)^2 - z_{na} \right)^2 \]
  \[ + 2 \cdot A_{st.w} \left( h - t_{bf.1} - t_{bf.2} + t_p \right) - z_{na} \right)^2 \]
  \[ + \left[ n_{st} \left( b_2 + 2 \cdot b_3 \right) + t_p \left( b_1 + b_{sub} \right) \right] + \left( b_{sub} \cdot 0.2 \cdot t_p \right) \left( z_{na} - z_{sl.1} \right)^2 + n_{st} \cdot l_{sl.1} \]
  \[ I_{tot} = 5.014m^4 \]

3.2.3.5 Effective area of the bottom flange

3.2.3.5.1 General
In the following, the ultimate resistance of the longitudinal stiffened bottom plate is determined according to EN 1993-1-5, Section 3, Section 4 and Annex A.

3.2.3.5.2 Plate parameters

Geometry of the panel
- Number of stiffeners (equally spaced): \( n_{st} = 6 \ (\geq 3!) \)
- Length of panel: \( a_p = 4.0 \text{ m} \)
- Width of panel: \( b_p = 6.5 \text{ m} \)
- Thickness of panel: \( t_p = 75 \text{ mm} \)

Geometry of the trapezoidal stiffeners
- Distance between webs of stiffener: \( b_1 = 0.5 \text{ m} \)
- Width of stiffener flange: \( b_2 = 0.2 \text{ m} \)
- Height of stiffener: \( h_{st} = 0.4925 \text{ m} \)
- Thickness of stiffener: \( t_{st} = 15 \text{ mm} \)

Figure 3-12: Geometry of the trapezoidal stiffeners.
EN 1993-1-5, 4.5.1, General

(1) For plates with longitudinal stiffeners the effective\(^p\) areas from local buckling of the various subpanels between the stiffeners and the effective\(^p\) areas from the global buckling of the stiffened panel should be accounted for.

(2) The effective\(^p\) section area of each subpanel should be determined by a reduction factor in accordance with 4.4 to account for local plate buckling. The stiffened plate with effective\(^p\) section areas for the stiffeners should be checked for global plate buckling (by modelling it as an equivalent orthotropic plate) and a reduction factor \(\rho\) should be determined for overall plate buckling.

(3) The effective\(^p\) area of the compression zone of the stiffened plate should be taken as:

\[
A_{c,\text{eff,loc}} = \rho_c \cdot A_{c,\text{eff,loc}} + \sum b_{\text{edge,eff}} \cdot t
\]

where \(A_{c,\text{eff,loc}}\) is the effective\(^p\) section areas of all the stiffeners and subpanels that are fully or partially in the compression zone except the effective parts supported by an adjacent plate element with the width \(b_{\text{edge,eff}}\), see example in Figure 4.4.

(4) The area \(A_{c,\text{eff,loc}}\) should be obtained from:

\[
A_{c,\text{eff,loc}} = A_{s,\text{eff}} + \sum \rho_{\text{loc}} \cdot b_{c,\text{eff}} \cdot t
\]

where \(\sum\) applies to the part of the stiffened panel width that is in compression except the parts \(b_{\text{edge,eff}}\) see Figure 4.4;

\(A_{s,\text{eff}}\) is the sum of the effective\(^p\) sections according to 4.4 of all longitudinal stiffeners with gross area \(A_s\) located in the compression zone;

\(b_{c,\text{loc}}\) is the width of the compressed part of each subpanel;

\(\rho_{\text{loc}}\) is the reduction factor from 4.4(2) for each subpanel.

Figure 4.4: Stiffened plate under uniform compression

*NOTE: For non-uniform compression see Figure A.1.*
Resulting parameters

Width of each subpanel:
\[ b_{\text{sub}} := \frac{b_p - n_{\text{st}} b_1}{(n_{\text{st}} + 1)} = 0.5 \text{ m} \]

Width of each stiffener web:
\[ b_3 := \sqrt{\frac{2}{h_{\text{st}}} + \left(\frac{b_1 - b_2}{2}\right)^2} = 0.515 \text{ m} \]

Equivalent thickness of stiffener web:
\[ t_{\text{st.eq}} := b_3 \frac{h_{\text{st}}}{t_{\text{st}}} = 15.68 \text{ mm} \]

### 3.2.3.5.3 Effective\( ^p \) cross-section of subpanels and stiffeners

Stress distribution:
\[ \psi = 1 \]

Buckling factor for internal compression elements:
\[ k_\sigma = 4 \]

Slenderness of analysed plate:
\[ \lambda_{\text{local}}(b, t) := \frac{b}{t \cdot 28.4 \cdot \sqrt{k_\sigma}} \]

Reduction factor for internal compression elements:
\[ \rho_{\text{local}}(b, t) := \begin{cases} 1 & \text{if } \lambda_{\text{local}}(b, t) < 0.673 \\ \frac{\lambda_{\text{local}}(b, t) - 0.22}{\lambda_{\text{local}}(b, t)^2} & \text{otherwise} \end{cases} \]

Geometry of local panels and resulting effective\( ^p \) width due to local buckling:

<table>
<thead>
<tr>
<th>Panel</th>
<th>( b )</th>
<th>( t )</th>
<th>( \lambda_{\text{local}} )</th>
<th>( \rho_{\text{local}} )</th>
<th>( b_{\text{eff}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5 m</td>
<td>75 mm</td>
<td>0.138</td>
<td>1.000</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>0.2 m</td>
<td>15 mm</td>
<td>0.289</td>
<td>1.000</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>0.515 m</td>
<td>15 mm</td>
<td>0.743</td>
<td>0.948</td>
<td>0.488</td>
</tr>
<tr>
<td>sub</td>
<td>0.5 m</td>
<td>75 mm</td>
<td>0.138</td>
<td>1.000</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Effective local area (without edges):
\[ A_{\text{c.eff.loc}} := n_{\text{st}} \left[ t_{\text{st}} \left( b_{2,\text{eff}} + 2 \cdot b_{3,\text{eff}} \right) + t_p \left( b_{1,\text{eff}} + b_{\text{sub.eff}} \right) \right] = 0.556 \text{ m}^2 \]
EN 1993-1-5, Annex A, Calculation of critical stresses for stiffened plates

A.1 Equivalent orthotropic plate

(1) Plates with at least three longitudinal stiffeners may be treated as equivalent orthotropic plates.

(2) The elastic critical plate buckling stress of the equivalent orthotropic plate may be taken as:

\[ \sigma_{cr,p} = k_{\sigma,p} \cdot \sigma_E \]  \hspace{1cm} (A.1)

where

\[ \sigma_E = \frac{\pi^2 \cdot E \cdot t^2}{12(1-\nu^2) \cdot b^2} = 190000 \left( \frac{t}{b} \right)^2 \text{ in [MPa]} \]

- \( k_{\sigma,p} \) is the buckling coefficient according to orthotropic plate theory with the stiffeners smeared over the plate;
- \( b \) is defined in Figure A.1;
- \( t \) is the thickness of the plate.

NOTE1: The buckling coefficient \( k_{\sigma,p} \) is obtained either from appropriate charts for smeared stiffeners or relevant computer simulations; alternatively charts for discretely located stiffeners may be used provided local buckling in the subpanels can be ignored and treated separately.

NOTE2: \( \sigma_{cr,p} \) is the elastic critical plate buckling stress at the edge of the panel where the maximum compression stress occurs, see Figure A.1.

NOTE3: Where a web is of concern, the width \( b \) in equations (A.1) and (A.2) should be replaced by \( h_w \).

NOTE4: For stiffened plates with at least three equally spaced longitudinal stiffeners the plate buckling coefficient \( k_{\sigma,p} \) (global buckling of the stiffened panel) may be approximated by:

\[ k_{\sigma,p} = \begin{cases} 
2 \left( \frac{(1 + \alpha^2)^2 + \gamma - 1}{\alpha^2 (\psi + 1)(1 + \delta)} \right) & \text{if } \alpha \leq \sqrt[4]{\gamma} \\
\frac{4 \left( 1 + \gamma \right)}{(\psi + 1)(1 + \delta)} & \text{if } \alpha > \sqrt[4]{\gamma}
\end{cases} \]  \hspace{1cm} (A.2)

with

\[ \psi = \frac{\sigma_2}{\sigma_1} \geq 0.5 \quad ; \quad \gamma = \frac{I_w}{I_p} \quad ; \quad \delta = \frac{\Sigma A_{st}}{A_p} \quad ; \quad \alpha = \frac{a}{b} \geq 0.5 \]

where:
- \( I_w \) is the second moment of area of the whole stiffened plate;
- \( I_p \) is the second moment of area for bending of the plate \( = \frac{bt^3}{12(1-\nu^2)} = \frac{bt^3}{10.92} \);
- \( \Sigma A_{st} \) is the sum of the gross areas of the individual longitudinal stiffeners;
- \( A_p \) is the gross area of the plate \( = bt \);
- \( \sigma_1 \) is the larger edge stress;
- \( \sigma_2 \) is the smaller edge stress.
Gross area (without edges):

$$A_c := n_{st} \left[ t_{st} \left( b_2 + 2 \cdot b_3 \right) + t_p \left( b_1 + b_{sub} \right) \right] = 0.561 \, \text{m}^2$$

### 3.2.3.5.4 Effective\(^p\) cross-section of the whole bottom flange

**Determination of the elastic critical plate buckling stress (global buckling)**

Plate parameters:

$$z_{sl} := \frac{n_{st} \left[ \left( h_{st} + t_p \right) t_{st,eq} \cdot h_{st} + \left( h_{st} + \frac{t_p - t_{st}}{2} \right) t_{st} \cdot b_2 \right]}{A_c} = 63.674 \, \text{mm}$$

$$I_{sl} := n_{st} \left[ 2 \left( \frac{b_{st} \cdot t_{st,eq}^3}{12} + h_{st} \cdot t_{st,eq} \left( \frac{h_{st}}{2} - z_{sl} \right)^2 \right) + \frac{t_{st}^3 \cdot b_2}{12} + b_2 \cdot t_{st} \left( h_{st} - z_{sl} \right)^2 \right] + \frac{b_p \cdot t_p^2}{12} + b_p \cdot t_p \cdot z_{sl}^2$$

$$I_{p} := \frac{b_p \cdot t_p^3}{12 \left( 1 - \nu^2 \right)} = 2.511 \times 10^4 \, \text{cm}^4$$

$$A_{sl} := n_{st} \cdot t_{st} \left( b_2 + 2 \cdot b_3 \right) = 1.107 \times 10^5 \, \text{mm}^2$$

$$A_{p} := b_p \cdot t_p = 4.875 \times 10^5 \, \text{mm}^2$$

$$\gamma := \frac{I_{sl}}{I_{p}} = 41.724$$

$$\delta := \frac{A_{sl}}{A_{p}} = 0.227$$

$$\alpha := \frac{a_p}{b_p} = 0.615 = 0.5$$

$$k_{\sigma,p} := \begin{cases} \frac{2 \left[ \left( 1 + \alpha \right)^2 + \gamma - 1 \right]}{\alpha \cdot \left( \psi + 1 \right) \cdot (1 + \delta)} & \text{if } \alpha \leq \frac{4}{\sqrt{\gamma}} \\ \frac{4 \left( 1 + \sqrt{\gamma} \right)}{\left( \psi + 1 \right) \cdot (1 + \delta)} & \text{otherwise} \end{cases}$$

$$k_{\sigma,p} = 91.732$$

**Euler stress:**

$$\sigma_E := \frac{\pi^2 \cdot E \cdot t_p^2}{12 \left( 1 - \nu^2 \right) \cdot b_p^2} = 25.269 \, \text{N} \cdot \text{mm}^{-2}$$
EN 1993-1-5, 4.5.2, Plate type behaviour

(1) The relative plate slenderness $p_l$ of the equivalent plate is defined as:

$$\hat{\lambda}_p = \frac{\beta_{A,c} \cdot f_y}{\sigma_{cr,p}}$$

with $\beta_{A,c} = \frac{A_{c,eff,loc}}{A_c}$

(4.7)

where $A_c$ is the gross area of the compression zone of the stiffened plate except the parts of the subpanels supported by an adjacent plate, see Figure 4.4 (to be multiplied by the shear lag factor if shear lag is relevant, see 3.3);

$A_{c,eff,loc}$ is the effective area of the same part of the plate (including shear lag effect, if relevant) with due allowance made for possible plate buckling of subpanels and/or stiffeners.

(2) The reduction factor $\rho$ for the equivalent orthotropic plate is obtained from 4.4(2) provided $\lambda_p$ is calculated from equation (4.7).

NOTE: For calculation of $\sigma_{r,p}$ see Annex A.

EN 1993-1-5, 4.5.3, Column type buckling behaviour

(1) The elastic critical column buckling stress $\sigma_{cr,c}$ of an unstiffened (see 4.4) or stiffened (see 4.5) plate should be taken as the buckling stress with the supports along the longitudinal edges removed.

(2) For an unstiffened plate the elastic critical column buckling stress $\sigma_{cr,c}$ may be obtained from

$$\sigma_{cr,c} = \frac{\pi^2 \cdot E \cdot t^2}{12 \left(1 - \nu^2\right) \cdot a^2}$$

(4.8)

(3) For a stiffened plate $\sigma_{cr,c}$ may be determined from the elastic critical column buckling stress $\sigma_{cr,s}$ of the stiffener closest to the panel edge with the highest compressive stress as follows:

$$\sigma_{cr,c} = \frac{\pi^2 \cdot E \cdot I_{st,1}}{A_{st,1} \cdot a^2}$$

(4.9)

where $I_{st,1}$ is the second moment of area of the gross cross-section of the stiffener and the adjacent parts of the plate, relative to the out-of-plane bending of the plate;

$A_{st,1}$ is the gross cross-sectional area of the stiffener and the adjacent parts of the plate according to Figure A.1.

NOTE: $\sigma_{cr,c}$ may be obtained from $\sigma_{cr,c} = \sigma_{cr,s} \cdot b_s / b_{s,1}$, where $\sigma_{cr,c}$ is related to the compressed edge of the plate, and $b_{s,1}$ and $b_s$ are geometric values from the stress distribution used for the extrapolation, see Figure A.1.

(4) The relative column slenderness $\hat{\lambda}_c$ is defined as follows: [...]
Elastic critical plate buckling stress of equivalent orthotropic plate:

\[ \sigma_{cr,p} := k_{\sigma,p} \cdot \sigma_E = 2.318 \times 10^3 \cdot \text{N}\cdot\text{mm}^{-2} \]

**Plate type behaviour**

Reduction factor \( \beta_{A,c} \):

\[ \beta_{A,c} := \frac{A_{\text{eff.loc}}}{A_c} = 0.991 \]

Relative slenderness of the equivalent plate:

\[ \lambda_p := \sqrt{\frac{\beta_{A,c} \cdot t_p (t_p)}{\sigma_{cr,p}}} = 0.373 \]

Reduction factor for internal compression elements:

\[ \rho_p := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \lambda_p - 0.22 & \text{otherwise} \end{cases} \]

**Column type buckling behaviour**

Effective gross-section of stiffener:

\[ b_{1,sl} := (b_{\text{sub}} + b_1) = 1 \text{ m} \]

\[ A_{sl.1} := \frac{(b_2 + 2 \cdot b_3) + t_p \cdot b_{1,sl}}{t_{st}} = 9.345 \times 10^4 \cdot \text{mm}^2 \]

\[ z_{sl.1} := \frac{(h_{st} + t_p) \cdot t_{st,eq} \cdot h_{st} + \left( \frac{h_{st}}{2} + \frac{t_p - t_{st}}{2} \right) \cdot t_{st} \cdot b_2}{A_{sl.1}} = 63.674 \cdot \text{mm} \]

\[ I_{sl.1} := \frac{2}{12} \left[ \frac{h_{st}^3 \cdot t_{st,eq}}{12} + h_{st} \cdot t_{st,eq} \left( \frac{h_{st}}{2} - z_{sl.1} \right)^2 \right] + \frac{t_{st}^3 \cdot b_2}{12} + b_2 \cdot t_{st} \cdot \left( h_{st} - z_{sl.1} \right)^2 \]

\[ I_{sl.1} = 1.718 \times 10^9 \cdot \text{mm}^4 \]

Effective net cross-section of stiffener:

\[ b_{1,sl,eff} := (b_{\text{sub,eff}} + b_{1,eff}) = 1 \text{ m} \]

\[ A_{sl.1,eff} := t_{st} \left( b_{2,eff} + 2 \cdot b_{3,eff} \right) + t_p \cdot b_{1,sl,eff} = 9.264 \times 10^4 \cdot \text{mm}^2 \]

Elastic critical bucking stress for an equivalent column:

\[ \sigma_{cr,sl} := \frac{\pi^2 \cdot E \cdot I_{sl.1}}{A_{sl.1,eff} \cdot a_p} = 2.382 \times 10^3 \cdot \text{N}\cdot\text{mm}^{-2} \]
EN 1993-1-5, 4.5.3, Column type behaviour

(5) The reduction factor $\gamma_c$ should be obtained from 6.3.1.2 of EN 1993-1-1. For unstiffened plates $\alpha = 0.21$ corresponding to buckling curve a should be used. For stiffened plates its value should be increased to:

$$\alpha_v = \alpha + \frac{0.09}{\sqrt{i/e}}$$  \hspace{1cm} (4.12)

with

$$i = \sqrt{\frac{I_{sl,1}}{A_{sl,1}}}$$

$e = \max (e_1, e_2)$ is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the effective column, see Figure A.1; $\alpha = 0.34$ (curve b) for closed section stiffeners; $\alpha = 0.49$ (curve c) for open section stiffeners.

---

EN 1993-1-5, 4.5.4, Interaction between plate and column buckling

(1) The final reduction factor $\rho_c$ should be obtained by interpolation between $\gamma_c$ and $\rho$ as follows:

$$\rho_c = (\rho - \gamma_c) \xi (1 - \xi) + \gamma_c$$  \hspace{1cm} (4.13)

where

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1$$ \hspace{1cm} but \hspace{1cm} $0 \leq \xi \leq 1$

- $\sigma_{cr,p}$ is the elastic critical plate buckling stress, see Annex A.1(2);
- $\sigma_{cr,c}$ is the elastic critical column buckling stress according to 4.5.3(2) and (3), respectively;
- $\gamma_c$ is the reduction factor due to column buckling.
- $\rho$ is the reduction factor due to plate buckling, see 4.4(1).
Reduction factor $\beta_{\text{A.c.}}$:

$$\beta_{\text{A.c.}} := \frac{A_{\text{sl.1.eff}}}{A_{\text{sl.1}}} = 0.991$$

Relative column slenderness:

$$\lambda_c := \sqrt{\frac{\beta_{\text{A.c.}} \cdot t_p}{\sigma_{\text{cr.sl}}}} = 0.368$$

$$i := \sqrt{\frac{I_{\text{sl.1}}}{A_{\text{sl.1}}}} = 0.136 \text{ m}$$

$$e_1 := \frac{t_p}{2} + \frac{h_{\text{st}}}{2} \left( h_{\text{st}} - h_{\text{st} \cdot b_2} \right) - z_{\text{sl.1}} = 260.127 \cdot \text{mm}$$

$$e_2 := z_{\text{sl.1}} = 63.674 \cdot \text{mm}$$

$$e := \begin{cases} e_1 & \text{if } e_1 \geq e_2 \\ e_2 & \text{otherwise} \end{cases}$$

Imperfection factor $\alpha_e$:

$$\alpha_e := \alpha_0 + \frac{0.09}{e} = 0.513$$

Reduction factor for column buckling:

$$\chi_c(\phi) := \begin{cases} 1 & \text{if } \lambda_c < 0.2 \\ \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} & \text{otherwise} \end{cases}$$

$$\phi := 0.5 \left[ 1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2 \right] = 0.611$$

$$\chi_c := \chi_c(\phi) = 0.911$$

**Interaction between plate and column buckling**

Weighting factor $\xi$:

$$\xi := \frac{\sigma_{\text{cr.p}}}{\sigma_{\text{cr.sl}}} - 1$$

$$\xi = 0$$

Final reduction factor $\rho_c$:

$$\rho_c := \left( \rho_p - \chi_c \right) \xi \cdot (2 - \xi) + \chi_c = 0.911$$
Effective area of compression zone:

\[ A_{c\text{,eff}} := \rho_c \cdot A_{c\text{,eff loc}} + b_{\text{sub,eff}} \cdot t_p = 5.437 \times 10^5 \text{mm}^2 \]

### 3.2.3.5.5 Parameter study

Figure 3-13 summarises the results of the above performed design calculation for a variation of number of stiffeners \( n_{st} \) and thickness of bottom plate \( t_p \). From the diagram the following conclusions can be drawn:

1. With increasing bottom plate thickness \( t_p \), the critical buckling stress \( \sigma_p \) and thus the reduction factor \( U_c \) decreases. This is due to the fact that with increasing plate thickness and with constant stiffener-geometry the stiffening effect of the stiffeners decreases. → Decreasing of the continous lines (utilisation level \( \eta \)).

   **NOTE:** The parameter study has been performed with the hand-calculation formulae of EN 1993-1-5, Annex A. This effect can be minimized by using EBPlate with discrete stiffeners.

2. With increasing bottom plate thickness \( t_p \), the characteristic value of the yield strength \( f_y \) decreases. In conclusion the slenderness \( \lambda \) increases. → slight non-linearity of the behaviour described in 2. (continous lines are not straight).

3. With increasing plate thickness \( t_p \) the effective area \( A_{c\text{,eff}} \) and thus the maximum axial force in the bottom plate increases (broken lines).

4. Apart from case \([t_p = 35; \ n_{st} = 3]\) there is no reduction due to local buckling of the bottom plate.

5. From the \( A_{c\text{,eff}} \)-function it can be seen that by increasing the thickness \( t_p \) about 8 mm the number of stiffeners can be decreased down to four stiffeners. This would mean a benefit with regard to number of welds and amount of labour.

![Figure 3-13: Utilisation level (left ordinate) and effective area (right ordinate) of bottom plate in function of bottom plate thickness \( t_p \); curve parameter = number of stiffeners \( n_{st} \).](image-url)
EN 1993-1-5, 3.1, General

(1) Shear lag in flanges may be neglected if \( b_0 < \frac{L_e}{50} \) where \( b_0 \) is taken as the flange outstand or half the width of an internal element and \( L_e \) is the length between points of zero bending moment, see 3.2.1(2).

EN 1993-1-5, 3.2.1, Effective width

(1) The effective width \( b_{\text{eff}} \) for shear lag under elastic conditions should be determined from:

\[
b_{\text{eff}} = \beta b_0
\]

where the effective factor \( \beta \) is given in Table 3.1.

This effective width may be relevant for serviceability and fatigue limit states.

(2) Provided adjacent spans do not differ more than 50% and any cantilever span is not larger than half the adjacent span the effective lengths \( L_e \) may be determined from Figure 3.1. For all other cases \( L_e \) should betaken as the distance between adjacent points of zero bending moment.

Figure 3.1: Effective length \( L_e \) for continuous beam and distribution of effective width

Table 3.1: Effective width factor \( \beta \)

<table>
<thead>
<tr>
<th>( \kappa )</th>
<th>Verification</th>
<th>( \beta ) value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \kappa \leq 0.02 )</td>
<td>sagging bending</td>
<td>( \beta = \beta_1 = \frac{1}{1 + 6.4 \kappa^3} )</td>
</tr>
<tr>
<td>( 0.02 &lt; \kappa \leq 0.70 )</td>
<td>hogging bending</td>
<td>( \beta = \beta_1 = \frac{1}{1 + 6.0 \kappa^3} )</td>
</tr>
<tr>
<td>( \kappa &gt; 0.70 )</td>
<td>sagging bending</td>
<td>( \beta = \beta_2 = \frac{1}{5.9 \kappa} )</td>
</tr>
<tr>
<td></td>
<td>hogging bending</td>
<td>( \beta = \beta_2 = \frac{1}{8.6 \kappa} )</td>
</tr>
<tr>
<td>all ( \kappa )</td>
<td>end support</td>
<td>( \beta_1 = (0.65 + 0.025 \kappa) \beta_1 ) but ( \beta_2 &lt; \beta_1 )</td>
</tr>
<tr>
<td>all ( \kappa )</td>
<td>Cantilever</td>
<td>( \beta = \beta_2 ) at support and at the end</td>
</tr>
</tbody>
</table>

\( \kappa = \frac{a_{b0}}{L_e} \) with \( a_{b0} = \sqrt{1 + \frac{A_{c0}}{b_0}} \) in which \( A_{c0} \) is the area of all longitudinal stiffeners within the width \( b_0 \) and other symbols are as defined in Figure 3.1 and Figure 3.2.

EN 1993-1-5, 3.3, Shear lag at the ultimate limit state

NOTE3: Elastic-plastic shear lag effects allowing for limited plastic strains may be taken into account using \( A_{\text{eff}} \) as follows

\[
A_{\text{eff}} = A_{\text{eff}} \beta^\kappa \geq A_{\text{eff}} \beta
\]

where \( \beta \) and \( \kappa \) are taken from Table 3.1.
3.2.3.5.6 Reduction due to shear lag effect

Verification if shear lag effect has to be taken into account:

- Bridge span: \( L_1 = 120 \text{ m} \) and \( L_2 = 120 \text{ m} \)
- Effective length: \( L_e = 0.25 (L_1 + L_2) \cdot 120 \text{ m} = 60 \text{ m} \)
- Considered width: \( b_0 = b_p/2 = 3.25 \text{ m} \)

\[ b_0 < L_e/50 \text{ requirement not fulfilled! Shear lag effect has to be taken into account.} \]

Shear lag parameters:

\[ \alpha^0 := \sqrt{1 + \frac{A_{sl}}{2 \frac{b_0}{t_p}}} = 1.108 \]

\[ \kappa := \frac{b_0}{L_e} = 0.06 \]

\[ \beta_{ult} := \begin{cases} 1 & \text{if } \kappa \leq 0.02 \\ \frac{1}{1 + 6 \left( \kappa - \frac{1}{2500 \cdot \kappa} \right) + 1.6 \cdot \kappa^2} & \text{if } 0.02 < \kappa \leq 0.7 \\ \frac{1}{8.6 \cdot \kappa} & \text{otherwise} \end{cases} \]

\[ \beta_{ult} = 0.754 \]

3.2.3.5.7 Effective area of the stiffened plate

Effective area of compression zone taking into account effects of plate buckling and shear lag:

\[ A_{effEP} := A_{c.eff} \cdot b_{ult}^k = 0.535 \text{ m}^2 \]

3.2.3.5.8 New mechanical properties of the cross-section

The new mechanical properties of the cross-section are then calculated by replacing the gross area of the bottom flange by its effective area.

The web has been stiffened by a longitudinal closed stiffener located at mid-depth due to shear verifications. For simplification reasons, this stiffener is not considered in the bending verification.

New mechanical properties of the steel part of the box-section

The new mechanical properties of the steel part (structural steel only) of the box section are:

- Area:

\[ A_{tot.a.eff} := A_{effEP} + 2 \left( h - t_{tf.1} - t_{tf.2} - t_p \right) t_{w,h} + 2 \left( b_{tf.1} t_{tf.1} + b_{tf.2} t_{tf.2} \right) \]

\[ A_{tot.a.eff} = 1.331 \text{ m}^2 \]
• First moment of area:

\[
S_{\text{a,na}} := 2 \left[ b_{\text{tf,1}} \cdot t_{\text{tf,1}} \left( h - \frac{t_{\text{tf,1}}}{2} \right) + b_{\text{tf,2}} \cdot t_{\text{tf,2}} \left( h - \frac{t_{\text{tf,1}} - t_{\text{tf,2}}}{2} \right) \right] + 2 \cdot t_{\text{w,h}} \left( h - t_{\text{tf,1}} - t_{\text{tf,2}} - t_{p} \right) \left( \frac{h - t_{\text{tf,1}} - t_{\text{tf,2}} + t_{p}}{2} \right) + A_{\text{eff,EP}} z_{\text{sl,1}}
\]

\[
S_{\text{a,na}} = 2.666 \cdot m^{3}
\]

• Distance between the center of gravity and the lower face of the bottom flange:

\[
z_{\text{tot,a,na}} := \frac{S_{\text{a,na}}}{A_{\text{tot,a,eff}}} = 2.003 \text{ m}
\]

• Second moment of area:

\[
I_{\text{tot,a,eff}} := 2 \left[ \frac{b_{\text{tf,1}} \cdot t_{\text{tf,1}}^{3}}{12} + \left( b_{\text{tf,1}} \cdot t_{\text{tf,1}} \right) \left( h - \frac{t_{\text{tf,1}}}{2} - z_{\text{tot,a,na}} \right)^{2} \right] + 2 \left[ \frac{b_{\text{tf,2}} \cdot t_{\text{tf,2}}^{3}}{12} + \left( b_{\text{tf,2}} \cdot t_{\text{tf,2}} \right) \left( h - t_{\text{tf,1}} - \frac{t_{\text{tf,2}}}{2} - z_{\text{tot,a,na}} \right)^{2} \right] + 2 \left[ t_{\text{w,h}} \left( h - t_{\text{tf,1}} - t_{\text{tf,2}} - t_{p} \right) \left( h - t_{\text{tf,1}} - t_{\text{tf,2}} + t_{p} \right) - z_{\text{tot,a,na}} \right]^{2}
\]

\[
I_{\text{tot,a,eff}} = 4.308 \text{ m}^{4}
\]

**New mechanical properties of the composite box-section**

The mechanical properties of the composite box section (structural steel part and reinforcement) are:

• Area:

\[
A_{\text{tot,eff}} := A_{\text{tot,a,eff}} + A_{\text{sur}} + A_{\text{tslr}} = 1.416 \text{ m}^{2}
\]

• First moment of area:

\[
S_{\text{na}} := S_{\text{a,na}} + A_{\text{sur}} \cdot \left( h + t_{\text{slab}} - c_{ur} \right) + A_{\text{tslr}} \cdot \left( h + c_{lr} \right) = 3.023 \cdot m^{3}
\]

• Distance between the center of gravity and the lower face of the bottom flange:

\[
z_{\text{tot,na}} := \frac{S_{\text{na}}}{A_{\text{tot,eff}}} = 2.134 \text{ m}
\]
EN 1993-1-1, Table 5.2 (sheet 1 of 3), Maximum width-to-thickness ratios for compression parts

<table>
<thead>
<tr>
<th>Class</th>
<th>Part subject to bending (compression positive)</th>
<th>Part subject to compression</th>
<th>Part subject to bending and compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( c/t \leq 72 \varepsilon )</td>
<td>( c/t \leq 33 \varepsilon )</td>
<td>when ( \alpha &gt; 0.5 ): ( c/t \leq \frac{396 \varepsilon}{13 \alpha - 1} ) ( \text{when } \alpha \leq 0.5 ): ( c/t \leq \frac{36 \varepsilon}{\alpha} )</td>
</tr>
<tr>
<td>2</td>
<td>( c/t \leq 83 \varepsilon )</td>
<td>( c/t \leq 38 \varepsilon )</td>
<td>when ( \alpha &gt; 0.5 ): ( c/t \leq \frac{456 \varepsilon}{13 \alpha - 1} ) ( \text{when } \alpha \leq 0.5 ): ( c/t \leq \frac{41.5 \varepsilon}{\alpha} )</td>
</tr>
<tr>
<td>3</td>
<td>( c/t \leq 124 \varepsilon )</td>
<td>( c/t \leq 42 \varepsilon )</td>
<td>when ( \psi &gt; -1 ): ( c/t \leq \frac{42 \varepsilon}{0.67 + 0.33 \psi} ) ( \text{when } \psi \leq -1^{(*)} ): ( c/t \leq 62 \varepsilon (1 - \psi) \sqrt{(-\psi)} )</td>
</tr>
</tbody>
</table>

\[ \varepsilon = \sqrt{\frac{235}{f_y}} \]

<table>
<thead>
<tr>
<th></th>
<th>( f_y )</th>
<th>235</th>
<th>275</th>
<th>355</th>
<th>420</th>
<th>460</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \varepsilon )</td>
<td>1.00</td>
<td>0.92</td>
<td>0.81</td>
<td>0.75</td>
<td>0.71</td>
<td></td>
</tr>
</tbody>
</table>

\(^{(*)}\) \( \psi \leq -1 \) applies where either the compression stress \( \sigma \leq f_y \) or the tensile strain \( \varepsilon > f_y / E \)
### Cross-section verifications - Box-girder bridge at the internal support P3

**Second moment of area:**

\[
I_{\text{tot.eff}} := A_{\text{surl}} \left( h + t_{\text{slab}} - c_{ur} - z_{\text{tot.na}} \right)^2 + A_{\text{tslr}} \left( h + c_{lr} - z_{\text{tot.na}} \right)^2 + 2 \left( b_{\text{tf.1}} \cdot t_{\text{tf.1}} \cdot k_{1} \right) \left( h - t_{\text{tf.1}} / 2 - z_{\text{tot.na}} \right)^2 \\
+ 2 \left( b_{\text{tf.2}} \cdot t_{\text{tf.2}} \cdot k_{1} \right) \left( h - t_{\text{tf.2}} / 2 - z_{\text{tot.na}} \right)^2 \\
+ 2 \left( t_{w,h} \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - t_{p} \right) \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} + t_{p} / 2 - z_{\text{tot.na}} \right)^2 \\
+ \frac{t_{w,h} \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - t_{p} \right)^3}{12} \right) \\
+ A_{\text{effEP}} \left( z_{\text{tot.na}} - z_{\text{sl.1}} \right)^2 + n_{\text{st}} \cdot I_{\text{sl.1}}
\]

\[I_{\text{tot.eff}} = 4.69 \text{ m}^4\]

### 3.2.3.6 Effective area of the web

#### 3.2.3.6.1 General

From the values of the bending moments \( M_a \) and \( M_c \) (see Paragraph 3.2.3.3) and of the mechanical properties in Paragraph 3.2.3.5, the normal extreme stresses in the web at ULS are as follows:

\[
\sigma_{\text{abfu}} = \frac{-M_a \left( z_{\text{tot.na}} - t_{p} \right)}{I_{\text{tot.a.eff}}} + \frac{-M_c \left( z_{\text{tot.na}} - t_{p} \right)}{I_{\text{tot.eff}}} = 328.263 \text{ MPa}
\]

\[
\sigma_{\text{aflf}} = \frac{M_a \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - z_{\text{tot.na}} \right)}{I_{\text{tot.a.eff}}} + \frac{M_c \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - z_{\text{tot.na}} \right)}{I_{\text{tot.eff}}} = -290.373 \text{ MPa}
\]

#### 3.2.3.6.2 Determination of the cross-section class

The bottom flange is already a class 4 element due to the webs of the stiffeners, so that the whole section is already classified (elastic analysis should be performed). Thus, it has to be only determined if the web is a Class 3 or Class 4 panel in order to eventually reduce it.

Reasoning is based on the elastic stress distribution at ULS given by Paragraph 3.2.3.6 which takes into account the effects of plate buckling and shear lag:

\[
\sigma_{\text{abfu}} = 328.263 \text{ N/mm}^2
\]

\[
\sigma_{\text{aflf}} = -290.373 \text{ N/mm}^2
\]

And the elastic stress distribution at ULS:

\[
\psi_w = \frac{\sigma_{\text{aflf}}}{\sigma_{\text{abfu}}} = -0.885 > -1
\]

Thus, the limiting slenderness between Class 3 and Class 4 is given by:

\[
\frac{c_w}{I_w} = \frac{h_w}{t_w} = 167.872 > \frac{42e}{0.67 + 0.33 \cdot \psi_w} = 91.681
\]

It is deduced that the steel web is in Class 4.
EN 1993-1-5, 4.4 Table 4.1, Internal compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective width $b_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Stress distribution diagram" /></td>
<td>$\psi = 1$: $b_{eff} = \rho \vec{b}$</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = 0.5 b_{eff}$ $b_{eff} = 0.5 b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$1 &gt; \psi &gt; 0$: $b_{eff} = \rho \vec{b}$</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = \frac{2}{5 - \psi} b_{eff}$ $b_{eff} = b_{eff} - b_{eff} \cdot b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$\psi &lt; 0$: $b_{eff} = \rho b_{c} = \rho \vec{b} / (1 - \psi)$</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = 0.4 b_{eff}$ $b_{eff} = 0.6 b_{eff}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi = \sigma_{2}/\sigma_{1}$</th>
<th>1</th>
<th>$1 &gt; \psi &gt; 0$</th>
<th>0</th>
<th>0</th>
<th>$0 &gt; \psi &gt; -1$</th>
<th>-1</th>
<th>-1</th>
<th>$\psi &gt; -3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckling factor $k_{b}$</td>
<td>4.0</td>
<td>8.2</td>
<td>$(1.05 + \psi)$</td>
<td>7.81</td>
<td>7.81 - 6.29$\psi + 9.78\psi^{2}$</td>
<td>23.9</td>
<td>5.98 $(1 - \psi)^{2}$</td>
<td></td>
</tr>
</tbody>
</table>

EN 1993-1-5, 4.4 Table 4.2, Outstand compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective width $b_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Stress distribution diagram" /></td>
<td>$1 &gt; \psi &gt; 0$: $b_{eff} = \rho \vec{c}$</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = \rho b_{c}$</td>
</tr>
<tr>
<td></td>
<td>$\psi &lt; 0$: $b_{eff} = \rho b_{c} = \rho \vec{c} / (1 - \psi)$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi = \sigma_{2}/\sigma_{1}$</th>
<th>1</th>
<th>0</th>
<th>-1</th>
<th>$1 &gt; \psi &gt; -3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckling factor $k_{b}$</td>
<td>0.43</td>
<td>0.57</td>
<td>0.85</td>
<td>0.57 - 0.21$\psi + 0.07\psi^{2}$</td>
</tr>
</tbody>
</table>

EN 1993-1-5, 4.4(3)

(3) For flange elements of I-sections and box-girders the stress ratio $\phi$ used in Table 4.1 and Table 4.2 should be based on the properties of the gross cross-sectional area, due allowance being made for shear lag in the flanges if relevant. For web elements the stress ratio $\psi$ used in Table 4.1 should be obtained using a stress distribution based on the effective area of the compression flange and the gross area of the web.
Conclusion: the cross-section at the internal support P1, P2 and P3 is in Class 4 and is checked by an elastic section analysis.

The web is in Class 4, so that its effective cross-section under bending moment has to be calculated according to EN 1993-1-5, 4.4.

**Web in bending**

\[ k_{ow} = 7.81 - 6.29 \cdot \psi + 9.78 \cdot \psi^2 = 21.027 \]

(see EN1993-1-5, 4.4, Table 4.1, Internal compression elements)

\[
\lambda_{pw} = \frac{B}{t} \cdot \frac{h_w}{28.4 \cdot \sqrt[4]{k_{\sigma}}} = \frac{t_w}{28.4 \cdot \sqrt[4]{k_{\sigma}}} = 1.562 > 0.673
\]

\[ \Rightarrow \rho_w = \frac{\lambda_{pw} - 0.055(3 + \psi_w)}{\lambda_{pw}^2} = 0.593; \text{ There is a reduction of the height of the steel web} \]

Thus, the effective height of the web in compression can be calculated:

\[ h_{weff} = \frac{\rho_w h_w}{(1 - \psi_w)} = 1.425 \text{ m} \]

And this effective height of the web can be distributed as shown in EN1993-1-5, Table 4.1:

\[ h_{we1} = \left( \frac{-\psi_w}{1 - \psi_w} \right) \cdot h_w + 0.6 h_{weff} = 2.127 + 0.855 = 2.983 \text{ m} \]

\[ h_{we2} = 0.4 h_{weff} = 0.57 \text{ m} \]

**NOTE:** The effective area of the box section webs is determined after that of its stiffened bottom flange. The reverse calculation would not lead to the same effective area of the cross-section at P3 and it would not comply with EN 1993-1-5.

**3.2.3.6.3 Effective mechanical properties of the box section**

The final effective mechanical properties of the cross-section are calculated by replacing the gross area of the bottom flange and web by their effective areas.

**Final mechanical properties of the steel part of the box-section**

The final mechanical properties of the effective steel box section (structural steel only) of the box section are:

- **Area:**
  \[
  A_{\text{eff},w} := (h_{w,e1} + h_{w,e2}) \cdot t_w = 0.096 \text{ m}^2
  \]
  \[
  A_{\text{tot},a,\text{eff}} := A_{\text{eff},EP} + 2 \cdot A_{\text{eff},w} + 2(b_{tf,1} \cdot t_{tf,1} + b_{tf,2} \cdot t_{tf,2}) = 1.278 \text{ m}^2
  \]

- **First moment of area:**
  \[
  S_{\text{a,na}} := 2 \left[ b_{tf,1} \cdot t_{tf,1} \left( h - t_{tf,1} - \frac{t_{tf,2}}{2} \right) + b_{tf,2} \cdot t_{tf,2} \left( h - t_{tf,1} - \frac{t_{tf,2}}{2} \right) \right] + A_{\text{eff},EP} \cdot z_{sl,1} \]
  \[
  + 2 \cdot t_w \cdot h_{w,e1} \left( h - t_{tf,1} - t_{tf,2} - \frac{h_{w,e1} \cdot \cos(q_w)}{2} \right) + 2 \cdot t_w \cdot h_{w,e2} \cdot \frac{h_{w,e2} \cdot \cos(q_w)}{2}
  \]
  \[
  S_{\text{a,na}} = 2.613 \cdot \text{m}^3
  \]
Cross-section verifications - Box-girder bridge at the internal support P3

- Distance between the center of gravity and the lower face of the bottom flange:

\[
z_{\text{tot.na}} := \frac{S_{a,na}}{A_{\text{tot.eff}}} = 2.044 \text{ m}
\]

- Second moment of area:

\[
I_{\text{tot.eff}} := 2 \left[ \frac{b_{\text{tf.1}} \cdot t_{\text{tf.1}}^3}{12} + \left( b_{\text{tf.1}} \cdot t_{\text{tf.1}} \right) \left( h - \frac{t_{\text{tf.1}}}{2} - z_{\text{tot.na}} \right)^2 \right] + 2 \left[ \frac{b_{\text{tf.2}} \cdot t_{\text{tf.2}}^3}{12} + \left( b_{\text{tf.2}} \cdot t_{\text{tf.2}} \right) \left( h - t_{\text{tf.2}} - z_{\text{tot.na}} \right)^2 \right] + 2 \left[ \frac{t_{w,h} \cdot h_{w,e1} \cdot \cos(q_w)}{12} \right]^3 + \frac{t_{w,h} \cdot h_{w,e1} \cdot \cos(q_w)}{2} \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - \frac{h_{w,e1} \cdot \cos(q_w)}{2} - z_{\text{tot.na}} \right)^2 \right] + \frac{A_{\text{effEP}} \left( z_{\text{tot.na}} - z_{\text{sl.1}} \right)^2 + n_{st} \cdot I_{\text{sl.1}}}{\text{}}
\]

\[
I_{\text{tot.eff}} = 4.244 \text{ m}^4
\]

Final mechanical properties of the composite box-section

The final mechanical properties of the composite box section (structural steel part and reinforcement) are:

- Area:

\[
A_{\text{tot.eff}} := A_{\text{tot.eff}} + A_{\text{sur}} + A_{\text{slr}} = 1.364 \text{ m}^2
\]

- First moment of area:

\[
S_{\text{na}} := S_{a,na} + A_{\text{sur}} \cdot (h + t_{\text{slab}} - c_{ur}) + A_{\text{slr}} \cdot (h + c_{lr}) = 2.97 \cdot \text{m}^3
\]

- Distance between the center of gravity and the lower face of the bottom flange:

\[
z_{\text{tot.na}} := \frac{S_{\text{na}}}{A_{\text{tot.eff}}} = 2.178 \text{ m}
\]

- Second moment of area:

\[
I_{\text{tot.eff}} := A_{\text{sur}} \left( h + t_{\text{slab}} - c_{ur} - z_{\text{tot.na}} \right)^2 + A_{\text{slr}} \left( h + c_{lr} - z_{\text{tot.na}} \right)^2 \right] + \frac{b_{\text{tf.1}} \cdot t_{\text{tf.1}}^3}{12} + \left( b_{\text{tf.1}} \cdot t_{\text{tf.1}} \right) \left( h - \frac{t_{\text{tf.1}}}{2} - z_{\text{tot.na}} \right)^2 \right] + 2 \left[ \frac{b_{\text{tf.2}} \cdot t_{\text{tf.2}}^3}{12} + \left( b_{\text{tf.2}} \cdot t_{\text{tf.2}} \right) \left( h - t_{\text{tf.2}} - z_{\text{tot.na}} \right)^2 \right] + 2 \left[ \frac{t_{w,h} \cdot h_{w,e1} \cdot \cos(q_w)}{12} \right]^3 + \frac{t_{w,h} \cdot h_{w,e1} \cdot \cos(q_w)}{2} \left( h - t_{\text{tf.1}} - t_{\text{tf.2}} - \frac{h_{w,e1} \cdot \cos(q_w)}{2} - z_{\text{tot.na}} \right)^2 \right] + \frac{A_{\text{effEP}} \left( z_{\text{tot.na}} - z_{\text{sl.1}} \right)^2 + n_{st} \cdot I_{\text{sl.1}}}{\text{}}
\]

\[
I_{\text{tot.eff}} = 4.61 \text{ m}^4
\]
EN 1993-1-5, 4.6(1)

Member verification for uniaxial bending should be performed as follows:

\[
\eta = \frac{N_{Ed}}{f_y A_{eff}} + \frac{M_{Ed} + N_{Ed} e_N}{f_y W_{eff}} \leq 1.0 \quad (4.14)
\]

Where

- \( A_{eff} \) is the effective cross-section area in accordance with 4.3(3);
- \( e_N \) is the shift in the position of neutral axis, see 4.3(3);
- \( M_{Ed} \) is the design bending moment;
- \( N_{Ed} \) is the design bending moment;
- \( W_{eff} \) is the effective elastic section modulus, see 4.3(4);
- \( \gamma_{M0} \) is the partial safety factor, see application parts EN 1993-2 to 6.

**NOTE:** For members subject to compression and biaxial bending the above equation 4.14 may be modified as follows:

\[
\eta = \frac{N_{Ed}}{f_y A_{eff}} + \frac{M_{y,Ed} + N_{Ed} e_{y,N}}{f_y W_{y,eff}} + \frac{M_{z,Ed} + N_{Ed} e_{z,N}}{f_y W_{z,eff}} \leq 1.0 \quad (4.15)
\]

\( M_{y,Ed}, M_{z,Ed} \) are the design bending moments with respect to \( y \) and \( z \) axes respectively; \( e_{y,N}, e_{z,N} \) are the eccentricities with respect to the neutral axis.

---

**Further explanations on the shear resistance check**

See Paragraph 3.1.2.5.2, page 89ff.
### 3.2.3.7 Bending resistance verification

From the values of the bending moments $M_a$ and $M_c$ (see Paragraph 3.2.3.3) and of the mechanical properties in Paragraph 3.2.3.6, the normal extreme stress at ULS are:

\[
\sigma_{abfl} = -\frac{M_a (z_{tot, a, mu})}{I_{tot, a, eff}} - \frac{M_c (z_{tot, c, mu})}{I_{tot, c, eff}} = 353.374 \text{ MPa}
\]

\[
\sigma_{eff, 21} = -\frac{M_a (h - t_{gf, 1} - t_{gf, 2} - z_{tot, a, mu})}{I_{tot, a, eff}} - \frac{M_c (h - t_{gf, 1} - t_{gf, 2} - z_{tot, c, mu})}{I_{tot, c, eff}} = -287.808 \text{ MPa}
\]

\[
\sigma_{eff, 21} = \frac{M_a (h - t_{gf, 1} - z_{tot, a, mu})}{I_{tot, a, eff}} - \frac{M_c (h - t_{gf, 1} - z_{tot, c, mu})}{I_{tot, c, eff}} = -302.954 \text{ MPa}
\]

\[
\sigma_{eff, 31} = -\frac{M_a (h - z_{tot, a, mu})}{I_{tot, a, eff}} - \frac{M_c (h - z_{tot, c, mu})}{I_{tot, c, eff}} = -319.783 \text{ MPa}
\]

\[
\sigma_{s, reinf} = -\frac{M_s (h + t_{slab} - c_{surf} - z_{tot, mu})}{I_{tot, eff}} = -145.599 \text{ MPa}
\]

It is then clearly verified that:

\[
\sigma_{abfl} \leq \frac{f_{yd}}{\gamma_{M0}} = 325 \text{ MPa} \quad \Rightarrow \eta_{1, abfl} = 1.087 > 1.0
\]

\[
\sigma_{eff, 21} \geq \frac{f_{yd}}{\gamma_{M0}} = 315 \text{ MPa} \quad \Rightarrow \eta_{1, eff} = 0.914 < 1.0
\]

\[
\sigma_{eff, 31} \geq \frac{f_{yd}}{\gamma_{M0}} = 315 \text{ MPa} \quad \Rightarrow \eta_{1, eff} = 0.962 < 1.0
\]

\[
\sigma_{s, reinf} \leq \frac{f_s}{\gamma_{s}} = 434.783 \text{ MPa} \quad \Rightarrow \eta_{1, s, reinf} = 0.335 < 1.0
\]

The effective box section has been checked here with the calculated bending moment in the cross-section at support P3. The stress in the lower flange and in the upper flange is too high ($\eta_1 > 1.0$). The calculation should normally be carried out with a lower value calculated in the cross-section located at the distance $\min [0.4 \cdot a; 0.5 \cdot h_w]$ from the support P3. Moreover, the stresses can be checked at mid-depth of the flanges. By doing this, the stresses should decrease under the limit value ($\eta_1 = 1.0$).

**NOTE:** The bending moment resistance does not take into account in the calculation the presence of the web stiffener.

### 3.2.3.8 Shear resistance verification

#### 3.2.3.8.1 Shear in the box section webs

The box section web is transversally stiffened on both sides at the internal support P3 ($a_w = 2.5$ m).

**Stiffened webpanel**

To evaluate the shear buckling coefficient of the stiffened webpanel, the second moment of area of the longitudinal stiffener must be calculated according to EN 1993-1-5, Figure 5.3:
EN 1993-1-5, 4.6(3)

The plate buckling verification of the panel should be carried out for the stress resultants at a distance 0.4·a or 0.5·b, whichever is the smallest, from the panel end where the stresses are the greater. In this case the gross sectional resistance needs to be checked at the end of the panel.

EN 1993-1-5, Annex A3, Shear buckling coefficients

(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient $k$, can be obtained as follows:

$$
k = 5.34 + 4 \left( \frac{h_w}{a} \right)^2 + k_{st} \text{ when } a/h_w \geq 1
$$

$$
\text{or } k = 4 + 5.34 \left( \frac{h_w}{a} \right)^2 + k_{st} \text{ when } a/h_w < 1
$$

where $k_{st} = 9 \left( \frac{h_w}{a} \right)^4 \left( \frac{I_{sl}}{I_{sl} t^2 h_w} \right)^3$ but not less than $k_{st} = \frac{2.1}{t} \sqrt{\frac{I_{sl}}{h_w}}$

$a$ is the distance between transverse stiffeners (see Figure 5.3);

$I_{sl}$ is the second moment of area of the longitudinal stiffener about the $z$-axis, see Figure 5.3 (b).

For webs with two or more longitudinal stiffeners, not necessarily equally spaced, $I_{sl}$ is the sum of the stiffness of the individual stiffeners.

NOTE: No intermediate non-rigid transverse stiffeners are allowed for in equation (A.5)

![Figure 5.3: Web with transverse and longitudinal stiffeners](image)

(2) The equation (A.5) also applies to plates with one or two longitudinal stiffeners, if the aspect ratio $\alpha = \frac{a}{h_w}$ satisfies $\alpha \geq 3$. For plates with one or two longitudinal stiffeners and an aspect ratio $\alpha < 3$ the shear buckling coefficient should be taken from:

$$
k = 4.1 + \frac{6.3 + 0.18 \frac{I_{sl}}{r^2 h_w}}{\alpha^2} + 2.2 \sqrt{\frac{I_{sl}}{r^2 h_w}}
$$

(A.6)
The elastic neutral axis of the web stiffener with the width \( 15\varepsilon(t_u)\) on both sides of the stiffener is:

\[
z_{zz,w} = \frac{2h_{z,z,w}b_{z,z,w}}{2h_{z,z,w} + b_{z,z,w} + b_{z,z,w} + (2 \cdot 15\varepsilon(t_u) + 1) t_w} = 0.111 \text{ m}
\]

The second moment of area of the web stiffener is:

\[
I_{zz,w} = I_{zz,w} = b_{z,z,w} t_w (h_{z,z,w} - z_{zz,w})^2 + 2 \left[ \frac{t_w h_{z,z,w}}{12} + t_w h_{z,z,w} \left( \frac{h_{z,z,w}}{2} - z_{zz,w} \right)^2 \right] + (2 \cdot 15\varepsilon(t_u) + 1) t_w z_{zz,w}^2
\]

\[
= 1.215 \times 10^{-3} \text{ m}^4
\]

According to EN1993-1-5, Annex A3(2), as there is only one stiffener in the web and the aspect ratio is \( \frac{a_w}{h_w} = 0.552 \leq 3 \), the shear buckling coefficient is:

\[
k_{w,z} = 4.1 + \frac{6.3 + 0.18 I_{zz,w}}{t_w h_w} + 2.2 \sqrt{I_{zz,w}} = 38.119
\]

The transverse stiffeners of the bracings frames bordering the webpanel close to support P3 are assumed to be rigid.

\[
\frac{h_w}{t_w} = \frac{4533}{27} = 167.872 > \frac{31}{\eta} \varepsilon(t_u) \sqrt{k_{w,z}} = 131.636
\]

Thus, the stiffened webpanel must be checked against shear buckling.

The reduced slenderness of the stiffened webpanel:

\[
\bar{\lambda}_w = \frac{h_w}{37.4 \varepsilon(t_u) \sqrt{k_{w,z}}} = 0.881
\]

**Web subpanels**

It is possible that one or both of the two web subpanels are more critical than the stiffened webpanel. Thus, the two web subpanels must be also checked. As the longitudinal stiffeners is located at the middle of the height of the web, the two webs subpanels have the same width and the same reduced slenderness.

According to EN 1993-1-5, Annex A3(1), as the aspect ratio is \( \frac{a_w}{b_{w,sp}} = 2.5 = 1.24 \geq 1 \), the shear buckling coefficient is:

\[
k_{w,sp,z} = 5.34 + 4 \left( \frac{b_{w,sp}}{a_w} \right)^2 = 7.942
\]

\[
\frac{b_{w,sp}}{t_w} = \frac{2016}{27} = 74.677 > \frac{31}{\eta} \varepsilon(t_u) \sqrt{k_{w,z}} = 60.085
\]

Thus, the web subpanels must be checked against shear buckling.

The reduced slenderness of the web subpanel:

\[
\bar{\lambda}_{w,sp} = \frac{b_{w,sp}}{37.4 t_w \varepsilon(t_u) \sqrt{k_{w,z}}} = 0.858
\]
EN 1993-1-1, 6.2.6, Shear

(1) The design value of the shear force $V_{Ed}$ at each cross-section should satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \quad (6.17)$$

where $V_{c,Rd}$ is the design shear resistance. For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in (2). For elastic design $V_{c,Rd}$ is the design elastic shear resistance calculated using (4) and (5).

(2) In the absence of torsion the design plastic shear resistance is given by:

$$V_{pl,Rd} = \frac{A_i \left( f_y / \sqrt{3} \right)}{\gamma_{M0}} \quad (6.18)$$

where $A_i$ is the shear area.

EN 1993-1-5, 5.5, Verification

(1) The verification should be performed as follows:

$$\eta_i = \frac{V_{Ed}}{V_{b,Rd}} \leq 1$$

where $V_{Ed}$ is the design shear force including shear from torque.

EN 1993-1-1, 6.2.7(9), Torsion

(9) For combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T,Rd}$ and the design shear force should satisfy:

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \quad (6.25)$$

in which $V_{pl,T,Rd}$ may be derived as follows:

- for a structural hollow section:

$$V_{pl,T,Rd} = \left[ 1 - \frac{\tau_{Ed}}{\left( f_y / \sqrt{3} \right) / \gamma_{M0}} \right] V_{pl,Rd} \quad (6.28)$$

where $V_{pl,Rd}$ is given in 6.2.6.
Cross-section verifications - Box-girder bridge at the internal support P3

Shear resistance verification

Thus, it is the stiffened webpanel which is critical: \( \overline{\tau}_w = \max(\overline{\tau}_w, \overline{\tau}_{w,op}) = 0.881 \)

As the webpanel close to support P3 is assumed to be rigid and \( 0.8 \leq \overline{\tau}_w \leq 1.08 \), the reduction factor is:

\[ \chi_{w} = \frac{0.83}{\overline{\tau}_w} = 0.942 \]

The maximum design value of the shear resistance is given by

\[ V_{Rd} = \min (V_{b,Rd}; V_{pl,a,Rd}) \] with \( V_{b,Rd} = V_{bw,Rd} \)

neglecting the flange contribution to the resistance:

\[ V_{bw,Rd} = \frac{\chi_{w} f_y(t_w) h_w t_w}{\sqrt{3} \gamma_{M1}} = 20.881 \text{ MN} \]

\[ V_{b,Rd} = \min \left( V_{bw,Rd}; \frac{\eta f_y(t_w) h_w t_w}{\sqrt{3} \gamma_{M1}} \right) = 20.881 \text{ MN} \]

\[ V_{pl,a,Rd} = \frac{\eta f_y(t_w) h_w t_w}{\sqrt{3} \gamma_{M1}} = 29.251 \text{ MN} \]

so that \( \eta = \frac{V_{Ed}}{V_{Rd}} = \frac{20.165}{20.881} = 0.966 \leq 1 \)

Shear resistance is verified!

Addition of torsional effect

The maximum torque on the box-girder bridge at the internal support P3 is equal to \( M_T = 1.5 \cdot 20.761 \) MNm = 31.142 MNm (see Figure 2-35).

The area inside the median line of the box-girder bridge:

\[ S = \frac{\left( b_i + b_o \right) \left( h + \frac{t_{web}}{2} \right)}{2} = \frac{12 + 6.5 \left( 4 + \frac{0.325}{2} \right)}{2} = 38.503 \text{ m}^2 \]

The shear stress in the web is given by the Bredt formula:

\[ \tau_{Ed,web} = \frac{M_T}{2S t_w} = 14.978 \text{ MPa} \]

The shear force in the web due to torque is:

\[ V_{Ed,web} = \tau_{Ed,web} h_w = 1.833 \text{ MN} \]

Thus, the verification of shear including the torsional effect gives:

\[ \eta_3 = \frac{V_{Ed} + V_{T,web}}{V_{Rd}} = \frac{20.165 + 1.833}{20.713} = 1.061 > 1 \]

\( \eta_3 > 1 \), but the maximum torque value is combined with the maximum design value of the shear resistance which do usually not appear at the same time.
EN 1993-1-5, Annex A3, Shear buckling coefficients

(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient \( k_i \) can be obtained as follows:

\[
\begin{align*}
    k_i &= 5.34 + 4 \left( \frac{h_w}{a} \right)^2 + k_{iisl} \quad \text{when} \quad a/h_w \geq 1 \\
    k_i &= 4 + 5.34 \left( \frac{h_w}{a} \right)^2 + k_{iisl} \quad \text{when} \quad a/h_w < 1
\end{align*}
\]

where \( k_{iisl} = 9 \left( \frac{h_w}{a} \right)^2 \left( \frac{I_{isl}}{t'h_w} \right)^{3/2} \) but not less than \( k_{iisl} = \frac{2.1}{r} \left( \frac{I_{isl}}{h_w} \right) \)

\( a \) is the distance between transverse stiffeners (see Figure 5.3);

\( I_{isl} \) is the second moment of area of the longitudinal stiffener about the z-axis, see Figure 5.3 (b).

For webs with two or more longitudinal stiffeners, not necessarily equally spaced, \( I_{isl} \) is the sum of the stiffness of the individual stiffeners.

NOTE: No intermediate non-rigid transverse stiffeners are allowed for in equation (A.5)

Figure 5.3: Web with transverse and longitudinal stiffeners

(2) The equation (A.5) also applies to plates with one or two longitudinal stiffeners, if the aspect ratio \( \alpha = \frac{a}{h_w} \) satisfies \( \alpha \geq 3 \). For plates with one or two longitudinal stiffeners and an aspect ratio \( \alpha < 3 \) the shear buckling coefficient should be taken from:

\[
    k_i = 4.1 + \frac{6.3 + 0.18 \frac{I_{isl}}{t'h_w}}{\alpha^2} + 2.2 \frac{I_{isl}}{r't'h_w}
\]  

(A.6)
Shear verification including torsional effect

The shear area of the box section:

\[ A_v = h_w t_w = 0.122 \text{ m}^2 \]

The design plastic shear resistance:

\[ V_{pl,rd} = \frac{\eta A_v (f_y / \sqrt{3})}{\gamma_{M0}} = 29.251 \text{ MN} \]

The reduced design plastic shear resistance:

\[ V_{pl,Trd} = \left[ 1 - \frac{\tau_{Ed}}{(f_y / \sqrt{3})/\gamma_{M0}} \right] V_{pl,rd} = 27.272 \text{ MN} \]

Thus, the verification in shear including torsional effect gives:

\[ \frac{V_{Ed} + V_{T,web}}{V_{rd}} = \frac{20.165 + 1.833}{27.272} = 0.806 < 1 \]

Shear resistance including shear from torque is verified!

3.2.3.8.2 Shear in the stiffened bottom flange of the box section

Calculation of the shear stress in the bottom flange

The shear stress in the bottom flange varies from \( \tau_{Ed,\text{min}} = 0 \) in the vertical symmetry axis of the cross-section to \( \tau_{Ed,\text{max}} \) at the junction of the bottom flange with the main web. \( \tau_{Ed,\text{max}} \) is calculated considering the construction phases and using the initial gross cross-section.

The shear force \( V_{Ed} = 33.234 \text{ MN} \) at support P3 is broken down into:

- \( V_{Ed,a} = 19.675 \text{ MN} \) applied to the structural steel box section only (\( I_{tot,a} = 4.588 \text{ m}^4 \), \( z_{na,a} = 1.882 \text{ m} \)) and which corresponds to a shear stress in the bottom flange equal to:

\[ \tau_{Ed,a} = \frac{V_{Ed,a}}{I_{tot,a}} \frac{\mu_{a}}{t_p} = 26.25 \text{ MPa} \]

where \( \mu_{a} \) is the moment of area of the bottom flange with respect to the elastic neutral axis of the cross-section:

\[ \mu_{a} = \frac{b_p}{2} t_p z_{na,a} = 0.459 \text{ m}^3 \]

- \( V_{Ed,c} = 13.559 \text{ MN} \) applied to the composite box section (\( I_{tot} = 5.015 \text{ m}^4 \), \( z_{na,a} = 2.01 \text{ m} \)) and which corresponds to a shear stress in the bottom flange equal to:

\[ \tau_{Ed,c} = \frac{V_{Ed,c}}{I_{tot}} \frac{\mu_{c}}{t_p} = 17.674 \text{ MPa} \]

where \( \mu_{c} = \frac{b_p}{2} t_p z_{na,a} = 0.49 \text{ m}^3 \)

\[ \tau_{Ed,\text{max}} = \tau_{Ed,a} + \tau_{Ed,c} = 43.924 \text{ MPa} \]

The shear stress due to torsion should be added to this value by using the same way than in previous Paragraph 3.2.3.8.1.
The shear stress in the web is given by the Bredt formula:

\[
\tau_{Ed,T,ly} = \frac{M_{f}}{2St_{p}} = 4.853 \text{MPa}
\]

\[
\Rightarrow \quad \tau_{Ed,max} = \tau_{Ed,\alpha} + \tau_{Ed,\varepsilon} + \tau_{Ed,T,ly} = 48.776 \text{ MPa}
\]

**Shear stress check in the global stiffened bottom flange**

The bottom flange is transversally stiffened on both sides at the internal support P3 (a_{w} = 2.5 m) and longitudinally stiffened every b_{sub} = 0.5 m by six closed-section stiffeners equally spaced.

To evaluate the shear buckling coefficient of the stiffened bottom flange panel, the second moment of area of the bottom flange stiffener must be calculated according to EN 1993-1-5, Figure 5.3:

\[
15\frac{\varepsilon(t_{p})}{t_{p}}t_{p} = 0.957 \text{ m} \geq \frac{b_{1,at.w}}{2} = 0.25 \text{ m} \quad \text{and} \quad 15\frac{\varepsilon(t_{p})}{t_{p}}t_{p} = 0.957 \text{ m} \geq \frac{b_{sub}}{2} = 0.25 \text{ m}
\]

Then the elastic neutral axis of the web stiffener with the width 15\frac{\varepsilon(t_{p})}{t_{p}}t_{p} (with the upper limit b_{sub}/2 or b_{1,at.w}/2) on both sides of the stiffener is:

\[
z_{\alpha,p} = \frac{2h_{at.w}t_{at.w} + h_{st.w} + b_{st.w}h_{st.w}}{2} \left( h_{at.w} + \frac{t_{p} - t_{at.w}}{2} \right) = 63.674 \text{ mm}
\]

The second moment of area of one bottom flange stiffener is:

\[
I_{st,p} = \frac{1.718 \times 10^{-3} \text{ m}^{4}}{}
\]

The second moment of area of the six bottom flange stiffeners is:

\[
I_{st,p} = 6I_{st,p} = 0.01 \text{ m}^{4}
\]

According to EN 1993-1-5, Annex A3 (2), as there are six stiffeners in the bottom flange and the aspect ratio \(\alpha_{p} = \frac{a_{w}}{b_{p}} = 0.385 \leq 1\), the shear buckling coefficient is:

\[
k_{t,p} = 4 + 5.34 \left( \frac{b_{p}}{a_{w}} \right)^{2} + k_{at,p} = 204.342
\]

where \(k_{at,p} = 9 \left( \frac{b_{p}}{a_{w}} \right)^{2} \left( \frac{I_{at,p}}{t_{p}b_{p}} \right)^{3/2} = 164.243 \geq \frac{21}{t_{p} \sqrt{b_{p}}} = 3.265
\]

The transverse stiffeners of the bracings frames bordering the bottom flange panel close to support P3 are assumed to be rigid.

\[
\frac{b_{p}}{t_{p}} = 6500 \times 75 = 86.667 < \frac{31}{\eta} \varepsilon(t_{p}) \sqrt{\kappa_{t,p}} = 314.015
\]

Thus, the bottom flange must not be checked against shear buckling. It is therefore deduced that no global plate buckling occurs due to shear stress in the bottom flange.

\[
\Rightarrow \quad \tau_{Ed,max} / 2 = 48.776 \text{ MPa}/2 = 24.388 \text{ MPa} \leq \tau_{Rd} = \frac{\eta f_{c(t_{p})}}{\gamma_{M1} \sqrt{3}} = 204.697 \text{ MPa} \quad \text{(with} \quad \eta = 1.2\text{)}
\]

\[
\Rightarrow \quad \eta_{3} = \frac{\tau_{Ed,max} / 2}{\tau_{Rd}} = 0.119 < 1
\]
EN 1993-1-1, 6.2.6 (4)

(4) For verifying the design elastic shear resistance $V_{c,Rd}$ the following criterion for a critical point of the cross-section may be used unless the buckling verification in EN 1993-1-5, Section 5 applies:

$$\frac{\tau_{Ed}}{f_s/\sqrt{3} \gamma_{M0}} \leq 1,0$$

(6.19)

where $\tau_{Ed}$ may be obtained from:

$$\tau_{Ed} = \frac{V_{Ed} S}{It}$$

(6.20)

where $V_{Ed}$ is the design value of the shear force

$S$ is the first moment of area about the centroidal axis of that portion of the cross-section between the point at which the shear is required and the boundary of the cross-section

$I$ is second moment of area of the whole cross-section

$t$ is the thickness at the examined point

**NOTE:** The verification according to (4) is conservative as it excludes partial plastic shear distribution, which is permitted in elastic design, see (5). Therefore it should only be carried out where the verification on the basis of $V_{c,Rd}$ according to equation (6.17) cannot be performed.

---

**Further explanations on the M-V-interaction**

See Paragraph 3.1.2.5.3, page 93.

---

EN 1993-1-5, 7.1(2)

(2) The criterion given in (1) should be verified at all sections other than those located at a distance less than $h_w/2$ from a support with vertical stiffeners.

---

EN 1993-1-5, 7.1(5)

(5) A flange in a box-girder should be verified using 7.1(1) taking $M_{fs} = 0$ and $\tau_{fs}$ taken as the average shear stress in the flange which should not be less than half the maximum shear stress in the flange and $\eta_1$ is taken as $\eta_1$ according to 4.6(1). In addition the subpanels should be checked using the average shear stress within the subpanel and $\chi_s$ determined for shear buckling of the subpanel according to 5.3, assuming the longitudinal stiffeners to be rigid.
Shear stress check in each subpanel of the bottom flange

The longitudinal stiffeners are assumed to be rigid. In the bottom flange they define subpanels of the size \( a_w = 2500 \text{ mm} \) and \( b_{sub} = 500 \text{ mm} \). These subpanels should be individually checked for shear resistance. The verification is only performed in the most loaded subpanel, namely the one bordering the main steel web of the box section where the average shear stress reaches.

\[
\tau_{Ed} = \tau_{Ed,max} + \left( \frac{\tau_{Ed,hy} - \tau_{Ed,max}}{b_p/2} \right) b_{sub} = 48.776 + \frac{(4.853 - 48.776)500/2}{6500/2} = 45.398 \text{ MPa}
\]

According to EN 1993-1-5, Annex 3(2), as the aspect ratio \( \alpha_{sub} = \frac{a_w}{b_{sub}} = 5 \geq 1 \), the shear buckling coefficient is:

\[
k_{t,p} = 5.34 + 4 \left( \frac{b_{sub}}{a_w} \right)^2 + k_{tsub} = 5.5
\]

where \( k_{tsub} = 0 \)

The transverse stiffeners of the bracings frames bordering the bottom flange panel close to support P3 are assumed to be rigid.

\[
\frac{b_{sub}}{t_p} = \frac{500}{75} = 6.67 < \frac{31}{\eta} e(t_p) \sqrt{k_{tsub}} = 51.517
\]

Thus, the subpanels of the bottom flange must not be checked against shear buckling. It is therefore deduced that no local buckling occurs in the bottom flange due to shear.

\[
\Rightarrow \tau_{Ed} = 45.398 \text{ MPa} \leq \tau_{b,Rd} = \frac{\eta f_y(t_p)}{\sqrt{3}} = 204.697 \text{ MPa (with } \eta = 1.2 \}
\]

\[
\Rightarrow \eta = \frac{\tau_{Ed}}{\tau_{b,Rd}} = 0.22 \leq 1
\]

Shear resistance is verified!

### 3.2.3.9 Interaction between bending moment and shear force

#### 3.2.3.9.1 M-V-interaction in the box-girder webs

The section to be verified is located at a distance \( h_w/2 = 2.266 \text{ m} \) from support P3. In this section \( M_{Ed} = -670.487 \text{ MNm} \) and \( V_{Ed} = 18.932 \text{ MN} \) (considering the inclination of the web).

\[
\bar{\eta}_3 = \frac{V_{Ed}}{V_{b, Rd}} = 0.914 \geq 0.5
\]

The M-V-interaction should be checked by justifying the following criterion in the box section webs:

\[
\bar{\eta}_1 + \left[ 1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right] \left[ 2\bar{\eta}_3 - 1 \right]^2 \leq 1 \text{ if } \bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}} \geq \frac{M_{f,Rd}}{M_{pl,Rd}}
\]

The plastic bending moment resistance of the section, as well as the plastic bending moment resistance of the flanges only, is calculated with the effective cross-sections of the flanges (taking into account the shear lag effect and the possibility of plate buckling).

The web is in tension in its upper part and in compression in its lower part. To calculate \( M_{f,Rd} \) the position of the Plastic Neutral Axis (PNA) is determined as follows:
Relations to find the location of the Plastic Neutral Axis (PNA) under negative moment $M_{PL,Rd}$

<table>
<thead>
<tr>
<th>RELATIONS</th>
<th>PNA LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{abf} \geq N_{af,1} + N_{af,2} + N_{sl} + N_{su}$</td>
<td>PNA in the bottom flange</td>
</tr>
<tr>
<td>$N_{abf} + N_{af,2} \geq N_{af,1} + N_{sl} + N_{su}$ and $N_{abf} &lt; N_{af,1} + N_{af,2} + N_{sl} + N_{su}$</td>
<td>PNA in the top flange 2</td>
</tr>
<tr>
<td>$N_{abf} + N_{af,2} + N_{af,1} \geq N_{sl} + N_{su}$ and $N_{abf} + N_{af,2} &lt; N_{af,1} + N_{sl} + N_{su}$</td>
<td>PNA in the top flange 1</td>
</tr>
<tr>
<td>$N_{sl} + N_{su} &gt; N_{abf} + N_{af,1} + N_{af,2}$</td>
<td>PNA in the slab</td>
</tr>
</tbody>
</table>
Design plastic resistance of the bottom flange:
\[ N_{abf} = \left[ n_a \left[ t_{a,f} f_{yd} (t_{slw}) (b_{2,eff} + 2b_{3,eff}) + t_p f_{yd} (t_p) (h_{eff} + b_{sub,eff}) \right] \rho_v + t_p f_{yd} (t_p) (0.2m + b_{sub,eff}) \right] \rho_{st} = 181.359 \text{ MN} \]

Design plastic resistance of the two structural steel top flanges 1:
\[ N_{a.tf.1} = 2 \cdot b_{tf.1} \cdot t_{tf.1} \cdot f_{yd} (t_{tf.1}) = 94.5 \text{ MN} \]

Design plastic resistance of the two structural steel top flanges 2:
\[ N_{a.tf.2} = 2 \cdot b_{tf.2} \cdot t_{tf.2} \cdot f_{yd} (t_{tf.2}) = 79.38 \text{ MN} \]

Design plastic resistance of the upper steel reinforcement:
\[ N_{su} := A_{s.ur} f_{sd} = 22.59 \text{ MN} \]

Design plastic resistance of the lower steel reinforcement:
\[ N_{sl} := A_{s.lr} f_{sd} = 14.458 \text{ MN} \]

Location of the Plastic Neutral Axis (PNA)
\[ N_{abf} + N_{a.tf.2} = 275.857 \text{ MN} \geq N_{a.tf.1} + N_{su} + N_{sl} = 131.548 \text{ MN} \]
\[ N_{abf} = 181.357 \text{ MN} < N_{a.tf.1} + N_{a.tf.2} + N_{sl} + N_{su} = 210.928 \text{ MN} \]

Thus the PNA is deduced to be located in the top flange 2 at a distance \( z_{pl} \) from the extreme lower fiber of the bottom flange. Writing the force equilibrium around the PNA deduced:
\[ z_{pl} = \frac{4(h - t_{gf.1}) b_{gf.2} f_{yd} (t_{gf.2}) + N_{a.gf.1} + N_{su} + N_{sl} - N_{abf} - N_{a.gf.2}}{4b_{gf.2} f_{yd} (t_{gf.2})} = 3.827 \text{ m} \]

The design plastic resistance moment of the flanges only is calculated from the position of the PNA:
\[ M_{f,Rd} = N_{su} (h + t_{slab} - c_{ur} - z_{pl}) + N_{sl} (h + c_{ur} - z_{pl}) \]
\[ + \frac{(h - t_{gf.1} - z_{pl})^2}{2} 2b_{gf.2} f_{yd} (t_{gf.2}) + \frac{(h - t_{gf.1} - t_{gf.2} - z_{pl})^2}{2} 2b_{gf.2} f_{yd} (t_{gf.2}) = 714.623 \text{ MNm} \]
\[ + N_{a.gf.1} \left( h - t_{gf.1} - z_{pl} \right) + N_{a.bf} \left( z_{pl} - \frac{h_p}{2} \right) \]

As \( |M_{Ed}| = 709.513 \text{ MNm} < M_{f,Rd} = 714.623 \text{ MNm} \), there is finally no need to verify the interaction criteria.

### 3.2.3.9.2 Interaction M-V in the bottom flange of the box section

The value \( \eta_3 = 0.119 \) has already been calculated in Paragraph 3.2.3.8.2. Thus, \( \eta_3 < 0.5 \).

\[ \Rightarrow \text{ There is no need to check the M-V-interaction.} \]
4 Verifications during erection

4.1 Twin-girder bridge

4.1.1 General

The construction phasing of the twin-girder bridge is done according to the following scheme: firstly, the steel superstructure of the twin-girder bridge is erected by the incremental launching technique. Secondly, the concrete slab is cast in-situ according to the order already described in Section 2.1.4. Finally, the non-structural equipment is installed. During erection, usually each cross-section has to be verified at every construction stage which cannot all be part of this section. Thus, in the following, the verifications during erection focus on the patch loading resistance of the steel girders during launching.

The bridge is launched from one side (abutment C0) only. In order to recover and to reduce the deflection of the cantilever part when approaching a support, a launching nose is used. The launching nose has a length of 11.75 m and its total weight decreases from 18 kN/m at the cross-section connected to the bridge girders to 12 kN/m at its free end. A provisional wind bracing is added between the two steel girders for the launching process. At the supports, sliding skates with a loading length of $s_s = 1.5$ m are used.

The determination of the patch loading resistance is done both according to Section 6, EN 1993-1-5 in Section 4.1.2 and according to Section 10, EN 1993-1-5 in Section 4.1.3. However, initially the most unfavourable launching situation has been determined as shown in Figure 4-1 which corresponds to a location of the steel girders at a position of $x = 111.75$ m.

![Figure 4-1: Most unfavourable launching situation.](image)

The dimensions of the studied panel at the decisive cross-section are given in Figure 4-2. The web is longitudinally unstiffened and the spacing of the vertical stiffeners is based on the verifications against lateral torsional buckling, i.e. 3.5 m here. On the safe side, additional stiffeners which may have been added for shear verification are not considered.
Figure 4-2: Dimensions of the studied panel in [m].

From the global analysis, the internal design forces (for use with Section 6, EN 1993-1-5) are given below for one main steel girder.

\[
\begin{align*}
M_{Ed} &= -19.26 \text{ MNm} \\
V_{Ed,max} &= 0.73 \text{ MN} \\
V_{Ed,applied} &= V_{Ed,max} - F_{Ed}/2 = 0 \text{ MN} \\
F_{Ed} &= 1.46 \text{ MN}
\end{align*}
\]

The resulting stress field (for use with Section 10, EN 1993-1-5) acting at the studied panel is shown in Figure 4-3.

\[
\begin{align*}
\sigma_{x,Ed,\text{top}} &= -96.1 \text{ MPa} \\
\tau &\approx 0 \text{ MPa} \\
\sigma_{x,Ed,\text{bot}} &= 79.7 \text{ MPa} \\
\sigma_{z,Ed} &= 45.3 \text{ MPa}
\end{align*}
\]

Figure 4-3: Stress field acting at the studied panel.
Buckling value $k_F$

The formula given for the buckling value $k_F$ must be exclusively used for the determination of the critical load $F_{cr}$ because the reduction curve $\chi_F(\lambda_F)$ has been calibrated based on this formula for $k_F$. 

4.1.2 Verifications according to Sections 6 and 7, EN 1993-1-5

In the following, the patch loading resistance according to Section 6, EN 1993-1-5 is determined. The interaction between transverse force and bending moment is checked according to Section 7, EN 1993-1-5.

Determination of the critical load $F_{cr}$

\[ k_F := 6 + 2 \left( \frac{h_w}{a} \right)^2 \quad \text{and} \quad k_F = 6.80 \]

\[ F_{cr} := 0.9 \cdot k_F \cdot 210000 \cdot \frac{t_w^3}{h_w} \quad \text{so} \quad F_{cr} = 3.99 \text{MN} \]

Determination of the yield load $F_y$

\[ m_1 := \frac{f_y f}{f_y w t_w} \quad \text{so} \quad m_1 = 48.05 \]

\[ m_2 := 0.02 \left( \frac{h_w}{t_f} \right)^2 \quad \text{so} \quad m_2 = 10.82 \]

\[ l_y := s_s + 2 \cdot t_f \cdot \left( 1 + \sqrt{m_1 + m_2} \right) \]

\[ l_y := \begin{cases} 1 \cdot l_y & \text{if } l_y \leq a \\ a & \text{if } l_y > a \end{cases} \quad \text{so} \quad l_y = 3148 \text{ mm} \]

\[ F_y := l_y \cdot t_w \cdot f_y w \quad \text{so} \quad F_y = 20.63 \text{ MN} \]

Determination of the slenderness parameter $\lambda_F$

\[ \lambda_F := \frac{F_y}{F_{cr}} \quad \text{so} \quad \lambda_F = 2.27 \]

Here, the slenderness parameter is larger than $\lambda_F = 0.5$ which is a precondition to use the above formula for $m_2$. For $\lambda_F \leq 0.5$, $m_2$ should be set to zero according to Paragraph 6.5(1), EN 1993-1-5.

Determination of the reduction factor $\chi_F$

\[ \chi_F := \begin{cases} 1.0 & \text{if } \lambda_F \leq 0.5 \\ 0.5 & \text{if } \lambda_F > 0.5 \end{cases} \quad \text{so} \quad \chi_F = 0.22 \]
Improved resistance to transverse forces

In the COMBRI research project and in [31] it has been shown that in case the value m_2 is set to zero and, due to the changed definition of the yield load, the reduction curve is recalibrated, not only the resistance to transverse forces can be improved but also the scatter of the resistance model becomes smaller. For details, see the COMBRI Final report [7].

Interaction between transverse force and bending moment

In Section 7.2(1), EN 1993-1-5, the interaction should be determined as follows:

\[ \eta_2 + 0.8 \cdot \eta_1 \leq 1.4 \]

If \( \eta_2 = 1.0 \) is assumed, it can be shown that the influence of bending moment must be taken into account only for \( \eta_1 > 0.5 \).
Determination of the patch loading resistance

\[ L_{\text{eff}} := \chi F \cdot t_y \]

\[ L_{\text{eff}} = 691.9 \text{ mm} \]

\[ F_{\text{Rd}} := \frac{f_{yw} \cdot L_{\text{eff}} \cdot t_w}{\gamma_M 1} \]

\[ F_{\text{Rd}} = 4.12 \text{ MN} \]

\[ \eta_2 := \frac{F_{\text{Ed}}}{F_{\text{Rd}}} \]

\[ \eta_2 = 0.353 \]

Interaction between transverse force and bending moment

Here, \( \eta_1 = 0.265 < 0.5 \) so that the interaction with bending moment is not decisive.

However, in the following the calculation according Section 7, EN 1993-1-5 is shown:

\[ \eta_2 + 0.8 \cdot \eta_1 \leq 1.4 \]

Here \( \eta_1 = 0.270 \) and \( \eta_2 = 0.353 \), so that the interaction verification becomes

\[ 0.353 + 0.8 \cdot 0.270 = 0.569 < 1.4 \]

4.1.3 Verifications according to Section 10, EN 1993-1-5

In the following, the patch loading resistance according to Section 10, EN 1993-1-5 is determined.

Determination of \( \alpha_{cr} \)

The determination of the minimum load amplifier \( \alpha_{cr} \) for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- for each component of the stress field (e.g. by a hand-calculation)

\[ \sigma_E := 189800 \text{ MPa} \left( \frac{t_w}{h_w} \right)^2 \]

\[ \sigma_E = 14.03 \text{ MPa} \]

Elastic critical longitudinal stress acc. to Table 4.1, Section 4.4, EN 1993-1-5

\[ k_{\sigma, x} := \begin{cases} 8.2 & \text{if } 1 \geq \psi > 0 \\ 1.05 + \psi & \text{if } 0 \geq \psi > -1 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } -1 \geq \psi > -3 \\ 5.98(1 - \psi)^2 & \text{if } -3 \geq \psi > -3 \end{cases} \]

\[ k_{\sigma, x} = 29.08 \]

\[ \sigma_{cr, x} := k_{\sigma, x} \cdot \sigma_E \]

\[ \sigma_{cr, x} = 407.95 \text{ MPa} \]

\[ \alpha_{cr, x} := \frac{\sigma_{cr, x}}{\sigma_{x, Ed\bot}} \]

\[ \alpha_{cr, x} = 5.12 \]
Elastic critical transverse stress with \( k = 2.08 \) acc. to Table 8.12 [38]

\[
\sigma_{cr,z} := k \cdot \sigma_E \cdot \frac{1}{s_s + 2 \cdot \tau_f} \\
\sigma_{cr.z} = 60.43 \text{ MPa}
\]

\[
\alpha_{cr,z} := \frac{\sigma_{cr,z}}{\sigma_{z,Ed}} \\
\alpha_{cr,z} = 1.33
\]


\[
k_\tau := \begin{cases} 
5.34 + 4.00 \cdot \left( \frac{h_w}{a} \right)^2 & \text{if } \frac{a}{h_w} \geq 1 \\
4.00 + 5.34 \cdot \left( \frac{h_w}{a} \right)^2 & \text{if } \frac{a}{h_w} < 1 
\end{cases} \\
k_\tau = 6.93
\]

\[
\tau_{cr} := k_\tau \cdot \sigma_E \\
\tau_{cr} = 97.29 \text{ MPa}
\]

\[
\alpha_{cr,\tau} := \frac{\tau_{cr}}{\tau_{Ed}} \\
\alpha_{cr,\tau} := \infty
\]

Minimum load amplifier acc. to Eq. (10.6), Section 10, EN 1993-1-5

\[
\alpha_{cr} := \frac{1}{1 + \psi + \frac{1}{2 \alpha_{cr,z}} + \sqrt{\left( \frac{1 + \psi}{4 \alpha_{cr,x}} \right) + \frac{1}{2 \alpha_{cr,z}}} + \frac{1}{\alpha_{cr,\tau}}} \\
\alpha_{cr} = 1.276
\]

- for the complete stress field (e.g. by using appropriate software)

In case e.g. software EBPlate is used, the minimum load amplifier can be determined in a single step as \( \alpha_{cr} = 1.259 \).

In the following calculations, the value \( \alpha_{cr} = 1.276 \) is further used.

Determination of \( \alpha_{ult,k} \)

\[
\sigma_{eq} := \sqrt{\sigma_{x,Ed,bot}^2 + \sigma_{z,Ed}^2 - \sigma_{x,Ed,bot} \cdot \sigma_{z,Ed} + 3 \cdot \tau_{Ed}^2} \\
\sigma_{eq} = 69.26 \text{ MPa}
\]

\[
\alpha_{ult,k} := \frac{f_{yw}}{\sqrt{\sigma_{x,Ed,bot}^2 + \sigma_{z,Ed}^2 - \sigma_{x,Ed,bot} \cdot \sigma_{z,Ed} + 3 \cdot \tau_{Ed}^2}} \\
\alpha_{ult,k} = 4.98
\]
**Reduction curve for transverse stresses, column-like behaviour and interpolation function**

In the COMBRI project it has been shown that the interpolation function acc. to Eq. (4.13), Section 4.5.4, EN 1993-1-5, which takes into account column-like buckling, is not appropriate for patch loading stresses in the transverse direction. Moreover, the ratio of $V_{cr,p}/V_{cr,c}$, for which column-like buckling needs to be considered, should be 2.7 (and not 2.0 as given in Eq. (4.13)).

Basically, either a new interpolation function is required or a reduction curve should be used which can be used with the existing interpolation function. A new interpolation function has been derived e.g. by Seitz [40] and this method is introduced in the COMBRI Final report [7]. However, e.g. for the new German DIN-Fachbericht 103 [12], it has been decided to use the reduction curve of Annex B, EN 1993-1-5, for transverse stresses because this approach complies well with current Eurocode rules and could be easier implemented. Thus, for a welded girder Table B.1, Annex B.1, EN 1993-1-5 gives:

$$\bar{O}_p = 0.80 \quad D_p = 0.34 \quad I_p = 0.5 \quad 1$$

which is to be used in

$$\rho := \frac{1}{\phi_{p,} + \sqrt{\phi_{p,}^2 - \lambda_p}}$$

This reduction curve is used on the right hand-side instead of Eq. (4.13), Section 4.5.4, EN 1993-1-5.

---

**Elastic critical column-buckling stress $\sigma_{cr,c}$**

The determination of $\sigma_{cr,c}$ in the transverse direction should take into account the nonlinear stress distribution in the web for which no hand-calculation method exists at the moment. Instead, a common approach is to assume a linearly varying stress distribution for a pin-ended strut which is zero at one end and which can be calculated acc. to DIN 4114 as follows: $\sigma_{cr,c} = 1.88 \cdot \sigma_E$.

It should be noted that this simplification may lead to unsafe results because $\sigma_{cr,c}$ is underestimated and in turn the ratio $\sigma_{cr,p}/\sigma_{cr,c}$ is overestimated so that column-like behaviour is not detected correctly. The shorter the loading lengths is, the larger this deviation is.
Determination of the slenderness parameter $\bar{\lambda}_p$

$$\bar{\lambda}_p := \frac{\alpha_{\text{ult},k}}{\alpha_{\text{cr}}}$$

$\lambda_p = 1.976$

Determination of the reduction factors

The determination of the reduction factors for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- **using different buckling curves**

  Longitudinal stress acc. to Eq. (4.2), Section 4.4, EN 1993-1-5

  $$\rho_x := \frac{\lambda_p - 0.055 \cdot (3 + \gamma)}{\lambda_p^2}$$

  $\rho_x = 0.481$

  Check of column-like behaviour in the longitudinal direction

  Column-like buckling in the longitudinal direction for plates should be checked according to Section 4.4(6), EN 1993-1-5, for panel aspect ratios $a < 1.0$. Therefore column-like behaviour is not calculated in detail here.

  Transverse stress acc. to Annex B.1, EN 1993-1-5

  $$\rho_z := \frac{1}{\phi_{p..} + \sqrt{\phi_{p..}^2 - \lambda_p^2}}$$

  with $\phi_{p..} = 1.688$

  $\rho_z = 0.381$

  Check of column-like behaviour in the transverse direction

  Due to the nonlinear distribution of transverse stresses in the web, the determination of the critical column-buckling stress has been done here most accurately based on the energy method which takes into account the nonlinear stress distribution.

  $$\sigma_{\text{cr},c} := 28.55\text{MPa}$$

  $$\frac{\sigma_{\text{cr},z}}{\sigma_{\text{cr},c}} = 2.12 \quad \xi := \frac{\sigma_{\text{cr},z}}{\sigma_{\text{cr},c}} - 1 \quad \xi = 1.12$$

  Because $\xi > 1.0$, column-like behaviour does not have to be considered acc. to the definition in the existing Section 4.5.4, EN 1993-1-5.

  Shear stress acc. to Table 5.1, Section 5.3, EN 1993-1-5 with $\eta = 1.2$

  $$\chi_w := \begin{cases} 1.0 & \text{if } \lambda_p < \frac{\eta}{0.83} \\ 0.83 & \text{if } \lambda_p \geq \frac{\eta}{0.83} \end{cases}$$

  $\chi_w = 0.420$
Use of a corrected interpolation function instead of the reduction curve acc. to Annex B.1, EN 1993-1-5

For comparison, the use of the newly derived interpolation function acc. to Seitz [40] would give:

$$\eta_{\text{Seitz}} = 0.477$$

For comparison with Section 4.1.2 “Application of Section 6, EN 1993-1-5”, the pure patch loading resistance leads to $F_{Rd} = 3.13$ MN. This also shows that the application of existing Section 10, EN 1993-1-5 [23] in combination with Section 4, EN 1993-1-5 overestimates the patch loading resistance by 22.4% ($F_{Rd} = 3.83$ MN). In contrast to this, results from the newly derived interpolation function ($F_{Rd} = 3.13$ MN) and from the buckling curve based on Annex B.1, EN 1993-1-5 ($F_{Rd} = 3.17$ MN) correspond well with a difference of about 1.3%.
Verifications during erection - Twin-girder bridge

- using a single buckling curve

According to Table B.1, Annex B.1, EN 1993-1-5

\[
\begin{align*}
\lambda_{p0} & := 0.80 & \alpha_p & := 0.34 \\
\phi_p & := 0.5 \left[ 1 + \alpha_p \left( \lambda_p - \lambda_{p0} \right) + \lambda_p \right] \\
\rho & := \frac{1}{\phi_p + \sqrt{\phi_p^2 - \lambda_p}} & \phi_p & = 1.688 \\
\rho & = 0.381
\end{align*}
\]

Determination of the patch loading resistance

- using different buckling curves

\[
\eta_{\text{diff}} := \sqrt{\left( \frac{\sigma_{x, Ed, \text{bot}}}{f_{yw}} \right)^2 + \left( \frac{\sigma_{z, Ed}}{\rho_x \gamma_{M1}} \right)^2 - \left( \frac{\sigma_{x, Ed, \text{bot}}}{f_{yw}} \right)^2 \left( \frac{\sigma_{z, Ed}}{\rho_x \gamma_{M1}} \right)} + 3 \left( \frac{\tau_{Ed}}{f_{yw}} \right)^2
\]

\[
\eta_{\text{diff}} = 0.472
\]

For comparison with Section 4.1.2 “Application of Section 6, EN 1993-1-5”, the pure patch loading resistance leads to $F_{Rd} = 3.17$ MN.

- using a single buckling curve

\[
\alpha_{Rd} = \frac{\rho \cdot \alpha_{ult, k}}{\gamma_{M1}} & \quad \alpha_{Rd} = 1.727 \\
\eta_{\text{single}} = \frac{1}{\alpha_{Rd}} & = 0.579
\]

For comparison with Section 4.1.2 “Application of Section 6, EN 1993-1-5”, the pure patch loading resistance leads to $F_{Rd} = 3.17$ MN.
4.1.4 Results

In Figure 4-4 the distribution of the patch loading resistances is summarised along the whole bridge length. It can be shown that the most unfavourable launching situation ($F_{Ed} = 1.456$ MN) can be easily verified not only with the cross-section near the support $P_2$ ($x = 111.75$) but also with other cross-sections in the span region.

![Figure 4-4: Distribution of patch loading resistances according to EN 1993-1-5 along the bridge length.](image)
4.2 Box-girder bridge

4.2.1 General

The construction phasing of the box-girder bridge is done according to the following scheme: firstly, the steel superstructure of the box-girder bridge is erected by the incremental launching technique. Secondly, the concrete slab is cast in-situ according to the order already described in Section 2.2.4. Finally, the non-structural equipment is installed. During erection, usually each cross-section has to be verified at every construction stage which cannot all be part of this section. Thus, in the following, the verifications during erection focus on the patch loading resistance of the steel girders during launching.

The bridge is launched from one side (abutment C0) only. In order to recover and to reduce the deflection of the cantilever part when approaching a support, a launching nose is used. The launching nose has a length of 28 m and its total weight decreases from 57 kN/m at the cross-section connected to the bridge girder to 28 kN/m at its free end. A provisional wind bracing is added between the upper flanges for the launching process. At the supports, sliding skates with a loading length of $s_s = 3.0$ m are used.

The determination of the patch loading resistance is done both according to Section 6, EN 1993-1-5, in Section 4.2.2 and according to Section 10, EN 1993-1-5, in Section 4.2.3. In the following, the different launching situations which will be checked are introduced. In all diagrams the internal forces and support reactions are given for one half of the box-girder.

The most unfavourable launching situation for maximum bending has been determined as shown in Figure 4-1 which corresponds to a location of the steel girder at a position of $x = 448$ m.

![Figure 4-1](image)

**Figure 4-5:** Launching situation “1” (most unfavourable both for maximum bending and pier cross-section).
The most unfavourable launching situation for the weakest cross-section has been determined as shown in Figure 4-6 which corresponds to a location of the steel girder at a position of $x = 484$ m.

Figure 4-6: Launching situation “2” (most unfavourable for weakest end-span cross-section).

The most unfavourable launching situation for unequal shear distribution has been determined as shown in Figure 4-7 which corresponds to a location of the steel girder at a position of $x = 408$ m.

Figure 4-7: Launching situation “3” (most unfavourable for weakest mid-span cross-section).

The dimensions of the studied panel at the decisive cross-sections are given in Table 4-1 and Figure 4-2. The web is longitudinally stiffened at $0.2\cdot h_w$ with reference to the neutral axis of the stiffener, which is an efficient stiffener position in case of patch loading and might not necessarily be in accordance with the choice of stiffener position in the other sections of this document, and the spacing of the vertical stiffeners is based on the verifications against lateral torsional buckling, i.e. $4.0$ m here.
Verifications during erection - Box-girder bridge

Figure 4-8: Notations of the studied panel in [mm].

Table 4-1: Dimensions of the studied panels in [mm].

<table>
<thead>
<tr>
<th>Launching situation</th>
<th>“1”</th>
<th>“2”</th>
<th>“3”</th>
</tr>
</thead>
<tbody>
<tr>
<td>b_{f, top} [mm]</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>t_{f, top} [mm]</td>
<td>184</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>h_w [mm]</td>
<td>4539.4</td>
<td>4726.2</td>
<td>4762.6</td>
</tr>
<tr>
<td>t_w [mm]</td>
<td>27</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td>b_{f, bot} [mm]</td>
<td>1015.3</td>
<td>527.1</td>
<td>405.1</td>
</tr>
<tr>
<td>t_{f, bot} [mm]</td>
<td>75</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td>b_1 [mm]</td>
<td>657.9</td>
<td>695.2</td>
<td>702.5</td>
</tr>
</tbody>
</table>

From the global analysis, the internal design forces (for use with Section 6, EN 1993-1-5) are given below. In case of internal forces and support reactions the values are given for each web.

Table 4-2: Internal design forces.

<table>
<thead>
<tr>
<th>Launching situation</th>
<th>“1”</th>
<th>“2”</th>
<th>“3”</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{Ed} [MNm]</td>
<td>-217.93</td>
<td>-99.35</td>
<td>-50.62</td>
</tr>
<tr>
<td>V_{Ed,max} [MN]</td>
<td>4.25</td>
<td>3.17</td>
<td>2.16</td>
</tr>
<tr>
<td>V_{Ed,applied} = V_{Ed,max} - F_{Ed}/2 [MN]</td>
<td>-0.83</td>
<td>-0.29</td>
<td>-0.83</td>
</tr>
<tr>
<td>F_{Ed} [MN]</td>
<td>8.37</td>
<td>5.69</td>
<td>4.92</td>
</tr>
<tr>
<td>F_{Ed,web} [MN]</td>
<td>10.15</td>
<td>6.91</td>
<td>5.97</td>
</tr>
<tr>
<td>F_{Ed,bottom plate} [MN]</td>
<td>5.75</td>
<td>3.91</td>
<td>3.38</td>
</tr>
</tbody>
</table>

The calculations in Section 4.2.2 showed that the negative bending moment has an influence on the patch loading resistance only for launching position “1”. In all cases, the additional shear force is almost negligible. For these reasons, the calculation for launching situation “1” is given in detail in Section 4.2.2.1 whereas for launching situations “2” and “3”, which are relevant for the mid- and end-span verifications, only results are presented in Sections 4.2.2.2 and 4.2.2.3.
**Determination of the critical load \( F_{cr} \)**

In EN 1993-1-5 the determination of the critical load of a longitudinally stiffened web is based on the lowest buckling value \( k_F \). In the diagram below buckling values for different panel aspect ratios and stiffener positions are given. Based on that, the formula in EN 1993-1-5 describes only the increasing branch and is therefore limited to a range of application \( b_1/h_w \leq 0.3 \).

Thus, according to an EN 1993-1-5 calculation, the most advantageous stiffener position is at \( b_1/h_w = 0.3 \) although this might be not true in reality. In general, the patch loading resistance increases with decreasing distance between stiffener and loaded flange. In the COMBRI project, this paradox has been solved within the COMBRI project, see [7] as well as [5] and [9].

**Improved resistance to transverse forces**

In the COMBRI research project and in [5], [9] it has been shown that in case the value \( m_2 \) is set to zero and, due to the changed definition of the yield load, the reduction curve is recalibrated, not only the resistance to transverse forces can be improved but also the scatter of the resistance model becomes smaller. Whereas the work in [5] is related to the improved resistance model for longitudinally unstiffened girder according to [31], the work of [9] can be used with current Eurocode rules. For this reason, in the German National Annex to EN 1993-1-5 [11] the proposal of [9] has been adopted for the national choice of the patch loading resistance model for longitudinally stiffened girders. For details, see also the COMBRI Final Report [7].

On the following page, the example calculation according to [9] is shown.
4.2.2 Verifications according to Section 6, EN 1993-1-5

4.2.2.1 Launching situation “1”

In the following, the patch loading resistance according to Section 6, EN 1993-1-5 is determined. The interaction between transverse force and bending moment is checked according to Section 7, EN 1993-1-5.

Determination of the critical load $F_{cr}$

$$\gamma_s := 10.9 \cdot \frac{I_{sl.1}}{h_w \cdot t_w^3}$$

$$\gamma_{s.min} := 13 \cdot \left( \frac{a}{h_w} \right)^3 + 210 \cdot \left( 0.3 - \frac{b_1}{a} \right)$$

$$\gamma_s := \begin{cases} 10.9 \cdot \frac{I_{sl.1}}{h_w \cdot t_w^3} & \text{if } \gamma_s \leq \gamma_{s.min} \\ 13 \cdot \left( \frac{a}{h_w} \right)^3 + 210 \cdot \left( 0.3 - \frac{b_1}{a} \right) & \text{if } \gamma_s > \gamma_{s.min} \end{cases}$$

$$\gamma_s = 37.36$$

$$k_F := 6 + 2 \left( \frac{h_w}{a} \right)^2 + \left( 5.44 \cdot \frac{b_1}{a} - 0.21 \right) \cdot \sqrt{\gamma_s}$$

$$k_F = 12.76$$

$$F_{cr} := 0.9 \cdot k_F \cdot 210000 \cdot \frac{t_w^3}{h_w}$$

$$F_{cr} = 10.46 \text{ MN}$$

Determination of the yield load $F_y$

$$m_1 := \frac{f_{yf} \cdot b_f}{f_{yw} \cdot t_w}$$

$$m_1 = 35.42$$

$$m_2 := 0.02 \left( \frac{h_w}{t_f} \right)^2$$

$$m_2 = 73.26$$

$$l_y := s_s + 2 \cdot t_f \left( 1 + \sqrt{m_1 + m_2} \right)$$

$$l_y = 4000 \text{ mm}$$

$$F_y := l_y \cdot t_w \cdot f_{yw}$$

$$F_y = 37.26 \text{ MN}$$
Example calculation based on the procedure according to Davaine [9]

**Determination of the critical load $F_{cr}$**

\[ F_{cr.1} := F_{cr} \quad \text{with} \quad F_{cr.1} = 10.46 \text{ MN} \]

\[ k_{F.2} := \left[ 0.8 \left( \frac{s_s + 2 \cdot t_f}{a} \right) + 0.6 \right] \left( \frac{a}{b_1} \right)^{0.6} \left( \frac{s_s + 2 \cdot t_f}{a} \right)^{0.5} \quad \text{with} \quad k_{F.2} = 7.12 \]

\[ F_{cr.2} := k_{F.2} \cdot 189800 \text{ MPa} \cdot \frac{b_1^3}{t_w} \quad \text{with} \quad F_{cr.2} = 40.41 \text{ MN} \]

\[ F_{cr.} := \frac{F_{cr.1} \cdot F_{cr.2}}{F_{cr.1} + F_{cr.2}} \quad \text{with} \quad F_{cr.} = 8.31 \text{ MN} \]

**Determination of the yield load $F_y$**

The factor $m_2$ has to be set to zero in this procedure.

\[ m_2 := 0 \]

\[ l_y := s_s + 2 \cdot t_f \cdot \left( 1 + \sqrt{m_1} \right) \]

\[ l_y := \begin{cases} l_y & \text{if} \quad l_y \leq a \\ a & \text{if} \quad l_y > a \end{cases} \quad l_y = 4000 \text{ mm} \]

\[ F_{y} := l_y \cdot t_w \cdot f_yw \quad \text{with} \quad F_{y} = 37.26 \text{ MN} \]

**Determination of the slenderness parameter $\lambda_F$**

\[ \lambda_F := \frac{F_{y}}{F_{cr.}} \quad \text{with} \quad \lambda_F = 2.12 \]

**Determination of the reduction factor $\chi_F$**

\[ \phi := 0.5 \left[ 1 + 0.21 \left( \lambda_F - 0.80 \right) + \lambda_F \right] \quad \phi = 1.56 \]

\[ \rho := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_F^2}} \quad \rho = 0.436 \]

**Determination of the patch loading resistance**

\[ L_{eff.} := \rho \cdot l_y \quad \text{with} \quad L_{eff.} = 1744.7 \text{ mm} \]

\[ F_{Rd.} := \frac{f_yw \cdot L_{eff.} \cdot t_w}{\gamma M_1} \quad \text{with} \quad F_{Rd.} = 14.77 \text{ MN} \]

\[ \eta_2 := \frac{F_{Ed.web}}{F_{Rd.}} \quad \eta_2 = 0.687 \]
Determination of the slenderness parameter $\lambda_F$

$$\lambda_F := \sqrt{\frac{F_y}{F_{cr}}}$$

$\lambda_F = 1.89$

Here, the slenderness parameter is larger than $\lambda_F = 0.5$ which is a precondition to use the above formula for $m_2$. For $\lambda_F \leq 0.5$, $m_2$ should be set to zero according to Paragraph 6.5(1), EN 1993-1-5.

Determination of the reduction factor $\chi_F$

$$\chi_F := \begin{cases} 1.0 & \text{if } \lambda_F \leq 0.5 \\ 0.5 & \text{if } \lambda_F > 0.5 \\ \frac{1}{\lambda_F} & \text{if } \lambda_F > 0.5 \end{cases}$$

$\chi_F = 0.26$

Determination of the patch loading resistance

$$L_{\text{eff}} := \chi F^{-1}y$$

$L_{\text{eff}} = 1059.6 \text{ mm}$

$$F_{\text{Rd}} := \frac{f_{yw} L_{\text{eff}} t_w}{\gamma_{M1}}$$

$F_{\text{Rd}} = 8.97 \text{ MN}$

$$\eta_2 := \frac{F_{\text{Ed.web}}}{F_{\text{Rd}}}$$

$\eta_2 = 1.132$

As $\eta_2 = 1.132 > 1.0$ the patch loading verification cannot be fulfilled and thus also the interaction cases are not fulfilled. Instead they are shown for the sake of completeness using the value of the patch loading resistance according to Davaine [9].
Example calculation based on the procedure according to Davaine [9] (cont.)

Based on the recommended procedure in EN 1993-1-5, the patch loading verification can not be fulfilled and a solution would be to increase the web thickness $t_w$ to 30 mm. When using EN 1993-1-5, another stiffener position or longer loading length would not increase the calculated resistance decisively. However, the design models developed in [9] and [5] which are summarised in [7] allow to assess the existing higher patch loading resistance without using numerical simulation tools. Here, the procedure acc. to [9] has been shown to improve the patch loading resistance with regard to the drawbacks described in the remark to the determination of the critical load $F_{cr}$.

Interaction between transverse force and bending moment

In Section 7.2(1), EN 1993-1-5, the interaction should be determined as follows:

$$\eta_2 + 0.8 \cdot \eta_1 \leq 1.4$$

If $\eta_2 = 1.0$ is assumed, it can be shown that the influence of bending moment must be taken into account only for $\eta_1 > 0.5$.

Interaction between transverse force and shear force

In the frame of the COMBRI project, experimental and numerical studies on steel plated girders have been conducted in order to review and to propose an interaction equation for combined shear and patch loading. From the investigations, it is obvious that the interaction between shear and patch loading is not negligible. Although the interaction might appear severely at first sight, the specific conditions during bridge launching have to be taken into account. For this reason, the figure below shows two relevant construction stages: a) when the bridge girder is about to arrive at the support and a cantilever is existent; b) when the bridge girder has reached the pier. In stage a) the introduced patch load is almost equally equilibrated resulting in a pure patch loading situation where the shear is already considered in the patch load model.

![Diagram showing interaction between shear force and bending moment](image-url)
Interaction between transverse force and bending moment

Here, $\eta_1 = 0.580 > 0.5$ so that the interaction with bending moment has to be considered.

$$\sigma_{x,Ed} := \frac{M_{Ed}}{W_{bot}}$$

$\sigma_{x,Ed} = 189.42 \text{ MPa}$

$$\eta_1 := \frac{\sigma_{x,Ed}}{f_{yf}} \frac{J}{\gamma_{M0}}$$

$\eta_1 = 0.58$

$$\eta_2 + 0.8 \cdot \eta_1 = 1.60$$

$W_{bot}$ takes into account the effects of plate buckling in the bottom flange and in the webs.

Interaction between transverse force and shear force

Although the considered panel is subjected to an additional shear force of $V_{Ed} = 0.83 \text{ MN}$ which is not induced by the patch loading, this interaction is not taken into account in EN 1993-1-5.
Interaction between transverse force and shear force (cont.)

In stage b) the maximum internal shear force approximates the value of the applied patch load which leads to an asymmetric patch loading condition. For this situation the interaction becomes relevant, however, the average will result in reductions of around 10%, see interaction diagram below.

![Interaction Diagram](image-url)

Interaction equation proposed in [7]:

\[
\left( \frac{V - 0.5 \cdot F}{V_F} \right)^a \cdot \left( \frac{F}{F_F} \right)^b \leq 1.0 \quad \text{with} \quad a = 1.6, \quad b = 1.0
\]

which gives in this example with the patch loading resistance according to [9]:

\[
\left( \frac{0.83}{20.71} \right)^{1.6} + \left( \frac{10.15}{14.77} \right) = 0.01 + 0.67 = 0.68 < 1.0
\]
4.2.2.2 Launching situation “2”

The results for launching situation “2” are summarised below:

\[ F_{cr} = 4.19 \text{ MN} \quad F_y = 27.60 \text{ MN} \]
\[ F_{Rd} = 4.89 \text{ MN} \quad \eta_2 = 1.413 \]
\[ \eta_1 = 0.456 \]

Here, \( \eta_1 = 0.456 < 0.5 \) so that the interaction with bending moment is not considered.
The cross-section cannot be verified for launching situation “2”, see also Figure 4-4 of the summary.

4.2.2.3 Launching situation “3”

The results for launching situation “3” are summarised below:

\[ F_{cr} = 3.05 \text{ MN} \quad F_y = 24.84 \text{ MN} \]
\[ F_{Rd} = 3.96 \text{ MN} \quad \eta_2 = 1.510 \]
\[ \eta_1 = 0.294 \]

Here, \( \eta_1 = 0.294 < 0.5 \) so that the interaction with bending moment is not considered.
The cross-section cannot be verified for launching situation “3”, see also Figure 4-4 of the summary.
4.2.3 Verifications according to Section 10, EN 1993-1-5

4.2.3.1 Webpanel (Launching situation “1” only)

In the following, the patch loading resistance according to Section 10, EN 1993-1-5 is determined. For this example launching situation “1” has been chosen. The resulting stress field acting at the studied panel is shown in Figure 4-3.

Figure 4-9: Stress field acting on the studied panel.

The moment of inertia of the gross steel section at position \( x = 448 \text{ m} \) is \( I_a = 4.524 \cdot 10^8 \text{ cm}^4 \). Shear lag does not have to be considered because \( b_0 < L_e/50 \), cf. EN 1993-1-5, Section 3.1, so that the values according to Table 4-3 have been determined directly.

\[
\begin{align*}
  b_p & := 6.5 \text{ m} \\
  L_3 & := 92 \text{ m} \\
  b_0 & := \frac{b_p}{2} = 3.25 \text{ m} \\
  L_e & := 2 \cdot L_3 = 3.68 \text{ m}
\end{align*}
\]

Table 4-3: Values of the studied panel, see Figure 4-3 (compression is taken as positive).

<table>
<thead>
<tr>
<th>Position [Index]</th>
<th>z-axis [mm]</th>
<th>W [cm³]</th>
<th>( \sigma_{x,Ed} ) [MPa]</th>
<th>( \sigma_{z,Ed} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>“top,w”</td>
<td>-1906.3</td>
<td>-1186573</td>
<td>-183.7</td>
<td>0.0</td>
</tr>
<tr>
<td>“top,st”</td>
<td>880.5</td>
<td>2569047</td>
<td>84.8</td>
<td>72.9</td>
</tr>
<tr>
<td>“st”</td>
<td>1086.5</td>
<td>2081882</td>
<td>104.7</td>
<td>81.7</td>
</tr>
<tr>
<td>“bot,st”</td>
<td>1292.5</td>
<td>1750027</td>
<td>124.5</td>
<td>87.7</td>
</tr>
<tr>
<td>“bot,w”</td>
<td>1834.7</td>
<td>1232880</td>
<td>176.8</td>
<td>98.4</td>
</tr>
</tbody>
</table>
From section 4.2.1 the following dimensions are already known.

Subpanel widths: \( b_{11} = b_1 = 657.9 \text{ mm} \)  
\( b_{12} = 3381.5 \text{ mm} \)

Subpanel widths up to centreline of stiffener: \( b_{11,\text{st}} = 907.9 \text{ mm} \)  
\( b_{12,\text{st}} = 3631.5 \text{ mm} \)

The value of additional shear stress is \( \tau_{\text{Ed}} = 0.6 \text{ MPa} \).

The stress ratios can be determined from Table 4-3:

- Overall: \( \psi_{ov} = -1.04 \)
- Lower subpanel “11”: \( \psi_{11} = 0.70 \)
- Upper subpanel “12”: \( \psi_{12} = -2.17 \)

In the following verification procedure, the subpanels “11” and “12” and the longitudinal stiffener are verified individually.

### 4.2.3.1.1 Verification of subpanel “11”

The subpanel aspect ratio is \( \alpha_{11} = 6.1 \).

#### Determination of \( \alpha_{cr} \)

The determination of the minimum load amplifier \( \alpha_{cr} \) for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- for each component of the stress field (e.g. by a hand-calculation)

\[ \sigma_{E.11} := 189800 \text{ MPa} \left( \frac{t_w}{b_{11}} \right)^2 \]

\[ \sigma_{E.11} = 319.7 \text{ MPa} \]

Elastic critical longitudinal stress acc. to Table 4.1, Section 4.4, EN 1993-1-5

\[ k_{\sigma.x.11} := \begin{cases} 
8.2 & \text{if } 1 \geq \psi_{11} > 0 \\
1.05 + \psi_{11} & \text{if } 0 \geq \psi_{11} > -1 \\
7.81 - 6.29 \psi_{11} + 9.78 \psi_{11}^2 & \text{if } -1 \geq \psi_{11} > -3 \\
5.98 (1 - \psi_{11})^2 & \text{if } -3 \geq \psi_{11} > -6 \\
0 & \text{if } \psi_{11} \leq -6 
\end{cases} \]

\[ k_{\sigma.x.11} = 4.674 \]

\[ \sigma_{x.11} := k_{\sigma.x.11} \cdot \sigma_{E.11} \]

\[ \sigma_{x.11} = 1494.2 \text{ MPa} \]

\[ \alpha_{cr.x.11} := \frac{\sigma_{cr.x.11}}{\sigma_{x.\text{Ed.bot.w}}} \]

\[ \alpha_{cr.x.11} = 8.453 \]
Elastic critical transverse stress acc. to EBPlate

Because of the large subpanel aspect ratio of $\alpha = 6.1$, the determination of the elastic critical transverse stress is not possible by a hand-calculation. Thus, in this case software EBPlate has been used:

$$\sigma_{cr.z.11} := 360.7 \text{MPa}$$

$$\alpha_{cr.z.11} := \frac{\sigma_{cr.z.11}}{\sigma_{z.Ed}} \quad \alpha_{cr.z.11} = 3.67$$


$$k_{\tau.11} := \begin{cases} 5.34 + 4.00 \cdot \left( \frac{b_{11}}{a} \right)^2 & \text{if} \quad \frac{a}{b_{11}} \geq 1 \\ 4.00 + 5.34 \cdot \left( \frac{b_{11}}{a} \right)^2 & \text{if} \quad \frac{a}{b_{11}} < 1 \end{cases}$$

$$k_{\tau.11} = 5.448$$

$$\tau_{cr.11} := k_{\tau.11} \cdot \tau_{E.11} \quad \tau_{cr.11} = 1741.8 \text{MPa}$$

$$\alpha_{cr.\tau.11} := \frac{\tau_{cr.11}}{\tau_{Ed}} \quad \alpha_{cr.\tau.11} = 3162.6$$

Minimum load amplifier acc. to Eq. (10.6), Section 10, EN 1993-1-5

$$\alpha_{cr.11} := \frac{1}{1 + \psi_{11} + \frac{1}{2\alpha_{cr.z.11}} + \sqrt{\left( \frac{1 + \psi_{11}}{4\alpha_{cr.x.11}} + \frac{1}{2\alpha_{cr.z.11}} \right)^2 + \frac{1 - \psi_{11}}{2\alpha_{cr.x.11}^2} + \frac{1}{\alpha_{cr.\tau.11}^2}}}$$

$$\alpha_{cr.11} = 2.638$$

- For the complete stress field (e.g. by using appropriate software)

In case e.g. software EBPlate is used, the minimum load amplifier can be determined in a single step as $\alpha_{cr} = 3.490$.

In the following calculations, the value $\alpha_{cr} = 2.638$ is further used.
Reduction curve for transverse stresses, column-like behaviour and interpolation function

Elastic critical column-buckling stress $\sigma_{cr,c}$

The determination of $\sigma_{cr,c}$ in the transverse direction should take into account the nonlinear stress distribution in the web for which no hand-calculation method exists at the moment. Instead, a common approach is to assume a linearly varying stress distribution for a pin-ended strut which is zero at one end and for which the buckling length $s_k$ can be calculated acc. to DIN 4114 as follows:

$$s_k := b \sqrt{\frac{1 + 0.88 \frac{\sigma_{z,Ed,bot.st}}{\sigma_{z,Ed}} \frac{s_{bot.st}}{s + 2 \cdot t_f}}{1.88}}$$

It should be noted that this simplification may lead to unsafe results because $\sigma_{cr,c}$ is underestimated and in turn the ratio $\sigma_{cr,p}/\sigma_{cr,c}$ is overestimated so that column-like behaviour is not detected correctly. The shorter the loading lengths is, the larger this deviation is.

This reduction curve is used on the right hand-side instead of Eq. (4.13), Section 4.5.4, EN 1993-1-5.
Determination of $\alpha_{\text{ult.k}}$

$$\sigma_{\text{eq.11}} := \sqrt{\sigma_{x, \text{Ed.bot.w}}^2 + \sigma_{z, \text{Ed}}^2 - \sigma_{x, \text{Ed.bot.w}} \sigma_{z, \text{Ed}} + 3 \cdot \tau_{\text{Ed}}}^2$$

$\sigma_{\text{eq.11}} = 153.4 \text{ MPa}$

$$\alpha_{\text{ult.k.11}} := \frac{f_{yw}}{\sqrt{\sigma_{x, \text{Ed.bot.w}}^2 + \sigma_{z, \text{Ed}}^2 - \sigma_{x, \text{Ed.bot.w}} \sigma_{z, \text{Ed}} + 3 \cdot \tau_{\text{Ed}}}^2}$$

$\alpha_{\text{ult.k.11}} = 2.249$

Determination of the slenderness parameter $\bar{\lambda}_p$

$$\bar{\lambda}_{p.11} := \frac{\alpha_{\text{ult.k.11}}}{\alpha_{\text{cr.11}}}$$

$\bar{\lambda}_{p.11} = 0.923$

Determination of the reduction factors

The determination of the reduction factors for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- **using different buckling curves**

  Longitudinal stress acc. to Eq. (4.2), Section 4.4, EN 1993-1-5

  $$\rho_{x.11} := \frac{\bar{\lambda}_{p.11} - 0.055 \cdot (3 + \psi_{11})}{\bar{\lambda}_{p.11}^2}$$

  $\rho_{x.11} = 0.844$

  Check of column-like behaviour in the longitudinal direction

  Column-like buckling in the longitudinal direction for plates under pure bending is only relevant for panel aspect ratios $\alpha < 0.15$. Therefore column-like behaviour is not calculated in detail here.

  Transverse stress acc. to Annex B.1, EN 1993-1-5

  $$\rho_{z.} := \frac{1}{\phi_{p.} + \sqrt{\phi_{p.}^2 - \bar{\lambda}_{p.11}}}$$  with  $\phi_{p.} = 0.983$

  $\rho_{z.} = 0.842$

  Check of column-like behaviour in the transverse direction

  Due to the nonlinear distribution of transverse stresses in the web, the determination of the critical column-buckling stress has been done here most accurately based on the energy method which takes into account the nonlinear stress distribution.

  $$\sigma_{\text{cr.c.11}} = 333.2 \text{ MPa}$$

  $$\frac{\sigma_{\text{cr.z.11}}}{\sigma_{\text{cr.c.11}}} = 1.08$$

  $$\xi_{11} := \frac{\sigma_{\text{cr.z.11}}}{\sigma_{\text{cr.c.11}}} - 1$$

  $$\xi_{11} = 0.08$$
Use of the corrected interpolation function

For comparison, here the use of the newly derived interpolation function acc. to Seitz [40] is used. The calculation gives:

\[ \eta_{\text{Seitz,11}} = 0.598 \]
Here $\xi$ is close to 0.0, i.e. pure column-like behaviour.

\[
\lambda_{\text{c.11}} := \frac{f_{yw}}{\sigma_{\text{cr.c.11}}} \quad \lambda_{\text{c.11}} = 1.051
\]

\[
\Phi_{11} := 0.5 \left[ 1 + 0.21 \left( \lambda_{\text{c.11}} - 0.2 \right) + \lambda_{\text{c.11}}^2 \right] \quad \Phi_{11} = 1.142
\]

\[
\chi_{\text{c.11}} := \frac{1}{\Phi_{11} + \sqrt{\Phi_{11}^2 - \lambda_{\text{c.11}}^2}} \quad \chi_{\text{c.11}} = 0.630
\]

\[
\rho_{\text{c.z.11}} := \left( \rho_{\text{z.11}} - \chi_{\text{c.11}} \right) \xi_{11} \left( 2 - \xi_{11} \right) + \chi_{\text{c.11}} \quad \rho_{\text{c.z.11}} = 0.663
\]

Shear stress acc. to Table 5.1, section 5.3, EN 1993-1-5

\[
\chi_{\text{w.11}} := \begin{cases} 
1.0 & \text{if } \lambda_{\text{p.11}} < 0.83 \\
0.83 & \text{if } \lambda_{\text{p.11}} \geq 0.83 \\
\lambda_{\text{p.11}} & \text{if } \lambda_{\text{p.11}} = 0.83
\end{cases} \quad \chi_{\text{w.11}} = 0.899
\]

- **using a single buckling curve**

  According to Table B.1, Annex B.1, EN 1993-1-5

\[
\lambda_{\text{p}0} := 0.80 \quad \alpha_p := 0.34
\]

\[
\Phi_{11} := 0.5 \left[ 1 + \alpha_p \left( \lambda_{\text{p.11}} - \lambda_{\text{p}0} \right) + \lambda_{\text{p.11}} \right] \quad \Phi_{11} = 1.142
\]

\[
\rho_{11} := \frac{1}{\Phi_{11} + \sqrt{\Phi_{11}^2 - \lambda_{\text{p.11}}^2}} \quad \rho_{11} = 0.842
\]

\[
\rho_{\text{c.11}} := \chi_{\text{c.11}} + \left( \rho_{11} - \chi_{\text{c.11}} \right) \xi_{11} \quad \rho_{\text{c.11}} = 0.651
\]

**Determination of the patch loading resistance**

- **using different buckling curves**

\[
\eta_{\text{diff.11}} := \left( \frac{\sigma_{\text{x.Ed.bot.w}}}{\rho_{\text{x}11} \frac{f_{yw}}{\gamma_{\text{M1}}}} \right)^2 + \left( \frac{\sigma_{\text{z.Ed}}}{\rho_{\text{c.z.11}} \frac{f_{yw}}{\gamma_{\text{M1}}}} \right)^2 - \left( \frac{\sigma_{\text{x.Ed.bot.w}}}{\rho_{\text{x}.1} \frac{f_{yw}}{\gamma_{\text{M1}}}} \right)^2 - \left( \frac{\sigma_{\text{z.Ed}}}{\rho_{\text{c.z.11}} \frac{f_{yw}}{\gamma_{\text{M1}}}} \right)^2 + 3 \cdot \left( \frac{\tau_{\text{Ed}}}{\chi_{\text{w.11}} \frac{f_{yw}}{\gamma_{\text{M1}}}} \right)^2
\]

\[
\eta_{\text{diff.11}} = 0.595
\]
using a single buckling curve
\[
\alpha_{\text{Rd.11}} = \frac{\rho_{e.11} \cdot \alpha_{\text{ult.k.11}}}{\gamma_{\text{M1}}} = 1.332 \quad \eta_{\text{single.11}} = \frac{1}{\alpha_{\text{Rd.11}}} = 0.751
\]

4.2.3.1.2 Verification of subpanel “12”
The subpanel aspect ratio is \( \alpha_{12} = 1.18 \).
In the following only the results of the calculation are shown. For further information the reader is referred to the previous section “verification of subpanel “11”.

Determination of \( \alpha_{\text{cr}} \)
The determination of the minimum load amplifier \( \alpha_{\text{cr}} \) for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- for each component of the stress field (e.g. by a hand-calculation)
  Elastic critical longitudinal stress acc. to Table 4.1, Section 4.4, EN 1993-1-5
  \[
k_{\sigma.x.12} = 59.907 \quad \sigma_{\text{cr.x.12}} = 724.9 \text{ MPa} \quad \alpha_{\text{cr.x.12}} = 8.546
  \]
  Elastic critical transverse stress with \( k = 3.8 \) acc. to Table 8.12 [38]
  \[
  \sigma_{\text{cr.z.12}} := k_{12} \cdot \sigma_{E.12} \cdot \frac{1}{s_{\text{top.st}}} \quad \sigma_{\text{cr.z.12}} = 58.05 \text{ MPa}
  \]
  \[
  \alpha_{\text{cr.z.12}} := \frac{\sigma_{\text{cr.z.12}}}{\sigma_{Z.\text{Ed.top.st}}} \quad \alpha_{\text{cr.z.12}} = 0.60
  \]
  \[
k_{\tau.12} = 8.199 \quad \tau_{\text{cr.12}} = 99.2 \text{ MPa} \quad \alpha_{\text{cr.\tau.12}} = 180.137
  \]
  Minimum load amplifier acc. to Eq. (10.6), Section 10, EN 1993-1-5
  \[
  \alpha_{\text{cr.12}} = 0.616
  \]

- for the complete stress field (e.g. by using appropriate software)
  In case e.g. software EBPlate is used, the minimum load amplifier can be determined in a single step as \( \alpha_{\text{cr}} = 0.889 \).

In the following calculations, the value \( \alpha_{\text{cr}} = 0.616 \) is further used.

Determination of \( \alpha_{\text{ult.k}} \)
\[
\sigma_{\text{eq.12}} = 91.3 \text{ MPa} \quad \alpha_{\text{ult.k.12}} = 3.735
\]
Elastic critical column-buckling stress $\sigma_{cr,c}$

The determination of $\sigma_{cr,c}$ in the transverse direction should take into account the nonlinear stress distribution in the web for which no hand-calculation method exists at the moment. Instead, a common approach is to assume a linearly varying stress distribution for a pin-ended strut which is zero at one end and which can be calculated acc. to DIN 4114 as follows:

$$\sigma_{cr,c} = 1.88 \sigma_E$$

It should be noted that this simplification may lead to unsafe results because $\sigma_{cr,c}$ is underestimated and in turn the ratio $\sigma_{cr,p}/\sigma_{cr,c}$ is overestimated so that column-like behaviour is not detected correctly. The shorter the loading lengths is, the larger this deviation is.

Reduction curve for transverse stresses, column-like behaviour and interpolation function

In the COMBRI project it has been shown that the interpolation function acc. to Eq. (4.13), Section 4.5.4, EN 1993-1-5, which takes into account column-like buckling, is not appropriate for patch loading stresses in the transverse direction. Moreover, the ratio of $\sigma_{cr,p}/\sigma_{cr,c}$, for which column-like buckling needs to be considered, should be 2.7 (and not 2.0 as given in Eq. (4.13)).

Basically, either a new interpolation function is required or a reduction curve should be used which can be used with the existing interpolation function. A new interpolation function has been derived e.g. by Seitz [40] and this method is introduced in the COMBRI Final report [7]. However, e.g. for the new German DIN-Fachbericht 103 [12], it has been decided to use the reduction curve of Annex B, EN 1993-1-5, for transverse stresses because this approach complies well with current Eurocode rules and could be easier implemented. Thus, for a welded girder Table B.1, Annex B.1, EN 1993-1-5 gives:

$$\lambda_{p0} := 0.80 \quad \alpha_p := 0.34 \quad \phi_p := 0.5 \left[1 + \alpha_p \left(\lambda_p - \lambda_{p0}\right) + \lambda_p \right]$$

which is to be used in

$$\rho := \frac{1}{\phi_p + \sqrt{\phi_p^2 - \lambda_p}}$$

This reduction curve is used on the right hand-side instead of Eq. (4.13), Section 4.5.4, EN 1993-1-5.
Determination of the slenderness parameter $\bar{\lambda}_p$

$\lambda_{p.12} = 2.462$

Determination of the reduction factors

The determination of the reduction factors for the design loads to reach the elastic critical load of the plate under the complete stress field can be either determined

- **using different buckling curves**
  
  Longitudinal stress acc. to Eq. (4.2), Section 4.4, EN 1993-1-5

  $\rho_{x.12} = 0.399$

  Check of column-like behaviour in the longitudinal direction

  Column-like buckling in the x-direction for plates under pure bending is only relevant for panel aspect ratios $a < 0.15$. Therefore column-like behaviour is not calculated in detail here.

  Transverse stress acc. to Annex B.1, EN 1993-1-5

  $\rho_{z.12} = 0.305$

  Check of column-like behaviour in the transverse direction

  Due to the nonlinear distribution of transverse stresses in the web, the determination of the critical column-buckling stress has been done here most accurately based on the energy method which takes into account the nonlinear stress distribution.

  \[
  \sigma_{\text{cr.c.12}} = 27.86 \text{ MPa} \quad \frac{\sigma_{\text{cr.z.12}}}{\sigma_{\text{cr.c.12}}} = 2.08 \quad \xi_{12} = 1.08
  \]

  Because $\xi > 1.0$, column-like behaviour does not have to be considered according to the code calculation. However, as shown by Seitz, the interpolation function is not safe-sided.

  Shear stress acc. to Table 5.1, section 5.3, EN 1993-1-5

  $\chi_{w.12} = 0.337$

- **using a single buckling curve**

  According to Table B.1, Annex B.1, EN 1993-1-5

  $\phi_{p.12} = 2.014 \quad \rho_{12} = 0.305$
Use of the corrected interpolation function

For comparison, here the use of the newly derived interpolation function acc. to Seitz [40] is used. The calculation gives:

\[ \eta_{Seitz,12} = 0.915 \]

DIN 18800 Part 3, Element (801)

Additional verification for plates with transverse stresses \( \sigma_y \)

For longitudinally stiffened plates with transverse stresses \( \sigma_y \), the longitudinal stiffeners should be verified using a second order elastic method of analysis based on the following assumptions:

- The considered longitudinal stiffener is treated as simply supported member with an initial sinusoidal imperfection \( w_0 \) equal to \( bik/250 \), where \( bik \) is the smallest width of the adjacent subpanels.
- The adjacent longitudinal boundaries are straight, simply supported and rigid.
- Hinged boundaries are usually assumed between the subpanels and the longitudinal stiffener. If the subpanels are considered to be rigidly connected to the longitudinal stiffener, the loading on the subpanels when they act together with the longitudinal stiffener should be taken into account.
Determination of the patch loading resistance

- using different buckling curves
  \[ \eta_{\text{diff.12}} = 0.897 \]
- using a single buckling curve
  \[ \alpha_{Rd.12} = 1.037 \quad \eta_{\text{single.12}} = \frac{1}{\alpha_{Rd.12}} = 0.965 \]

4.2.3.1.3 Verification of the longitudinal stiffener

Generally, the resistance of the longitudinal stiffener can be either calculated according to second-order theory or with the help of a buckling coefficient and a reduction curve. In the latter case, it is of utmost importance to determine the buckling coefficient correctly to which advanced analysis software such as EBPlate [13] is able to highly contribute nowadays. However, in this example, a buckling coefficient for the longitudinal stiffener could not be explicitly determined because the stiffener deflection was always found in combination with local subpanel eigenmodes of higher order. Thus, a verification on the basis of second-order theory is followed here.

In contrast to EN 1993-1-5, in German standard DIN 18800 Part 3, Element (801) [10] rules are provided for the verification of longitudinal stiffeners in panels which are subjected to transverse loading. These complementary rules will be used to determine the stiffener resistance in this example.

Determination of the initial stiffener imperfection

\[ w_0 := \min \left( \frac{b_{11,\text{st}}}{250}, \frac{b_{12,\text{st}}}{250} \right) \quad w_0 = 3.63 \text{ mm} \]

Normal force acting on the stiffener

The stiffener force is calculated with the direct stress \( \sigma_{x,\text{st}} \) and the gross cross-section of the stiffener \( A_{\text{st}} \). The gross cross-section of the stiffener is determined according to EN 1993-1-5, Figure A.1:

\[
\begin{align*}
b_{11,\text{part}} &:= \frac{3 - \psi_{11}}{5 - \psi_{11}} \cdot b_{11} & b_{11,\text{part}} &= 351.6 \text{ mm} \\
\text{b}_c &:= b_{12} - z_{\text{top,w}} & \text{b}_c &= 1475.2 \text{ mm} \\
\text{b}_{c,\text{part}} &:= 0.4 \cdot \text{b}_c & \text{b}_{c,\text{part}} &= 590.1 \text{ mm} \\
A_{\text{st}} &:= A_{\text{sl.1}} + t_w \cdot \left( b_{11,\text{part}} + b_{\text{st}} + b_{c,\text{part}} \right) & A_{\text{st}} &= 575.84 \text{ cm}^2 \\
N_{x,\text{st}} &:= A_{\text{st}} \cdot \sigma_{x,\text{Ed.st}} & N_{x,\text{st}} &= 6.03 \text{ MN}
\end{align*}
\]
Transverse loading acting on the stiffener

The transverse stress $\sigma_{z,Ed, st}$ acts on the stiffener along the loading length $s_{st}$ only. It can be recalculated to an equivalent transverse stress $\sigma_{z,Ed,st,eq}$ which acts on the whole stiffener length and a factor $q$ as follows:

$$\sigma_{z,Ed,st,eq} := \sigma_{z,Ed,st} \left( \frac{s_{st}}{a} + \frac{1}{\pi} \cdot \sin \left( \frac{\pi \cdot s_{st}}{a} \right) \right)$$  \hspace{1cm} \sigma_{z,Ed,st,eq} = 80.4 \text{ MPa}

$$q := \sigma_{z,Ed,st,eq} \cdot t_w \left( \frac{1}{b_{11,st}} + \frac{1}{b_{12,st}} \right) + \left( \frac{\pi}{a} \right)^2 \cdot N_{x, st}$$  \hspace{1cm} q = 6.706 \text{ MPa}

Second-order theory verification of the stiffener

With the critical force $N_{x,cr}$ of the stiffener and the influence of the elastic foundation $c_f$, the elastic deformation $w_{el,II}$ and the bending moment $M_{II}$ according to second-order theory can be determined and with the maximum distance $z_{st,1} = 393.3$ mm to the outer fibre and the moment of inertia $I_{sl,1} = 154894$ cm$^4$ of the longitudinal stiffener, the verification can be done.

$$N_{x,cr} := \frac{2 \cdot 210000 \text{ MPa} \cdot I_{sl,1}}{a^2}$$  \hspace{1cm} N_{x,cr} = 200.65 \text{ MN}

$$c_f := \left( \frac{\pi}{a} \right)^2 \cdot N_{x,cr}$$  \hspace{1cm} c_f = 123.8 \text{ MPa}

$$w_{el,II} := \frac{q \cdot w_0}{c_f - q}$$  \hspace{1cm} w_{el,II} = 0.21 \text{ mm}

$$M_{II} := N_{x,cr} \cdot w_{el,II}$$  \hspace{1cm} M_{II} = 0.042 \text{ MNm}

$$\sigma := \frac{M_{II}}{I_{sl,1}} \cdot z_{st,1}$$  \hspace{1cm} \sigma = 115.3 \text{ MPa}

$$\eta := \frac{\sigma}{f_{yw}}$$  \hspace{1cm} \eta = 0.37
4.2.3.2 Bottom plate

4.2.3.2.1 Application of Sections 3, 9 and 10, EN 1993-1-5 to launching situation “1”

In the following, the ultimate resistance of the longitudinal stiffened bottom plate is determined acc. to Sections 3, 9 and 10 of EN 1993-1-5. Resistance is calculated for the erection phase, taking into account the horizontal component of the patch load, cp. Section 4.2.2.1.

Specific cross-section parameters of considered bottom plate (cp. Section 2.2.3):
- Thickness of bottom plate: $t_f = 75$ mm
- Number of stiffeners: $n_{st} = 6$ (Geometry cp. Figure 2-9)
- Yield strength of steel: $f_{yf} = 325$ N/mm²

Determination of the stress field components acc. to elastic bending theory:
- Bending moment in cross-section: $M_{Ed} = 217.93$ MNm
- Section modulus at bottom plate: $W_{bot} = 1150500$ cm³
- Resulting axial stress in bottom plate: $\sigma_{x,Ed} = 189.4$ N/mm²
- Horizontal component of patch load: $F_{Ed,bot} = 5.75$ MN
- Loading length: $s_s = 3.0$ m
- Resulting transverse stress: $\sigma_{y,Ed} = 25.6$ N/mm²

Remark: The above used „section modulus at bottom plate“y takes conservatively into account shear lag effect. In disregard to the consistency this fact is neglected in the following to provide an entire design example.

Figure 4-10 shows the idealized assumption of the stress distribution in the bottom plate acc. to the elastic bending theory. The next required calculation steps are:

1. Correction of the axial stress distribution due to shear lag effect, acc. to EN 1993-1-5 Section 3, if required.
2. Verification of the subpanel resistance acc. to EN 1993-1-5, Section 10.
3. Verification if the longitudinal stiffeners fulfil the minimum requirements to act as fix supports for the subpanels acc. to EN 1993-1-5, Section 9.

![Stress field in bottom plate acc. to elastic bending theory (left hand side) and resulting conservative assumption for stress field in subpanel (right hand side).](image)
EN 1993-1-5, 3.1, General
(1) Shear lag in flanges may be neglected if \( b_0 < L_e/50 \) where \( b_0 \) is taken as the flange outstand or half the width of an internal element and \( L_e \) is the length between points of zero bending moment, see 3.2.1(2).

EN 1993-1-5, 3.2.1, Effective width
(2) Provided adjacent spans do not differ more than 50% and any cantilever span is not larger than half the adjacent span the effective lengths \( L_e \) may be determined from Figure 3.1. For all other cases \( L_e \) should be taken as the distance between adjacent points of zero bending moment.

\[
\begin{align*}
\beta_1: L_e &= 0.85 L_1 \\
\beta_2: L_e &= 0.25 (L_1 + L_2) \\
\beta_3: L_e &= 0.70 L_2 \\
\beta_4: L_e &= 2 L_3
\end{align*}
\]

Figure 3.1: Effective length \( L_e \) for continuous beam and distribution of effective width

<table>
<thead>
<tr>
<th>( \kappa )</th>
<th>Verification</th>
<th>( \beta )-value</th>
<th>( \beta )-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \kappa \leq 0.02 )</td>
<td>sagging bending</td>
<td>( \beta = \beta_1 = \frac{1}{1 + 6.4 \kappa^2} )</td>
<td>( \beta = 1.0 )</td>
</tr>
<tr>
<td>( 0.02 &lt; \kappa \leq 0.70 )</td>
<td>hogging bending</td>
<td>( \beta = \beta_2 = \frac{1}{1 + 6.0 \left( \kappa - \frac{1}{2500 \kappa} \right) + 1.6 \kappa^2} )</td>
<td>( \beta = \beta_2 )</td>
</tr>
<tr>
<td>( \kappa &gt; 0.70 )</td>
<td>sagging bending</td>
<td>( \beta = \beta_1 = \frac{1}{5.9 \kappa} )</td>
<td>( \beta = \beta_2 = \frac{1}{8.6 \kappa} )</td>
</tr>
<tr>
<td>( \kappa )</td>
<td>hogging bending</td>
<td>( \beta = \beta_2 )</td>
<td>( \beta = \beta_2 ) at support and at the end</td>
</tr>
</tbody>
</table>

\( \kappa = \alpha_0 b_0 / L_e \) with \( \alpha_0 = \sqrt{1 + \frac{A_{st}}{b_0 \beta}} \)

in which \( A_{st} \) is the area of all longitudinal stiffeners within the width \( b_0 \) and other symbols are as defined in Figure 3.1 and Figure 3.2.
Verification if shear lag effect has to be taken into account (EN 1993-1-5, 3.1):

- Effective length: \( L_e = 120 \) m (conservative assumption)
- Considered width: \( b_0 = \frac{b_p}{2} = 3.25 \) m

\[ b_0 < \frac{L_e}{50} \text{ requirement not fulfilled! Shear lag effect has to be taken into account.} \]

Correction of axial stress distribution due to shear lag effect:

Gross area of longitudinal stiffeners:

\[ A_{sl} := n_{st} \cdot t_{st} \left( b_2 + 2 \cdot b_3 \right) = 0.109 \text{ m}^2 \]

Shear lag parameters:

\[ \alpha_0 := \frac{A_{sl}}{b_p \cdot t_f} = 1.106 \]

\[ \kappa := \frac{b_0}{L_e} = 0.03 \]

\[ \beta := \begin{cases} 1 & \text{if } \kappa \leq 0.02 \\ \frac{1}{1 + 6 \left( \kappa - \frac{1}{2500 \cdot \kappa} \right) + 1.6 \cdot \kappa^2} & \text{if } 0.02 < \kappa \leq 0.7 \\ \frac{1}{8.6 \cdot \kappa} & \text{otherwise} \end{cases} \]

\[ \beta = 0.908 \]

The axial stress \( \sigma_{1.x,Ed} \) in the bottom plate (at location of web) is that value, which delivers a mean stress value \( \sigma_{x,mean} \) in the bottom plate, which is equal to the axial design stress \( \sigma_{x,Ed} \) derived by the elastic bending theory. The mean stress \( \sigma_{x,mean} \) in the bottom plate is given by the integral

\[ \sigma_{x,mean} = \frac{1}{b_0} \int_0^{b_0} \sigma(y) \, dy \]

which results in

\[ \sigma_{x,mean} := \sigma_{1.x,Ed} - \frac{4}{5} \left( \sigma_{1.x,Ed} - \sigma_{2.x,Ed} \right) \]

with the axial stress \( \sigma_{2.x,Ed} \) in the middle of the bottom plate

\[ \sigma_{2.x,Ed} := 1.25 \cdot (\beta - 0.2) \cdot \sigma_{1.x,Ed} \text{ if } \beta > 0.2 \]

the mean stress \( \sigma_{x,mean} \) becomes equal to the axial stress \( \sigma_{x,Ed} \) for the following value of \( \sigma_{1.x,Ed} \):

\[ \sigma_{1.x,Ed} := \frac{\sigma_{x,Ed}}{\beta} = 208.6 \cdot \text{N} \cdot \text{mm}^{-2} \]

Cross-check:

\[ \sigma_{x,mean} := \sigma_{1.x,Ed} - \frac{4}{5} \left( \sigma_{1.x,Ed} - \sigma_{2.x,Ed} \right) = 189.4 \cdot \text{N} \cdot \text{mm}^{-2} \]

\[ \sigma_{x,Ed} = 189.4 \cdot \text{N} \cdot \text{mm}^{-2} \]
EN 1993-1-5, 3.2.2, Stress distribution due to shear lag

(1) The distribution of longitudinal stresses across the flange plate due to shear lag should be obtained from Figure 3.3.

Figure 3.3: Distribution of stresses due to shear lag

EN 1993-1-5, 10, Reduced stress method

(4) In determining $\alpha_{ult,k}$ the yield criterion may be used for resistance:

$$\frac{1}{\alpha_{ult,k}^2} = \left(\frac{\sigma_{x,Ed}}{f_y}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y}\right)\left(\frac{\sigma_{z,Ed}}{f_y}\right) + \left(\frac{\tau_{Ed}}{f_y}\right)^2$$

(10.3)

where $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and $\tau_{Ed}$ are the components of the stress field in the ultimate limit state.

NOTE: By using the equation (10.3) it is assumed that the resistance is reached when yielding occurs without plate buckling.

EN 1993-1-5, 4.4, Plate elements without longitudinal stiffeners

(4) $k_\alpha$ is the buckling factor corresponding to the stress ratio $\psi$ and boundary conditions. For long plates $k_\alpha$ is given in Table 4.1 or Table 4.2 as appropriate;

Table 4.1: Internal compression elements

<table>
<thead>
<tr>
<th>Stress distribution (compression positive)</th>
<th>Effective(^4) width $b_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Stress Diagram 1]</td>
<td>$\psi = 1$:</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = \rho \overline{b}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c1} = 0.5 b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c2} = 0.5 b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$1 &gt; \psi &gt; 0$:</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = \rho \overline{b}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c1} = 2 \frac{1}{\psi} b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c2} = b_{eff} - b_{c1}$</td>
</tr>
<tr>
<td></td>
<td>$\psi &lt; 0$:</td>
</tr>
<tr>
<td></td>
<td>$b_{eff} = \rho \overline{b}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c1} = 0.4 b_{eff}$</td>
</tr>
<tr>
<td></td>
<td>$b_{c2} = 0.6 b_{eff}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\psi = \sigma_y / \sigma_t$</th>
<th>1</th>
<th>$1 &gt; \psi &gt; 0$</th>
<th>0</th>
<th>$0 &gt; \psi &gt; -1$</th>
<th>-1</th>
<th>$-1 &gt; \psi &gt; -3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckling factor $k_\alpha$</td>
<td>4.0</td>
<td>8.2 / $(1.05 + \psi)$</td>
<td>7.81</td>
<td>7.81 - 6.29$\psi$ + 9.78$\psi^2$</td>
<td>23.9</td>
<td>5.98 $(1 - \psi)^2$</td>
</tr>
</tbody>
</table>
Axial stress distribution in bottom flange (half bottom plate):

\[
\sigma_x(y) := \begin{cases} 
\sigma_{2.x.Ed} + \left(\sigma_{1.x.Ed} - \sigma_{2.x.Ed}\right) \left(1 - \frac{y}{b_0}\right)^4 & \text{if } \beta > 0.2 \\
\sigma_{1.x.Ed} \left(1 - \frac{y}{5 \cdot \beta \cdot b_0}\right)^4 & \text{otherwise}
\end{cases}
\]

Figure 4-11: Axial stress distribution in bottom flange.

Buckling verification of subpanels:
- Minimum load amplifier \(\alpha_{ult,k}\) of subpanel:

\[
\alpha_{ult.k} := \frac{f_y f}{\sqrt{\sigma_{1.x.Ed}^2 + \sigma_{y.Ed}^2 - (\sigma_{1.x.Ed} \cdot \sigma_{y.Ed})}} = 1.649
\]

Stress distribution in subpanel:

\[
\sigma_{1.sub} := \sigma_{1.x.Ed} = 208.6 \cdot \text{N} \cdot \text{mm}^{-2}
\]

\[
\sigma_{2.sub} := \sigma_x(b_{sub}) = 196.7 \cdot \text{N} \cdot \text{mm}^{-2}
\]

\[
\psi := \frac{\sigma_{2.sub}}{\sigma_{1.sub}} = 0.943
\]

Buckling factor for internal compression elements:

\[
k_\sigma := \frac{8.2}{1.05 + \psi} = 4.115
\]

Critical local buckling stress in x-direction:

\[
\sigma_{cr,p.sub.x} := \frac{\pi^2 E \cdot f_y^2}{12(1 - \nu^2) b_{sub}^2} \cdot k_\sigma = 16701.1 \cdot \text{N} \cdot \text{mm}^{-2}
\]
Remark from author
The exact elastic critical plate buckling stress $\sigma_{cr,x}$ for a plate with an aspect ratio $\alpha \leq 1$ results from:

$$\sigma_{cr,x} = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot (1 - \mu^2) \cdot b^2} \cdot \left[ \alpha + \frac{1}{\alpha} \right]^2$$

where $a$ = length of the panel in direction of load
$b$ = width of panel (transvers to load-direction)
$\alpha = a/b$

EN 1993-1-5, 10, Reduced stress method

(6) Where $\alpha_{cr}$ values for the complete stress field are not available and only $\alpha_{cr,i}$ values for the various components of the stress field $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and $\tau_{Ed}$ can be used, the $\alpha_{cr}$ value may be determined from:

$$\frac{1}{\alpha_{cr}} = \frac{1 + \psi_x}{4\alpha_{cr,x}} + \frac{1 + \psi_z}{4\alpha_{cr,z}} + \left( \frac{1 + \psi_x}{4\alpha_{cr,x}} + \frac{1 + \psi_z}{4\alpha_{cr,z}} \right)^2 + \frac{1 - \psi_x}{2\alpha_{cr,x}} + \frac{1 - \psi_x}{2\alpha_{cr,z}} + \frac{1}{2\alpha_{cr,z}}$$

(10.6)

$$\alpha_{cr,x} = \frac{\sigma_{cr,x}}{\sigma_{x,Ed}}$$

$$\alpha_{cr,z} = \frac{\sigma_{cr,z}}{\sigma_{z,Ed}}$$

$$\alpha_{cr,\tau} = \frac{\tau_{cr}}{\tau_{Ed}}$$

and $\sigma_{cr,x}$, $\sigma_{cr,z}$, $\tau_{cr}$, $\psi_x$, $\psi_z$ are determined from Section 4 and Section 6.

EN 1993-1-5, 10, Reduced stress method

(3) The plate slenderness $\bar{\lambda}_{q}$ should be taken from

$$\bar{\lambda} = \sqrt{\frac{\alpha_{alt,k}}{\alpha_{cr}}}$$

(10.2)

where $\alpha_{cr}$ is the minimum load amplifier for the design loads to reach the elastic critical load of the plate under the complete stress field, see (6)

NOTE1: For calculating $\alpha_{cr}$ for the complete stress field, the stiffened plate may be modelled using the rules in section 4 and 5 without reduction of the second moment of area of longitudinal stiffeners as specified in 5.3(4).

NOTE2: When $\alpha_{cr}$ cannot be determined for the panel and its subpanels as a whole, separate checks for the subpanel and the full panel may be applied.
Minimum load amplifier $\alpha_{\text{cr,sub},x}$:

$$\alpha_{\text{cr,sub},x} := \frac{\sigma_{\text{cr,p.sub}.x}}{\sigma_{\text{l.sub}}} = 80.063$$

Critical local buckling stress in y-direction:

$$\sigma_{\text{cr,p.sub}.y} := \frac{\pi^2 \cdot E \cdot t_f^2}{12 \cdot (1 - \nu^2) \cdot a^2 \cdot \left( \frac{b_{\text{sub}}}{a} + \frac{a}{b_{\text{sub}}} \right)^2} = 4193.6 \cdot \text{N} \cdot \text{mm}^{-2}$$

Minimum load amplifier $\alpha_{\text{cr,sub},y}$:

$$\alpha_{\text{cr,sub},y} := \frac{\sigma_{\text{cr,p.sub}.y}}{\sigma_{\text{y.Ed}}} = 164.041$$

Resulting minimum load amplifier $\alpha_{\text{cr}}$ for local subpanel buckling (with $\psi \approx 1$):

$$\alpha_{\text{cr,sub}} := \frac{1}{\alpha_{\text{cr,sub},x} + \frac{1}{\alpha_{\text{cr,sub},y}}} = 53.803$$

- Interaction between plate and column-type buckling behaviour:

  The buckling behaviour in x-direction of the subpanel is pure plate-like, due to its aspect ratio. The buckling behaviour in y-direction of the subpanel has to be checked.

  Critical local column buckling stress in y-direction:

  $$\sigma_{\text{cr,c.sub}.y} := \frac{\pi^2 \cdot E \cdot t_f^2}{12 \cdot b_{\text{sub}}^2} = 3693.8 \cdot \text{N} \cdot \text{mm}^{-2}$$

  Weighting factor $\xi$:

  $$\xi := \frac{\sigma_{\text{cr,p.sub}.y}}{\sigma_{\text{cr,c.sub}.y}} - 1$$

  $$\xi = 0.135$$

- Reduction due to local buckling of subpanels:

  Global plate slenderness of subpanel:

  $$\lambda := \sqrt{\frac{\alpha_{\text{ult.k}}}{\alpha_{\text{cr,sub}}}} = 0.175$$

  Reduction factor for internal compression elements:

  $$\rho_{\text{sub}}(\lambda, \psi) := \begin{cases} 1 & \text{if } \lambda < 0.673 \\ \frac{\lambda - 0.055 \cdot (3 + \psi)}{\lambda^2} & \text{otherwise} \end{cases}$$
EN 1993-1-5, 10, Reduced stress method

(5) The reduction factor \( \rho \) may be determined using either of the following methods:

a) the minimum value of the following reduction factors:

\[
\begin{align*}
\rho_x & \quad \text{for longitudinal stresses from 4.5.4(1) taking into account column-like behaviour where relevant;}
\rho_z & \quad \text{for transverse stresses from 4.5.4(1) taking into account column-like behaviour where relevant;}
\chi_w & \quad \text{for shear stresses from 5.2(1);}
\end{align*}
\]

each calculated for the slenderness \( \bar{\lambda}_p \) according to equation (10.2).

b) a value interpolated between the values of \( \rho_x \), \( \rho_z \) and \( \chi_w \) as determined in a) by using the formula for \( a_{ult, k} \) as interpolation function

\[
\text{NOTE: This method leads to the verification format:}
\]

\[
\left( \frac{\sigma_{x,Ed}}{\rho_x f_y / \gamma_{M1}} \right)^2 + \left( \frac{\sigma_{z,Ed}}{\rho_z f_y / \gamma_{M1}} \right)^2 - \left( \frac{\sigma_{z,Ed}}{\rho_z f_y / \gamma_{M1}} \right) \left( \frac{\sigma_{x,Ed}}{\rho_x f_y / \gamma_{M1}} \right) + 3 \left( \frac{\tau_{Ed}}{\chi_w f_y / \gamma_{M1}} \right)^2 \leq 1
\]

(10.5)

\[
\text{NOTE 1: Since verification formulae (10.3), (10.4) and (10.5) include an interaction between shear force, bending moment, axial force and transverse force, section 7 should not be applied.}
\]

\[
\text{NOTE 2: The National Annex may give further information on the use of equations (10.4) and (10.5). In case of panels with tension and compression it is recommended to apply equations (10.4) and (10.5) only for the compressive parts.}
\]

EN 1993-1-5, 9.2.1, Minimum requirements for transverse stiffeners

(1) In order to provide a rigid support for a plate with or without longitudinal stiffeners, intermediate transverse stiffeners should satisfy the criteria given below.

(2) The transverse stiffener should be treated as a simply supported member subject to lateral loading with an initial sinusoidal imperfection \( w_0 = s/300 \), where \( s \) is the smallest of \( a_1 \), \( a_2 \) or \( b \), see Figure 9.2, where \( a_1 \) and \( a_2 \) are the lengths of the panels adjacent to the transverse stiffener under consideration and \( b \) is the height between the centroids of the flanges or span of the transverse stiffener. Eccentricities should be accounted for.

\[
\begin{align*}
\text{Figure 9.2: Transverse stiffener}
\end{align*}
\]

(3) The transverse stiffener should carry the deviation forces from the adjacent compressed panels under the assumption that both adjacent transverse stiffeners are rigid and straight together with any external load and axial force according to the NOTE: to 9.3.3(3). The compressed panels and the longitudinal stiffeners are considered to be simply supported at the transverse stiffeners.

(4) It should be verified that using a second order elastic method analysis both the following criteria are satisfied at the ultimate limit state:
Reduction factor for column buckling:

\[
\chi_{\text{sub}}(\phi) := \begin{cases} 
1 & \text{if } \lambda < 0.2 \\
\frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} & \text{otherwise}
\end{cases}
\]

\[
\phi := 0.5 \left[ 1 + 0.34 \cdot (\lambda - 0.2) + \lambda^2 \right] = 0.511
\]

\[
\chi_{\text{sub}} := \chi_{\text{sub}}(\phi) = 1
\]

Resulting reduction factors

\[
\rho_{\text{sub}, x} := \rho_{\text{sub}}(\lambda, \psi) = 1
\]

\[
\rho_{\text{sub}, y} := \rho_{\text{sub}}(\lambda, 1.0) = 1
\]

\[
\rho_{\text{c}, \text{sub}, y} := (\rho_{\text{sub}, y} - \chi_{\text{sub}}) \xi (2 - \xi) + \chi_{\text{sub}} = 1
\]

Subpanel verification:

\[
\eta := \left( \frac{\sigma_{1, \text{sub}}}{\rho_{\text{sub}, x} \frac{f_{yf}}{\gamma_{M1}}} \right)^2 + \left( \frac{\sigma_{y, \text{Ed}}}{\rho_{\text{c}, \text{sub}, y} \frac{f_{yf}}{\gamma_{M1}}} \right)^2 - \left( \frac{\sigma_{1, \text{sub}}}{\rho_{\text{sub}, x} \frac{f_{yf}}{\gamma_{M1}}} \right) \left( \frac{\sigma_{y, \text{Ed}}}{\rho_{\text{c}, \text{sub}, y} \frac{f_{yf}}{\gamma_{M1}}} \right) = 0.445
\]

Global buckling verification of longitudinal stiffener:

- General: Local subpanel buckling is governing, if the longitudinal stiffeners satisfy the criteria of a rigid support. Therefore the minimum requirements for transverse stiffeners given in EN 1993-1-5 Section 9.2 are applied to the longitudinal stiffeners. As the bottom plate is designed for the Ultimate Limit State, it can be expected that for the launching loads the stiffeners fulfill the simplified conservative check acc. to Section 9. In case that this handcalculation check fails, an exact buckling check, using an appropriate computer program (e.g. EBPlate), has to be performed.

- Minimum requirements for the longitudinal stiffener:

  Maximum stress in stiffener:

  \[
  \sigma_{\text{max}} \leq \frac{f_{yf}}{\gamma_{M1}}
  \]

  Maximum additional deflection:

  \[
  w := \frac{a}{300}
  \]

- Simplified second order elastic analysis of longitudinal stiffener:

  Discription of used parameters:

  \[
  \begin{align*}
  t, t_{\text{st}}, b_1, b_2, h_x, t, & \text{ (dimensions of the stiffener)} \\
  t_{\text{st}, \text{eq}}, b_1, b_{\text{sub}}, b_1, & \text{ (dimensional parameters)}
  \end{align*}
  \]

  \[\textbf{Figure 4-12: Detail of bottom plate cross-section with trapezoidal stiffeners.}\]
Commentary and worked examples to EN 1993-1-5 “Plated structural elements”, 9.2.1 [34]
Minimum requirements for transverse stiffeners (p. 110-111)

[...] For single sided transverse stiffeners the mechanical model is shown in Figure 9.2. The equilibrium equation (9.10) is still valid; only the boundary conditions change due to end moments \( M_{EN} = N_{st,Ed} \epsilon_0 \), where \( \epsilon_0 \) is the eccentricity of the centroid of single sided stiffener relative to the mid-plane of the web. With new boundary conditions the solution of (9.10) becomes much more complicated than the solution given by (9.17) and is not suitable for practical use. To overcome this problem, a simplified approach may be used, based on the expression for maximum displacements and stresses at mid height of double sided stiffeners (9.19) and (9.20).

**Figure 9.2: The mechanical model of a single sided stiffener**

It is considered that \( N_{st,Ed} \) is related to the maximum eccentricity \( \epsilon_0 + w_o \) and \( \Delta N_{st,Ed} \) from deviation force only to \( w_o \). In this case expression (9.20) rewrites as follows:

\[
\sigma_{max} = \frac{N_{st,Ed}}{A_{st}} + \epsilon_{max} \left( \frac{\Sigma N_{st,Ed}}{I_{st}} \frac{W_0 \cdot \epsilon}{I} + \frac{N_{st,Ed}}{N_{cr,st}} \frac{\Sigma N_{st,Ed}}{I} \frac{e_0}{I - \frac{N_{st,Ed}}{N_{cr,st}}} \right)
\]

(9.21)

and after rearranging:

\[
\sigma_{max} = \frac{N_{st,Ed}}{A_{st}} + \frac{\Sigma N_{st,Ed}}{I_{st}} \frac{W_0 \cdot \epsilon_{max}}{I} \left( I + q_m \right) \leq \frac{f_y}{\gamma \cdot M_I}
\]

(9.22)

where:

\[
q_m = \frac{N_{st,Ed}}{\Sigma N_{st,Ed}} \frac{e_0}{W_0}
\]
Gross cross-section of stiffener:

\[
\begin{align*}
\text{bst.1} := \begin{pmatrix} 2.15 \cdot t_f \\ b_{\text{sub}} \end{pmatrix} &= \begin{pmatrix} 1.913 \\ 0.513 \end{pmatrix} \text{m} \\
\text{bst.2} := \begin{pmatrix} 2.15 \cdot t_f \\ b_1 \end{pmatrix} &= \begin{pmatrix} 1.913 \\ 0.485 \end{pmatrix} \text{m} \\
\text{bst} := \min(\text{bst.1}) + \min(\text{bst.2}) &= 0.998 \text{m} \\
A_{\text{st}} := t_{\text{st}} \cdot (b_2 + 2 \cdot b_3) + t_f \cdot b_{\text{st}} &= 93059.4 \cdot \text{mm}^2 \\
z_{\text{st}} := \frac{h_{\text{st}} \cdot (t_{\text{st.eq}} \cdot h_{\text{st}} + t_{\text{st.eq}} \cdot b_2)}{A_{\text{st}}} &= 55.6 \cdot \text{mm} \\
I_{\text{st}} := \frac{2}{12} \left[ h_{\text{st}} \cdot t_{\text{st.eq}} \left( \frac{h_{\text{st}}}{2} - z_{\text{st}} \right)^2 + b_2 \cdot t_{\text{st.eq}} \left( h_{\text{st}} - z_{\text{st}} \right)^2 + \frac{b_{\text{st}} \cdot t_f^3}{12} + b_{\text{st}} \cdot t_f \cdot z_{\text{st}}^2 \right] \\
\end{align*}
\]

Eccentricity of single sided stiffener:

\[
e_0 := z_{\text{st}} = 0.056 \text{m}
\]

Maximum eccentricity of stiffener:

\[
e_{\text{max}} := z_{\text{st}} + \frac{t_f}{2} = 0.093 \text{m}
\]

Amplitude of initial imperfection:

\[
w_0 := \min \left[ \frac{a}{300} \left( \frac{b_{\text{sub}} + \frac{b_1}{2}}{300} \right) \right] = 2.52 \cdot \text{mm}
\]

Axial load in stiffener:

\[
N_{\text{st.Ed}} := A_{\text{st}} \cdot \sigma_{x.Ed} = 17.63 \cdot \text{MN}
\]

Reduction due to plate type behaviour of the adjacent panels (subpanels):

\[
\frac{\sigma_{\text{cr.c.sub},y}}{\sigma_{\text{cr.p.sub},y}} \geq 0.5
\]

Deviation force from patch load:

\[
N_{\text{Ed}} := F_{\text{Ed.bot}} = 5.75 \cdot \text{MN}
\]

Maximum deviation stress from patch load:

\[
\sigma_m := \frac{\sigma_{\text{cr.c.sub},y}}{\sigma_{\text{cr.p.sub},y}} \cdot \frac{N_{\text{Ed}}}{a} \left( \frac{2}{b_{\text{sub}} + \frac{b_1}{2}} \right) = 3.35 \cdot \text{N} \cdot \text{mm}^{-2}
\]
If the same amplification factor \((1+q_m)\) is applied to the displacements, equation (9.19) rewrites as follows:

\[
w = w_0 \frac{I}{N_{cr,Ed}} \left(1 + q_m \right) \leq \frac{b}{300} \tag{9.23}
\]

Expressions (9.21) and (9.22) were tested against the solution of the equilibrium equation (9.10). Based on an extensive parametric study it was found (Beg and Dujc [1]), that safe and very accurate results are obtained, when \(q_m\) is multiplied by a factor 1.11 in (9.22) and by a factor 1.25 in (9.23). This means that single sided transverse stiffeners may be checked to fulfil the requirements (9.1) with the following simplified expressions:

\[
\sigma_{\text{max}} = \frac{N_{st,Ed}}{A_{st}} + \frac{\Sigma N_{st,Ed} e_{\text{max}} w_0}{I_{st}} \cdot \frac{I}{I} \left(1 + 1.11 q_m \right) \leq \frac{f_y}{\gamma_M} \tag{9.24}
\]

\[
w = w_0 \frac{I}{N_{cr,Ed}} \left(1 + 1.25 q_m \right) \leq \frac{b}{300} \tag{9.25}
\]

For single sided stiffeners \(e_{\text{max}}\) has to be understood as the distance from the web surface (opposite to the stiffener) to the stiffener centroid, if this distance is smaller than \(e_{\text{max}}\). This is due to the fact that the most unfavourable situation is present when the initial bow imperfection \(w_0\) extends to the stiffener side of the web. In this case compression stresses from the axial force and from bending sum up at the web side of the stiffener.

See also [1].
Additional axial force in stiffener:

\[
\Delta N_{\text{st.Ed}} := \frac{\sigma_{\text{m}} \cdot a^2}{\pi^2} = 5.44 \text{ MN}
\]

Resulting axial force in stiffener:

\[
\Sigma N_{\text{st.Ed}} := N_{\text{st.Ed}} + \Delta N_{\text{st.Ed}} = 23.06 \text{ MN}
\]

Euler load of stiffener:

\[
N_{\text{cr.st}} := \frac{\pi^2 \cdot E \cdot I_{\text{st}}}{a^2} = 216.29 \text{ MN}
\]

Amplifier of deviation force:

\[
q_m := \frac{N_{\text{st.Ed}} \cdot e_0}{\Sigma N_{\text{st.Ed}} \cdot w_0} = 16.864
\]

Maximum axial stress in stiffener:

\[
\sigma_{\text{max}} := \frac{N_{\text{st.Ed}}}{A_{\text{st}}} + \frac{\Sigma N_{\text{st.Ed}} \cdot e_{\text{max}} \cdot w_0}{I_{\text{st}}} \left(1 + 1.11 \cdot q_m\right) \frac{1}{1 - \frac{\Sigma N_{\text{st.Ed}}}{N_{\text{cr.st}}}}
\]

\[
\sigma_{\text{max}} = 260.9 \text{ N} \cdot \text{mm}^{-2} \leq \frac{f_{\text{yf}}}{\gamma_{M1}} = 295.5 \text{ N} \cdot \text{mm}^{-2}
\]

\(\Rightarrow\) Minimum requirement for „maximum allowable stress“ fulfilled!

Maximum additional deflection:

\[
w := w_0 \left(1 + 1.25 \cdot q_m\right) \frac{N_{\text{cr.st}}}{\Sigma N_{\text{st.Ed}}} - 1
\]

\[
w = 6.6 \text{ mm} \leq \frac{a}{300} = 13.3 \text{ mm}
\]

\(\Rightarrow\) Minimum requirement for „maximum allowable additional deflection“ fulfilled!

**Final conclusion:**

The bottom plate fulfilles the verification acc. to Sections 9 and 10 for the launching situation “1”.

Remark 1: The performed verification via hand-calculation acc. to Section 9 is a safe but conservative approach. In case that the longitudinal stiffeners do not satisfy these requirements, an exact global buckling analysis via appropriate computer software has to be performed.

Remark 2: The verification of the local buckling resistance of the trapezoidal stiffeners can be done separately, whereas the minimum load amplifier is governing the design of the whole cross-section. This calculation is not covered by the calculation example given above.
Table 4-4: Summary of parameter study for launching situation “1”; Variation parameter $n_{st}$.

<table>
<thead>
<tr>
<th>Number of stiffeners $n_{st}$</th>
<th>[-]</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subpanel width $b_{sub}$</td>
<td>[m]</td>
<td>1.84</td>
<td>1.26</td>
<td>0.91</td>
<td>0.68</td>
<td>0.51</td>
</tr>
<tr>
<td>Maximum axial stress in bottom plate due to shear lag effect $\sigma_{1x,Ed}$</td>
<td>[N/mm²]</td>
<td>205.4</td>
<td>206.2</td>
<td>207.0</td>
<td>207.8</td>
<td>208.6</td>
</tr>
<tr>
<td>Axial stress in bottom plate due to shear lag effect at location of 1st stiffener $\sigma_{2x,Ed}$</td>
<td>[N/mm²]</td>
<td>185.5</td>
<td>185.2</td>
<td>185.0</td>
<td>184.8</td>
<td>184.6</td>
</tr>
<tr>
<td>Minimum load amplifier $\alpha_{kst}$</td>
<td>[-]</td>
<td>1.676</td>
<td>1.669</td>
<td>1.663</td>
<td>1.656</td>
<td>1.649</td>
</tr>
<tr>
<td>Minimum load amplifier $\alpha_{cr,sub}$</td>
<td>[-]</td>
<td>4.732</td>
<td>9.520</td>
<td>17.611</td>
<td>31.111</td>
<td>53.803</td>
</tr>
<tr>
<td>Weighting factor $\xi$</td>
<td>[-]</td>
<td>0.615</td>
<td>0.328</td>
<td>0.216</td>
<td>0.163</td>
<td>0.135</td>
</tr>
<tr>
<td>Global plate slenderness of subpanel $\lambda$</td>
<td>[-]</td>
<td>0.595</td>
<td>0.419</td>
<td>0.307</td>
<td>0.231</td>
<td>0.175</td>
</tr>
<tr>
<td>Utilisation level of subpanel $\eta$</td>
<td>[-]</td>
<td>0.430</td>
<td>0.432</td>
<td>0.437</td>
<td>0.441</td>
<td>0.445</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable axial stress in stiffener $\eta_{le,\text{max}}$</td>
<td>[-]</td>
<td>0.828</td>
<td>0.834</td>
<td>0.848</td>
<td>0.865</td>
<td>0.883</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable additional deflection in stiffener $\eta_{w}$</td>
<td>[-]</td>
<td>0.549</td>
<td>0.511</td>
<td>0.496</td>
<td>0.496</td>
<td>0.496</td>
</tr>
</tbody>
</table>
4.2.3.2.2 Application of Sections 3, 9 and 10, EN 1993-1-5 to launching situation “2”

Specific cross-section parameters of considered bottom plate (cp. Section 2.2.3):

- Thickness of bottom plate: \( t_f = 35 \text{ mm} \)
- Geometry of stiffeners: cp. Figure 2-9
- Yield strength of of steel: \( f_{yt} = 345 \text{ N/mm}^2 \)

Determination of the stress field components acc. to elastic bending theory:

- Bending moment in cross-section: \( M_{Ed} = 99.35 \text{ MNm} \)
- Section modulus at bottom plate: \( W_{bot} = 630829 \text{ cm}^3 \)
- Resulting axial stress in bottom plate: \( \sigma_{x,Ed} = 157.5 \text{ N/mm}^2 \)
- Horizontal component of patch load: \( F_{Ed,bot} = 3.912 \text{ MN} \)
- Loading length: \( s_s = 3.0 \text{ m} \)
- Resulting transverse stress: \( \sigma_{y,Ed} = 37.3 \text{ N/mm}^2 \)

Verification if shear lag effect has to be taken into account:

- Effective length: \( L_e = 120 \text{ m} \) (conservative assumption)
- Considered width: \( b_0 = b_p/2 = 3.25 \text{ m} \)

\[ b_0 < L_e/50 \] requirement not fulfilled! Shear lag effect has to be taken into account.

Table 4-5: Summary of parameter study for launching situation “2”: Variation parameter \( n_{st} \).

<table>
<thead>
<tr>
<th>Number of stiffeners ( n_{st} )</th>
<th>[-]</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subpanel width ( b_{sub} )</td>
<td>[m]</td>
<td>1.84</td>
<td>1.26</td>
<td>0.91</td>
<td>0.68</td>
<td>0.51</td>
</tr>
<tr>
<td>Maximum axial stress in bottom plate due to shear lag effect ( \sigma_{x,Ed} )</td>
<td>[N/mm²]</td>
<td>172.3</td>
<td>173.7</td>
<td>175.0</td>
<td>176.3</td>
<td>177.5</td>
</tr>
<tr>
<td>Axial stress in bottom plate due to shear lag effect at location of 1st stiffener ( \sigma_{x,1,Ed} )</td>
<td>[N/mm²]</td>
<td>153.8</td>
<td>153.5</td>
<td>153.1</td>
<td>152.9</td>
<td>152.6</td>
</tr>
<tr>
<td>Minimum load amplifier ( a_{ult,k} )</td>
<td>[-]</td>
<td>2.197</td>
<td>2.178</td>
<td>2.161</td>
<td>2.144</td>
<td>2.128</td>
</tr>
<tr>
<td>Minimum load amplifier ( a_{cr,sub} )</td>
<td>[-]</td>
<td>1.033</td>
<td>2.026</td>
<td>3.704</td>
<td>6.501</td>
<td>11.2</td>
</tr>
<tr>
<td>Weighting factor ( \xi )</td>
<td>[-]</td>
<td>0.615</td>
<td>0.328</td>
<td>0.216</td>
<td>0.163</td>
<td>0.135</td>
</tr>
<tr>
<td>Global plate slenderness of subpanel ( \bar{\lambda} )</td>
<td>[-]</td>
<td>1.458</td>
<td>1.037</td>
<td>0.764</td>
<td>0.574</td>
<td>0.436</td>
</tr>
<tr>
<td>Utilisation level of subpanel ( \eta )</td>
<td>[-]</td>
<td>0.726</td>
<td>0.428</td>
<td>0.287</td>
<td>0.259</td>
<td>0.264</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable axial stress in stiffener ( \eta_{\sigma, max} )</td>
<td>[-]</td>
<td>0.713</td>
<td>0.710</td>
<td>0.708</td>
<td>0.731</td>
<td>0.760</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable additional deflection in stiffener ( \eta_w )</td>
<td>[-]</td>
<td>0.444</td>
<td>0.436</td>
<td>0.436</td>
<td>0.444</td>
<td>0.457</td>
</tr>
</tbody>
</table>
4.2.3.2.3 **Application of Sections 3, 9 and 10, EN 1993-1-5 to launching situation “3”**

**Specific cross-section parameters of considered bottom plate (cp. Section 2.2.3):**

- Thickness of bottom plate: \( t_f = 25 \text{ mm} \)
- Geometry of stiffeners: cp. Figure 2-9)
- Yield strength of of steel: \( f_{yf} = 345 \text{ N/mm}^2 \)

**Determination of the stress field components acc. to elastic bending theory:**

- Bending moment in cross-section: \( M_{Ed} = 50.62 \text{ MNm} \)
- Section modulus at bottom plate: \( W_{bot} = 499908 \text{ cm}^3 \)
- Resulting axial stress in bottom plate: \( \sigma_{x,Ed} = 101.3 \text{ N/mm}^2 \)
- Horizontal component of patch load: \( F_{Ed,bot} = 3.38 \text{ MN} \)
- Loading length: \( s_s = 3.0 \text{ m} \)
- Resulting transverse stress: \( \sigma_{y,Ed} = 45.1 \text{ N/mm}^2 \)

**Verification if shear lag effect has to be taken into account:**

- Effective length: \( L_e = 60 \text{ m} \)
- Considered width: \( b_0 = b_p/2 = 3.25 \text{ m} \)

\( b_0 < L_e/50 \) requirement not fulfilled! Shear lag effect has to be taken into account.

**Table 4-6: Summary of parameter study for launching situation “3”; Variation parameter \( n_{st} \).**

<table>
<thead>
<tr>
<th>Number of stiffeners ( n_{st} )</th>
<th>[-]</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subpanel width ( b_{sub} )</td>
<td>[m]</td>
<td>1.84</td>
<td>1.26</td>
<td>0.91</td>
<td>0.68</td>
<td>0.51</td>
</tr>
<tr>
<td>Maximum axial stress in bottom plate due to shear lag effect ( \sigma_{x,Ed} )</td>
<td>[N/mm²]</td>
<td>134.2</td>
<td>136.1</td>
<td>137.9</td>
<td>139.6</td>
<td>141.2</td>
</tr>
<tr>
<td>Axial stress in bottom plate due to shear lag effect at location of 1st stiffener ( \sigma_{x,Ed}^{1st} )</td>
<td>[N/mm²]</td>
<td>93.0</td>
<td>92.6</td>
<td>92.2</td>
<td>91.7</td>
<td>91.3</td>
</tr>
<tr>
<td>Minimum load amplifier ( \alpha_{ad,k} )</td>
<td>[-]</td>
<td>2.917</td>
<td>2.873</td>
<td>2.833</td>
<td>2.796</td>
<td>2.762</td>
</tr>
<tr>
<td>Minimum load amplifier ( \alpha_{cr,sub} )</td>
<td>[-]</td>
<td>0.588</td>
<td>1.117</td>
<td>2.005</td>
<td>3.477</td>
<td>5.939</td>
</tr>
<tr>
<td>Weighting factor ( \xi )</td>
<td>[-]</td>
<td>0.615</td>
<td>0.328</td>
<td>0.216</td>
<td>0.163</td>
<td>0.135</td>
</tr>
<tr>
<td>Global plate slenderness of subpanel ( \bar{\lambda} )</td>
<td>[-]</td>
<td>2.228</td>
<td>1.604</td>
<td>1.189</td>
<td>0.897</td>
<td>0.682</td>
</tr>
<tr>
<td>Utilisation level of subpanel ( \eta )</td>
<td>[-]</td>
<td>0.844</td>
<td>0.482</td>
<td>0.304</td>
<td>0.206</td>
<td>0.150</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable axial stress in stiffener ( \eta_{\sigma,\max} )</td>
<td>[-]</td>
<td>0.513</td>
<td>0.512</td>
<td>0.511</td>
<td>0.511</td>
<td>0.526</td>
</tr>
<tr>
<td>Utilisation level of maximum allowable additional deflection in stiffener ( \eta_{w} )</td>
<td>[-]</td>
<td>0.308</td>
<td>0.308</td>
<td>0.308</td>
<td>0.308</td>
<td>0.316</td>
</tr>
</tbody>
</table>
4.2.4 Results

In Figure 4-4 the distribution of the patch loading resistances is summarised along the whole bridge length. It can be shown that with the current resistance model of Section 6, EN 1993-1-5 the patch loading resistance cannot be verified for any of the cases. It can be however shown that the improvements made in the COMBRI project [7] lead to increase in the calculated resistances so that launching situation “1” can be verified as it is. For launching situations “2” and “3” a small increase in web thickness from $t_w(2) = 20\,\text{mm}$ to $22\,\text{mm}$ and $t_w(3) = 18\,\text{mm}$ to $20\,\text{mm}$ allows for a verification against patch loading.

The calculation according to Section 6, EN 1993-1-5 showed that at least for launching situation “1” the patch loading resistance could be verified, see Section 4.2.3.1.

![Figure 4-13: Distribution of patch loading resistances according to EN 1993-1-5 [23] along the bridge length.](image)

Figure 4-14 and Figure 4-15 summarise the results of all parameter variations described in Section 4.2.3.2 with regard to the bottom plate. It can be seen that, in spite of the conservatism of the used hand-calculation approach and the used “section modulus at bottom plate”, the level of utilisation for subpanel and stiffener resistance is always below 100%, even if only two stiffeners are used. This means in fact, that for the bottom plate the erection phase is not governing the design.

Furthermore it can be seen that:

1) for thin bottom plates (launching situation “2”, launching situation “3”) the number of stiffeners play a significant roll for its resistance.

2) with increasing number of stiffeners the maximum axial stress in the stiffeners is increasing. This is due to the fact that with decreasing width of the plate the buckling behaviour of the subpanel becomes more column-like. The ratio $\frac{\sigma_{cr}}{\sigma_{crp}}$ in the hand-calculation approach accounts for the influence of column type behaviour in the subpanels transverse direction that increases deviation forces and thus the equivalent axial stress in the stiffener.
Figure 4-14: Utilisation level of subpanel resistance $\eta$ in function of number of stiffeners.

Figure 4-15: Utilisation level of stiffener resistance $\eta_{\text{e, max}}$ in function of number of stiffeners.
5 Summary

This publication is Part I of the Design Manual based on results from the research project „Competitive Steel and Composite Bridges by Improved Steel Plated Structures - COMBRI“ [7] and the subsequent dissemination project “Valorisation of Knowledge for Competitive Steel and Composite Structures - COMBRI+”, both sponsored by RFCS. Part II is a separate publication [8], which shows the state-of-the-art and the conceptual design of steel bridges and the steel parts of composite bridges and it is based on the rules in EN 1993-1-5, EN 1993-2 and EN 1994-2. This Part I focuses in detail on the application of Eurocodes on a composite I-girder bridge and a composite box-girder bridge which are related to plate buckling verifications but an overall view on bridge design could not be covered in depth due to its very wide field.

In Chapter 2 the deck of the twin-girder and the box-girder bridge were described and the global analysis of both bridges were introduced. For this purpose, an overview on the bridge geometry, material distribution and construction sequences were firstly given. Secondly, a general section followed in which common data such as material properties and actions as well as combinations thereof were given. The global analysis were presented for both bridges and the relevant results - internal forces and moments - were summarised and set the basis for the verifications in Chapters 3 and 4 which look at the verifications during the final stage and the execution stage.

In Part II of the Design Manual [8] the standard design of the calculation examples of this Part I is enhanced with regard to the conceptual possibilities of the Eurocode rules and the application of results of the COMBRI research project [7]. Part II of the Design Manual is structured according to certain main topics such as steel grades, flanges, webs, cross bracings and diaphragms as well as launching of steel- and composite bridges. Besides general design recommendations in these chapters, relevant parts of the calculation examples of this Part I are recalculated in order to compare and to show the application of new rules or proposals. Especially the following topics related to this Part I are addressed in Part II of the Design Manual [8]:

- Hybrid girders with higher strength in the flanges than in the webs are economic in many applications. Thus, the box-girder is redesigned from S355 to a hybrid girder with S460 and S690 and it will be shown that the cost of the material is reduced by 10% in the spans and 25% at the piers.

- Double composite action with both top and bottom flanges being composite has been used for some large bridges in Germany and France. The top flange is as usual the bridge deck and the bottom flange has a concrete slab at the piers where the bottom flange is in compression. The design of bridges with double composite action is more complicated than the design of a normal composite bridge so that past experience is summarised and recommendations for design are given.

- Here and in general, it is common that transverse stiffeners are used at the locations of the cross bracings of which the transverse stiffeners form a part. Besides that, the effect of the transverse stiffeners on the resistance of the web is basically an increase in the shear buckling resistance. However, unless the distance between the transverse stiffeners is very short this effect is small and it does not justify the cost of the stiffeners. The possibility of omitting the transverse stiffeners is discussed. Moreover, longitudinal stiffeners on webs increase the resistance for bending as well as for shear so that the economy of using longitudinal stiffeners and their detailing is studied.

- Cross-bracings and diaphragms are to prevent lateral torsional buckling and to transfer lateral loads on the girders to the deck. As used in Chapter 3, traditional cross bracings can be of truss
type or frame type including transverse stiffeners on the webs. Although it is not much material used for cross bracings, from an economical point of view it is important to minimize the man hours for fabrication. This is discussed in terms of eliminating parts and possibly also the transverse stiffeners leading to straightforward solutions.

- The technique of launching bridges has become very popular. As shown in Chapter 4, the resistance to patch loading is of importance as very high support reactions have to be resisted in combination with high bending moments. This has been studied in the COMBRI research project and it resulted in improved design rules. These rules allow the utilisation of quite long loaded lengths and accordingly quite high resistance can be achieved. This may make it possible to launch bridges with parts of the concrete slab or the reinforcement in place. For the twin-girder bridge of this Part I, these two possibilities are studied and the results are compared.
References


[13] EBPlate: A piece of software developed in the frame of the COMBRI research project [7]. Its aim is to assess the elastic critical stresses of plates. EBPlate is free of charge and can be downloaded from the web site of cticm: www.cticm.com


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