ECSC Steel RTD Programme

project carried out with a financial grant of the
European Coal and Steel Community

Composite Bridge Design for Small and Medium Spans

Design Guide

with standard solutions documented in drawings and static analysis

Contract Number: 7210-PR/113

Contractors:
Institute of Steel Construction, RWTH Aachen (D) – Co-ordinator
Centre Technique Industrielle de Construction Metallique, Paris (F)
Division of Steel Structures, Luleå University of Technology (S)
Service Ponts et Carpentes, Université de Liège (B)
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REGISTER OF ILLUSTRATION

1 INTRODUCTION

This Design Guide provides all necessary information about the construction and static calculation of the superstructure of composite bridges for small and medium spans by using new innovative standard solutions.

The Design Guide includes all main topics concerning erection, construction details and the static analyses. Design charts are added to enable a fast forward pre-design.

This report is based on the ECSC-Project "Composite bridge design for small and medium spans" which is shortly described later on. The results of the test programmes performed in this ECSC-project are summarised in standard solutions in this Design Guide.

This Design Guide covers composite bridges with standard solutions by applying partially- and fully prefabricated slabs as well as solid slabs up to a span of 50 m.

Furthermore a software has been developed to verify different kinds of constructions and enable a very fast static calculation by the practical engineer.

Two examples of a single span bridge with fully prefabricated elements and a two span bridge with partially prefabricated slab are attached as tender documents including the static analyses by using the software and drawings with all details.

2 COMPOSITE BRIDGE CONCEPT

2.1 General

2.1.1 ECSC-Project: Composite bridge design for small and medium spans

This Design Guide has been developed in the frame of the ECSC-Project: "Composite bridge design for small and medium spans" (ECSC Contract No. 7210 – PR 113).

The usual span lengths for composite bridges built so far have been in the range of 40 m to 80 m. Bridges with a span lengths of 15 m to 30 m have been mostly built as concrete bridges due to the fact that the superstructure only represents a small part of the total construction works for the main contractor who usually deals with concrete foundations, -piers and –abutments and therefore tends to maintain the building technique he is accustomed to.

The aim of the project has been achieved by developing composite bridge concepts with standard solutions that are easily buildable and therefore attractive for being executed by the main contractor without any problems on site.

The composite bridge concepts cover the span range of about 15 m to 50 m to link the traditional span lengths of composite bridges – by that range they cover about 75 % of all span requirements for road bridges.

Within the scope of the project an extensive test programme has been performed including numerical simulations with parameter and sensibility studies. The test programme has covered:

- Serviceability and fatigue tests with hybrid girders
- Push-out tests with dismountable shear connectors and shear studs ∅ 25 mm
- Behaviour of joints of partially- and fully prefabricated slabs
- Fatigue tests on special joints of beams
- Plate buckling of stocky and slender webs

The results of these test programmes are summarised in standard solutions in this Design Guide. Furthermore a software has been developed to verify different kinds of constructions and enable a very fast static calculation by the practical engineer.

The following partners have been involved in the project:

- Centre Technique Industriel de la Construction Métallique, Paris
- Service Ponts et Charpentes, Institut du Génie Civil, Université de Liège
- Division of Steel Structures, Luleå University of Technology
- Fachgebiet Stahlbau und Verbundkonstruktionen, BUG Wuppertal
- Profil ARBED-Research, Esch sur Alzette
- Scandiaconsult, Luleå
- Lehrstuhl für Stahlbau, RWTH Aachen (Co-ordinator)

The research partners hereby acknowledge the European Commission for the financial support and the F6-Executive Committee for the accompaniment during the research period.

2.1.2 Documents

This Design Guide includes technical drawings and static calculations for the superstructure which have been elaborated in the ECSC-Project “Composite bridge design for small and medium spans”.

The main part of this Design Guide is the summary of standard solutions for the main girders, connections in span or on support, shear connectors, the bridge deck and the cross girder. Furthermore ways of fabrication and erection are shown.

Two examples are laid down in tender drawings:

1. Single span bridge (span length of 16.20 m) with fully prefabricated elements
2. Two span bridge (each span lengths 26 m) with partially prefabricated elements

The drawings for the steel construction which are described in detail are only for disposition and therefore no workshop drawings have been made.
For the concrete deck all necessary formwork- and reinforcement drawings are given including the details for the partially- and fully prefabricated elements as well as for the cap for the sidewalks.

Concrete foundations, piers and abutments are only secondary in this Design Guide.

2.2 Boundary Conditions

2.2.1 Assumptions

This Design Guide considers single and continues superstructures out of composite cross sections whereas the main girder has been prefabricated in the workshop and the concrete slab is completed on site using different techniques.

These standard solutions and examples have been designed for different widths of road profiles and the load model 1 according to the EC 1, 4.3.2.

2.2.2 Standards

The Design is based on the following Eurocodes:		Version:
EN 1991 Eurocode 1	Basis of design and actions on structures	ENV 1991 : 1994
EN 1992 Eurocode 2	Design of concrete structures	ENV 1992 : 1991
EN 1993 Eurocode 3	Design of steel structures	ENV 1993 : 1997
EN 1994 Eurocode 4	Design of composite steel and concrete structures	ENV 1994 : 1992

2.3 Superstructure

2.3.1 Static system

This Design Guide comprises single and continues composite bridges with different levels of prefabrication for the concrete slab.

The concrete slab consists out of a solid slab or a partially- as well as fully prefabricated deck elements. For the main girders rolled and welded) sections (including LP-plate sections) can be used. The amount of the main girders is depending on the required road width or -class.

Solid slabs need a complete formwork before concreting the bridge deck. By using fully prefabricated slabs the openings for the shear connectors in the elements (pockets or covered channel) and the joints between the elements have to be filled with mortar. By pre-stressing the fully prefabricated elements before grouting the joints between the elements can be closed up to a minimum.

Partially prefabricated elements are the formwork during casting and shall also be used later as a part of the total slab height. The distance between the main girders can be determined by varying the slab thickness and the used slab technique (e.g. partially prefabricated slab,
$d_{Slab} = 10 + 20 = 30$ cm $\Rightarrow$ max $b_{Girder\ Distance} \approx 3{,}00$ m).

An appealing design of the superstructure can be reached by a slenderness of about $L / h = 25$
(L = span length, h = total construction height).

After completion of the slab all main girders are connected in-between by the concrete slab. To guarantee the stability of the main girders against torsional-flexural buckling the girders shall be lateral stiffened by using cross girders in the main axes of the piers and abutments only. These cross girders can be designed in steel or in concrete. The bearings can be placed under the cross girders or directly under the webs of the main girder. Additional torsional stiffeners can be applied to obtain a knuckle support in the moment zero point.

One main aspect of this design is the erection of the bridge without cost-intensive temporary supports or proppings. Furthermore the casting sequences have to be considered for continues bridges.

The following Table 2-1 shows the construction sequences for an in-situ casted solid slab or a fully pre-fabricated slab (example for a two-span bridge). The formwork for the in-situ casted slab is placed in-between the main girders on the lower flange and is considered for the system 2 only.

Table 2-1: Construction Sequences for a solid- and fully prefabricated slab

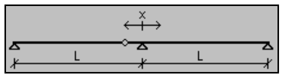
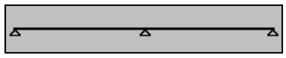
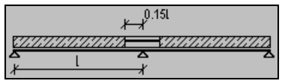
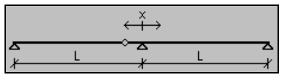
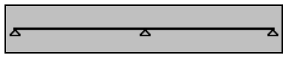
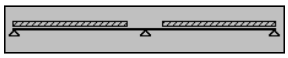
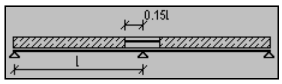
Construction Situation	Description	Load	
System 1	**Multi-beam bridges: Steel beam with a hinge**	**Dead load of the steel beam**	
System 2	**Steel beam with welded hinge**	**Dead load of the solid slab or the fresh concrete and construction loads**	
System 4	**Full composite action considering cracks in concrete in the hogging moment area**	**Infrequent and frequent load**	

Table 2-2 shows the construction sequences for a bridge by using a partially prefabricated slab.

Table 2-2: Construction Sequences for a partially prefabricated slab

Construction Situation	Description	Load	
System 1	**Multi-beam bridges: Steel beam with a hinge**	**Dead load of the steel beam**	
System 2	**Steel beam with Welded hinge**	**Dead load of the partially prefabricated slab**	
System 3	**Partial composite action**	**Dead load of the fresh concrete and construction loads**	
System 4	**Full composite action considering cracks in concrete in the hogging moment area**	**Infrequent and frequent load**	

The steel girders are acting as simple beams during the first construction situation and are laterally supported by the cross girders in the main axes. The dead load of the steel girders is carried by these simple beams only (System 1).

After welding the hinge continues steel beams are obtained which have to carry the partially- or fully prefabricated deck elements as well as the solid slab in the second construction situation (System 2).

By using fully prefabricated elements or a solid slab this continues steel beams have to carry the dead load of the solid slab or the fresh concrete and construction loads only (System 2).

By using partially prefabricated slabs in the construction situation 3 a partial composite action can be achieved by grouting the joints and pockets in the elements. The supplement fresh concrete and the construction loads are carried after the setting of the mortar by the partial composite cross section (System 3). The partially prefabricated elements are now acting as one shell (e.g. for wind loads). The static utilisation of the partial composite action provides a very economic and optimised construction.

The last construction situation (System 4) is the continues and finished composite cross section considering the cracked concrete in the area of the hogging moment (simplification). On this static system infrequent (e.g. finished permanent loads like caps and guard-rails) and frequent (e.g. traffic) loads are applied under considering the time dependent behaviour of the cross sections (creep- and shrinkage effects).

The relevant forces for each construction phase are multiplied by safety- and combination factors and summed up. With the load combinations and the estimated cross sections the following checks for the ultimate limit state and the serviceability state are performed:

Ultimate limit state (ULS) of the steel girder (System 1 and 2):
- ultimate resistance against positive bending
- ultimate resistance against negative bending
- ultimate resistance against positive / negative bending taking interaction of shear in account
- ultimate resistance against shear
- ultimate resistance against torsional-flexural buckling

Ultimate limit state (ULS) of the composite beam (System 3 and 4):
- ultimate resistance against positive bending
- ultimate resistance against negative bending
- ultimate resistance against shear
- ultimate resistance against negative bending taking interaction of shear in account
- bonding strength of the shear connectors, number of shear connectors
- ultimate resistance against torsional-flexural buckling

Serviceability limit state (SLS) of the composite beam (System 3 and 4):
- stress analysis
- crack width limitation and check of minimum reinforcement
- deflection check

To complete the design of a structure, the following items have to be carried out, but are not considered in the design guide:
- distribution of the shear connectors
- shear resistance of the concrete chord
- check of the contour area of the shear connector
- fatigue design
- vibration behaviour of the structure could be checked potentially (especially for slender structures using HSS and HSC)

2.3.2 Steel construction

The main girders should be prefabricated and painted with protection against corrosion in the workshop before arriving on site. By using welded plates for the girders the required cross sections can be suited very well to the stress distribution. A cost optimised steel construction can be reached by using hybrid girders with lower strength steel for the web and higher strength steel for the flanges. The number of different cross sections of welded plate girder in longitudinal direction should be minimised to obtain a small number of joints and different plate thickness. In the frame of the ECSC-Project hybrid girders have been tested under fatigue loads. The results of these tests are summarised in the chapter 4.2.

A further decrease of weight can be achieved by using LP-plates with a variable thickness in longitudinal direction; these LP-plates are rolled by a few mills only.

The main girders are connected on support or in span by using welded as well as bolted connections and temporary cams. In the area of welded connections the corrosion protection has to be removed before the welding procedure and has to be completed after finishing the welding activities.

The usual steel grades for composite bridges are S235, S355 or S460. By using special steel, e.g. HISTAR from Profil ARBED, a reduction of steel strength according to the steel thickness can be precluded.

Table 2-3: characteristic data of the steel

Steel grade	**thickness t**	
	t ≤ 40 mm	**40 mm < t**
	f_y [N/mm²]	**f_y [N/mm²]**
S 235	**235**	**215**
S 275	**275**	**255**
S 355	**355**	**335**
S 420	**420**	**390**
S 460	**460**	**420**

2.3.3 Shear connectors

In general headed shear studs are used as shear connectors. Studs with a diameter of 16, 19, 22 and 25 mm are commonly available on the market. Studs with a diameter of 25 mm are not considered in the Codes up to now. In the frame of the ECSC-Project these studs have been tested under fatigue loads. The results of these tests are summarised in the chapter 4.2.

By using partially prefabricated slabs with in situ concrete for the partial composite action in the construction situation 3 double headed shear studs are used.

The tensile strength and the yield strength for headed shear studs is based on a cold pulled S235 – the S460 has not been tested yet.

Besides the shear studs several alternative shear connectors, e.g. fixing slips, dismountable shear connectors, etc. can be used.

2.3.4 Slab

In general three different kinds of slabs can be specified:

1. Solid slabs (casting with in situ concrete)
2. Partially prefabricated slabs
3. Fully prefabricated slabs

The following concrete strengths are possible:

Table 2-4: characteristic data of the concrete strength

Concrete strength	C 20 / 25	C 25 / 30	C 30 / 37	C 35 / 45	C 40 / 50	C 45 / 55	C 50 / 60
f_{ck} [N/mm²]	20	25	30	35	40	45	50
f_{ctm} [N/mm²]	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_{cm} [kN/cm²]	2900	3050	3200	3350	3500	3600	3700

The concrete strength of C 40 / 50 for prefabricated elements and C 30 / 37 for in situ concrete are usually used for composite bridges of small and medium spans. By using high strength steel for the girders, e.g. S460, and high strength steel for concrete, e.g. C70/85, a high slenderness and bearing capacity for the composite cross section can be reached.

2.3.5 Prefabricated elements

This Design Guide considers fully and partially prefabricated slabs.

Compared to concrete bridges one major advantage of composite bridges is that the steel girders can carry the weight of the formwork (lost or used for the final stage), prefabricated elements and the wet concrete making the need for temporary structures superfluous. Another advantage is the saving of construction time by fast erections with a high grade of prefabrication which saves money for the contractor, but even more so for the road users, a fact that is commonly neglected when evaluating alternative bridge designs.

The partially prefabricated slabs are used as formwork during casting and in the final state as a part of the total slab depth (no lost formwork). The dimension of the elements should be suited to the width of the superstructure. For a narrow bridge with only two lanes, small sidewalks and two main girders and only one prefabricated element in transversal direction could be used. For bridges with more than two main girders the prefabricated element on the outside should include the cantilever area of the sidewalk. Therefore these prefabricated elements need pockets with small tolerances for the shear connectors. The longitudinal and transversal reinforcement in these pockets has to be suited to the local distribution of the shear studs. A simple pattern for the shear studs and the reinforcement is commendable. The inner prefabricated elements should be used as simple span slabs with a continues longitudinal joints on the steel beams due to the statically determined supports.

The outside elements should be dimensioned with an edge acting as a face formwork (with a height of 15 to 25 cm). Furthermore the cantilever elements shall be designed in a way that no further support is necessary for the in-situ casting of the slab.

The completion reinforcement for the in-situ concrete should not be thread through the bars of the prefabricated elements but lay on top of them. This requires special lattice girders with garlands without top bars. The lattice girder of the prefabricated elements shall be designed for dynamic loads (official approval based on test reports) to obtain a composite action of the full slab depth.

The in-situ concrete deck is casted on top of the partially prefabricated elements. In the area of the end cross-girders conventional formwork is necessary for oblique-angled bridges. Furthermore often more shear connectors are necessary over these critical areas which requires the whole width of the top flange.

To prevent reaction of constrains all prefabricated slabs should be supported statically determined. The steel beams are delivered on site with special supporting strips (heights of 10-30 mm) on top out of synthetic elastomer to obtain:

- a sealing joint during casting between steel beam and prefabricated element
- a compensation for tolerances and superelevations of the steel beams (especially for oblique-angled bridges)
- a waterproof joint between steel and concrete in the final state
- a compensation for the transverse slope

The supporting strips should be accurately defined according to the weight of the slab and the geometrical boundary conditions (specified stress–deformation–behaviour). The market offers homogenous and two-component systems.

The prefabricated elements are placed according to a laying drawing by considering sufficient tolerances for the concrete and the reinforcement. To prevent flaking effects of the elements edges are broken under an angle of 45°. The dimension of prefabricated slabs have to be suited to the allowed sized of road haulage.

A further step to improve the competitiveness of composite bridges is the erection of fully prefabricated concrete deck on site. The main advantages obtained by adopting this concept are listed below.

- A shorter construction time.
- The deck elements are cast indoors, which is believed to result in improved quality.

- An improved working environment for the workers while erecting the formwork, placing the reinforcement bars and casting the concrete.

For instance in Sweden and France this concept has been used for many bridges, with reinforcement splices in situ cast joints. Furthermore, a new concept with dry joints between match cast deck elements has been realised. The composite action is achieved by injection of a shear stud channel in the elements, leaving its top surface dry.

2.3.6 Cross girder and End-cross girder

In general cross girders out of concrete and steel can be applied.

For a competitive composite bridge cross girders should be only applied in the main axes of the supports and not in span.

To reach a minimum amount of bearings the supports should be placed between the main girders. Therefore the cross girders are loaded under bending and shear forces. Supporting points for jacks can be considered directly under the main girder to exchange the bearings.

A very easy erection can be performed by placing twin-beams on site. The connections between these twin-beams can act as cross girders later on. After placing the main girders the missing cross girders between the twin-beams can be supplemented on site.

Chapter 4.3.4 offers some innovative solutions of connections between cross and main girders.

2.4 Design Charts

The pre-design of a composite bridge is a time intensive procedure. Therefore design charts for common cross sections with spans from 10 m upwards are given to easily get an estimation of the necessary beam section for the composite structure.

The following design charts are based on the EC design rules and carried out with the software CBD.

The design is based on the assumption, that torsion of the main girders is negligible. Therefore main girders are considered to be separately and plane in vertical direction. Investigation of rigid strips for each steel girder is possible. Vertical loads are applied according to the lever principle over the cross section.

The included bridge constructions are simple span and two-span bridges using prefabricated elements. The indicated span of the diagrams for two-span bridges represent the span length of each span. Consequently the total bridge span is twice the span length. The following parameters are kept constant for all design charts:

- The total slab thickness is 300 mm
- The thickness of the prefabricated element is 100 mm
- BSt 500 S is applied as reinforcement
- C 35 / 45 is used as concrete
- The degree of longitudinal reinforcement in the field is 6 %
- The degree of longitudinal reinforcement at support is 11 %
- The degree of transversal reinforcement 0.5 %
- main beams in S 460
- 200 mm Shear studs with a diameter of 22 mm in S 460 are used
- The applied traffic loads are conform to EC1 Part 2
- All constructions situation are taken into account
- The bridge is unpropped during construction
- The maximal deflection is limited to L / 200
- Creep, shrinkage and moment distribution are taken into account
- The national safety factors of Germany are used

Varying parameters are:

- The span length
- The bay width of the cross section.

2.4.1 Simple bridge with end supports pinned

2.4.1.1 H-beams with a bay width of 1.80 m

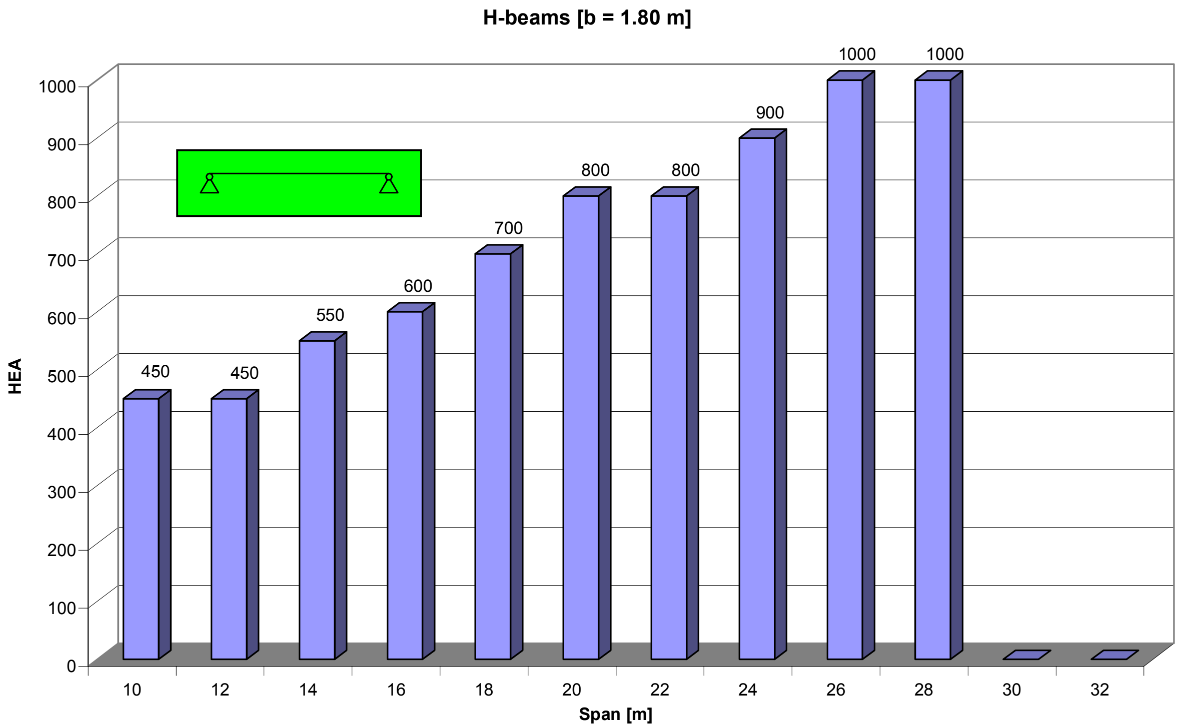


Figure 2-1: Design Chart: Simple Bridge, HEA, b = 1.80 m, supports pinned

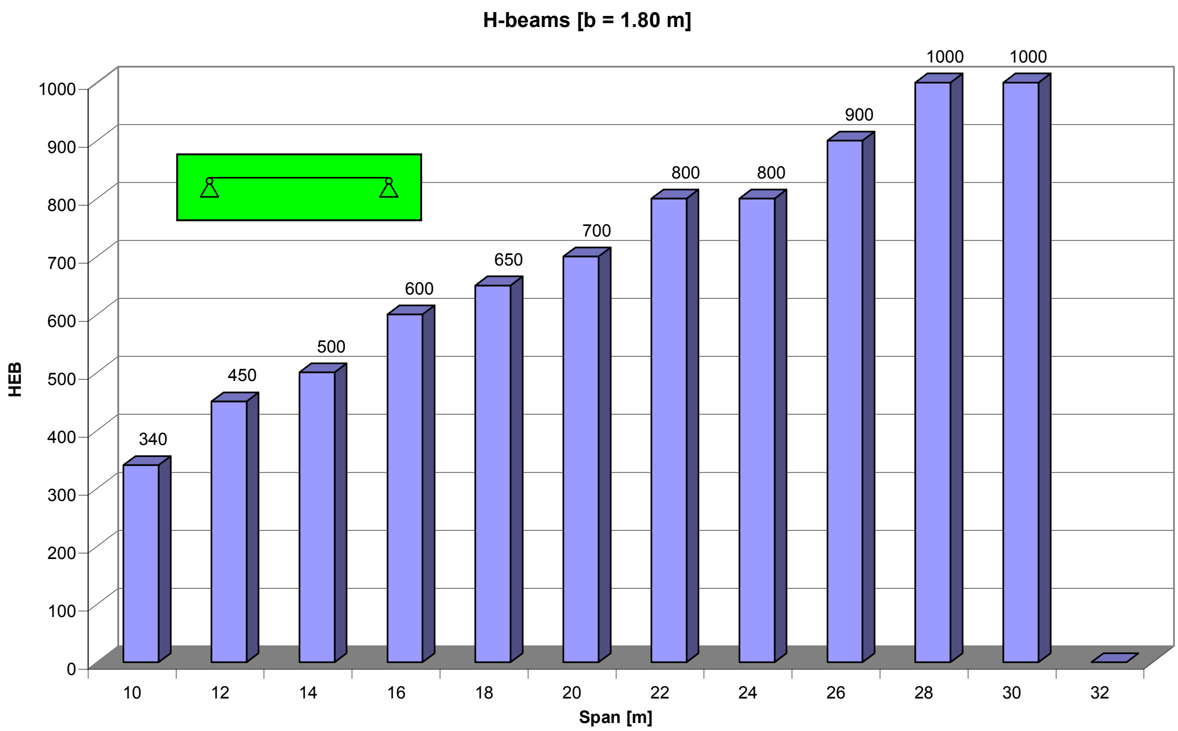


Figure 2-2: Design Chart: Simple Bridge, HEB, b = 1.80 m, supports pinned

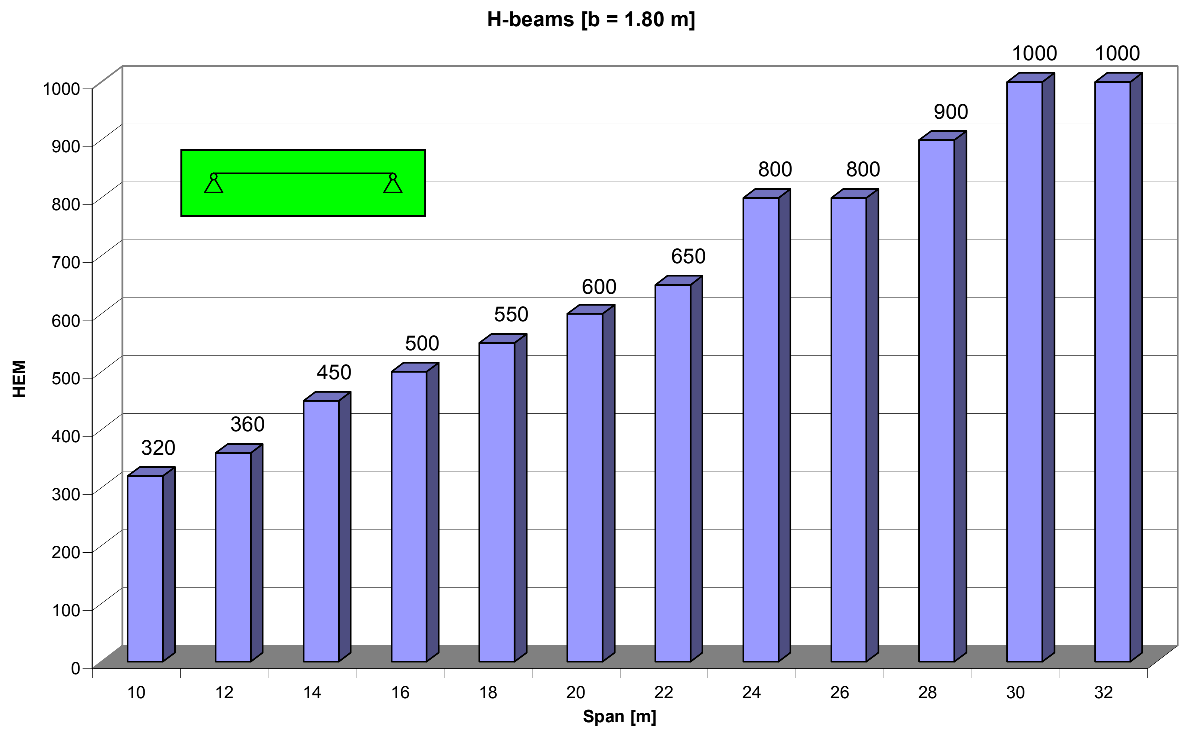


Figure 2-3: Design Chart: Simple Bridge, HEM, b = 1.80 m, supports pinned

2.4.1.2 H-beams with a bay width of 2.67 m

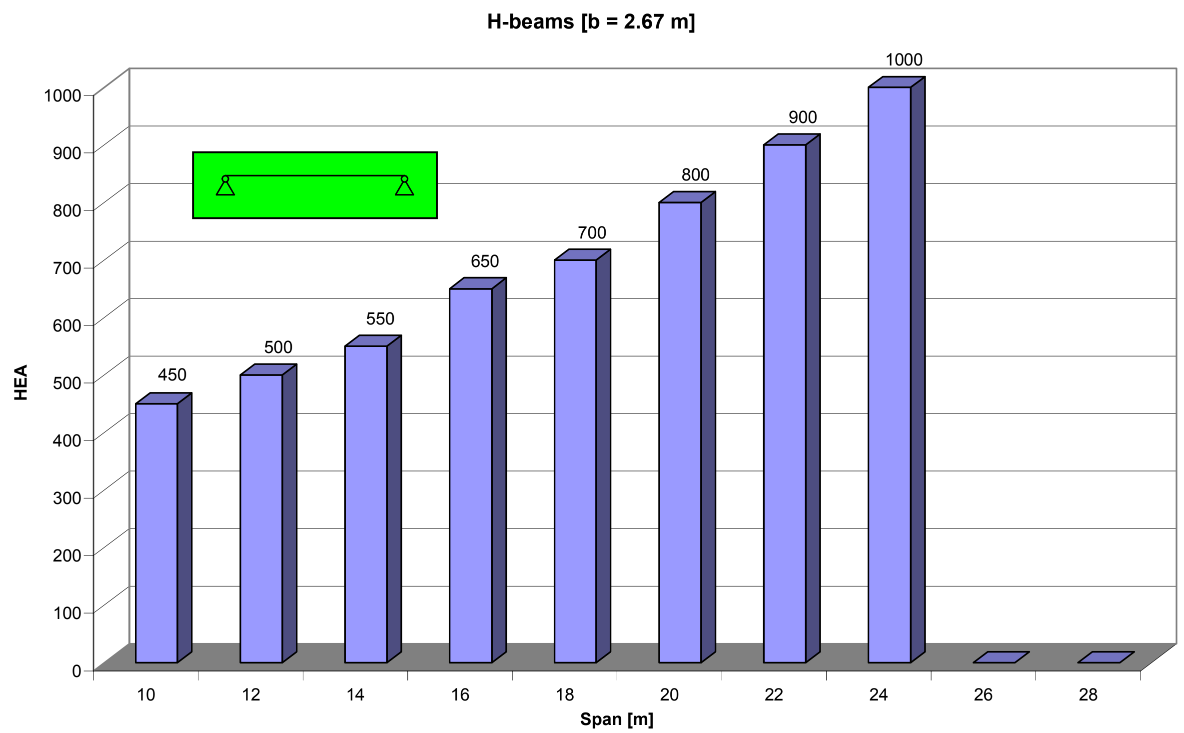


Figure 2-4: Design Chart: Simple Bridge, HEA, b = 2.67 m, supports pinned

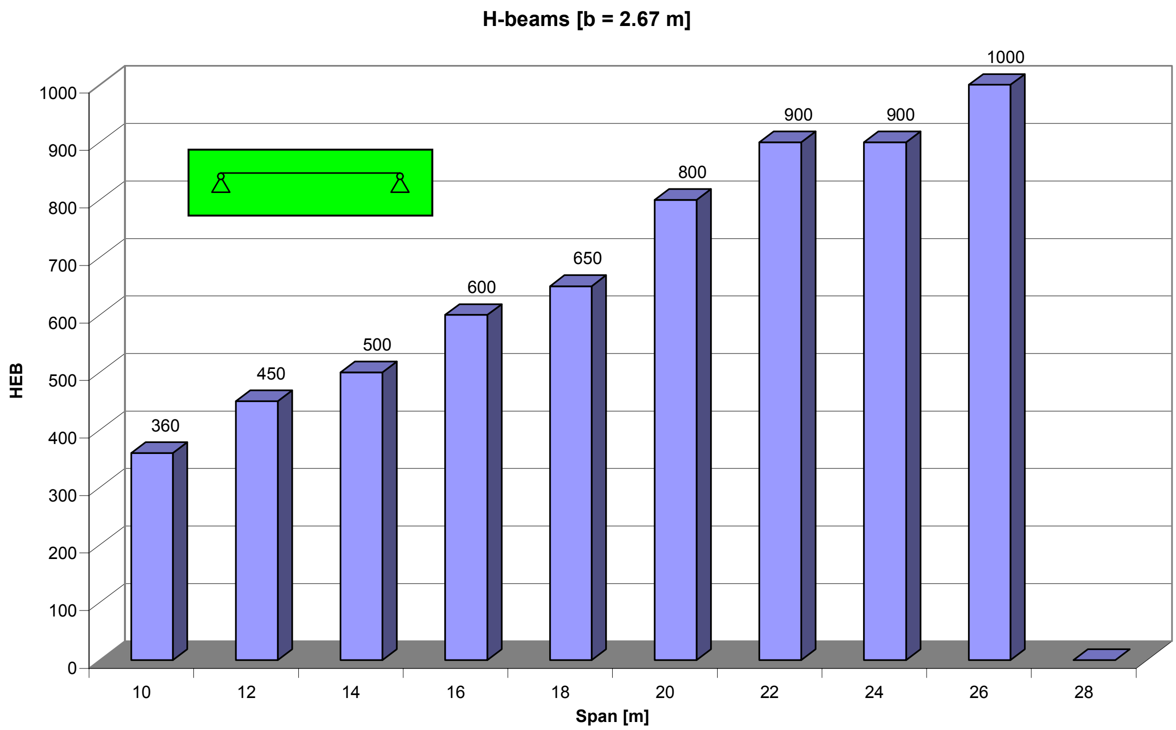


Figure 2-5: Design Chart: Simple Bridge, HEB, b = 2.67 m, supports pinned

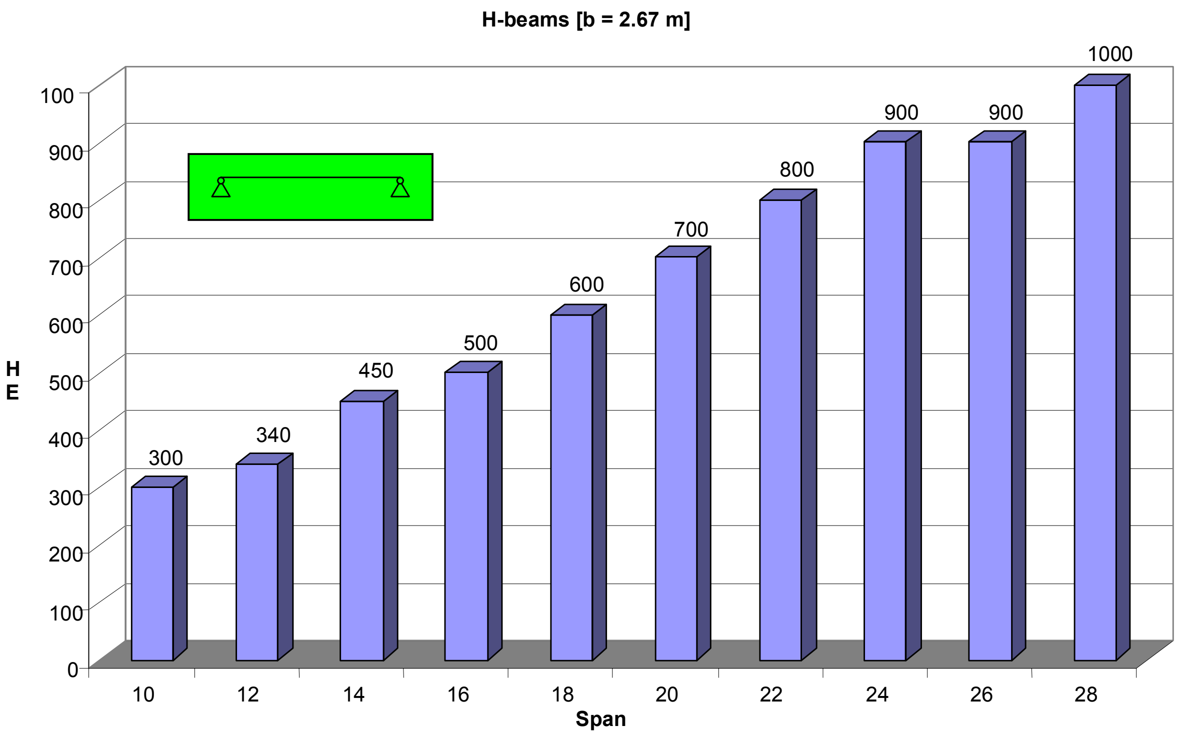


Figure 2-6: Design Chart: Simple Bridge, HEM, b = 2.67 m, supports pinned

2.4.2 Simple bridge with end supports clamped

2.4.2.1 H-beams with a bay width of 1.80 m

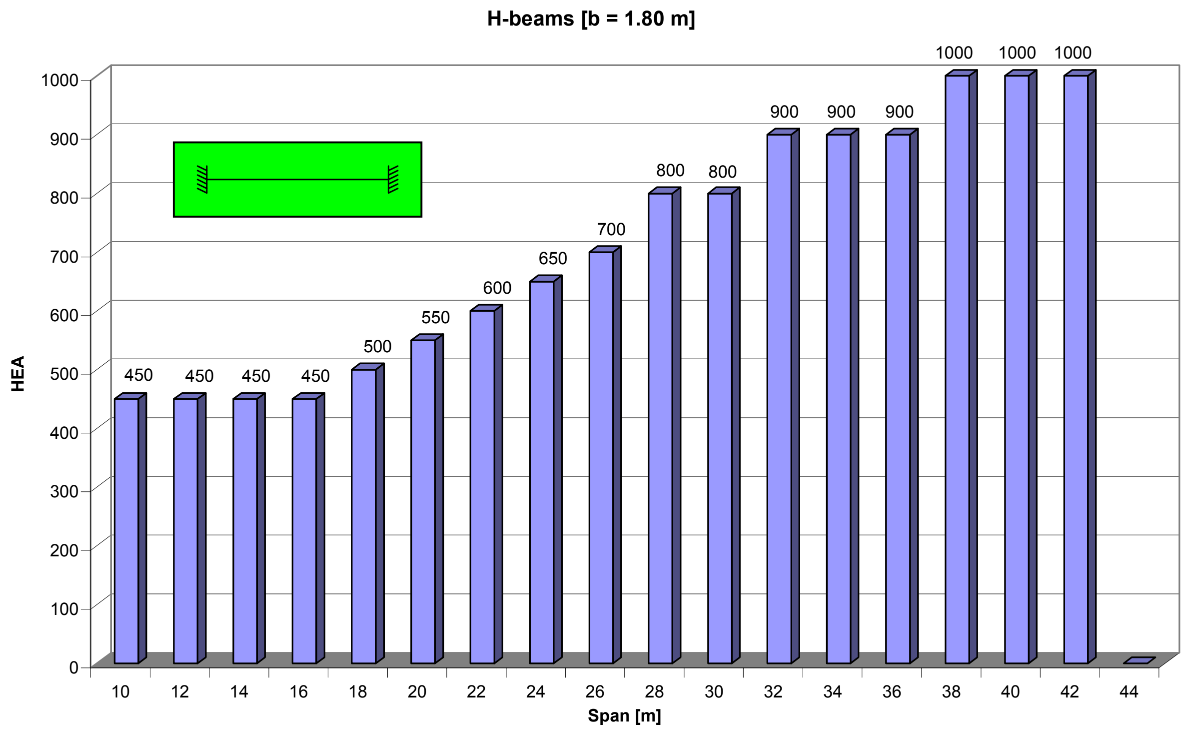


Figure 2-7: Design Chart: Simple Bridge, HEA, b = 1.80 m, supports clamped

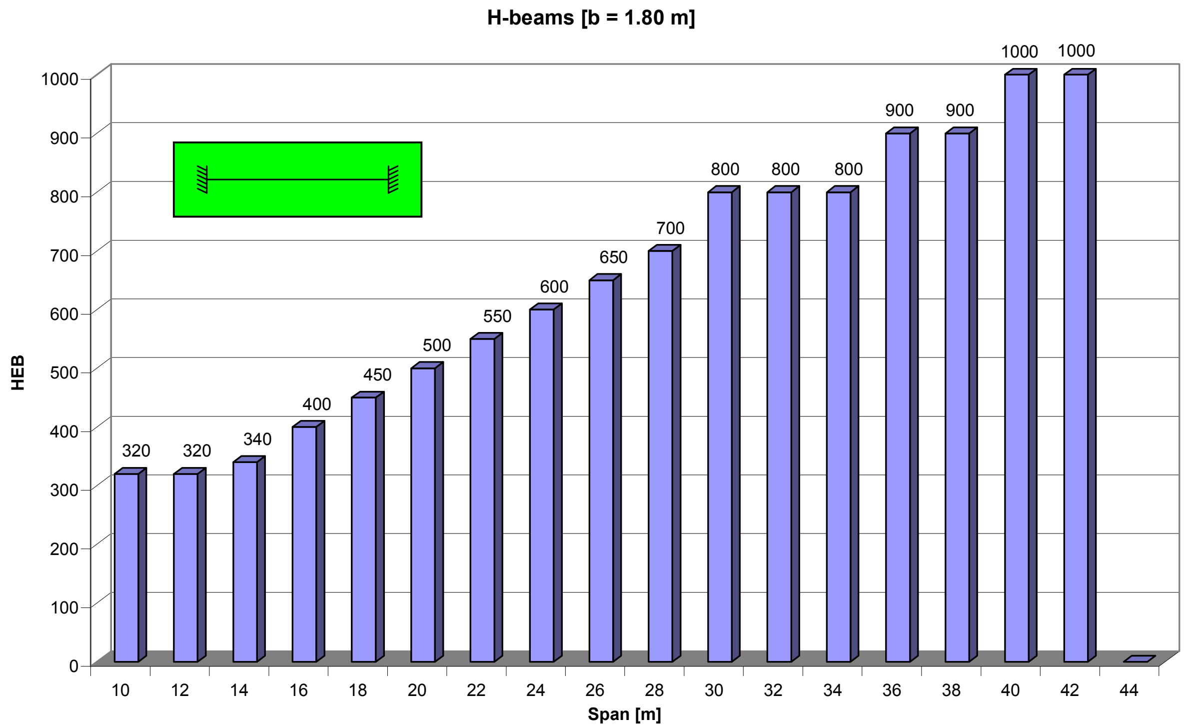


Figure 2-8: Design Chart: Simple Bridge, HEB, b = 1.80 m, supports clamped

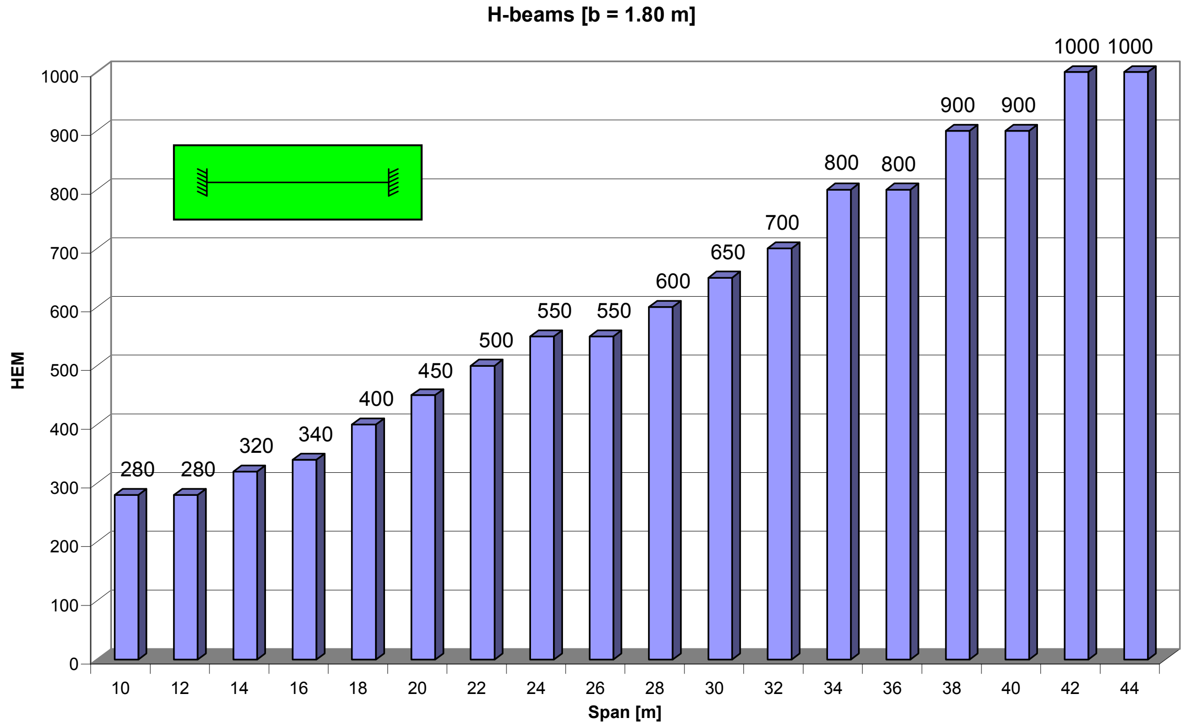


Figure 2-9: Design Chart: Simple Bridge, HEM, b = 1.80 m, supports clamped

2.4.2.2 H-beams with a bay width of 2.67 m

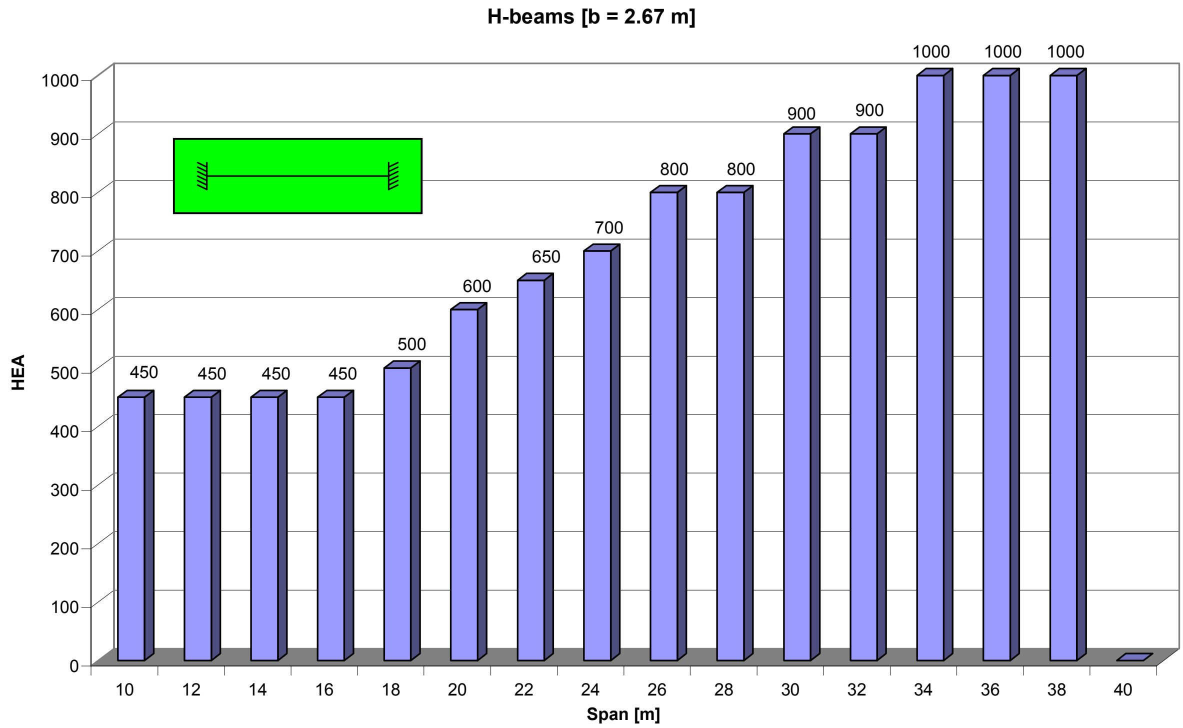


Figure 2-10: Design Chart: Simple Bridge, HEA, b = 2.67 m, supports clamped

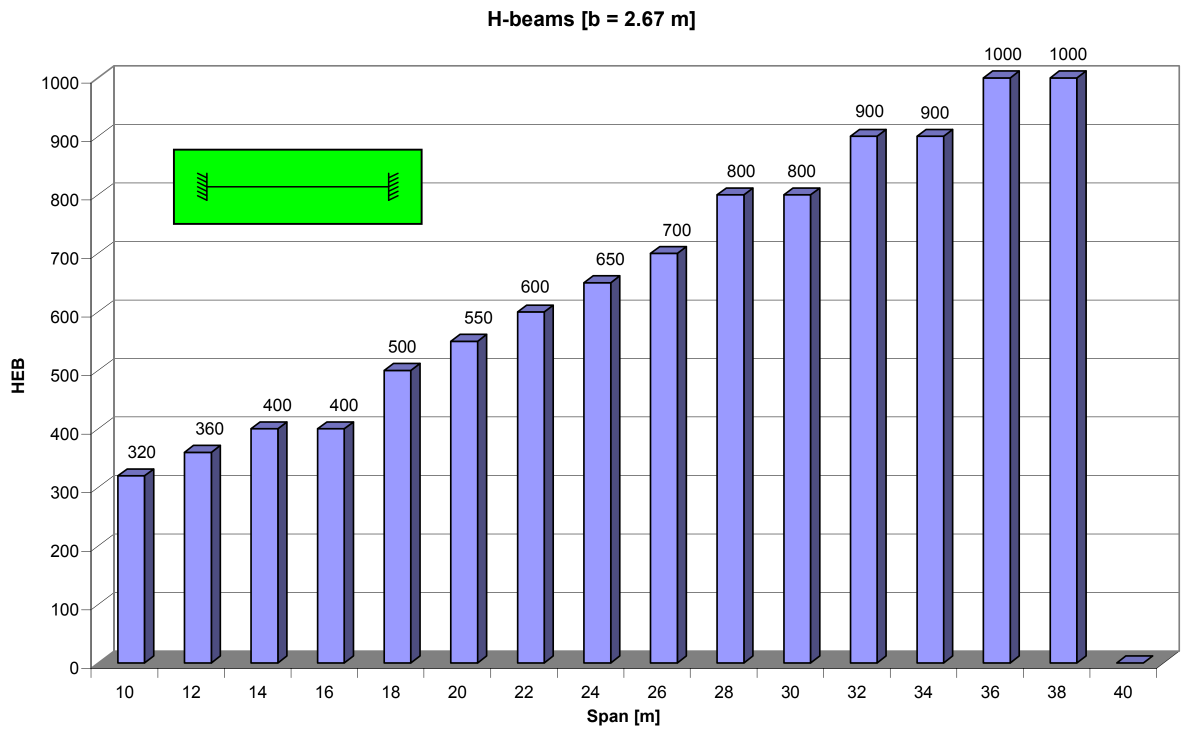


Figure 2-11: Design Chart: Simple Bridge, HEB, b = 2.67 m, supports clamped

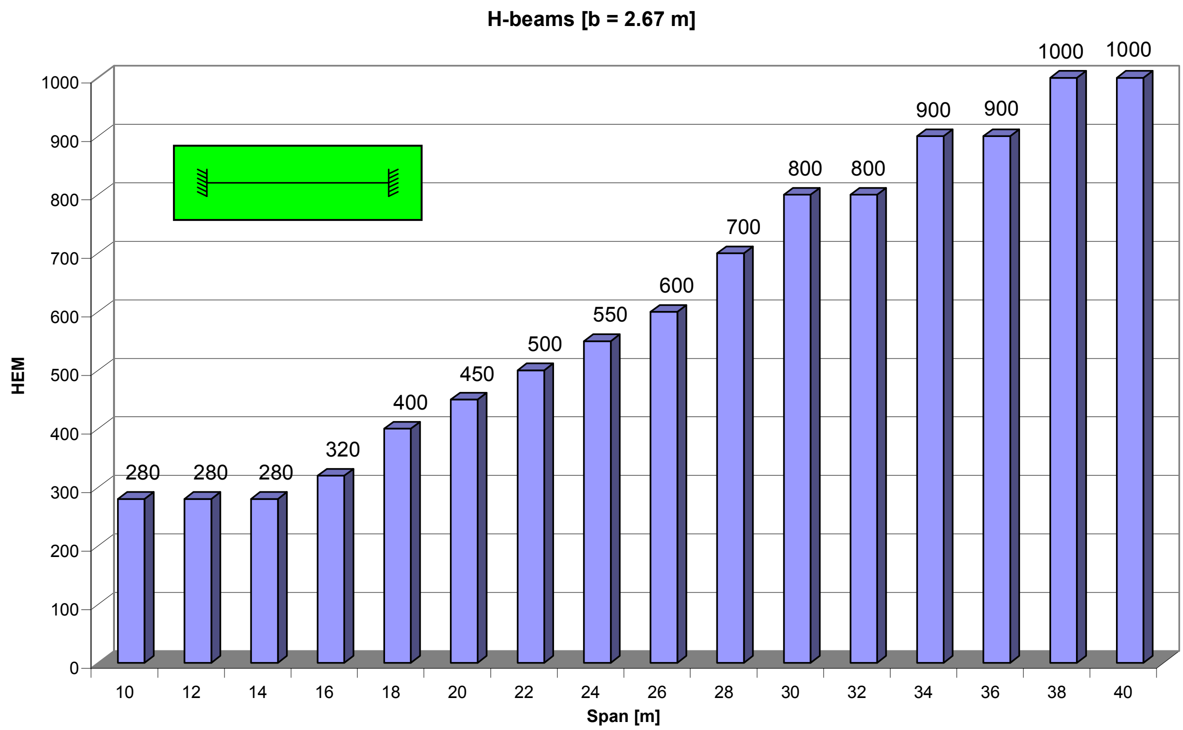


Figure 2-12: Design Chart: Simple Bridge, HEM, b = 2.67 m, supports clamped

2.4.3 Two-span bridge with end supports and the middle support pinned

2.4.3.1 H-beams with a bay width of 1.80 m

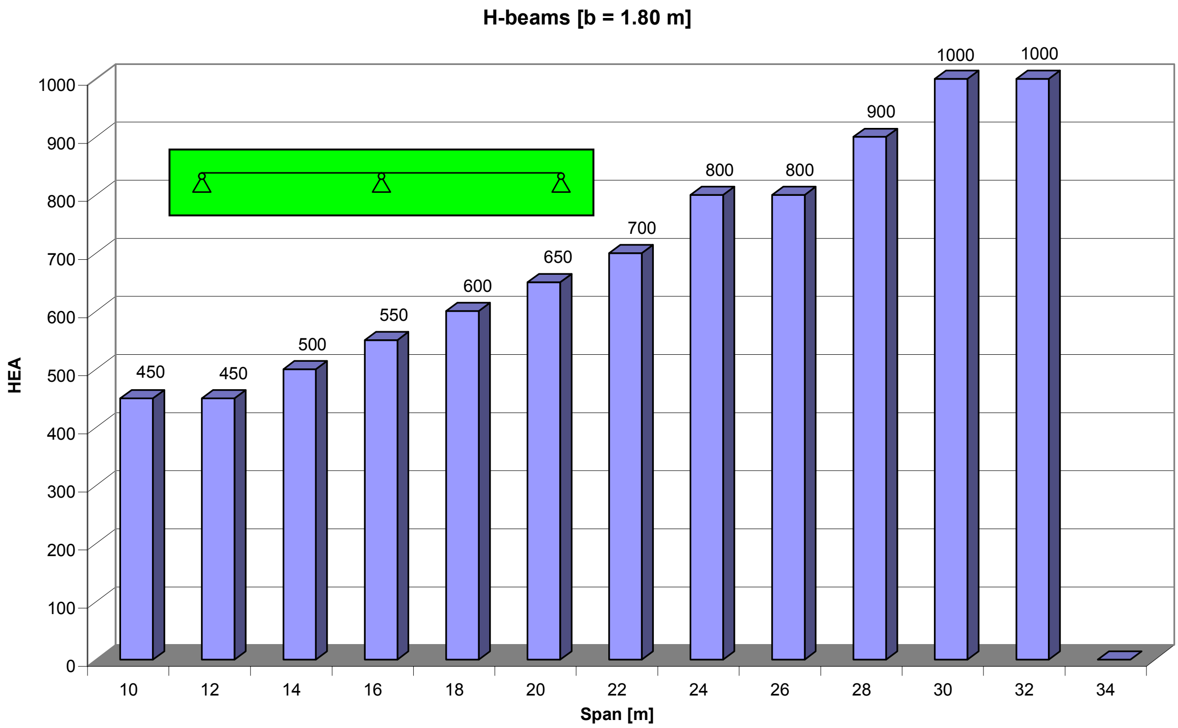


Figure 2-13: Design Chart: Two-span Bridge, HEA, b = 1.80 m, supports pinned

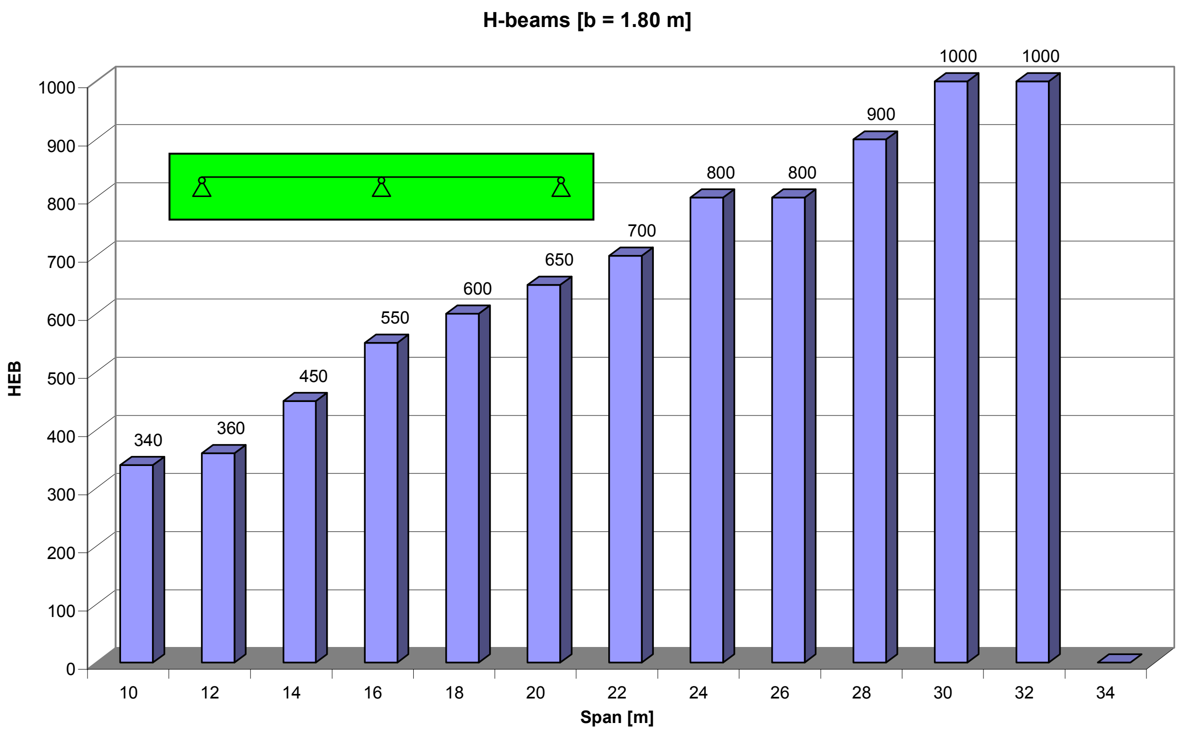


Figure 2-14: Design Chart: Two-span Bridge, HEB, b = 1.80 m, supports pinned

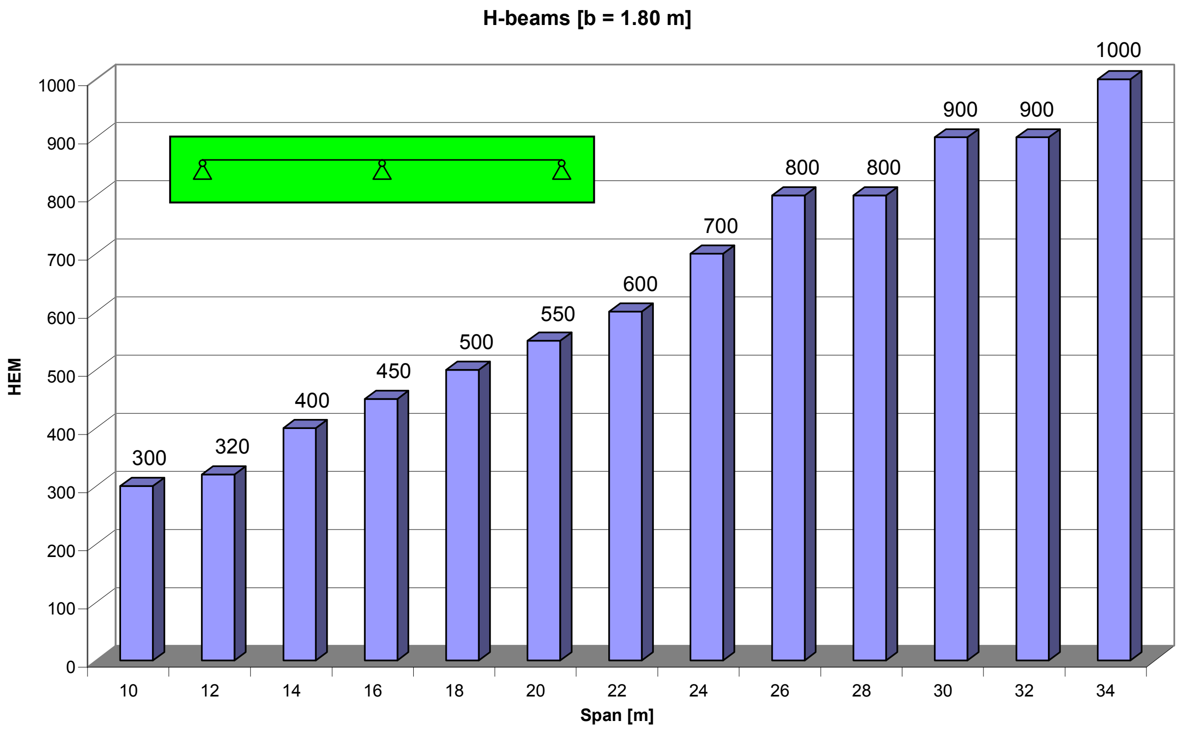


Figure 2-15: Design Chart: Two-span Bridge, HEM, b = 1.80 m, supports pinned

2.4.3.2 H-beams with a bay width of 2.67 m

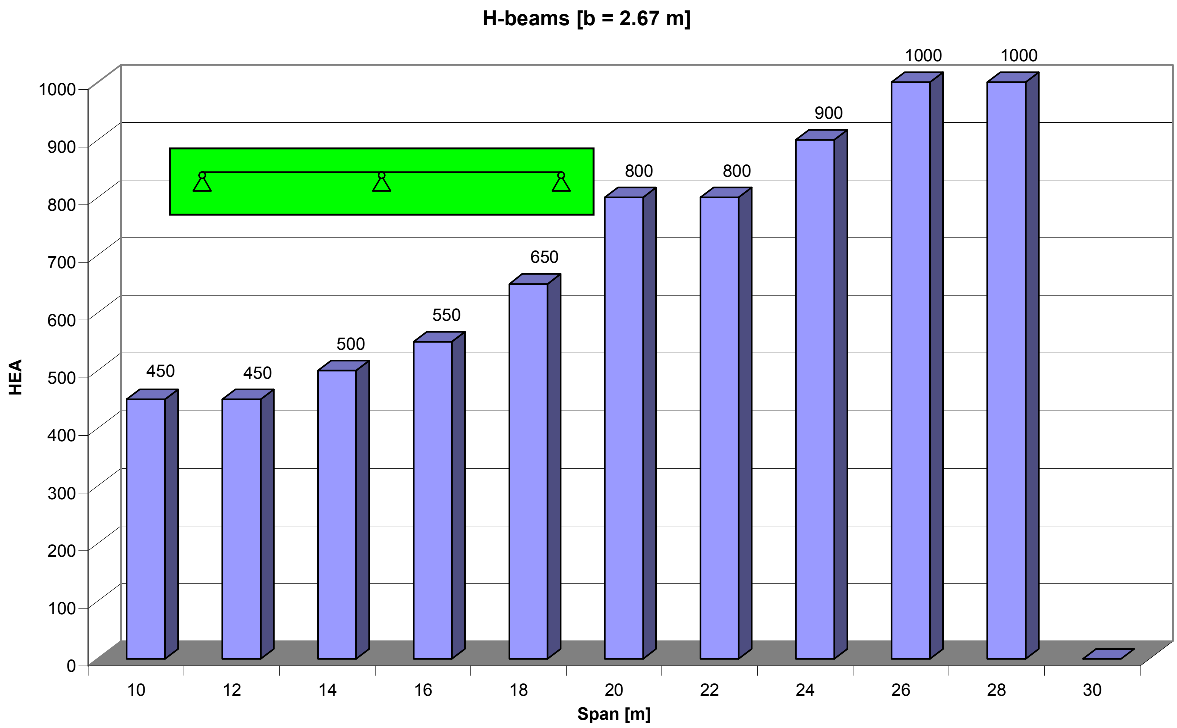


Figure 2-16: Design Chart: Two-span Bridge, HEA, b = 2.67 m, supports pinned

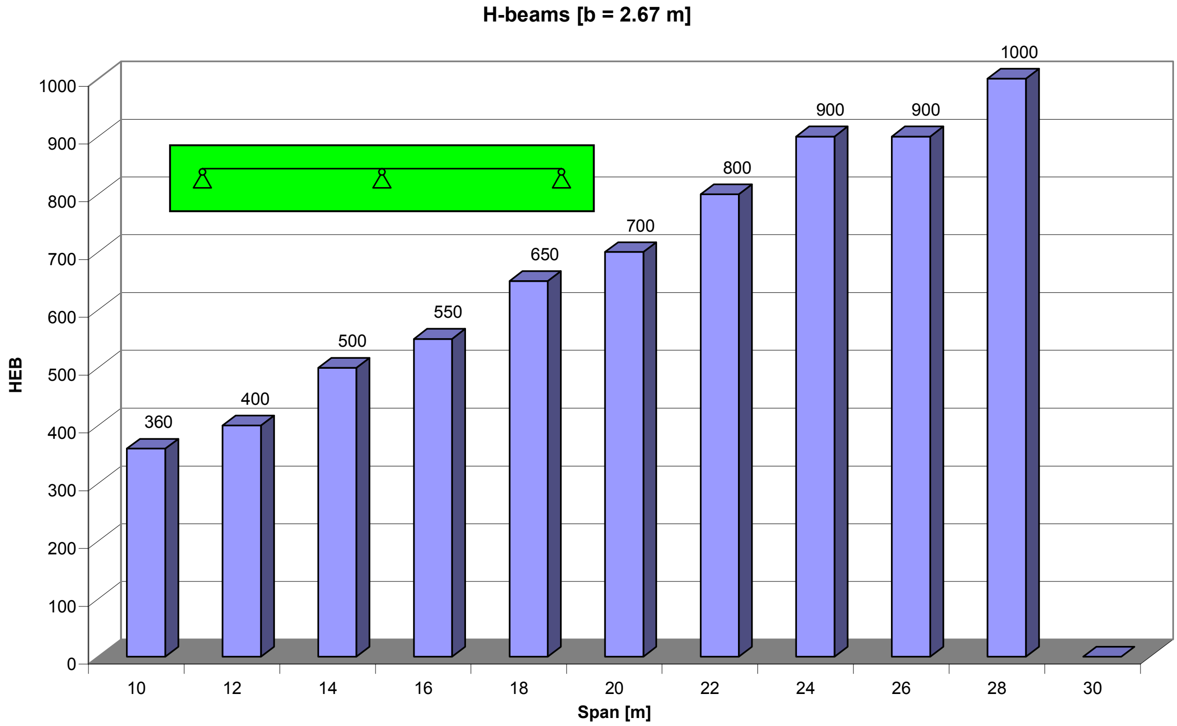


Figure 2-17: Design Chart: Two-span Bridge, HEB, b = 2.67 m, supports pinned

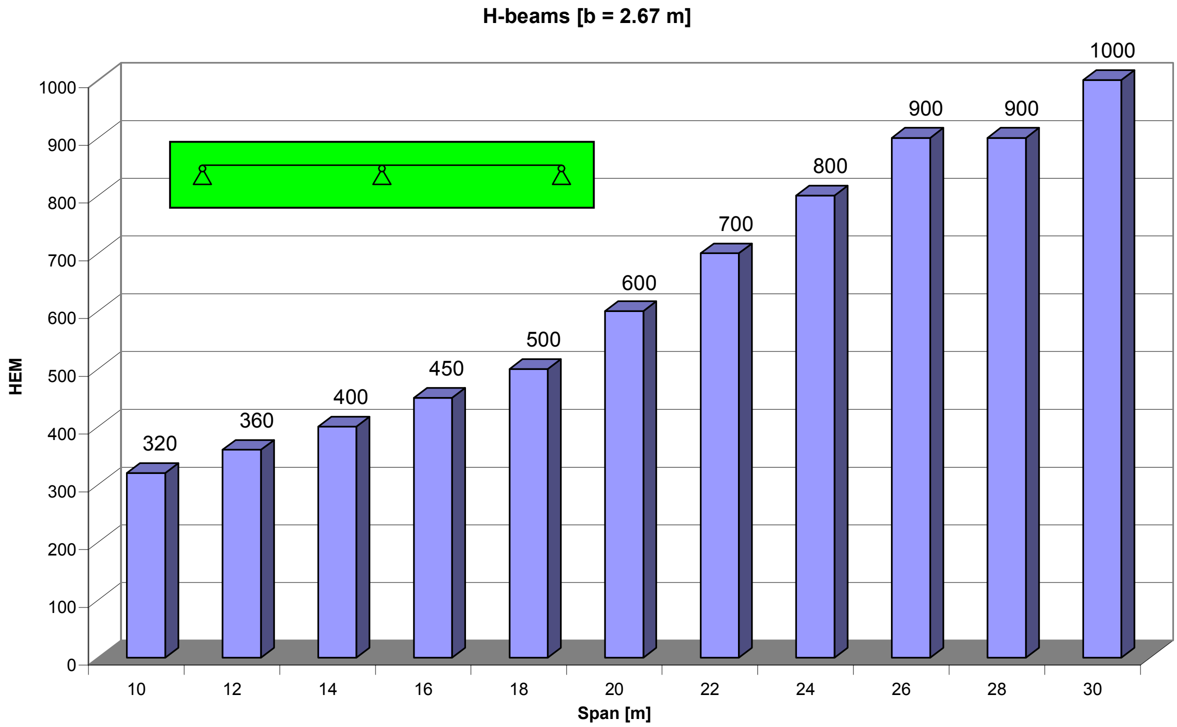


Figure 2-18: Design Chart: Two-span Bridge, HEM, b = 2.67 m, supports pinned

2.4.4 Two-span bridge with end supports clamped and the middle support pinned

2.4.4.1 H-beams with a bay width of 1.80 m

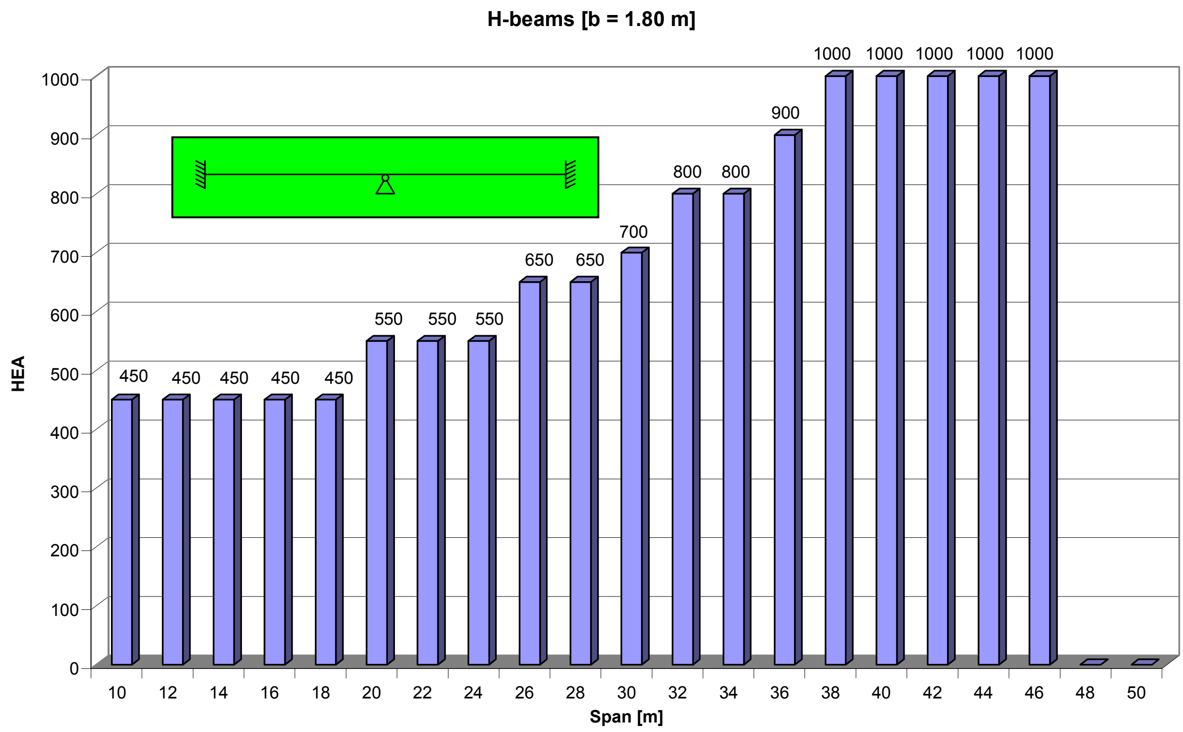


Figure 2-19: Design Chart: Two-span Bridge, HEA, b = 1.80 m, end supports clamped

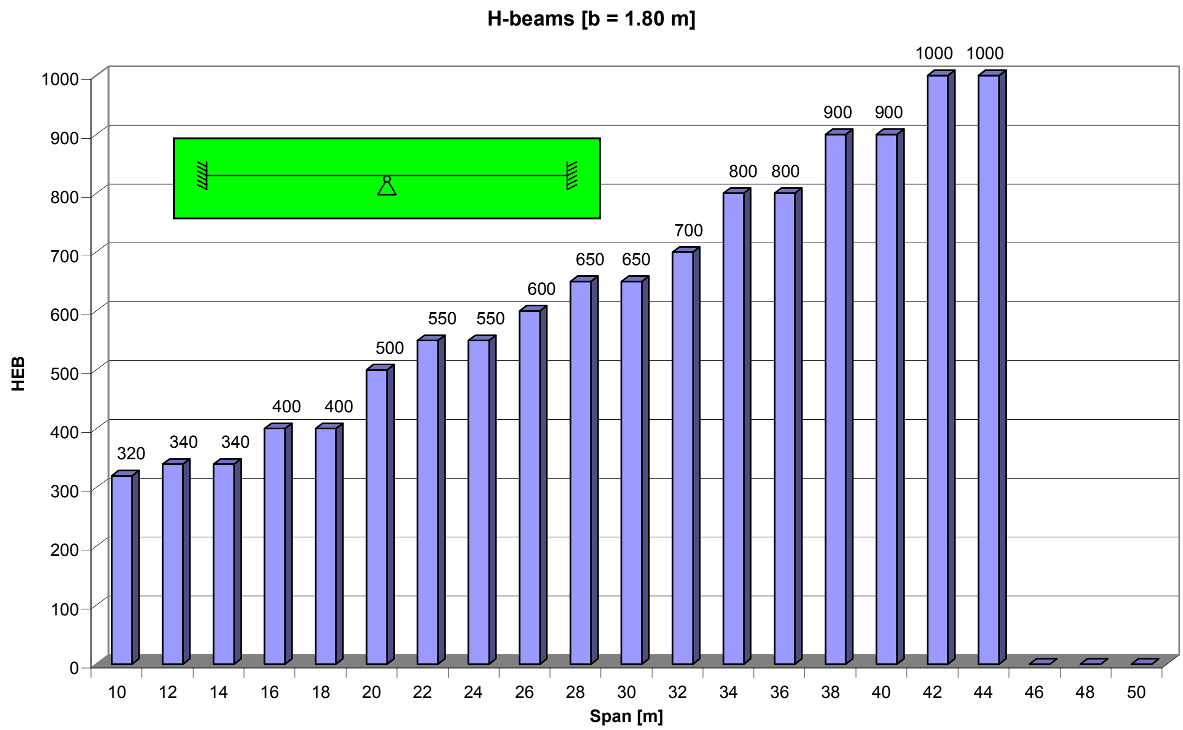


Figure 2-20: Design Chart: Two-span Bridge, HEB, b = 1.80 m, end supports clamped

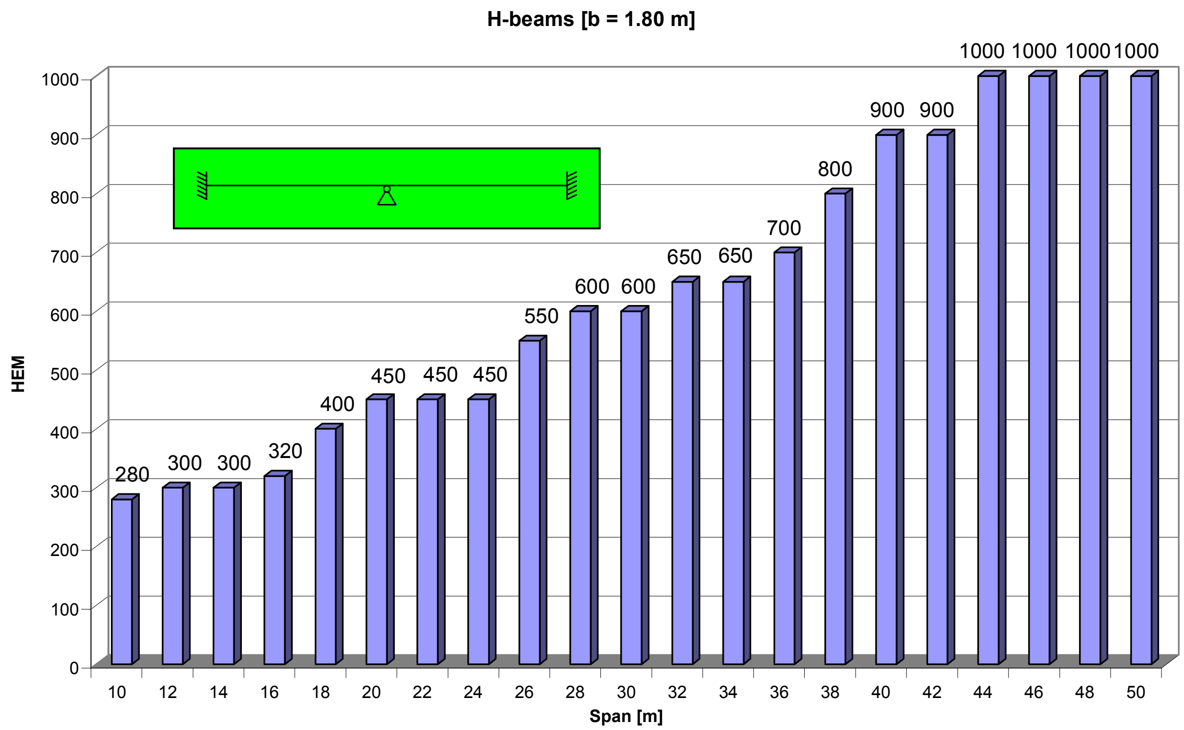


Figure 2-21: Design Chart: Two-span Bridge, HEM, b = 1.80 m, end supports clamped

2.4.4.2 H-beams with a bay width of 2.67 m

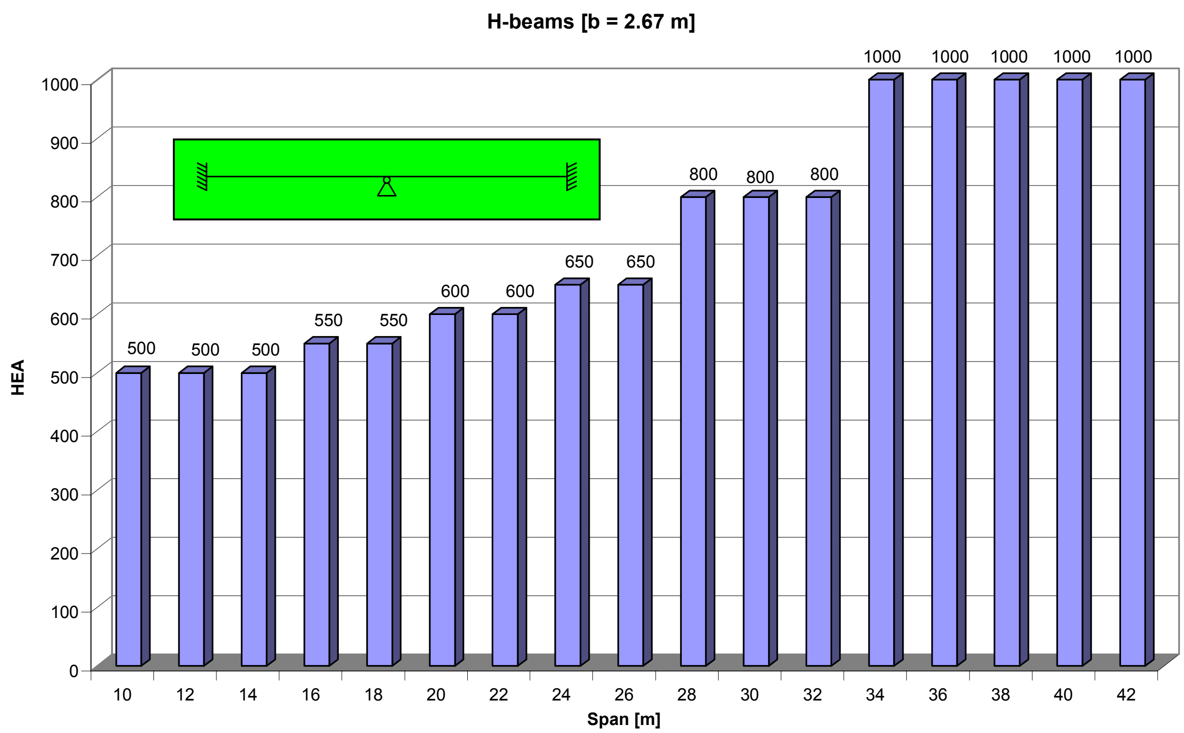


Figure 2-22: Design Chart: Two-span Bridge, HEA, b = 2.67 m, end supports clamped

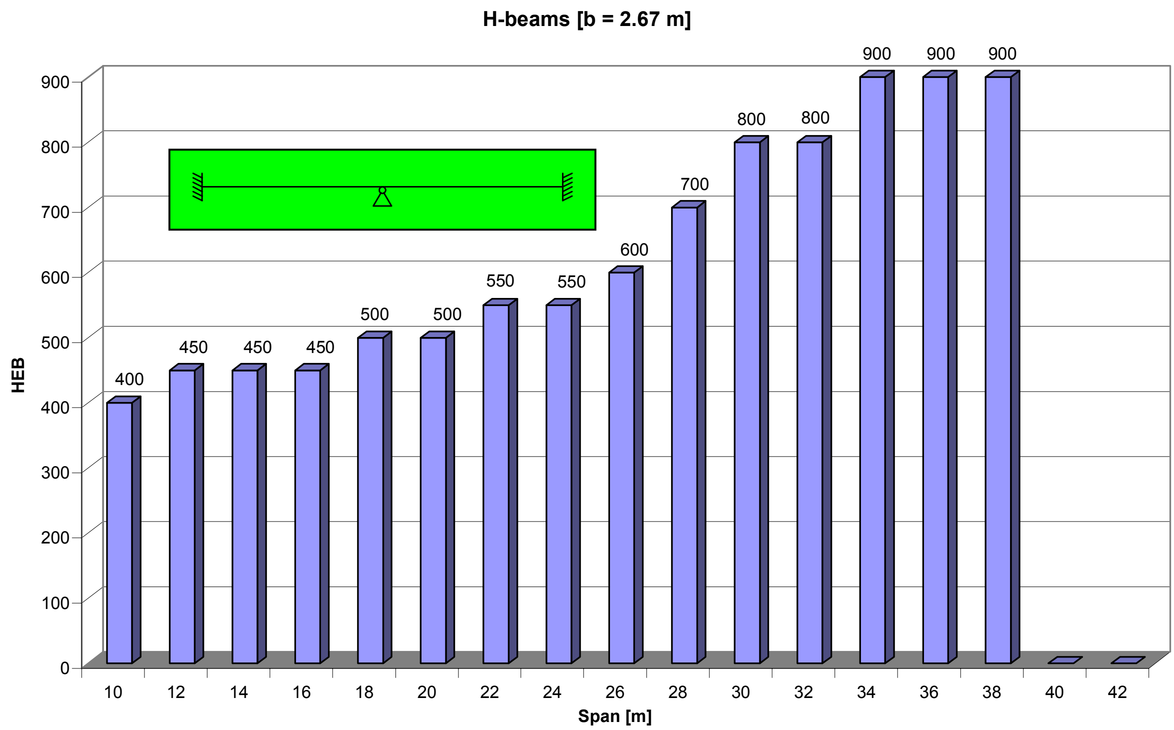


Figure 2-23: Design Chart: Two-span Bridge, HEB, b = 2.67 m, end supports clamped

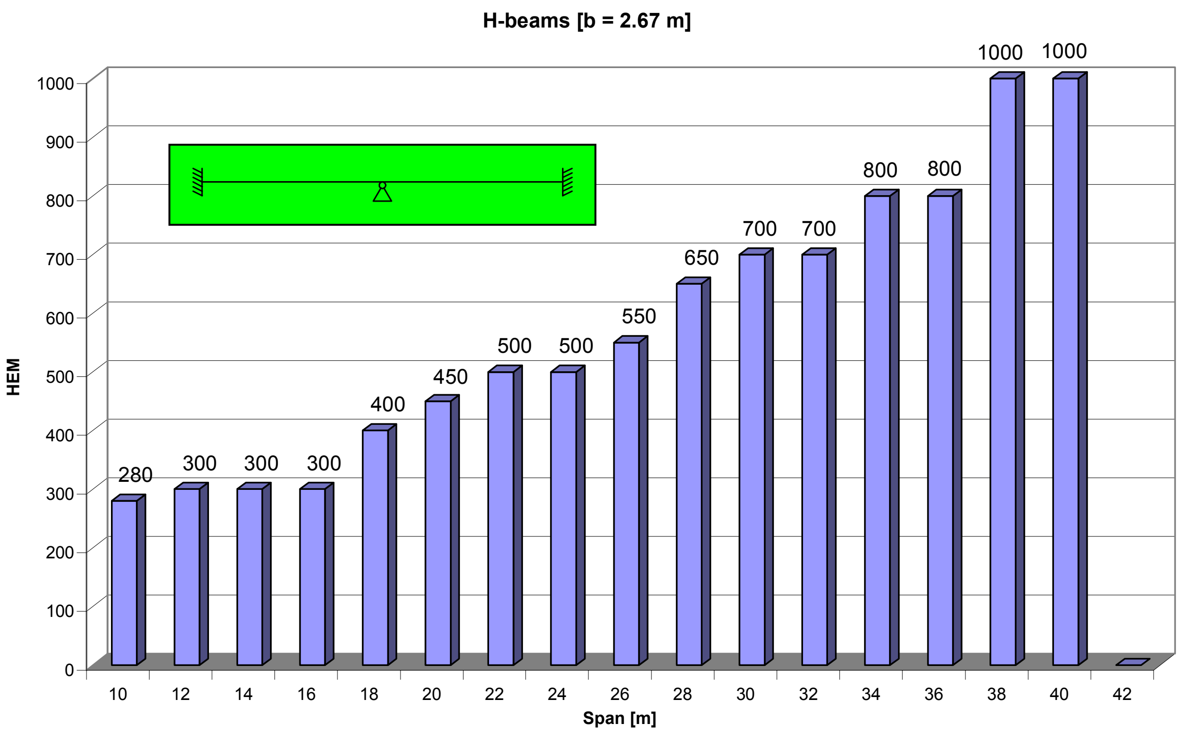


Figure 2-24: Design Chart: Two-span Bridge, HEM, b = 2.67 m, end supports clamped

3 SOFTWARE

In frame of the project a computer programme named CBD (Composite Bridge Design) has been developed to pre-design composite bridges for small and medium spans. The design is based on the Eurocode.

The Software is enclosed in ANNEX A.

The calculable constructions are rectangular, simple or multispan composite bridges consisting of a prefabricated steel beam and a concrete slab. The slab can be solid or constructed using prefabricated elements. The longitudinal and transverse reinforcement is considered not to be pre-stressed.

The program is designed in three layers:

- The first layer of the program is the main menu. All main actions of the program are initialised out of the main menu using pull-down menus.

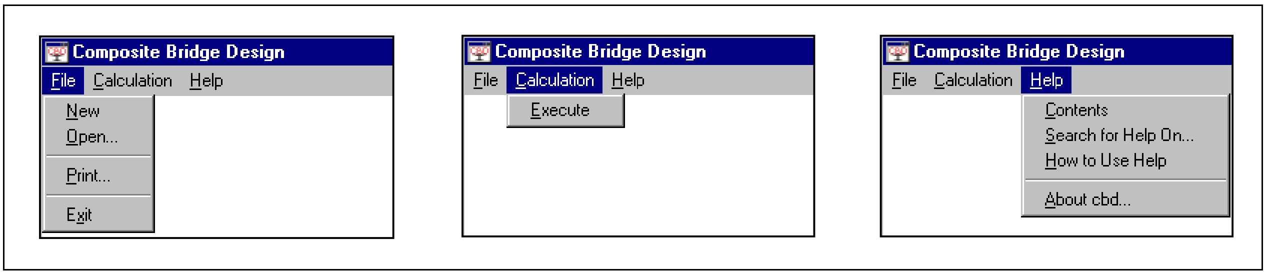


Figure 3-1: Pull-down menus of the main menu

One action is to call user-friendly dialog boxes where input data of a bridge is entered. The user is able to apply data for a complete new superstructure or edit a former specified bridge data input file. Additionally it is possible to print input or output files out of the main menu. The calculation is also started out of the main. Online-Help to the programme is available.

- The second layer of the program is represented by the dialog boxes, the user interface to apply data for the superstructure. The user can choose between the pre-design of a simple, two-span or multispan bridge with different support situations. The slab can be constructed as a solid slab or with the help of partially or fully prefabricated elements. The cross section in the field is differentiated from the cross section over the support. Rolled or welded steel girder sections are implemented. Choosing a rolled section means, that the program determines the section, increasing standard profile sections out of tables, until the checks for the ultimate limit state and serviceability limit state of EC3 and EC 4 are satisfied in all construction and final situations. For welded sections, the height of the profile is increased from 500 [mm] up to maximal 2500 [mm] in 10 [mm]-increments up to the point where the former specified limitations are satisfied.

  Common materials are included in the program and additionally high-tensile steel and high-strength concrete are implemented. Studs with a variable height, diameter and steel strength can be chosen as shear connectors. The possibility to use an alternative shear connector with a user specified bearing capacity is added. The distribution of the shear connectors is up to the user and has to be checked additionally. The specification of the loads are differentiated between their type, kind, place and temporary occurrence. The specification has an influence on safety and combination factors in the checks. The self-weight of the steel girder, the concrete slab and the actions due to creep and shrinkage at $t = \infty$ are estimated by the program itself. Permanent , traffic, temperature and breaking loads have to be added by the user. Generally loads have to be determined considering their transversal distribution over the cross section of the bridge. All checks are done for variable construction situation and final situations. Safety factors of different nations can be chosen. It is up to the user, if creep, shrinkage, moment distribution and a user specified deflection limit is taken into account.

- The third layer of the program is the calculation module. After opening an input file the pre-design calculation referring to EC 3 and EC 4 is initialised. During the calculation design points are defined at the quarter points and end points of each bridge span. All checks of the ultimate limit state and serviceability limit state are carried out at these points. Starting with the determination of the steel and composite cross sections for all construction and final situations, differentiated between field sections and sections over the support, the loads are applied and the inner forces are estimated.

Table 3-1: Construction Sequences for a solid- and fully prefabricated slab

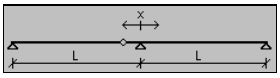
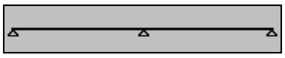
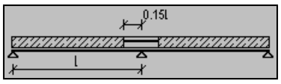
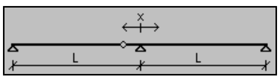
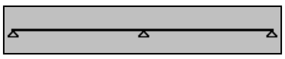
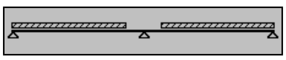
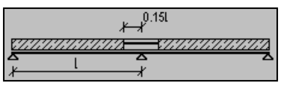
Construction Situation	Description	Load	
System 1	Multispan bridges: Steel beam with a hinge	Dead load of the steel beam	
System 2	Steel beam with welded hinge	Dead load of the solid slab or the fresh concrete and construction loads	
System 4	Full composite action considering cracks in concrete in the hogging moment area	Infrequent and frequent load	

Table 3-2: Construction Sequences for a partially prefabricated slab

Construction Situation	Description	Load	
System 1	Multispan bridges: Steel beam with a hinge	Dead load of the steel beam	
System 2	Steel beam with welded hinge	Dead load of the partially prefabricated slab	
System 3	Partial composite action	Dead load of the fresh concrete and construction loads	
System 4	Full composite action considering cracks in concrete in the hogging moment area	Infrequent and frequent load	

The relevant forces for each construction phase are multiplied by safety and combination factors and summed up. With the load combinations and the estimated cross sections the programme is able to perform the following checks of the ultimate limit state and serviceability limit state

Ultimate limit state of the steel girder referring to EC 3:

- ultimate resistance against positive bending
- ultimate resistance against negative bending
- ultimate resistance against positive bending moments taking interaction of shear into account
- ultimate resistance against negative bending moments taking interaction of shear into account
- ultimate resistance against shear
- ultimate resistance against torsional buckling

Ultimate limit state of the composite section referring to EC 4:

- ultimate resistance against positive bending
- ultimate resistance against shear
- ultimate resistance against negative bending moments taking interaction of shear into account
- bonding strength of the shear connectors, number of shear connectors
- ultimate resistance against torsional buckling

Serviceability limit state referring to EC 4:

- stress analysis
- crack width limitation and check of the minimal reinforcement
- deflection check

The results of each iteration step of the calculation are written into a similar named output file with the extension (.OUT). The last and relevant iteration step is summarised into a similar named data file with the extension (.RTF). The output is arranged such as the user is able to follow the result information easily. The check formats of the EC are included in the output file

INSTALL.bat installs the program in a former specified directory on the computer.

4 ADVANCED STANDARD SOLUTIONS

4.1 Abutments

4.1.1 Abutments with expansion joints

The most common way to arrange the end supports is using expansion joints which are supposed to absorb the movements from temperature, as well as any horizontal movements of the supports. These expansion joints often have to be replaced, due to leakage of the joints. Furthermore, they lead to discomfort for the drivers and passengers of the vehicles over-passing them. In other words, the bridge in many cases would be better of without any expansion joint.

4.1.2 Abutments without expansion joints

Instead of using expansion joints the movements from temperature changes can be absorbed in the backfilling behind the backwalls, which are directly connected to the steel girders. The concept is commonly used in Sweden for bridges up to some 50-70 m total length.

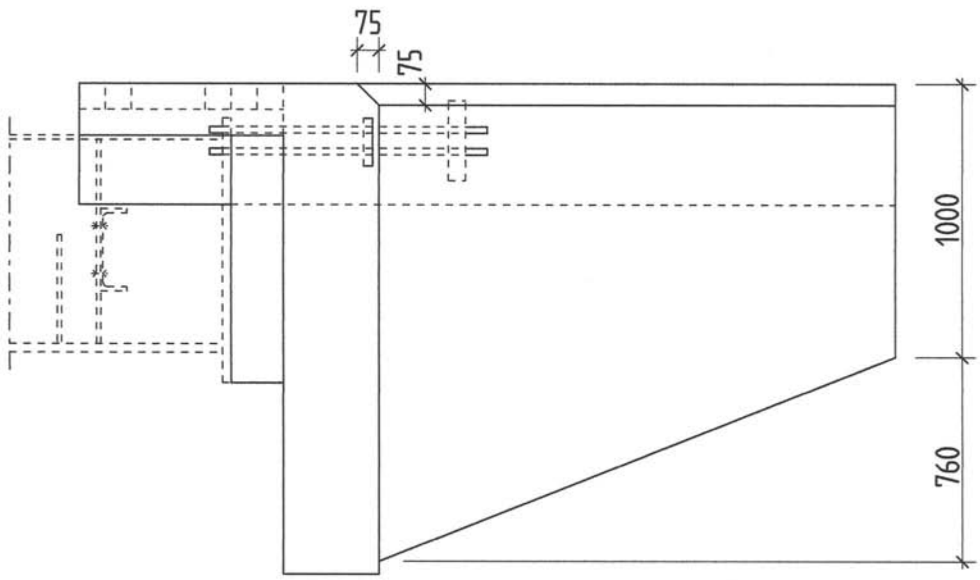


Figure 4-1: Prefabricated back-wall with side-wings, used for a composite bridge.

4.1.3 Prefabricated foundations

When old bridges are being replaced, it is sometimes essential that the construction time is short. In many cases the foundations can be prefabricated, and lifted out in their final position. The Figure 4-2 shows the prefabricated foundations of such a bridge, which contributed to a traffic disturbance of only 30 hours.

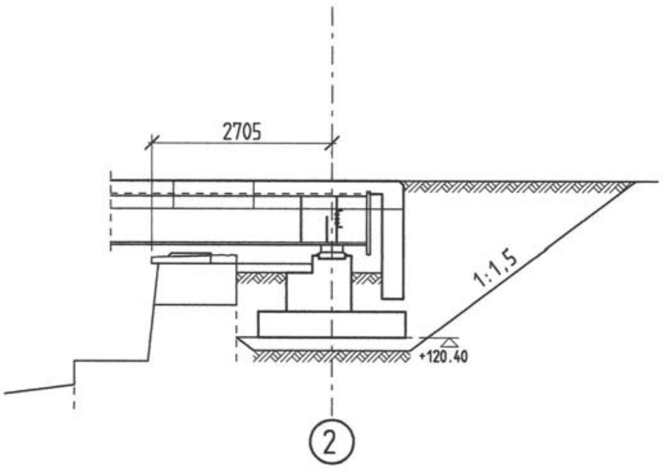


Figure 4-2: Prefabricated piers used to minimise the construction time

4.1.4 Integral abutments

Experience from the U.S. shows that bridges with integral abutments are increasingly outclassing the traditional bridges with joints, the former being not only less expensive to maintain, but also more affordable to build. The concept is commonly used in USA, up to 120 m total length of the bridge.

Analysing the load bearing capacity of piles subjected to lateral movements is complex as it contains two co-dependent elements; the flexural pile and the soil. To further complicate matters, soils are often inhomogeneous.

Within a research project at Luleå University of Technology, a bridge was built in the northern Sweden, completed in September 2000.

Figure 4-3: Bridge with integral abutments.

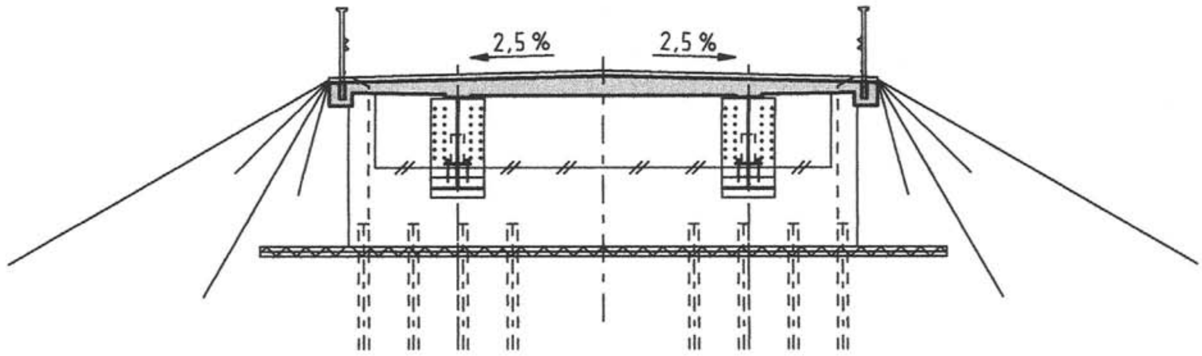


Figure 4-4: Cross section of a bridge with integral abutments. Eight X-shaped steel piles 180x24 mm carry the vertical load.

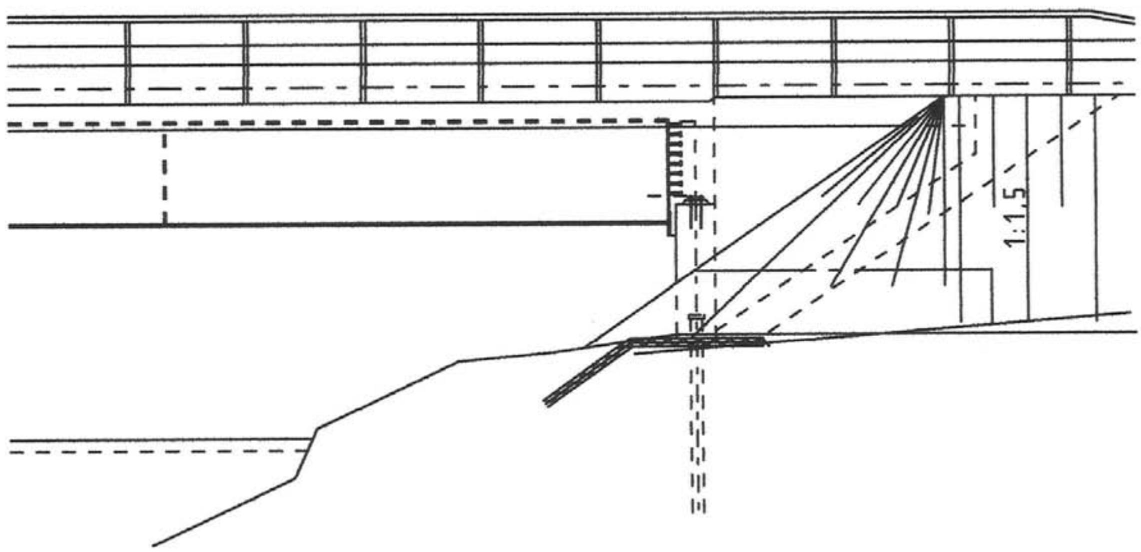


Figure 4-5: Elevation of the bridge shown in Figure 3-5, where the steel piles are positioned in a straight line.

4.2 Main Girder

4.2.1 Hybrid girder

Hybrid plate girders may be used in steel and composite bridges provided the limitations and the criteria for the resistance at the ultimate limit states, the serviceability limit states and fatigue, are satisfied.

At the stage of drafting of this guide, the reference is the ENV's versions of Eurocodes and especially : ENV 1993-1-5 (November 1997), ENV 1993-1-1 (February 1992), ENV 1993-2 (October 1997) and ENV 1994-2 (December 1997). The designer should take any modification adopted in the final EN version of these documents into account.

4.2.1.1 Limitations

4.2.1.1.1 Cross-section

This guide concerns the design of hybrid plate girders for small and medium span bridges. The application of this guide covers the design of the main elements. The main elements can be symmetrical or non-symmetrical I section girders or box girders without longitudinal stiffeners in the flange.

4.2.1.1.2 Yield strength ratio

The particular focus of this guide is towards hybrid plate girders which are built-up sections where the flange plate yield strength f_{yf} is greater than that of the web plate f_{yw}.

According to ENV 1993-1-5, the use of hybrid girders is explicitly covered in Clause 2.2.2(6). According to this clause hybrid girders may have a flange yield strength f_{yf} up to **2** f_{yw}. This limitation is based on available tests in this field.

$$f_{yw} \leq f_{yf} \quad and \quad \frac{f_{yf}}{f_{yw}} \leq 2 \tag{4–1}$$

4.2.1.1.3 Weld metal

The fabrication and erection should be in accordance with ENV 1993-2 Chapter 7.

For hybrid plate girders and at the web-to-flange junctions, the weld metal of the fillet weld should match at least with the metal of the web in terms of the mechanical properties. Elsewhere, the provisions of ENV 1993-1-1 Clause 6.6 and ENV 1993-2 Clause 6.5 should be applied.

4.2.1.2 Ultimate limit states other than fatigue

4.2.1.2.1 Classification of hybrid plate cross-section

For the classification of a hybrid plate cross-section, the basis given in ENV 1993-1-1 Clause 5.3 should be applied.

a) Compression flange

The Table 5.3.1 of ENV 1993-1-1 should be used for the classification of the compression flange. In addition, compression flanges connected to concrete flange according to ENV 1994-2 may be classified as Class 1 sections.

b) Web

For the classification of the cross section the plastified zone in the web should be taken into account. According ENV 1993-1-5 Clause 2.2.2 (6-b), the classification and the calculation of the effective area of the web should be determined with f_{yf} rather than f_{yw} to consider the effect due to the plastified zone in the web.

4.2.1.2.2 Bending moment resistance

The increase of flange stresses caused by yielding of the web should be taken into account as indicated in ENV 1993-1-5 Clause 2.2.2 (6-a).

Depending on the class of the cross section, the distribution of the stresses should be one of those given in Figure 4-6 to Figure 4-8.

The effect of shear lag should be considered by the use of an effective width of the slab and of the flanges if necessary.

a) Class 1 or 2 cross-sections

The bending moment resistance should be calculated (see Figure 4-6) from a plastic distribution of the stresses in the gross cross-section according to ENV 1993-1-1 Clause 5.4.5.2 for steel cross-sections and ENV 1994-1-1 Clause 4.4 for composite cross-sections.

b) Class 3 cross-sections

The bending moment resistance should be calculated (see Figure 4-7) from an elastic distribution of the stresses in the gross cross-section according to ENV 1993-1-1 Clause 5.4.5.2 for steel cross-sections and ENV 1994-1-1 Clause 4.4 for composite cross-sections.

c) Class 4 cross-sections

In this guide, Class 4 cross-section concerns only the web due to the limitation adopted in Figure 4-8. To calculate the effective area of the web, ENV 1993-1-5 Clause 2.2.2 (6-b) indicates the use of f_{yf} rather than f_{yw} . The verification should be according to the expression (2-1) in Clause 2.2.1(2) of ENV 1993-1-5. In Figure 4-8 are given some examples of possible stress distributions.

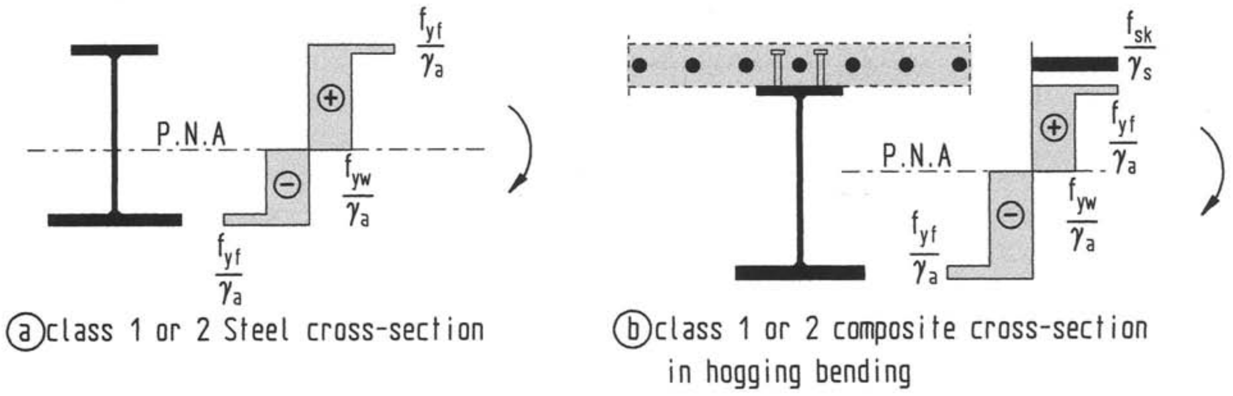


Figure 4-6: *bending moment resistance of a plastic section for a steel and composite beam*

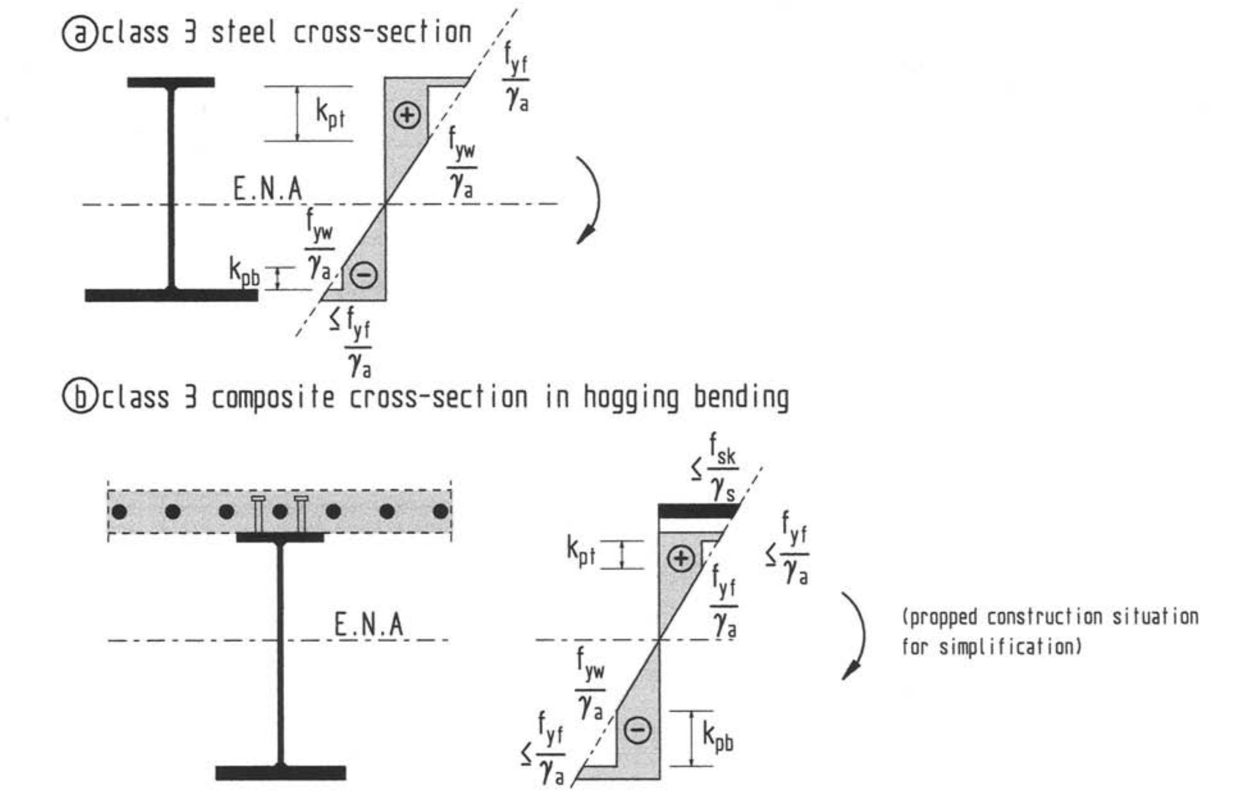


Figure 4-7: *bending moment resistance of an elastic section for a class 3 steel and composite section*

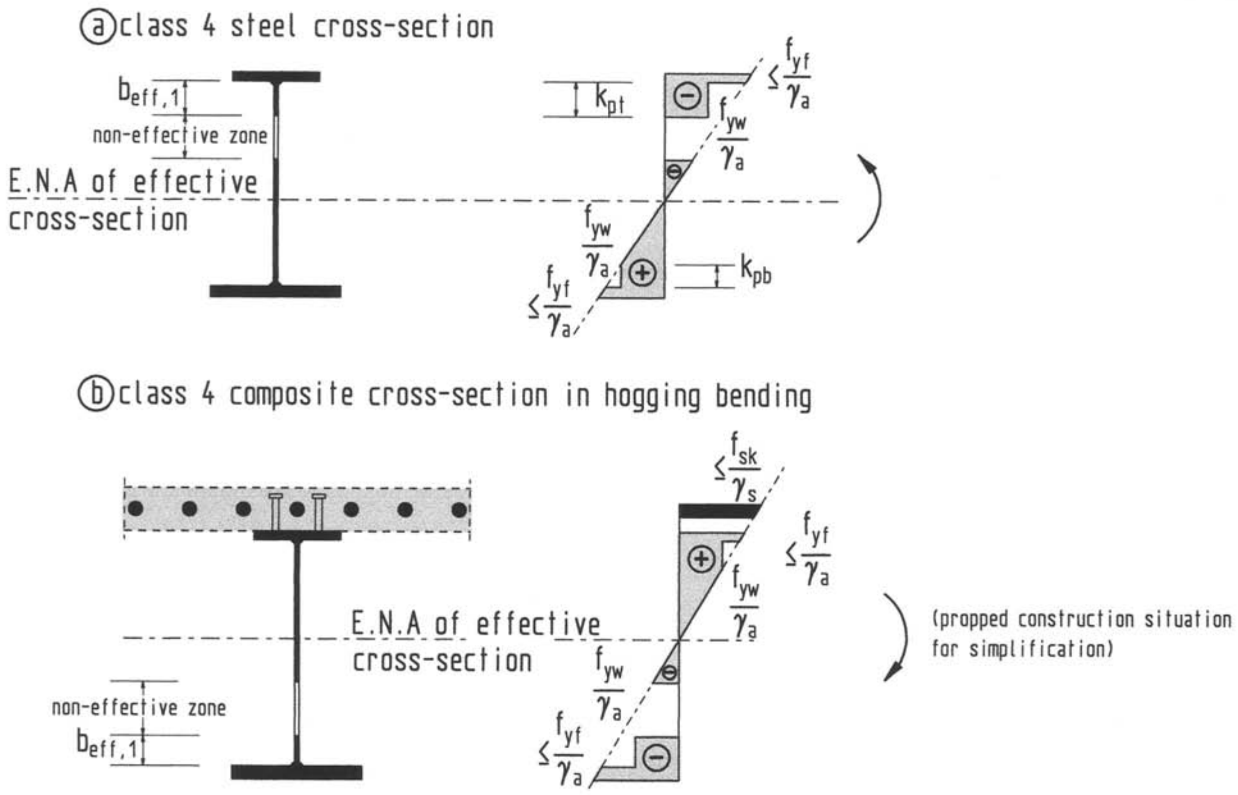


Figure 4-8: bending moment resistance of an elastic section for a class 4 steel and composite section

4.2.1.2.3 Shear resistance, shear buckling resistance and interaction between shear and bending

The shear resistance, shear buckling resistance and the interaction between shear and bending should be calculated as for homogeneous girders according to ENV 1993-2 and ENV 1993-1-5 Clauses 2.2.1 and 4.3 respectively using f_{yf} for flanges and f_{yw} for webs.

4.2.1.2.4 Resistance of webs to transverse forces and interaction

The resistance should be calculated according to ENV 1993-1-5 Clause 4.4.

The interaction between transverse force, bending moment and axial force should be verified according to ENV 1993-1-5 Clause 2.2.3.2.

4.2.1.3 Serviceability limit state

4.2.1.3.1 Stresses

At serviceability limit state, ENV 1993-2 Clause 4.3(1) should be applied for hybrid plate girders.

The nominal stresses in all elements of the bridge resulting from characteristic (rare) load combinations $\sigma_{Ed,ser}$ and $\tau_{Ed,ser}$, should be limited as follows :

$$\sigma_{Ed,ser} \leq f_y / \gamma_{M,ser} \tag{4–2}$$

$$\tau_{Ed,ser} \leq (f_y / \sqrt{3}) / \gamma_{M,ser} \tag{4–3}$$

And for the interaction

$$\left[(\sigma_{Ed,ser})^2 + 3(\tau_{Ed,ser})^2\right]^{0,5} \leq f_y / \gamma_{M,ser} \tag{4–4}$$

4.2.1.3.2 Web breathing

Hybrid plate girders should be verified as homogeneous girders against web breathing. Reference should be made to ENV 1993-2 Clause 4.4.

4.2.1.3.3 Deformations (Deflections)

In general, the effect of the yielding in the web on the deflection is negligible and may be disregarded in the calculation of the deflections of steel and composite bridges with hybrid plate girders.

When accurate deflections are required, an analysis taken into account the plastified zone in the web should be adopted.

4.2.1.4 Fatigue

This paragraph proposes some recommendations and guidelines concerning the fatigue assessment of hybrid plate girders in steel and composite bridges with small and medium spans. By reference to homogeneous girders the main difference is the effect of the plastified zone in the web and particularly at the web-to-flange junction on the fatigue resistance of different details.

4.2.1.4.1 Limitations

All nominal stresses in the flanges should be within the elastic limits of the material. The range of the design values of such stresses shall not exceed $1.5\, f_{yf}/\gamma_{Mf}$ for normal stresses or $1.5\, f_{yw}/(\sqrt{3}\,\gamma_{Mf})$ for shear stresses.

For hybrid plate girders, the stress ranges shall be determined by an elastic analysis of the structure under fatigue loading (ENV 1993-1-1). Stress calculations should take into account the effect of shear lag but normally without the effect of local plate buckling. For composite bridges the effect of the cracking of the concrete should also be considered.

4.2.1.4.2 Details

Figure 4-10 gives the main detail categories for hybrid plate girders for bridges.

Cope holes (see Figure 4-9 and Figure 4-10) can be adopted for hybrid plate girders. Fatigue resistance of this detail may be enhanced by the execution of a mouth shape hole rather than a circular one as indicated in Figure 4-10 and plugging the hole with weld.

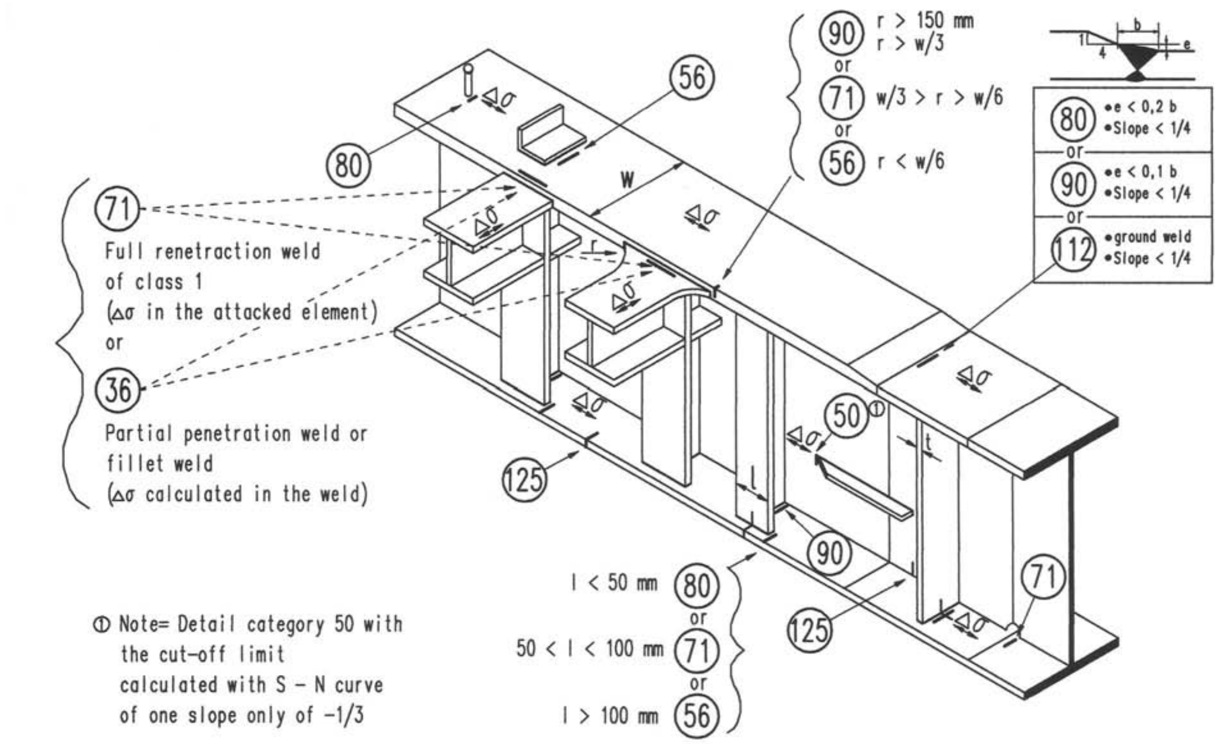


Figure 4-9: Cope holes in hybrid girders

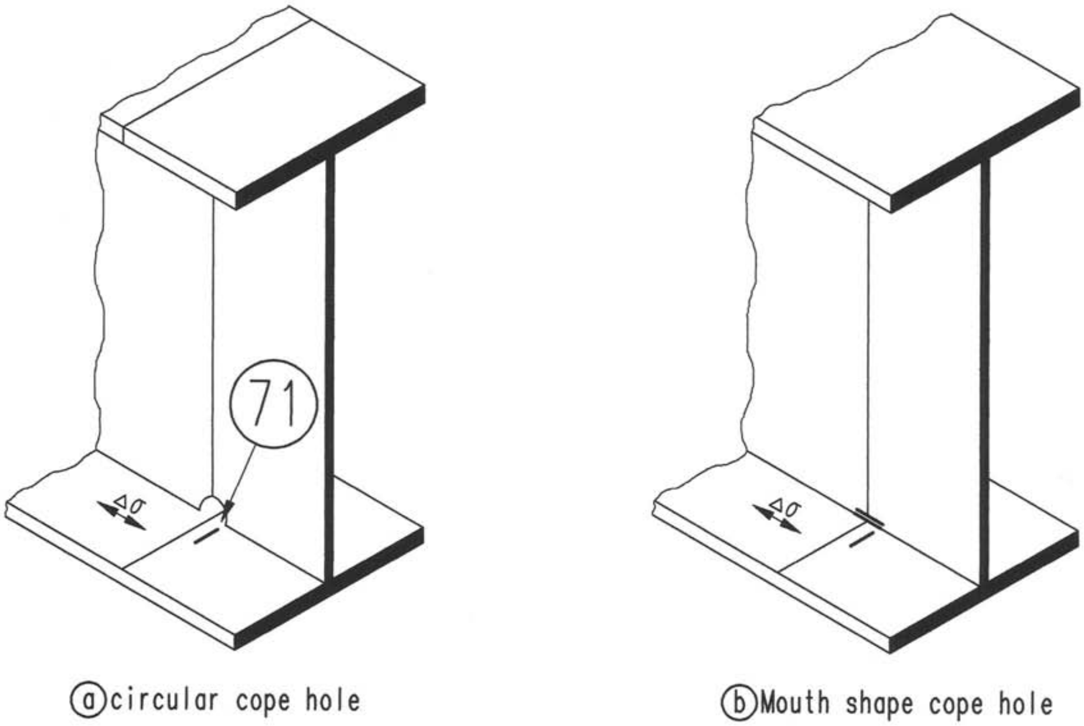


Figure 4-10: Circular and mouth shaped cope holes

4.2.2 Studs as shear connectors

4.2.2.1 General

For the shear connection and concrete grades in accordance with ENV 1994-2:1997, headed studs with diameters of 19,22 and 25 mm may be used. The design resistance for the ultimate limit state and the limit state of fatigue should be determined according to 4.2.2.2 and 4.2.2.3. In case of partially prefabricated elements or precast slabs the influence of appropriate infill around the studs should be taken into account, if the infill around the studs is smaller than 25 mm. See Chapter 4.2.2.2.

4.2.2.2 Design resistance of headed studs in combination with precast slabs

The design resistance of headed studs, automatically welded in accordance with EN ISO 14555 and EN ISO 13918 is given by equations (3-5) and (3-6) whichever is smaller.

$$P_{Rd} = \kappa_p \left(0{,}8\, f_u \frac{\pi d^2}{4} \right) \frac{1}{\gamma_v} \tag{4–5}$$

$$P_{Rd} = \kappa_p \left(0{,}25\, \alpha\, d^2 \sqrt{(f_{ck}\ E_{cm})} \right) \frac{1}{\gamma_v} \tag{4–6}$$

where :

γ_v = 1,25 is the nominal partial safety factor for the ultimate limit state ;

d is the diameter of the shank of the stud ;

f_u is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm^2 ;

f_{ck} is the characteristic cylinder strength of the concrete at the age considered ;

E_{cm} is the mean value of the secant modulus of the concrete in accordance with EN 1992-1 for normal and for lightweight concrete ;

α = 0,2 [(h_{sc} / d) + 1] for $3 \le h_{sc} / d \le 4$

α = 1 for $h_{sc} / d > 4$

h_{sc} is the overall nominal height of the stud,

κ_p is a reduction factor in case of use of precast slabs taking into account the influence of the distance between the stud and the side faces of the precast element. The reduction factor should be determined from Figure 4-11 where $e_s \ge 5$ mm is the distance between the outside of the shank of the stud and the side or front face of the precast element.

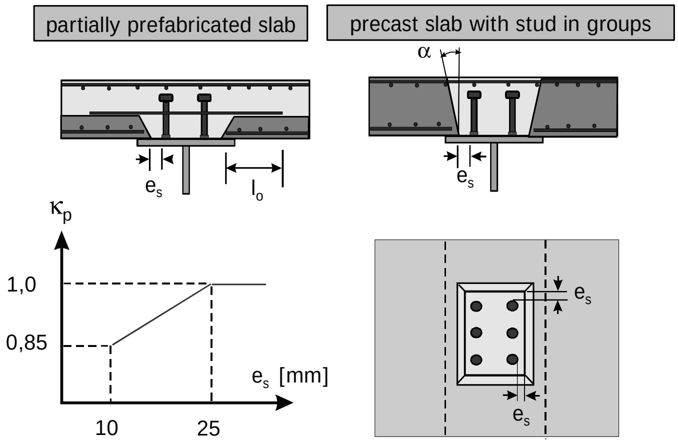


Figure 4-11: Reduction factor κ_p in case of partially or totally prefabricated slabs

An inclination of the side faces of the openings in precast elements should be avoided. The design resistance according to equations (3-5) and (3-6) should be reduced by 20%, if the side faces are inclined according to Figure 4-11 and $\tan\alpha \leq 0{,}20$.

4.2.2.3 Fatigue verification

For solid slabs, partially prefabricated slab and precast elements with studs in groups, the fatigue strength of headed studs should be determined in accordance with 6.3.8 of ENV 1994-2 but with a characteristic fatigue strength $\Delta\tau_c$=90N/mm^2

4.2.3 Dismountable shear connectors

4.2.3.1 General

For temporary bridges and where during the design life an exchange of the concrete deck should be considered, shear connection with headed studs in combination with an additional steel plate and bolts of grade 10.9 can be provided (see Figure 4-12). The distance between the edge of the head of the bolt and the edge of the steel plate should be not less than 20mm.

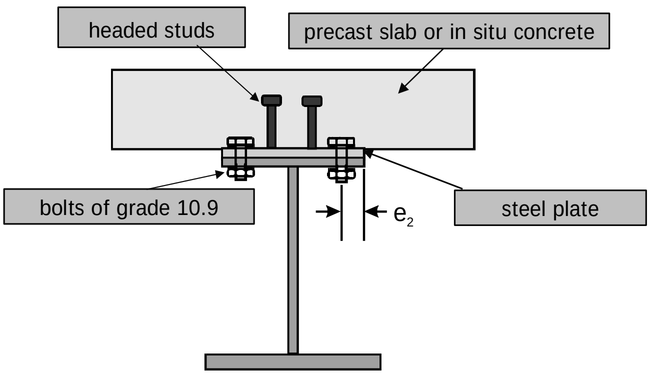


Figure 4-12: Dismountable shear connection

4.2.3.2 Design resistance for longitudinal shear

At ultimate limit states other than fatigue, the design resistance for longitudinal shear should be determined from the resistance in the critical sections I-I and II- II according Figure 4-13.

In the critical section I-I the design resistance per unit length is given by:

$$V_{L,Rd,I-I} = n_s\ P_{Rd,stud} + n_b\ P_{cd,bolt} \tag{4–7}$$

where

	$P_{Rd,stud}$ is the shear resistance according to equations (3-5) and (3-6) with κ_p= 1,0 in case of in situ solid concrete slabs and κ_p according to Figure 4-11 for partially or fully prefabricated slabs,
$P_{c,bolt}$	is the resistance of the heads of the bolts given by

$$P_{cd,bolt} = \sigma_{c,Rd}\ s_b\ h_{hb} \quad \text{with} \quad \sigma_{c,Rd} = \beta\ f_{ck} / \gamma_c \tag{4–8}$$

where

s_b and k_b	are the width and height of the head of the bolt and β is a factor taking into account the effect of local stress concentration in front of the head of the bolt,
β	factor to take into account effects of concentrated stresses which should be taken as β=2,0 ,
n_s and n_b	are the number of studs and bolts per unit length.

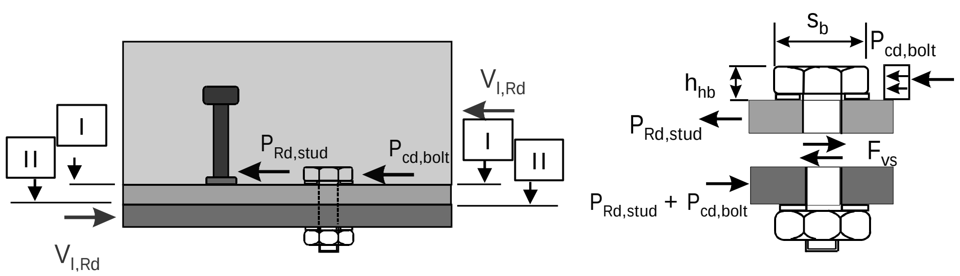


Figure 4-13: Model for the determination of longitudinal shear resistance

The longitudinal shear resistance $V_{l,Rd,II-II}$ in the critical section II – II is given by the shear resistance of the bolts and should be determined in accordance with 6.5.5 of ENV 1993-1-1: 1:1992. It should be verified that the longitudinal shear resistance in the critical sections I-I and II is in the range

$$0{,}9 \le \frac{V_{l,Rd,I-I}}{V_{l,Rd,II}} \le 1{,}1 \tag{4–9}$$

4.2.3.3 Fatigue verification

The fatigue resistance of headed studs for the critical section I-I should be verified according 4.2.3 for an equivalent constant shear stress range $\Delta\tau_E^*$ for 2 million cycles in accordance with ENV 1994-2: 1997, where $\Delta\tau_E^*$ is given by

$$\Delta\tau_E^* = \frac{\Delta V_{E,l}}{V_{l,Rd,I-I}} 1{,}1\ P_{Rd,stud} \frac{1}{\pi d^2/4} \qquad (4\text{–}10)$$

where

$P_{Rd,stud}$	is the design resistance of the headed stud according to 3.2.2.2,
$V_{l,Rd,I\text{-}I}$	is the longitudinal shear resistance in the critical section II,
n_s	is the number of headed studs per unit length,
$\Delta V_{l,E}$	is the equivalent longitudinal shear range at two million cycles, determined with a slope m=8 of the fatigue strength curve,
d	is the diameter of the shank of the stud.

The fatigue verification of bolts in the critical section II-II should follow Section 9 of ENV 1993-2: 1997.

4.2.4 Bolted Connections of Girders in Span

4.2.4.1 General

There are several positions to connect two girders of multi-beam bridges to each other with a bolted connection in span: these positions are limited on the one hand by the zero crossover of the maximum and on the other hand by the zero crossover of the minimum moment process (Figure 4-18).

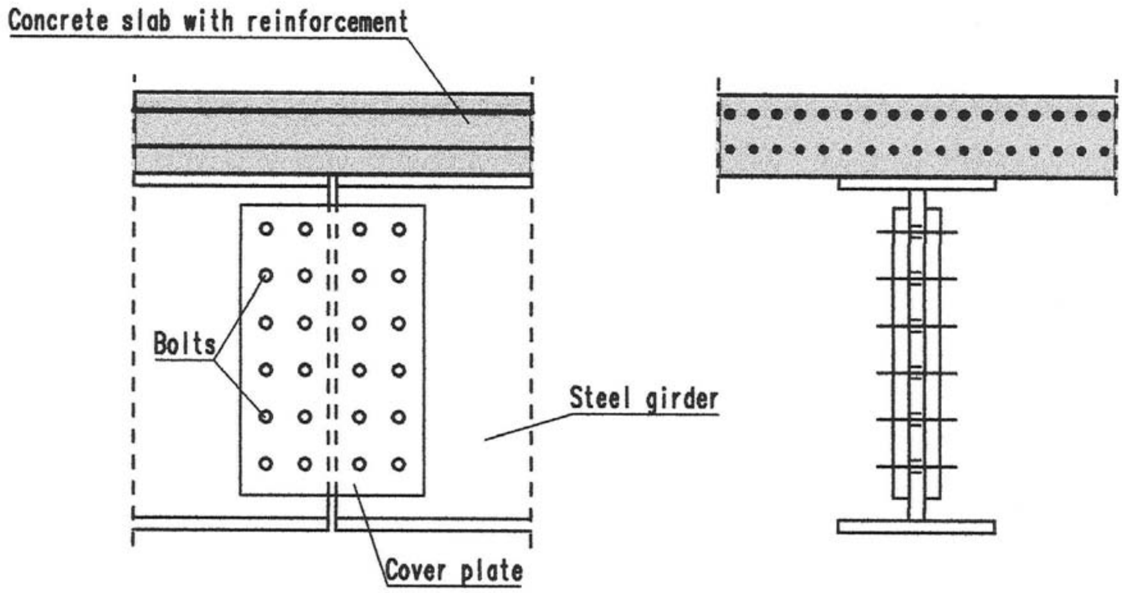


Figure 4-14: Bolted connection of two girders for multi-beam bridges

4.2.4.2 Static design

If the connection is in the area of the positive moment distribution, the failure occurs in the bolts, the cover plates or the web of the girder. If the connection is in the area of the negative moment distribution, the failure occurs in the reinforcement of the concrete slab.

4.2.4.2.1 Dimensioning of the Reinforcement

The static dimensioning of the reinforcement is described in [EC 2.1].

Only if a negative moment is acting in the connection section, the reinforcement takes up force.

4.2.4.2.2 Dimensioning of the bolts

4.2.4.2.2.1 Ultimate Limit State

The design resistance of the bolts is described in [EC 3.1.1 – 6.5.5].

4.2.4.2.2.2 Serviceability Limit State

High strength bolts in slip-resistant connections are described in [EC 3.1.1 – 6.5.8].

The design slip resistance of a preloaded high-strength bolt is dependent of the slip factor μ, which itself dependent on the specified class of surface treatment as given in Reference Standard 8. The high strength bolts, conform to Reference Standard 3, shall be controlled tightened, in conformity with Reference Standard 8, to at least the specified minimum design preloading force $F_{p,Cd}$, which results to

$$F_{p,Cd} = 0.7\, f_{ub}\, A_s \tag{4–11}$$

with: f_{ub} tensile strength of the bolt

A_s tensile stress area of the bolt

If other types of preloaded bolts or other types of preloaded fasteners were used, the design preloading force $F_{p,Cd}$ shall be agreed between the competent authority.

All tests have shown, that no slip has been occurred in the bolted connection during the calculated lifetime.

4.2.4.2.3 Positive Resistance Moment of the Bolted Connection for Elastic Design

In this category preloaded high strength bolts with controlled tightening shall be used. Slip shall not occur at Serviceability Limit State (SLS). The serviceability shear design load should not exceed the design slip resistance. Slip is accepted in the Ultimate Limit State (ULS). The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance.

The neutral axis of the connection section is the same as the neutral axis of the concrete-girder-cross section (without cover plates and bolts) in the area of the connection section.

Because of the test results it is not necessary to calculate the plastic resistance of the bolted connection.

The resistance results of the slip resistance of the bolts and the resistance of the concrete. The maximum stress in the concrete must be lower than the design compressive strength of the concrete f_{cd} multiply with α.

Therefore the positive resistance moment results to

$$M_{Rd,elast} = \sum F_{s,Rd}\; n\; y_i + F_c\, y_i \tag{4–12}$$

with: n row of bolts

y_i distance of each bolt or the force of the concrete to the neutral axis

$F_{s,Rd}$ slip resistance of the bolts

F_c force of the concrete

f_{cd} design compressive strength of the concrete

α coefficient taking account of long term effects on the compressive strength and unfavourable effects resulting from the way the load is applied

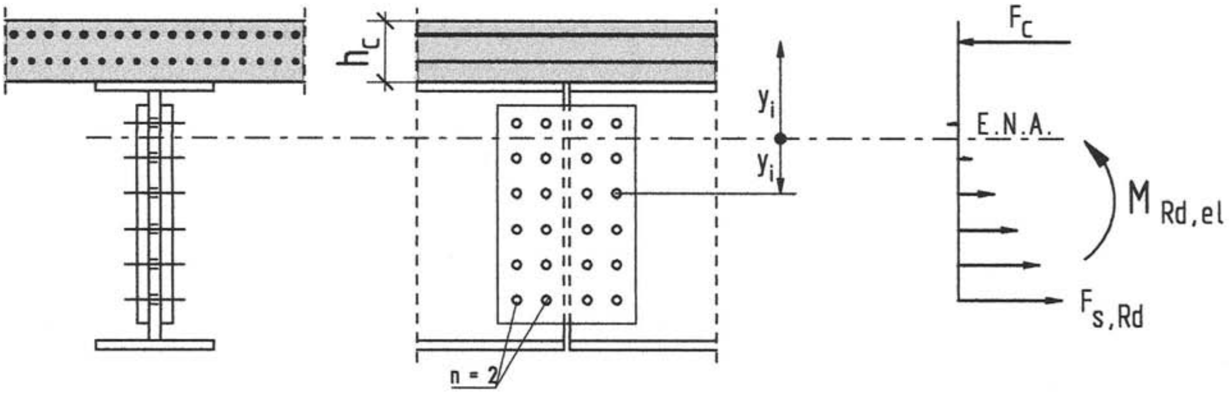


Figure 4-15: Positive resistance of a bolted connection for elastic design

4.2.4.2.4 Negative Resistance Moment of the Bolted Connection for Elastic Design

In this category preloaded high strength bolts with controlled tightening shall be used. Slip shall not occur at Serviceability Limit State (SLS). The serviceability shear design load should not exceed the design slip resistance. Slip is accepted in the Ultimate Limit State (ULS). The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance.

The neutral axis of the connection section is the same as the neutral axis of the concrete-girder-cross section (without cover plates and bolts) in the area of the connection section.

Because of the test results it is not necessary to calculate the plastic resistance of the bolted connection.

The resistance results of the slip resistance of the bolts and the elastic resistance of the reinforcement (the concrete is fully cracked). The maximum stress in the reinforcement must be lower than the design strength of the reinforcement..

Therefore the negative resistance moment results to

$$M_{Rd,elast} = F_{s,Rd}\, n \sum y_i + A_{si}\ \sigma_{si} \sum y_i \tag{4–13}$$

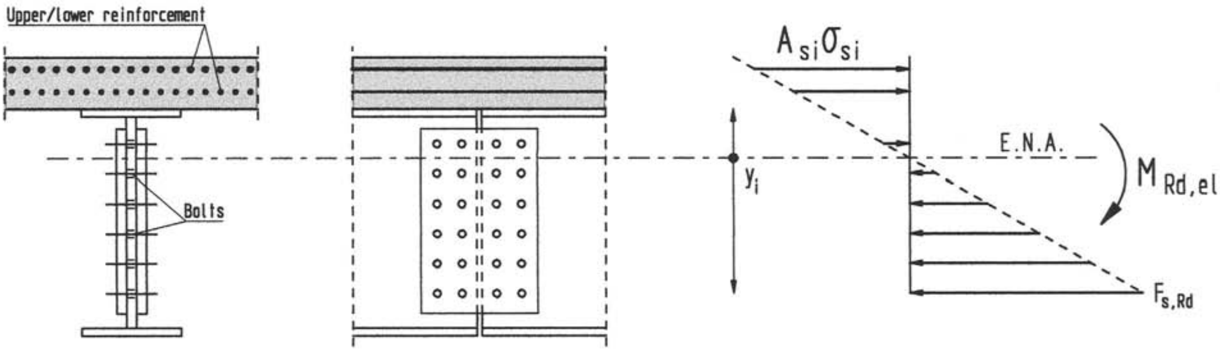


Figure 4-16: Resistance of a bolted connection for elastic design (negative moment)

with: n row of bolts at one girder

y_i distance of each bolt to the neutral axis

$F_{s,Rd}$ slip resistance of the bolts

F_s force in the reinforcement

A_s total area of the longitudinal reinforcement

f_{sd} design strength of the reinforcement

4.2.4.2.5 Critical Resistance Moment of the Web

Here the failure should be expected (following EC 3) in the web of the girder. Due to the test results it is possible to neglect the holes in the web and to calculate the resistance of the web in a distance of the thickness of the cover plate t_{covpla} plus half the diameter of the washer d_{wa} (Figure 4-17).

Due to the results of the tests, for a positive action moment cracks have been detected in the cover plates and for a negative action moment in the web of the girder. No slip occurred in both cases.

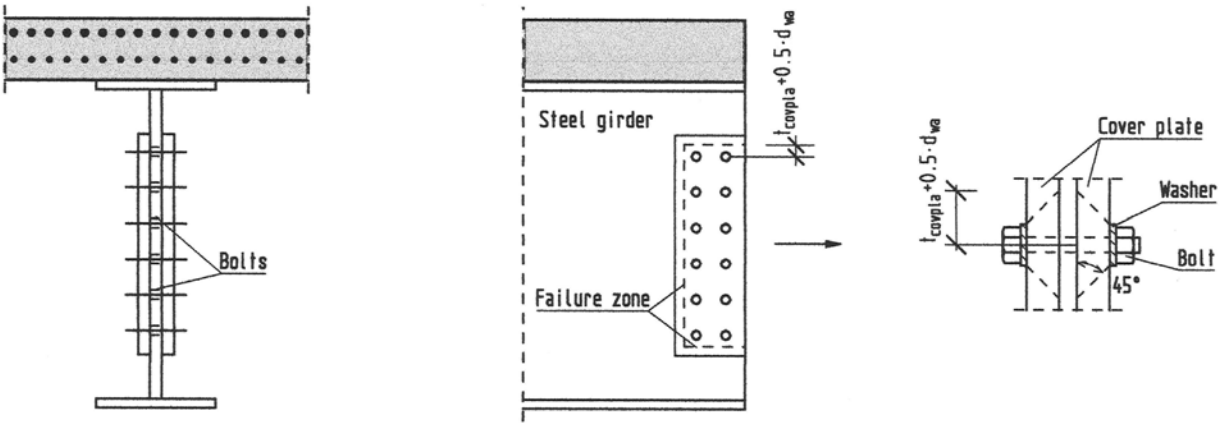


Figure 4-17: Failure zone of the web

If it is necessary to raise the critical resistance of the web, than it is possible to elongate the cover plates and to use more bolts.

4.2.4.2.5.1 Resistance Moment of a Bolted Connection for Elastic Design

In calculation it could be supposed, that the neutral axis is located in the centre of the concrete slab. So the resistance of the concrete could be neglected. This is an assumption to maximise the thickness of the cover plate (4.2.4.2.6).

Therefore, if there is no slip, the resistance moment for elastic design for the full section results to

$$M_{Rd,elast} = \frac{f_{yk}}{\gamma_{M0}} \frac{\sum k\, A_i\, y_i^2}{y_{max}} \tag{4–14}$$

with: k 1.0 in tension and 1/√3 in shear

y_i distance of each bolt to the neutral axis

y_{max} max. distance of the bolts to the neutral axis

A_i area of the failure zone (Figure 4-17)

4.2.4.2.5.2 Resistance Moment of a Bolted Connection for Plastic Design

In calculation it could be supposed, that the neutral axis is located at the level of the upper steel reinforcement in the concrete slab. So the resistance of the concrete could be neglected. This is an assumption to maximise the thickness of the cover plate (4.2.4.2.6)

Therefore, if there is slip, the resistance moment for plastic design for the net section results to

$$M_{Rd,plast} = \frac{f_{yk}}{\gamma_{M0}} \sum k\, A_i\, y_i \tag{4–15}$$

with: k 1.0 in tension and 1/√3 in shear

y_i distance of each bolt to the neutral axis

A_i area of the failure zone (Figure 4-17)

4.2.4.2.6 Resistance of the Cover Plate

With the elastic resistance moment (4.2.4.2.5.1), the thickness of the cover plate results to

$$t = \frac{M_{Rd,elast}\, y_{max}\, \gamma_{M0}}{n\left(\frac{h_{cov\,pla}^{3}}{12} + h_{cov\,pla}\, y_{cov\,pla,1}^{2}\right) f_{yk}} \tag{4–16}$$

And with the plastic resistance moment (4.2.4.2.5.2), the thickness of the cover plate results to

$$t = \frac{M_{Rd,plast}\,\gamma_{M0}}{n\,h_{\text{cov}\,pla,net}\,f_{yk}\,y_{\text{cov}\,pla,2}} \tag{4–17}$$

with: h_{covpla} the height of the cover plate

$h_{covpla,net}$ the height of the cover plate minus the hole of the bolts

$y_{covpla,1}$ distance of the centre of the cover plate to the centre of the concrete

$y_{covpla,2}$ distance of the centre of the cover plate to the upper steel reinforcement in the concrete slab

y_{max} distance of the lower side of the cover plate to the centre of the concrete

n number of the cover plates

Finally the maximum of these values is the proper one.

4.2.4.3 Fatigue design [EC 3.1.1 – 6.2]

The fatigue results of the tests are:

- the fatigue resistance of the connections has been much higher than calculated
- for a positive action moment cracks have been detected in the cover plates
- for a negative action moment cracks have been detected in the web of the girder
- no slip occurred in both cases

The fatigue resistance of the reinforcement is not relevant for bolted connections in span, because the lower classified detail categories of the cover plate or the web of the girder failed first.

The fatigue detail category of the bolted connection is equal to 112 for preloaded high strength bolts or preloaded injection bolts and 90 for fitted bolts or non preloaded injection bolts.

The fatigue detail category of the girder is equal to 160.

4.2.4.4 Position of the joints

The first step is to evaluate the positive and negative resistance moment of the connection ($M_{Rd.conn}$).

After this evaluation, it could be decided where to put the connection section. The possible connection area is specified with A in Figure 4-18.

Because of the test results, that means that the lifetime of a bolted connection in span in the area of the negative moment is much higher than in the area of the positive moment, the connection should be performed in the area with a minimum possible positive moment.

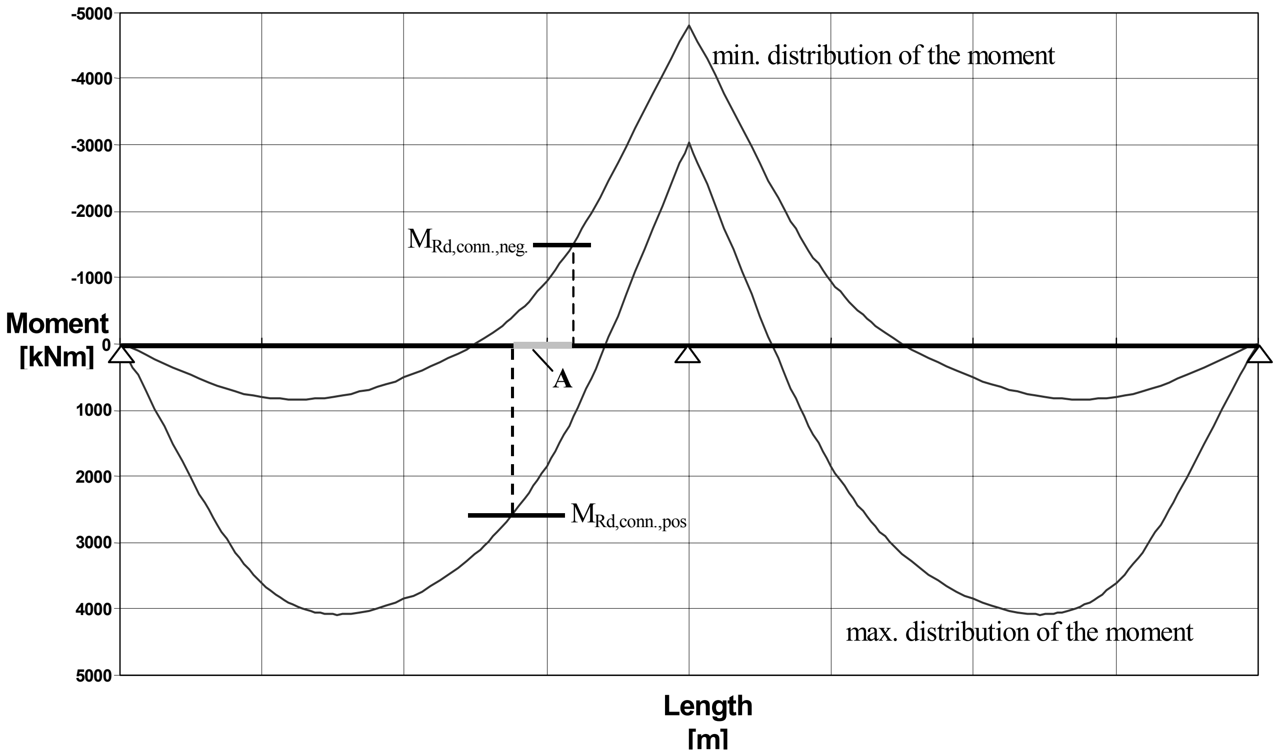


Figure 4-18: Suggested position of a bolted connection

4.2.5 Shear resistance of the girders

The resistance according to vertical shear should be calculated according to EC 3-1-5 (4.3). From the investigation on steel girders of high strength steel S420 and S460 it is stated that the contribution from the web to shear buckling resistance can be expected to be as high as for steel with lower nominal yield strength. The contribution factor χ_w may be taken from Table 4-1.

Table 4-1: Contribution from the web, χ_w, to shear buckling

Contribution from the web, χw, to shear buckling		
$\bar{\lambda}_w$	Rigid end post	Non-rigid end post
<0,69	1,20	1,20
$0{,}69 \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

4.2.6 Lateral torsional buckling of the beams in service

When the composite bridge deck is in place it acts like a bracing of the top flange and consequently prevents lateral deflection of the flange. This means for a simply supported beam that lateral buckling cannot occur, taking into account that the cross bracings at the supports are sufficient. This can be assumed to be the case if the cross braces are designed for horizontal loads from wind and skew braking forces according to EC 1-2.

For a continuous bridge girder the bottom flange will be in compression close to the piers and hence it may buckle laterally. The worst load case may be assumed to be that giving maximum hogging moment at the considered support. Two different designs will be distinguished; girders with intermediate cross braces and girders without cross braces other than at the supports.

4.2.6.1 Girders with intermediate cross braces

A typical moment distribution is shown in Figure a. It can be approximated to a linear variation of the bending moment without much error. A design procedure for this case is given in 5.5.2.4 of EC3-2 (ENV-version). This procedure makes it possible to consider also the rotational restraint provided by the bridge deck. This leads to distortional buckling and the dominating flexibility is the plate bending of the web. For welded girders the effect is not worth calculating but for rolled beams the increase in lateral buckling resistance is noticeable.

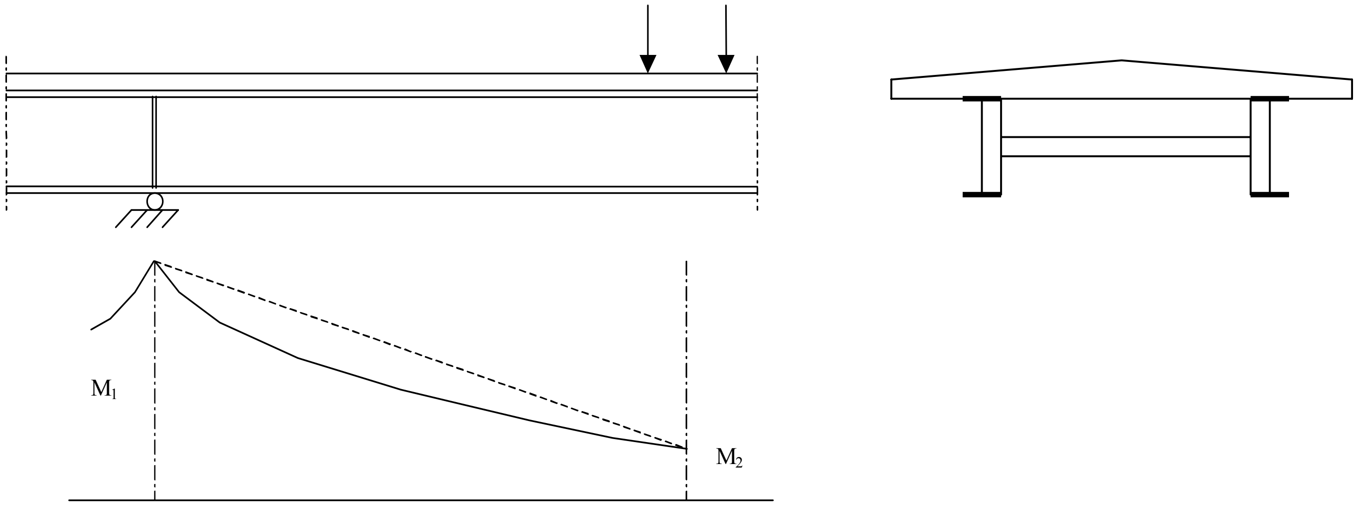


Figure 4-19: Typical moment distribution of bending moment close to a pier.

4.2.6.2 Girders without cross braces other than at support

As suggested in 4.5.1.7 of this report it is sometimes feasible to delete all intermediate cross braces. This is only of interest for rolled beams due to the stability during erection, see 4.5.3. In this case the lateral buckling of the bottom flange will cause lateral deflection of the whole bottom flange and the critical hogging moment will depend on the whole moment distribution. The design procedure in 5.5.2 of EC3-2 is applicable but it exclude any information about the critical moment. Solutions for continuous girders with the top flange braced can be found in *Nylander, 1964, Hanswille, 2000* and in *Hanswille et al 1998.* The former articles of Hanswille neglects the St. Venant torsional stiffness. The latter articel takes the St. Venant torsional stiffness into account. A summary is given below.

The system is defined in Figure 4-20. The distribution of bending moments is characterised by M_0, which is the maximum sagging moment if the span is considered as simply supported, the largest hogging moment αM_0 and the smaller hogging moment $\psi \alpha M_0$, see Figure d. The rotational restraint stiffness c_ζ is defined in Figure c and the critical hogging moment is expressed as follows.

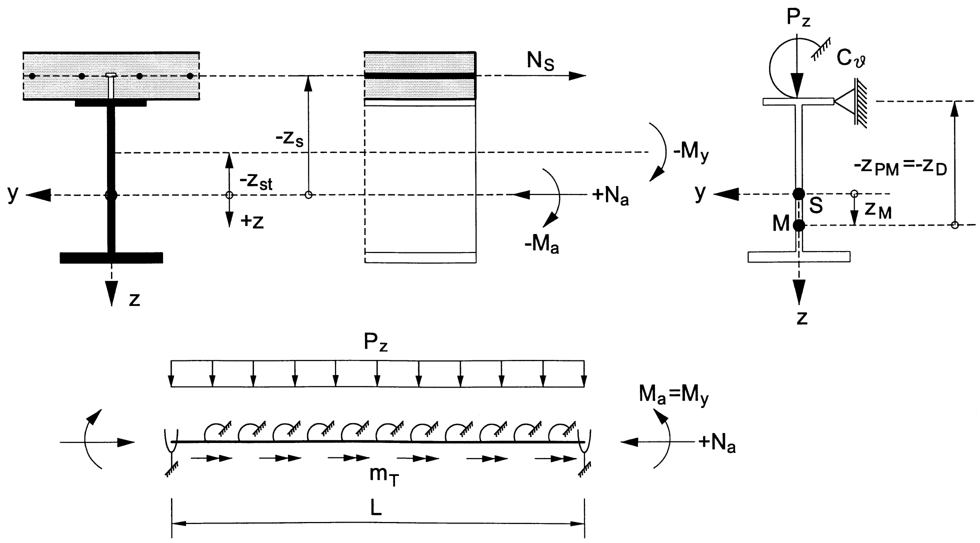
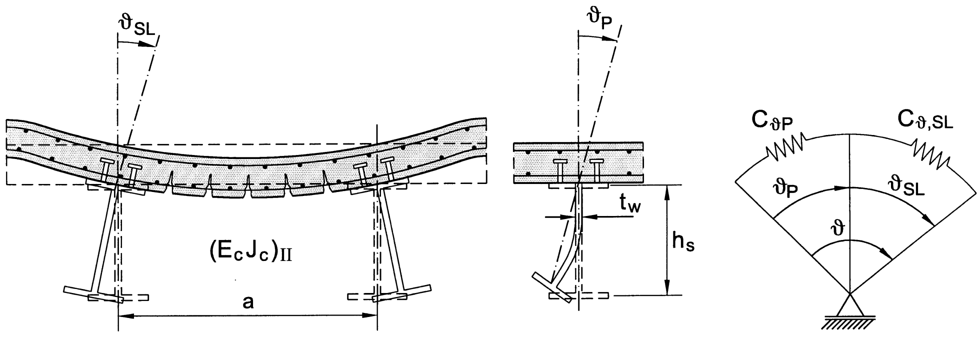


Figure 4-20: Definition of system and notations. S is the centre of gravity of the structural steel section and M is its shear centre.

Cross section and notations



Stiffness $c_{\vartheta,P}$ due to deformations of the web

$$c_{\vartheta,P} = \frac{1}{4}\frac{E_a}{1-\nu_a^2}\frac{t_w^3}{h_s}$$

Stiffness $c_{\vartheta,SL}$ of the cracked concrete slab

$$c_{\vartheta,SL} = k\frac{(E J_c)_{II}}{a}$$

a	span length of the slab perpendicular to the axis of the beam
$(EJ_c)_{II}$	flexural stiffness of the cracked slab in transverse direction
k	factor taking into account the supporting conditions of the slab (k=2 for a simply supported and k=4 for a continuous slab)

Effective stiffness c_ϑ

$$\frac{1}{c_\vartheta} = \frac{1}{c_{\vartheta,P}} + \frac{1}{c_{\vartheta,SL}}$$

Figure 4-21: Definition of rotational restraint stiffness c_ζ.

$$k_z = \left[\frac{(z_D + z_M)^2 + i_P^2}{z_e} + 2\,z_D - r_{Mz}\right]\frac{J_{ay}}{J_{st,y}}$$

$$M_{cr} = \frac{1}{k_z}\left[\frac{\pi^2\ E\ J_{\omega D}}{(\beta\ \ L)^2} + GJ_{T,eff}\right]$$

$$(GJ_T)_{eff} = A(1,5-0,5\psi)GI_T$$

$$\eta = \sqrt{c_\vartheta\ \ L^4\ /\ (E\ \ J_{\omega D})}$$

A	parameter given in Figure d
A_a	area of the structural steel section

A_s area of reinforcement

E modulus of elasticity for structural steel section

G shear modulus for structural steel section

J_{ay} second moment of area of the structural steel section about the y-axis

J_{az} second moment of area of the structural steel section about the z-axis

$J_{\omega D}$ $= J_{af,z}\, h_s^2$

$J_{af,z}$ second moment of area of the bottom flange of the steel section about the z-axis

$J_{st,y}$ second moment of area of the composite section about the y-axis but neglecting concrete

J_T St. Venant torsion section constant of the structural steel section

h_s distance between the centroids of the flanges

i_p^2 $= (J_{ay} + J_{az})/A_a$

z_e $= J_{ay}/(z_{st}A_a)$

z_{st} $= A_s z_s/(A_a + A_s)$

z_s coordinate of the reinforcement

z_{st} coordinate of the centroidal axis of the composite section, but neglecting concrete

z_D coordinate of lateral restraint related to the shear centre of the structural steel section

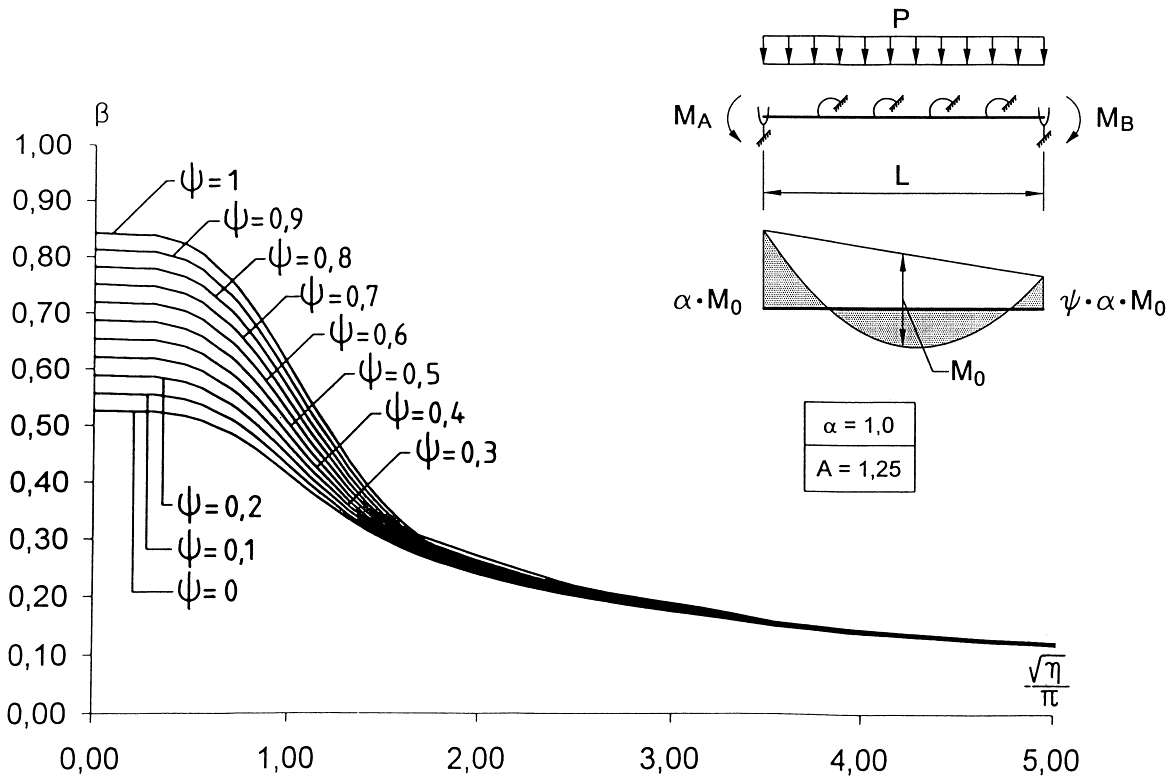
z_M coordinate of shear centre

z_{pM} coordinate of transverse load pz related to the shear centre

r_{Mz} $= \left(\frac{1}{I_y} \int z \ (y^2 + z^2) \ dA \right) - 2 \ z_M$

β effective length factor from Figure d

ν_a Poisson's ratio for structural steel



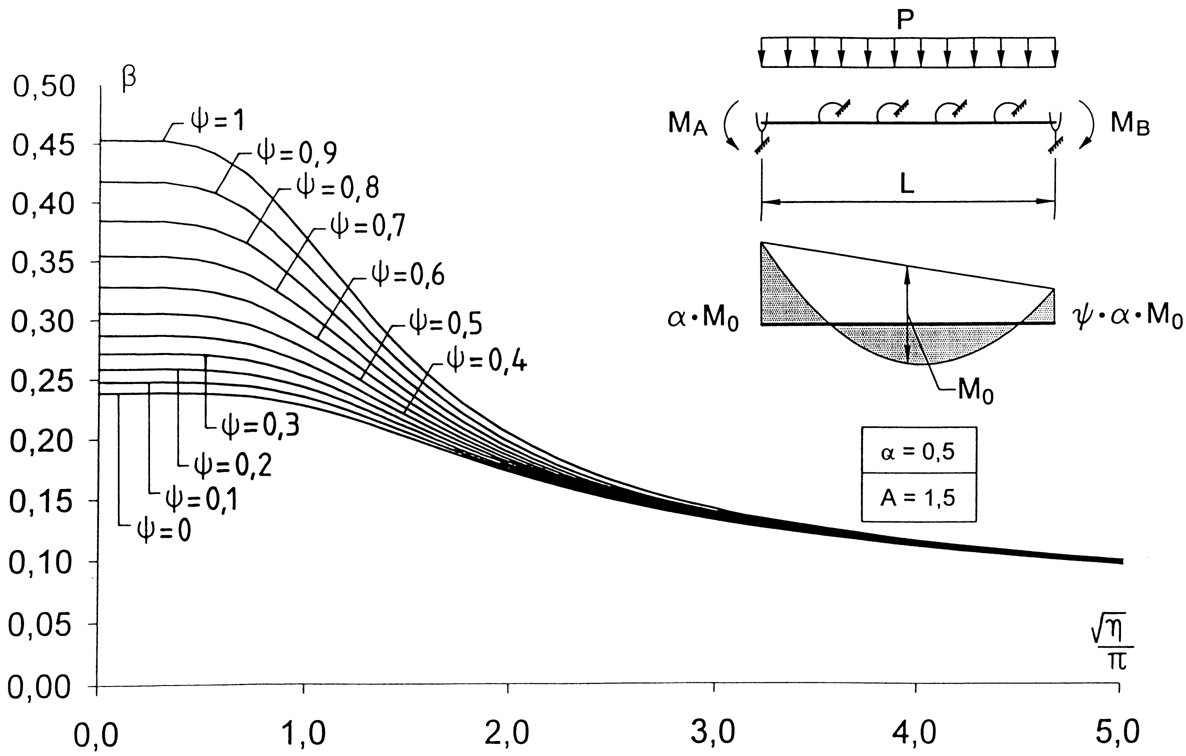


Figure 4-22: Effective length factor β.

4.2.6.3 WORKED EXAMPLE

For the end span of a composite beam in Figure e the critical bending moment is determined based on the presented method. The slab is continuous in transverse direction

Properties of the structural steel profile (HE 800 A):

$A_a = 286$ cm² $J_{ay} = 30{,}34$ cm²m² $J_{az} = 1{,}26$ cm²m²

$J_{af,z} = J_{ay}/2 = 0{,}63$ cm²m² JT = 0,0597 cm²m² $i_p^2 = (30{,}34+1{,}26)/286 = 0{,}11$ m²

Properties of the composite section neglecting concrete:

$A_{st} = 286 + 30 = 316$ cm² $z_s = -(0{,}79/2 + 0{,}1) = 0{,}495$ m

$J_{st,y} = 30{,}34 + (286 \cdot 30 \cdot 0{,}495^2) / 316 = 37$ cm²m² $z_{st} = -0{,}495 \cdot 30/316 = -0{,}047$ m

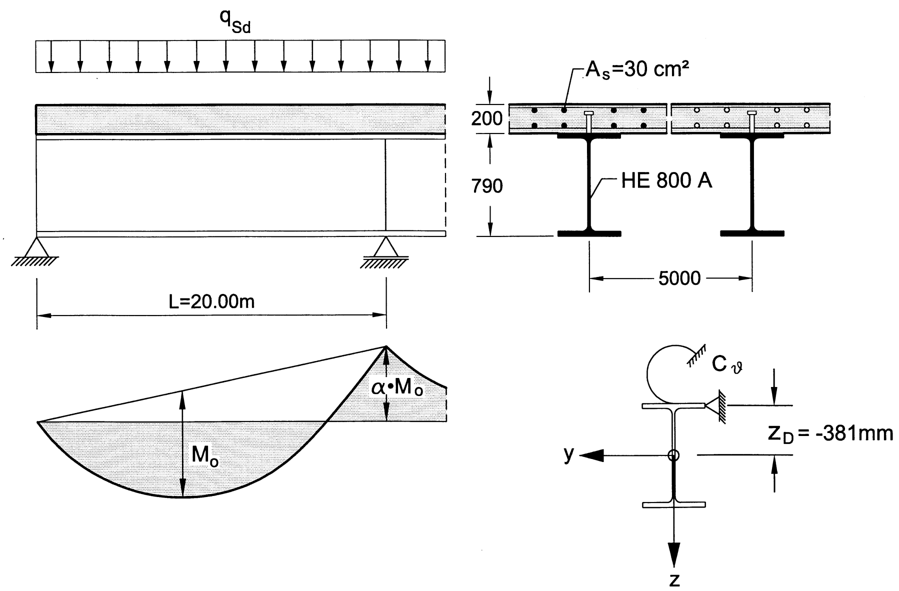


Figure 4-23: Cross-section and loading conditions

With the flexural stiffness of the concrete slab $(EJ_c)_{II} = 4550$ kNm²/m the effective stiffness of the torsional restraint c_ϑ results from Figure c

$$c_{\vartheta,P} = \frac{1}{4}\,\frac{E_a}{1-\nu_a^2}\,\frac{t_w^3}{h_s} = \frac{1}{4}\,\frac{21000}{1-0{,}3^2}\,\frac{1{,}5^3}{76{,}2} = 255{,}5\ \text{kNm/m}$$

$$c_{\vartheta,Sl} = k\,\frac{(EJ_c)_{II}}{a} = \frac{4\ \ 4550}{5} = 3640\ \text{kNm/m}$$

$$\frac{1}{c_\vartheta} = \frac{1}{c_{\vartheta,P}} + \frac{1}{c_{\vartheta,Sl}} = \frac{1}{255{,}5} + \frac{1}{3640} = \frac{1}{239}$$

$\Rightarrow$ c_ϑ= 239 kNm/m

With z_D = -0,381m and z_M = 0 and r_{Mz} = 0 for the double-symmetrical structural steel section the values z_e and k_z are given by:

$$z_e = \frac{J_{ay}}{z_{st,a}\,A_a} = \frac{30{,}34}{-0{,}047 \cdot 286} = -2{,}26\text{m}$$

$$k_z = \left[\frac{z_D{}^2 + i_P^2}{z_e} + 2\,z_D\right]\frac{I_{ay}}{I_{st,y}} = \left[\frac{(-0{,}381)^2 + 0{,}11^2}{-2{,}26} - 2\cdot(-0{,}381)\right]\frac{30{,}34}{37{,}0} = -0{,}68\text{ m}$$

The warping constant $J_{\omega D}$ and the factor ηbecomes.

$J_{\omega D}$= $J_{af,z}h_s^2$ = 0,63*0,762² = 0,366 cm²m4

$$\eta = \sqrt{\frac{c_\vartheta L^4}{E\,J_{\omega D}}} = \sqrt{\frac{239\cdot 20^4}{21000\cdot 0{,}366}} = 70{,}53$$

The effective length factor β = 0,17 results from figure d with α = 1 and Ψ = 0. The effective St. Venant torsional stiffness $(GJ_T)_{eff}$ and the elastic critical bending moment M_{cr} are given by

$$M_{cr} = \frac{1}{k_z}\left[\frac{\pi^2\,E J_{\omega D}}{(\beta_B\,L)^2} + GJ_{T,eff}\right]$$

$$M_{cr} = \frac{1}{-0{,}68}\left[\frac{\pi^2\,21000\cdot 0{,}366}{(0{,}17\cdot 20)^2} + 1{,}25\cdot 1{,}5\cdot 8100\cdot 0{,}0597\right] = -10974\text{ kNm}$$

4.2.6.4 REFERENCES

Nylander, H. Stability of continuous I beams with top flanges braced in a lateral direction, Bulletin No. 50 of the Division of Building Stratics and Structural Engineering, Royal Institute of Technology, Stockholm 1964

Hanswille, G. Lateral Torsional buckling of composite beams. Comparison of more accurate methods with Eurocode 4, Procedings of the conference in Banff, Alberta, Canada. May 28-June 2, 2000. Volume 2.

Hanswille, G., Lindner, J., Münich, D. Lateral torsional buckling of composite beams. Stahlbau 67, 1998.

4.2.7 Detailing of the cross beams or cross braces

The crossbeams of I-girder bridges have several purposes. These are mainly:

1) to guarantee the stability during lifting, launching and casting
2) to guarantee the stability of the lower flange near support
3) to take up forces from torsional moments when the bridge is situated in a horizontal curve
4) to transfer horizontal forces from wind etc. from the steel girders to the concrete deck
5) at supports, the crossbeams serve as lifting points if the bearings have to be replaced
6) furthermore, the web stiffeners to which the crossbeams are connected will prevent the web from buckling.

It is important that the detailing is adjusted for the workshop, with no expensive welding to be performed at the site.

Three main types of crossbeams are:

- Trusses with rolled or welded profiles
- Welded crossbeams
- Hot-rolled crossbeams

The first alternative often offers a superior flexural rigidity, but the costs from welding or bolting the truss can be high.

The second alternative can be very useful for crossbeams at support, where the horizontal forces will be transferred to the piers. As indicated above, they are often designed to carry the dead load of the bridge, in case the bearings will have to be replaced in the future.

The third alternative is commonly used in Sweden, with e g rolled U-profiles. For higher beams, L-profiles used as diagonals can increase the flexural rigidity.

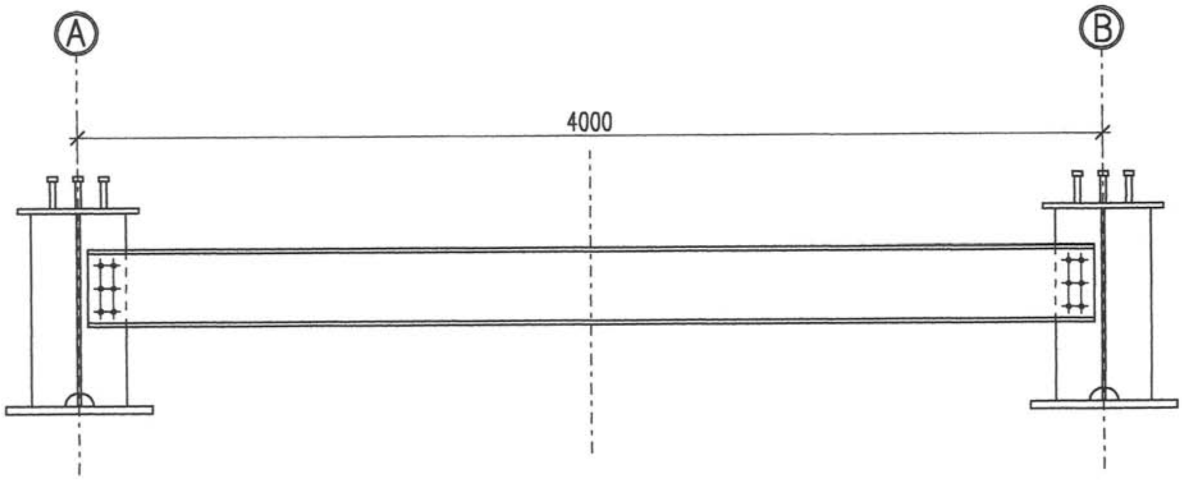


Figure 4-24: Crossbeam made out of hot rolled UPE 300.

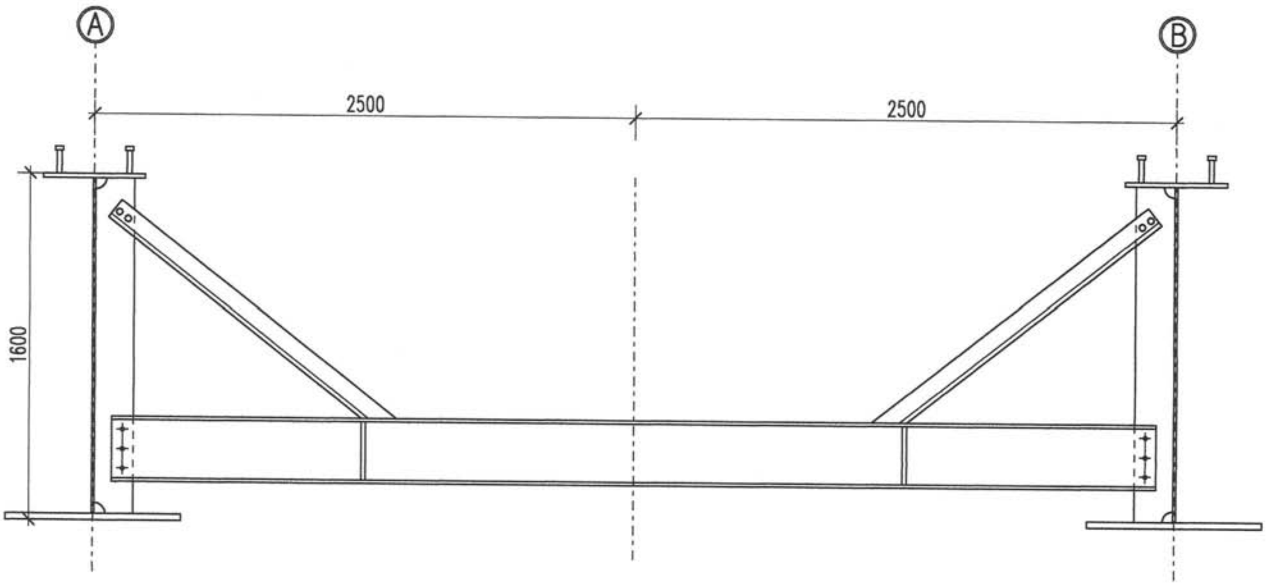


Figure 4-25: Crossbeam using UPE 300 and L 100x100x10.

4.3 Concrete Deck

4.3.1 Joints in fully prefabricated slabs

One solution is to use prefabricated concrete slabs with dry joints. Hence the joints are dry, special concern has to be taken into account concerning the vertical wheel load from a vehicle passing over the joint. If there is no physical contact between the elements in the joint, then the elements will move separately from each other in the vertical direction. The vertical movement will affect the isolation and bitumen on top of the element in a negative way.

In order to prevent this vertical movement, a solution of overlapping concrete tongues has been developed. In Figure 4-26 the overlapping concrete tongues are shown.

Figure 4-26: Scaled model of fully prefabricated concrete elements, showing the overlapping concrete tongues.

Hence the concrete can not resist high-tension stress it is of great importance that the concrete tongues will be properly reinforced in such a way that the reinforcement will be able to resist the tension stress caused by the wheel load. Figure 4-27 shows a typical solution of one way to reinforce the concrete tongues.

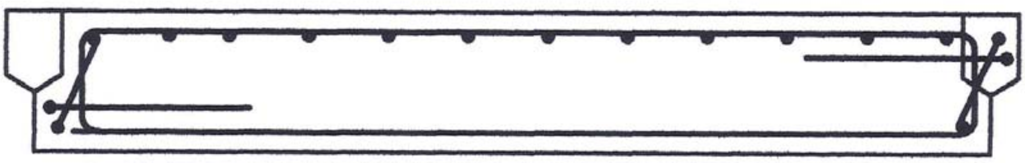


Figure 4-27: Typical solution of one way to reinforce the concrete tongues

The cover of reinforcement on the front of the concrete tongues may possibly be reduced due to the fact that this surface is always protected from water. A reduction of the cover of reinforcement would make it possible to place the reinforcement even further out in the concrete tongues. That would be positive for the resistance. Or it would also be possible to reduce the size of the tongues, which will make it easier to assemble the element on site. Figure 3.3-3 shows a fully prefabricated element.

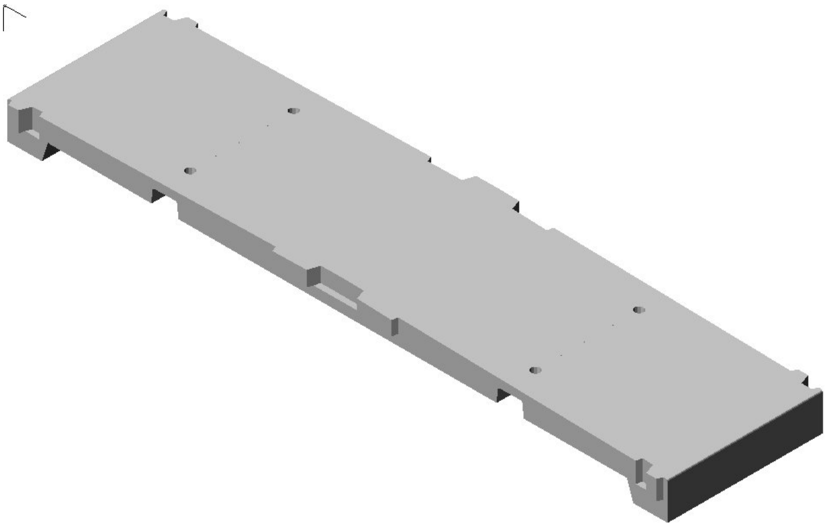


Figure 4-28: Fully prefabricated concrete element.

To achieve composite action between the steel and the concrete it is necessary to connect the concrete to the steel beam. The method normally used is applying headed studs to achieve the composite action. The shear connectors have to be embedded into the concrete slab. One solution, that will keep the prefabrication rate high, is to make a void channel, a trough, on the underside of the concrete slab. The trough in the element will surround the headed studs on the steel flange. Several grout holes will be placed on top of the element to make it possible to fill the trough with concrete on site. Several air holes between the grout holes should also be applied. The air holes are important in order to prevent air pockets inside the trough after casting.

This method gives a dry surface that can be covered with isolation and bitumen immediately. Therefore it is possible to cast a bridge in winter times. If the temperature drops under –20 °C the deck can be heated by radiators. Additionally, due to the fact that the surface is dry, it is possible to cut the construction time.

Inside the trough transverse rebars are placed in order to prevent a collapse of the console during erection and to prevent the concrete to split around the studs. It is important that the spacing of the transverse rebars in the trough and the spacing of the headed studs is co-ordinated to prevent problem during erection. Figure 4-29 shows a section of the element and the steel beam.

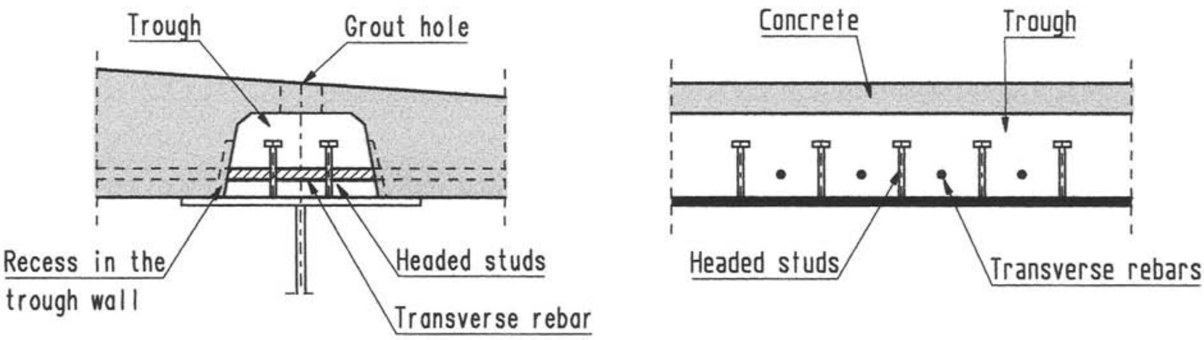


Figure 4-29: Section of element and steel beam.

The concept of prefabricated concrete elements with dry joints is applicable for simply supported bridges when the joints are in compression. The concrete slabs will then act like a continuos concrete slab if the fit between the concrete elements is good enough. According to tests and full scale construction match-casting is satisfactory. When the concept is used for a multiple span bridge the joint openings over intermediate supports have to be considered. The joints will open horizontally because of the strains in the top flange. This in turn will increase the vertical movement between adjacent elements when a wheel passes. The isolation and bitumen may suffer damage from these movements. It is also of great importance to correctly estimate the forces on the headed studs and to check their fatigue resistance. A full-scale test is the recommended next step in the development of this concept.

4.3.2 Concrete technology for prefabricated slabs and in situ concrete

The competitiveness of composite bridges depends on several circumstances such as site conditions, local costs of material and staff and the contractor's experience. One major advantage compared to concrete bridges is that the steel girders can carry the weight of the formwork and the wet concrete, which means that there is no need for temporary structures. Another advantage are the savings in construction time, which saves some money for the contractor but is even more comfortable for the road users; a fact that is usually neglected when considering advantages and disadvantages of bridge designs.

Different construction methods have been developed to improve either one or both of these advantages. Generally the methods can be divided up into three main groups:

- the full in-situ concrete method: the whole slab is poured in place
- the full precast concrete method: the whole slab is a precast unit
- a combination of both: a precast unit is used as formwork and supplemented by an in-situ concrete layer, so that a two parted slab is created

The above-mentioned methods are varied slightly regarding special properties in order to emphasise certain advantages, so that they can be divided up into subgroups.

In the following unit the construction methods are described and their advantages and disadvantages are mentioned.

4.3.2.1 The full in-situ concrete method

The steel girders are laid over the gap that has to be bridged. There after the formwork for the concrete slab is erected on the steel girders. If statically necessary the horizontal rigidity is improved by integrating reinforcing braces. Now the shear connectors are welded onto the steel girder and the reinforcement is installed. Thereafter the concrete is poured. After the time of after treatment the surface is sealed. The formwork often makes a complicated static necessary that in complexity might surpass the one of the bridge itself.

On the other hand it enables the architect to design forms that are not possible to build with precast units. Except for the welding, which also can be done by the steel company, there is nothing to do on site that requires especially skilled workers so that the work can be executed by a large number of companies without a special preparation. At least for the customer this has the advantage, that a great competition between the executing companies on the marked exists and so drops the prices and makes it easy to find an adequate firm.

This construction method results in that the steel girders carry the whole dead load of the bridge's superstructure and in addition the weight of the formwork during construction time. So extra horizontal reinforcing braces are almost essential and the steel girders must be dimensioned bigger.

The total amount of steel is the highest of all methods. Due to the formwork the execution time is very high and man-hours are numerous. The whole shrinkage of the concrete affects the bridge. Fore the whole term of concreting it is essential that the environmental temperature is above 5°C what makes a construction in winter terms impossible in colder tempered zones such as Northern Europe.

However, since the required techniques are quite easy to execute it has become the most popular solution to set up a composite bridge.

Table 4-2 gives a brief summary of the advantages and disadvantages of the conventional concreting method.

Table 4-2: Summary of the advantages and disadvantages of the full in-situ concrete method

Advantages	**Disadvantages**
- almost each design of the slab is possible - large number of capable firms cause competition and low prices - most popular method	- no composite action for dead load - long construction time - large amount of man-hours - large amount of steel is used - impossible in winter terms with temperatures below 5°C

4.3.2.2 The concrete slab as full precast unit

Precast units are laid on rolled steel joist with pre-mounted shear connectors. As it is not planed to fix in mortar to the joints afterwards, a dry construction method has to be used. In order to receive a good fit between the elements one finished element should be used as formwork for the next one during production, what make a numeration of the elements necessary. The connection between the elements should be made using special dowels. Within one day the units can be mounted and afterwards the elements can be prestressed by tightening four M24 bolts at each end of the girders. Thus the steel girders was used as tension bars. The construction can be made during winter down to temperatures about –10°C but in this case the environmental temperature has to be increased by at least 15K to concrete the connections between the slab and the girders. To achieve this raise in temperature the bridge e.g. can be covered by plastic film and heated using gas heatings. As the temperature rise to 5°C the concrete can be injected to the shear connectors through holes with a diameter not smaller than 20mm in the precast units.

While using precast units has the advantage of high quality concrete and gives the opportunity of surveillance it also requires a high accuracy producing the units. Obviously very exact working is required producing the precast units and mounting the shear connectors so that later when positioning the elements no problems occur due to shear connectors that get into conflict with the reinforcement of the slab.

As said before one advantage of composite bridges is the possible short construction time. The most favourable way of constructing composite bridges in order to save time is the above mentioned one. In Sweden an investigation was carried out in order to determine the costs that might be saved by a shorter construction phase. The investigation shows a possible time saving of about ten weeks not yet considered the possible disturbance of cold weather when concreting is impossible. The possible savings where calculated referring to a real carried out project and assuming that the time saving of each road user, using the bridge, is 5 minutes, that daily 2000 cars pass the bridge and that one hour values approximately 10 EURO for the road user. Due to that assumptions the sum of savings after 10 weeks is 117000 EURO. On the other hand, the problem is that no customer considers this fact what is quite understandable as nobody who is concerned with the decision how to build the bridge has to pay these expenses. So this argument might be a political question.

The following table gives the advantages and disadvantages of the use of fully prefabricated concrete slabs in short:

Table 4-3: Summary of the advantages and disadvantages of the concrete method using the concrete slab as full precast unit

Advantages	**Disadvantages**
- high and equal concrete quality - shortest construction time of all methods - best solution for the road users - reduces man-hours due to work in fabrics - set-up is possible in freezing periods - reduced forces from shrinkage as the elements get to the site older	- high technical know-how is necessary - this includes high costs - limited design possibilities - steel girders carry the whole dead load - only very few precast unit factories are able to the job - only few companies have experience with that method

4.3.2.3 *Precast units supplemented with in-situ concrete*

This method has to be divided up into two major groups. It can be carried out so, that the precast unit is just a permanent formwork. It is also possible to give the precast unit a static function in longitudinal direction. This method was improved by the SSF company in Germany.

The basic idea using these precast units is to simplify the formwork. Prefabricated elements are laid as formwork onto the steel girders that are mounted equal to the other construction methods. Next the reinforcement is installed and the concrete is poured from the middle of the bridge to the ends. The direction of concreting is quite important in order to avoid cracking. Starting in the middle lets deflect the bridge first so that when concreting the locations with negative moments and deflections, they already have their final forms. So the just poured concrete does not get subject to tension. To ensure the transmission of shear stress, the shear connectors are so long that they pass through the precast unit and enter into the in-situ concrete slab. Also shear garlands are mounted that connect the precast unit and the in-situ concrete. This sequence of construction results in that the whole dead load of the bridge is carried in longitudinal direction by the steel girders. As already mentioned other ways of building have been developed in order to give the prefabricated elements also in longitudinal direction a static function.

One possibility is to fix in mortar to the transverse joints before poring the in-situ concrete. This however requires an extra work step and extra costs. The advantage is that the main dead load of the bridge, the load of the reinforcement and the in-situ concrete, is carried by the partly composite girder. The precast units act as compression boom and give the construction a better horizontal stiffness while concreting. Horizontal braces are not a must. In every case the amount of construction steel required is reduced.

Table 4-4 sums up the advantages and disadvantages of the above described concrete method.

Table 4-4: Summary of the advantages and disadvantages of the concrete method using precast elements as formwork

Advantages	**Disadvantages**
- no additional formwork is needed - short construction time - possibility of reducing effects from shrinkage - especially if composite action is activated first the amount of steel that is necessary is reduced as the dead load is carried by the composite girder	- more complicate technique that is not executable for every company - restricted design possibilities - long transportation ways as only a few factories can produce the elements

The SSF engineering society improved this method again. They developed a type of composite girder that can be almost completely prefabricated in the factory. These have an upper boom which acts as an economical cross section under compression, as a shell element for the bridge floor slab and as a horizontal stabilisation. Constructing and tilting braces are not longer needed for concreting the in-situ concrete plate. The amount of steel required is once again considerably reduced, since this is determined by the amount needed in construction and not the amount contained in the bridge structure. Simple connection rod links in the support area allow the girders to be coupled in the construction phase, in order to achieve a continuous effect. After the installation and reinforcing stages, the in-situ concrete can be pored continuously and therefore more economically. The already executed projects include large bridges over valleys as well as double-span bridges over motor ways and show that also this very special method which is only supported by a few precast unit factories can be favoured by certain customers. However, it remains a very special solution and includes a very complicate transportation as the girders can not be piled up. Due to the fact that only a few factories support the method, long transportation ways are almost sure, this might be the most serious problem for this method to become more popular.

The advantages and disadvantages using this invention are given in Table 4-5.

Table 4-5: Summary of the advantages and disadvantages using the above mentioned invention

Advantages	Disadvantages
- The high quality of the finished structure keeps maintenance costs low, due to the strong design with easily accessible steel girders and a monolithic in-situ concrete slab - Extensive prefabrication of the girders reduces the amount of work on the building site, so that construction deployment is very economical with efficient construction time - The concrete flanges that form the upper boom make buckling braces in the steel girders unnecessary, considerably reduce the amount of steel used in construction, since the composite material is mounted in advance, and increase the tilt resistance, i.e. no assembly braces are needed.	- The tall girder-slab composition is not pile able and makes huge trucks necessary for transportation. - High costs due to low competition - The rare available precast unit fabrics make far transportation necessary.

Regarding all the mentioned arguments it is impossible to favour one of the above mentioned construction methods. However it is obvious that the main disadvantage of the more developed methods, which all include the use of precast units, is the required know-how. Another disadvantage is that only a few precast unit fabrics are able to produce the required elements. If it would be possible to raise political interest in the costs caused by traffic disturbance the fastest method would always be the most favourable. Only the dry construction method as tested in Sweden allows construction during freezing period. Since these arguments only apply to very few bridges or to anyone, the use of pre-concreted elements as formwork seems to be the most reasonable method. The very special solution of SSF’s VFT© is only reasonable for very few opportunities since transportation costs can be too high.

4.3.3 Creep and shrinkage effects

The long-term behaviour of concrete causes complicate processes in composite girders, especially if the slab consists of two different concrete with different ages.

To estimate the creep and shrinkage effects, in the case that fully prefabricated concrete elements as well as full in-situ concrete slabs were used, the so called ‘hole cross-section method’ according to EC 4- part 2 can be used.

In the case that the slab consists of two different concrete with different ages three possibilities exist for calculation:

- One possibility is the ‘hole cross-section method’ and the rather safesided assumptions that firstly the prefabricated part is not taken into account calculating the stiffness of the cross section and secondly the creep and shrinkage multiplier like for in-situ concrete are used. These assumptions lead to perspicuously safesided results.
- The second possibility is the ‘hole cross-section method’ and more refined assumptions mentioned below. It has the advantages of a manual calculation and also its disadvantages. In the case of the above described construction method it requires the separation of the construction process (three different static systems) and the separation of the load conditions (at least three). Then the hyperstatic force has to be determined in an iteration process, while cracking is still not considered. Considering cracking requires an assumption for the cracking zone or an additional iteration process. If also tension stiffening should be considered the calculation is not anymore accessible for manual calculation. But also neglecting tension stiffening and cracking, it might be doubted that this possibility is really a solution. In every case the determination of the creep multipliers is still necessary, what can be done using the program ‘CASE’.
- The third possibility is the use of the program ‘CASE V2.2’ or other software. In the case of ‘CASE’ it can be said that no assumptions are made that are not allowed in the Eurocode. It only executes the calculations explained in EC4-2 and considers the facts given there. It is easy and comfortable to use and allows the control of the calculation.

The above mentioned program ‘CASE’ was developed in the frame of the ECSC research project “Composite Bridge Design for Small and Medium Spans” and is written in Visual Basic 6.0® as plug-in for Microsoft Excel 9.0® part of Microsoft Office2000®. A detailed description can be taken from the final report, chapter 7.3.3 “The program ‘CASE’ (Creep and Shrinkage Effects)”. It has to be remarked that for no input an error check is made. This means that entering wrong values results in an error warning and the interruption of the running process or simply in wrong results. The entering numbers must compromise with the system settings (i.e.1,5 or 1.5).

Therefore this program should only used by experts which are able to identify ‘wrong results’!

4.3.4 Cracking of the joints and of the slabs

A comparison of 21 test results found in literature with the calculated values of the crack width according four different calculation methods and the interpretation of this comparison was carried out in the frame of the ECSC research project “Composite Bridge Design for Small and Medium Spans”. From the that the following conclusion can be drawn:

- The calculation methods according to [1] and [2] work very well.
- The ENV 1994-2 : 1997 gives simplified design rules to avoid impermissible cracks, for example values of minimum reinforcement, a table of maximum bar diameter and maximum bar spacing are given.
- The simplified design rules given in ENV 1994-2 : 1997 are in a good accordance with the results evaluated with the calculation methods according to [1].

In conclusion the simplified design rules are applicable to composite bridges in small and medium spans made of normal or high strength steel and in-situ or prefabricated concrete slabs. Only the influence of creep and shrinkage of partially prefabricated concrete slabs could not clarified so that in this case safesided assumptions are recommended.

For a more refined calculation of the crack width by experts the calculation method according to [1] should be used.

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[1] ECSC-Research Project “Composite Bridge Design Improvement for High Speed Railways” – Report 4. “Composite Behaviour at Intermediate Supports”, Agreement 7210-SA/128, University of Wuppertal, March 1998

[2] Maurer, R.: “Grundlagen zur Bemessung des Betongurtes von Stahlverbundträgern”, Dissertation, TH-Darmstadt, 1992

4.3.5 Vibration of bridges

The measurements on 7 composite bridges and their evaluation as well as an extensive parameter-study carried out using the FE-Program DYNACS can be summarised as follows:

- For composite bridges in small and medium spans usually no adverse comments of pedestrian crossing the bridge simultaneously with traffic occurred.
- Due to some strong perception of vibration occurred in one of the carried out measurements it seems possible that the application of high strength steel and concrete slabs of low weight in future result in vibration problems.
- If it is necessary composite bridges should be examined using VDI-Standard 2057.
- Finally it can be stated that the FE-Program DYNACS (developed from the Institute of Steel Construction, RWTH Aachen) is useful to predict the dynamic behaviour of composite bridges of small an medium spans under vehicle traffic.

4.4 Cross girder

For the structural design of a concrete cross girder section, two general designs may be considered in order to limit the stress in the concrete.

4.4.1 Connection always in negative moment area

If the connection is always in a negative moment area, the connection should be perform with a concrete cross girder and thick cap plates (Figure 4-30).

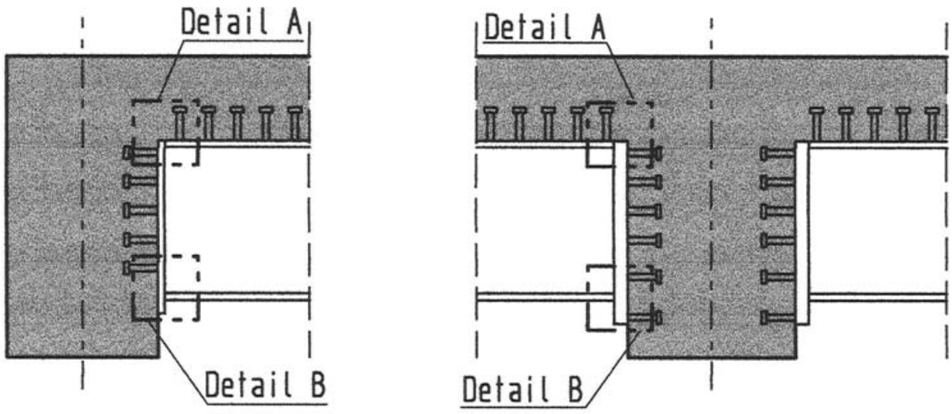


Figure 4-30: Concrete cross girder with thick cap plates

The thickness of the welds results of (4.4.3.3) and the thickness of the cap plates results of (4.4.3.1).

It is also possible to apply the studs only below the neutral axis (in the compression zone).

4.4.2 Connection in alternating moment area

If the connection is in a alternating moment area, the connection should be perform with a concrete cross girder, thick cap plates and tension anchoring (Figure 4-31).

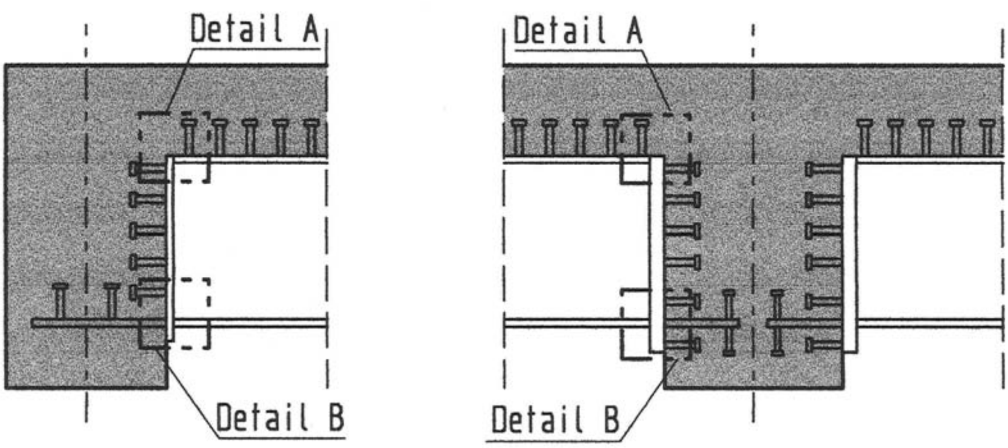


Figure 4-31: Concrete cross girder with thick cap plates and tension anchoring

The thickness of the welds results of (4.4.3.3 and Figure 4-34) and the thickness of the cap plates results of (4.4.3.1).

4.4.3 Static design

4.4.3.1 Dimensioning of the cap plate

For a uniform distribution of load on an area A_0 (Figure 4-32) the concentrated resistance force may be determined following [EC 2.1 – 6.7].

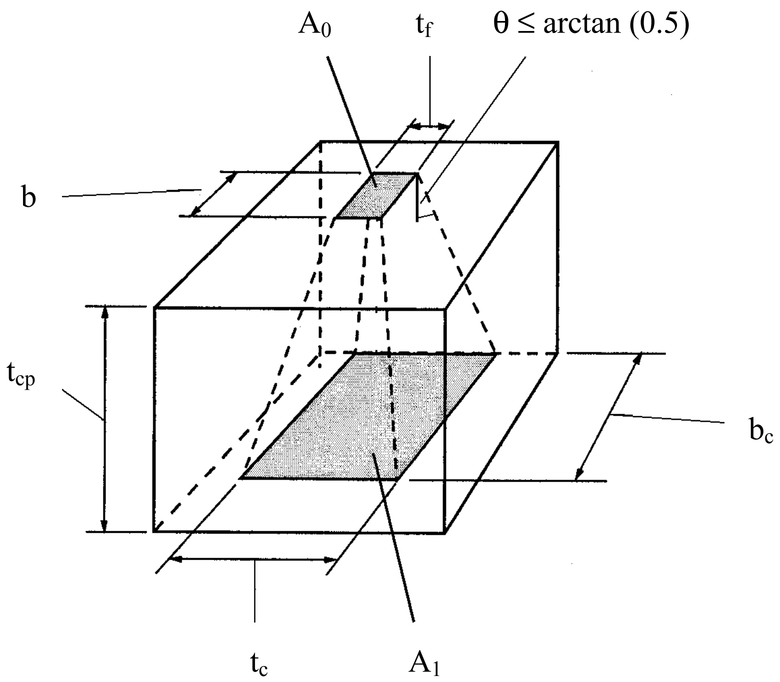


Figure 4-32: Determination of areas in partially loaded areas

Therefore the thickness of the cap plate t_{cp} results to the minimum of

$$t_{cp} \leq 2\, t_f \quad and \quad t_{cp} \leq 2b \tag{4–18}$$

with: t_f and b of the flange of the steel girder

because the highest stress occurs in the lower flange of the steel girder.

4.4.3.2 Dimensioning of the reinforcement

The dimensioning of the reinforcement is described in [EC 2.1].

4.4.3.3 Dimensioning of the welds

For welding the thick cap plates on the girder, there are two options. First it is possible to make a fillet weld all around the profile (Figure 4-33)

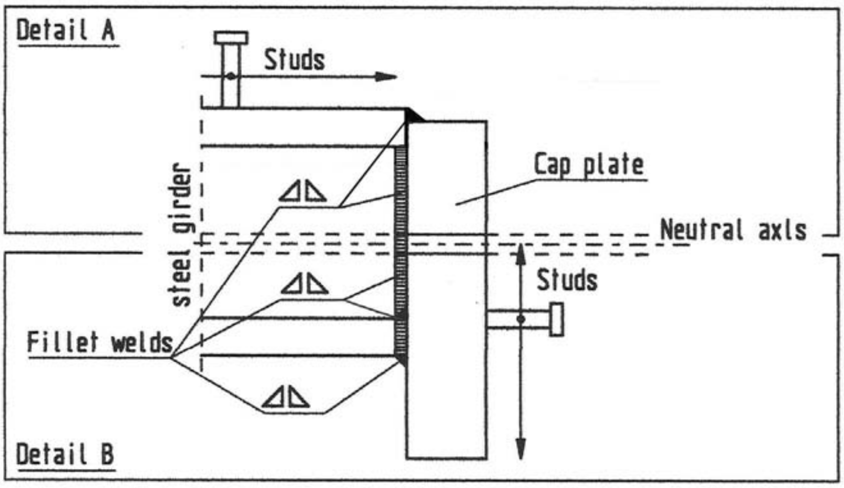


Figure 4-33: Cap plate solution with fillet welds

and it is also possible to make a full penetration weld (Figure 4-34).

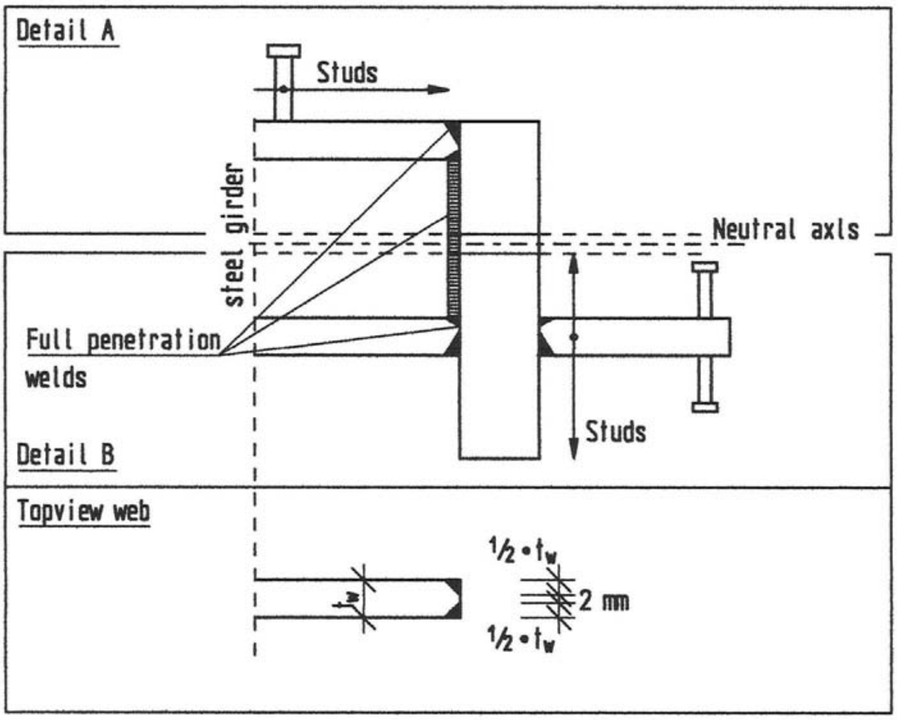


Figure 4-34: Cap plate solution with full penetration welds

The fillet weld is the cheaper version but following EC 3 the expected fatigue life of the proper lower flange-to-cap-plate-weld is less than the fatigue life of the full penetration weld version.

In the static case the thickness of the fillet welds [EC 3 – 6.6.5] results to

$$3.0 \leq a_w \geq \frac{t\sqrt{3}}{2\sqrt{2}} \tag{4–19}$$

with: t thickness of the flange or the web and the thickness of the full penetration welds [EC 3 – 6.6.5] at the outside of the flanges results to

$$a_w \geq \frac{2\, t_f}{3} \tag{4–20}$$

and at the inside to

$$\begin{aligned} a_w &\geq \frac{t_w}{2} \quad \text{in the area of the web and} \\ a_w &\geq \frac{t_f}{3} \quad \text{in the area of the flange.} \end{aligned} \tag{4–21}$$

In the centre of the full penetration welds in the web there is a gap of 2 mm.

4.4.4 Fatigue design [EC 3.1.1 – 6.2]

The fatigue results of the tests (only the fillet welded solution has been tested 2 times) are:

- the fatigue resistance of the connections has been much higher than calculated
- the fillet welds failed first, but in both cases no crack growth has been occurred
- the main cause of failure has been the failure of the reinforcement

The fatigue detail category of the fillet welded joint is equal to 36* and of the full penetration welded joint 40 up to 80 in subject to the geometry of the jointed details.

The fatigue detail category of the reinforcement is 162.5 at 10^6 cycles [EC 2.1 - 6.8.4].

The fatigue detail category of the girder is equal to 160.

4.4.5 Example

For example of the cross girder solution in the following the three span bridge in Differdange (South of Luxembourg) is shown (Figure 4-35).

The bridge carries the Differdange town centre bypass road over the 9 tracks of a main railway line. Spans of the heavily skewed and curved 12.75 m wide bridge are 25 – 40 – 25 m long. The 6 main girders are 1 m deep rolled beams (HL 1000 M / W40x277) in high strength Fe E 460 – HISTAR QST steel grade.

Figure 4-35: Three span bridge in Differdange

Figure 4-36: Main girders with thick cap plates and studs

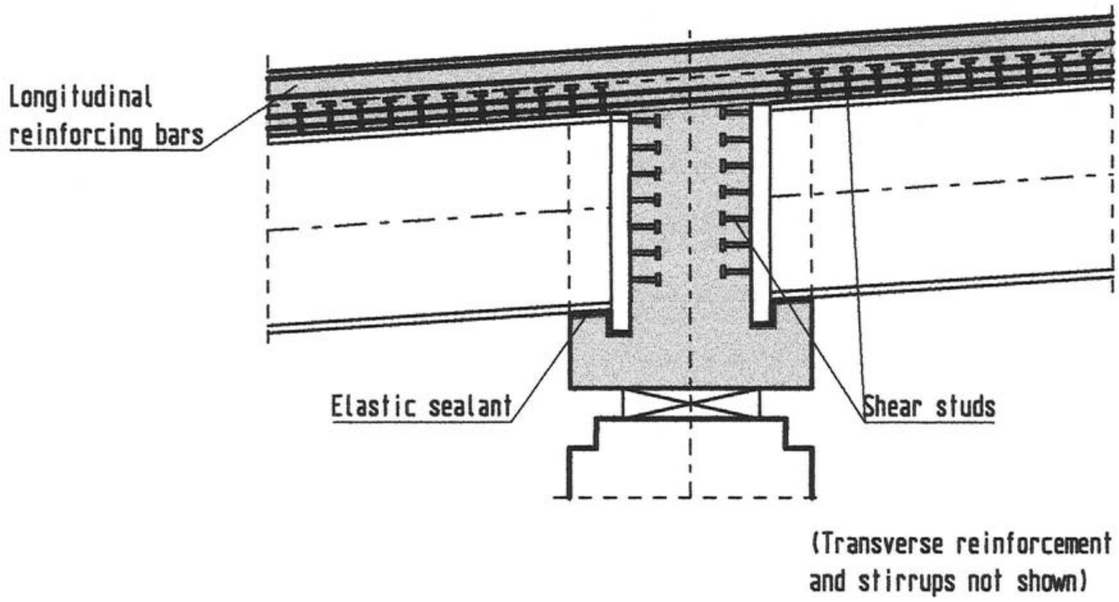


Figure 4-37: Plan of the connection section on support

4.4.6 Recommendations

If the connection is always in a negative moment area, the connection should be perform with a concrete cross girder and thick cap plates Figure 4-30. The using of a fillet weld is the better solution to weld the profile on the cap plate, because of the advantages of the simple installation of the bending-, shear- and splitting-tensile reinforcement, the less reinforcement and no welding activities for the additional compression plates necessary. Additionally the fillet weld is the cheaper solution.

Furthermore it has been proved, that the fillet weld solution reaches a fatigue life which has been higher as for the full penetration weld has been calculated.

If the connection is in a alternating moment area, the connection should be perform with a concrete cross girder, thick cap plates and tension anchoring Figure 4-31. In this case the using of a full penetration weld is the better solution to weld the profile on the cap plate.

To calculate the stress range for the fatigue analysis a linear strain distribution over the whole section could be assumed. This means, that a continuous steel girder without a joint could be supposed.

According to the DIN-Fachbericht 104 the following principles of design shall be considered:

The shear reaction forces in the connection between the concrete cross girder and the main girder have to be transmitted through a concentrated arrangement of studs at the end of the girder. In the case of a positive moment over the support the tension force of the lower flange of the steel girder has to be transmitted through adequate force transmission constructions and with reinforcement into the concrete.

In the area of the lower flange of the steel girder a splitting tensile reinforcement is necessary (Figure 4-31). Splitting tensile reinforcement resulting from other loadings can not take into account.

To avoid the change of the reinforcement a continuous thick cap plate shall be provided (Figure 4-31). The thickness of the plate results of 4.4.3.1.

For short spans the cross girders and the concrete slab have to be concreted in one procedure with retarding admixture. The concrete cross girders have to be compacted.

If it is not possible to concrete bridges with longer spans in one procedure the sections in span have to be preliminarly concreted and after that, the support sections with the concrete cross girders have to be concreted.

If the abutments will be pre-concreted, a working joint has to be arranged horizontally between the cross girder and the slab.

4.5 Fabrication and Erection

4.5.1 Girder Type Composite Bridges

4.5.1.1 Principle & Applications

Composite deck construction consists of steel girders, which support a reinforced concrete slab. Composite action is achieved by connecting both materials by means of shear studs. Transverse bracing over supports provides lateral restraint.

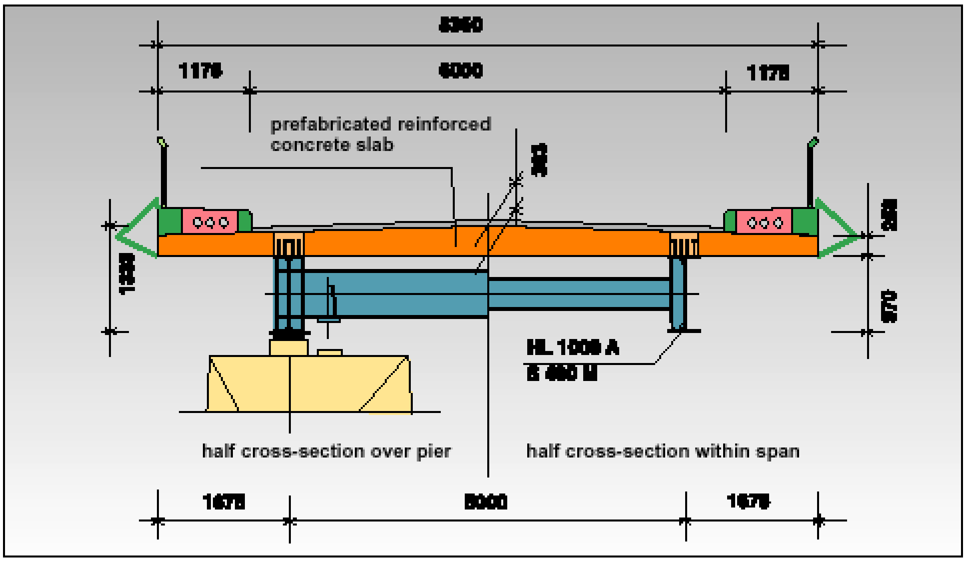


Figure 4-38: Example of twin girder arrangement (cross section of A16 over-bridge)

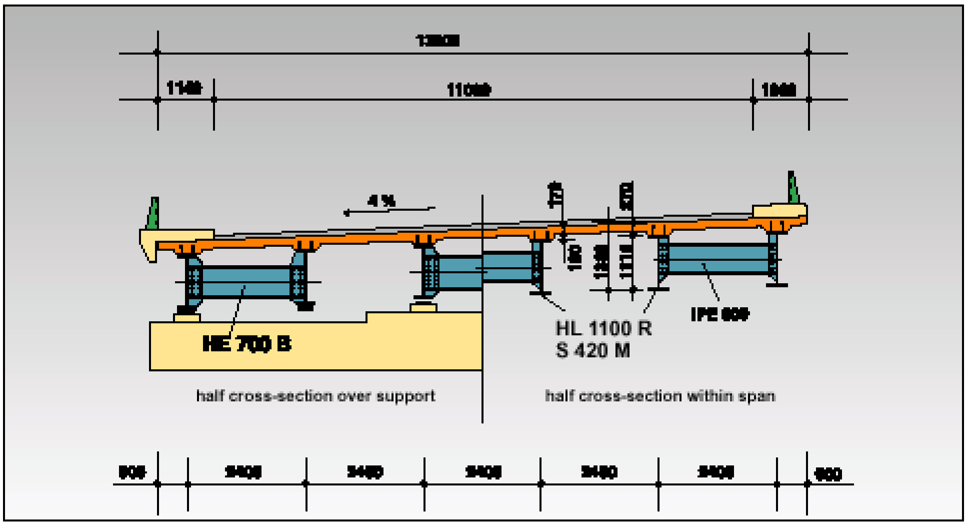


Figure 4-39: Example of a 6 girder arrangement (cross section of road bridge, by-pass for the town of Luxembourg)

Composite deck construction is recommended wherever construction depth is not, or only slightly, restricted. Feasible spans for road bridges range up to about 35 m for simply supported spans and up to about 40 m for continuous spans.

4.5.1.2 Static system

For a narrow deck, two main girders are required. When the deck is wider or when construction depth is restricted, more than two girders will be needed.

Girders of single span bridges are simply supported on the abutments. Multiple span bridges are designed either as successive simply supported or as continuous structures.

Continuous girders are statically better suited: bending moments are lower and deflections are smaller. In addition they offer a major constructional advantage: the number of bearings and expansion joints, which cause high costs by the need for regular maintenance, is reduced.

Depending on the overall bridge length and transport conditions the beams may be erected as unspliced pieces (delivered ex works in lengths up to 34 m or in exceptional cases up to 45 m) or site splicing will be necessary. In the latter base both splicing by butt-welding and by bolting (high strength friction grip bolts) have proved successful. A newly developed method consists in connecting the beams to a concrete cross girder through end plates and additional slab.

Figure 4-40: Railway bridge over the A23 motorway in Fretin, France. Four-span bridge with spans of 16,9 - 21,9 -23,0 - 17,8. The two continuous main girders and the bracings within spans are rolled beams; the cross beams at supports are made of reinforced concrete.

4.5.1.3 Cambering and bending of main girders

Girders are cambered to compensate for deflections under permanent loads. Additional bending may be required to form the girders to the shape of the longitudinal profile. If the bridge is horizontally curved bending along the weak axis may be necessary. Both cambering and bending are carried out in the rolling mill on a press.

4.5.1.4 Surface Treatment

In addition to the significant aesthetic function surface treatment has to provide an effective protection against corrosion.

The following are conditions for a durable protection:

- careful surface preparation
- controlled application of the protective coating
- regular inspection during the life of the bridge and immediate repair of any damage.

Over the last few years, much progress has been made with respect to paint formulations, their application, their durability and their environmental friendliness. In general, modern coating systems last for at least 20 years before major maintenance.

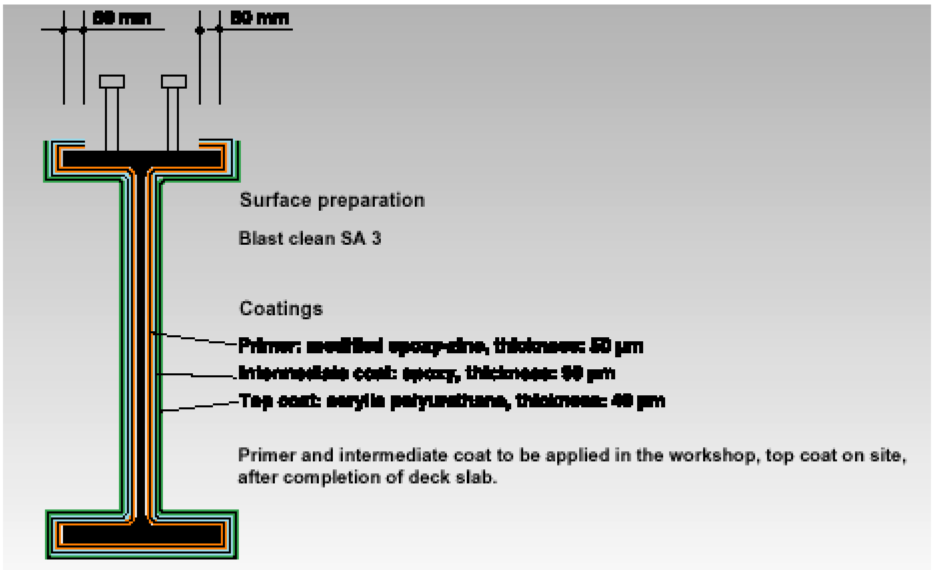


Figure 4-41: Example of protective treatment for composite decks

The choice of a suitable system and the correct application are essential for effectiveness and durability. The available protective systems include the following:

- painting
- metal spraying
- hot-dip galvanizing
- galvanizing + painting (duplex system)

A number of aspects have to be considered for the choice of the appropriate system: type and expected life of the bridge, local climate and other environmental site conditions, constraints relating to maintenance, possibilities of surface preparation and painting, metal spaying or galvanizing at workshop, and feasibility of applying the final coating on site.

Steel components which are encased in concrete do not require coating. However transition zones must be carefully designed.

It is recommended to carry out as much of the surface treatment as possible in the controlled environment of a workshop. Usually only the finishing coat is applied after erection on site. Here, rolled beams can offer the advantage of already being coated with the required systems in the rolling mill.

4.5.1.5 Steel grades

Steel with a yield strength of 355 N/mm2 (S 355) and more recently of 460 N/mm2 (S 460), are used primarily. With the latter type, special attention should be paid to the stiffness requirements. The use of S 460 high-strength steel in place of the more traditional S 355 results in a substantial reduction in weight and corresponding savings in material costs. Fabrication costs are also lower, with a full butt joint, for example, the weld volume is considerably reduced.

The use of fine grain structural steel is particularly advantageous: for example, grade S 355 M or grade S 460 M in accordance with EN 10113-3.

Shapes made from these low alloy fine-grained steel are produced using a thermo-mechanical rolling process with an increased cooling rate and subsequent self-tempering. These grades demonstrate excellent toughness at low temperatures and are characterised by their outstanding weldability. Due to the low carbon equivalent value, pre-heating is not required before flame cutting and welding.

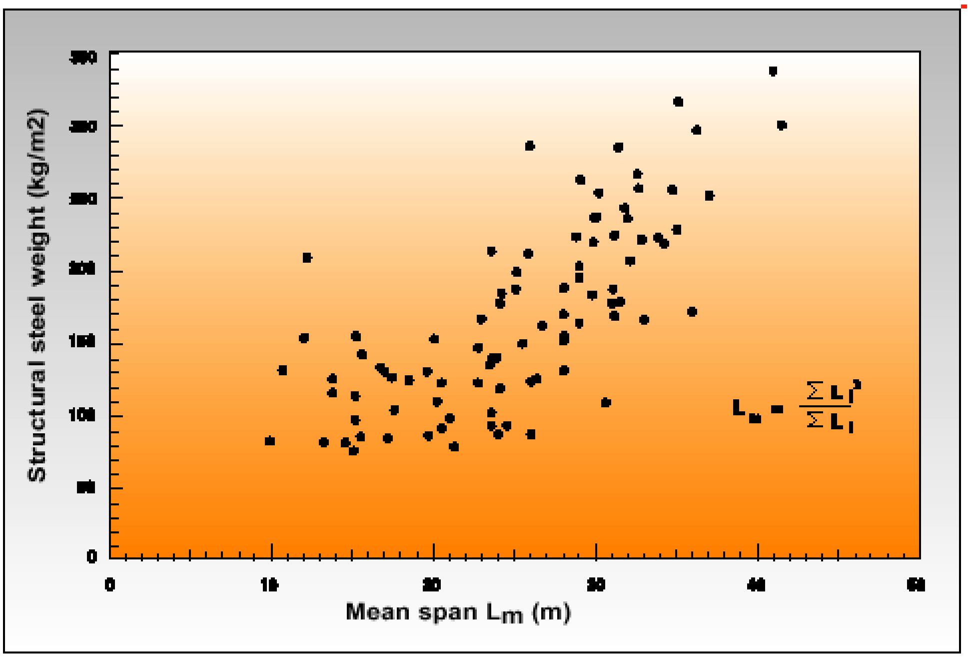


Figure 4-42: Composite road bridges. Structural steel weight per square meter of deck area.

4.5.1.6 Transverse bracing at supports

At supports, bracing is required to transfer horizontal loads to the bearings and to provide lateral and torsional restraint to the girders. Bracing is often designed to carry additional jacking loads in case of replacement of bearings.

Bracing consists of:

- either steel beams which are rigidly connected to the main girders by bolting or welding
- or reinforced concrete cross beams, where the reinforcing steel passes through holes in the web of the longitudinal girders.

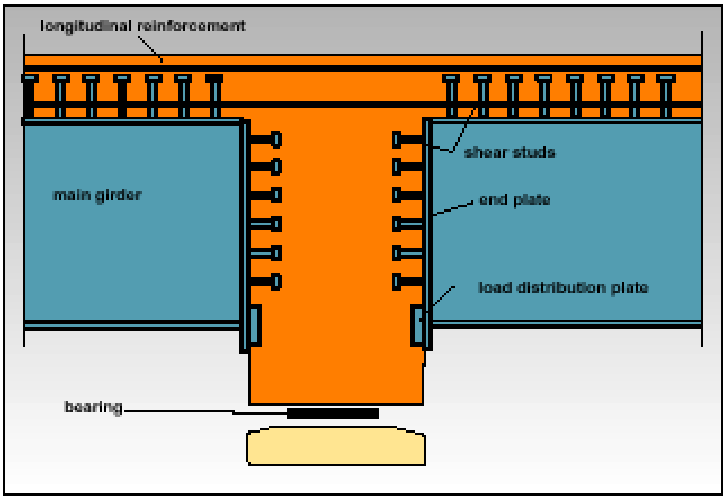


Figure 4-43: Concrete connection of main girders. - Cross section through bracing (schematic)

Reinforced concrete bracing over intermediate supports of multiple span bridges may be designed as splices of longitudinal girders. This construction method combines the following advantages:

- the longitudinal girders are erected as single-span girders;
- there is no need for welded or bolted splices.

Continuity is achieved by the use of vertical end plates and additional reinforcing bars in the deck slab. During concreting loads due to the dead weight of steel girders, formwork and wet concrete are carried by simply supported beams. After the concrete has hardened, moment resistance is provided at splices and subsequent loads are supported by continuous girders. Thus hogging bending is produced at supports only by super-imposed dead loads and variable actions. Forces are transmitted as follows:

The compressive force is directed through the end plate from the lower flange to the concrete, by using a thin end plate with a load distribution plate or a thick end plate. The tensile force flows from the upper beam flange through the shear studs into the longitudinal slab reinforcement. Studs welded to the vertical end plates transfer the shear force from the steel beams to the concrete bracing.

Figure 4-44: Concrete connection of main girders. - A pair of girders are lifted by crane into the recesses of the prefabricated formwork.

4.5.1.7 Intermediate transverse bracing

Vertical loads are laterally distributed by the means of the deck slab. Bracing within the span is needed to stabilise the girders, but does not participate in load distribution.

During construction, bracing prevents girders from lateral torsional buckling in sagging moment regions. After hardening of the concrete, the slab takes over this stabilizing action and bracing may be removed. With continuous girders, lateral buckling of the lower compression flange in hogging moment regions must be avoided. This is achieved using the bracing at the supports and, if needed, additional intermediate permanent bracing.

Figure 4-45: New public transport system in Oberhausen, Germany. –Crossing the DB railway line.

4.5.1.8 Deck slab

The deck consists of a non prestressed concrete slab with longitudinal and transverse reinforcement. Longitudinal reinforcement of continuous decks must be specially designed for crack width control.

Hogging moments of two and three span bridges may be reduced by lowering the structure at intermediate supports after concrete hardening.

4.5.1.9 Bearings and supports

In general, simple elastomeric bearing pads are used with composite bridges. The advantage of the low construction weight of composite construction results in smaller dimensions for the sub-structure, including abutments, piers and foundations (in particular, pile foundations). The resulting savings in construction costs are characteristic for this construction method.

4.5.1.10 Fabrication, transport and erection

Fabrication consists of the finishing of the rolled beams, i.e. cutting to length, drilling, cambering or bending about the strong axis and, if required, about the weak axis, welding of shear studs and bearing plates, surface preparation and application of a corrosion protection system. These operations can be carried out at the rolling mill's finishing department in a both cost effective and time-saving process. Alternatively the work can also be carried out fully or partially in a steel fabricators' workshop.

The ready-to-erect girders are transported to the construction site by rail or by lorry. The single components are relatively light and therefore only low capacity lifting equipment is required on site.

Figure 4-46: Railway bridge over the A23 motorway in Fretin, France. –Launching of the steel structure and assembled formwork.

The girders are often pre-assembled in pairs in order to get erection units with increased stability. The girders or pairs of girders are lifted into final position by mobile cranes. Alternatively elements are assembled in a nearly area and subsequently launched.

The low masses of steel components enable rapid assembly of the structure. In most cases, there is no need for temporary supports. When the routes crossed are in service, disruption to traffic can be kept to a minimum, especially if works are scheduled during off-peak hours.

4.5.1.11 In-situ Concrete Slab

The concrete slab can be cast in situ either on reusable formwork, or on precast concrete planks or profiled steel sheeting.

Figure 4-47: Precast reinforced concrete panels used for permanent formwork.

If certain conditions relating to construction and reinforcement are complied with, the precast planks contribute to carrying loads in the transverse direction, together with in situ concrete. For cantilever parts, traditional formwork is generally used, with supports attached to the edge beams.

Figure 4-48: Example of formwork support for cantilever part of slab.

4.5.1.12 Prefabricated Slab

As an alternative to in-site concreting, precast deck elements can be used. The main advantage of this method consist in the reduction of the number of site operations and a substantial saving in the construction time.

With twin girder - type bridges, the precast elements span the full width of the bridge as a single component. To allow connection with steel girders, slab elements have pockets for shear studs.

The prefabricated elements are placed in a mortar bedding on the girder flanges. Alternatively support may be designed with a small gap between flange and slab which is later filled in with grout.

Figure 4-49: Prefabricated slab. Example of transverse joint detailing:

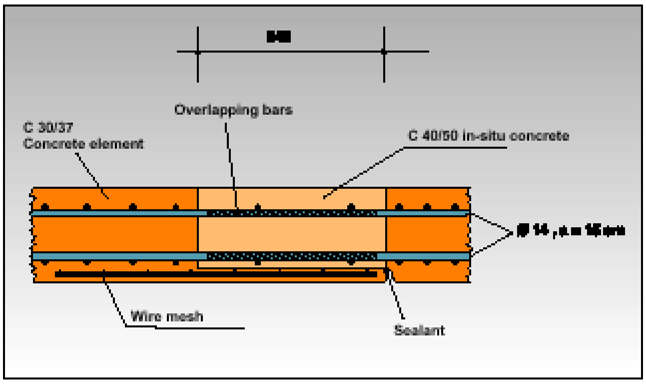


Figure 4-50: Deck units before concreting of pockets and joints

Consequently transverse joints and pockets are filled with concrete to connect the slab to the steel structure.

Hogging moments in continuous span girders can be reduced by lowering (jacking) the structure or intermediate supports after hardening of the concrete. The slab is prestressed longitudinally.

4.5.2 *Prestressed Composite Girders*

4.5.2.1 *Principle & Applications*

When a rolled beam is bent the tension flange is elastically stressed. In this state it is encased in concrete. Shear connections are provided for composite action.

After hardening of the concrete, bending is released. The concrete part is thus compressed –it is prestressed.

After erection on site the other flange is connected to the concrete slab. By this procedure double composite action is given.

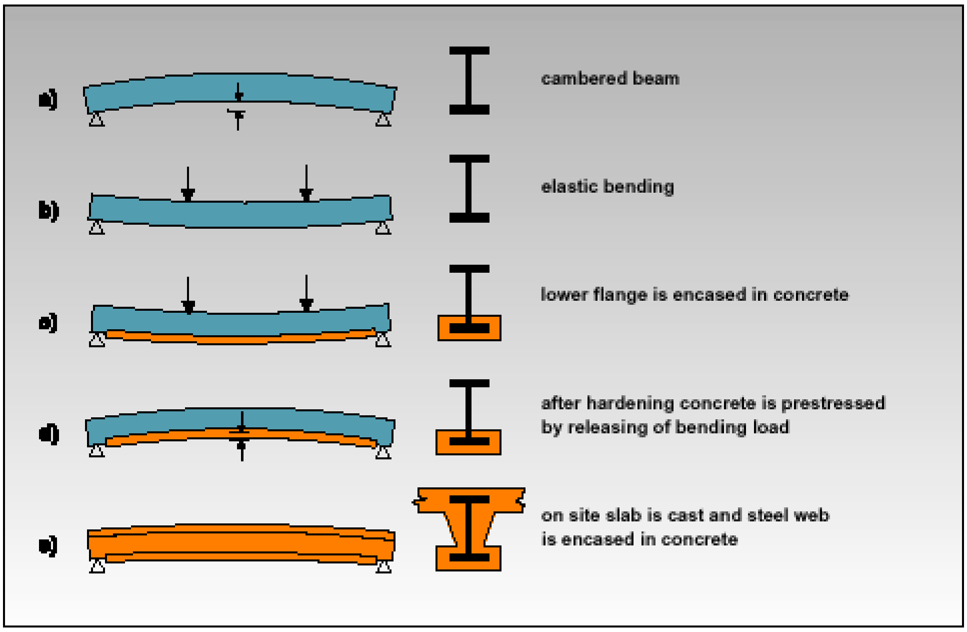


Figure 4-51: Fabrication of prestressed composite girder

A two-fold aim is achieved:

- the concrete slab increases the bending capacity and stiffness of the girder;
- under service load pre-compression stress in the concrete of the lower flange is reduced, but not totally. Thus no cracking occurs; concrete of the lower flange increases the flexural stiffness and reduces deflection.

Prestressed composite girders have:

- a very high moment capacity; they are suited to the construction of bridges carrying heavy loads, in particular railway bridges;
- a very high stiffness; deflections under service loading are small.

Due to these properties, prestressed girders are particularly suited to structures when the available construction depth is highly restricted. The slenderness ratio value (ratio of the span divided by the structural depth) may reach 45 for road bridges.

4.5.2.2 Fabrication and Erection

Beams are cambered at the rolling mill.

At the fabrication shop shear connectors and additional cover plates (if any) are welded to the flanges. Then elastic bending, encasing of lower flange with concrete and prestressing are carried out as described above.

Figure 4-52: Transportation of a Preflex beam

Figure 4-53: Single span bridge in Kerpen Horrem, Germany. - 41,25 m long HE 1000 A beams used for prestressed composite girders of " Preflex " type.

Most prestressed composite girders are simply supported beams. Sometimes girders are spliced at supports for continuous action.

After erection of the prefabricated girders formwork elements are placed on the lower flanges. They are specially shaped for casting of the slab and simultaneous encasing of the steel webs. Thus a protective treatment against corrosion is not necessary.

If particularly high loads are to be carried (for example railway loads) prestressed composite girders may be arranged side by side. No formwork is needed. By encasing in concrete girders are integrated into a massive slab.

4.5.3 Lateral torsional buckling of the beams during erection

The erection phase may involve very different procedures. The recommendations below are limited to the stage of casting the concrete slab. It is assumed that the formwork does not provide lateral restraint for the top flange. If the formwork is designed to brace the top flange the recommendations in 3.2.6 are applicable.

For continuous bridges the concrete is usually cast in stages. The first stage is usually casting the sagging region of an end span, see Figure 3-14. This creates a high sagging bending moment while the top flange is still unsupported, which may be critical for lateral torsional buckling. Next stage will be casting the sagging region of the adjacent span. If the beam is symmetric this stage is less critical than the previous but if there is no symmetry also this stage may need to be checked. The third stage would be to cast the remaining part over the pier. This stage creates the maximum hogging bending during erection but this is not likely to be critical. Worse cases will occur during operation.

4.5.3.1 Girders with intermediate cross braces

The cross braces will prevent twisting of the beams. Although the lateral displacement is not prevented the only possible lateral buckling mode is one with nodes at the locations of the cross braces. For this case the recommendations in 5.5.2.4 of EC3-2 are applicable. During casting it is usually the top flange that has to be checked. The bottom flange will normally be no problem.

4.5.3.2 Girders without cross braces other than at support

The beam will be free to buckle laterally over the whole length. It is a load case that gives maximum sagging moment that will be critical, like that in Figure 3-14. Unless the possible rotational restraint from the formwork is investigated it is recommended to assume the load to be applied at the top flange. For simple cases critical loads can be found in StBk-K2 (Swedish handbook, English or German to be added). It covers simply supported and continuous beams with uniform load. Usually it is assumed that the flanges are simply supported for lateral bending at the support in such solutions. In this case it may be worth considering the continuity also in lateral direction. The best recommendation for finding the critical load is to use a computer simulation.

Having found the critical load, the design according to 5.5.2 of EC3-2 is straightforward.

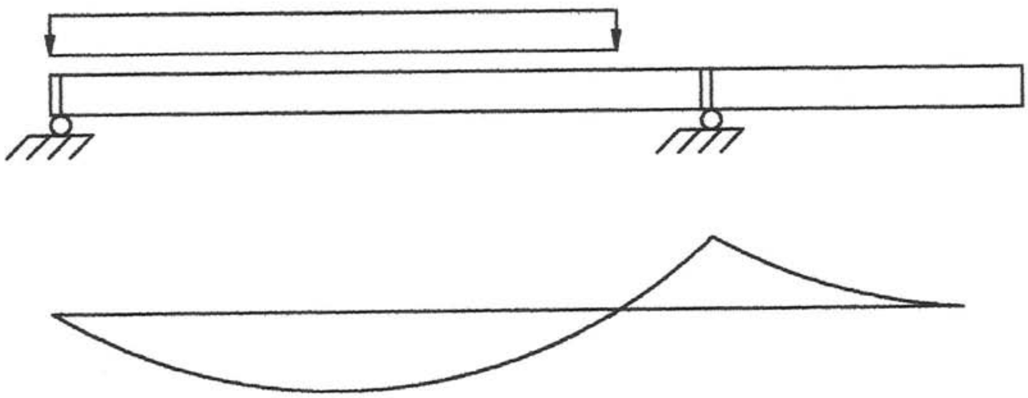


Figure 4-54: Typical load case during casting of the concrete deck.

Reference: Kommentarer till Stålbyggnadsnorm 70, StBK-K2. Bulletin nr. 100. 1973. Division of Building Statics and Structural Engineering, Royal Institute of Technology, Stocholm, Sweden

5 EXAMPLES

In this chapter two examples are carried out:

- 5.1 Two-span bridge using partially prefabricated elements
- 5.2 Simple bridge using fully prefabricated elements

The design examples are performed using the ANNEX A: Software CBD.

Therefore the user interfaces and a list of input data are added.

Tender drawings to the examples are added in ANNEX B: Tender Drawings.

5.1 Composite Bridge with Partially Prefabricated Elements

5.1.1 Static System

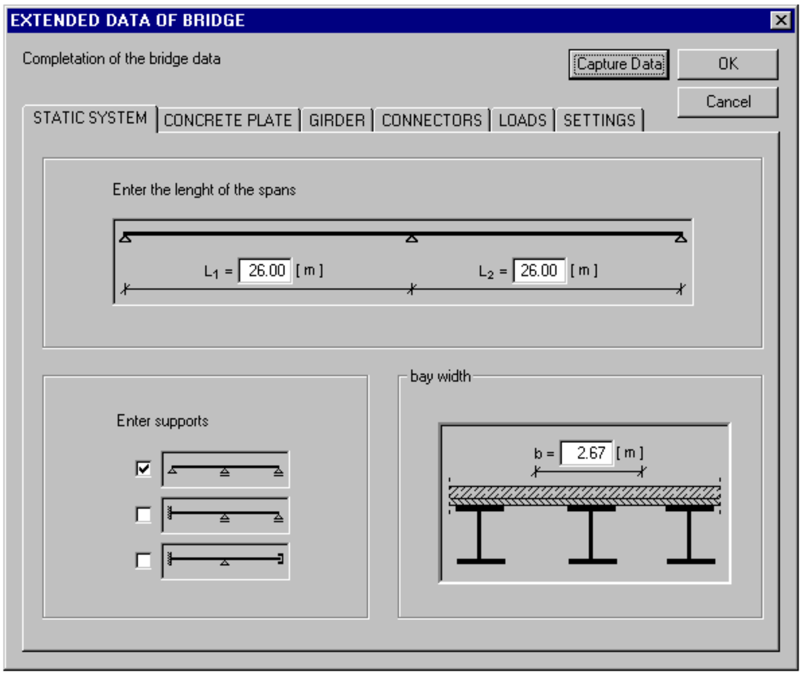


Figure 5-1: Static system

5.1.2 Concrete Plate

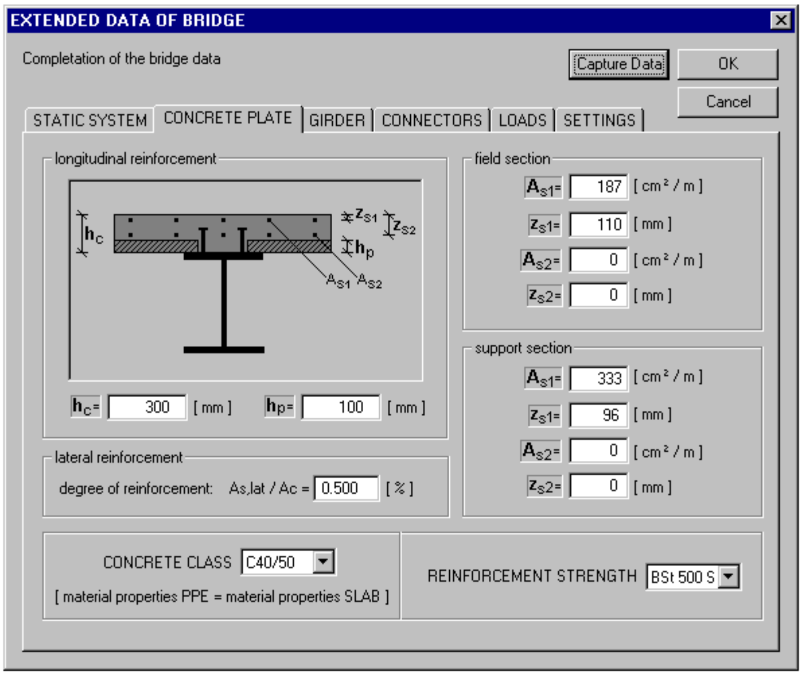


Figure 5-2: Concrete Plate

5.1.3 Hybrid Girder (inner beam)

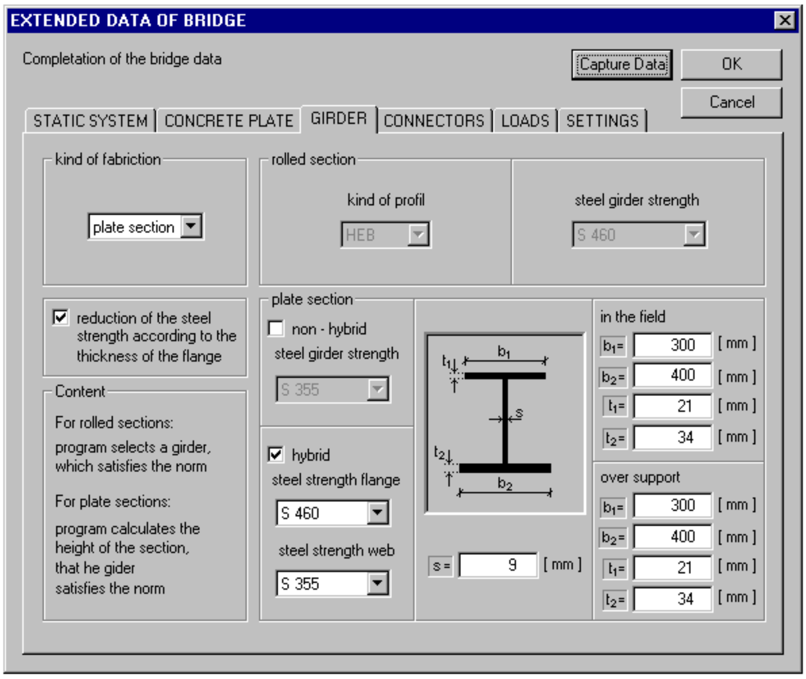


Figure 5-3: Hybrid girder (inner beam)

5.1.4 Hybrid Girder (outer beam)

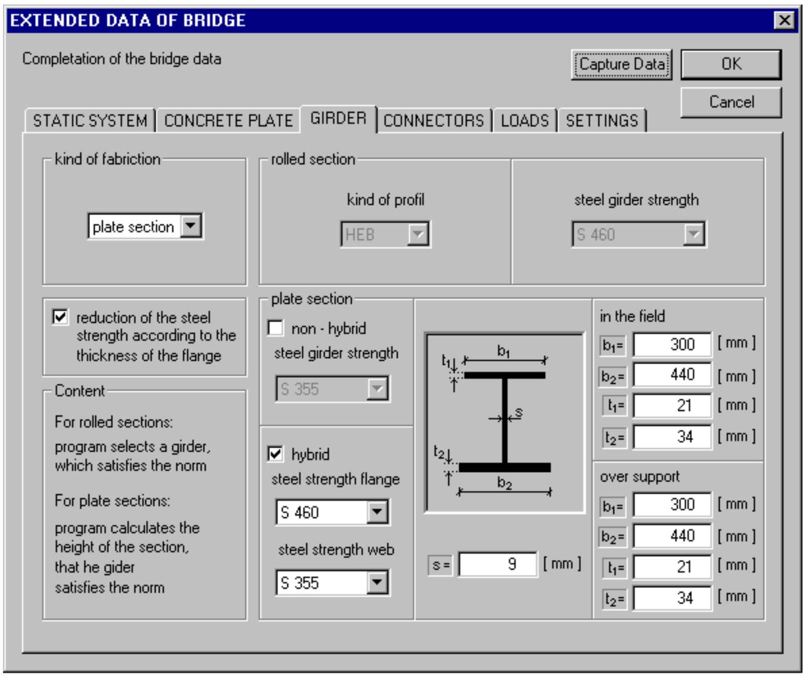


Figure 5-4: Hybrid girder (outer beam)

5.1.5 Shear Connectors

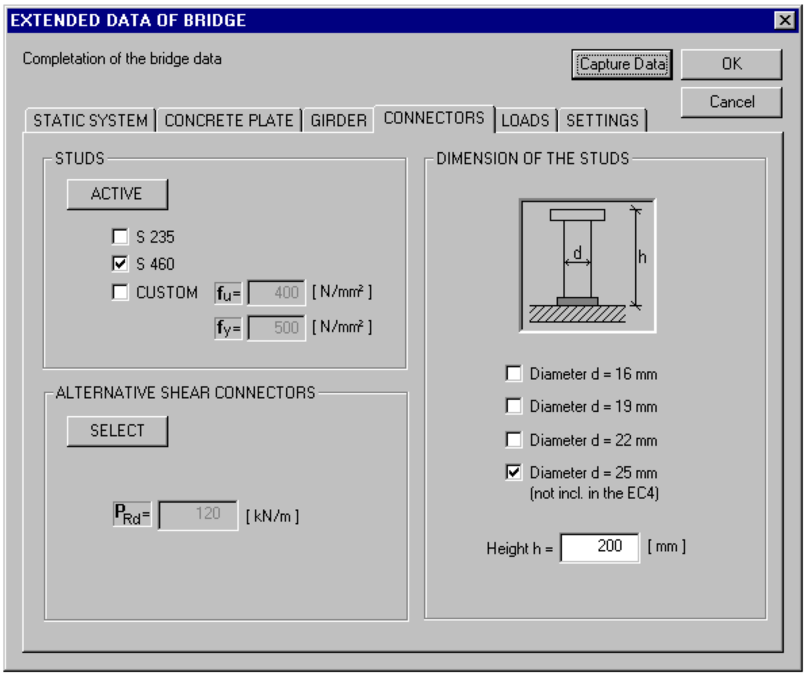


Figure 5-5: Shear connectors

5.1.6 Loads on the Structure

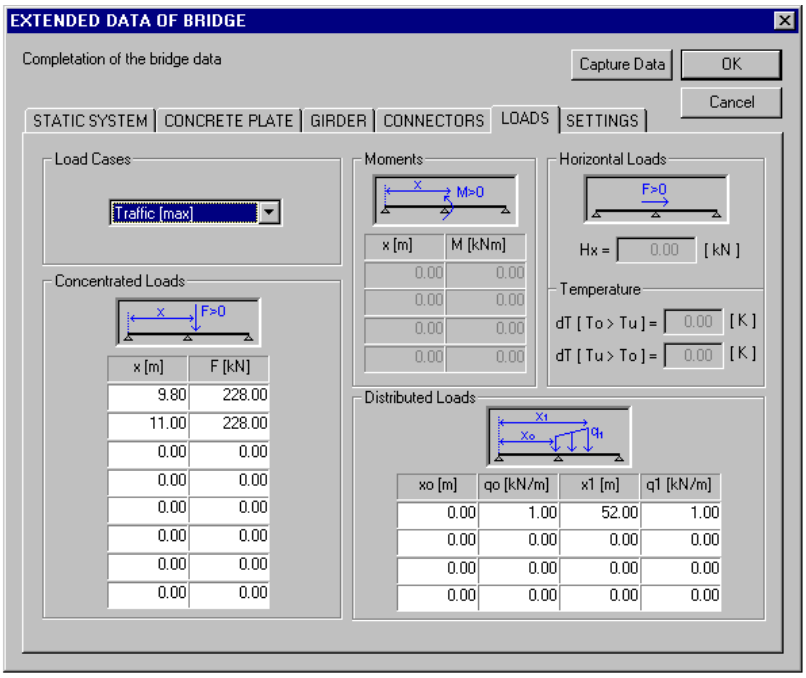


Figure 5-6: Loads on the structure

5.1.6.1 Loads on an Inner Beam

Table 5-1: Loads on the inner beam

Loads	Distributed	Start point	End point	Point Load	Place	Moment	Place	dT [To > Tu]
	[kN/m]	[m]	[m]	[kN]	[m]	[kNm]	[m]	[K]
Casting Loads	2,00	0,00	52,00	-	-	-	-	-
	2,00	8,90	11,90	-	-	-	-	-
Permanent Loads	6,46	0,00	52,00	-	-	-	-	-
max Traffic Loads	-	-	-	228,00	9,80	-	-	-
	-	-	-	228,00	11,00	-	-	-
	1,00	0,00	52,00	-	-	-	-	-
min Traffic Loads	-	-	-	228,00	14,22	-	-	-
	-	-	-	228,00	15,42	-	-	-
	1,00	0,00	52,00	-	-	-	-	-
Breaking Loads	-	-	-	900,00	52,00	419,00	26,00	-
Temperature Loads	-	-	-	-	-	-	-	10,00
Temperature Loads	-	-	-	-	-	-	-	-7,00

5.1.6.2 Loads on an Outer Beam

Table 5-2: Loads on the outer beam

Loads	Distributed	Start point	End point	Point Load	Place	Moment	Place	dT [To > Tu]
	[kN/m]	[m]	[m]	[kN]	[m]	[kNm]	[m]	[K]
Casting Loads	1,00	0,00	52,00	-	-	-	-	-
	1,00	8,90	11,90	-	-	-	-	-
Permanent Loads	16,03	0,00	52,00	-	-	-	-	-
max Traffic Loads	-	-	-	138,00	9,80	-	-	-
	-	-	-	138,00	11,00	-	-	-
	0,60	0,00	52,00	-	-	-	-	-
min Traffic Loads	-	-	-	138,00	14,22	-	-	-
	-	-	-	138,00	15,42	-	-	-
	0,60	0,00	52,00	-	-	-	-	-
Breaking Loads	-	-	-	900,00	52,00	419,00	26,00	-
Temperature Loads	-	-	-	-	-	-	-	10,00
Temperature Loads	-	-	-	-	-	-	-	-7,00

5.1.7 Settings for the Calculation

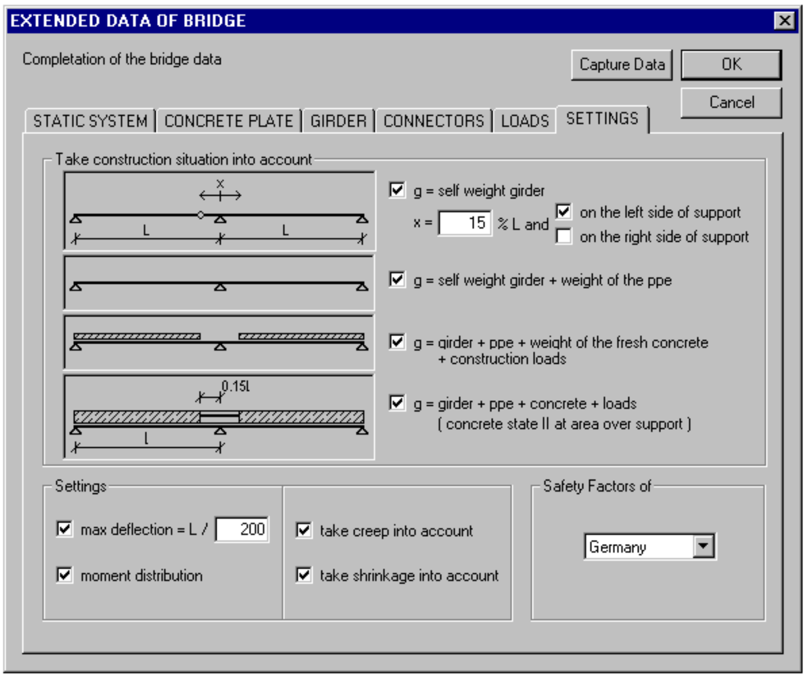


Figure 5-7: Settings for the calculation

5.1.7.1 Results of the Calculation (inner beam)

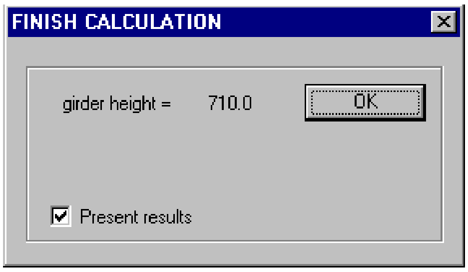


Figure 5-8: Results of the Calculation

5.1.7.2 List of Input data

Table 5-3: List of Input data: example 1

Dialog	**Designation**	**Value**	**Unit**	**Specification**
Static System	L_1	26.0	[m]	Length of span 1
	L_2	26.0	[m]	Length of span 2
	L_3	00.0	[m]	Length of span 3
	Enter support 1	√		Specification of support situation
	Enter support 2			Specification of support situation
	Enter support 3			Specification of support situation
	b	2.67	[m]	Enter bay width

Concrete Plate	h_c	300	[mm]	Height of concrete plate
	h_p	100	[mm]	Height of pre-fabricated element
	$A_{s,\,lat} / A_c$	0.5	%	Degree of lateral reinforcement
	A_{s1} (field)	187	cm²/m	Upper reinforcement area of field section
	z_{s1} (field)	110	[mm]	Distance of upper reinforcement from the top of the concrete plate
	A_{s2} (field)	0	cm²/m	Lower reinforcement area of field section
	z_{s2} (field)	0	[mm]	Distance of lower reinforcement from the top of the concrete plate
	A_{s1} (support)	333	cm²/m	Upper reinforcement area of support section
	z_{s1} (support)	96	[mm]	Distance of upper reinforcement from the top of the concrete plate
	A_{s2} (support)	0	cm²/m	Lower reinforcement area of support section
	z_{s2} (support)	0	[mm]	Distance of lower reinforcement from the top of the concrete plate
	Concrete class	C 40 / 50		
	Reinforcement strength	BSt 500 S		

Girder	Kind of fabrication	Plate section		Rolled section or plate section
	Kind of profile	-		Specify the rolled profile type
	Steel girder strength	-		Specify the steel strength of rolled profile
	Plate section	hybrid		Enter kind of plate layout
	Steel strength flange	S 460		Steel strength of the flange
	Steel strength web	S 355		Steel strength of the web
	Reduction of steel strength	√		Reduction of steel strength according to the thickness of the plate
	s	9	[mm]	Web thickness
	b_1 (field)	300	[mm]	Upper flange width of field section
	b_2 (field)	400	[mm]	Lower flange width of field section
	t_1 (field)	21	[mm]	Upper flange thickness of field section
	t_2 (field)	34	[mm]	Lower flange thickness of field section
	b_1 (support)	300	[mm]	Upper flange width of support section
	b_2 (support)	400	[mm]	Lower flange width of support section
	t_1 (support)	21	[mm]	Upper flange thickness of support section
	t_2 (support)	34	[mm]	Lower flange thickness of support section

Connectors	Studs	active		Select studs as shear connectors
	S 235			Use S 235 for studs
	S 460	√		Use S 460 for studs
	CUSTOM			Use a customised material for studs
	f_u	-	[N/mm²]	Ultimate strain
	f_y	-	[N/mm²]	Yield strength
	Diameter d = 16 mm			Diameter of studs
	Diameter d = 19 mm			Diameter of studs
	Diameter d = 22 mm			Diameter of studs
	Diameter d = 25 mm	√		Diameter of studs
	height h =	200	[mm]	Height of studs
	alternative shear connectors	select		Use an alternative shear connector
	P_{Rd}		[kN/m]	Bearing resistance of the alternative shear connector

Loads	**Load cases**	**Casting**		**Select load case**
	$x_{[1]-[8]}$	0	[m]	Distance of point load 1 – 8 from left support
	$F_{[1]-[8]}$	0	[kN]	Magnitude of point load 1 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	2	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	52	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	2	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2]}$	8.9	[m]	Distance starting point of distributed load 2 from left support
	$q_{0\,[2]}$	2	[kN/m]	Magnitude of the distributed load 2 at starting point
	$x_{1\,[2]}$	11.9	[m]	Distance end point of distributed load 2 from left support
	$q_{1\,[2]}$	2	[kN/m]	Magnitude of distributed load 2 at end point
	$x_{0\,[3],[4]}$	0	[m]	Distance starting point of distributed load 3, 4 from left support
	$q_{0\,[3],[4]}$	0	[kN/m]	Magnitude of the distributed load 3, 4 at starting point
	$x_{1\,[3],[4]}$	0	[m]	Distance end point of distributed load 3, 4 from left support
	$q_{1\,[3],[4]}$	0	[kN/m]	Magnitude of distributed load 3, 4 at end point

Loads	**Load cases**	**Permanent**		**Select load case**
	$x_{[1]-[8]}$	0	[m]	Distance of point load 1 - 8 from left support
	$F_{[1]-[8]}$	0	[kN]	Magnitude of point load 1 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	6.46	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	52	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	6.46	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2],\,[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\,[2],\,[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\,[2],\,[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\,[2],\,[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	**Load cases**	**Traffic [max]**		**Select load case**
	$x_{[1]}$	9.8	[m]	Distance of point load 1 from left support
	$F_{[1]}$	228	[kN]	Magnitude of point load 1
	$x_{[2]}$	11.0	[m]	Distance of point load 2 from left support
	$F_{[2]}$	228	[kN]	Magnitude of point load 2
	$x_{[3]-[8]}$	0	[m]	Distance of point load 3 – 8 from left support
	$F_{[3]-[8]}$	0	[kN]	Magnitude of point load 3 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	1.0	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	52	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	1.0	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2],[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\,[2],[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\,[2],[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\,[2],[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	Load cases	Traffic [min]		Select load case
	$x_{[1]}$	14.22	[m]	Distance of point load 1 from left support
	$F_{[1]}$	228	[kN]	Magnitude of point load 1
	$x_{[2]}$	15.42	[m]	Distance of point load 2 from left support
	$F_{[2]}$	228	[kN]	Magnitude of point load 2
	$x_{[3]-[8]}$	0	[m]	Distance of point load 3 – 8 from left support
	$F_{[3]-[8]}$	0	[kN]	Magnitude of point load 3 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	1.0	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	52	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	1.0	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2],\,[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\,[2],\,[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\,[2],\,[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\,[2],\,[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	Load cases	Breaking		Select load case
	$x_{[1]}$	26.0	[m]	Distance of moment 1 from left support
	$F_{[1]}$	419	[kNm]	Magnitude of moment 1
	$x_{[2]-[8]}$	0	[m]	Distance of moment 2 – 4 from left support
	$F_{[2]-[8]}$	0	[kNm]	Magnitude of moment 2 - 4
	H_x	0	[kN]	Horizontal load due to breaking or acceleration

Loads	Load cases	Temperature		Select load case
	dT [$T_o > T_u$]	10	[K]	Top of concrete warmer than bottom concrete
	dT [$T_u > T_o$]	7	[K]	Top of concrete colder than bottom concrete

Settings	Constr. Sit. 1			Take construction situation 1 into account
	x	15	% L	Distance of the hinge in construction sit. 1
	Right side of support			Place of the hinge referring to the column
	Left side of support	√		Place of the hinge referring to the column
	Constr. Sit. 2	√		Take construction situation 2 into account
	Constr. Sit. 3	√		Take construction situation 3 into account
	Constr. Sit. 4	√		Take construction situation 4 into account
	Max deflection	√		Perform deflection check
	L /	200		Deflection limit
	Moment distribution	√		Take moment distribution into account
	Creep	√		Take creep into account
	Shrinkage	√		Take shrinkage into account
	Safety factors	Germany		Specify set of combination factors for various countries

Combination factors (ULS)	ψ (Casting)	1.0		Combination factor for casting loads (ULS)
	ψ (Traffic TS)	1.0		Combination factor for single traffic loads (ULS)
	ψ (Traffic UDL)	0.4		Combination factor for uniformly distributed traffic loads (ULS)
	ψ (Breaking)	0.7		Combination factor for breaking loads (ULS)
	ψ (Temperature)	0.6		Combination factor for temperature loads (ULS)

Combination factors (SLS)	ψ (Casting)	1.0		Combination factor for casting loads (SLS)
	ψ (Traffic TS)	1.0		Combination factor for single traffic loads (SLS)
	ψ (Traffic UDL)	0.4		Combination factor for uniformly distributed traffic loads (SLS)
	ψ (Breaking)	0.7		Combination factor for breaking loads (SLS)
	ψ (Temperature)	0.6		Combination factor for temperature loads (SLS)

5.1.7.3 *Partial safety factors [NAD]*

Table 5-4: Partial safety factors

μ_g	μ_q	μ_{M0}	μ_{M1}	ν_{vec}	μ_c	μ_s	μ_v
1.35	1.50	1.10	1.10	0.80	1.50	1.15	1.25

5.1.7.4 Cross Section

5.1.7.4.1 Cross Section over support

	Section	Units	Value
Steel beam	I_{yy}	[cm^4]	236492
	$W_{c,o}$	[cm^3]	0
	$W_{c,u}$	[cm^3]	0
	$W_{st,o}$	[cm^3]	- 5213
	$W_{st,u}$	[cm^3]	8879
Partially composite slab			
Short term	I_{yy}	[cm^4]	982197
	$W_{0,c,o}$	[cm^3]	-250536
	$W_{0,c,u}$	[cm^3]	-435813
	$W_{0,st,o}$	[cm^3]	-72636
	$W_{0,st,u}$	[cm^3]	16796
Long term	I_{yy}	[cm^4]	753360
	$W_{b,c,o}$	[cm^3]	-506032
	$W_{b,c,u}$	[cm^3]	-565653
	$W_{b,st,o}$	[cm^3]	-24445
	$W_{b,st,u}$	[cm^3]	18293
Shrinkage	I_{yy}	[cm^4]	843296
	$W_{s,c,o}$	[cm^3]	-354499
	$W_{s,c,u}$	[cm^3]	-478000
	$W_{s,st,o}$	[cm^3]	-35142
	$W_{s,st,u}$	[cm^3]	17568
Fully composite slab			
Short term	I_{yy}	[cm^4]	1332023
	$W_{0,c,o}$	[cm^3]	-473156
	$W_{0,c,u}$	[cm^3]	609675
	$W_{0,st,o}$	[cm^3]	101613
	$W_{0,st,u}$	[cm^3]	15651
Long term	I_{yy}	[cm^4]	1182758
	$W_{b,c,o}$	[cm^3]	-1105768
	$W_{b,c,u}$	[cm^3]	-30137254
	$W_{b,st,o}$	[cm^3]	155419
	$W_{b,st,u}$	[cm^3]	14857
Long term	I_{yy}	[cm^4]	1241473
	$W_{s,c,o}$	[cm^3]	-598816
	$W_{s,c,u}$	[cm^3]	3109858
	$W_{s,st,o}$	[cm^3]	132595
	$W_{s,st,u}$	[cm^3]	15259

5.1.7.4.2 Cross Section in the field

	Section	Units	Value
Steel beam	I_{yy}	[cm^4]	236492
	$W_{c,o}$	[cm³]	0
	$W_{c,u}$	[cm³]	0
	$W_{st,o}$	[cm³]	- 5213
	$W_{st,u}$	[cm³]	8879
Partially composite slab			
Short term	I_{yy}	[cm^4]	863940
	$W_{0,c,o}$	[cm³]	-220372
	$W_{0,c,u}$	[cm³]	-383341
	$W_{0,st,o}$	[cm³]	-63890
	$W_{0,st,u}$	[cm³]	14774
Long term	I_{yy}	[cm^4]	638070
	$W_{b,c,o}$	[cm³]	-422298
	$W_{b,c,u}$	[cm³]	-471327
	$W_{b,st,o}$	[cm³]	-20859
	$W_{b,st,u}$	[cm³]	15409
Shrinkage	I_{yy}	[cm^4]	725756
	$W_{s,c,o}$	[cm³]	-304395
	$W_{s,c,u}$	[cm³]	-410159
	$W_{s,st,o}$	[cm³]	-30313
	$W_{s,st,u}$	[cm³]	15102
Fully composite slab			
Short term	I_{yy}	[cm^4]	1270879
	$W_{0,c,o}$	[cm³]	-395312
	$W_{0,c,u}$	[cm³]	711928
	$W_{0,st,o}$	[cm³]	118655
	$W_{0,st,u}$	[cm³]	15365
Long term	I_{yy}	[cm^4]	1089774
	$W_{b,c,o}$	[cm³]	-848025
	$W_{b,c,u}$	[cm³]	-4462591
	$W_{b,st,o}$	[cm³]	236949
	$W_{b,st,u}$	[cm³]	14227
Long term	I_{yy}	[cm^4]	1163055
	$W_{s,c,o}$	[cm³]	-511810
	$W_{s,c,u}$	[cm³]	5718311
	$W_{s,st,o}$	[cm³]	159917
	$W_{s,st,u}$	[cm³]	14672

5.1.8 Design

The effective width of the concrete plate $b_{eff} = 2.67$ [m]

The beam is rated into class 2 therefore the checks can be carried out plastically.

5.1.8.1 Bearing Resistance in the negative moment area

Steel beam:	$V_{sd} = 941.9$	[kN]
	$M_{sd} = 2688.4$	[kNm]

The beam has to be restrained against torsional buckling during construction.

Fully Composite beam [t = 28d]:	$V_{sd} = 947.8$	[kN]
	$M_{sd} = 6464.0$	[kNm]
Fully Composite beam [t = ∞d]:	$V_{sd} = 947.8$	[kN]
	$M_{sd} = 6039.5$	[kNm]

5.1.8.2 Bearing Resistance in the positive moment area

Steel beam:	$V_{sd} = 941.9$	[kN]
	$M_{sd} = 2688.4$	[kNm]

The beam has to be restrained against torsional buckling during construction.

Partially Composite beam:	$V_{sd} = 947.8$	[kN]
	$M_{sd} = 5115.0$	[kNm]
Fully Composite beam [t = 28d]:	$V_{sd} = 947.8$	[kN]
	$M_{sd} = 6718.2$	[kNm]
Fully Composite beam [t = ∞d]:	$V_{sd} = 947.8$	[kN]
	$M_{sd} = 6718.2$	[kNm]

5.1.8.3 Further Design Checks

The reinforcement diameter has to be smaller than 44 [mm] or the maximal distance between the reinforcement bars has to be smaller than 25 [cm] to satisfy the crack width design checks of the Eurocode.

The shear connectors have got a design resistance of P_{rd} = 172 [kN].

The following criteria for the studs have to be satisfied:

- average welded bead diameter:	d_w	> 1.25 d	= 31.25	[mm]
- average welded bead height:	h_{wm}	> 0.2 d	= 5.00	[mm]
- minimum welded bead height:	$h_{w,min}$	> 0.15 d	= 3.75	[mm]
- minimum distance in direction of shear:	e_{min}	> 5 d	= 125.00	[mm]
- minimum distance vertical to direction of shear:	e_{min}	> 2.5 d	= 62.50	[mm]

If studs are not placed directly over the web, the diameter of the studs should be less than 22.5 [mm]

The following number of shear connectors have to be applied:

- 63 between support 1 and M_{max} in span 1
- 148 between M_{max} in span 1 and support 2
- 148 between support 2 and M_{max} in span 2
- 64 between M_{max} in span 2 and support 3

The deflection checks are satisfied for all spans.

5.1.9 Results of the Calculation (outer beam)

5.1.9.1 Partial safety factors [NAD]

Table 5-5: Partial safety factors, example 1

μ_g	M_q	μ_{M0}	μ_{M1}	ν_{vec}	μ_c	μ_s	μ_v
1.35	1.50	1.10	1.10	0.80	1.50	1.15	1.25

5.1.9.2 *Cross Section*

5.1.9.2.1 Cross Section over support

	Section	Units	Value
Steel beam	I_{yy}	[cm^4]	244539
	$W_{c,o}$	[cm³]	0
	$W_{c,u}$	[cm³]	0
	$W_{st,o}$	[cm³]	- 5247
	$W_{st,u}$	[cm³]	9631
Partially composite slab			
Short term	I_{yy}	[cm^4]	1049331
	$W_{0,c,o}$	[cm³]	-255950
	$W_{0,c,u}$	[cm³]	-431275
	$W_{0,st,o}$	[cm³]	-71879
	$W_{0,st,u}$	[cm³]	18281
Long term	I_{yy}	[cm^4]	797238
	$W_{b,c,o}$	[cm³]	-517760
	$W_{b,c,u}$	[cm³]	-576490
	$W_{b,st,o}$	[cm³]	-24691
	$W_{b,st,u}$	[cm³]	20076
Shrinkage	I_{yy}	[cm^4]	895554
	$W_{s,c,o}$	[cm³]	-361878
	$W_{s,c,u}$	[cm³]	-481440
	$W_{s,st,o}$	[cm³]	-35302
	$W_{s,st,u}$	[cm³]	19205
Fully composite slab			
Short term	I_{yy}	[cm^4]	1426104
	$W_{0,c,o}$	[cm³]	-492491
	$W_{0,c,u}$	[cm³]	677709
	$W_{0,st,o}$	[cm³]	112952
	$W_{0,st,u}$	[cm³]	16852
Long term	I_{yy}	[cm^4]	1267461
	$W_{b,c,o}$	[cm³]	-1149753
	$W_{b,c,u}$	[cm³]	-16658990
	$W_{b,st,o}$	[cm³]	183827
	$W_{b,st,u}$	[cm³]	16065
Long term	I_{yy}	[cm^4]	1330145
	$W_{s,c,o}$	[cm³]	-625557
	$W_{s,c,u}$	[cm³]	3858264
	$W_{s,st,o}$	[cm³]	152129
	$W_{s,st,u}$	[cm³]	16474

5.1.9.2.2 Cross Section in the field

	Section	Units	Value
Steel beam	I_{yy}	[cm^4]	244539
	$W_{c,o}$	[cm³]	0
	$W_{c,u}$	[cm³]	0
	$W_{st,o}$	[cm³]	- 5247
	$W_{st,u}$	[cm³]	9631
Partially composite slab			
Short term	I_{yy}	[cm^4]	918828
	$W_{0,c,o}$	[cm³]	-224118
	$W_{0,c,u}$	[cm³]	-377638
	$W_{0,st,o}$	[cm³]	-62940
	$W_{0,st,u}$	[cm³]	16007
Long term	I_{yy}	[cm^4]	670225
	$W_{b,c,o}$	[cm³]	-428194
	$W_{b,c,u}$	[cm³]	-475974
	$W_{b,st,o}$	[cm³]	-20918
	$W_{b,st,u}$	[cm³]	16772
Shrinkage	I_{yy}	[cm^4]	765904
	$W_{s,c,o}$	[cm³]	-308705
	$W_{s,c,u}$	[cm³]	-410398
	$W_{s,st,o}$	[cm³]	-30264
	$W_{s,st,u}$	[cm³]	16403
Fully composite slab			
Short term	I_{yy}	[cm^4]	1359545
	$W_{0,c,o}$	[cm³]	-411283
	$W_{0,c,u}$	[cm³]	802386
	$W_{0,st,o}$	[cm³]	133731
	$W_{0,st,u}$	[cm³]	16546
Long term	I_{yy}	[cm^4]	1165103
	$W_{b,c,o}$	[cm³]	-876617
	$W_{b,c,u}$	[cm³]	-3982746
	$W_{b,st,o}$	[cm³]	312471
	$W_{b,st,u}$	[cm³]	15385
Long term	I_{yy}	[cm^4]	1244101
	$W_{s,c,o}$	[cm³]	-532437
	$W_{s,c,u}$	[cm³]	9027632
	$W_{s,st,o}$	[cm³]	190309
	$W_{s,st,u}$	[cm³]	15841

5.1.10 Design

The effective width of the concrete plate $b_{eff} = 2.67$ [m]

The beam is rated into class 2 therefore the checks can be carried out plastically.

5.1.10.1 Bearing Resistance in the negative moment area

Steel beam:	$V_{sd} = 941.6$	[kN]
	$M_{sd} = 2635.0$	[kNm]

The beam has to be restrained against torsional buckling during construction.

Fully Composite beam [t = 28d]:	$V_{sd} = 945.2$	[kN]
	$M_{sd} = 6014.3$	[kNm]
Fully Composite beam [t = ∞d]:	$V_{sd} = 945.2$	[kN]
	$M_{sd} = 5642.0$	[kNm]

5.1.10.2 Bearing Resistance in the positive moment area

Steel beam:	$V_{sd} = 941.6$	[kN]
	$M_{sd} = 2635.0$	[kNm]

The beam has to be restrained against torsional buckling during construction.

Partially Composite beam:	$V_{sd} = 945.2$	[kN]
	$M_{sd} = 5424.8$	[kNm]
Fully Composite beam [t = 28d]:	$V_{sd} = 945.2$	[kN]
	$M_{sd} = 7087.0$	[kNm]
Fully Composite beam [t = ∞d]:	$V_{sd} = 945.2$	[kN]
	$M_{sd} = 7087.0$	[kNm]

5.1.10.3 Further Design Checks

The reinforcement diameter has to be smaller than 44 [mm] or the maximal distance between the reinforcement bars has to be smaller than 25 [cm] to satisfy the crack width design checks of the Eurocode.

The shear connectors have a design resistance of P_{rd} = 172 [kN].

The following criteria for the studs have to be satisfied:

- average welded bead diameter:	d_w	> 1.25 d	= 31.25	[mm]
- average welded bead height:	h_{wm}	> 0.2 d	= 5.00	[mm]
- minimum welded bead height:	$h_{w,min}$	> 0.15 d	= 3.75	[mm]
- minimum distance in direction of shear:	e_{min}	> 5 d	= 125.00	[mm]
- minimum distance vertical to direction of shear:	e_{min}	> 2.5 d	= 62.50	[mm]

If studs are not placed directly over the web, the diameter of the studs should be less than 22.5 [mm]

The following number of shear connectors have to be applied:

- 66 between support 1 and M_{max} in span 1
- 151 between M_{max} in span 1 and support 2
- 151 between support 2 and M_{max} in span 2
- 66 between M_{max} in span 2 and support 3

The deflection checks are satisfied for all spans.

5.2 Composite Bridge with Fully Prefabricated Elements

5.2.1 Static System

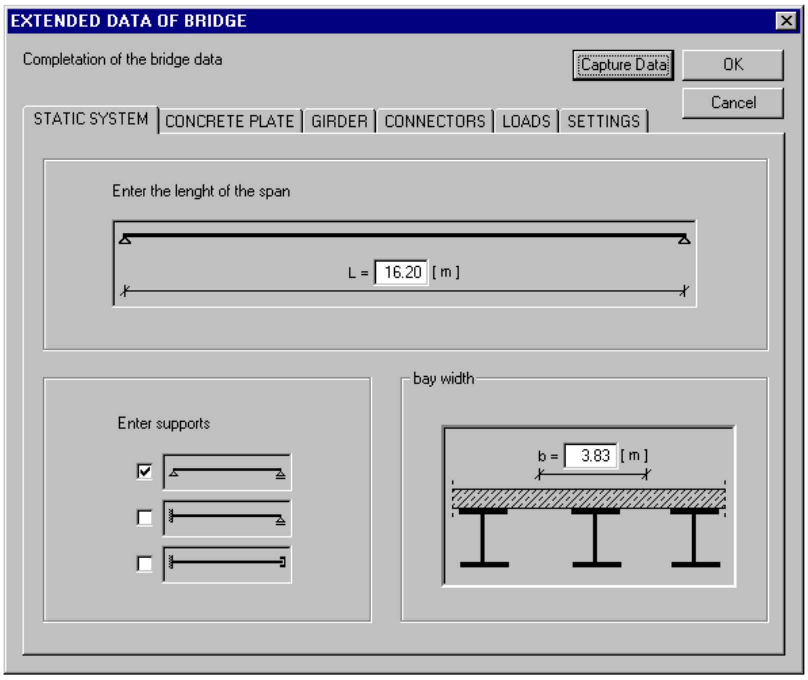


Figure 5-9: Static system

5.2.2 Concrete Plate

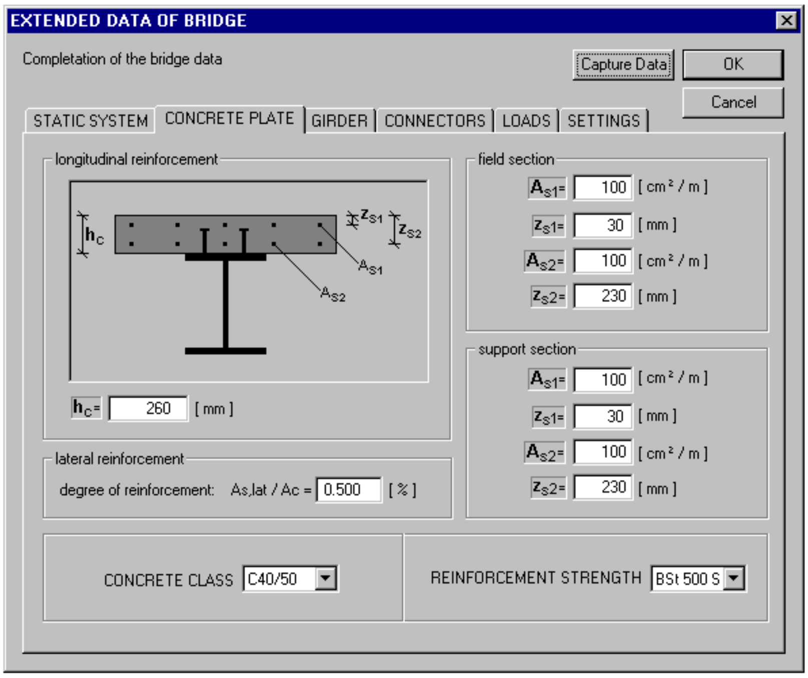


Figure 5-10: Concrete plate

5.2.3 Girder in S 355

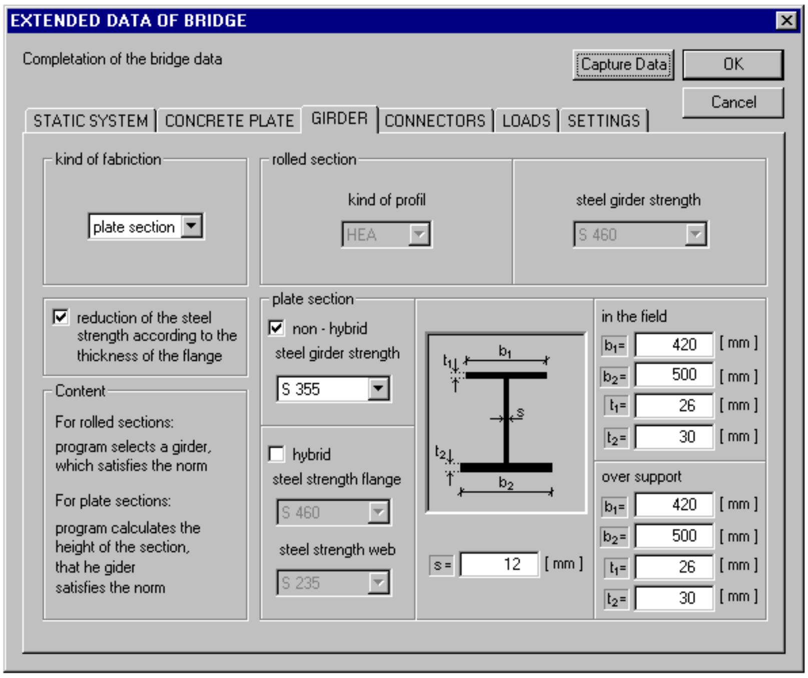


Figure 5-11: Girder in S 355

5.2.4 Shear Connectors

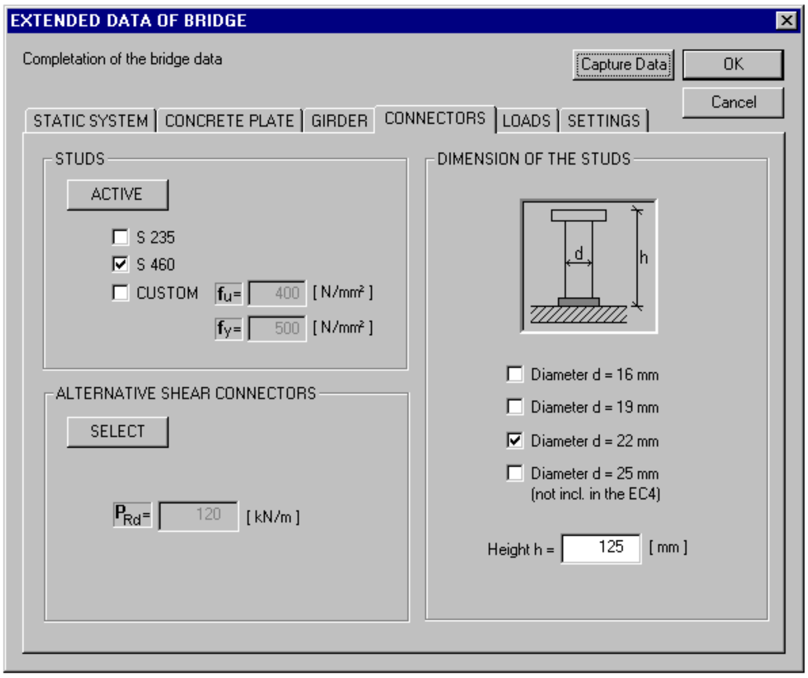


Figure 5-12: Shear connectors

5.2.5 Loads on the Structure

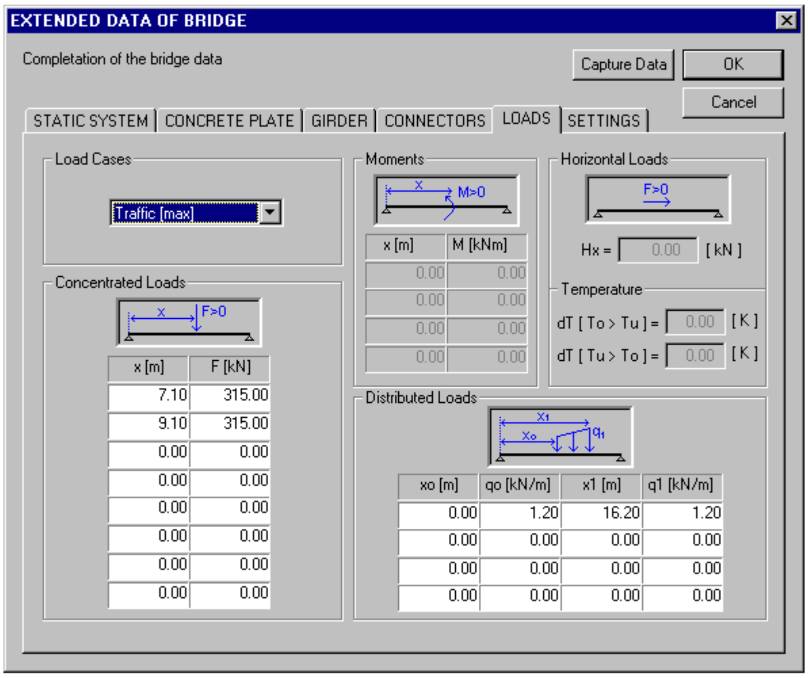


Figure 5-13: Loads on the structure

5.2.5.1 Loads on Beam

Table 5-6: Loads on beam

Loads	Distributed	Start point	End point	Point Load	Place	Moment	Place	dT [To > Tu]
	[kN/m]	[m]	[m]	[kN]	[m]	[kNm]	[m]	[K]
Casting Loads	2,90	0,00	16,20	-	-	-	-	-
	2,90	6,60	9,60	-	-	-	-	-
Permanent Loads	30,80	0,00	16,20	-	-	-	-	-
max Traffic Loads	-	-	-	315,00	7,10	-	-	-
	-	-	-	315,00	9,10	-	-	-
	1,20	0,00	16,20	-	-	-	-	-
min Traffic Loads	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-
	1,20	0,00	16,20	-	-	-	-	-
Breaking Loads	-	-	-	900,00	16,20	419,00	26,00	-
Temperature Loads	-	-	-	-	-	-	-	10,00
Temperature Loads	-	-	-	-	-	-	-	-7,00

5.2.6 Settings for the Calculation

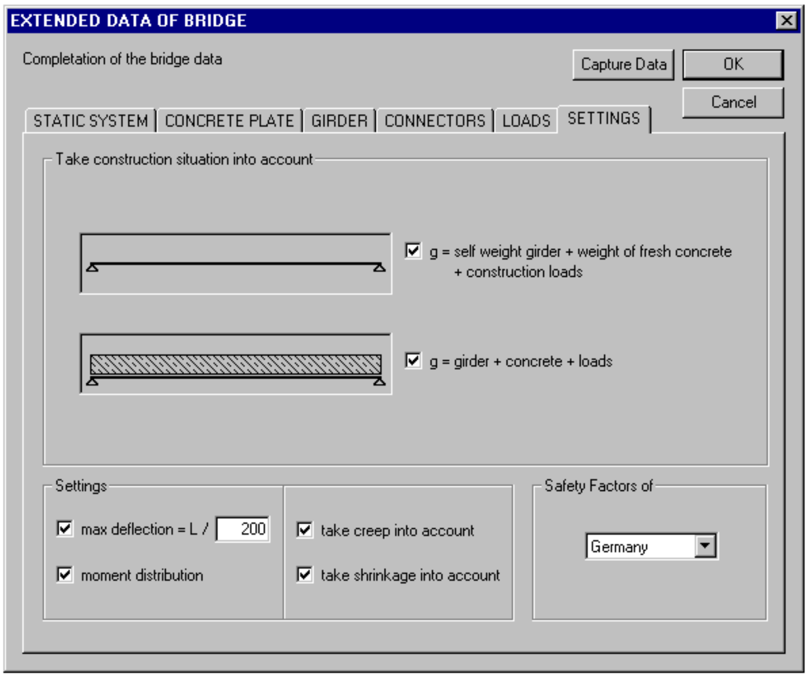


Figure 5-14: Settings for the calculation

5.2.7 Results of the Calculation

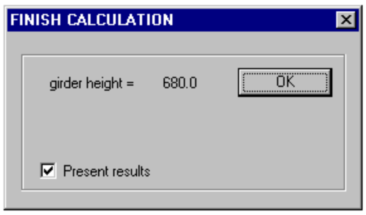


Figure 5-15: Results of the Calculation

5.2.7.1 List of Input data

Table 5-7: List of Input data: example 1

Dialog	Designation	Value	Unit	Specification
Static System	L_1	16.2	[m]	Length of span 1
	L_2	00.0	[m]	Length of span 2
	L_3	00.0	[m]	Length of span 3
	Enter support 1	√		Specification of support situation
	Enter support 2			Specification of support situation
	Enter support 3			Specification of support situation
	b	3.83	[m]	Enter bay width

Concrete Plate	h_c	260	[mm]	Height of concrete plate
	h_p		[mm]	Height of pre-fabricated element
	$A_{s,\,lat} / A_c$	0.5	%	Degree of lateral reinforcement
	A_{s1} (field)	100	cm²/m	Upper reinforcement area of field section
	z_{s1} (field)	30	[mm]	Distance of upper reinforcement from the top of the concrete plate
	A_{s2} (field)	100	cm²/m	Lower reinforcement area of field section
	z_{s2} (field)	230	[mm]	Distance of lower reinforcement from the top of the concrete plate
	A_{s1} (support)	100	cm²/m	Upper reinforcement area of support section
	z_{s1} (support)	30	[mm]	Distance of upper reinforcement from the top of the concrete plate
	A_{s2} (support)	100	cm²/m	Lower reinforcement area of support section
	z_{s2} (support)	230	[mm]	Distance of lower reinforcement from the top of the concrete plate
	Concrete class	C 40 / 50		
	Reinforcement strength	BSt 500 S		

Girder	Kind of fabrication	Plate section		Rolled section or plate section
	Kind of profile	-		Specify the rolled profile type
	Steel girder strength	-		Specify the steel strength of rolled profile
	Plate section	non-hybrid		Enter kind of plate layout
	Steel girder strength	S 355		Steel strength of the beam
	Reduction of steel strength	√		Reduction of steel strength according to the thickness of the plate
	s	12	[mm]	Web thickness
	b_1 (field)	420	[mm]	Upper flange width of field section
	b_2 (field)	500	[mm]	Lower flange width of field section
	t_1 (field)	26	[mm]	Upper flange thickness of field section
	t_2 (field)	30	[mm]	Lower flange thickness of field section
	b_1 (support)	420	[mm]	Upper flange width of support section
	b_2 (support)	500	[mm]	Lower flange width of support section
	t_1 (support)	26	[mm]	Upper flange thickness of support section
	t_2 (support)	30	[mm]	Lower flange thickness of support section

Connectors	Studs	active		Select studs as shear connectors
	S 235			Use S 235 for studs
	S 460	√		Use S 460 for studs
	CUSTOM			Use a customised material for studs
	f_u	-	[N/mm²]	Ultimate strain
	f_y	-	[N/mm²]	Yield strength
	Diameter d = 16 mm			Diameter of studs
	Diameter d = 19 mm			Diameter of studs
	Diameter d = 22 mm	√		Diameter of studs
	Diameter d = 25 mm			Diameter of studs
	height h =	125	[mm]	Height of studs
	alternative shear connectors	select		Use an alternative shear connector
	P_{Rd}		[kN/m]	Bearing resistance of the alternative shear connector

Loads	**Load cases**	**Casting**		**Select load case**
	$x_{[1]-[8]}$	0	[m]	Distance of point load 1 – 8 from left support
	$F_{[1]-[8]}$	0	[kN]	Magnitude of point load 1 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	2.90	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	16.20	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	2.90	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2]}$	6.60	[m]	Distance starting point of distributed load 2 from left support
	$q_{0\,[2]}$	2.90	[kN/m]	Magnitude of the distributed load 2 at starting point
	$x_{1\,[2]}$	9.60	[m]	Distance end point of distributed load 2 from left support
	$q_{1\,[2]}$	2.90	[kN/m]	Magnitude of distributed load 2 at end point
	$x_{0\,[3],[4]}$	0	[m]	Distance starting point of distributed load 3, 4 from left support
	$q_{0\,[3],[4]}$	0	[kN/m]	Magnitude of the distributed load 3, 4 at starting point
	$x_{1\,[3],[4]}$	0	[m]	Distance end point of distributed load 3, 4 from left support
	$q_{1\,[3],[4]}$	0	[kN/m]	Magnitude of distributed load 3, 4 at end point

Loads	**Load cases**	**Permanent**		**Select load case**
	$x_{[1]-[8]}$	0	[m]	Distance of point load 1 - 8 from left support
	$F_{[1]-[8]}$	0	[kN]	Magnitude of point load 1 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	30.80	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	16.20	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	30.80	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2],[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\,[2],[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\,[2],[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\,[2],[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	**Load cases**	**Traffic [max]**		**Select load case**
	$x_{[1]}$	7.10	[m]	Distance of point load 1 from left support
	$F_{[1]}$	315	[kN]	Magnitude of point load 1
	$x_{[2]}$	9.10	[m]	Distance of point load 2 from left support
	$F_{[2]}$	315	[kN]	Magnitude of point load 2
	$x_{[3]-[8]}$	0	[m]	Distance of point load 3 – 8 from left support
	$F_{[3]-[8]}$	0	[kN]	Magnitude of point load 3 - 8
	$x_{0\,[1]}$	0	[m]	Distance starting point of distributed load 1 from left support
	$q_{0\,[1]}$	1.20	[kN/m]	Magnitude of the distributed load 1 at starting point
	$x_{1\,[1]}$	16.20	[m]	Distance end point of distributed load 1 from left support
	$q_{1\,[1]}$	1.20	[kN/m]	Magnitude of distributed load 1 at end point
	$x_{0\,[2],\,[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\,[2],\,[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\,[2],\,[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\,[2],\,[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	Load cases	Traffic [min]		Select load case
	$x_{[1]-[8]}$	0	[m]	Distance of point load 3 – 8 from left support
	$F_{[1]-[8]}$	0	[kN]	Magnitude of point load 3 - 8
	$x_{0\ [1],[4]}$	0	[m]	Distance starting point of distributed load 2, 3& 4 from left support
	$q_{0\ [1],[4]}$	0	[kN/m]	Magnitude of the distributed load 2, 3& 4 at starting point
	$x_{1\ [1],[4]}$	0	[m]	Distance end point of distributed load 2, 3& 4 from left support
	$q_{1\ [1],[4]}$	0	[kN/m]	Magnitude of distributed load 2, 3 & 4 at end point

Loads	Load cases	Breaking		Select load case
	$x_{[1]-[8]}$	0	[m]	Distance of moment 2 – 4 from left support
	$F_{[1]-[8]}$	0	[kNm]	Magnitude of moment 2 - 4
	H_x	900	[kN]	Horizontal load due to breaking or acceleration

Loads	Load cases	Temperature		Select load case
	dT [$T_o > T_u$]	10	[K]	Top of concrete warmer than bottom concrete
	dT [$T_u > T_o$]	7	[K]	Top of concrete colder than bottom concrete

Settings	Constr. Sit. 1	√		Take construction situation 1 into account
	Constr. Sit. 2	√		Take construction situation 2 into account
	Max deflection	√		Perform deflection check
	L /	200		Deflection limit
	Moment distribution	√		Take moment distribution into account
	Creep	√		Take creep into account
	Shrinkage	√		Take shrinkage into account
	Safety factors	Germany		Specify set of combination factors for various countries

Combination factors (ULS)	ψ (Casting)	1.0		Combination factor for casting loads (ULS)
	ψ (Traffic TS)	1.0		Combination factor for single traffic loads (ULS)
	ψ (Traffic UDL)	0.4		Combination factor for uniformly distributed traffic loads (ULS)
	ψ (Breaking)	0.7		Combination factor for breaking loads (ULS)
	ψ (Temperature)	0.6		Combination factor for temperature loads (ULS)

Combination factors (SLS)	ψ (Casting)	1.0		Combination factor for casting loads (SLS)
	ψ (Traffic TS)	1.0		Combination factor for single traffic loads (SLS)
	ψ (Traffic UDL)	0.4		Combination factor for uniformly distributed traffic loads (SLS)
	ψ (Breaking)	0.7		Combination factor for breaking loads (SLS)
	ψ (Temperature)	0.6		Combination factor for temperature loads (SLS)

5.2.7.2 Partial safety factors [NAD]

Table 5-8: Partial safety factors, example 2

μ_g	μ_q	μ_{M0}	μ_{M1}	ν_{vec}	μ_c	μ_s	μ_v
1.35	1.50	1.10	1.10	0.80	1.50	1.15	1.25

5.2.7.3 Cross Section

5.2.7.3.1 Cross Section

	Section	Units	Value
Steel beam	I_{yy}	[cm^4]	294703
	$W_{c,o}$	[cm^3]	0
	$W_{c,u}$	[cm^3]	0
	$W_{st,o}$	[cm^3]	- 7784
	$W_{st,u}$	[cm^3]	9778
Fully composite slab			
Short term	I_{yy}	[cm^4]	1219257
	$W_{0,c,o}$	[cm^3]	-376017
	$W_{0,c,u}$	[cm^3]	1117786
	$W_{0,st,o}$	[cm^3]	186298
	$W_{0,st,u}$	[cm^3]	16356
Long term	I_{yy}	[cm^4]	1070452
	$W_{b,c,o}$	[cm^3]	-1075968
	$W_{b,c,u}$	[cm^3]	-9888418
	$W_{b,st,o}$	[cm^3]	494769
	$W_{b,st,u}$	[cm^3]	15257
Long term	I_{yy}	[cm^4]	1129775
	$W_{s,c,o}$	[cm^3]	-611655
	$W_{s,c,u}$	[cm^3]	3928389
	$W_{s,st,o}$	[cm^3]	277575
	$W_{s,st,u}$	[cm^3]	15676

5.2.8 Design

The effective width of the concrete plate $b_{eff} = 3.83$ [m]

The beam is rated into class 2 therefore the checks can be carried out plastically.

5.2.8.1 Bearing Resistance

Steel beam:	$V_{sd} = 1395$	[kN]
	$M_{sd} = 2993$	[kNm]

The beam has to be restrained against torsional buckling during construction.

Fully Composite beam [t = ∞d]:	$V_{sd} = 1395$	[kN]
	$M_{sd} = 6216$	[kNm]

5.2.8.2 Further Design Checks

The reinforcement diameter has to be smaller than 44 [mm] or the maximal distance between the reinforcement bars has to be smaller than 25 [cm] to satisfy the crack width design checks of the Eurocode.

The shear connectors have a design resistance of $P_{rd} = 133$ [kN].

The following criteria for the studs have to be satisfied:

- average welded bead diameter:	d_w	> 1.25 d	= 27.50	[mm]
- average welded bead height:	h_{wm}	> 0.2 d	= 4.40	[mm]
- minimum welded bead height:	$h_{w,min}$	> 0.15 d	= 3.30	[mm]
- minimum distance in direction of shear:	e_{min}	> 5 d	= 110.00	[mm]
- minimum distance vertical to direction of shear:	e_{min}	> 2.5 d	= 55.00	[mm]

If studs are not placed directly over the web, the diameter of the studs should be less than 30 [mm]

The following number of shear connectors have to be applied:

- 81 between support 1 and M_{max}
- 147 between M_{max} and support 2

The deflection checks are satisfied for all spans.

ANNEX A: SOFTWARE CBD

It is possible to download the Software CBD (Composite Bridge Design) under the following homepage:

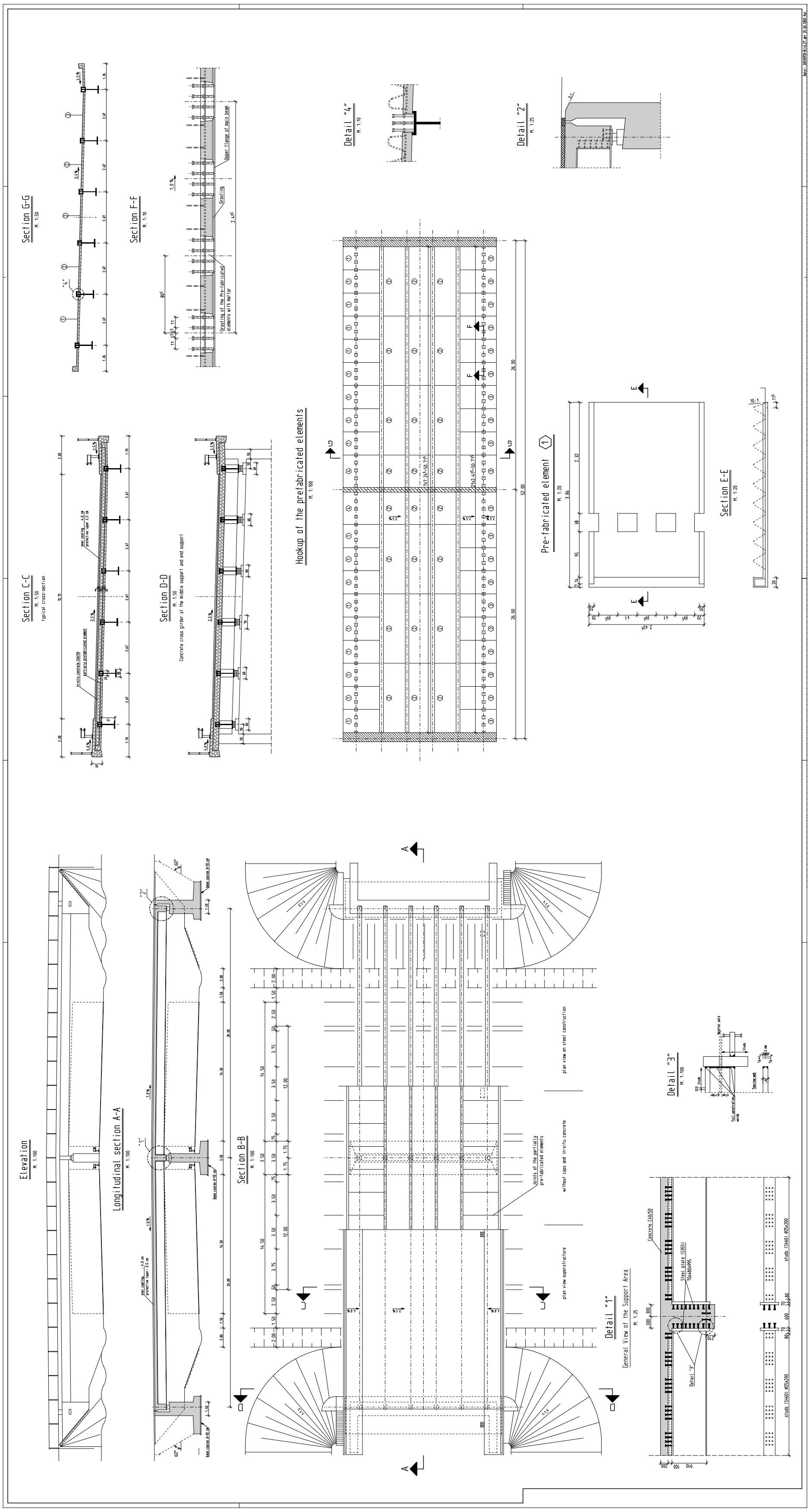
http://www.stb.rwth-aachen.de/Projekte/ECSC/CompBridges/index.html

ANNEX B: TENDER DRAWINGS

It is possible to download the Tender Drawings in *DIN A3* under the following homepage:

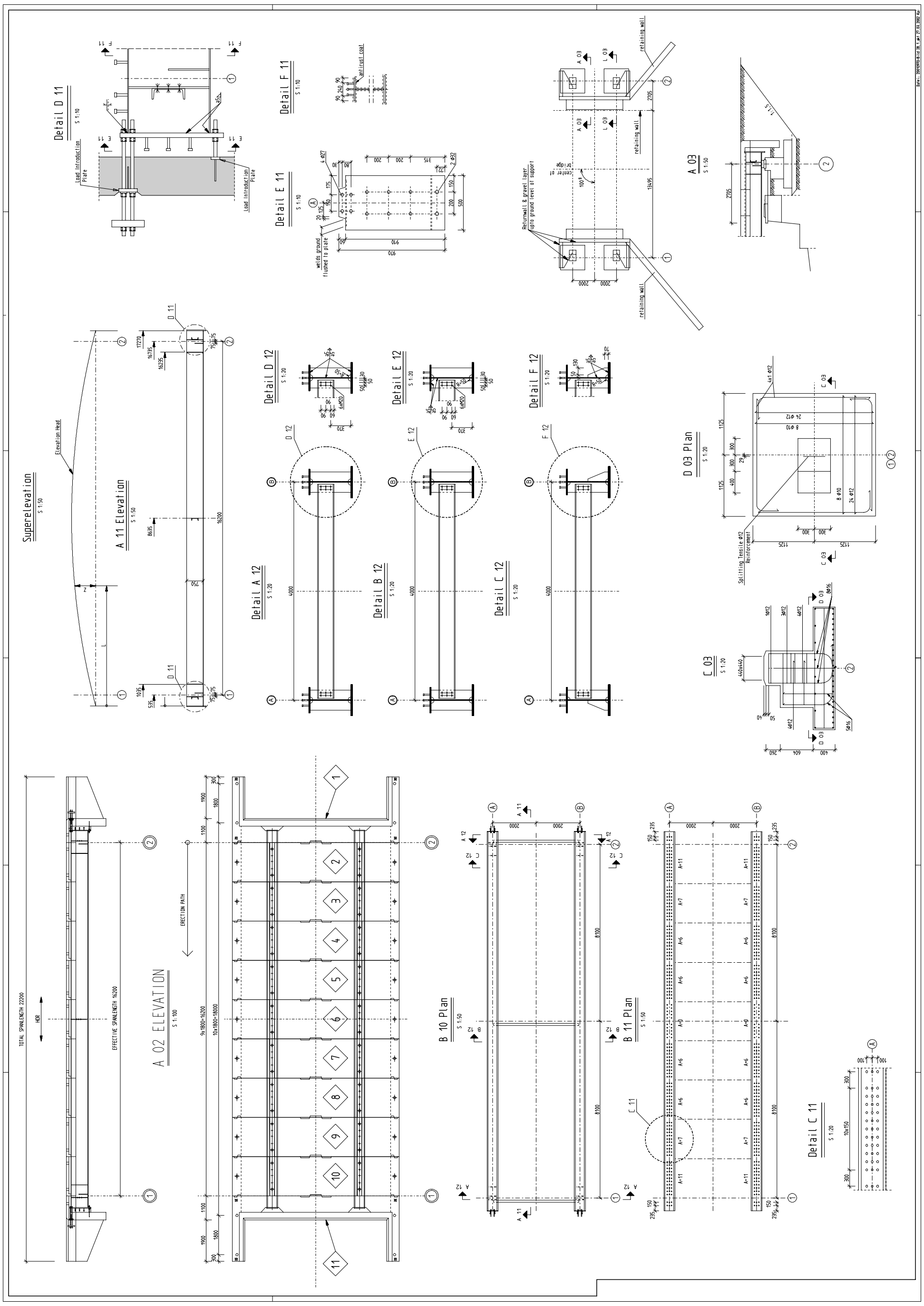
http://www.stb.rwth-aachen.de/Projekte/ECSC/CompBridges/index.html

B.1 Two-span bridge using partially prefabricated elements



B.2 Simple bridge using fully prefabricated elements

B.2.1 Simple bridge using fully prefabricated elements (Drawing 1)



B.2.2 Simple bridge using fully prefabricated elements (Drawing 2)