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Steel – Concrete Composite Bridges
Sustainable Design Guide

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Steel – Concrete Composite Bridges

Sustainable Design Guide

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Contents

Foreword

Steel-concrete composite bridges, both road and rail, have experienced a veritable boom in France since the late 1970s. Whilst highly competitive, primarily in the area of medium span structures, composite bridges have consistently extended their range of usage through development specifically towards large spans, the design of which has progressed ever further away from prestressed box girder and steel structures.

This guide reviews in detail the design and construction of the most common composite road structures, namely girder and box girder bridges. It is intended not only for Clients and Owners, but also design, construction and methods engineers.

The present guide therefore replaces and supersedes the Sétra guide entitled "Ponts mixtes acier-béton bipoutres / Guide de conception" [twin girder steel-concrete composite bridges / design guide], published in October 1985, updating of which was necessary in relation to specific points (new design and material standards, wide structures) as well as complementing of major areas, such as box girder deck design and construction methods. This new guide also replaces and supersedes Sétra technical report No. 8, entitled "Montage des ponts métalliques" [steel bridge construction], published in 1973.

The guide comprises seven sections:

- Section 1 introduces the road composite structures covered by the guide and provides a succinct review of relevant production over the last years; it recalls the most outstanding recent bridges and positions composite bridges with respect to sustainable development criteria;
- Sections 2 and 3 include first a general, then a much more detailed, description of standard twin girder and box girder composite structures;
- Sections 4 and 5 provide more technological descriptions of structural steelwork transportation-installation and concrete slab construction respectively;
- Section 6 describes the precautionary measures to be integrated at design stage in favour of structural maintenance and durability;
- Section 7 provides recommendations on both preparing French public tender DCEs [contractor consultation packages] for standard composite structures and the content of their text documents.

The guide also includes three appendices: the first lists the main composite bridges built in France between 1995 and 2005, the second contains a bibliography and the third a glossary of the principal composite bridge construction terms.

This document is the fruit of extensive group work and represents a powerful illustration of the know-how possessed by French engineers and builders.

1 - Introduction

This section details the purpose of this guide, its positioning with respect to other relevant Sétra documents and structural Eurocodes. It then provides information on the area of composite bridge usage and the place of these structures in French road bridge construction. Finally, it compares French practices with those of other European countries.

1.1 - Purpose and background of this guide

1.1.1 -Purpose of this guide

This guide deals with steel-concrete composite road bridges, i.e. bridges whose decks are composed of a concrete slab connected to a twin girder (figure 1.1), a multi-girder or a box girder (figure 1.2) steel frame. It therefore covers neither orthotropic deck bridges nor composite rail bridges nor filler beam, lateral beam or cable (bowstring and stayed) road bridges.

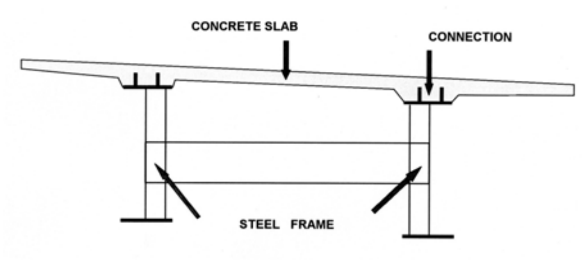

Figure 1.1. Twin girder composite

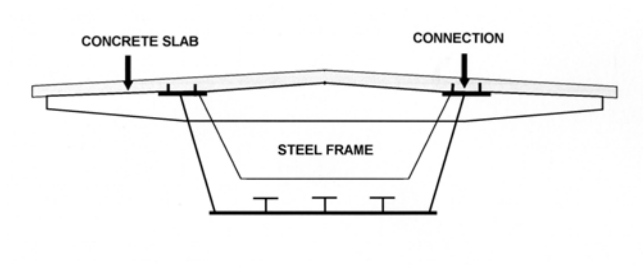

Figure 1.2. Box girder composite

This guide replaces and supercedes the Sétra guide entitled "Ponts mixtes acier-béton bipoutres / Guide de conception" [twin girder steel-concrete composite bridges / Design guide], published in October 1985 as well as Sétra technical report No. 8, entitled "Montage des ponts métalliques" [Steel bridge construction], published in 1973.

Compared with the above documents, the content of the present guide extends to composite box girders and a much more detailed description of steelwork installation, slab construction and tender package preparation. On the other hand, it contains very few design calculation data because these are now featured in the Sétra Eurocode 3 and 4 Application Guide published in 2007.Prise en compte des Eurocodes

1.1.2 -Eurocode consideration

All measures described in this guide, especially in Section 3 "Detailed Design" and Section 7 "Recommendations for DCE Preparation", comply fully with Standards NF EN 1990 to 1994, i.e. with Eurocodes 0 to 4, which are applicable to composite decks.Autres guides du Sétra relatifs aux ouvrages mixtes

1.1.3 -Other Sétra guides to composite structures

This technical guide should not be considered in isolation. It effectively complements previous guides published by Sétra covering both steel and composite structures, such as the Eurocode 3 and 4 application guide, the guide entitled "Travaux de construction des ponts en acier – Guide du maître d'œuvre" [Guide to steel bridge construction – A Client’s guide] or MEMOAR (Mémento pour la Mise en oeuvre sur Ouvrages d'Art) datasheets XVI to XVIII.

The Sétra/SNCF/CTICM guide entitled "Ponts métalliques et mixtes – Résistance à la fatigue" [Steel and composite bridges – Fatigue resistance] and the Sétra/LCPC guide entitled "Ponts mixtes – Recommandations pour maîtriser la fissuration des dalles" [Composite bridges – Recommendations for controlling slab cracking] are earlier and their design sections are now outdated. However, they do provide general information that is topical.

1.1.4 -Terminology and illustrations

For legibility, most figures in this guide are partial illustrations and should not be considered to scale.

1.2 - Reminder of composite bridge structural behaviour

As stated at the start of this section, composite bridge structures comprise a reinforced or prestressed concrete slab and a steel frame linked by connectors. The latter components prevent any relative movement between the bottom of the slab and the top of the steel frame and they govern the deformation identity of both slab bottom fibres and steel frame top fibres.

Under these conditions, composite bridge deck cross sections are usually subjected to longitudinal bending and according to the stress-deformation diagrams shown below (Figure 1.3) at serviceability limit states (SLS).

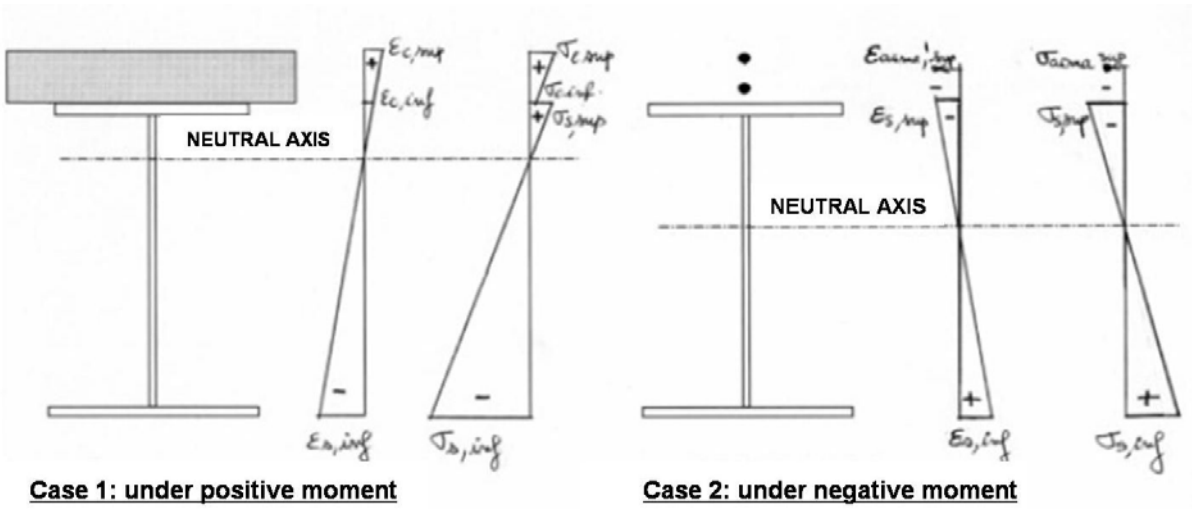

Figure 1.3. Stress-deformation in composite cross sections at SLS

The centre of gravity of a composite cross section is relatively high, thus the concrete slab is in compression and the steelwork is predominantly in tension in positive bending moment areas (Case 1). Structural operation is therefore highly economic because each material is effectively stressed in keeping with its inherent strength characteristics.

The slab concrete is subjected to high tension and is therefore considered to offer no resistance in negative bending moment areas (Case 2). Loads are then only taken up by the slab longitudinal passive reinforcement and the steel frame, whose bottom part is highly compressed and thus subjected to instability risks. This results in less efficient usage of the structural potential of the materials composing the cross section and usually requires the frame sectional areas to be increased.

1.3 - Area of composite bridge usage in france

1.3.1 -Range of usage

The table below shows the economic range of usage of girder or box girder composite bridges and for potentially competing structures based on a maximum span criterion (black bars represent the most common areas of usage).

Type	Span						
	35	70	90	120	150	200	300
Progressively cast prestressed concrete							
Prestressed concrete bridges built by the							
Pushed prestressed concrete bridges							
Girder composite bridges							
Box girder composite bridges							
Orthotropic box girder bridges							

Table 1.1. Range of usage of composite bridges and competing structures

In France, the range of usage of twin girder composite bridges is therefore 30 to 130 m, embracing a common range of 40 to 90 m, and that of box girder composite bridges is 50 to 150 m, embracing a common range of 70 to 120 m.

1.3.2 -Steel-concrete competition

As we can see from the table above, girder or box girder composite bridges are used over essentially the same maximum span range as prestressed concrete box segment/deck bridges. In France, these structures are built either by successive deck segment cantilevering, pushing or in-situ concreting operations.

A number of invitations to tender featuring two basic solutions (one a composite, the other a pushed concrete bridge) have revealed that, in the 40 to 65 m maximum span range, a pushed concrete bridge is usually more expensive than a composite bridge integrating similar spans, except when the obstruction to be crossed is very long, as in the Meaux and Bresle viaducts.

In the 60 to 110 m span range, prestressed concrete bridges built by successive deck segment cantilevering are also almost invariably more expensive than composite structures, except when the composite bridge steelwork is difficult to install (a tunnel very near the projected launching area, a complex road network, need for a highly variable deck height, etc.).

In conclusion, composite solutions are currently highly competitive over a very wide range of obstructions to be crossed and are only seriously challenged by concrete solutions for structures:

- in which at least one span exceeds 120 m,
- in which the steel frame is very difficult to install,
- doubling up existing concrete bridges,
- of very great length, for which precasting offers renewed competitivity.

Given the lifespan of this guide, it should be stated that the range of composite bridge competitivity that we have detailed above is closely dependent on the steel selling price applied by steel producers and can therefore vary depending on this price. This range of competitivity can also be invalid under certain local conditions (overseas territories, etc.) or when multiple composite bridges need to be built during the same time period.

1.4 - Statistics for composite bridge construction

1.4.1 -Main composite road bridges built from 1995 to 2005

Appendix A1 of this guide includes a list of the most important composite road bridges built between 1995 and 2005 along with their main characteristics. We have restricted this list to twin girder composite structures longer than 200 m and composite box girder structures longer than 100 m to ensure that this list does not provide data already given by those regularly published in OTUA "Bulletins Ponts Métalliques" [steel bridge reports published by the Office Technique de l'Utilisation de l'Acier]. Moreover, we have detailed the slab construction method for each bridge.

1.4.2 -Annual statistics

For each year between 1995 and 2004, the table below provides the total number of composite road bridges built in France as well as the corresponding total deck area and steelwork tonnage. This table has been drawn up based on production figures regularly published in OTUA "Bulletins Ponts Métalliques" [see above] and differentiates box girder composite and girder composite structures.

Quantities	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004
No. girder composite bridges	47	31	25	24	23	23	22	15	30	18
Girder bridge deck area	90 850	58 660	69 760	41 560	43 540	97 080	53 665	58 975	43 590	56 960
Girder bridge steelwork tonnage	17 620	13 750	23 790	10 205	10 520	21 255	15 990	21 035	7 640	15 500
No. box girder composite bridges	2	4	4	4	3	2	3	1	2	3
Box girder deck area	9 210	12 110	3 390	15 340	4 265	1 125	2 885	755	1 580	14 970
Box girder steelwork tonnage	2 330	3 840	865	5 945	1 230	290	730	190	820	3 850
Total number	**49**	**35**	**29**	**28**	**26**	**25**	**25**	**16**	**32**	**21**
Total deck area	**100 060**	**70 760**	**73 150**	**56 900**	**47 805**	**98 205**	**56 550**	**59 730**	**45 170**	**71 930**
Total tonnage	**19 950**	**17 600**	**24 660**	**16 150**	**11 750**	**21 545**	**16 720**	**21 225**	**8 460**	**19 350**

Table 1.2. Statistics for composite structures built in France between 1995 and 2004

This very brief summary indicates that, on average, approximately 30 composite road bridges (including only a few box girder structures), representing a total deck area of the order of 70,000 m² and requiring assembly/erection of approximately 18,000 tonnes of steelwork, were built each year.

1.4.3 -Statistics for bridge type and construction method

Twin girder structures represent a very major portion (75 to 85% depending on the parameter considered: number of bridges, deck area, tonnage) of composite road bridges built. Within this family, twin girder cross-beam bridges constitute approximately 70% of deck areas built and twin girder directly supporting cross-beam bridges make up the remaining 30%.

Box girder bridges only represent, on average, approximately 10% of composite road bridges built and these are mainly small structures with the exception of the second bridge over the River Rhône at Valence and the Verrières viaduct.

If we now consider construction methods, bridge steelwork is principally installed by deck launching (Section 4), except for smaller structures for which steel frames are often crane-lifted. Mobile formwork-based construction (Section 5) is by far the most frequently adopted slab construction method.

1.5 - A few significant structures

We mention below a number of the most significant structures built in France during the decade prior to writing this guide.

Longest

The three longest composite road bridges are located on French concessionary motorways. These are the Risle viaduct on the A28, the Dordogne viaduct on the A20 and the Vézère viaduct on the A89. Their total lengths are 1320, 1070 and 1002 m respectively.

Widest

Two road bridges are exceptionally wide:

- the Charles de Gaulle bridge in Paris, with a unique deck width of 34.90 m including an 18 m wide road width,
- the Saint-Denis canal viaduct in Saint-Denis, near Paris, on the A86 motorway, with two four girder decks forming a combined width of nearly 45 m.

Longest spans

The record span for a French cableless composite bridge is that of the Verrières viaduct with its 144 m central span. This longest span is followed by:

- Jassans bridge in Ain (130 m),
- downstream Centron viaduct in Savoie (125 m),
- Triel bridge in Yvelines (124 m).

Largest deck areas

Three twin girder cross-beam structures exceed the symbolic 20,000 m² mark. These are the River Charente viaduct on the A837 motorway, the Vézère viaduct on the A89 motorway and the River Dordogne viaduct on the A20 motorway.

1.6 - Composite bridges in other european countries

The great majority of European countries now build composite bridges, but it should be noted that the proportion of composite structures is smaller in these countries than in France.

In some countries, twin girder composite bridges are very rare because local engineers consider these structures to be insufficiently safe should one of the two girders be destroyed. In these countries, composite bridges are therefore either 4-girder or box girder structures.

Other major differences between European countries concern specific design issues. Thus, in France, heavy steel plate is frequently used (up to 150 mm for S355 grade steel), but this is not the case for some countries, in which plate thickness is often limited to 80 mm.

Another important difference involves steelwork assembly. In France, common composite bridges are fully welded for reasons of durability and aesthetics, whilst in some countries on-site assembly is implemented using high-strength (HS) bolting.

A final difference within Europe concerns the use of weathering steels. This practice is extremely rare in France: the colour of the patina, which is very similar to rust, is not liked and it is felt that the patina makes it difficult to detect fatigue cracks. This practice is more widespread in other countries, the above arguments not being considered unacceptable.

The reader interested in current composite bridge construction practices in other European countries will find Part II of the COMBRI design guide helpful. This guide was published by the CTICM [Centre Technique Industriel de la Construction Métallique, the French steelwork technical centre] in November 2008; it compares Belgian, Swedish, German, Spanish and French practices.

1.7 - Composite bridges and sustainable development

It is of interest to consider the position of composite bridges with respect to sustainable development criteria before considering more closely the design and construction of these structures.

Preamble

Sustainable development is the construction of durable, robust structures, which are sparing in material and energy terms and guarantee reduced environmental and human health impacts at an acceptable economic cost.

Optimisation of resources

In general, modern composite bridges are structures in which material consumption is effectively optimised because:

- supporting I-girders offer high structural efficiency,
- use of different thickness steel plate enables implementation of only strictly required minimum thickness throughout structure,
- deck lightness decreases size of supports and especially foundations,
- when longitudinal profile is not imposed, higher slenderness ratio allows lower longitudinal profile and thus lower approach embankments.

It should also be noted that steel plate waste collected in the fabrication shop can be reused because steel is a readily recyclable material.

Ecobalance

At the time of editing this guide, it is still difficult to draw up an accurate balance for emissions in CO_2 equivalent, energies and water consumed and hazardous products to be controlled on site or at end of life. Assumptions to be adopted in fact remain too inaccurate, especially in relation to steel production.

Human health

Composite bridge construction embraces phases, which may generate accidents, such as steel frame installation or slab construction. These phases, featuring movement of often very heavy parts, are usually well controlled but nevertheless call for both the contractor’s and the engineer’s extreme care. For a box girder, confined area welding- and painting-related risks must also be prevented, especially if it is closed.

For structures spanning busy roads, steelwork installation and slab constrruction are generally performed subject to minimum disturbance of the routes crossed, whilst the risk of falling objects must of course be contained.

The small volumes of concrete cast in situ mean that disturbance of local residents (due to truck mixer traffic, concrete vibration) is also curtailed.

Vulnerability to impacts

Modern composite bridge supports can be designed and built to resist truck impacts of unexceptional intensity. However, composite structure decks are more sensitive than those of concrete structures. This is particularly the case for the girder decks considered in this guide, whose bottom flanges (the most frequently damaged parts) are intrinsically weak. Overriding importance must therefore accorded to respecting sufficient clearance and avoiding a conventional girder composite structure at a location where frequent deck impacts can be foreseen.

Earthquake vulnerability

Modern composite bridges built in compliance with seismic codes are highly resistant to earthquakes. Their systematically continuous design makes them easy to maintain at support level and prevents any risk of support unseating. Moreover, composite bridges are lighter than equivalent span and width concrete bridges; this reduces the loads sustained by their supports in the event of an earthquake.

Vulnerability to fire

Instances of structures damaged by fire are extremely rare because the probability of a fire under a bridge is very low and temperatures reached by the steel are only dangerous if the deck is just a few metres above the flames. Having said this, a composite bridge deck is rather more sensitive to a major fire than a concrete bridge deck because steel engineering properties are more sensitive to a high rise in temperature than concrete properties.

Durability

Achieving a quality structure generally requires taking into account all the European normative system (Eurocodes, standards, technical certifications) and setting up quality assurance process. The Client or Owner must therefore ensure that:

- structural design and construction are performed by qualified, experienced personnel,
- construction materials and products are used as specified in Eurocodes and standards or by manufacturers,
- the quality of processes implemented in design offices, fabrication shops and on site is ensured,
- after completion the structure receives adequate maintenance.

In addition to these general requirements, the Client or Owner of a projected composite bridge must ensure that:

- the waterproofing course is thick and properly laid,
- slab cracking is controlled by suitable reinforcement and construction kinematics,
- steelwork corrosion protection is ensured in compliance with conditions and components (ACQPA-certified paint systems) recommended in CCTG [French general technical specifications for government contracts] fascicule 56,
- fatigue phenomena are properly considered during both construction and the life of the structure (especially setting up of specific monitoring if traffic and traffic loads increase).

Maintenance

A composite bridge usually only requires regular renewal of its corrosion protection and expansion joints, and sometimes renovation of its waterproofing course.

Repainting operations are basically only necessary every 20 to 30 years. These represent fairly major operations requiring extensive precautions, environmental care in particular demanding prevention of any paint discharge into nature. Painting must be performed as described under the “Durability” heading in this section (see above) to curtail their frequency to a minimum.

The slab requires no maintenance if its concrete has been correctly specified and reinforcement covers have been adapted to Eurocode exposure classes. Measures can be taken to replace it at mid-life, if its construction quality is doubtful or in an aggressive environment.

A defective waterproofing course must be very rapidly renewed, as in all structures.

Adaptability

In general, civil engineering structures are very difficult to upgrade, when new needs arise, and composite bridges are no exception to this rule. They are often a little more flexible than other types of structure insofar as their steel framework can sometimes be upgraded: strengthening of girder bottom flanges by adding plates, extension of tranverse members, strengthening of certain welds, etc.

Demolition and recycling capacity

Civil engineering structures are usually difficult to demolish and recycle. Composite bridges are no exception to this rule, but they do offer certain advantage compared with other types of structure:

- their supports are smaller, so less tedious to demolish,
- deck steelwork can be pull back or cut up into “easily” moved sections and its steel can be recycled,
- their deck slabs are not thick and thus “easy” to demolish and remove.

Overall cost

Life cycle consideration allows the Client or Owner of a projected composite bridge to integrate into his appraisal not only the immediate structure construction cost, but also the deferred costs of later management actions, such as maintenance, demolition and possible repairs.

Unfortunately, whilst structure construction cost is fairly easy to estimate, structure management costs are much more difficult to evaluate: management overheads are often inaccurately known and vary, depending on the age of the bridge; when sickness strikes, its consequences may only become apparent after a decade!

At the time of drafting this guide, determining the overall cost of a civil engineering structure remains a difficult exercise, but current research should prompt rapid advances in our knowledge and development of this methodology.

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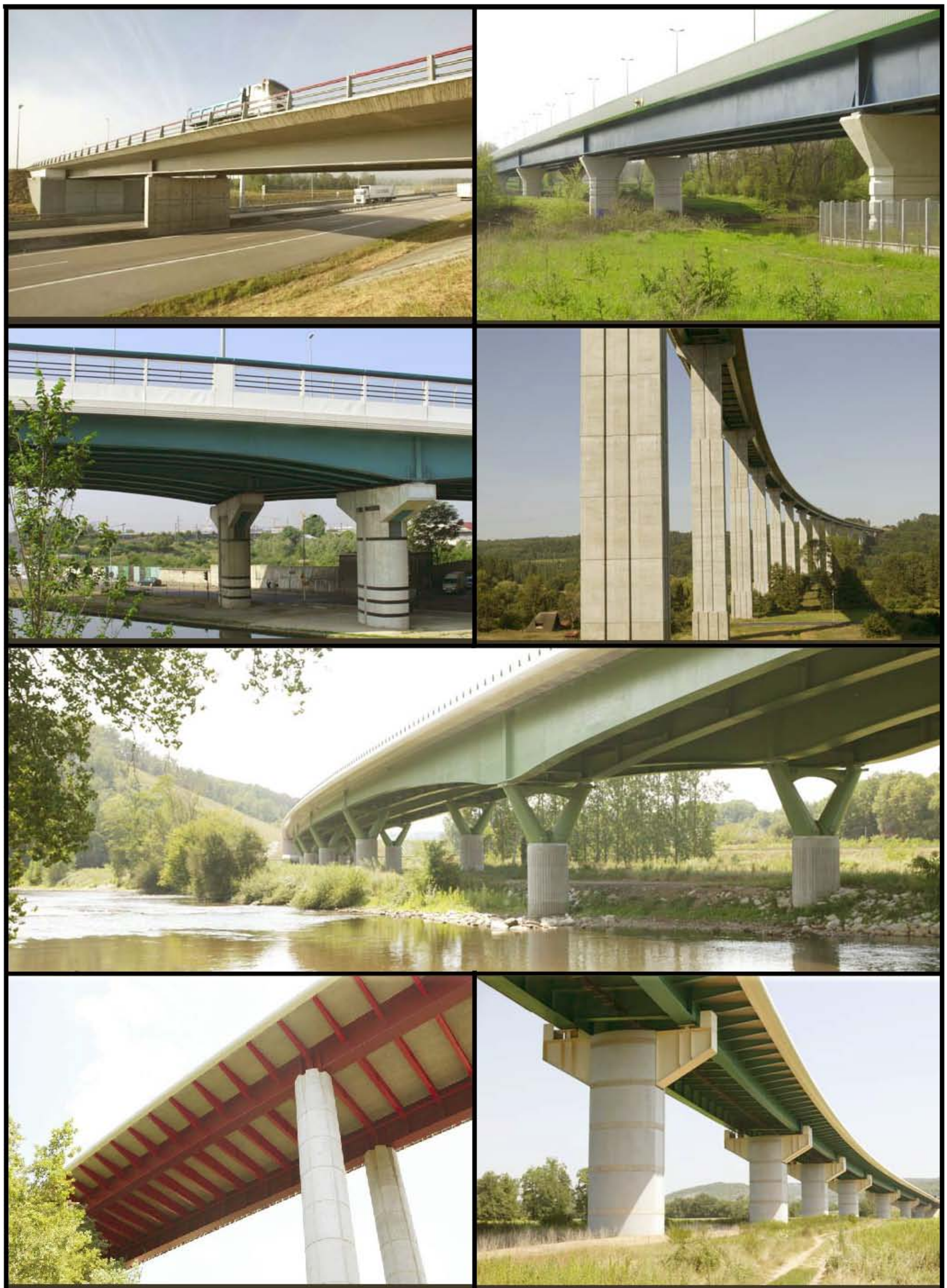
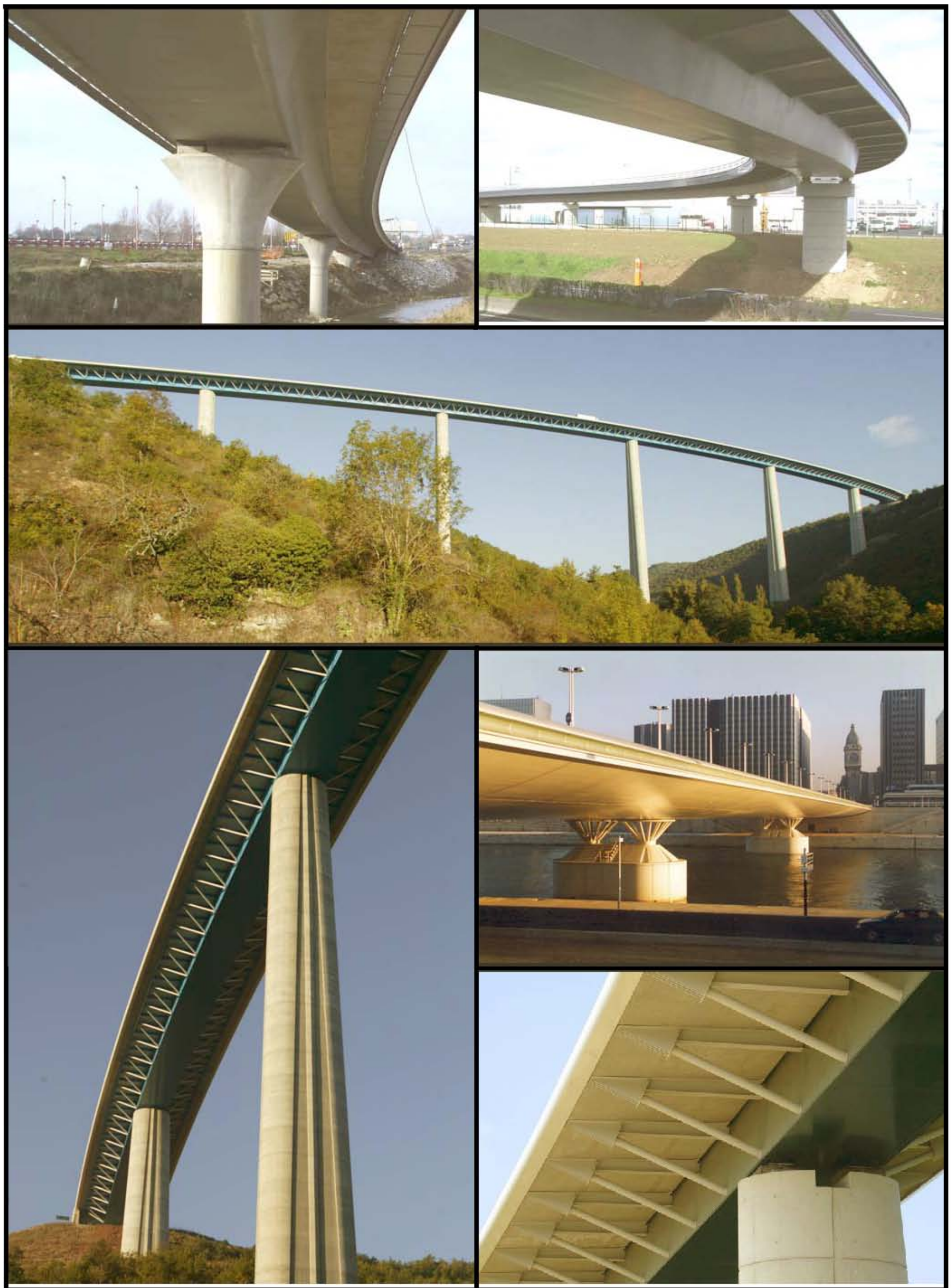


101: Twin girder composite overpass. 102: Triel viaduct in Yvelines. 103: Saint-Denis canal viaduct on A86. 104: Risle viaduct on A28. 105: Vézère viaduct on A89. 106: Planchette viaduct on A75. 107: Dordogne viaduct on A20.



111: Standard box girder composite at Toulouse. 112: Box girder and directly supporting cross-beam composite at Orly. 113 and 114: Verrières viaduct, a very wide propped cantilever box girder composite on A75. 115: Charles de Gaulle bridge in Paris.
116: Frocourt viaduct, a propped cantilever box girder composite

2 - General design of composite bridge

This section provides essential information for the general design of a conventional composite bridge and guidelines for an "Etude Préliminaire d'Ouvrage d'Art" (EPOA), the French preliminary structural design level. "Avant-Projet d'Ouvrage d'Art" (APOA) or "Projet d'Ouvrage d'Art" (POA), the French detailed structural design levels, are covered in Section 3 of the guide. Section 2 introduces successively design of girder composite structures, the most common box girder composite structures and finally a few special composite structures.

2.1 - Overall design of girder composite bridges

2.1.1 -General

Girder composite bridges are commonly used structures; they can be designed and built for a wide range of conditions: urban or rural environment, main span between 30 and 130 m, total length of a few tens of metres up to more than one kilometre, total width of 7 or 8 m up to approximately 20 m, highly economic standard or more sophisticated structure.

Tables A and B in Appendix 1 of this guide consolidate the main twin girder composite bridges built in France between 1995 and 2005. These structures represent approximately 90% of French composite road bridges, the balance being made up of box girder composite bridges.

2.1.2 -Transverse morphology

2.1.2.1 - Twin girder cross-beam bridges

The great majority of girder composite bridges are so-called twin girder cross-beam structures. Their deck is a concrete slab, usually simply reinforced, which is supported by a steel frame comprising two main girders connected to the deck slab and interlinked by secondary beams called cross-beams, which are at no point in contact with the slab (Figure 2.1).

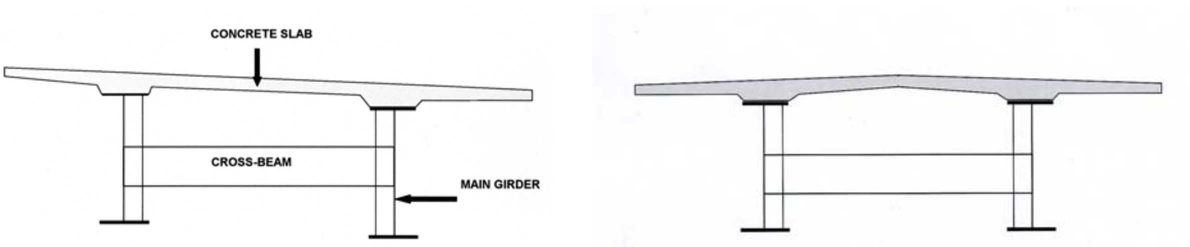


Figure 2.1. Twin girder cross-beam composite decks
(pavement unidirectional banking at left - bidirectionally banking at right)

Main girders

Composite bridge main girders are large structural steel members fabricated by shop welding into I-sections, except in the case of a few small span bridges in which they can be hot-rolled standard steel sections. In their longitudinal direction, flange width is generally constant, whilst flange thickness and web depth and thickness are variable. Connectors, usually studs, are fixed to the upper faces of the top flanges. These connectors inhibit bridge slab movement (sliding and uplift) with respect to the steel frame, thereby ensuring composite behaviour of the system.

When the supported road features symmetrical bidirectional banking, the two main girders are identical and positioned at the same elevation. When the supported road features unidirectional banking, the two main girders are identical but vertically offset by a height equal to the product of their centre-to-centre distance and the road banking.

Cross-beams

Steel frame secondary beams, called cross-beams, are at no point in contact with the concrete slab and usually comprise standard structural steel sections. Cross-beams at supports, which brace the main girders against horizontal loads (wind, earthquake) are generally deeper and fabricated by shop-welding. Cross-beams are welded to the main girders through T-sections, called posts, welded to the internal faces of the main girder webs and flanges.

Cross-beam centre-to-centre distance is less than or equal to 8 m; it is most often constant within a bridge span, but can vary from one span to another. In the future however, cross-beam centre-to-centre distance could be less near bridge piers than at mid-span due the severe lateral torsional buckling conditions imposed by Eurocodes 3 and 4.

Slab

The slab depth of a twin girder cross-beam composite bridge is constant in the longitudinal direction and most often variable in the transverse direction (usually between 24 and 40 cm). It is made of reinforced concrete, when its width does not exceed 15 or so metres, but can be transversely prestressed for greater widths (see below). Slab integrality with its supporting steel frame is ensured by connectors welded to the top flange of the two main girders. It is constructed after installing the steel frame, either by casting in situ or by assembling slab segments precast on site or at a casting yard (Section 5).

Special case of twin girder cross-beam composite bridges with a prestressed concrete slab

When the slab width exceeds 15 or so metres, its depth and therefore its weight can be reduced by building in transverse prestress. This is usually generated by 1T15S and 4T15S power cables arranged at centre-to-centre distances of 25 – 80 cm.

Implementation of this prestress requires extensive labour; twin girder composite brides with a prestressed slab have been replaced by twin girder directly supporting cross-beam composite bridges in recent years (see below).

2.1.2.2 - Twin girder directly supporting cross-beam composite bridges

The second major family of girder structures comprises twin girder directly supporting cross-beam composite bridges. In these structures, the steel frame is composed of two main girders interlinked by secondary beams called directly supporting cross-beams, which effectively support the slab and allow its depth to be reduced (Figure 2.2).

These bridges are more complex to build than twin girder cross-beam bridges and are generally used when the slab weight become excessive for the steel frame, i.e. when the deck width exceeds 13-14 m or when the maximum span exceeds approximately 90 m.

Sometimes, a directly supporting cross-beam bridge may be preferred to a cross-beam bridge for purely aesthetic reasons.

This second family, in which the main girders are similar to those of twin girder cross-beam structures, can be split into two sub-families depending on whether the directly supporting cross-beams extend as cantilevers or not.

Twin girder directly supporting cross-beam bridges with cantilevers

In this first sub-family, the length of the directly supporting cross-beams is essentially the same as that of the deck and they are in contact with the concrete slab right across its width.

In unidirectionally banked decks, the directly supporting cross-beams are inclined at the banking gradient. Their depth is constant between the main girders, but varies linearly in their cantilevered parts, which support the slab overhangs (Figure 2.2, left). In bidirectionally banked decks, the directly supporting cross-beams are horizontal and their depth varies linearly in their cantilevered parts. In the latter case, their depth usually varies linearly between the main girders with a maximum depth at the deck centre (Figure 2.2, right). In some cases, this cross-beam depth between main girders can be constant, which generates a need for concrete haunching or extra depth above each directly supporting cross-beam.

The centre-to-centre distance between directly supporting cross-beams cannot be varied, even slightly, without increasing slab construction complexity, so this distance must remain as constant as possible with a recommended value of approximately 4 m.

The reinforced concrete slab is of constant depth, usually 24 or 25 cm, in both longitudinal and transverse directions. It is connected to both the main girders and the directly supporting cross-beams.

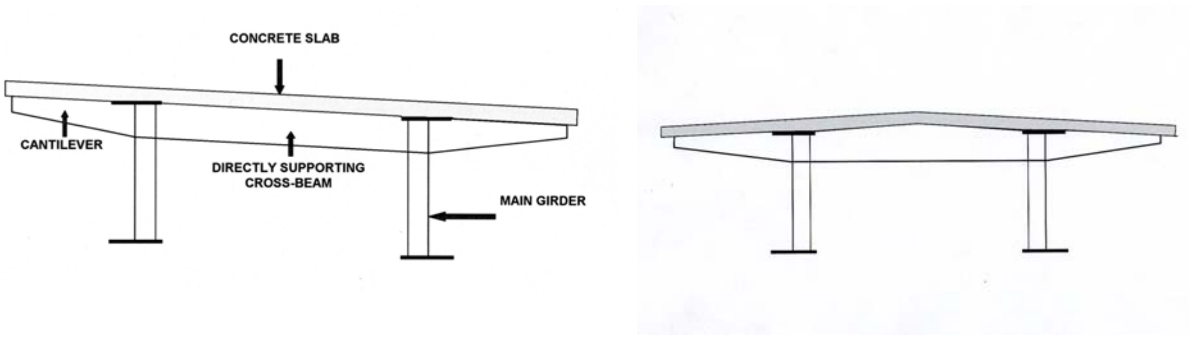


Figure 2.2. Twin girder directly supporting cross-beam composite bridge with cantilevers (pavement unidirectional banking at left - bidirectional banking at right)

Centron downstream viaduct on France's RN90 national road, the River Durance downstream bridge at Avignon and the River Lot bridge on the A20 motorway are good examples of twin girder directly supporting cross-beam composite bridges with cantilevers.

Twin girder directly supporting cross-beam bridges without cantilevers

The second sub-family of girder bridges with directly supporting cross-beams comprises directly supporting cross-beam structures without cantilevers. In this case, the steel frame is composed of two main girders interconnected by directly supporting cross-beams without cantilevers, which therefore only support the slab between the two main girders (Figure 2.3).

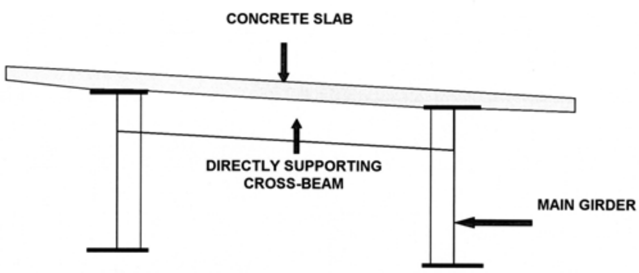


Figure 2.3. Twin girder directly supporting cross-beam bridge without cantilevers (pavement unidirectional banking)

These bridges share many characteristics with those of the previous sub-family, in particular a constant centre-to-centre distance of approximately 4 m between directly supporting cross-beams. However, the main girder centre-to-centre distance will be significantly different for a bridge of identical width. The slab overhanging or cantilever parts are in fact unsupported by the directly supporting cross-beams, so their width must be restricted to a fairly low value around 2 m, which leads to a large centre-to-centre distance between main girders. As an example, a 15 m wide deck with directly supporting cross-beams and cantilevers will require a main girder centre-to-centre distance of nearly 8 m and 3.50 m overhangs, whilst a similar width deck with directly supporting cross-beams and no cantilevers will require a main girder centre-to-centre distance of nearly 11 m and approximately 2 m overhangs.

Thus, twin girder directly supporting cross-beam bridges without cantilevers are chosen when a large main girder centre-to-centre distance is required for a given deck width; this is the case for both decks featuring large horizontal curvature and decks near the ground to be supported on piers featuring two independent shafts.

In recent years, this second sub-family has given rise to very few structures; solutions integrating directly supporting cross-beams with cantilevers being frequently preferred for aesthetic reasons; The Saulières viaduct on the Brive bypass and the River Sauldre viaduct on the A85 motorway are probably the largest of these bridges without cantilevers in France.

2.1.2.3 - Other girder composite bridges

Multi-girder composite bridges

Composite bridges can be designed to integrate more than two main girders (Figure 2.4). In the absence of specific constraints, these structures featuring secondary members embodied by cross-beams are more expensive to build than twin girder structures; they are therefore reserved for cases in which:

- deck width exceeds 25 m,
- insufficient height available for integrating the deck of a twin girder bridge,

- site constraints prohibit use of common lifting equipment, requiring lighter, and thus more, main girders,
- the span/width ratio is very low.

With its two 4-girder decks, the Saint-Denis canal viaduct on the A86 motorway is probably the largest multi-girder composite bridge built recently in France.

Another application involving the use of multi-girder decks is concrete deck and prestressed beam replacement. In this case, the new composite deck transfers its loads to the existing supports under conditions similar those applicable to the existing concrete deck to be replaced (the new deck for the bridge over the River Drôme on the A7 motorway provides an excellent illustration of this application).

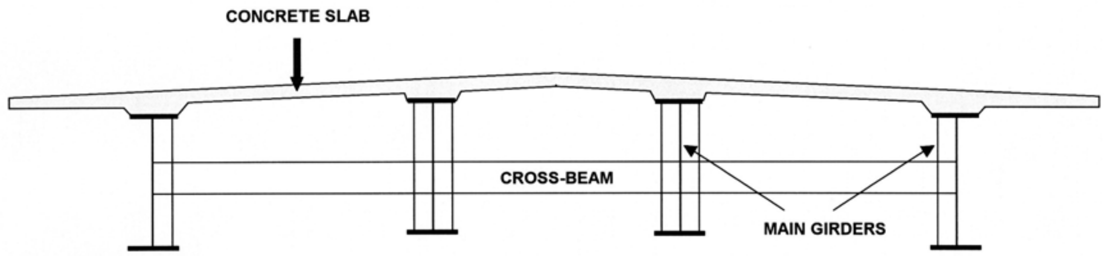


Figure 2.4. 4-girder composite bridge (pavement bidirectional banking)

Cross-beam composite bridges with deck local widening near abutment

In an urban environment, the last few metres of a bridge deck require fairly major local widening to allow efficient merging of traffic on a lane perpendicular to the bridge. In this case, it is possible to opt for a steel frame with a constant main girder centre-to-centre distance and special directly supporting cross-beams or props in the widened zone. This solution has the advantage of not increasing the complexity of steel frame launching.Ouvrages à poutres avec élargissement localisé près d'une culée

Variable width cross-beam composite bridges

When the width of a cross-beam composite bridge varies over a significant length, the main girder centre-to-centre distance must often be varied and this will increase the steel frame launching complexity, if this deck installation method is planned.

If the width variation is large, it may also be necessary to deepen the slab and even replace the cross-beams with directly supporting cross-beams, if the structure is initially a cross-beam composite bridge.

Special girder composite bridges

Several cross-beam composite bridges outside the categories introduced above have been in France within the context of either an innovative approach or a design competition. Amongst the most interesting, we can name:

- the Blois bridge over the River Loire and the Pritz bridge over the River Mayenne at Laval, whose main girders are variable depth lattice members,
- the South entry/exit bridge on the Lille ring road, which is a twin girder composite structure with cross-beams in contact with the main girder bottom flanges and permanently braced at the bottom,

- Overpass No.13 on the A85 motorway, whose main girders are micro box girders made up of two welded standard sections filled with concrete and whose slab is constructed from longitudinally prestressed bonded combined precast segments,
- the Monestier-de-Clermont viaduct on the A51 motorway, whose twin girder cross-beam composite deck loads are transferred to the piers through V-shaped props,
- Access viaducts on the Gustave Flaubert lifting bridge at Rouen, whose very high directly supporting cross-beams are open in their central sections.

Current research

During the ten or so years preceding completion of this guide, two major developments for further increasing the competitivity of composite road bridges have been researched.

The first involves integrating horizontal bracing composed of structural steel sections between the bottom flanges of the main girders of a twin girder composite structure. Designed to improve load distribution between the main girders, this development has prompted a number of designs, but only one full-scale application: the OA1 bridge on Lille’s eastern ring road.

The second major development involves replacing the conventional concrete slab, which is often cast in place, by longitudinally prestressed precast slab segments made of ultra-high performances fibers reinforced concrete (UHPFRC). Given the currently high price of this material, these segments incorporate an ultra-thin slab (approximately 5 cm) stiffened by longitudinal and transverse ribs, spaced at approximately 60 cm, which call to mind a waffle and prompt its name of “waffle slab”. At the end of 2009, this development was subjected to and extensive test programme but had not yet prompted any application in a real structure.

2.1.3 -Longitudinal morphology

Span distribution

Composite bridges, especially those of girder design, offer tremendous flexibility in terms of span distribution.

For a very long bridge crossing a natural topographic gap subject to no particular constraint, the ratio between the end span length and standard span length can reach 0.8, which allows the structure’s maximum span to be limited for a given number of supports (Figure 2.5).

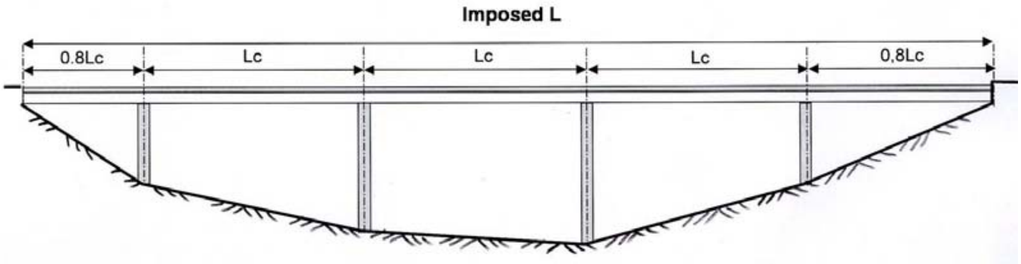


Figure 2.5. Bridge with imposed abutment postions and “long” end spans

Conversely, for a bridge crossing a fairly flat topographic gap featuring major obstructions (waterway, railways, roads or motorways), the ratio between the end span length and standard span length can decrease to 0.6 without support vertical adjustment and to as little as 0.5 with support vertical adjustment (Sections 3 and 4), thereby allowing the structure’s total length to be limited to an absolute minimum (Figure 2.6).

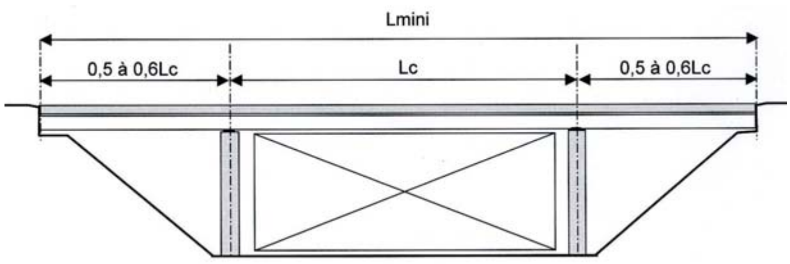


Figure 2.6. Bridge with imposed pier positions and "short" end spans

Composite bridges are also more suitable than most concrete structures (post-tensioned prestressed beam bridges, prestressed concrete box girder bridges built by successive cantilevering or installed by pushing) with an irregular span distribution.

For bridges featuring directly supporting cross-beams, it should be recalled that the cross-beam centre-to-centre distance must be as constant as possible thoughout the bridge length, which may prompt adaptation of the projected span lengths to ensure they are all a multiple of this cross-beam centre-to-centre distance.

Constant depth

The most common bridges integrate constant depth main girders throughout their length (.7). This provision in fact provides the most economic solution in terms of shop-fabricating, then installing, the girders.

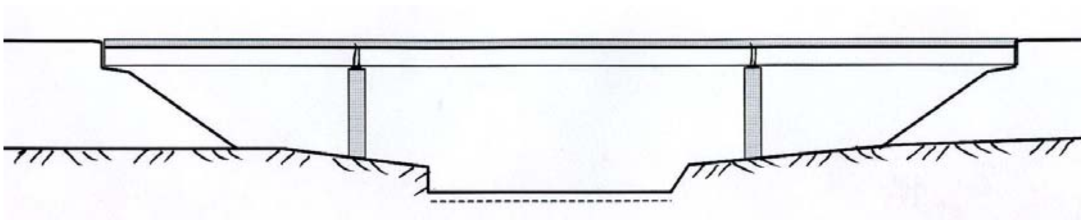


Figure 2.7. Bridge with constant deck depth throughout its length

Main girder depth linearly variable in end spans and constant elsewhere

An alternative to the previous morphology involves linearly reducing the main girder depth by approximately one third in the structure's end spans (Figure 2.8). This arrangement is well suited to bridges with short end spans and allows the structure's silhouette and its integration into the site to be enhanced without implicating the project economics.

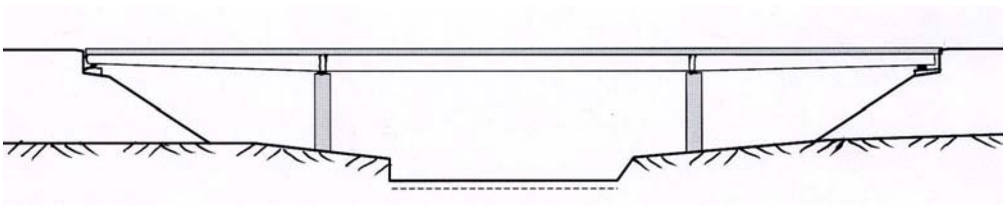


Figure 2.8. Bridge with linearly variable depth deck in end spans

Variable main girder depth

Composite bridges with their main girders varying in depth throughout their length can be designed. This depth variation is usually parabolic, but can also be cubic or even linear (Figure 2.9).

Variable girder depth invariable leads to more complex fabrication and installation of the steel frame. It is therefore only adopted in special cases (bridges with large spans or major clearance constraints requiring the lowest possible longitudinal profile, etc.). Sometimes, variable girder depth is retained in relation to purely aesthetic considerations.

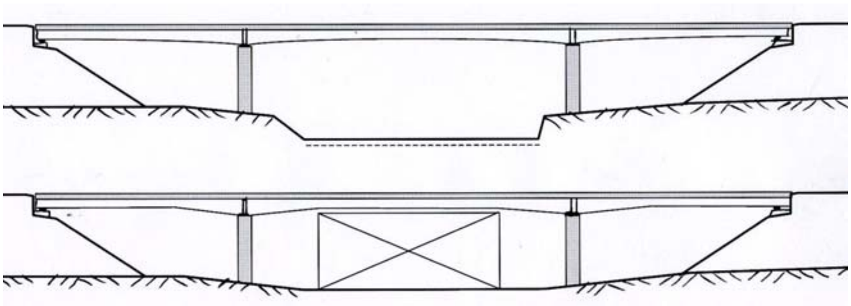


Figure 2.9. Bridges with deck depth varying thoughout their length

Saulières viaduct on the Brive bypass and Jassans bridge are good examples of structures with parabolically varying deck depths.

Deck with constant and variable depth parts

A number of bridges with decks combining constant and variable depth parts have been built in recent years (Figure 2.10). This design is highly advantageous for very long structures crossing a clearly identified obstruction. From an engineering standpoint, it allows large span lengths to be integrated over the obstruction and smaller, more economic, span lengths to be placed in areas free of major constraints. From an aesthetic standpoint, it highlights the main obstruction and breaks the monotony of constant depth spans.

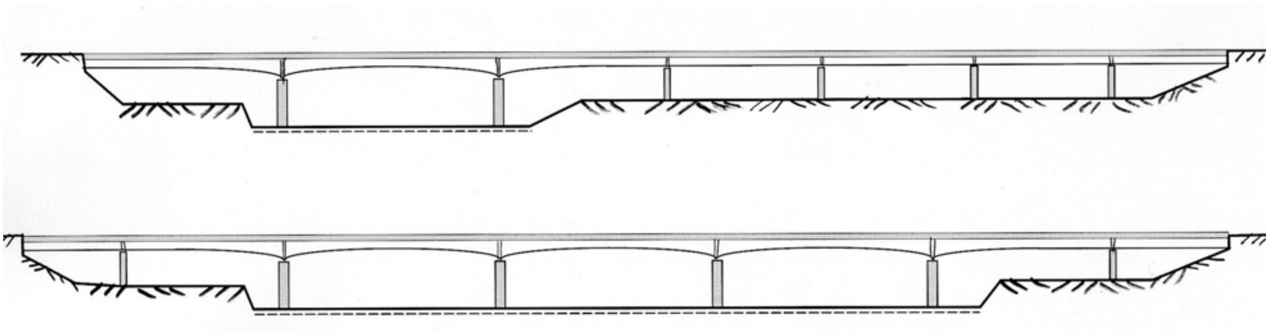


Figure 2.10. Bridges with decks combining constant and variable depth parts

Several recent large bridges embody this design basis: the A86 viaduct crossing the Saint-Denis canal, the A89 viaducts over the Vézère and Dordogne rivers and the A20 viaduct of the Lot river.

2.1.4 -Preliminary design data for twin girder composite bridges

2.1.4.1 - Preliminary design of twin girder cross-beam bridges

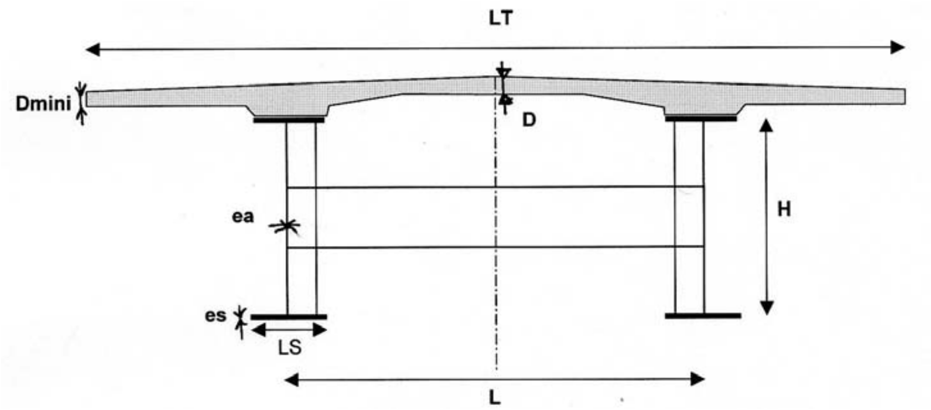


Figure 2.11. Design parameters for a twin girder cross-beam composite bridge

The table below consolidates steel frame and slab preliminary design data for twin girder cross-beam composite bridges using the notation defined in Figure 2.11.

Main girder depth H	*Max* ($\frac{X}{28}\left(\frac{LT}{12}\right)^{0.45}$, $0.40+\frac{X}{35}$) for a constant depth deck. *X*/24 at pier and *X*/36 at mid-span for a variable depth deck with more than 2 spans.
Main girder c/c distance	*L* = approx. 0.55 *LT*
Bottom flange width (Binf)	$\left(0.25+\frac{LT}{40}+\frac{X}{125}\right)\left(0.92+\frac{LT}{150}\right)$
Top flange width (Bsup)	*Binf* – 0.100 for a 2-lane deck *Binf* – 0.200 for a 4-lane deck
Standard cross-beams	IPE500 to IPE700 standard section or equivalent
Steelwork tonnage	$63+0.9X^{1.2}\left(1.34-\frac{LT}{40}\right)$ + 0.25 *X* in kg/m² of deck
Slab thickness	$0.13+\frac{(LT-L)}{26}$ at main girders $0.12+\frac{L}{50}$ at deck centre
Slab reinforcement ratio	Approx. 250 kg/m³

In the above relations, X is the standard span length or, for unequal spans, the weighted length of the two longest consecutive spans $X=(2\times l_i+l_{i+1})/3 \quad for \quad l_i>l_{i+1}$ (end span lengths are multiplied by 1.25 when applying this formula) or, for isostatic spans, $X=1.4\times l$.

2.1.4.2 - Preliminary design of twin girder directly supporting cross-beam bridges

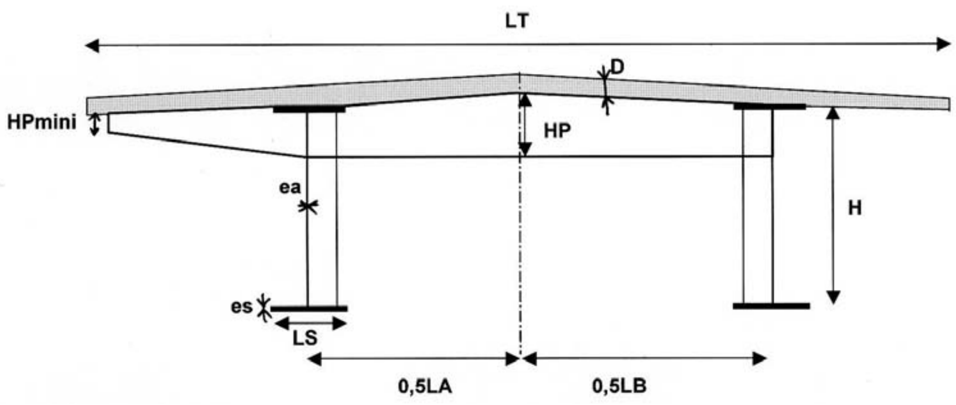


Figure 2.12. Design parameters for a twin girder directly supporting cross-beam composite bridge (with cantilevers at left, without cantilevers at right)

The table below consolidates steel frame and slab preliminary design data for twin girder directly supporting cross-beam bridges using the notation defined in Figure 2.12.

Main girder depth H	Max ($\frac{X}{28}\left(\frac{LT}{12}\right)^{0.333}$, $0.40+\frac{X}{35}$) for a constant depth deck. *X*/24 at pier and *X*/36 at mid-span for a variable depth deck.
Main girder c/c distance	*LA* = approx. 0.55 *LT* *LB* = *LT* - 4 m.
Bottom flange width (Binf)	$0.25+\frac{LT}{40}+\frac{X}{125}$
Top flange width (Bsup)	*Binf* – 0.100 for a 2-lane deck *Binf* – 0.200 for a 4-lane deck
Directly supporting cross-beam depth	*HP* = approx. 1/11th. of *LA* or *LB*. *HPmini* = approx. 300 mm.
Steelwork tonnage	$65 + 0.9\,X^{1.2}\left(1.43-\frac{LT}{30}\right)$ + 2 *LT* + 0.22 *X* in kg/m²
Slab thickness	24 to 26 cm
Slab reinforcement ratio	Approx. 275 kg/m³

The notation adopted is the same as for preliminary design of twin girder cross-beam bridges.

2.1.4.3 - Painting areas

The total area requiring corrosion protection can be evaluated once the steelwork tonnage P has been estimated based on the conditions detailed above. To do this, P is multiplied by the ratio given by the “twin girder” curves plotted on the chart below (Figure 2.13).

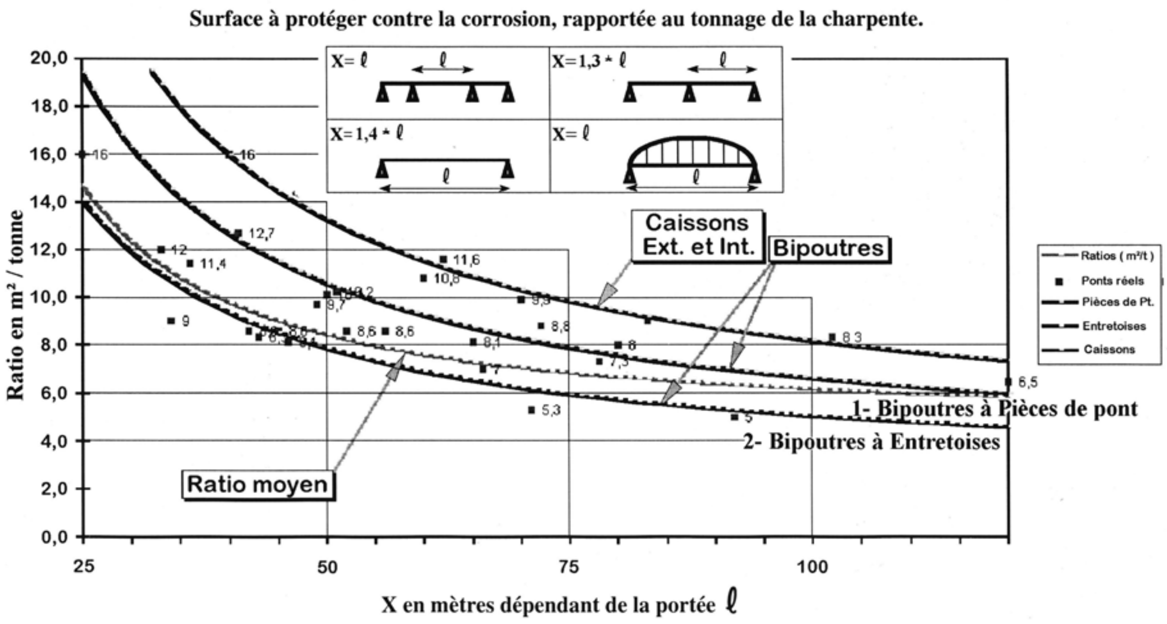


Figure 2.13. Required corrosion protection area per tonne steelwork with respect to X

2.2 - General design of box girder composite bridges

2.2.1 -General

Box girder composite bridges are much less common than conventional girder structures. In the absence of specific constraints, they are effectively more complex and so more expensive to build and maintain. However, they are well suited to cases, in which at least one of the following conditions is present:

- maximum span length exceeds approximately 90 m,
- deck width exceeds 20 or so metres,
- insufficient height available for integrating the deck of a conventional girder structure,
- horizontal curvature is high (angular span ratio P/R > 0.2).

A box girder structure is sometimes adopted in preference to a conventional girder structure for purely aesthetic reasons. A box girder almost always appears lighter than a conventional girder deck because it is both more slender and its inclined webs make it seem more slender than a twin girder deck for a given deck depth.

Moreover, a box girder is sometimes retained in preference to a conventional girder deck either because the space available for its supports is very small (case of bridges crossing roads or railways) or for purely architectural reasons. Box girder web inclination effectively means that its bearing devices are always significantly closer together than those of a conventional girder deck; this allows more compact pier designs.

To be totally comprehensive, it should be finally stated that box girders are sometimes preferred to twin girder decks simply because of their greater capacity for resisting vehicle or floating body impacts.

Tables C to E in Appendix 1 of this guide list the main box girder composite road bridges built in France between 1995 and 2005. These represent approximately 10% of all composite bridges built.

2.2.2 -Transverse morphology

Single open box girder composite bridges

The simplest box girder decks comprise a concrete slab and a U-shaped steel frame. The latter is made of longitudinal plates forming the external U (from top to bottom, 2 top flanges, 2 webs and a bottom flange) along with transverse elements of two types: bulkheads located at the bridge supports and transverse frames located within the spans (Figure 2.14).

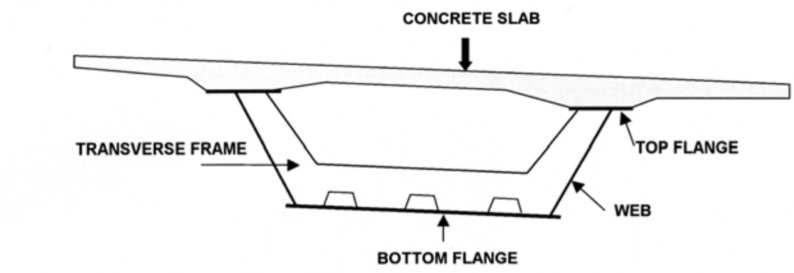


Figure 2.14. Single open box girder
(pavement unidirectional banking)

The slab has essentially the same characteristics and is subject to similar construction methods as a twin girder cross-beam bridge slab.

The top flange characteristics are effectively identical to those of twin girder structures.

The bottom flange is made up of plate sections of constant thickness. In most cases, it is stiffened by trapezoidal box ribs, known as bucklets, or by T-sections and sometimes by flat bars.

Box girder webs are composed of plate sections of constant thickness. They are usually inclined with respect to an axis perpendicular to the bottom flange and are normally stiffened by flat bars or T-sections.

The transverse frames are very open transverse elements welded to the top flanges, webs and bottom flange (Figure 2.15). They are designed to prevent excessive transverse deformation of the box girder and are spaced at a centre-to-centre distance of between 4 and 6 m.

The bulkheads are also transverse elements, but are located at the bridge supports. They are designed to take up multiple loads, including torsional and support reaction loads, and are composed of heavily stiffened plate sections, which effectively close the entire U-shaped cross section of the box girder except for a manhole (Figure 2.15). They are in contact with, and connected to, the concrete slab throughout the top width of the U-shaped cross section.

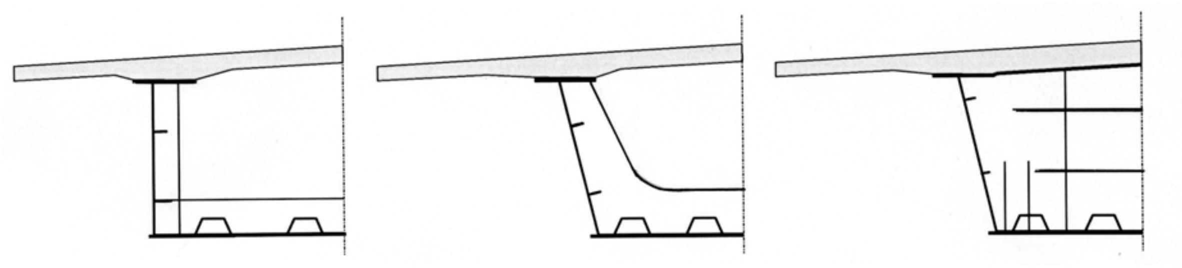


Figure 2.15. Transverse elements: transverse frames at left and centre; bulkhead over support at rights

The depth of an open steel box girders must not be less than 1.5 m; this condition is essential to both constructing the concrete slab and their inspection.

The most economical steel box girders can be completely assembled in the fabrication shop and their bottom flange is fabricated from a single steel plate. This is usually the case for box girders, whose overall breadth does not exceed 6 m and whose bottom flange width does not exceed 4.50 m. When one of these conditions cannot be met, a longitudinal weld is incorporated in the centre of the bottom flange either in the fabrication shop or on site; this naturally increases the cost of the structure.

Amongst recent bridges of this type, we can name the bridge DE at the Palays interchange in Toulouse, the Ners viaduct over the River Gardon and the bridge over the River Ante.

Single closed box girder composite bridges

An alternative to the open structure described above is the closed box girder; This is identical to the open box girder except that the two top flanges are replaced by a top plate (Figure 2.16).

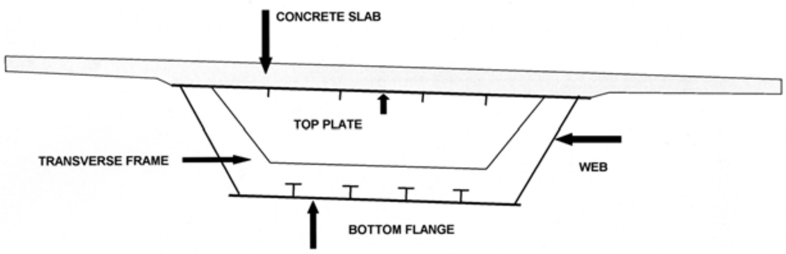


Figure 2.16. Single closed box girder (pavement unidirectional banking)

The top plate is stiffened by flat bars or T-sections in these structures.

These bridge decks require a little more steel than the previous described single open box girders for the same widths and spans, but use of a plate as the top member simplifies certain operations, especially deck construction, because the top plate is incorporated as permanent formwork. Single closed box girders are therefore structures that are particularly well suited to small size decks. They are also very suitable for curved bridges because their top plate avoids the need to temporary bracing.

Structures of this type are fairly unusual. The small box girder composite structures, which widen the Aquitaine bridge access viaduct, are nevertheless good examples.

Box girder composite bridges with directly supporting cross-beams and cantilevers

As in twin girder composite bridges, box girders can be designed with a secondary framework supporting the slab (Figure 2.17).

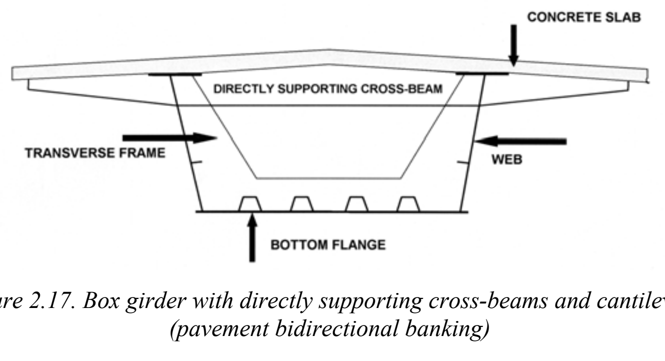


Figure 2.17. Box girder with directly supporting cross-beams and cantilevers (pavement bidirectional banking)

In common with cross-beam structures, box girders with directly supporting cross-beams are more difficult to build than single box girders. they are therefore usually used when:

- the deck width exceeds 13/14 m,
- the span length exceeds approximately 90 m.

In these structures, the characteristics of the top flanges, webs and bottom flange are similar to those of single box girders.

The centre-to-centre distance between the directly supporting cross-beams must be as constant as possible and near to 4 m. The cross-beams are associated with the box girder frames and are often extended as cantilevers, usually of linearly variable depth, beneath the overhanging parts of the slab.

The slab has the same characteristics and is subject to similar construction methods as a slab for a twin girder directly supporting cross-beam composite bridge. The slab is therefore thin and of constant thickness.

A few structures of this type have been built in recent years: the top deck of the Roche Bernard arch bridge, the Avignon ring road viaducts and the Trans-Val-de-Marne bridges crossing the A86 and A106 motorways in Isle de France.

Box girder composite bridges with directly supporting cross-beams and no cantilevers

In place of transverse frame, some box girder composite bridges incorporate bulkheads, whose top parts (above the manhole) play the part of directly supporting cross-beams for the slab central section (Figure 2.18). Bulkhead centre-to-centre distance is usually 4 m, which allows the slab thickness to be reduced to a minimum.

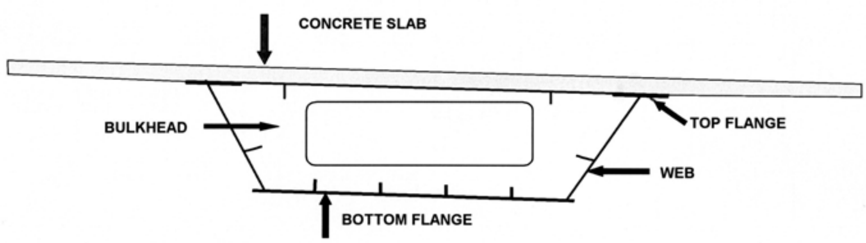


Figure 2.18. Box girder with directly supporting bulkheads and no cantilevers (pavement unidirectional banking)

The deck of the Boulogne-sur-Mer viaduct on French national road RN1 is a good example of a box girder composite bridge with directly supporting cross-beams and no cantilevers.

Box girder composite bridges with directly supporting cross-beams and propped cantilevers

Two very large box girder bridges with directly supporting cross-beams and propped cantilevers (Figure 2.19) have been built in recent years: the Verrières viaduct on the A75 motorway and the second bridge over the River Rhône at Valence.

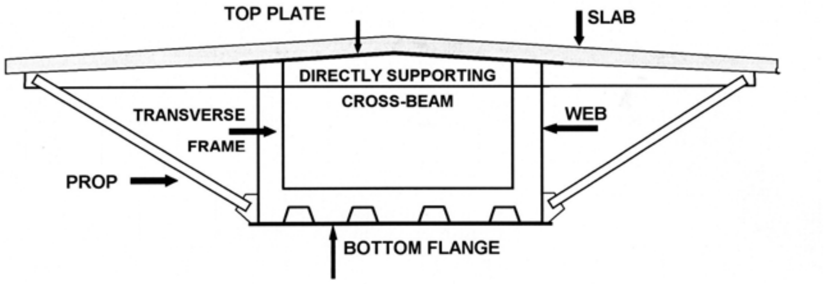


Figure 2.19. Box girder with directly supporting cross-beams and propped cantilevers (pavement bidirectional banking)

These box girder decks feature a central core composed of top and bottom plate flanges, two fairly close vertical webs and a series of transverse frames. Laterally, large overhangs are supported by cantilevers extending from the frames and these cantilevers are in turn supported by inclined props bearing on the base of the central core. The props are usually tubular and are positioned either perpendicular to the deck longitudinal axis, as on the Valence bridge, or in inclined longitudinal planes, in which they are triangulated, as on the Verrières viaduct.

Built in 2004/2005 near Beauvais, the Frocourt viaduct, although of smaller width (12.90 m), features a similar deck structure to the second bridge over the River Rhône, but for essentially aesthetic reasons.

Double box girder composite bridges

Decks comprising two small steel box girders and a single concrete slab can be envisaged for some very wide bridges with small spans. Despite the width of the deck, the main advantages of these structures are the possibility of transporting the fully assembled box girders from the fabrication shop to the site and the pier location flexibility for each box girder. Their principal drawbacks are the complexity of their behaviour and their slab construction conditions.

Other transverse design-related issues

Box girder composite bridges are often retained at sites, where piers must be of restricted size, because of the small width of their bottom flange. In some extreme cases, it may may necessary to restrict support to only one bearing device per pier, allowing the supporting crosshead to be reduced to a minimum, and to take up all torsional loads at the abutments.

2.2.3 -Longitudinal morphology

Total length and span distribution

Box girder composite bridges are as flexible in span distribution terms as conventional girder composite bridges.

If the box girder features directly supporting cross-beams, the centre-to-centre spacing of the latter must also be as constant as possible throughout the length of the structure, which may require adaptation of the projected spans to ensure they are all multiples of this cross-beam centre-to-centre distance.

Variation in depth

The great majority of box girder composite bridges feature a constant deck depth. This provision in fact simplifies greatly their construction and box girder installation, especially if this is performed by launching.

Structures departing from this rule are indeed extremely rare. Amongst these, we can mention the portal leg bridge over the Ante river, whose deck depth varies linearly, and the Vienne river bridge at Nouâtre, whose box girder bottom is horizontal whilst its top flanges are parallel to the parabolic longitudinal profile.

2.2.4 -Preliminary design data for box girder composite bridges

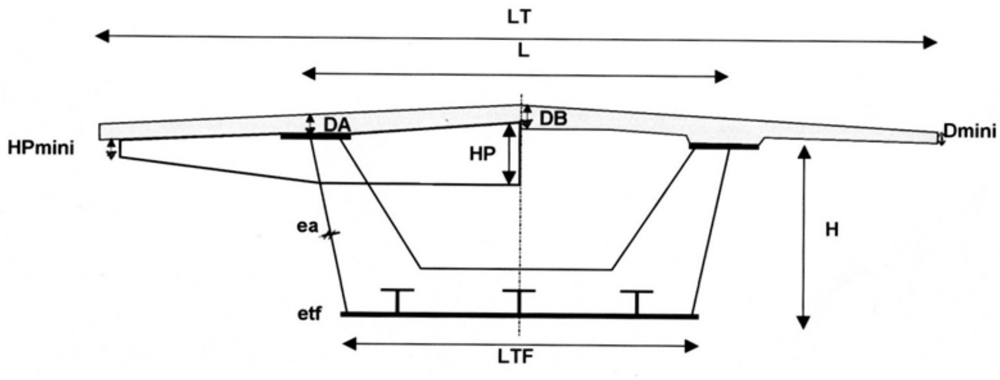


Figure 2.20. Design parameters for a box girder composite bridge (with directly supporting cross-beam and cantilevers at left, no cantilevers at right)

Web top c/c distance	L = 0.50 to 0.55 LT.
Web depth	H = 1/30th to 1/40th of maximum span distance.
Web inclination	0 to 50%.
Directly supporting cross-beam depth	HP = approx. 1/11th of L. HPmini = approx. 300 mm.
Top flange width + depth for open box girders	Same as for cross-beam composite bridges.
Web thickness	ea = 16 - 35 mm depending on cross section, width and span distance.
Bottom flange thickness	etf = 25 - 80 mm depending on cross section, width and span distance.
Slab thickness	Box girder without directly supporting cross-beams $0.13+\frac{(LT-L)}{26}$ at webs, $0.12+\frac{L}{50}$ at deck centre Box girder with directly supporting cross-beams 24 - 26 cm
Slab reinforcement ratio	Approx. 250 kg/m³ with simple frames, 275 kg/m³ with frames + directly supporting cross-beams or bulkheads

2.3 - Miscellaneous points possibly influencing general design

2.3.1 -Road alignment

At general design stage, particular attention should be given to the alignment of the road carried by the bridge because this can make it difficult and even impossible to install the steel frame under certain conditions.

Section 4 of this guide introduces multiple steel frame installation methods, of which the most common are launching and crane installation.

Installation by launching, i.e. rolling or sliding over temporary devices positioned on the bridge supports, is rather unsuitable for complex alignments, i.e. other than a circular arc or straight line. Thus, a deck whose horizontal alignment comprises a circular arc followed by a straight line must be launched from both its abutments and this is invariable more tedious and expensive than launching from one abutment. Similarly, intalling a deck featuring a clothoid curve at its centre will cause problems.

Crane installation is possible with all, even the most complex, road alignments. Except for very small bridges, which can be installed in a single operation from one of the abutments, this method can only be implemented if the deck is less than 15 or so metres above the ground and sufficiently powerful cranes can be brought to the site.

At preliminary design stage, simple, regular road geometries (single longitudinal and horizontal geometrical profile, constant banking and width) should be sought and alignment changes should be requested from the road designer, if necessary.

2.3.2 -Horizontal curvature

Horizontally curved composite bridge decks are almost always obtained by cutting the main girder and box girder bottom flanges according to the horizontal curvature of the road carried by the structure and constructing a slab with constant transverse characteristics above the steel frame thus fabricated.

In a twin girder composite deck with a curved steel frame, stresses are higher in the external girder than in the girders of a straight deck with the same axial span.

For information, in a circular isostatic deck under uniform load q, the maximum stress in the bottom flanges can be determined by multiplying the stress in the same flanges of an equivalent straight structure by:

$$\alpha = 1 + (P^2/2R)(5/12L + 1/K^2Ls)$$

in which P is the deck developed span, R is the road radius of curvature, L is the distance between main girders, K the cross-beam span/centre-to-centre distance ratio and Ls is the bottom flange width.

Moreover, for identical span distances, the cross-beams are subjected to higher stresses in a curved than in a straight bridge deck.

It is therefore advantageous to either reduce span distances or increase girder centre-to-centre distance for tightly curved cross-beam composite bridges. However, if these two parameters are imposed, the curvature quickly causes significant load increases in the external girder, which may prompt a preference for a far more torsionally rigid box girder structure.

2.3.3 - Skewness

A skew girder composite bridge, i.e. in which one of the axes of support is not perpendicular to its longitudinal axis, can be designed but it frequently suffers from numerous drawbacks:

- the transverse elements must follow the skew or else be subjected to specific loads created by the deflection difference at their ends; this results in their increased length and more costly assemblies;
- all or part of the slab segments are skew;
- if the structure is continuous, vertical support adjustments are not advisable because they generate high loads and deformations in the bearing areas.

We therefore recommend designing the composite bridge as non-skew as possible by firstly investigating the possibilities of altering the road alignment and by considering every possible pier shape, when the support ground position is decisive for the skew.

2.4 - Related bibliography

Some sections of this guide feature a "related" bibliography for the area specifically covered by the section concerned. Terms in square brackets indicate papers listed in Appendix 2, entitled "Bibliography", of this guide. Terms RT, BOA and OTUA indicate the French journal "Travaux", Sétra's structural engineering bulletin and ConstruirAcier's steel bridges bulletin respectively.

Twin girder cross-beam composite bridges

RT [MAR 95] [COU 95] [CHA 95] [MEU 96] [AMA 96] [AVR 01] [DEM 02] [STO 03] [MAR 07]

BOA [NOR 95] [GIL 96] [BAR 00] [VIO 08]

OTUA [HIP 96] [DEZ 03] [PRE 09] [BER 09]

Twin girder directly supporting cross-beam composite bridges

RT [ASF01 02] [ASF02 02] [CAL 02] [MAN 02] [BRI 03] [DUB 04] [DUM 06] [MOS 09]

OTUA [MOS 09]

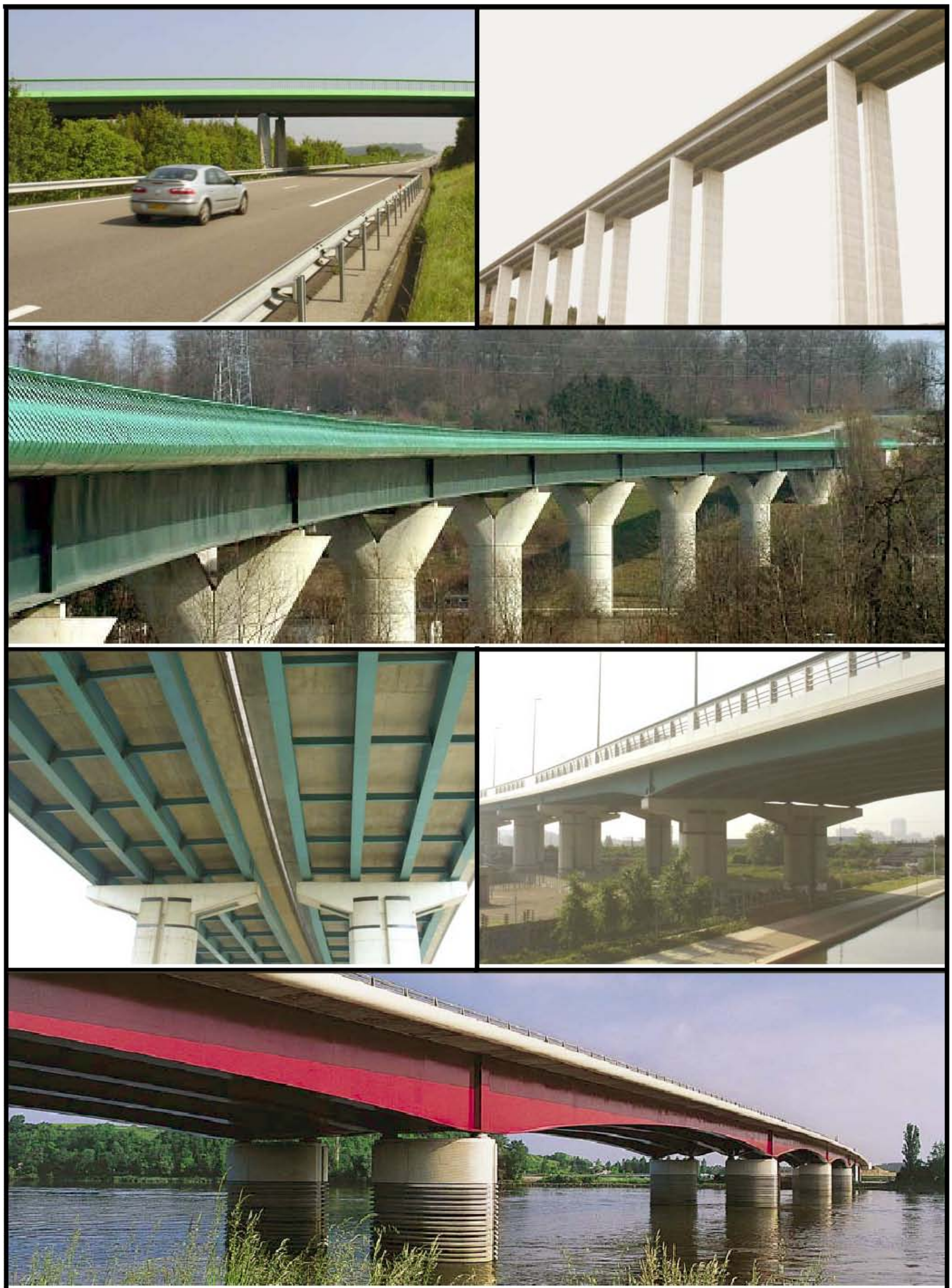
Box girder composite bridges

RT [POI 97] [CHA 00] [BOU 01] [GIL 01] [HAU 07]

BOA [FON 95] [DAI 05] [MON 96] [BAR 06]

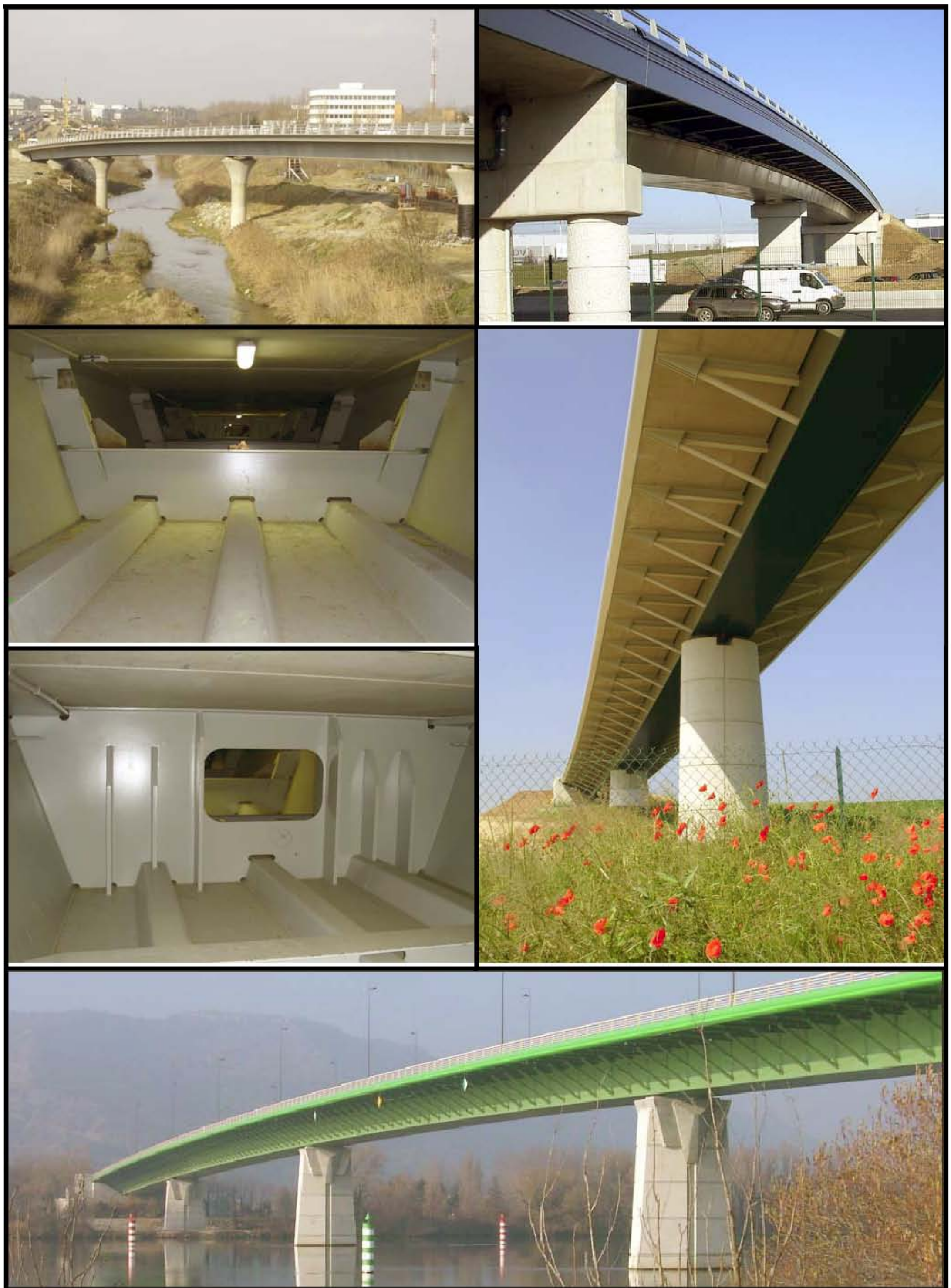
OTUA [VIL01 96] [TAV 04] [FLE 04] [GIL 04]

■ ■ ■



201: Twin girder cross-beam composite overpass. 202 and 203: Twin girder cross-beam composite bridges. 204: Underside of a double 4-girder composite bridge. 205: Double 4-girder cross-beam composite bridge with parabolic variable and constant depth areas. 206: Double twin girder cross-beam composite bridge with parabolic variable depth girders.

211 to 213: Twin girder directly supporting cross-beams and cantilevers.
214 to 215: Twin girder directly supporting cross-beams and no cantilevers (variable depth girders).



221: Single box girder. 222: Box girder with directly supporting cross-beams.
223: Standard frame. 224: Bulkhead at support.
225 and 226: Box girders with directly supporting cross-beams and propped cantilevers.

231: Propped composite bridge with linearly variable depth box girder (doubling in progress). 232: Twin girder bottom-braced composite bridge. 233: Twin girder cross-beam composite bridge with partial propping. 234: Twin girder composite bridge with open directly supporting cross-beams in central part. 235: Overpass with 2 HEA800 welded main beams

3 - Detailed design

This section covers the detailed design of the most common composite bridges. It deals successively with twin girder structural steel frame, box girder structural steel frame and concrete slab design.

3.1 – Choice of materials

3.1.1 - Structural steel and assemblies

French statutory texts

At the time of preparing this guide, the main texts governing structural steel characteristies are:

- Fascicule 4 Title III of CCTG [French general technical specifications for government contracts] complemented by the Sétra information memorandum of March 2007 entitled "Approvisionnement en tôles d'acier pour Ouvrages d'art" [procurement of steel plate for structures] for the “NF-Acier” quality mark,
- Fascicule 66 of CCTG [French general technical specifications for government contracts],
- Standards NF EN 10025-1 to 6,
- Standards NF EN 1993-1-10 and NF EN 1993-2, and their national appendices.

Steel designation

Steels used for building bridges are designated by a grade (e.g. S355) and a quality (e.g. K2+N, M, ML, etc.). We therefore refer to "S355K2+N", "S420M" or "S460ML" steels.

Steel grade

The grade comprises the letter S (for structural steel) followed by the elastic limit in N/mm^2 (355, 420, 460). The latter limit is for the thinnest steel plate in the range, elastic limit decreasing slightly with thickness.

The most common structural steel grade is S355, but even higher performance, so-called high elastic limit, steels are available (S420, S460, S690, etc.). It should be noted that Eurocode 4 Part 2 only covers steel grades inferior or equivalent to S460.

The longitudinal members of most structures are wholly constructed from S355 grade steels. A number of recent large bridges, such as the Verrières viaduct and second bridge over the River Rhône at Valence Textes réglementaires nevertheless include sections at their piers made of S460 steel. This provision allows the steel frame weight to be somewhat curtailed (up to approximately 10%) and facilitates its launching.

Steel quality

Quality (K2+N, N, M, etc.) is a parameter characterising steel toughness, in other words its capacity for absorbing impacts without risk of brittle fracture. This risk increases with the thickness of the piece and the steel grade; quality therefore depends on these two parameters.

For each type of steel considered (unalloyed, fine grained, normalised state), Standards NF EN 10025-1 to 6 specify the grades that can be used and the qualities, in which these grades can be supplied. Furthermore, the national appendix to Standard NF EN 1993-2 lays down additional requirements for steel toughness, which give the minimum quality to be used with respect to plate thickness (table 3.1).

Plate thickness	Minimum quality
t ≤ 30 mm	J2
t > 30 mm	N or M or Q (fine grained steels)

Table 3.1. Minimum steel quality w.r.t. plate thickness

For the French market, steel qualities frequently used in composite road bridges are as follows:

Grade	Thickness	Quality
S355	e ≤ 30 mm	K2
S355	30 < e ≤ 80 mm	N or M
S355	80 mm < e ≤150 mm	NL or ML
S460	e ≤ 50 mm	M
S460	50 mm < e ≤ 120 mm	ML

Table 3.2. Steel qualities frequently used in French composite road bridges

Limit of elasticity

The limit of elasticity to be applied in design calculations depends on the thickness of the piece being checked. Table 3.3 below, which is derived from different sections of Standard NF EN 10025, gives examples of the variation in the elastic limit of steel plate with respect to its thickness in mm

Designation	e≤16	16<e≤40	40<e≤63	63<e≤80	80<e≤100	100<e≤120	120<e≤150
S355N or NL	355 MPa	345 MPa	335 MPa	325 MPa	315 MPa	295 MPa	295 MPa
S460M or ML	460 MPa	440 MPa	430 MPa	410 MPa	400 MPa	380 MPa	-

Table 3.3. Elastic limit of steel plate w.r.t. thickness

Plate maximum thickness

Table 2.1 in Standard NF EN 1993-1-10 establishes the maximum thickness for steel plate of given grade and quality. Maximum thickness is governed by two parameters:

- the reference temperature T_{Ed}, defined in Standard NF EN 1993-1-5 and its national appendix,
- the level of stress in the element s_{Ed} , in conjunction with this temperature.

Table 3.4 below illustrates this derived maximum thickness using the example of two common steel grades subjected to a stress level *s* $_{Ed}$ less than 0.5.$f_y(t)$.

Désignation	$\sigma_{Ed} = 0{,}50.f_y(t)$		
	$T_{Ed} = -10°C$	$T_{Ed} = -20°C$	$T_{Ed} = -30°C$
S355N	110 mm	95 mm	80 mm
S355NL	155 mm	135 mm	110 mm

Table 3.4. Examples of maximum thickness based on Standards NF EN 1993-1-10 and NF EN 1993-1-10/NA

"NF-Acier" quality mark

In France, fascicule 4 title III of CCTG requires steel plate bearing the "NF-Acier" quality mark. However, plate production respecting *stricto sensu* this requirement is less than structural engineering needs, so Sétra and the LCPC [Laboratoire Central des Ponts et Chaussées – French civil engineering research laboratory] have developed transitional measures allowing exemption from this "NF-Acier" requirement without callling into question expected quality. These provisions are laid down in Sétra information memorandum of March 2007 entitled "Approvisionnement en tôles d'acier pour Ouvrages d'art" [procurement of steel plate for structures].

Delamination resistance

Certain steelwork plates can be stressed in tension in the direction of their thickness. This is especially the case for intermediate webs and intermediate cross-beam post flanges in multi-girder composite structures.

In compliance with Standard NF EN 1993-1-10 Section 3, it must be ensured that no rolling or welding fault can cause delamination (i.e. separation into leaves) of these members. To ensure this, plates of the appropriate quality Z should be used in compliance with Standard NF EN 10164 and ultrasound testing should be performed to ensure that there are no defects after fabrication.

Assemblies

Assembly is nearly always performed by arc welding for the French composite bridges covered by this guide. This is the most durable and aesthetic fabrication method.

Bolted assembly on site is very rare and only used for small structures or those for which assembly must be performed either as quickly as possible or under highly adverse climatic conditions (cold, wind). Moreover, this operation must only be performed using high-strength friction grip (HSFG) bolts.

3.1.2 - Slab concrete

French statutory texts and engineering guides

At the time of completing this guide, the main texts governing slab concrete characteristics are:

- Fascicule 65 of CCTG 2008, in particular its Section 8 entitled "Bétons et Mortiers" [concrete and mortar mixes],
- Standard NF EN 206-1 of April 2005 and its amendments,
- Standards in the NF EN 1992 series,
- Guides entitled "Recommandations pour la prévention des désordres dus à l'alcali-réaction" [recommendations for preventing damage due to alkaline reaction] and "Recommandations pour la prévention des désordres dus à la réaction sulfatique interne" [recommendations for preventing internal sulphate reaction] published by the LCPC in June 1994 and August 2007 respectively,
- Sétra/LCPC guide entitled "Ponts mixtes – Recommandations pour maîtriser la fissuration des dalles" [composite bridges – recommendations for controlling slab cracking], part of which has been rendered obsolete by the Eurocodes but which include concrete mix selection data that is still relevant,
- for standard structures, the LCPC 2009 guide entitled "Approche performantielle de la durabilité des bétons - Applications aux ouvrages courants - Recommandations provisoires" [performance approach to concrete durability – application to standard structures – provisional recommendations],
- for very large structures, the Association Française de Génie Civil (AFGC) guide entitled "Conception des bétons pour une durée de vie donnée des ouvrages" [design of concrete mixes for a given structural lifespan] published in July 2004.

Expected concrete qualities for a composite bridge slab

The main qualities, which are sometimes contradictory, required of a composite bridge concrete slab are:

- fluidity compatible with congested reinforcement,
- rapid gain in strength, allowing a quick mobile formwork cycle,
- limited shrinkage,
- excellent durability.

Strength class

When the slab is cast in place, the concrete strength class is most often C35/45 as defined by Standard NF EN 206-1; this provides a good compromise between shrinkage and durability. Higher performance concrete mixes are sometimes used, especially when the slab is precast.

Concrete mix design for a conventional approach

At the time of preparing this guide, Owners and Engineers offer contractors a degree of freedom in terms of mix design, whilst establishing a number of key assumptions such as:

- prevention level for alkaline and internal sulphate reaction-related risks,
- frost and salting intensity on roads carried and spanned by the structure,
- concrete strength class.

The CCTP also lays down several important parameters such as cement content, type and specific characteristics as well as the ratio Eeff/Leq [effective water/equivalent binder]. The table below shows the normally imposed characteristics for composite bridge slab concrete to ensure 100-year structural serviceability based on all exposure classes to which this slab may be subjected.

Exposure classes (1)	Strength class (2)	Min. equivalent binder content w.r.t. durability	Cement type w.r.t. durability	Additional cement durability characteristics (3)	Eeff/Leq w.r.t. durability	Additional characteristics
XF1 XC4	C30/37	330kg		CP (4)	0.50	RAG
XF3 XC4	C30/37	385kg	CEM I or CEM II/A or B (excpt W)	CP (4)	0.45	RAG G
XF4 XC4	C35/45	385kg	CEM I or CEM II/A (S or D)	PM or ES CP (4)	0.45	RAG G+S
XC4 XF1 XS1	C30/37	330kg		PM CP (4)	0.50	RAG
XC4 XF1 XS3	C35/45	350kg		PM CP (4)	0.50	RAG

Table 3.5. Normally imposed characteristics for composite bridge slab concrete

(1) This table only covers the most frequently applied exposure classes.
(2) Strength class quoted is a minimum class allocated only with respect to durability criteria.
(3) CP, PM and ES designate "cement for prestressed concrete", "seawater hardening" and "sulphate containing water" respectively.
(4) Cement quality required only for a prestressed concrete slab.

Concrete mix design for a performance-based approach

When implementing a performance-based approach, the Owner can leave the Contractor even greater freedom in his concrete mix designs as long as they achieve the durability targets laid down in the contract.

The AFGC 2004 guide entitled “Conception des bétons pour une durée de vie donnée des ouvrages” [concrete mix design for a given structural life] and the LCPC 2009 guide entitled "Approche performantielle de la durabilité des bétons - Applications aux ouvrages courants - Recommandations provisoires" [performance-based approach to concrete durability – applications to standard structures – provisional recommendations] suggest deciding on composite bridge slab concrete based on the following three durability indicators for reinforcement corrosion:

- porosity accessible to water by absorption in vacuo P_{water} expressed as a percentage and measured using the AFPC-AFREM procedure entitled "Détermination de la masse volumique apparente et de la porosité accessible à l’eau" [determining apparent density and porosity accessible to water],
- apparent permeability to gas K_{gas}, expressed as in 10^{-18} m² and measured using a constant load permeability meter (LPC test method No. 58.7),
- if the slab is subjected to de-icing salts or to a marine environment, the chloride diffusion coefficient D_{app}, expressed in m^2/s.

Electrical resistivity ρ, expressed in Ω.m, complements the above indicators.

Concrete design tests

The following table displays acceptability thresholds for these indicators at 90 days based on 100-year structural serviceability. These thresholds have been established for minimum concrete covers under environmental conditions $C_{min,dur}$ of 30 mm in relation to carbonation (XC) and of 50 mm in relation to chloride penetration depth (XD, XS). For other concrete covers, these thresholds should be adjusted in reference to the AFGC guide entitled “Conception des bétons pour une durée de vie donnée des ouvrages” [concrete mix design for a given structural life].

Exposure classes	Primary durability indicators w.r.t. reinforcement corrosions (thresholds at 90-day concrete age)		Secondary durability indicator w.r.t. reinforcement corrosion (threshold at 28-day concrete age)
	P_{eau}	K_{gaz}	D_{app}
XC4	$P_{eau\ 90j} < 13$ %	$K_{gaz\ 90j} < 150\ 10^{-18}$ m²	
XC4 + XS1	$P_{eau\ 90j} < 13$ %	$K_{gaz\ 90j} < 150\ 10^{-18}$ m²	$D_{app\ 90j} < 7$
XC4 + XS3	$P_{eau\ 90j} < 11$ %	$K_{gaz\ 90j} < 150\ 10^{-18}$ m²	$D_{app\ 90j} < 3$
XC4 + XD1	$P_{eau\ 90j} < 13$ %	$K_{gaz\ 90j} < 150\ 10^{-18}$ m²	$D_{app\ 90j} < 7$
XC4 + XD3	$P_{eau\ 90j} < 11$ %	$K_{gaz\ 90j} < 150\ 10^{-18}$ m²	$D_{app\ 90j} < 3$

Table 3.6. Thresholds for concrete durability indicators

When implementing this so-called performance-based approach, the concrete design test is invariably conducted in compliance with CCTG fascicule 65 requirements, but must also give measurement results for each 90-day durability indicator or provide convincing results for prequalified mix designs used previously under equivalent production and usage conditions.

28-day measurements of porosity to water and 28- and 90-day electrical resistivity measurements are also taken in view of concrete suitability tests or taken as reference values.

Concrete suitability tests

The concrete suitability test is conducted during the 4th week after concrete production, which is compatible with checking the material’s 28-day structural compressive strengths. It is also performed in compliance with CCTG fascicule 65 requirements, but includes measurements of accessible porosity to water and hardened concrete resistivity. For this, three additional 11 x 22 cm samples are taken, from which two 5 and 10 cm thick sections are cut for measuring the resistivity and porosity to water respectively.

The concrete suitability test is recognised as convincing if the following two conditions are confirmed:

- the accessible porosity to water (P_{water}) agrees with the value measured during concrete mix design, i.e. P_{water} (suitability) at 28 days < 1.1. P_{water} (design) at 28 days;
- the electrical resistivity ρ agrees with the value measured during concrete mix design, i.e. ρ(suitability) at 28 days > 0.8.ρ(design) at 28 days.

Concrete inspection tests

During construction, concrete compliance inspections foreseen under Clause 86.1 of CCTG fascicule 65 are complemented by durability indicator measurements specified at frequencies provided for in the contract. The concrete is declared in compliance if the following conditions are confirmed:

at 28 days:

- electrical resistivity (ρ): ρ(inspection) at 28 days > 0.8.ρ(design) at 28 days;
- accessible porosity to water (P_{water}): P_{water}(inspection) at 28 days < 1.1.P_{water}(design) at 28 days;

at 90 days:

- accessible porosity to water: P_{water}(inspection) at 90 days < P_{water}(contract-specified) at 90 days;
- gas permeability: K_{gas}(inspection) at 90 days < K_{gas}(contract-specified) at 90 days;
- chloride apparent diffusion coefficient: D_{app}(inspection) at 90 days < D_{app}(contract-specified) at 90 days.

In the event that one of the preceding conditions is not met, the contractor is required to undertake additional investigations.

In relation to frost and salt, if no durability indicators are available, the parameters to be measured are selected from the following:

- air bubble system spacing factor L_{bar},
- occluded air content t_{air} of fresh concrete,
- scaling E_c (scale mass under aggressive cycle),
- performance test for internal frost $\Delta\varepsilon$, strain measurement interlinked with measurement of $f^2/f0^2$ resonance frequencies.

The LCPC guide entitled "Recommandations pour la durabilité des bétons durcis soumis au gel" [recommendations for durability of hardened concrete subjected to frost] published in December 2003 quotes thresholds applicable to these indicators for different types of concrete from mix design to placement.

3.2 - Steelwork for twin girder composite bridges

3.2.1 - Main girder flanges

Material

Main girder flanges are most often made of S355N, M, NL or ML steel plate as defined by Standards NF EN 10025. S460 steel plate is also used sometimes.

General geometry

Flanges are horizontally rectilinear for a straight bridge and horizontally curved for a curved bridge. They are horizontal in the transverse direction.

Detailed geometry

Section 2 of this guide provides data on flange widths to be used with respect to deck width and span distances. These widths are almost always constant longitudinally, for the top flanges to facilitate slab formwork and, for the bottom flanges to facilitate lateral guidance of the steel frame during launching, if this installation method is adopted.

Flange thickness varies in relation to the deck cross section. The minimum thickness is close to 25 mm. The maximum thickness is determined by calculation, but rarely exceeds 150 mm for S355 steels and 100 mm for steels with higher elastic limits.

Flange thickness variations are usually integrated towards the web; for the top flange, to facilitate slab construction and, for the bottom flange, to facilitate steel frame launching (Figure 3.1). In crane-installed decks, the bottom flange thickness can be varied towards the bottom, which somewhat simplifies cutting of the web and welding of the bottom flange.

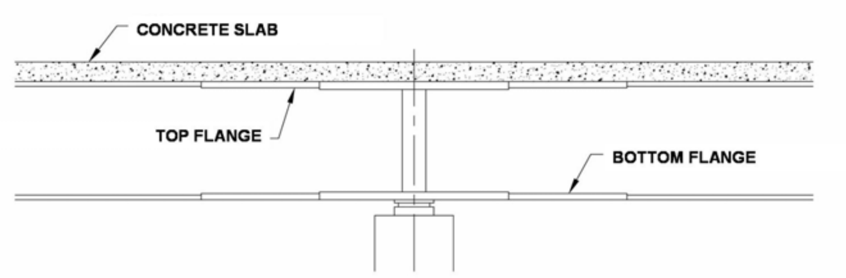


Figure 3.1. Usual direction of flange thickness variation

At changes of flange thickness, sudden variations can induce spurious bending moments and stress concentrations harmful to good structural behaviour and thus durability. Limiting the thickness variation of bottom flange plates to +50% and -33% is strongly recommended. These provisions can be relaxed for top flanges, which are restrained by the slab, as long as there is enough passive reinforcement to control cracking. All flange thickness variations must be materialised by gradual tapering in compliance with dimensional details shown in Figure 3.2. Standard NF EN 1090-2 specifies a 1/4 maximum transition slope.

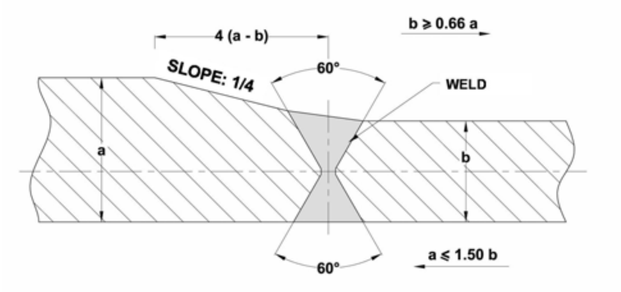


Figure 3.2. Detail of flange thickness variation

The number of flange thickness changes must be carefully chosen. Too few changes can lead to excessive steel consumption and too sudden variations in thickness. Too many changes will result in a high assembly cost.

Figure 3.3 provides guidelines for the desirable number of flange thickness changes in the central span of a constant depth deck integrating at least 3 spans with respect to its maximum span distance.

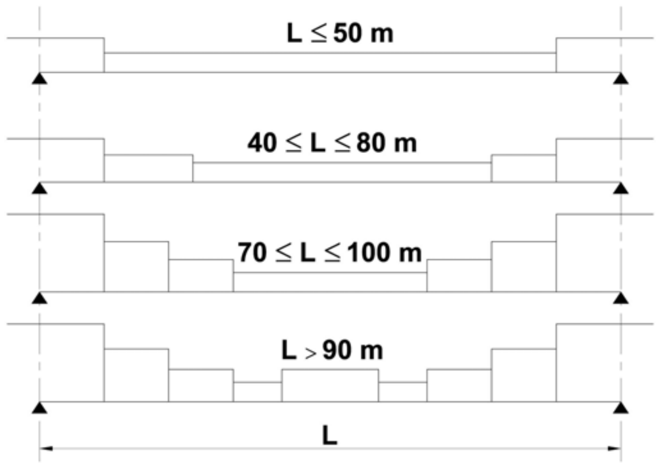


Figure 3.3. Guidelines to number of flange thickness changes in an intermediate span of a constant depth deck

For a constant depth, isostatic span, 2 to 3 flange thickness changes should be envisaged in relation to the span distance. The maximum thickness is reached at mid-span.

Flanges of continuously varying thickness

Continuously varying thickness flanges have been implemented in some bridges, but this provision, whilst excellent in terms of fatigue prevention, remains of marginal use for the following reasons:

- higher supply cost,
- plates non-reusable if structural steel distribution is changed.

Additional flanges

When the design leads to retaining flange thicknesses greater than the allowable maximum and neither girder depth nor flange width can be increased, additional plates may be used, i.e. secondary flanges welded to the main girder flanges.

Incorporation of additional flanges must remain exceptional and requires taking certain major precautions mostly designed to curtail risks of assembly fatigue between the two flanges and residual stresses due to weld shrinkage.

Figure 3.4 illustrates the majority of recommendations implemented in bridges built to old French regulations. It should be noted that the additional flanges are:

- 100 mm narrower than, and centred on, the main flanges,
- of minimum thickness equal to a maximum of 20 mm or 1/3 of the main flange thickness,
- without intermediate joints,
- tapered gradually at both ends, i.e. their thickness decreases gradually over a distance of at least 5 times their thickness,
- bevelled at both ends over a distance of at least their width.

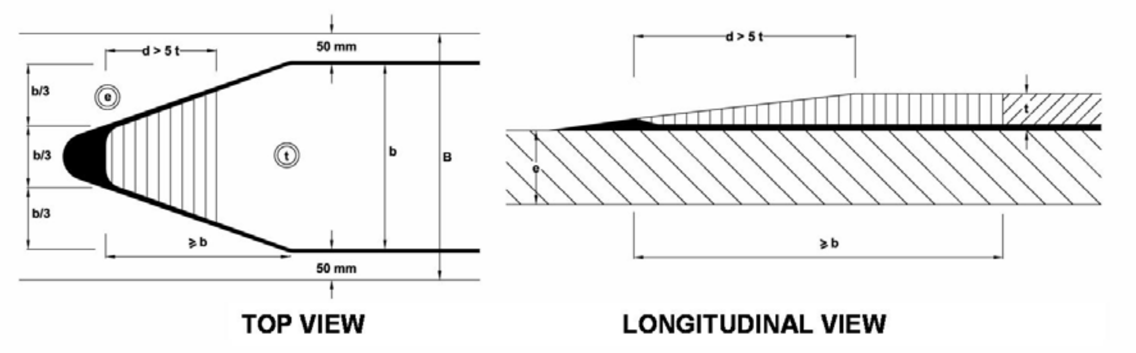


Figure 3.4. Former measures concerning additional flanges

Figure 3.5 illustrates the measures recommended by Eurocode 3.

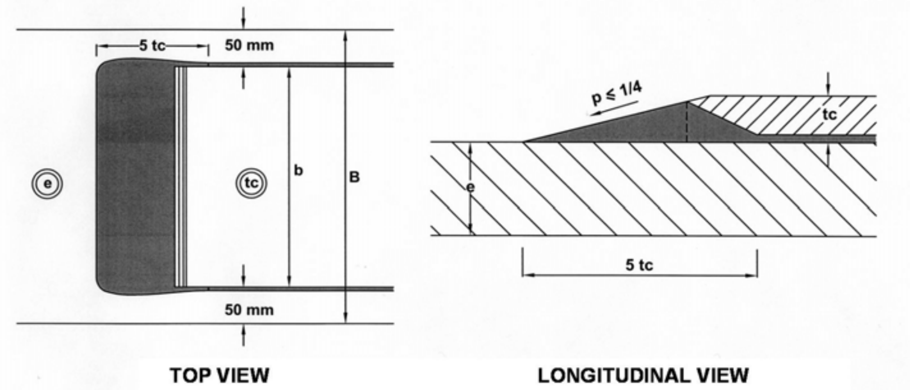


Figure 3.5. Measures concerning additional flanges recommendedby Eurocode 3

Design calculation checks

Main girder flanges must be checked based on information provided by Standards NF EN 1993-1-1, NF EN 1993-1-5, NF EN 1993-2, NF EN 1994-2 and their national appendices. This operation is commented and illustrated in Sections 8, 9.1, 10.1 et 10.2 of Part II of the Sétra Eurocodes 3 and 4 application guide.

3.2.2 - Main girder webs

Material

Webs and their ancillaries (cross-beam posts, directly supporting cross-beam posts, vertical and longitudinal stiffeners, etc.) are most often made of S355K2+N steel plate as defined by Standards NF EN 10025.

For deck depths less than 4.50 m, corresponding to the large majority of bridges, the web is cut from a single plate. For deeper decks, the web must be fabricated from 2 longitudinally welded plates, as in the case of the Jassans bridge (5 m maximum depth at piers).

General geometry

Webs are flat plates on a straight bridge. If the deck is horizontally curved, the webs are curved by tack welding when the flanges are welded.

Webs are cut based on longitudinal profile, cambers to be implemented and variations in flange thickness, in generally.

Detailed geometry

Web thickness varies in relation to deck cross sections. The recommended minimum thickness is 14 to 16 mm, which limits deformations due to transverse beam and stiffener welding to an aesthetically acceptable level. Maximum web thickness is determined by calculation, but rarely exceeds 30 to 35 mm.

Web thicknesses vary much less than flange thicknesses because the former variation are usually limited to between 2 and 5 mm. When webs vary by less than 4 mm, no grinding down is required; the weld ensures an acceptable thickness transition. On the other hand, maching to a 1/4 slope is desirable for web thickness variations exceeding 5 mm. These variations are generally symmetrical on each side of the web centreline.

Design calculation checks

Main girder webs must be checked based on information provided by Standards NF EN 1993-1-1, NF EN 1993-1-5, NF EN 1994-2 and their national appendices. This operation is commented and illustrated in Sections 8.2 and 10.3 of Part II of the Sétra Eurocodes 3 and 4 application guide.

3.2.3 - Cross-beams

Materials

Standard cross-beams are most often standard structural sections made of S355K2+N steel. In compliance with article 4 of Fascicule 4 Title III of CCTG, cross-beams must bear the “NF-Acier” quality mark in common with all plate required for a bridge steel frame. Standard sections may be subject to large rolling tolerances, so all sections should preferably be supplied from the same rolling mill production batch for gusset plate welding purposes (c.f. “Gusset plates and gussets” section below). Standard cross-beams are sometimes made of built-up welded sections, for example when design standard sections are unavailable at short notice.

Cross-beams at supports, which are larger, are almost always built-up welded sections made of S355K2+N steel.

Standard cross-beams

Standard cross-beams for twin girder composite bridges are usually 400 to 700 mm deep IPE or HEA sections, depending on the depth and centre-to-centre distance of the main girders.

Cross-beams in pier areas

Pier area cross-beams are subjected to much higher stresses than standard cross-beams. They are effectively required to resist the wind loads exerted on the deck, prevent lateral torsional buckling of the bottom flanges, which are heavily compressed by longitudinal bending, and take up the loads induced by operations involving jacking at locations not directly beneath the main girders.

For these reasons, cross-beams at piers are commonly 600 to 1600 mm deep built-up welded sections, depending on the bridge span distance (Case No. 1, Figure 3.6).

An alternative solution comprises integrating directly supporting cross-beams without cantilevers at the piers (c.f. section entitled "Directly supporting cross-beams" below). This solution is advantageous for a bridge located in a strongly seismic zone, in which pier horizontal dimensions prevent jacking beneath the main girders. However, this can hamper progress of the mobile formwork very frequently used for deck construction (Case No. 2, Figure 3.6) and so this solution is rarely adopted.

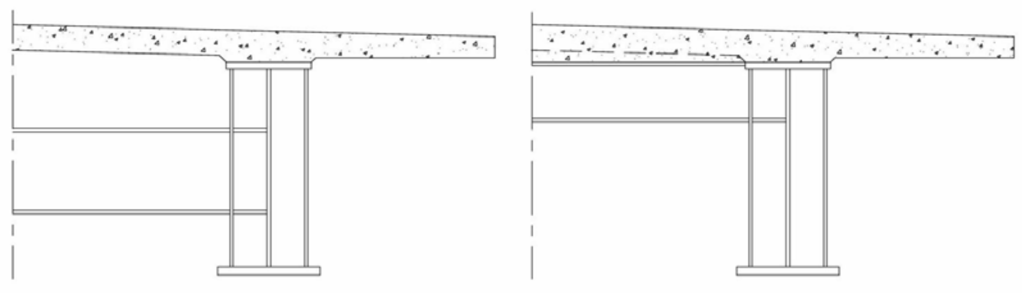


Figure 3.6. Different possible cross-beams at piers

Moreover, the fairly severe conditions governing prevention of girder lateral torsional buckling introduced by Eurocode 3 can lead to strengthening of standard cross-beams on either side of the cross-beams at piers. The former cross-beams can be strengthened by reducing their centre-to-centre distance as they approach the piers or by increasing their rigidity. The cross-beam system can also be braced by lowering the standard section and by complementing it with two diagonals connected, at the bottom, to the cross-beam top flange and, at the top, to the main girder top flanges. However, the current embodiment of the latter solution (Figure 3.7), described in Sub-section 8.6.7 of Part II of the Sétra Eurocodes 3 and 4 application guide, has the drawback of hampering mobile formwork progress.

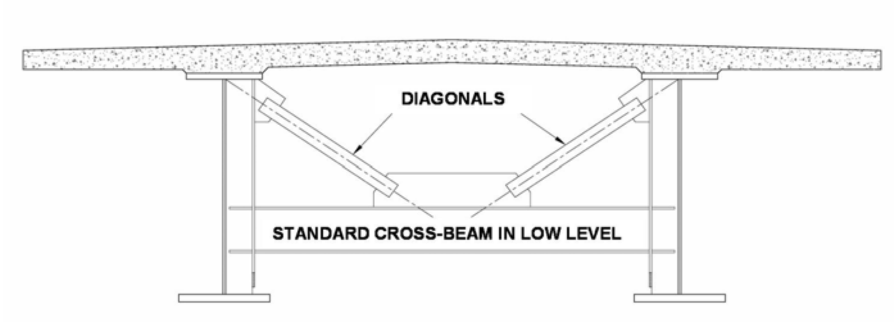


Figure 3.7. Low-level standard cross-beam braced by 2 diagonals

Cross-beam centre-to-centre distances

Cross-beams in each span are generally spaced at a constant centre-to-centre distance of no more than 6 to 8 m. As stated above, it may sometimes be necessary to reduce this centre-to-centre distance near a pier to prevent lateral torsional buckling of the main girder compressed bottom flanges in areas subjected to high negative bending moments. It should be noted that a local reduction of cross-beam centre-to-centre distance does not increase slab construction complexity.

Cross-beam vertical geometry

In elevation, standard cross-beams can be arranged vertically or radially, i.e. everywhere perpendicular to the longitudinal profile. Cross-beams at supports must be vertically positioned.

Cross-beam horizontal geometry

Standard cross-beams are horizontally arranged perpendicular to the webs in a square structure and radially in a curved structure.

In moderately skew (> 70 gr) bridges, all cross-beams are skew (Case No. 1, Figure 3.8).

A special design study must be conducted for very skew (< 70 gr) bridges. The above solution effectively leads to long, thus highly flexible, cross-beams and rather complex assemblies. An alternative solution involves opting for a transverse framework comprising square standard cross-beams and skew cross-beams at supports (Case No. 2, Figure 3.8), although other solutions can be envisaged for the cross-beams at supports.

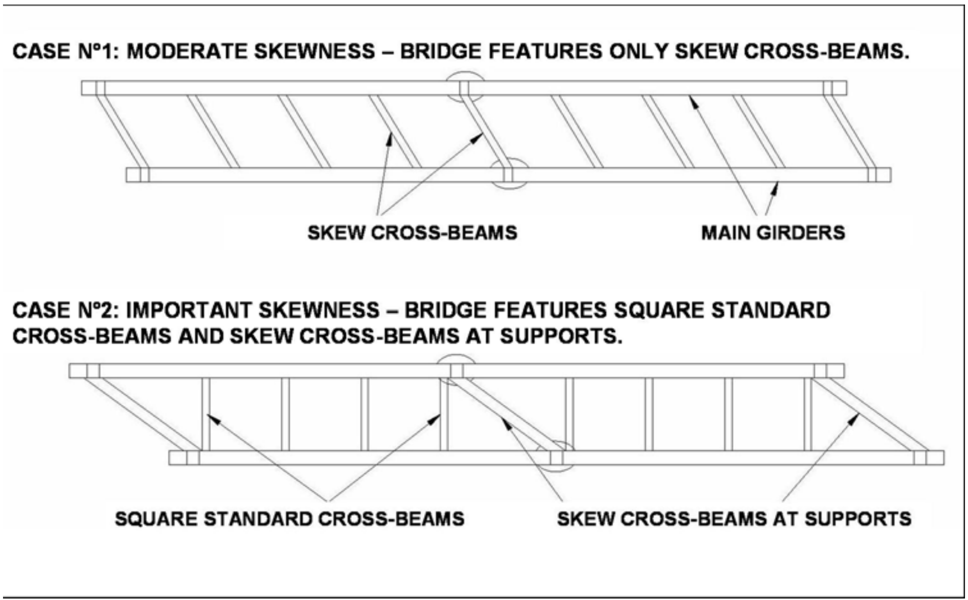


Figure 3.8. Cross-beam arrangement principle for twin girder skew decks

In the latter case, it is imperative to ensure the strength of standard cross-beams, if the steel frame is to be installed by launching. During this dynamic stage, deck skewness may cause the two girder cross sections, to which a standard cross-beam is welded, to be subjected to significantly different vertical deflections, thereby generating a high bending moment in the cross-beam. The problem is identical during operation.

Finally, when deck skewness is pronounced, the main girder cambers must be considered, when cutting the cross-beams, to ensure web verticality during operation.

Transverse geometry

Symmetry dictates horizontal cross-beams in the transverse direction for decks featuring bidirectional banking. They are also very often horizontal in a structure featuring unidirectional banking because this provision creates the simplest fabricated assemblies (c.f. Case No.1, Figure 3.9). However, in shallow decks or in those supporting a highly banked road, the cross-beams are sometimes positioned parallel to the deck slope (c.f. Case No.2, Figure 3.9) to give the required space above them for:

- repainting (a height essentially equal to the width of the cross-beam flange, i.e. approximately 30 cm, is usually considered essential for painting operations),
- moving the central shuttering platform, whose height may be considered to be approximately 50 cm, if the slab is to be built using moving formwork (Section 5).

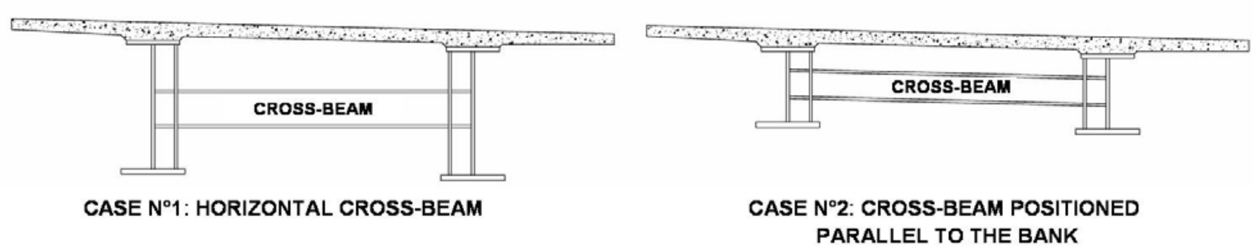


Figure 3.9. Cross-beam transverse orientation on a bridge with deck unidirectional banking

Vertical positioning

In general, maximum transverse rigidity is ensured when the cross-beams are positioned at girder mid-depth or even slightly below this level.

For bridges, whose span distance is less than 40 m or so, usually of constant deck depth, the cross-beams are all positioned at the same height on the girders to give the space required for displacing the moving formwork.

On variable depth structures, the standard cross-beams must be positioned lower and lower as the high-depth deck areas are approached to effectively counteract lateral torsional buckling of the main girder bottom flanges.

Posts linking main girders and cross-beams

Bridge deck main girders and cross-beams are linked by T-sections called cross-beam posts, which are welded to the girder webs (Figure 3.10). These tees are usually built-up welded sections, but standard half-sections are sometimes used on small bridge decks.

The cross-beam posts are welded to the girder webs based on the detail shown in Figure 3.10. At the top, the post flange must be welded to the main girder top flange to ensure its resistance against transverse bending loads. Conversely, at the bottom, the post flange must not be welded to the main girder bottom flange and must therefore be gradually terminated to curtail fatigue risks. Figure 3.10 illustrates the cut-down V shape of the crossbeam post flange used when these posts are built-up welded sections. On the other hand, when cross-beam posts are made from standard T-sections, their flange thickness is gradually reduced before narrowing down their webs.

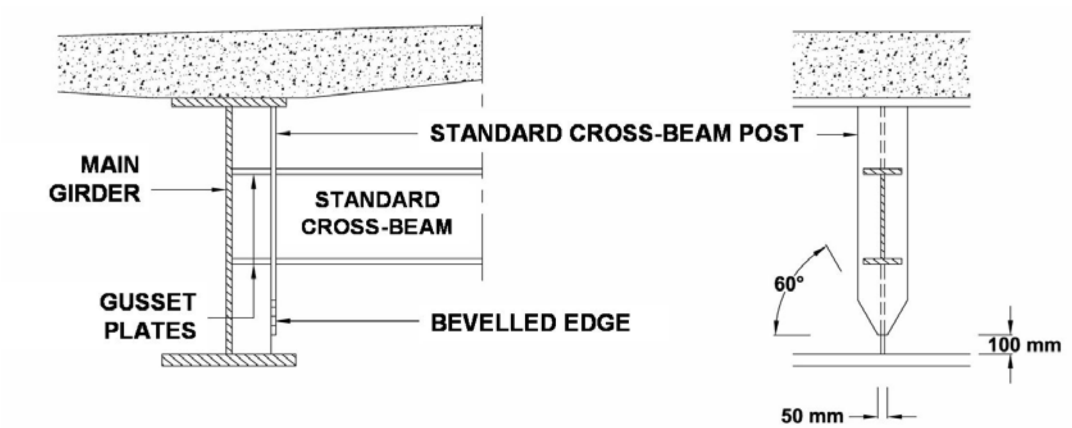


Figure 3.10. Standard cross-beam/main girder connection

To allow travelling platform to travel along the main girder bottom flanges, we strongly recommend positioning the cross-beam post flanges at least 100 mm from the edge of the main girder bottom flanges (Figure 3.11).

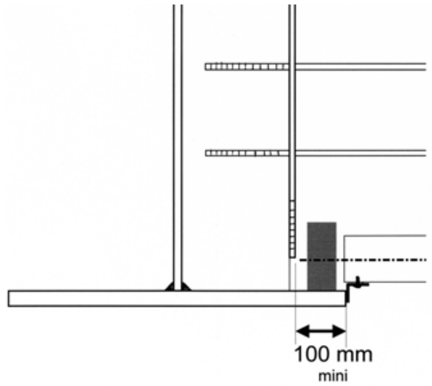


Figure 3.11. Minimum clearance to be ensured by cross-beam posts

Cross-beams at supports are also linked to the main girders by cross-beam posts, but in this case, the bottoms of the post flanges are welded to the main girder bottom flange. Deck-support bearing loads must be properly transferred and fatigue stresses are lower over the deck supports.

When the main girder bottom flanges feature a high longitudinal slope, either because of the bridge longitudinal profile or because the deck depth varies, we recommend cutting out a quarter round from the bottom of the cross-beam post web on the side it is welded to the main girder web. This half-moon cut-out effectively prevents rainwater stagnation and dirt accumulation upstream of the cross-beam posts.

Gusset plates and gussets

Loads are most often transferred between the cross-beam flanges and the main beams by gusset plates welded in extension of the cross-beam flanges.

These gusset plates can be triangular or rectangular for a twin girder cross-beam composite deck.

Triangular gusset plates are welded only on two sides (cross-beam post web and flange: Case No.1, Figure 3.12); this effectively limits assembly restraint. This type of gusset plate is only used when no other member can be fixed to it and the weld seam is long enough to transfer the load applied by the cross-beam flange.

Rectangular gusset plates are welded on three sides (Case No.2, Figure 3.12), when they are used for fixing temporary cross-bracing members.

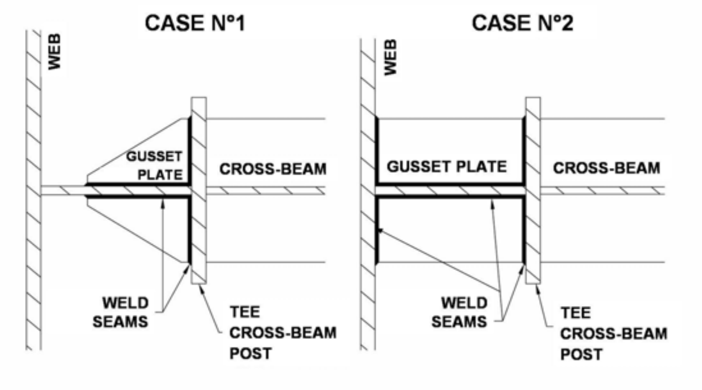


Figure 3.12. Horizontal cross-beam gusset plates

In the very common case of standard structural section cross-beams, the geometrical tolerances in centre-to-centre distance between the flanges of these sections are high, so gusset plates thicker than the cross-beam flanges should be adopted and they should, if possible, be positioned with respect to the actual centre-to-centre distance of the cross-beam flanges.

For a twin girder composite deck, vertical triangular gusset plates can represent an alternative to horizontal gusset plates (Figure 3.13). This arrangement is a little better from the structural standpoint and it overcomes the tolerance problems described above. However, this solution does have drawbacks: the gussets, shop-welded to the cross-beams, obstruct their handling and storage. The linear length of site welding is slightly greater and temporary cross-bracings cannot be fixed to the gusset plates. Finally, the top gussets can hamper displacement of the central shuttering platform, when the main girder depth is small and the slab is built on moving formwork.

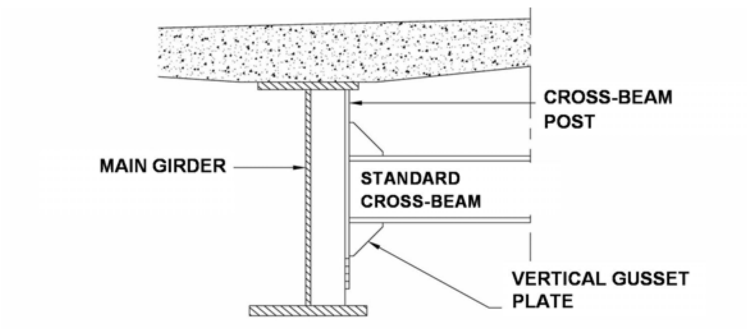


Figure 3.13. Vertical cross-beam gussets

In the case of bridge deck featuring more than two girders, gusset plates on the intermediate girder cross-beam posts must be rectangular and welded on three sides to ensure effective load transfer between the flanges of the different cross-beams.

Special cases of skew cross-beam posts and gusset plates

Cross-beam posts designed as in Case No.1, Figure 3.14 below can be adopted, when the standard cross-beams are positioned according to a skew deck and this skewness is moderate.

When the deck is highly skew, the welder and subsequently the painter may encounter problems in their operations on the closed angle side of the cross-beam post tees. In this case, it is preferable to opt for posts designed as in Case No.2, Figure 3.14. This case is more expensive insofar as it requires more angled welds than Case No.1 and skew cutting of the cross-beams, but it ensures more working clearance.

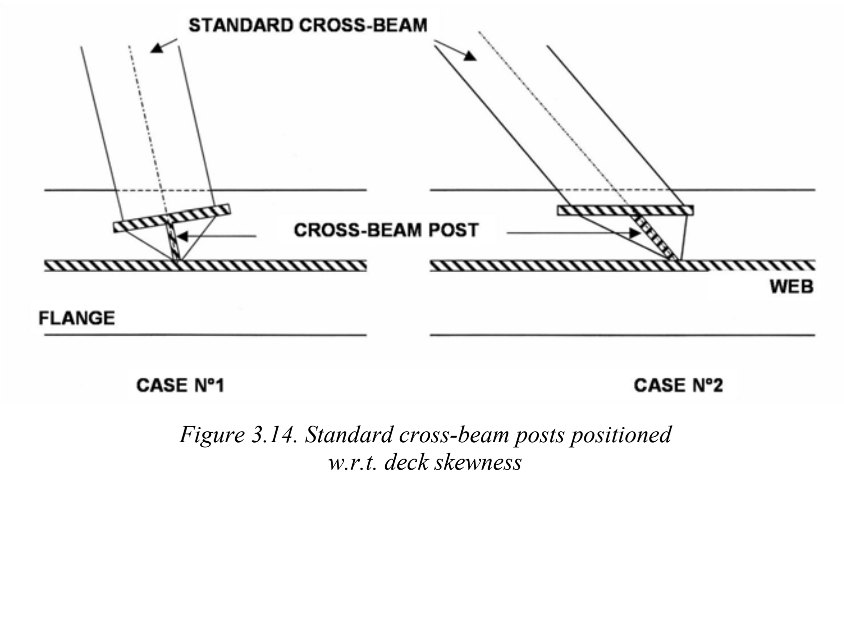


Figure 3.14. Standard cross-beam posts positioned w.r.t. deck skewness

Design calculation checks

Cross-beams should be checked based on information provided by Standard NF EN 1993-2 and its national appendix. Part of the check calculations is shown in Section 8.6 of Part II of the Sétra Eurocodes 3 and 4 application guide.

3.2.4 - Directly supporting cross-beams

Material

Directly supporting cross-beams are most frequently built-up welded sections made of S355K2+N steel as defined by Standards NF EN 10025.

Centre-to-centre distance

If the deck slab is to be built using mobile formwork, the directly supporting cross-beams must be spaced at a constant centre-to-centre distance, even if this means slightly displacing certain supports. The centre-to-centre distance is often taken as 4 m, but can be anywhere between 3.50 and 4.50 m.

If the bridge deck is to be built using pre-slabs or precast segments, a constant centre-to-centre distance is still recommended, but there is no major engineering problem in adopting two slightly different centre-to-centre distances (e.g. one for standards spans and one for end spans).

Horizontal and vertical geometries

In elevation, the directly supporting cross-beams must preferably be positioned in planes perpendicular to the longitudinal profile to facilitate welding of their top flanges to those of the main girders.

In plan, the directly supporting cross-beams are perpendicular to the main girder on a straight bridge deck and radial on a curved bridge deck.

Provisions identical to those recommended for cross-beam structures may be retained in relation to skew bridges. For very skew structures, combining skew and square directly supporting cross-beams leads to complex shaped slab segments near the supports, making the use of mobile formwork rather uneconomical (Section 5).

Transverse geometry

If the deck is unidirectionally banked, a cross section would show the directly supporting cross-beams positioned parallel to the banking (Case No.1, Figure 3.15).

If the deck is bidirectionally banked, the directly supporting cross-beams are most often essentially horizontal with a variable depth central section (Case No.2, Figure 3.15). We sometimes encounter bridges, on which the central section of the directly supporting cross-beams is of constant depth. This design requires variable depth concrete haunching, which is frequently tedious to reinforce and form, at each directly supporting cross-beam.

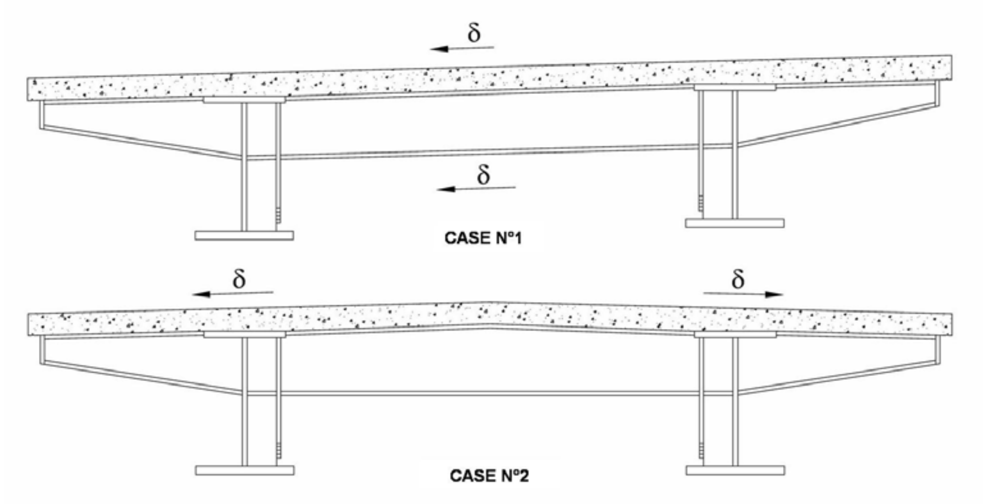


Figure 3.15. Influence of deck banking on directly supporting cross-beam shape

Directly supporting cross-beam cantilevers

Today, the vast majority of directly supporting cross-beam composite bridges feature lateral cantilevers.

Figure 3.16 below shows different cantilever types of equal, shorter or longer length than that of the slab overhangs (cantilevered sections).

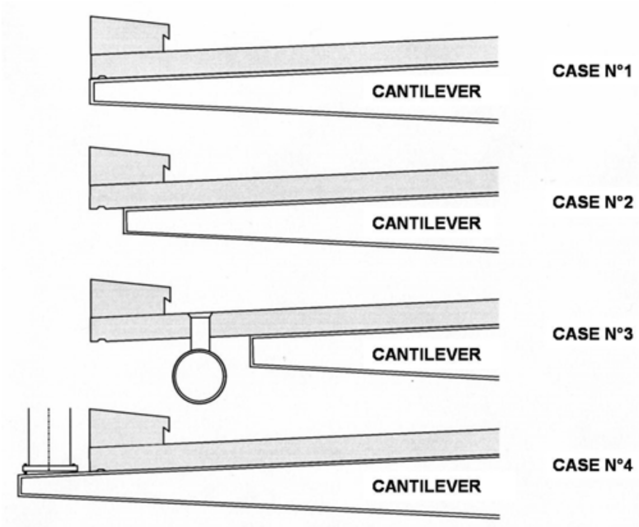


Figure 3.16. Directly supporting cross-beam cantilever lengths

Case No.1 is the most common. The cantilevers are the same length as the overhanging slab sections.

Case No.2 is also relatively common. The cantilevers are slightly shorter than the slab overhangs, facilitating their repainting and preserving continuity of the drips, which protect the underside of the slab. In cases in which the cantilevers do not extend beneath the road restraint systems, this arrangement also prevents congestion of the slab lateral areas – already heavily reinforced – by cantilever studs.

Case No.3 is fairly uncommon; it features cantilevers that are significantly shorter than the slab overhangs. This design is most often used to allow integration of services (rainwater drains in this case) as near as possible to the slab. This arrangement makes slab steelfixing and forming more complex.

Case No.4 is also fairly rare; it features cantilevers that extend beyond the slab overhangs, usually for fixing special equipment such as lighting columns, noise screens or open drains. This arrangement, which can be designed not to require a wider slab throughout its length and for architectural reasons, includes sections unprotected from the rain and so requires particularly careful anti-corrosion protection.

Furthermore, it should be noted that Figure 3.16 never shows a stringer at the free end of the cantilevers. This additional member, requiring numerous welds on site, has been virtually abandoned today.

Detailed geometry

Directly supporting cross-beam depth between main girder webs is basically 1/11th of the girder centre-to-centre distance. In the cantilevers, this depth generally varies between the depth of the above central area and a minimum value usually around 300 mm. Cross-beam flange width can also vary in the cantilevers.

Directly supporting cross-beams are made of steel plate, which is approximately 12 mm thick for the webs and 20 to 25 mm thick for the flanges. These thicknesses remain constant throughout the length of the cross-beams.

Integration of a vertical rectangular plate is recommended at the end of each cantilever. This forms an encastré end in torsion and curtails the risk of lateral torsional buckling of the bottom flange, which is maintained in compression. This end plate also facilitates fixing of mobile formwork lateral sections, when the latter equipment is to be used for slab construction; Finally, these end plates can facilitate fixing and stabilisation of steel drainage cornices, if the deck features these ancillaries.

Directly supporting cross-beams at piers

For short span bridges, directly supporting cross-beams at piers can be the same depth as the standard directly supporting cross-beams, but will incorporate larger flanges.

For longer span bridges, directly supporting cross-beams at piers generally feature a central section deeper than the standard directly supporting cross-beams and cantilevers, which are identical to the standard directly supporting cross-beam cantilevers (Figure 3.17).

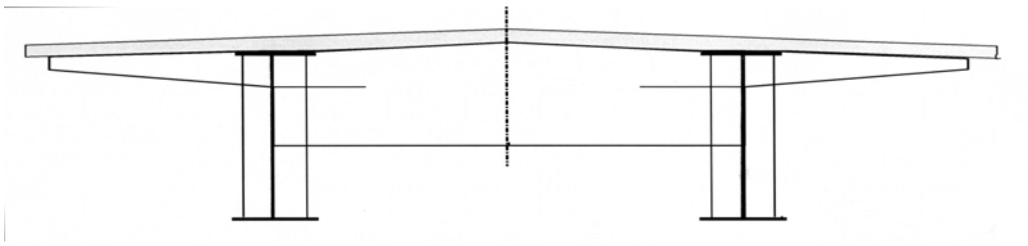


Figure 3.17. Directly supporting cross-beams at piers

Directly supporting cross-beams on either side of those at piers

In long span bridges, the very deep main girders and buckling prevention conditions introduced by Eurocode 3 can lead to strengthening of the directly supporting cross-beams on either side of those at the piers. Near the intermediate supports, either very deep directly supporting cross-beams or additional cross-beams at the bottom of the main girders (Case No.1, Figure 3.18) or gusset-strengthened standard directly supporting cross-beams (Case No.2, Figure 3.18) should be incorporated.

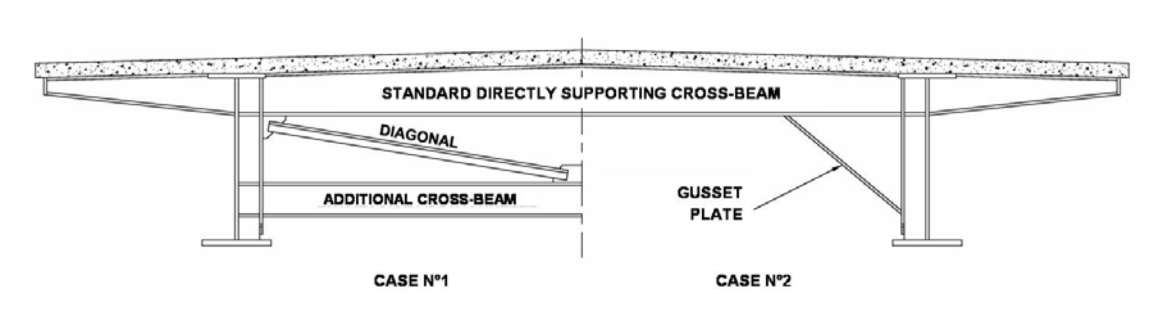


Figure 3.18. Directly supporting cross-beams in either side of those at piers

Connection with main girders

When the main girders are significantly deeper than the directly supporting cross-beams, both beam families are linked fairly similarly (Figure 3.19) to a main girder and cross-beam assembly. The top flange of a directly supporting cross-beam is invariably welded straight onto the main girder top flange. Moreover, gusset plates for directly supporting cross-beams must be rectangular to ensure proper load transfer between the bottom flanges of both the cantilevers and the central sections of the directly supporting cross-beams.

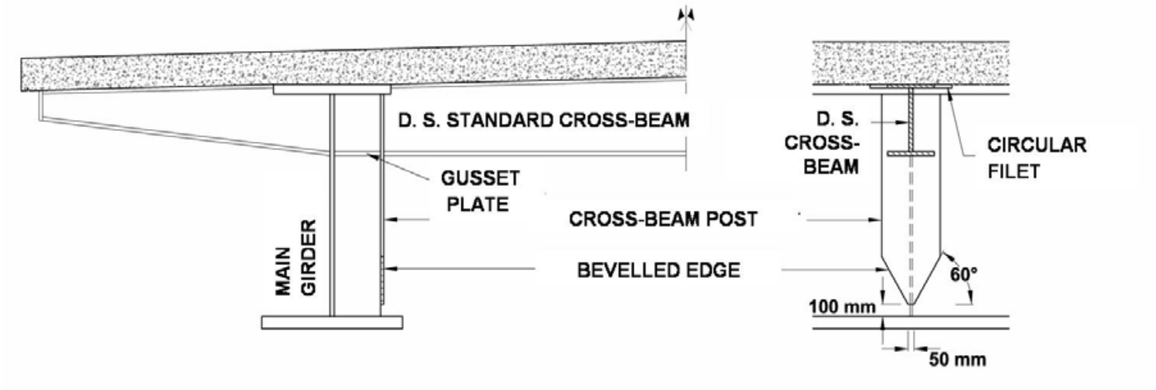


Figure 3.19. Main girder / Directly supporting cross-beam connection (standard case)

When the directly supporting cross-beam and main girder depths are the same, the bottom flange is welded straight onto the main girder bottom flanges. No post flange is then required (Figure 3.20), unless jacking operations are necessary.

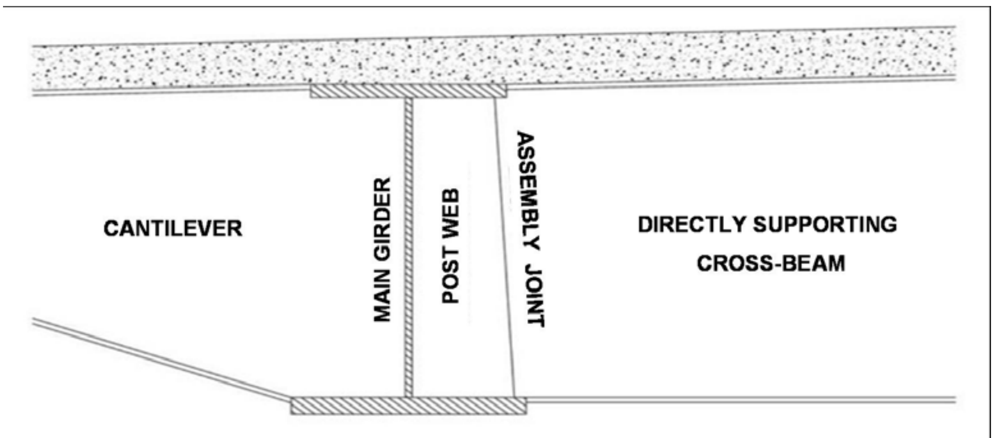


Figure 3.20. Main girder / Directly supporting cross-beam connection (special case)

Whatever the case, we strongly recommend incorporating inter-penetrating welds at the directly supporting cross-beam/main girder junction. These welds are difficult to inspect because of the slab and the consequences of their fracture are extremely serious.

Finally, incorporation of circular fillets between the directly supporting cross-beam and main girder top flanges ensures a stronger assembly and greater fatigue resistance (Figure 3.21). This constraining, expensive measure should not be systematically specified and can be limited to only areas determined by calculation.

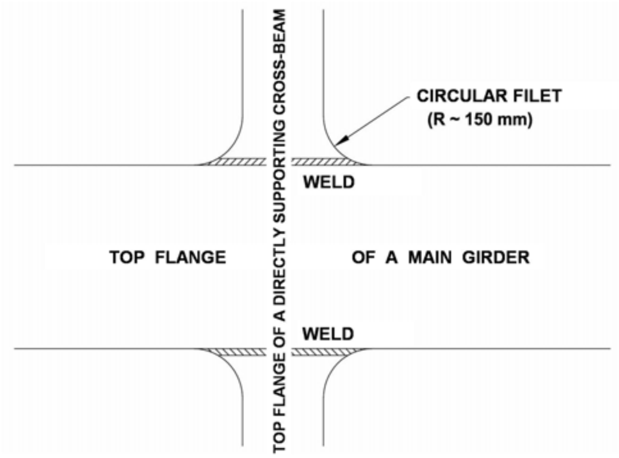


Figure 3.21. Assembly incorporating circular fillets connecting top flange of a directly supporting cross-beam with cantilevers to a main girder top flange

Design calculation checks

Directly supporting cross-beams must be checked in the same way as the bridge main girders, albeit considering stresses generated by transverse bending. Cantilevers are usually dimensioning.

3.2.5 - Secondary stiffeners

With a view to simplifying the steel frame, secondary stiffeners should not systematically be used and should respond to clearly identified stability problems.

Vertical stiffeners

Vertical stiffeners are the directly supporting cross-beam posts in a structure integrating this type of cross-beam.

Vertical stiffeners are sometimes designed at mid-distance between two successive cross-beam posts in a cross-beam structure. These stiffeners are then either flat bars welded on 3 sides or tees welded similarly to the standard cross-beams.

Longitudinal stiffeners

Longitudinal stiffeners are often provided on the webs to prevent local buckling during construction or operation.

These stiffeners are usually flat bars welded roughly at the web lower third point near the piers. They are not welded onto the cross-beam posts and their ends are bevelled (Figure 3.22). Their thickness is generally the same as that of the web plates they stiffen and their width is 10 to 12 x their thickness.

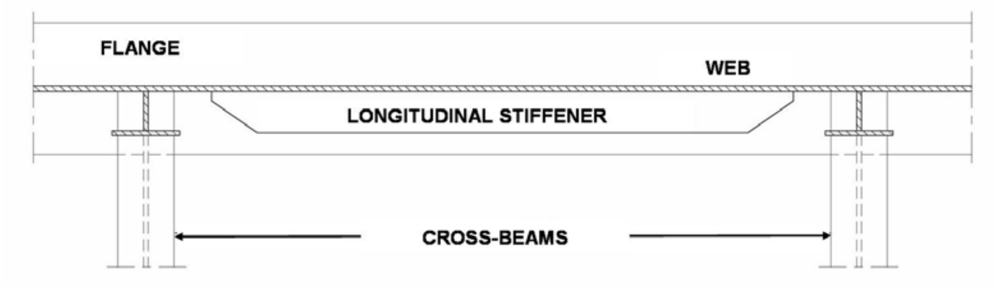


Figure 3.22. Plan view of horizontal stiffeners

The advantage of adding longitudinal stiffeners to a main girder should be carefully investigated, the extra cost of their implementation must be compensated by a reduction in web thickness. In practice, these stiffeners are particularly advantageous in structures, whose main girder webs are very slender, i.e. large span and narrow bridges, as well as those built using high strength steel.

3.2.6 - Pier bearing areas

Position of jacking locations

Deck jacking locations, usually along the main girder axes or either side of the support bearings, are provided beneath the steel frame for changing these support bearing (Figure 3.23).

Another arrangement is to incorporate these jacking locations between the main girders, beneath the pier cross-beam or directly supporting cross-beam (Figure 3.24). This arrangement is only used for standard bridges and often requires cross-beam strengthening, but this extra provision is compensated by the possibility of designing slightly narrower piers.

Support and jacking posts

The main girders should be stiffened at both their support bearings and jacking locations, if need be.

At the support bearings, the main girder internal face is stiffened by the T-section post used to fix the pier cross-beam or directly supporting cross-beam and its external face is stiffened by an additional vertical section. The latter is most often a T-section, but cases are encountered, in which this support post is a bucklet.

At the jacking locations, the main girders are stiffened by symmetrical vertical bucklet, T or simple flat bar sections, which are smaller than the sections stiffening the webs at the support bearings.

Figure 3.23 illustrates a twin girder crossbeam composite bridge pier area stiffened by both T-sections at the support bearings and bucklets at the jacking locations beneath the main girder webs.

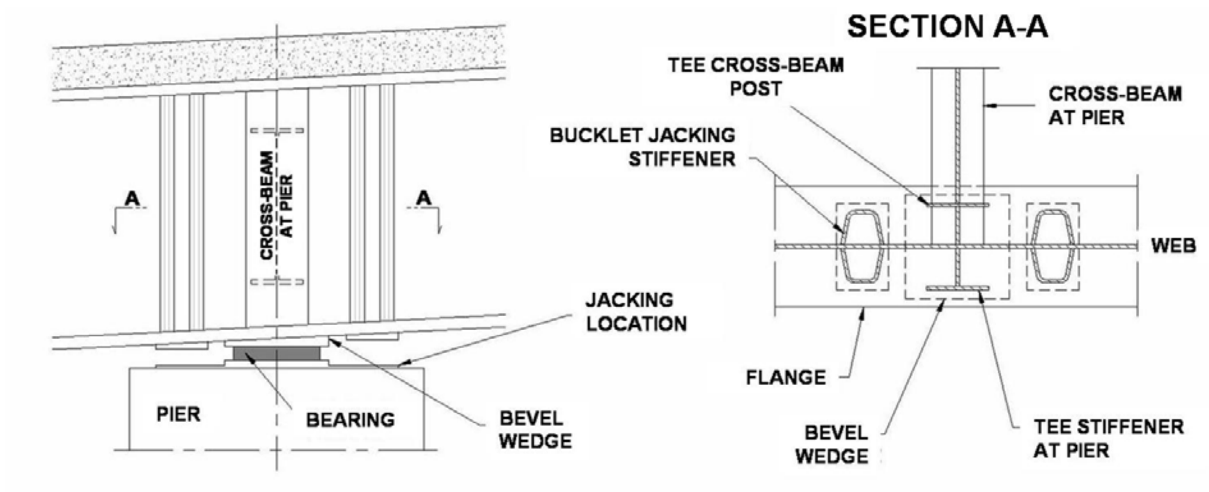


Figure 3.23. Pier area stiffening / Example 1

Figure 3.24 illustrates a twin girder directly supporting cross-beam composite bridge pier area stiffened by both T-sections at the support bearings and flat bar sections at the jacking locations beneath the directly supporting cross-beam.

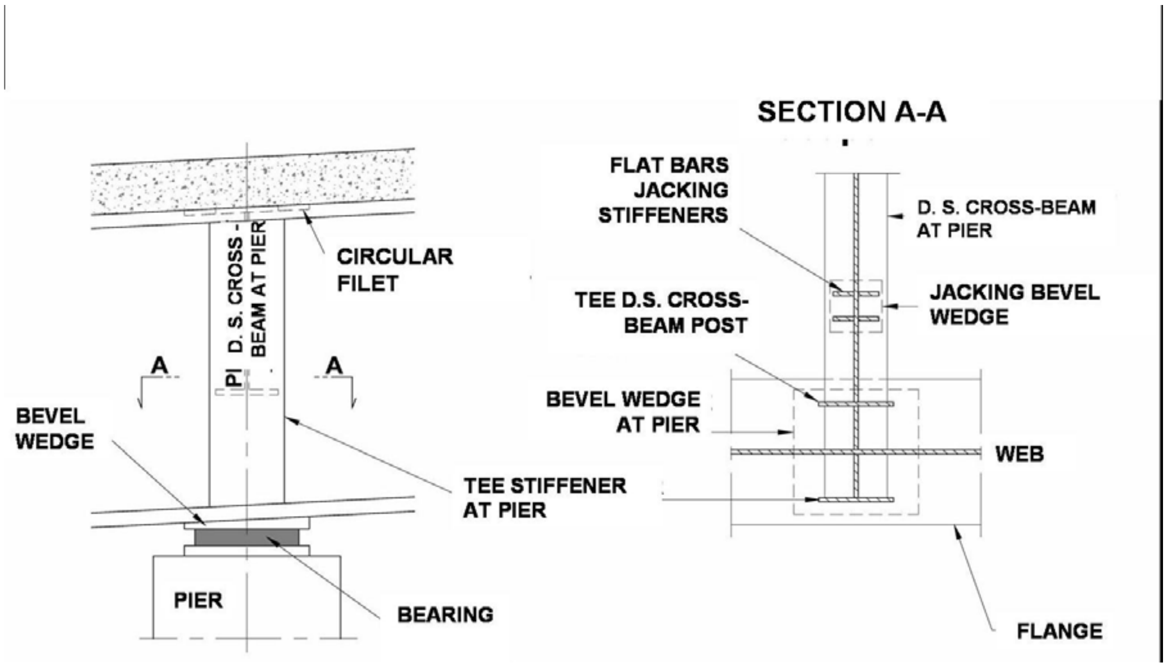


Figure 3.24. Pier area stiffening / Example 2

T-section support posts are sometimes used and complemented by two welded side plates to form a closed section. This design prevents rainwater and dirt accumulation at the bottom of the support posts and gives the posts a more aesthetic appearance. On the other hand, it makes inspection of the vertical welds between the T-section support posts and the main girder webs much more complex.

Gusset plates

When the cross-beam at support is very heavily stressed, e.g. when the jacking locations are beneath this member, it may be necessary to extend the gusset plates between the webs and the external support posts. The latter members are compulsorily T-sections in this case.

Bevel wedges

Bevel wedges are rectangular plates, whose thickness varies longitudinally and sometimes transversely, welded beneath the main beam bottom flanges at the support bearings and jacking locations (Figures 3.23 and 3.24). Despite the road longitudinal profile and construction imperfections, the bottom faces of these bevel wedges must be perfectly horizontal because it forms the contact surface with the support bearings and the jacks.

Bevel wedges are generally fabricated by grinding plates of the same steel grade and quality as the main beam bottom flanges. Their minimum thickness is of the order of 20 mm, but if pot-type support bearings have to be bolted to the wedges, it must be such that at least 40 mm is available at the support bearings. Their maximum thickness depends not only on the structure’s longitudinal profile and the wedge overall length, but also on the steel selection conditions.

When the jacking locations are beneath the main girder webs and the longitudinal profile is not too definite, a single large bevel wedge covering both the support bearing area and the jacking location areas can be provided. If not, independent bevel wedges for the support bearing and the jacking locations can be designed.

Problems specific to variable depth bridge decks

On variable depth bridge decks, main girder depth variation should not start at the supports to avoid creating a jagged point in the longitudinal profile. The most common arrangement involves a constant depth area over the pier of essentially the same length as the pier head width (Figure 3.25). This arrangement simplifies cutting of the bearing area stiffeners and machining of the bevel wedges.

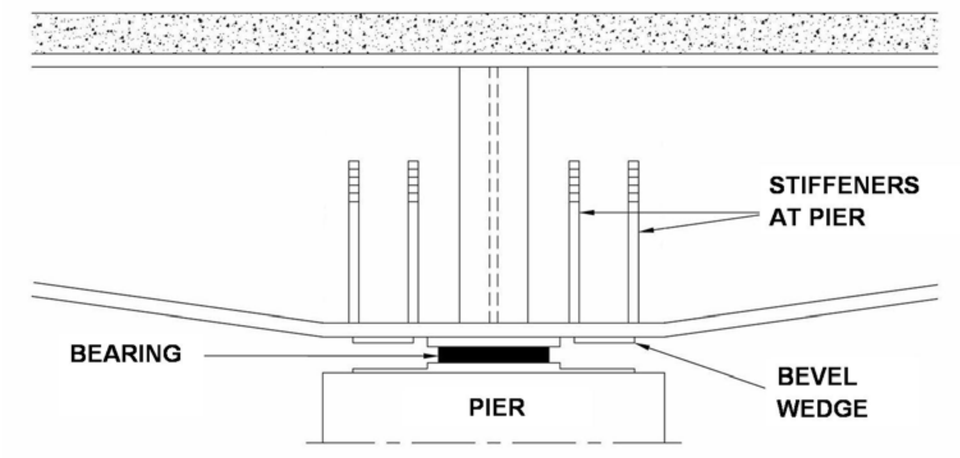


Figure 3.25. Bearing area for a variable depth deck

3.2.7 - Abutment bearing areas zones

Butt ends

In common with all bridges, composite decks feature an additional length called a butt end (Figure 3.26). This is usually 50 cm for standard structures, but can be up to 1 m for large structures.

Support and jacking posts, bevel wedges

Main girder areas at the abutment support bearings are provided with support and jacking posts, and bevel wedges in much the same way as pier areas. Jacking is nevertheless designed only on the span side, when it is performed beneath the main girder webs.

Slab strengthening at pavement expansion joints

Composite bridge slabs are relatively thin, so they often require significant strengthening at pavement expansion joints to permit satisfactory joint anchorage and greater slab resistance to the dynamic effects of trucks. A concrete minimum depth of 35 - 40 cm is therefore necessary throughout the joint slab width.

Several solutions are possible and commonly implemented for twin girder cross-beam composite structures.

When the standard slab thickness varies transversely, which is the most common cases, its maximum thickness at the main girders is often 35 – 40 cm. Under these conditions, this thickness can be simply generalised throughout the slab width over a 1 m length to give a generally satisfactory anchoring beam for the pavement expansion joint (Case No.1, Figure 3.26).

When the slab thickness obtained by this method remains insufficient, the depth of the main girders can be slightly reduced until the concrete depth necessary for joint anchorage is achieved (Case No.2, Figure 3.26).

A twin girder cross-beam bridge slab can be further stiffened by substituting the cross-beams at the abutments with directly supporting cross-beams, usually with cantilevers (Case No.3, Figure 3.26). We recommend this solution.

On a directly supporting cross-beam composite bridge, the standard slab is essentially of constant thickness (25 cm). A 35 – 40 cm thickness can therefore only be obtained by creating a concrete downstand behind the butt end of the deck and extending across its full width (Case No.4, Figure 3.26).

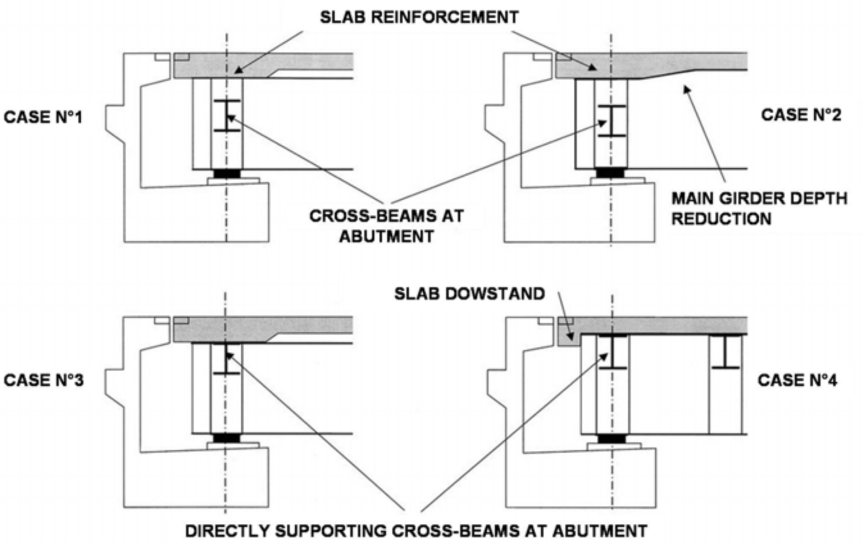


Figure 3.26. Slab strengthening at road expansion joint

Whatever the strengthening option retained, the concrete slab must extend 20 – 30 cm beyond the end of the steel frame to allow installation of a water collection channel accurately centred beneath the expansion joint.

3.2.8 Steel frame – concrete slab connection

Material and type

In the 1980s, a composite bridge steel frame and concrete slab was most often connected by sections of angle or studs with heads. Hoop connectors were sometimes used (Figure 3.27).

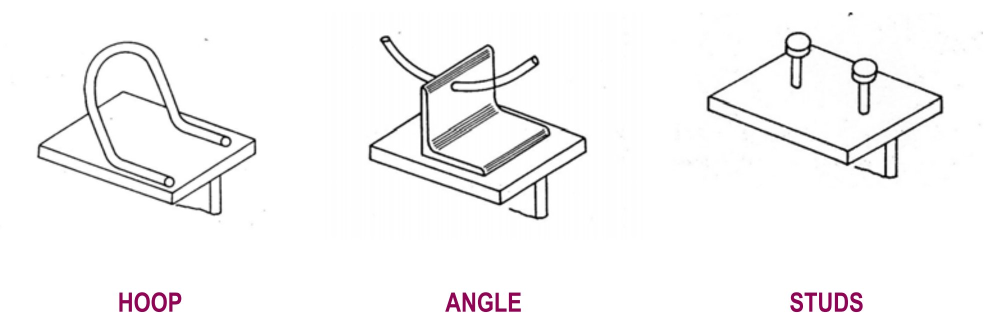


Figure 3.27. Main connector types

Nowadays, on the vast majority of bridges, steel frame – concrete slab connection is ensured by studs, i.e. small diameter cylindrical rods fixed by a semi-automatic process using a welding gun. The main girders are in fact more and more frequently assembled using automatic machinery requiring the connectors to be subsequently welded to the previously fabricated main girder. The top flange is restrained by the web, so welding of angles or hoops causes deformations that are very difficult to rectify, whilst this is not the case when using studs.

Connecting studs used on composite bridges most frequently comprise a 22 mm diameter rod topped by a 25 mm diameter, 10 mm high head. The most common stud total height is 200 mm, but 150, 175 and 225 mm heights are sometimes required for anchoring the connector above the bottom grid of passive reinforcement.

Studs are installed using a stud gun, an electrically powered special tool. When the stud fixer triggers the gun, an electromagnet slightly raises the stud until a high-intensity electric arc is created and fuses locally the plate and stud steel. The stud is then driven into the flange before the steel re-solidifies. To limit oxidation and confine the melt-metal, fusion is performed inside a refractory ring supplied with the stud and which has to be eliminated after welding.

Anchor studs and their refractory rings must comply with the requirements of Sections 9.6, 10.3 and 11.2 of Standard NF EN ISO 13918.

When the main girders are not fabricated using automatic machines, connection can also be ensured using sections of angle welded to the main girder top flanges before assembly with their webs. In this case, these are usually 150 – 200 mm long equal angles (120x120 mm to 160x160 mm) made of S235J2+N or S275J2+N steel.

During construction, some connectors can hamper positioning of certain erection devices, especially when the steel frame is crane-installed. In some cases, the connectors are not welded in the fabrication shop but only after removing the relevant erection devices, i.e. on site.

Stud layout on main girders

Anchor studs are most frequently laid out on the main girder top flanges in four and sometimes six rows, especially when the deck width is approaches 20 m.

Slab construction almost invariably requires movement of various items of mobile machinery (mobile formwork rigs, reinforcing cage trolleys, precast or pre-slab transporters, etc.) along the main girder top flanges. For this purpose, a wider gap is left between the two central rows of studs to allow this mobile machinery to travel along the top flanges in the web centreline (Cases No.1 and 2, Figure 3.28). On the other hand, the stud rows can be equally spaced in the transverse direction, when the contractor’s methods exclude any machine travelling along the top flanges, a fairly unusual circumstance however (Case No.3, Figure 3.28).

In the longitudinal direction, the studs are arranged at varying centre-to-centre distances, depending on the deck cross section. In mid-span areas, where the shear load is low, the stud centre-to-centre distance can reach 800 mm, a maximum value specified by indented paragraph (3), Sub-section 6.6.6.5 of Standard NF EN 1994-2. In pier areas, the stud centre-to-centre distance is far less and can be as little as 200 mm. It should be noted that stud centre-to-centre distance must be compatible with the slab transverse reinforcing bars, irrespective of the cross section considered.

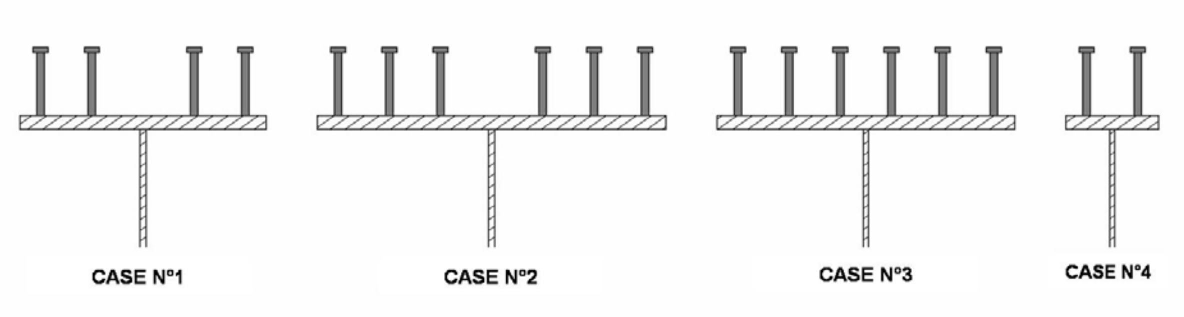


Figure 3.28. Common connector transverse distributions

Stud layout on main girders supporting full-width precast slabs

A twin girder cross-beam composite bridge deck sometimes comprises 2.50 - 4.00 m long, full-width precast segments (Section 5), which requires the stud connectors to be concentrated vertically beneath openings concreted after placing the slab segments; these openings are called slab connection recesses. In this case, the connectors are grouped together in 50 – 80 cm sided rectangular areas on the girder flanges at a maximum spacing of approximately 1.20 – 1.50 m between stud group centrelines. This grouping can require reducing the stud centre-to-centre distance to 100 or so millimetres (Figure 3.29).

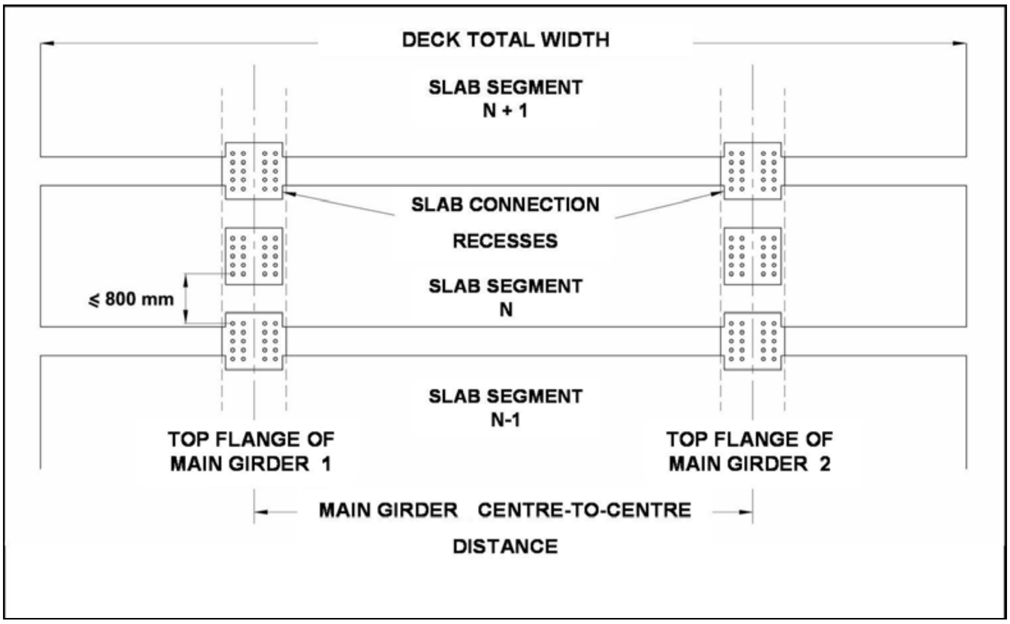


Figure 3.29. Grouped connector layout for a precast slab comprising full-width segments

Stud layout for directly supporting cross-beams

Studs welded to directly supporting cross-beams are similar to those welded to the main girders; they are usually arranged in two rows (Case No.4, Figure 3.28).

Design calculation checks

The steel frame-slab connection must be checked based on information provided by Standard NF EN 1994-2 and its national appendix. This operation is commented and illustrated in Section 11 of Part II of the Sétra Eurocodes 3 and 4 application guide.

3.2.9 Temporary bracing

General

When in service, bracing is ensured horizontally by the deck concrete slab and vertically by the frames formed by the cross-beams or directly supporting cross-beams and the web stiffening vertical posts.

During construction, as long as the slab has not been built, temporary horizontal bracing must be installed on the steel frame. This system prevents lateral torsional buckling of the main girders and allows them to resist wind loads during launching phases, especially in the cantilever part. This bracing system is also very useful during slab construction phases because it braces the steel frame against horizontal loads exerted by the machinery required for slab construction (mobile formwork rigs, slab pre-cast segments installation vehicles, etc.).

Composition

Temporary bracing comprises a horizontal triangulated system of prestressing rods, structural angles or U-sections arranged in pairs. The latter sections usually form an X (called a St. Andrew’s cross, Figure 3.30), sometimes a K, in plan between two consecutive cross-beams or directly supporting cross-beams.

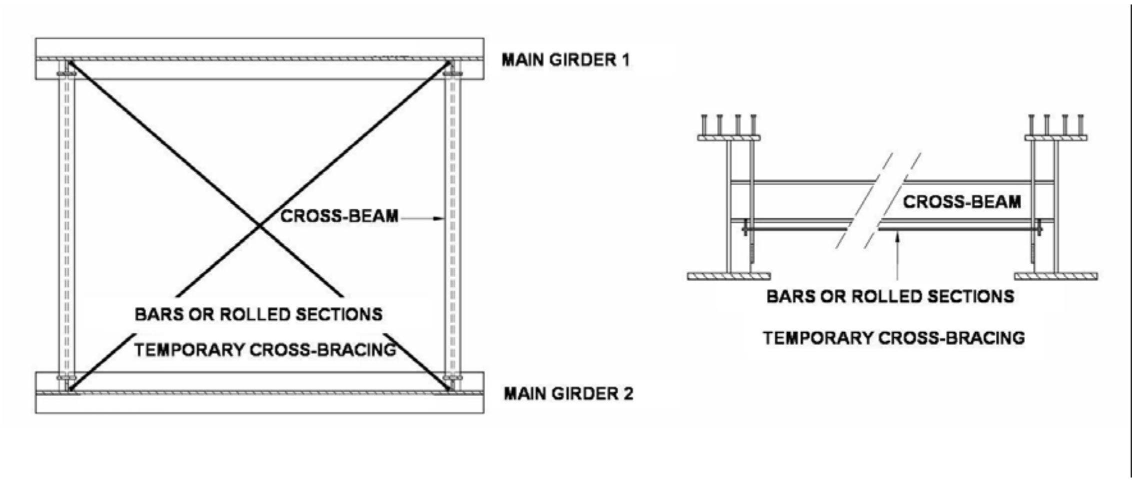


Figure 3.30. Outline diagram of temporary bracing (twin girder cross-beamcomposite bridge)

In the case of standard twin girder composite bridges, the temporary bracing is most often composed of prestressing rods bolted to the final horizontal gusset plates, which extend the cross-beam bottom flanges.

Temporary bracing must be installed and kept until the slab is strong enough to fulfil its function. It can be dismantled in one operation after slab completion or panel by panel in step with slab casting operations.

The compression diagonals of the St. Andrew’s crosses are liable to buckle so, in general, only the tension diagonals are taken into account in the design calculations.

3.2.10 - Anti-corrosion protection

All data concerning steel frame anti-corrosion protection have been consolidated at the start of Section 6 of this guide.

3.2.11 - Main dimensions of a few recent twin girder composite bridges

The following two tables consolidate the main dimensions of several recent twin girder (firstly cross-beam then directly supporting cross-beam) composite bridges. All dimensions are quoted in millimetres in these tables and an asterisk after the bridge name indicates that it was designed to the Eurocodes (other information on these structures may be found in Appendix 1 of this guide).

Twin girder cross-beam composite bridges

Bridge	Max. span distance and width	Top flange width	Top flange thickness	Bottom flange width	Bottom flange thickness	Web thickness	Cross-beams
Intermediate Viaduct	31m/14.80m	800	35 to 70	800	40 to 75	16 to 20	IPE400
Fos bridge (∗)	40m/12.40m	800	35 to 80	900	55 to 90	20 to 25	HEA500
Garrigue Viaduct	74m/10.85m	700	25 to 110	900	30 to 110	-	HEA600
Cher Viaduct	74.80m/14.80m	800	25 to 120	900	30 to 120	-	IPE600
Rieucros Viaduct	105m/12.70mini	900	40 to 130	1100	50 to 130	26 to 34	HEB700

Table 3.7. Main dimensions of a few recent twin girder cross-beam composite bridges

Twin girder directly supporting cross-beam composite bridges

Bridge	Max. span distance and width	Top flange width	Top flange thickness	Bottom flange width	Bottom flange thickness	Web thickness	Misc.
Clisson Viaduct	67.50m/13.20m	1100	45 to 120	1200	40 to 120	16 to 22	-
Elle Viaduct	80m/19.40m	1100	40 to 150	1200	40 to 150	20 to 22	-
Loing Viaduct (∗)	63.75m/19.34m	1200	40 to 140	1350	45 to 140	20	-
Durance Downsteam Viaduct (∗)	88m/21.50m	1300	35 to 120	1500	50 to 125	20 to 25	S460 steel
Planchette Viaduct	95.20m/23.50m	1100	40 to 120	1300	45 to 120	22	-
Saulières Viaduct	106m/10.90m	800	40 to 130	800	35 to 120	20 to 28	No cantilevers

Table 3.8. Main dimensions of a few recent twin girder directly supporting cross-beam composite bridges

3.3 - Box girder composite bridge steelwork

Section 3.4 introduces the detailed steelwork design of box girder composite bridges. Information identical to that provided for twin girder composite bridges is not restated.

3.3.1 General deck orientation

When the bridge carries a road featuring symmetrical bidirectional banking, the box girder has a horizontal bottom flange, identically inclined webs and a top slab integrating the same bidirectional banking as the road (Case No.1, Figure 3.31).

When the bridge carries a unidirectionally banked road, conditions are far more complex because several options are possible.

A first solution involves designing a symmetrical box girder tilted at an angle corresponding to the road banking (Case No.2, Figure 3.31). A structure so designed incorporates a symmetrical (but for the banking) top slab, webs of equal depth and a banked bottom flange. For roads banked at less than 2.5%, a structure of this type can be installed with its bottom flange initially horizontal and then tilted at the end of launching. For heavily banked roads, final tilting to create the same banking becomes a relatively expensive and delicate operation so it would appear preferable to launch the box girder with its bottom flange inclined, as long as suitable lateral guidance is used and a lateral stop is installed, if necessary.

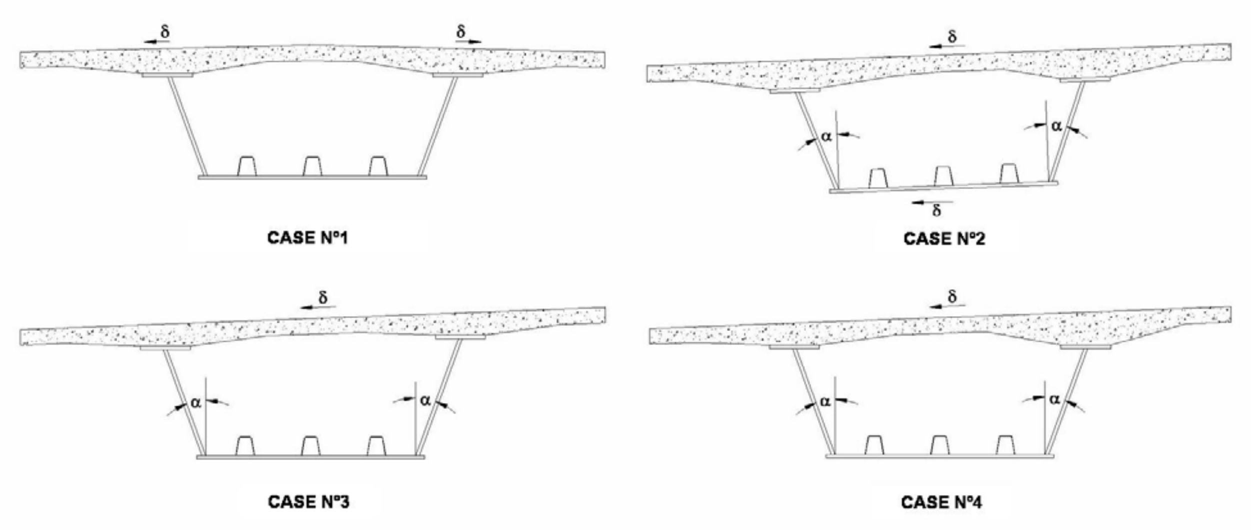


Figure 3.31. Box girder morphology w.r.t. banking of supported road

A second solution involves designing a box girder with a banked top slab, a horizontal bottom flange and different depth webs (Case No.3, Figure 3.31). If the box girder is installed by launching, this arrangement effectively simplifies this operation but the difference in web stiffness must imperatively be taken into account in the calculations in relation to both bridge construction and operation.

If the road is only moderately banked, if the bridge is curved or its banking varies, a third solution involves designing a box girder with a horizontal bottom flange, equal depth webs and a top slab featuring two different depth concrete haunches at its top flanges (Case No.4, Figure 3.31). This solution effectively simplifies installation of the box girder because it is totally symmetrical and its bottom flange is horizontal. However, it causes permanent twisting moments due to the weight difference of the two haunches.

3.3.2 Webs

Box girder composite bridge webs have similar characteristics to girder composite bridge webs except for their inclination.

The webs of most recent box girders are inclined outwards at a constant batter of between 30 and 50%. This arrangement has many advantages. For bridges less than 11 m wide, it ensures that the width of the box girder bottom flange is less than 5 m without excessively lengthening the cantilrvers. It also gives the structure a very satisfactory streamlined appearance from an aesthetic standpoint.

However, some box girders do feature vertical, or even slightly inwardly inclined, webs. This is particularly the case for composite bridges integrating closed box girders, directly supporting cross-beams and props, which have been built in recent years at Verrières, Valence and Frocourt. For these structures, this design allowed the top flange width to be curtailed, whilst remaining acceptable from an aesthetic standpoint because the highly inclined props compensate visually for web verticality.

3.3.3 Bottom flange

Detailed geometry

The bottom flange width is most frequency constant. However, a few bridges do feature a bottom flanges at their supports that are slightly wider than the standard width, usually in connection with special positioning of the support bearings.

For decks narrower than 11 m, we try not to exceed a maximum width determined by the fabrication and forge handling operations. This maximum width is near to 5 m for a straight bridge deck, but can be even less for a curved deck because a curved bottom flange must be cut from a 5 m wide rectangular plate.

For wide bridge decks, these maxima can be exceeded, but this requires the use of two plates transversely assembled and longitudinally welded.

The overall width of box girder bottom flanges is slightly greater than the distance between the bottoms of the webs. This design allows the box girder to be welded in its final position in the fabrication shop and, if the box girder is subsequently launched, the launching saddles or skids can be positioned at the webs (Figure 3.32).

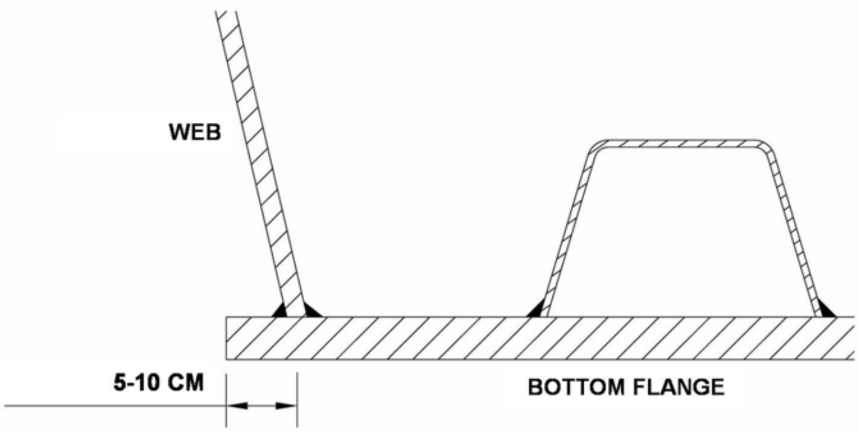


Figure 3.32. Web-bottom flange junction

The thickness of box girder bottom flanges usually varies longitudinally between 25/30 mm within the spans and 70/80 mm at the piers. This variation is normally upward as in the case of the bottom flange of girder composite structures. However, there are a few bridges whose bottom flange thickens downward; this allows transverse frames cutting to be standardised.

Stiffening

The bottom flange must be stiffened to resist the compressive loads it is subjected to during steel frame installation or when the bridge is in service.

For straight medium span box girder composite bridges, the bottom flange is often stiffened by bucklets spaced at a centre-to-centre distance of 0.80 to 1 m (Case No.1, Figure 3.33). This arrangement is economical because far fewer welds are required than with flat bar or T-section stiffeners. It is also advantageous for maintenance operations because the absence of sharp edges on the bucklets limits the risk of injury, if a person falls inside the box girder. Medium span box girder stiffening can also be provided by flat bars spaced at a 0.50 – 0.60 m centre-to-centre distance (Case No.2, Figure 3.33).

For straight long span and/or wide box girder composite bridges, the bottom flange must be stiffened by bucklets or T-sections (Case No.3, Figure 3.33) because flat bar inertia is insufficient.

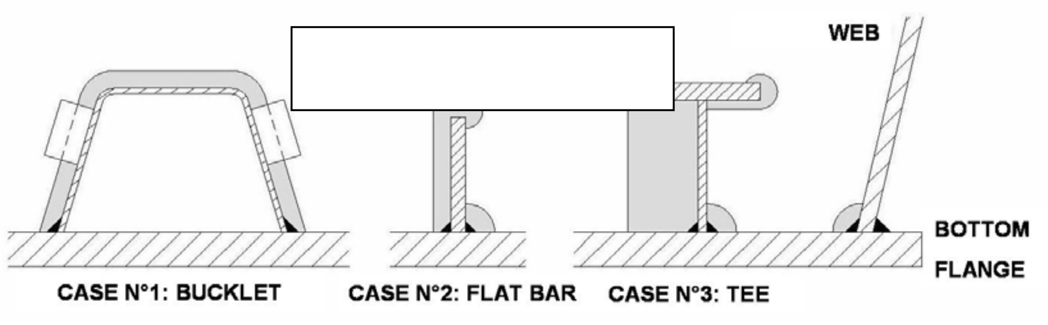


Figure 3.33. Bottom flange stiffening

On curved box girder composite bridges, flat bar and T-section stiffeners can be arranged parallel to the webs because the frames resist the outward thrust due to their horizontal curvature. Conversely, bucklets cannot follow the curvature because their transverse inertia is high. They therefore have to be welded along straight lines, which imposes different transverse frame crossing points and concentrates their outward thrust in just a few transverse frames. In practice, bucklets are only rarely used to stiffen the bottom flange of a curved box girder.

Bottom flange stiffening can be longitudinally constant on small-size box girders, whilst on larger structures it is often increased as the piers are approached either by increasing the number of stiffeners or by increasing their structural characteristics.

Stiffening is continous at the transverse frames to prevent buckling of the bottom flange, improve its fatigue behaviour and add to the box girder’s longitudinal bending resistance. Cut-out are therefore required in the transverse frames to facilitate fitting of these elements and curtail stress concentrations. Figure 3.33 illustrates these cut-outs, but it should be understood that other shapes are possible, particularly for bucklets.

In cases in which the steel frame is launched, it sometimes happens that the launching devices cannot be positioned at the web-bottom flange intersection because the pier head dimensions are insufficient. In this case, use of temporary pier bents supporting the installation devices during launching may be considered. If this solution is unfeasible, the design must include a permanent high-inertia T-section, called a launching tee, at the future launching system (Figure 3.34). this occurred on the box girder for the Vilaine bridge at Roche Bernard or, more recently, at the SD bridge on the Palays interchange at Toulouse. During launching, this tee was considered to bear on the transverse frames and/or bulkheads and to take up loads exerted by the launching saddle on the box girder.

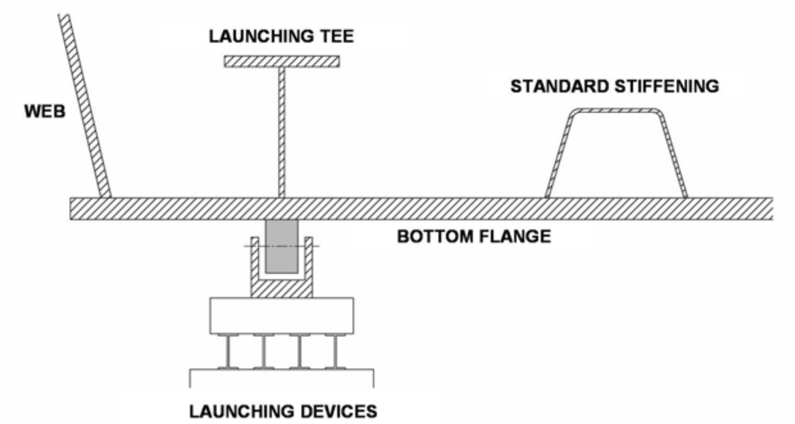


Figure 3.34. Launching tee

Number and positioning of stiffeners on bottom flange

When the box girder is transported in a single piece, the number of stiffeners can be either even or odd. This number must be even to avoid a stiffener located at the bottom flange longitudinal weld, when the box girder is transported in two halves.

In a box girder with no launching T-sections, the n bottom flange stiffeners are positioned so as to divide the flange plate into $n+1$ parallel panels of identical width. In a box girder with launching T-sections, the n bottom flange stiffeners divide the part of the bottom flange between the launching T-sections into $n+1$ panels of identical width.

Other points

We should mention the very special case of the bottom flange in the box girder for the Vilaine bridge at Roche Bernard. For this very wide (21.60 m) but small span (36 m max.) deck, the designers opted for a box girder bottom flange comprising two thick lateral plates (20 to 65 mm) linked by a thinner central plate (10 to 16 mm). Given the structural proportions and the effects of shear lag, which concentrates loads around the webs, the bottom flange central area was in fact little stressed.

Design calculation checks

The box girder bottom flange must be checked based on information provided by Standards NF EN 1993-1-1 and NF EN 1993-1-5 and their national appendices. This operation is commented and illustrated in Section 5 of Part III of the Sétra Eurocodes 3 and 4 application guide.

3.3.4 Top flanges

The top flanges of composite box girders have characteristics quite similar to cross-beam composite bridge flanges, albeit except for their inclination to the horizontal.

Top flanges are usually horizontal on bidirectionally banked bridge decks, whilst they can be horizontal or parallel to the banking on unidirectionally banked bridge decks.

In some box girders, the top flanges are sometimes a few centimetres eccentric towards the interior to reduce the steel box girder external breadth to 6 m and thereby allow it to be transported full width.

3.3.5 Top flange of closed box girders

In some cases, it may be advantageous to replace the two top flanges of the box girder by a large stiffened plate connected to the slab and contributing to structural stength: this is then called a closed box girder. This design leads to a slightly more expensive steel frame, but it greatly simplifies the remaining deck construction. The top plate acts as temporary bracing during construction and as formwork for the slab central area. These advantages make this arrangement very attractive for steel box girders of 1.50 m maximum depth or 4 m maximum width between webs.

Detailed geometry

Top plate thicknesses are close to those of the bottom flange, i.e. between 20 and 70 mm. They are usually stiffened by flat bars or T-sections.

Design calculation checks

The closed box girder top plate must be checked for launching along with the bottom flange; it may be decided to stiffen and take into account only its lateral sections. In service, the top plate is integral with the slab, which excludes any risk of buckling and permits slab composite behaviour.

3.3.6 Standard transverse frames and bulkheads

Detailed geometry

Transverse frames are transverse elements allowing the box girder stiffening with respect to torsion. They are spaced at a centre-to-centre distance of between 4 and 7.5 m, depending on the structure considered. On a box girder composite bridge with directly supporting cross-beams, the frames are associated with the latter members; their centre-to-centre distance is then constant and often around 4 m. On a box girder composite bridge without directly supporting cross-beams, transverse frame centre-to-centre distance can be both slightly greater and longitudinally variable, with a minimum value near the piers.

Figure 3.35 illustrates the most common types of box girder transverse frame. Case No.1 is a U-shaped fame without a directly supporting cross-beam. It comprises a 50 – 80 cm deep T-section bottom beam extended by two arms welded to the box girder top flanges; there is therefore no slab connection in this case. Case No.2 is a more recent type of tranverse frame designed for the Rocquencourt and Nevers box girder composite bridges and which proves very economical for narrow structures. Case No.3 features a frame associated with a directly supporting cross-beam and cantilevers. The transverse frame web thickness is generally between 14 and 20 mm.

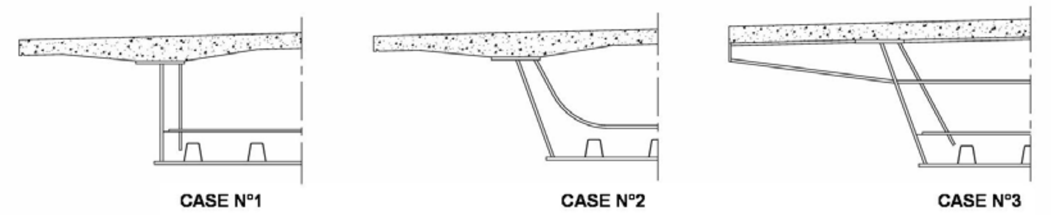


Figure 3.35. Different possible transverse frame types

Box girder transverse frames can be replaced by bulkheads, when torsional effects are very large, e.g. due to high horizontal curvature. Bulkheads are made up of vertical plates, which together totally close the box girder, except for a manhole essential to moving inside it (Figure 3.36). They have a top flange fitted with connectors embedded in the concrete.

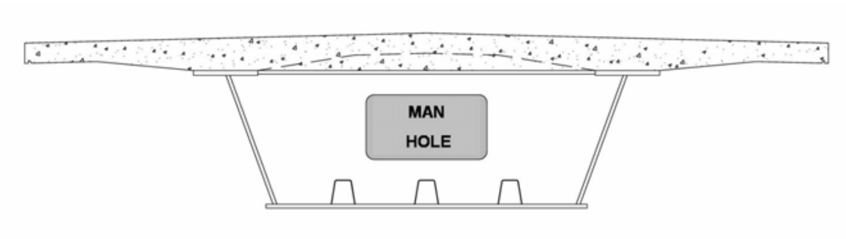


Figure 3.36. Example of standard bulkhead

Box girder transverse frame and bulkhead web thickness is commonly between 14 and 20 mm.

Design calculation checks

Box girder transverse frames and standard bulkheads must be checked in compliance with the requirements of Section 6.2 of Standard NF EN 1993-2.

3.3.7 Bulkheads at supports

Box girders incorporate bulkheads at their supports (Figure 3.37). These bulkheads are designed to resist torsional loads and loads exerted by the bearings and jacks.

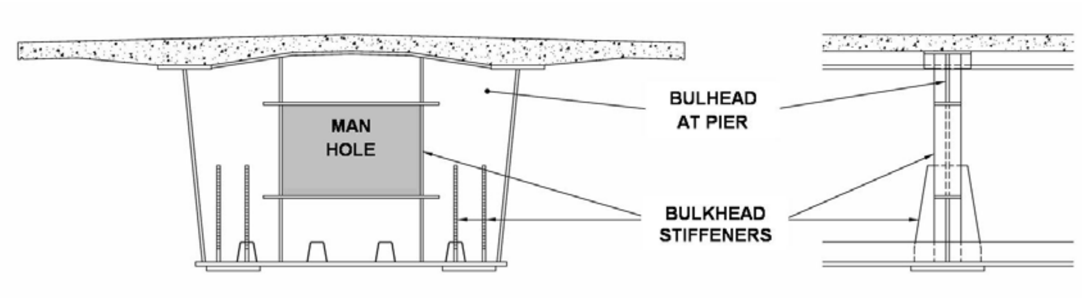


Figure 3.37. Example of a pier bulkhead

Detailed geometry

Design of bulkheads at supports is rather similar to that of standard bulkheads. They are composed of a vertical plate up to 50 mm thick, which is heavily stiffened at the bearings and jacking locations. In common with standard bulkheads, they have a top flange, which is connected to the slab, and they are penetrated by a manhole.

In the longitudinal direction, the bottom of the bulkhead vertical stiffeners at the bearings must be wide enough to stiffen the whole of the bottom flange area at these bearings, whatever the temperature. Vertical stiffeners are frequently of triangular or trapezoidal shape and their maximum width is on the bearing side for this reason.

Bulkheads at abutments with ears

Bulkheads at abutments are laterally extended as "ears" on some bridge decks. In other words, the lateral cantilevers extend far outside the standard box girder (Figure 3.38). These laterally extended bulkheads ensure that the abutment bearings can be more widely spaced than the box girder bottom flange alone would have allowed, thereby curtailing the effect of twisting moments due to eccentric loading and preventing negative bearing reactions. Ears are frequently designed for narrow box girders subjected to high torsion (horizontal curvature, traffic loads, wind, etc.) and for those with only one bearing at each pier, in which all torsion is transferred to the abutments.

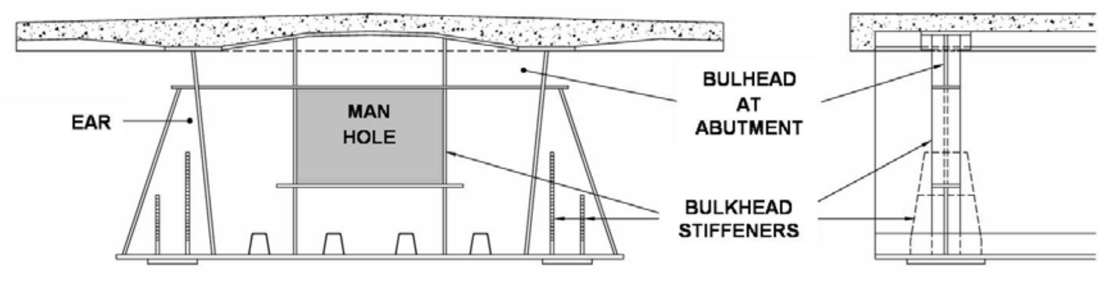


Figure 3.38. Example of abutment bulkhead with ears

Design calculation checks

Bulkheads must be checked based on information given in Section 6.2 of Standard NF EN 1993-2 and its national appendix.

3.3.8 Box girder – slab connection

When the box girder is open, i.e. when its top comprises two flanges, it is connected to the slab in the same way as the steel frame of a cross-beam composite bridge.

When the box girder is closed, the top plate is connected to the slab in three areas: two lateral areas near the webs, in which the connectors are installed at the design centre-to-centre distance, and the central area, in which the connectors are installed at the regulatory maximum centre-to-centre distance.

3.3.9 Temporary bracing

Temporary bracing should be installed in open box girders. This closes the U cross section and increases significantly torsional inertia and combined stability of the steel frame during assembly phases, which is particularly useful for curved or skew structures.

On the most common open box girders, temporary bracing is most often located at the top flange level to increase its efficiency and to facilitate movement inside the box girder as much as possible.

On shallow box girders, temporary bracing disassembly and removal is somewhat tedious and it may be preferable to leave it permanently in place rather than removing it. In this case, its fixings to the structure must be suitably designed and its members must receive the same anti-corrosion protection as the rest of the steel frame.

3.3.10 Main dimensions of a few recent box girders

The table below consolidates the main steelwork dimensions (in millimetres) of a few recent box girder composite bridges.

Bridge	Max. span and width	Top flange width	Top flange thickness	Bottom flange width	Bottom flange thickness	Stiffening	Web thickness
OA205 on A75	35.8m/10m	800	25 to 120	4000	? to 60	3 bucklets	22
Boulogne sur Mer viaduct	40m/9.15m	900	35 to 55	3900	40 to 75	5 flats bars	18
SD bridge at Toulouse	51.3m/9.5m	800 to 1000	45 to 110	3700	35 to 80	4 bucklets	16 to 18
OA4 at Embrun	55m/12m	800	20 to 150	3900	20 to 75	3 bucklets	14 to 20
DE bridge at Toulouse	60m/9.20m	1000	40 to 100	3700	55 to 90	3 bucklets + 2 launching tees	20
Monistrol d'Allier bridge	70m/10m	800	30 to 100	4100	30 to 100	3 to 7 flat bars	-
Loire bridge at Nevers	70m/10.7m	800	25 to 115	4000	20 to ?	3 bucklets	14 to 18

Table 3.9. Main dimensions of a few recent box girders

3.4 - Slab geometry and reinforcement

3.4.1 Slab geometry

Standard geometry of slabs above a steel frame with directly supporting cross-beams

The geometry of concrete slabs combined with twin girders or box girders with directly supporting cross-beams is very simple because these are constant thickness slabs (preliminary design formulae, Section 2). However, the tightness complicates operations involving formwork equipment movement in the case of a box girder.

C Small volumes of haunching concrete are almost always designed. These form an extra concrete depth of several centimetres above the main girders and directly supporting cross-beams to simplify the latter, absorb thickness variations in the main girder top flanges or take up differences in transverse slope.

Standard geometry of slabs above a steel frame without directly supporting cross-beams

For decks narrower than 7 or 8 m, an approximately 25 cm constant thickness slab is generally chosen.

For a wider deck, a variable thickness slab must be designed (Section 2). Minimum slab thicknesses are reached at the cantilever ends and between the webs and the maximum thickness is above the main girders. Transition between one or other of these limiting values is ensured by haunches, whose length is 1/5 – 1/4 of the distance between the webs, in the central area and by haunches or continuously in the cantilevered lateral areas (Figure 3.39).

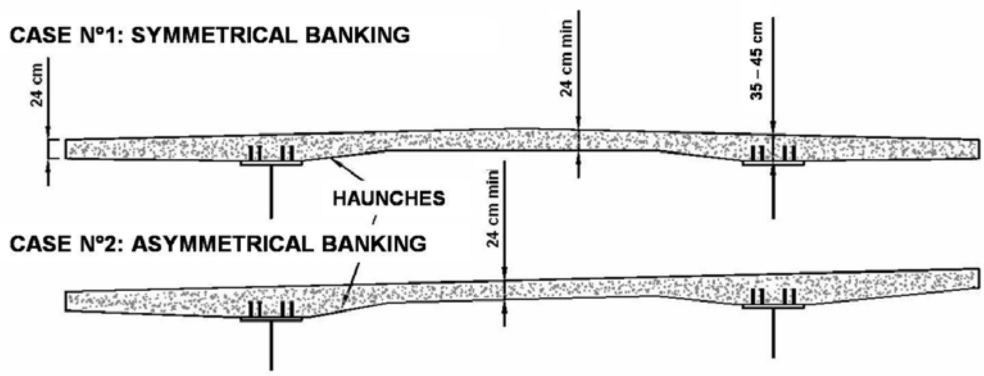


Figure 3.39. Slab geometry for a deck without directly supporting cross-beams

Concerning the slab geometry detail in the vicinity of the main girder top flanges, it would seem unnecessary to place haunching concrete because the top flange thickness variation is almost always towards the inside of the girder. On the other hand, when the slab is cast in situ and to facilitate formwork installation, an approximately 10 cm wide horizontal area should be kept between the top flanges and the start of the slab haunches (Figure 3.40).

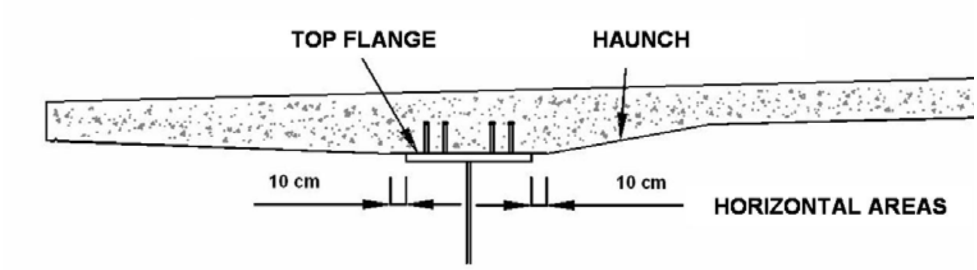


Figure 3.40. Slab geometry near main girder top flanges

3.4.2 Slab reinforcement

3.4.2.1 - General

Composite bridge with directly supporting cross-beams

For a directly supporting cross-beam composite bridge, the slab bears longitudinally on the two main girders and transversely on the directly supporting cross-beams. The slab deflects mainly between two successive directly supporting cross-beams, so the main reinforcement is longitudinal and its layers are most often fixed outside the secondary reinforcement, thereby giving the main bars a longer lever arm (Figure 3.41). However, it may be advantageous to fix the longitudinal reinforcement layers inside the secondary reinforcement to improve their restraint by the transverse bars.

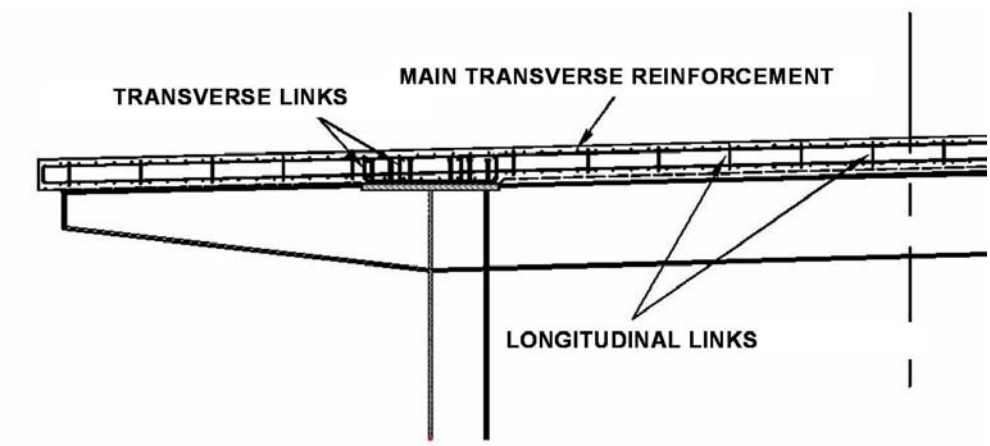


Figure 3.41. Standard slab reinforcement principle for a deck with directly supporting cross-beam (excluding restraint system anchorage)

Composite bridge without directly supporting cross-beams

For a composite bridge without directly supporting cross-beams, the slab bears only on the main girder webs. The main reinforcement providing local bending strength of the slab is therefore fixed transversely, generally as layers outside the secondary reinforcement (Figure 3.42). Longitudinal secondary reinforcement is also heavy in some cases and Eurocode 4 requirements for controlling slab cracking can lead to longitudinal reinforcement of approximately 1%.

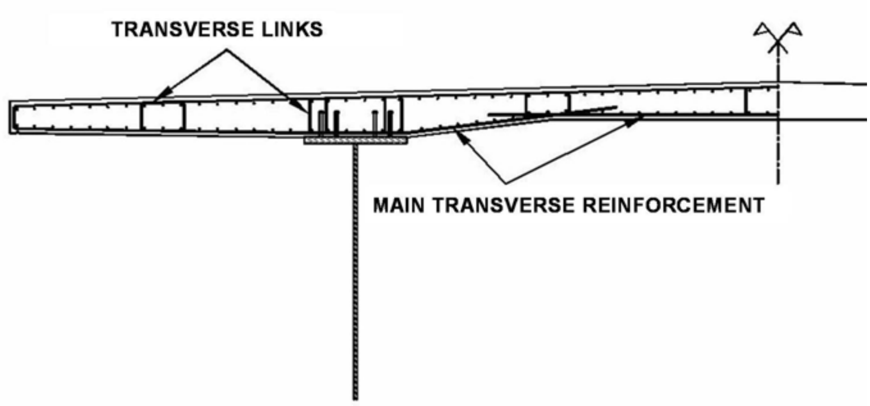


Figure 3.42. Standard slab reinforcement principle for a deck without directly supporting cross-beams (excluding restrant system anchorage)

Standard reinforcement for composite slabs also includes:

- - standard transverse (slabs for steel frames without directly supporting cross-beams) or longitudinal (slabs for steel frames with directly supporting cross-beams) secondary reinforcement usually comprising HA12 (12 mm high-tensile) links fixed at 2 links/m² minimum,
- - transverse tie bars resisting shear loads above the main girder top flanges and connectors,
- - restraint system anchorage bars.

Construction tolerances make lapping of two successive reinforcement cages very difficult in the common case of a cast in-situ slab reinforced by prefabricated cages. This means that their longitudinal linkage is usually ensured by small straight bars called pull-out bars, which are delivered fixed to one of the cages and pulled towards the contiguous cage. This system is not ideal and is only tolerated for prefabricated reinforcement cages; it requires rigorous supervision by the contractor and Engineer's Representative (ties, concrete covers, lap lengths).

The principles of keying precast slab segments and steelfixing for cast in-situ slabs over permanent formwork are described in Section 5 of this guide.

3.4.2.2 - Special construction conditions

Reminder

Eurocode 2 includes a number of recommendations on lapping of passive reinforcing bars.

Clause 8.7.2(2) recommends offsetting laps and excluding them from heavily stressed areas and this requirement should be included in the CCTP, especially for directly supporting cross-beam composite bridges.

Sub-section 8.7.4.1 defines three increasing requirement levels for transverse bars in a lapping area for bars in tension with a diameter ϕ (notion of transverse bars referring here to bars fixed perpendicular to the lapping direction).

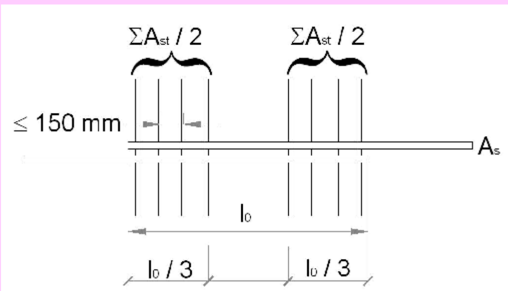
Case 1 $\phi < 20$ mm or $\rho_1 < 25\,\%$	Necessary transverse bars required elsewhere are sufficient. They may not be in layers outside the main bars.
Case 2 $\phi \geq 20$ mm and $\rho_1 \leq 50\,\%$ or $a > 10\phi$	Transverse bars should be fixed in layers outside the main bars and perpendicular to the lapping direction. They must satisfy $\sum A_{st} \geq A_s$
Case 3 $\phi \geq 20$ mm and $\rho_1 > 50\,\%$ and $a \leq 10\phi$	Transverse bars should be fixed in layers outside the main bars and perpendicular to the lapping direction. Transverse bars must be links, stirrups or ties and must satisfy $\sum A_{st} \geq A_s$

Table 3.10. Transverse reinforcing bars to be provided in lapping area

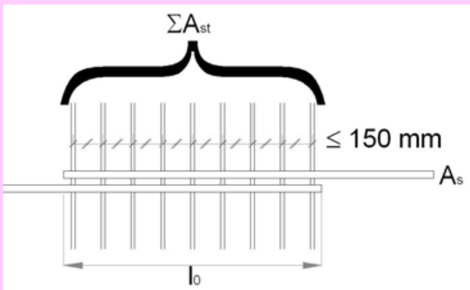
where ρ_1 *= proportion of lapped bars in a given cross section,*
A_{st} *= transverse bar diameter,*
A_s *= cross section of 1 lapped bar,*
a = distance between adjacent laps in a given cross section

Thus, the maximum diameter of bridge deck slab longitudinal bars is limited to 16 mm, when they are fixed in layers outside the main bars.

In the lapping area between bars in tension with diameters ≥ 20 mm (Cases No.2 and No.3, Table 3.10), two transverse reinforcing bar arrangements are possible: concentrated at the ends (Case No.1, Figure 3.43) or uniformly distributed (Case No.2, Figure 3.43).



CASE N°1



CASE N°2

Figure 3.43. Possible transverse reinforcement arrangements in lapping area for tension bars with diameters ≥ 20 mm

Sub-section 8.7.4.2 of Eurocode 2 lays down identical rules for transverse reinforcement in a lapping area for compression bars but subject to an additional requirement: on each side of the lap, a tranverse bar must be fixed at a distance < 4ϕ from the end of the lap.

Slabs combined with a steel frame without directly supporting cross-beams

Links, stirrups or ties should also be used in lapping areas for longitudinal reinforcing bars with diameters ≤ 16 mm.

In relation to transverse bars normally fixed in a layer outside the main reinforcement, Eurocode 2 does not allow lapping between bars with diameters ≥ 20 mm. Laps should therefore be offset and arranged in areas, in which they can comprise bars with diameters < 20 mm. In this case, the conditions to be applied are effectively those for the smallest diameter bar. This is no particular problem in practice.

Slabs combined with a steel frame with directly supporting cross-beams

In a slab combined with a steel frame with directly supporting cross-beams, we recommend arranging the longitudinal links before and after those located at the directly supporting cross-beams because the shear load is high in this area (Figure 3.44).

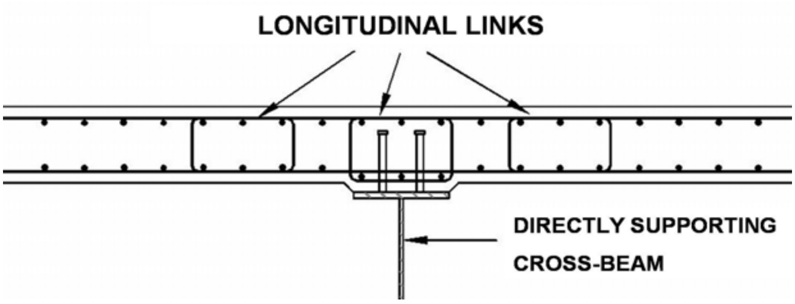


Figure 3.44. Longitudinal links required near to a directly supporting cross-beam

3.4.2.3 - Other Points

Connectors are rigid, multiple obstructions so the slab detailed reinforcement design should consider connector longitudinal and transverse distributions. This is especially important for directly supporting cross-beam composite decks, which incorporate many connectors, and for precast segmental slab decks, which feature areas in which connectors are closely spaced.

Section 5 (Slab Construction) of this guide provides further important information on slab reinforcement.

3.4.3 Design calculation checks

A composite bridge slab must be checked based on the information given in Standards NF EN 1994-2 and NF EN 1992-2 and their national appendices. This operation is commented and illustrated in Part II of the Sétra Eurocodes 3 and 4 application guide.

3.5 - Support Vertical Adjustment

Commonly applied to composite bridges, the support vertical adjustment method involves building the deck higher than its final position and then jacking it at several points to adjust it to its permanent position after its slab has been fully completed. This method enables the structure to be subjected to a positive bending moment, which tensions the bottom members and re-compresses the concrete slab, thereby contributing to the ideal behaviour of both materials.

Support vertical adjustment operations usually involve lowering the deck at some of the piers at the end of the construction phase. Conversely, in certain more unusual cases, the deck is raised at one of its two abutments. These operations should be avoided on skew composite bridges and to a lesser extent on curved bridges because they can introduce interfering effects.

The efficiency of support vertical adjustment is related to the deck curvature it introduces. This means that very large level adjustments are necessary for bridges integrating many spans. The method is therefore limited to 2- or 3-span structures, in which the support vertical adjustment heights are often approximately 1/00th of the main span distance, i.e. between 20 and 75 cm.

Support vertical adjustment are performed on site based on a detailed procedure developed by the design and construction methods department. When level adjustments are made over several supports, this procedure describes in particular the phasing of operations and establishes the level adjustments to be implemented at each stage.

Section 4.1 of the Sétra/LCPC guide entitled "Ponts mixtes – Recommandations pour maîtriser la fissuration des dalles" [Composite bridges - Recommendations for controlling slab cracking] provides valuable information on considering the effect of support vertical adjustment in design calculations. It is recalled that, given the uncertainties to which this operation is subject, the above guide limits the unweighted effect of support vertical adjustment to □e4, where □e is the elastic limit of the steel at the relevant point.

3.6 - Waterproofing

As stated in the Sétra/LCPC guide entitled "Ponts Mixtes - Recommandations pour maîtriser la fissuration des dalles" [Composite bridges - Recommendations for controlling slab cracking], extreme care should be applied to both the selection and implementation quality of the waterproofing course, which must fully protect the bridge slab.

In this connection, it is recommended in Section 7 of this guide that a prefabricated sheet + gritted asphalt protection system should be retained. This type of waterproofing course possesses the elasticity and robustness qualities suited to the conditions sustained by a composite bridge slab.

Incorporation of a thin film-type waterproofing layer, bonded to the support on the top and sides of the restraint system anchoring stringers, is also recommended.

3.7 - Related bibliography

Twin girder cross-beam composite bridges

RT [CHA 95] [MAR 95] [ROU 98] [POU 01] [CAR 00] [AVR 01] [DEM 02] [ASF03 02] [BOR 03] [STO 03]

BOA [NOR 95]] [VIO 08]

OTUA [HIP 96] [ABI 96] [DEZ 03] [ARG 04] [FLE 04]

Twin girder directly supporting cross-beam composite bridges

RT [CHA 01] [CAL 02] [MAN 02] [ASF01 02] [ASF02 02] [BRI 03] [DUB 04] [DUM 06] [JOL 08]

Simple box girder composite bridges

RT [POI 97] [CHA 00] [ARG 00] [MAT 01]

BOA [LEG 05] [LAC 96] [DAI 05]

OTUA [FLE 04]

Box girder composite bridges with directly supporting cross-beams

RT [VIL 96] [MON 96]

BOA [FON 95] [VIL02 96]

OTUA [VIL01 96] [VIL 99]

Box girder composite bridges with directly supporting cross-beams and props

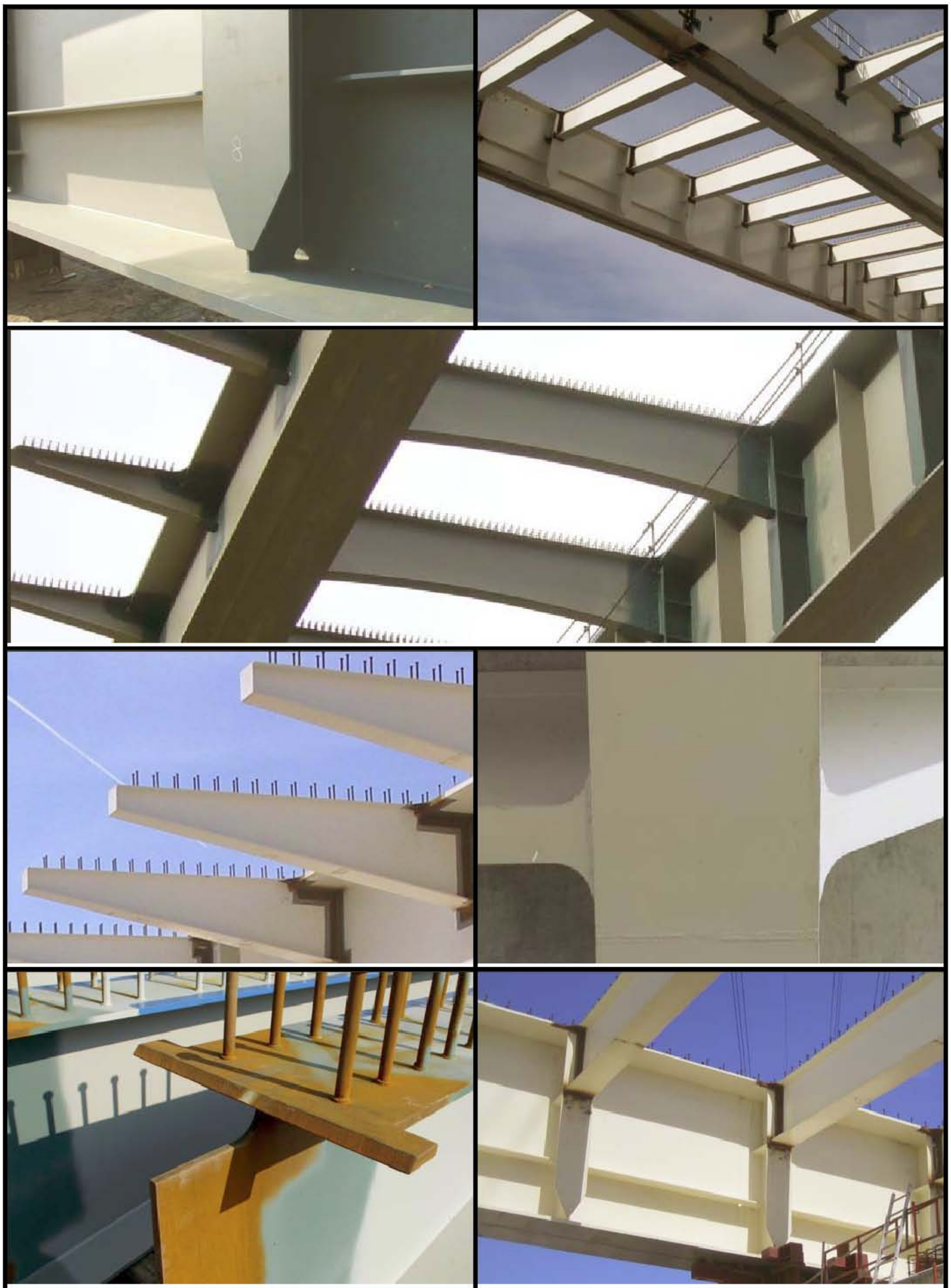
RT [GIL 01] [CHA 03]

BOA [BOU 01]

OTUA [TAV 04] [GIL 04]

■ ■ ■

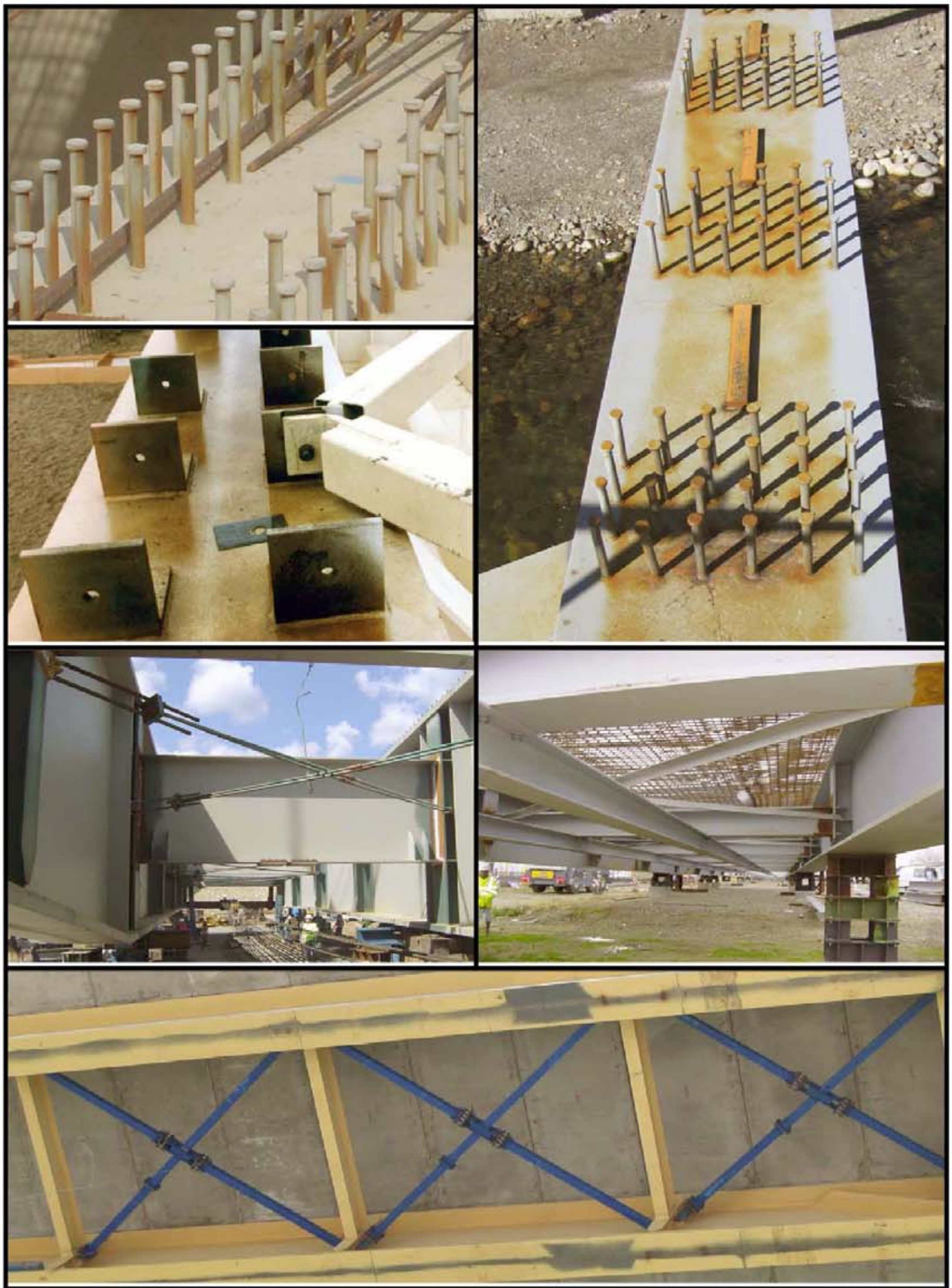
301: Flange thickness variation w.r.t. cross section. 302: Flange change of thickness showing tapering and half-moon cut-out. 303: Variable width flanges on 4-girder variable width deck. 304: Additional flange. 305 and 306: Cross-beam/Post connections with horizontal gusset plates (305) or vertical gussets (306).



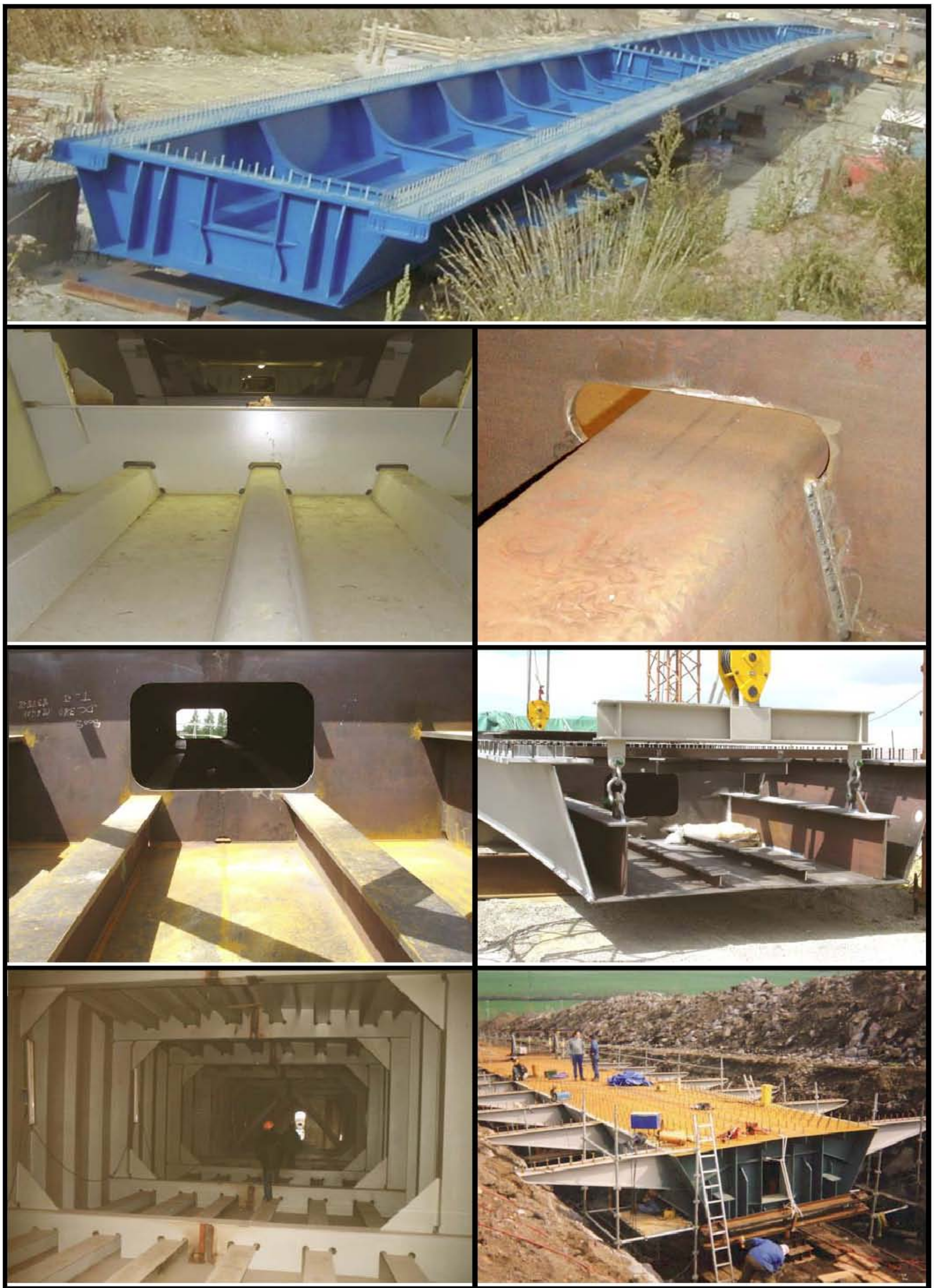
311: Bottom of standard cross-beam post showing gradual flange termination. 312: Directly supporting cross-beams with cantilevers. 313: Directly supporting cross-beams with variable depth central section. 314: Cantilevers with end plates. 315: Flanges circular fillets at main girder/directly supporting cross-beam junction. 316: Preparing cantilever top weld on main girder side. 317: Standard directly supporting cross-beam posts.

321: Directly supporting cross-beams strengthened by gussets. 322 and 323: Twin girder bridge longitudinal stiffeners. 324 to 326: Single tee support posts (324, jacking location beneath pier cross-beam) strengthened by tee and 2 lateral triangles (325), by 2 bucklets (326).

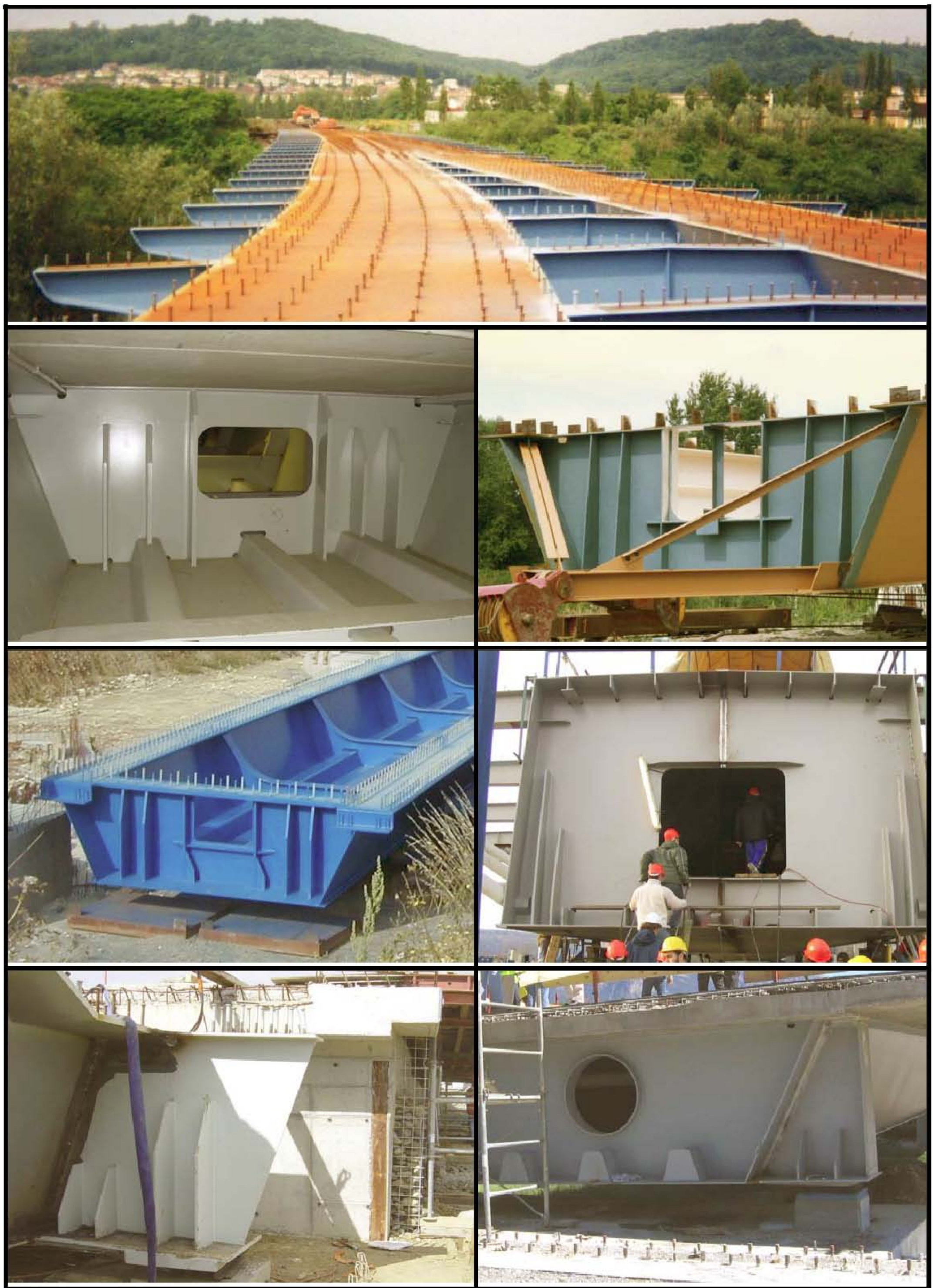
331 and 332: Piers areas for variable depth decks. 333: Directly supporting cross-beam with jacking location. 334: Seismic restraint integrated into abutment cross-beam. 335: Reduced main girder depth at abutment for slab strengthening. 336 and 337: Bevel wedges.



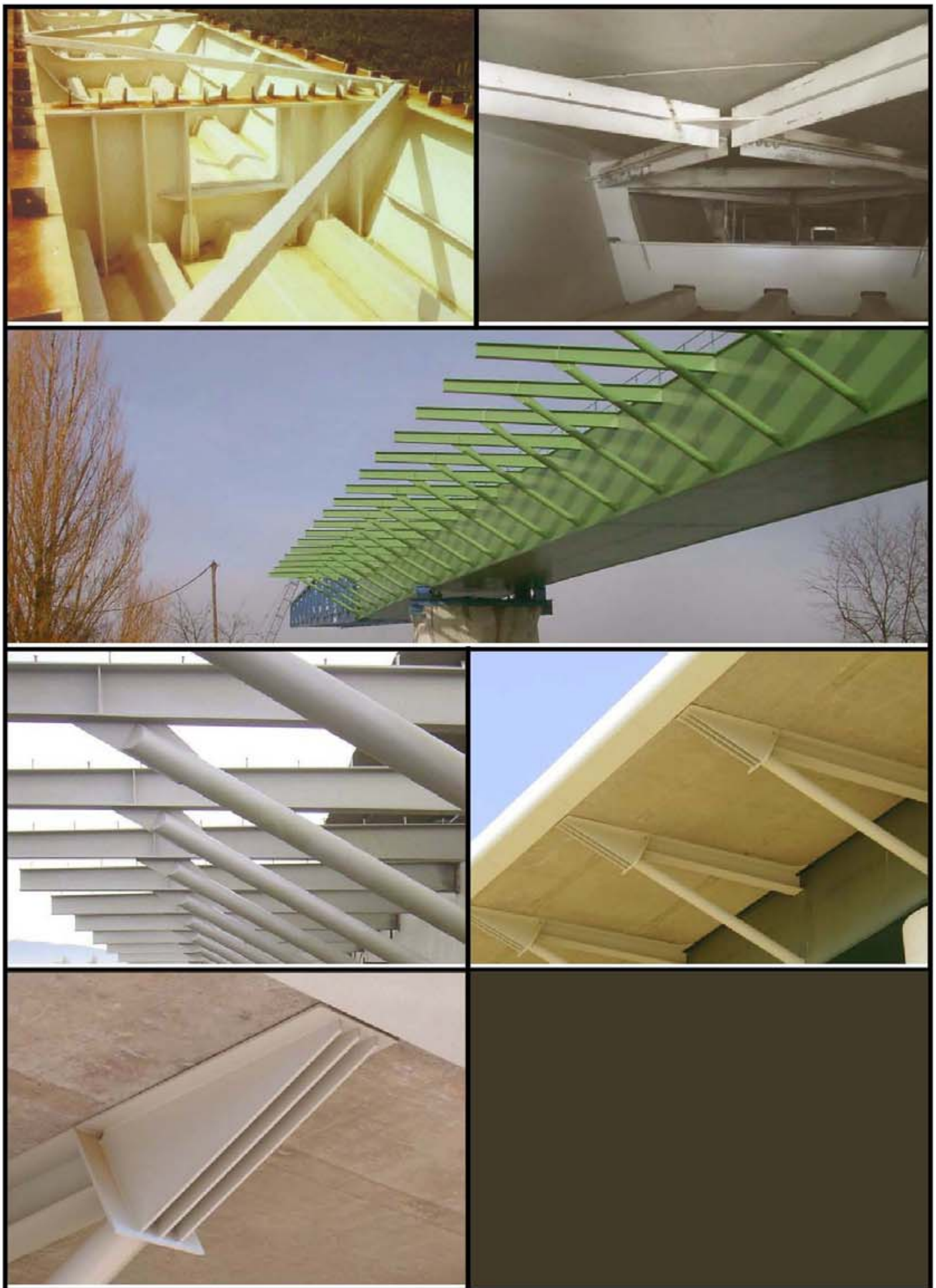
341: Stud connectors. 342: Concentrated stud connectors
for full-width precast segmental slab. 343: Angle connectors.
344: Prestressing rod cross-bracing 345: Angle cross-bracing. 346: Tubular cross-bracing.



351: Single box girder fully assembled on site before installing. 352: Bottom flange stiffened by 3 bucklets. 353: Transverse frame web cut around a bucklet. 354: Bottom flange stiffened by tees. 355: Launching tees. 356: Interior of closed box girder stiffened top and bottom by tees. 357: Closed box girder with directly supporting cross-beams.



361: Steel frame comprising 2 closed box girders linked by directly supporting cross-beams. 362: Pier bulkhead. 363 to 365: Abutment bulkheads. 366 to 367: Abutment bulkhead ears.



371: Temporary bracing of an open box girder. 372: Bracing left in place on an open box girder with a top plate acting used as permanent formwork. 373: Launching of a box girder with directly supporting cross-beams and props. 374 and 375: Directly supporting cross-beam cantilevers supported by cylindrical props. 376: Architecture of node connecting directly supporting cross-beam cantilever and cylindrical prop.

4 - Steel frame transport and installation

This section introduces transport and installation outline methods for the steel frame of a twin girder, multi-girder or box girder composite bridge. Its initial sub-section deals with steel frame transport from the fabrication shop to site. Its second sub-section considers steel frame installation using several methods, the most common of which are launching and crane installation. Sub-sections 3 and 4 are dedicated to final positioning on supports and possible support vertical adjustments after installation.

4.1 - Steel frame transport

4.1.1 General considerations

Composite bridge design depends on steel frame fabrication methods. The particular nature of these structures originates in the fact that steel is produced and worked in a steel plant, that its products are transformed in a steel fabrication facility and that final installation takes place on site. More specifically, bridge steel frame elements need to be transported from the fabrication shop to site.

Dimensional constraints inherent to steel frame transport impose its fabrication in elementary sections, which are then assembled on site. This explains the importance of steel frame assembly methods and their impact on project overall economics.

It is generally advantageous to prefabricate the steel frame sections in the largest possible sizes (longitudinally for girders, transversely for box girders in particular) at the fabrication shop to reduce the number of joints to be welded on site. These site assembly joints are necessarily welded under more difficult conditions than in the controlled environment of the fabrication shop (less favorable atmospheric conditions, unforeseen events), hence their higher cost for an equivalent quality of workmanship and greater risks of quality non-compliance.

In practice, the length of shop-fabricated steel frame sections results from seeking the best engineering-financial balance between fabrication facility part dimensions (to be maximised), total transport cost, including escorting of abnormal convoys, route adjustments, site handling, etc., (to be minimised) and site assembly cost (number of joints to be welded, installation equipment capacity, etc.), which must also be minimised.

Steel frame section limiting dimensions and weights are dictated by:

- fabrication shop capacity,
- site assembly and installation procedures adopted,
- site assembly area dimensions (unloading, storage, handling, etc.),
- available lifting equipment capacity at possible breaks of load and on site,
- most commonly usable transport capacities (road, rail, river, etc. clearances).

Transport conditions must be considered right from project design stage. Site access conditions can effectively make some solutions technically impossible or financially exhorbitant.

In every case, a steel frame can be conveyed using several transport modes (Figure 4.1) including:

- road transport,
- rail transport,
- river or sea transport.

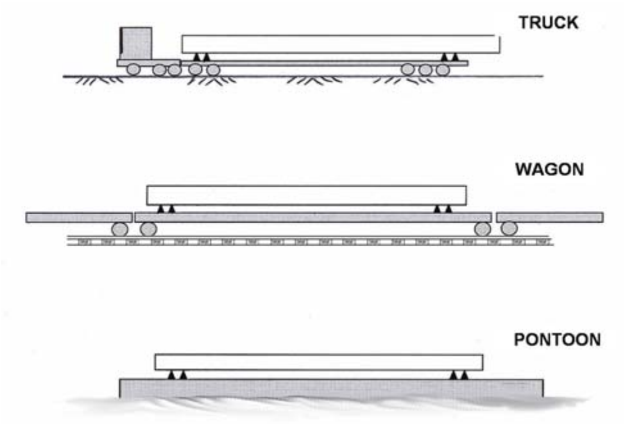


Figure 4.1. Steel frame transport

Apart from the characteristics specific to each form of transport, one mode can be resorted to in preference to another because the former is dictated by factors specific to each manufacturer, such as:

- special access to certain industrial sites for some transport modes (waterway, rail branch, etc.),
- the fact that one transport mode can be less expensive than another at a given time.

4.1.2 Road transport

Road transport is the mode most frequently retained by the steelwork contractor and is effectively the most flexible method because it allows almost every site to be reached.

Road transport is ensured by abnormal convoys in most cases. The sizes of the elements and packages to be transported indeed usually exceed the limits authorised by the highway code (in France, restricted to 2.55 m wide, 18.75 m long and 40 t weight).

Maximum characteristics (total vehicle load, overall length, overall width) of the three abnormal convoy categories are recalled in the table below.

Convoy characteristics	Category 1	Category 2	Category 3
Overall length	≤ 20 m	20 m < L ≤ 25 m	> 25 m
Overall width	≤ 3 m	3 m < l ≤ 4 m	> 4 m
Total vehicle load	40 t < M ≤ 48 t	48 t < M ≤ 72 t	M > 72 t

Table 4.1. Characteristics of abnormal convoy categories

Category 1 or 2 abnormal convoys are normally used, but Category 3 convoys may be used for very large loads.

The most common limits placed on parts transportable by abnormal convoys are shown in the following table.

	Case in which length is favoured	Case in which width is favoured
Length	33/35 m	25/27 m
Width	Approx. 3 m	5 to 6 m

Table 4.2. Common limits for parts transportable by abnormal convoys

However, longer elements and packages can still be transported as long as their width remains small (e.g. single girder sections). Maximum length is therefore approximately 40 m.

Element or package weight turns out to be only occasionally restrictive and frequently reaches 70 to 80 t. Loads of around 100 t have even been made up, in particular for construction of the Verrières viaduct and of the second bridge over the river Rhône at Valence.

A preliminary survey of the route to be followed by the convoy is essential at construction study stage to check its feasibility (overpass clearance, structure allowable loads, possible crossings at grade, urban areas with narrow roads, small curvatures at bends, swerving impossibilities, specific obstructions such as electrical poles, etc.). Trailer height should be taken as approximately 1 m, when checking road crossing clearance.

Twin girder or multi-girder steel frames are usually too large to be transported fully assembled. After being transported usually upright to simplify site unloading, the main girders therefore need to be assembled with their transverse members on site.

Box girder steel frames are much more difficult to assemble on site. An attempt should therefore be made to design box girders that can be transported in single transverse sections by specifically optimising their web centre-to-centre distance and, if necessary, by slightly offsetting their top flanges inward.

When the box girder is too wide to permit it to be transported in one piece, the steel frame has to be conveyed to site in half-box girders loaded with their webs horizontal onto the trailer. They are then rotated into a vertical position on site using an overhead gantry crane.

4.1.3 River or sea transport

River or sea transport is especially economical, but remains little used because it requires a loading quay in the immediate vicinity of the steel frame fabrication shop and an unloading quay near the site on the waterway used. Moreover, it must be compatible with the steel frame assembly method on site.

This mode allows transport of large elements or packages (possible full spans or complete structures). However, it should be noted that, for international container transport, the steel frame elements must be packed to standard container dimensions. Thus, element or package length is limited to 12 m and weight to 26 t for standard 40-foot containers with internal dimensions of 12 m long x 2.33 m wide x 2.35 m high.

For river transport, dimensions are limited by the characteristics of the waterways used, size of locks and navigation clearances when passing beneath bridges.

The table below recalls the European waterway classification drawn up at the European Transport Ministery conference in 1992 (CEMT 92) and details the maximum authorised length and mass for each class as well as the minimum vertical clearance required beneath bridges for new infrastructure projects. It should be noted that existing bridges do not necessarily comply with the minimum vertical clearance quoted.

	CEMT class	Length l (m)	Mass m (t)	Minimum headroom (m)
Waterways of regional interest	I	40	180 - 400	3 or 4 *(1)*
	II	50 - 60	400 - 650	3 or 4-5 *(1)*
	III	60 - 80	650 - 1000	4 or 4-5 *(1)*
Waterways of international interest	IV	80 - 85	1000 - 1500	5.25 or 7 *(1,2)*
	Va	95 - 110	1500 - 3000	5.25 or 7 or 9.10 *(2,3)*
	Vb	170 - 185	3200 - 6000	5.25 or 7 or 9.10 *(2,3)*
	VIa	95 - 110	3200 - 6000	7 or 9.10 *(2,3)*
	VIb	140 - 195	6400 - 12000	7 or 9.10 *(2,3)*
	VIc	195 - 280	9600 - 18000	9.10 *(2,3)*
	VII	285	14500 - 27000	9.10 *(2,3)*

Table 4.3. Classification of European waterwars based on CEMT 92

(1) *Depending on whether waterway is west or east of the River Elbe (minimum headroom east of Elbe).*

(2) *Includes safety margin of 30 cm between highest point on boat or its load and headroom beneath bridges.*

(3) *5.25 m, 7 m and 9.10 m for boats transporting 2, 3 and 4 levels of containers respectively.*

4.1.4 Rail transport

Rail transport may prove suitable, if the steel fabrication facilities and the bridge site are both near a railway line equipped with loading/unloading platforms for transferring the steel frame (limiting breaks of load). On the other hand, the site must offer extensive storage capacity because a rail convoy is only profitable if it transports a large number of steel frame sections.

Maximum characteristics of elements or packages that can be transported by rail are as follows:

- maximum weight of around 100 t with multi-axle rail cars,
- maximum length close to 40 m with shorter flatbed tenders inserted between conventional twin axle rail cars,
- maximum height of 2.50 m (4 m vertical clearance including 1.50 m already taken by the load bearing formation).

The maximum width depends on the length (e.g. 1.30 m for 50 m length).

These lengths, widths and heights are obviously interdependent and it is essential to acquire information from the rail operator's department managing abnormal convoys to ensure the feasibility of transport using this mode.

At the time of writing this guide, rail transport is, in practice, only rarely used particularly because of clearance limitations, higher cost and less flexibility than road transport.

4.1.5 Special precautions during transport

Handling and transport phases must be carefully studied.

Girder and beam sections must be fully equipped with their ancillaries (stiffeners, etc.) before transportation. In every case, care must be taken to stiffen sufficiently the girders or beams for transport and to wedge properly the parts at the vertical posts and immobilise them.

The elements must be to design supporting calculations under their temporary support conditions. In particular, the steel frame resistance to its self weight must be checked for long elements transported on two independent wheel trains.

In some cases, dynamic effects and a long journey can cause fatigue problems since both plates and assemblies are sensitive to alternating stresses and load concentrations. Propping nodes can create stress concentrations in adjacent parts that are already subjected to fatigue; risks of fracture do arise from this. Weld seams on transported parts should be visually inspected on arrival.

Visual inspection still also allow detection of possible deterioration of paint layers applied in the fabrication shop for anti-corrosion protection purposes or possible damage associated with impacts during handling and transport.

I-sections must be transported upright (except if there is a clearance problem requiring transport on their sides). They must be provided with props between members to prevent buckling and overturning. On the other hand, box girders can be transported flat because their transverse inertia, which is much higher than that of I-sections, allows this horizontal transport position.

4.2 - Steel frame assembly

4.2.1 - General

The cost of site assembly, including transport, represents 20 - 40% of the steel frame total cost and it represents the cost item subject to the highest risk for the contractor.

At design stage, possible steel frame assembly methods should therefore be considered and their potential problems examined. A type of structure and an appropriate compatible assembly method can be proposed on this basis. At tender stage, an assembly principle will be indicated in the DCE, but it is rarely contractual and the contractor can modify it, in particular adapting it to its available equipment.

Speed of construction and minimising interruptions to traffic on roads crossed by the bridge represent, in some cases, determining criteria in relation to selecting a steel frame assembly method.

Following actual shop fabrication of steel frame sections, assembly of steel structures integrates a number of basic operations including:

- temporary assembly of sections at fabrication facility,
- assembly of sections on site,
- installation of steel frame vertically above its final position.

Temporary assembly of sections at fabrication facility

For complex geometry bridges (box girders, curved or skew or variable width steel frames), the contract must include trial assembly, i.e. temporary assembly of all or part of the structure at the fabrication shop.

This operation involves aligning the steel frame elements end to end, wedged on benches, in the relative positions shown on the construction drawings and taking into account their fabrication camber, horizontal curvature and banking. They are then adjusted so that the joint edges to be welded on site display the required shape, dimensions and tolerances. These elements are then fitted with clamps, so that the different sections can be reset to their relative positions on site (Figure 4.2).

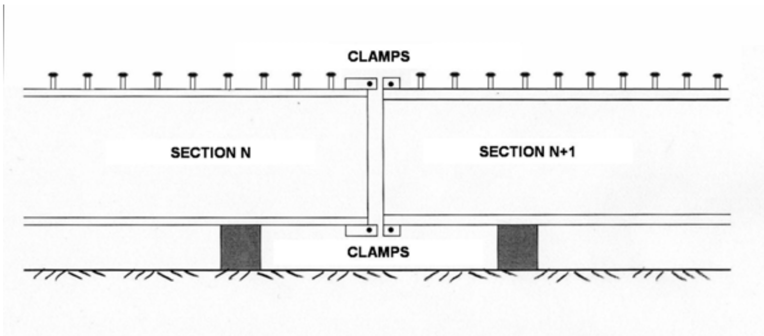


Figure 4.2. Trial assembly at fabrication shop with clamps

For simple geometry bridges (e.g. constant width twin girder composite bridge), geometrical checking by the contractor at every fabrication stage, comprising dimensional tolerance compliance checks, can simply allow a virtual trial assembly. Resorting to physical trial assembly would not be essential, if the contractor can demonstrate sufficient proficiency in this virtual process. However, trail assembly of initial elements could be required to confirm fabrication shop geometrical checks.

Assembly of sections on site and steel frame installation

On-site assembly methods most frequently applied are launching and crane installation, but other methods, including shifting and hoisting, are referred to at the end of this section. The latter methods may prove to be more appropriate under certain conditions, whilst much more rarely used.

4.2.2 - Installation by launching

4.2.2.1 - Launching Principle

Steel frame launching is the most commonly implemented installation method. It may be envisaged for a determining span distance of up to 80/90 m for isostatic spans and up to 130/140 m for hyperstatic spans.

Its principle involves causing the steel frame to travel over supports up to its final position after fully or partly reconstructing it on an assembly area located behind one or both abutments (Figure 4.3).

To reduce the cantilever loads, a temporary steel structure, called a launching nose, is fixed to the front of the permanent steel frame.

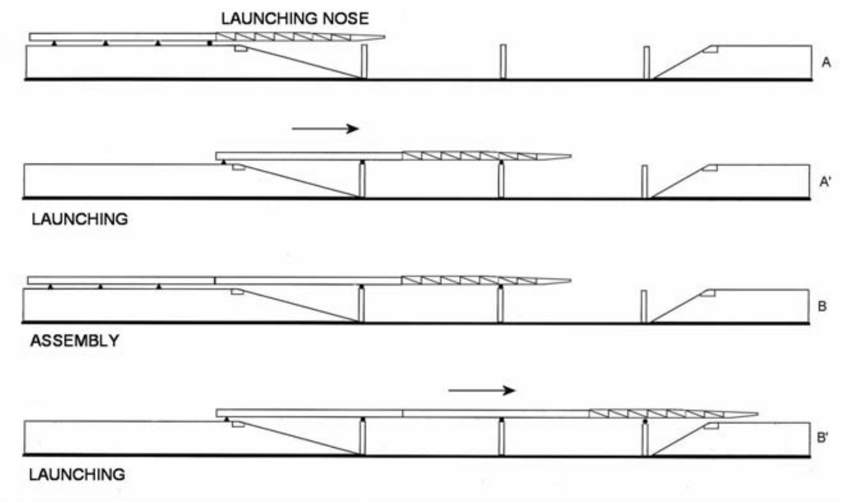


Figure 4.3. Steel frame launching principle

The steel frame can be moved by rolling over saddles incorporating rollers or by sliding on skids. The required pulling force is usually generated by winches, less frequently by cable launching jacks or by a thrust frame.

The steel frame is launched, at a higher level than its final level, onto temporary supports called stacks, which effectively comprise a stack of properly stiffened and braced I-sections or H-beams. After moving the steel frame to a position vertically above its permanent bearings, it is lowered onto temporary slab concreting supports (cf. Sub-section 4.3 “Installation on permanent bearings”).

The deck is lowered onto its permanent bearings after concreting the slab.

4.2.2.2 - Launchable steel frame geometries

To be launchable, a steel frame must generally be horizontally aligned such that it can be superposed on itself by rotation or translation, i.e. a straight or circular profile.

For a horizontally curved alignment, it is desirable for the radius of curvature to be constant, but launching a steel frame with a slightly variable curvature remains feasible.

Moreover, steel frames of constant or varying depth or width can be launched.

Special precautions to be adopted when launching a complex geometry (variable depth, width, curvature, etc.) steel frame are detailed in the following paragraphs.

It is, of course, advisable to avoid as much as possible steel frame geometries combining several difficulties (e.g. variable depth and unsuitable horizontal alignment) because such configurations make launching operations very expensive and, above all, much more difficult or even impossible.

When the longitudinal profile is made up of a constant gradient followed by a parabolic arc, the steel frame can be launched from a single abutment, if the radius of the parabola is not too small with respect to the girder depth. Support level differences are in fact induced during launching due to the fact that the launching curve cannot be superposed on itself, but the resulting stresses in the steel frame remain moderate and require no structural strengthening. Conversely, if the intrinsic flexibility of the steel frame does not allow it to take up the imposed deformations, it may be necessary to launch from both abutments, which invariably makes the operation more expensive.

4.2.2.3 - Assembly and launching area

General

The installation operation requires an assembly and launching area extending behind one or both abutments, when launching from one or both sides respectively.

When the steel frame can be launched from either abutment, priority should be given to launching on a rising gradient for obvious safety reasons. If not in contradiction with the above principles, the steel deck should be launched from the side with the shortest edge span and all the more so, when the bridge is composed of two spans.

Setting up of an assembly area at both ends of the bridge is expensive. This is why the steel frame is usually launched from one side and two-sided launching is effectively only implemented for special cases such as relatively long, highly variable depth decks, a bridge with a partly straight, partly curved horizontal alignment, a curved bridge with a point of inflexion, etc.

An especially favorable case would be that of building a bridge on a new road project. Construction of the approach embankments at the right time ensures a good quality assembly formation without additional costs.

Assembly and launching area characteristics

Geometry of the launching area located on the bridge extended axis must be strictly adhered to on site in terms of both horizontal and vertical profile.

The minimum length of an assembly area is essentially equal to:

- twice the length of the adjoining end span for obvious reasons of static equilibrium (otherwise, temporary pier bents must be resorted to),
- the length of first end span to be crossed plus the length of the launching nose.

The optimum length of the assembly area will allow single operation launching for standard length decks or a minimum number of launching phases for long decks. Hence, it is often advantageous for Owners to make large areas available to contractors.

Assembly area width must permit access of transport vehicles and lifting machinery for unloading, storing and assembling the steel frame sections. A continuous access track along the steel frame being assembled is therefore essential in the unloading area. Its width must be at least equal to the overall width of the steel frame (including possible directly supporting cross-beam cantilevers) plus 1.50 to 2.00 m on each side. Ideally, the available width should be 8/10 m on one side to provide space not only for trucks and cranes, but also for the crane outriggers (Figure 4.4).

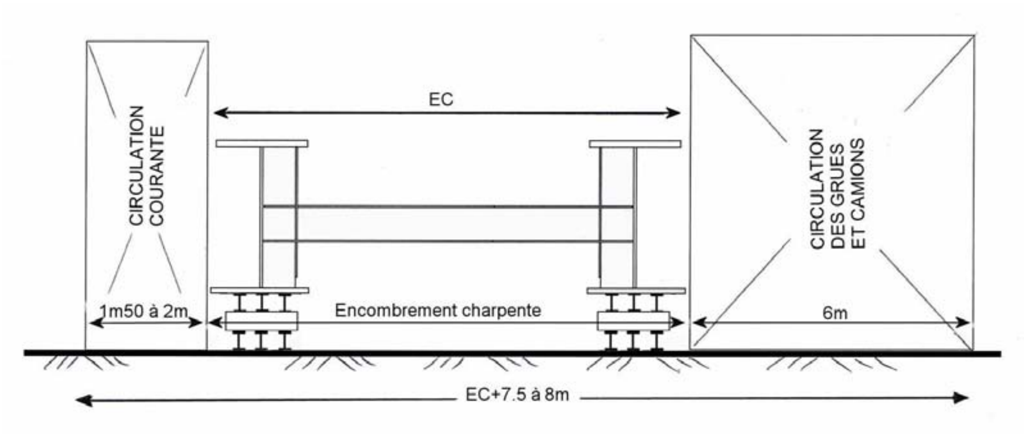


Figure 4.4. Ideal assembly area width

The steel frame assembly area elevation must be as close as possible to the level of the abutment crosshead top face to avoid launching the steel frame too high with respect to its permanent level.

When the bridge approaches are embankments, these should simply be built in two stages; the top of the first stage corresponding to the level required for launching and the top of the second stage corresponding to their final level after construction of the abutment retaining walls.

When the bridge approaches are in cut, the most economical solution involving excavating the assembly area (which may prove to be extremely expensive, e.g. in rocky ground) or launching at a very high level must be retained.

Materials used to build the launching area and their degree of compaction must be selected and carefully monitored because of very heavy abnormal convoy movements and the magnitude of the loads exerted by the launching supports on the supporting formation.

The assembly area must be accessible to steel frame transport and unloading equipment. Commonly required access characteristics are as follows:

- maximum gradient of 5%,
- minimum horizontal radius of 25 m,
- bend outside borders cleared over 5 m,
- bearing capacity: approximate total load 110 t, axle load 10 t.

Assembly of steel frame at assembly area

Steel frame sections are unloaded from the transport convoy at the assembly area using slings hooked into eyes welded on the girder top flanges. They are then placed on assembly supports called stacks, which have been accurately levelled so that the deflections and rotations at the ends of the frame sections are equal and these are therefore aligned to achieve an accurate welded butt joint. Sections limited to girder or beams must be temporarily held safe, e.g. using horizontal tackle or props, if they are unstable in a transverse direction.

In the case of twin girder decks, each main girder is assembled first. These assemblies are butt-welded, starting with the flange welds and finishing with the web to curtail the restraint induced by weld shrinkage (Figure 4.5).

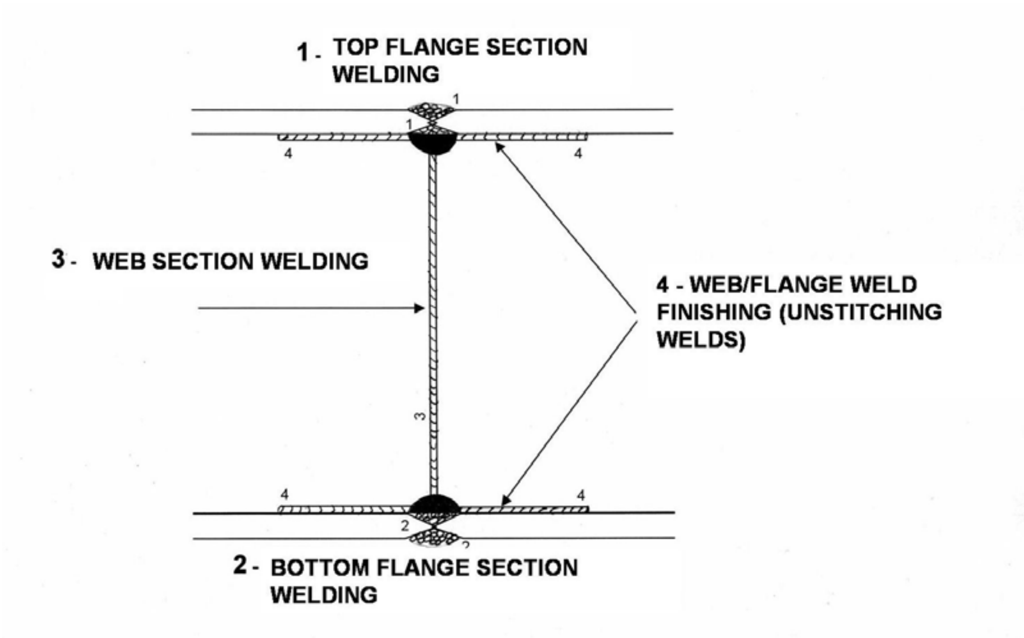


Figure 4.5. Sequencing of main girder butt welds on a twin girder steel frame (steps 1 and 2 can be reversed)

The transverse members are then positioned (cross-beams or directly supporting cross-beams) and assembled with the girder sections by tack welding. These members then ensure assembly stability and the systems or devices used to temporarily hold the different elements can be removed.

In the case of box girders, the sections only need to be butt-welded if they are delivered full width with all their transverse elements. Otherwise, the transverse cross section needs to be reconstructed, requiring expensive longitudinal welding of the bottom flange.

When locating devices have been shop-welded to the steel frame (invariably the case when a physical trial assembly has been performed), bolting of the locating devices at the right-hand end of section n to those at the left-hand end of section n+1 facilitates reconstruction of the required geometry.

When the steel frame has been fully assembled, it is positioned on the launching system by successive jacking operations. To ensure this transfer, jacking operations involving load application/release at the different bearing lines are performed by ensuring simultaneous displacements at the same bearing line; the jacks being connected to the same hydraulic power unit and actuated simultaneously through their displacement interlocking system.

4.2.2.4 - Launching over roller saddles

Principle and technological details

The steel frame moves over roller saddles in most launching operations. These saddles comprise a steel frame and rollers and ensure not only that the steel frame rolls over them under very low friction, but also efficient transfer of load to their supports.

There are two types of roller launching saddles (Figure 4.6):

- rocker saddles, whose bases are articulated to ensure that the rollers, usually arranged in pairs, remain in contact with the bottom flange irrespective of its longitudinal profile,
- cable saddles, in which the roller axles bear on a tensioned, endless cable ensuring uniform distribution of loads to the rollers.

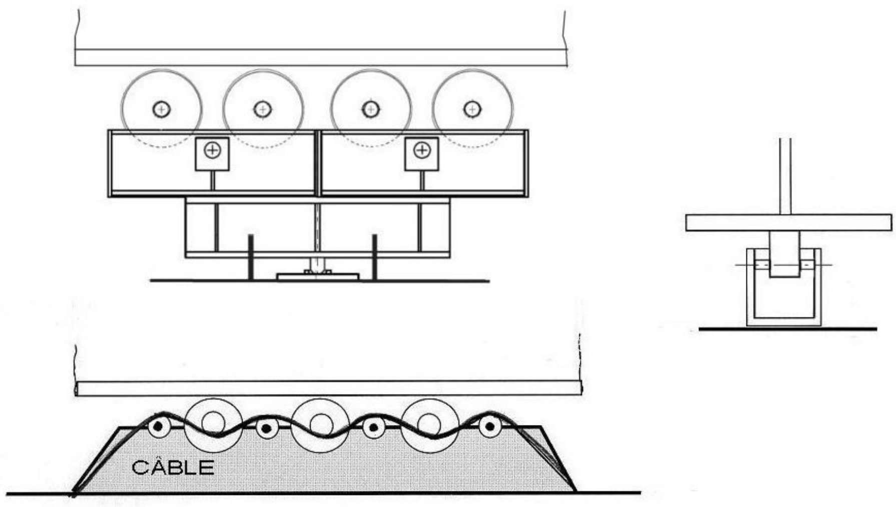


Figure 4.6. Roller saddles: rocker type (above), cable type (below)

Rocker saddles are by far the most commonly used and it would appear that contractors only very rarely resort to cable saddles.

The number of rollers required beneath each web depends on the loads to be resisted, given that the capacity of commonly used rollers is 40 – 60 t per roller, but can be up to 100 t on some rocker saddles. The number of rollers per cable saddle is currently limited to 6, whilst, for rocker saddles, up to 12 rollers can be incorporated. Roller loads (contact or so-called Hertz pressure) must be checked.

The table below gives rough overall dimensions for rocker launching saddles.

Type	Overall length	Overall height
2-roller saddles	1.30 m	0.50 m
3-roller saddles	1.80 m	0.90 m
4-roller saddles	3.00 to 3.30 m	1.25 m
6-roller saddles	3.50 to 4.00 m	1.00 to 1.60 m
8-roller saddles	4.00 to 4.50 m	1.00 to 1.60 m

Table 4.4. Approximate dimensions of rocker saddles

Roller saddles are ideal for medium loads (approximately 300 t per support) and especially for high launching speeds.

Launching precautions

At the launching area, the roller saddles bear on stacks and, at bridge supports, they are either fixed to stacks or directly lashed to the bearing head to prevent them accidentally falling during manoeuvres involving dynamic effects, such as docking at supports. We recommend that these devices (both roller saddles and stacks) be complemented by transverse bracing using props to resist the tangential loads.

Saddle-steel frame contact must be concentrated on the bottom flanges vertically beneath the web to ensure direct transfer of the support reaction load through the web. If this contact is eccentric with respect to the web, the I-section girder bottom flange would be subjected to significant lateral rotation. When the launching devices for a box girder have to be transversely positioned rather far away from the web axes, a T-section (called a launching tee) is provided at the launching support (Section 3).

The launching devices must be adjusted to ensure perfect distribution of loads to the various points of support. Improper load distribution to rollers on the same saddle or between saddles on the same support line can effectively cause significant load increase in a web that has not been designed for this.

Accidental steel frame movements must be prevented during each stop between launching stages, e.g. by inserting steel wedges between the bottom flange and the saddle.

4.2.2.5 - Steel frame launching by sliding on skids

General

Some manufacturers have recently turned to skid systems for sliding the steel frame.

These systems are based on one or other of the following principles:

- sliding of the steel frame bottom flange on skids fixed to the supports,
- sliding of skids solidly fixed to the steel frame bottom flange over the supports.

The first system is more frequently used because it is suitable for small and medium size bridges.

The second system is less used and is suitable for heavy steel frames or those launched with part of the slab. It is implemented to ensure that the skids can always be located at the vertical posts and stiffeners to prevent the occurrence of a buckling phenomenon under particularly high concentrated loads.

Case of steel frame sliding on skids fixed to supports

The supports are fitted with saddles articulated on bearing shells (called rocker mountings) beneath each girder, which take up steel frame rotations during launching (Figure 4.7). These saddles are placed on stacks at the launching area and are fixed directly to the bearing heads or placed on stacks at the bridge supports.

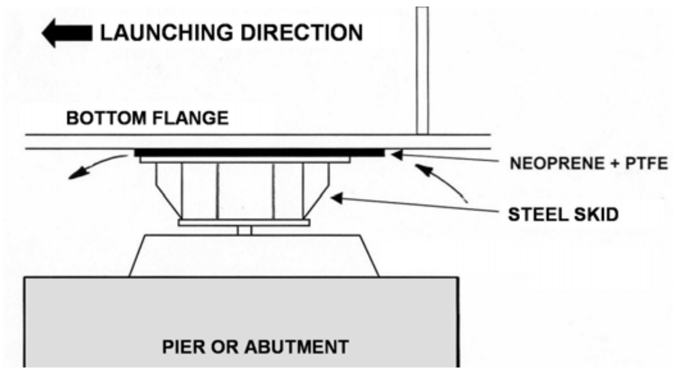


Figure 4.7. Sliding skid fixed to support

Several configurations are possible for ensuring sliding action during launching:

- either PTFE skids are introduced between the rocker mountings and the girder bottom flanges, in which case the top faces of the rocker mountings are plated with polished stainless steel to reduce the friction coefficient (the sliding parts can also be lubricated with soap or grease); the PTFE skids are recovered at the front and reintroduced as launching progresses,
- or the top faces of the rocker mountings are pre-fitted with a sheet of PTFE glued to an elastomer support bearing and the steel frame bottom flange then slides over the saddles lubricated with, for example, soft soap or grease to reduce the friction coefficient; a stainless steel sheet can also be inserted between the steel frame and the saddle instead of applying grease.

It should be noted that some manufacturers have the girder bottom flanges (painted) slide directly over the skid PTFE without resorting to “soap cakes”.

Rocker mounting lengths are designed for the supporting reactions. It should be remembered that rocker mountings can support approximately 450 t/lm (exceptional launching of the Verrières viaduct, in which the rocker mountings subjected to the highest loads were up to 3.80 m long). A spring system also ensures proper load distribution over the support length.

Case of skids solidly fixed to the deck sliding over supports

In this case, the bridge deck is fixed with respect to the sliding skids, i.e. each skid is temporarily fixed to the steel frame at a post (Figure 4.8). The bottom face of these skids are fitted with an elastomer support bearing, beneath which a PTFE sheet is fixed. At the launching area, the skids bear and slide on concrete stringers, whose top faces are plated with stainless steel sheet. The bearing heads are also fitted with a stainless steel slideway, along which the skids travel.

The steel frame is then pushed or pulled to the end of its travel and then jacked up to release the skids and move them back to their starting positions. The steel frame is then relowered onto the skids to begin the next pushing or pulling cycle.

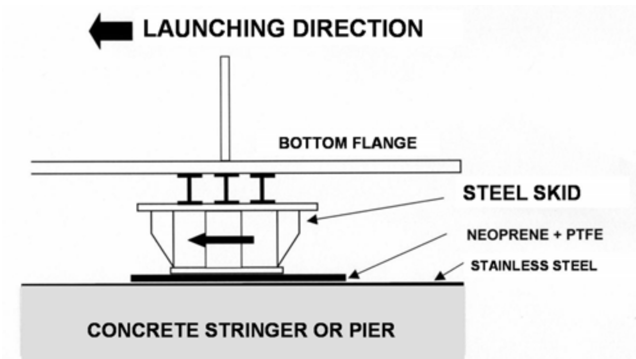


Figure 4.8. Sliding skids solidly fixed to deck

This method requires large support crossheads.

Other points

Skid-based sliding systems have the advantage of providing a larger contact area, allowing greater transverse leeway for the launching operation and a higher bearing capacity than roller saddle systems (the latter characteristic is especially essential for launching the steel frame with the slab).

4.2.2.6 - Steel frame pulling with winches

In most cases, the steel frame is pulled by an electric winch through a pulley block (Figure 4.9) as many times as the load requires (the block being a pulley arrangement used to multiply the pulling force).

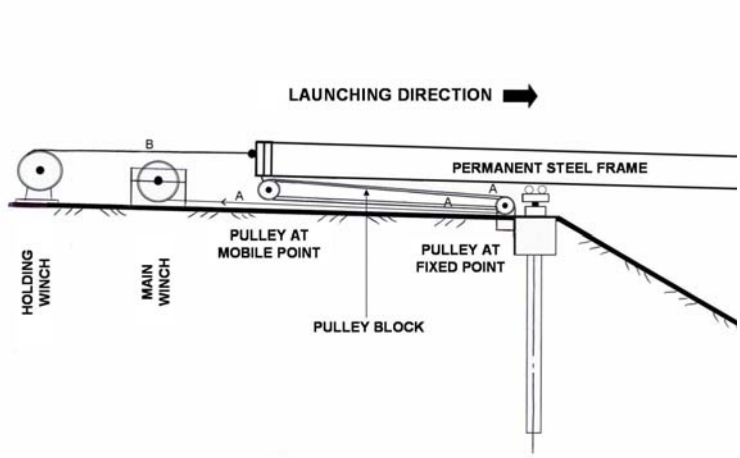


Figure 4.9. Principle of pulling with a winch

Main winches

Main winch capacity is usually 10 t per line (although this can reach 35 t in exceptional cases), whilst the pulling cable is often reeved for 8, 10 or even 12 lines. The developed traction force, equal to the winch capacity per line multiplied by the number of cable block lines, is therefore approximately 100 t in common cases. Its purpose is to overcome:

- friction between the steel frame and the launching saddles,
- friction between the pulling cables and their grooves,
- the bridge overall longitudinal gradient,
- the local bottom flange gradient at each line of saddles, which can rise and fall in succession depending on the camber profile and possible girder depth variation,
- the gradient associated with bevel wedges passing over the saddles, if they have been welded to the undersides of the girder bottom flanges before launching (which is not recommended, cf. section describing installation on permanent bearings).

In general cases, in which the steel frame is launched along an upward slope, the gradient of the longitudinal profile is limited to 5/6%, although this value should be adjusted based on the tonnage to be launched.

To advance the structure, we must attach:

- one pulley (fixed point) to the launching abutment (or exceptionally to the first pier),
- the other pulley (mobile point) to a cross bar or pulling end, which transfers the load to the steel frame.

Between these two points, the pulley block, one of whose lines is wound onto the main winch drum, shortens as the drum rotates and thereby causes the steel frame to advance.

The pulling end is a small temporary steel frame, several metres long, which is fixed to the permanent steel frame at the end of the last assembled section (Figure 4.10). On completion of launching, the pulling end prevents main pulley block congestion and facilitates removal of the launching equipment. If a holding winch is foreseen, its cable is attached to the pulling end.

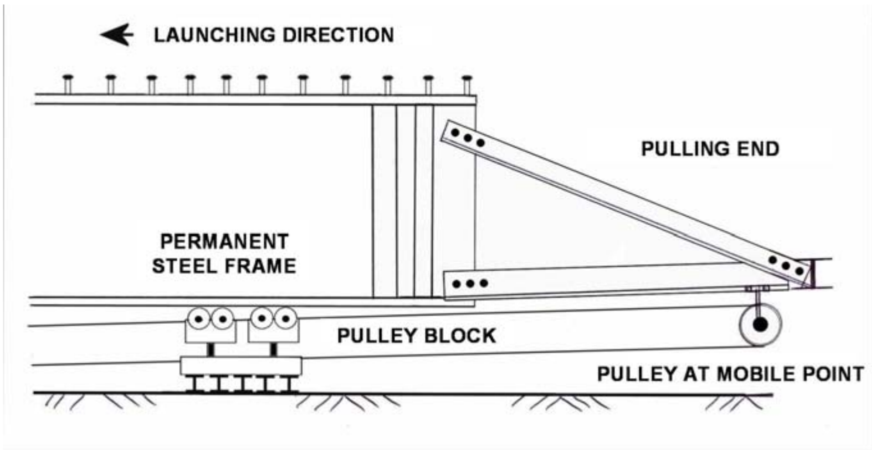


Figure 4.10. Principle of pulling end

Holding winches

When the steel frame is launched along a downward slope, an electric holding winch is usually provided for restraining the steel frame in case of cable breakage or attachment point failure. This winch also allows reversal of the launching operation if necessary and must therefore be suitably sized. The gradient of the longitudinal profile is limited to 9/10% under these conditions, although this value should again be adjusted, based on the tonnage to be launched.

When launching is performed along an upward slope, use of a holding winch is not systematic because safety can also be ensured by doubling the main winch.

The holding winch, which is generally positioned behind the launching area, is anchored by a dead weight buried or stabilised by a rear-mounted counterweight.

The holding winch is locked between two launching stages.

For a variable depth deck, the holding winch is active each time the steel frame tends to descend overall.

If the downward slope is steep (e.g. Monistrol d'Allier viaduct), the steel frame needs to be continuously restrained, whilst the force required for advancement is very small. The number of main winch lines can then be reduced, whilst the number of holding winch lines must be increased. Moreover, a second holding winch may need to be implemented.

Friction forces

When launching over roller saddles, the horizontal loads can be evaluated at 2 – 5% of the steel frame weight. The pulling force to be exerted when starting the launch is reckoned to be 6 – 10% of the steel frame weight (roller friction coefficient plus safety coefficient taking into account force required to overcome the structure’s inertia), although this coefficient should be increased or reduced in relation to the longitudinal gradient.

When launching on sliding skids, the pulling loads depend on the PTFE-stainless steel friction coefficient, which decreases with contact pressure. The friction coefficient is lowered by applying lubricant. Experience from a number of launching operations has shown that, prior to sliding, the elastomer support bearing distorts just before movement begins. The horizontal load should therefore not be underestimated. In practice, we also take into account a friction coefficient of 10% when starting the launch, a value that reduces to 5% once sliding has become steady.

To detect any fault (locked roller, skid installed wrong way round, etc.), a dynamometer or load cell is fitted to the dead line of the pulley block and the real pulling load is thus monitored.

4.2.2.7 - Steel frame pulling by cable launching jacks

The steel frame can also be moved by cable launching jacks. Anchored to the abutment, each jack pulls and draws through itself a prestressing cable fixed to the rear of the steel frame section to be launched (Figure 4.11). If there is a horizontal curvature, a cable deviator can be installed on the formation between the abutment and the rear of the steel frame.

Cable launching jacks are used when the loads to be applied for launching exceed 200 – 300 t. These jacks can effectively exert loads up to 600 t. This type of equipment can also be resorted to when there is insufficient rearward distance for installing a winch.

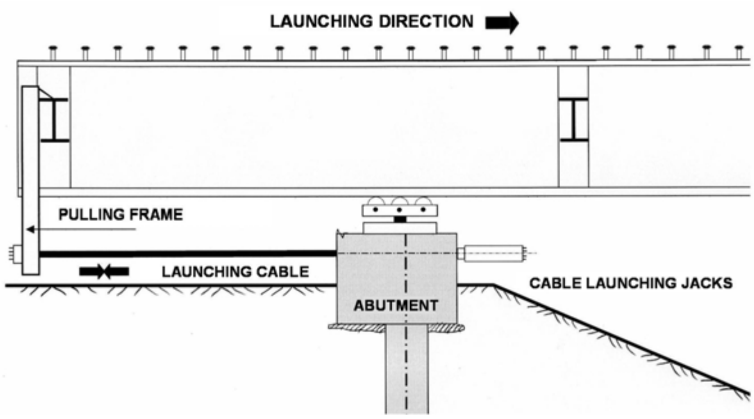


Figure 4.11. Principle of cable launching jacks

4.2.2.8 - Steel frame pushing by double acting jacks

Some contractors have designed a pushing process based on methods developed for pushing prestressed concrete decks. This process implements double acting jacks that push the steel frame through a temporarily fixed steel thrust frame against jack bearing devices inserted into temporary reinforced concrete stringers (Figure 4.12).

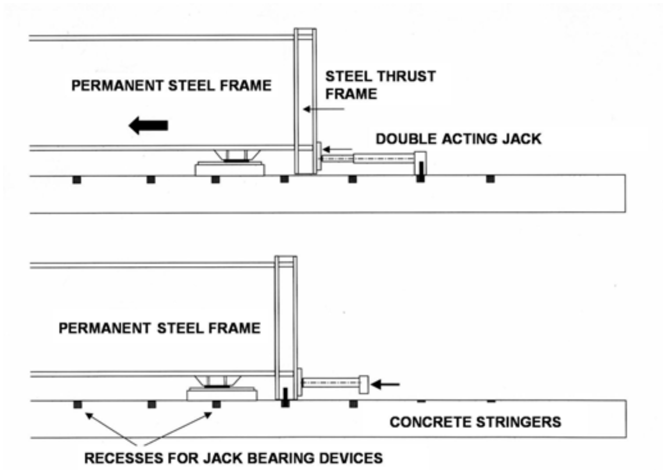


Figure 4.12. Thrust frame and stringers: jack retracted (above), jack at end of travel (below)

In this process, the steel frame slides over the concrete stringers on elastomer plates with their bottom faces clad with a PTFE sheet. These slide along the stringer top face, which is covered with a pre-greased stainless steel sheet.

The two concrete stringers (one under each web) are built in extension of the structure horizontal axis and must be constructed such that they scrupulously respect the levelling dimensions determined by the launching design studies. In addition to providing support at the launching area, the stringers ensure the reaction to the force generated by the pushing jacks. For this purpose, they incorporate recesses approximately every 75 cm, in which the jack bearing devices are located.

One of the advantages of this launching equipment is that it avoids tedious saddle release manoeuvres. On the other hand, using this system, reverse movements, sometimes required by the Engineer to prevent immobilisation over busy roads, are much more difficult than when using a winch system.

4.2.2.9 - Launching nose

Usage conditions

A launching nose fixed to the front of the steel frame must be used for launching isostatic spans and large/medium continuous spans. It effectively fulfils three essential functions:

- substantial weight reduction of the cantilever part of the structure because the dead weight of a launching nose – 1 t/lm for standard span bridges, up to 3 t/lm for very large span bridges – is approximately three times less than that of the steel frame,
- compensation for the large deflection at the end of the steel frame because the underside of the launching nose rises in a forward direction to allow docking,
- for isostatic stage launching, maintenance of the structure's static equilibrium (allowing the next bearing to be docked before tilting occurs).

Make-up

A launching nose is composed of different types of cross-braced beams and cross-beams: lattice beams, solid beams or "hybrid" beams featuring both solid and lattice webs.

Launching noses for standard bridges comprise sections, whose lengths can be adjusted from 5 to 10 m. In this case, the contractor simply has to adapt a head section based on the bridge span and construct a junction segment to ensure connection with the steel frame.

Launching noses are custom built for large bridges.

The beams making up the launching nose are most often fixed to the steel frame by high-strength friction grip (HSFG) bolting onto plates welded at the front end of the steel frame, but they are sometimes welded. This connection must be strong enough to resist loads due to roller or skid forces exerted on the front part of the system (Figure 4.13).

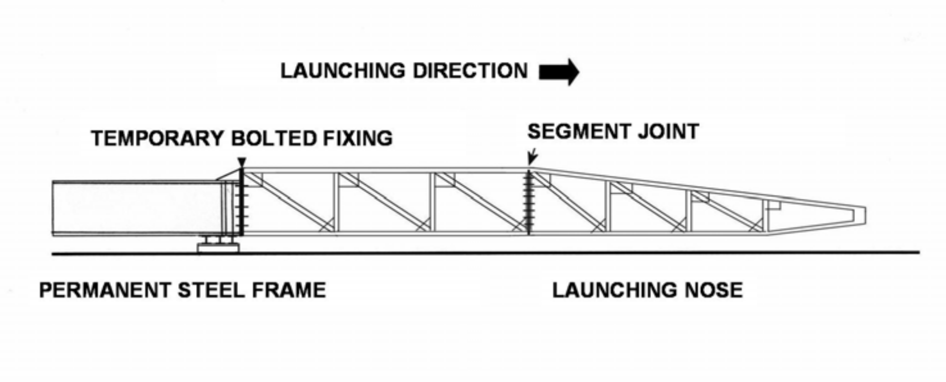


Figure 4.13. Outline diagram of a launching nose

The height difference between the launching nose and the main girders means that connecting gussets must be provided at the girder/launching nose connection to ensure proper load transfer.

Length

The length of a launching nose depends on the spans to be crossed.

An initial approach would indicate that the length must be such that, in a maximum cantilever configuration, the bearing section is a section near to a support in the final structural design.

At preliminary design stage, the launching nose length can be taken as equal to the difference between the lengths of the main span and edge span adjacent to the launching nose. The length of the latter is usually between 15 and 40 m. However, in certain exceptional cases, the launching nose can be up to 60 m long, e.g. the 63.50 m long nose for the Triel viaduct, which has 124 m long main spans.

Implementation

The launching nose is installed at the assembly area prior to the first launching operation. It is disassembled either before reaching the opposite abutment, if the abutment rear retaining wall has been built, or otherwise the common, recommended practice is to disassemble the launching nose after docking beyond this arrival abutment.

The launching nose must be disassembled as it passes over the arrival abutment, if sufficient area for nose disassembly beyond this abutment cannot be provided.

Other points

For curved bridges, the launching nose is usually fabricated using horizontally straight sections positioned to follow the structure's axis as closely as possible. This arrangement causes angular discontinuities at each joint, which are produced by bevel wedges, at which a bracing system must be installed.

4.2.2.10 - Rear launching nose

Usage conditions

In the case of an isostatic span, it may prove necessary to install a rear launching nose to the steel frame during launching. A rear launching nose allows the end of the deck to be moved into place on the departure abutment at the end of launching, whilst overcoming congestion of the traction pulley block. The rear launching nose can then be used to attach the pulling and holding systems, and to support a balancing counterweight or ballast if necessary.

Use of a rear launching nose may also be necessary:

- when the bearing saddle must be positioned behind the abutment rear retaining wall because, for various reasons, it cannot be positioned on the abutment itself,
- for automatically unloading a saddle, an especially useful facility when the launching area is long and there are many saddles to "release" at each launching stage,
- when launching has to be performed "through the piers", i.e. when there is no launching areas behind the abutments (cf. section on special launching operations).

Make-up

In common with the launching nose, the rear nose is a temporary steel framework, whose beams and cross-beams are braced and welded or HSFG-bolted to the permanent steel frame. It is between 5 and 10 m long (Figure 4.14).

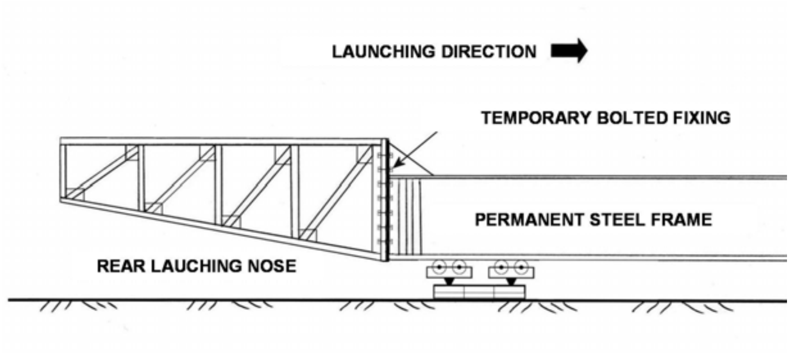


Figure 4.14. Outline diagram of a rear launching nose

When the steel frame is launched in successive stages, the rear launching nose must be removed at the end of each launching operation to allow assembly of the following steel frame sections; it must then be refitted for the next launching stage.

4.2.2.11 - Ballast or counterweight

When launching involves isostatic stages, it may be necessary to install ballast at the back of the steel frame or on the rear launching nose. Its weight is designed to prevent overturning of the steel frame before the docking, and to control the bearing reactions.

This ballast must be accurately centred on the bridge axis and be fitted with anti-slip devices in both longitudinal and transverse directions. It is sometimes formed by casting a concrete slab section or cross-beam at the back of the steel frame.

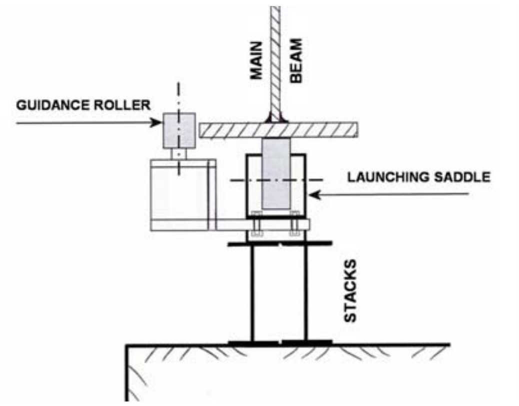
4.2.2.12 - Lateral guidance devices

Lateral guidance prevents the steel frame from unseating from the launching support devices and ensures that the saddle bearing locations remain beneath the webs.

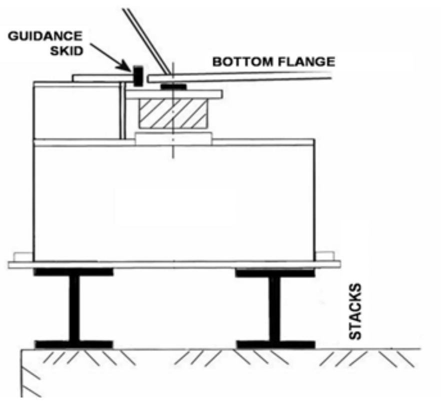
These lateral guidance devices must be fitted to at least two support heads, usually including the launching abutment, based on the fact that the distance between heads provided with guidance must be as great as possible. The exact number of guidance points depends on checking of the lateral guidance devices under wind loading, when launching is stopped.

Lateral guidance can be ensured by (Figure 4.15):

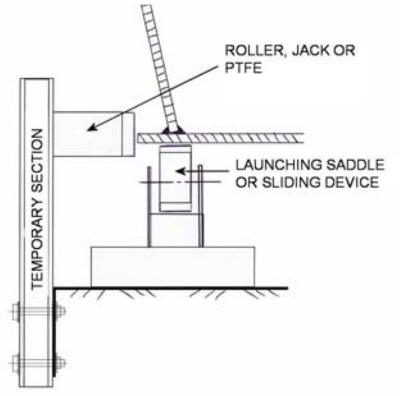
- either a roller with no vertical groove, solidly fixed to the roller saddle and bearing on the edge of the bottom flange (Case 1)
- or a steel frame, adjustable in height and width, fixed either to the rocker mounting (Case 2) or to a steel section temporarily lashed to the support head (Case 3), this frame bearing on the edge of the bottom flange through a PTFE skid,
- or by combining a rail welded to the underside of the bottom flange and grooved rollers (Case 4), the groove depth being less than that of the rail.



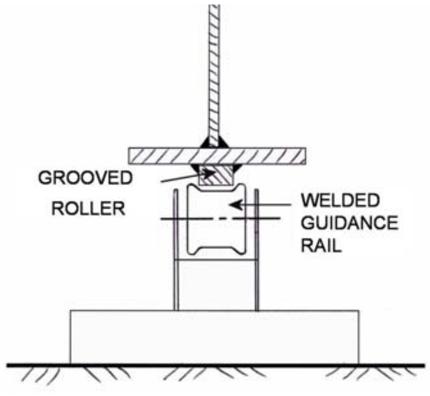
Case 1



Case 2



Case 3



Case 4

Figure 4.15. Lateral guidance details

In the case of the vertical axis roller solidly fixed to the saddle or the frame fixed to the rocker mounting, the lateral guidance system only performs its function effectively if the saddle or rocker mounting supports a sufficiently large vertical load. Lateral friction is then very low compared with the support reaction. On the other hand, if the support is subjected to unloading (foreseen or untimely), a disturbing force (winch skewness, side wind, etc.) may cause unbalancing of the saddle or rocker mounting in the deck transverse direction. In this type of configuration, it may then be necessary to fit a temporary lateral stop lashed directly to the relevant support head.

Moreover, implementation of a launching rail welded beneath the girder bottom flange has become somewhat obsolete specifically because of fatigue-related problems. This solution is now only found on complex geometry bridges, such as variable depth and flange box girders, and on bridges on which lateral guidance is very difficult to install (very deep cantilevers, etc.). If launching rails are permanently installed on the steel frame, they must be continuous and designed for the loads related to their participation in bridge operation. The ends of these rails must then be machined down to prevent fatigue phenomena and stress concentrations.

Finally, the rail sections in bearing areas must be removed to be able to weld the bearing and jacking plates before installing the deck on its permanent bearings.

In the case of guidance by lateral rollers with no groove, a maximum tolerance of 2 cm is incorporated to limit steel frame transverse drift when advancing: the gap between the edge of the bottom flange and the guide roller can then vary from 0 to 4 cm. Saddle roller width must therefore be at least 6 cm for them to remain directly beneath the web.

Guidance devices must be designed for wind loading, taking into account a mimimum transverse load corresponding to 1% of the bearing reaction.

Passing of the launching nose must be handled very carefuly. The bottom flange of the launching nose is not usually located in extension of the bottom flange of the steel frame, but is in fact offset inward. This means that lateral guidance of launching beam flange must be dismantled and offset to adjust its position to that of the steel frame flange, when the front of the launching nose is approaching. It must therefore by ensured that another guidance location must be provided for full passage of the launching nose (or else an additional flange must be provided, when the launching flange starts to pass through).

4.2.2.13 - Temporary pier bents

Usage condition

Pier bents are steel structures used as temporary supports during construction for limiting steel frame overhang, when large continuous spans are crossed or for providing an intermediate support, when launching an isostatic span (Figure 4.16). Pier bents are primary temporary structures and must therefore be designed and built with the same care as the permanent structures.

A temporary pier bent can also be located within an edge span, which extends an insufficiently long steel frame assembly area.

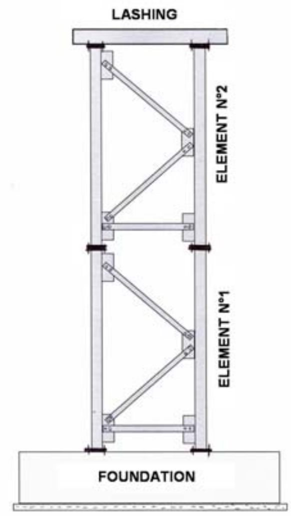


Figure 4.16. Temporary pier bent

Design calculation checks

The temporary pier bent structure must be subjected to detailed checking for static equilibium, bending (compound or eccentric), buckling stability, bracing, etc.

These checks must be performed specifically considering:

- dead weight actions,
- longitudinal loads generated by friction between the steel frame and the launching systems and by thermal effects,
- transverse loads induced by the wind and by deck movement due to horizontal curvature.

For temporary pier bents located in a river, the following phenomena must additionally be considered, depending on the case:

- effect of river current or flood flow,
- river bed loads likely to accumulate in front of the pier bent,
- accidental boat impact, if impact protection is not provided.

Pier bent foundations, which are most often shallow or comprising steel H-beam or cylindrical piles, must be subjected to design calculation checks in the same way as the permanent support foundations. In the case of shallow foundations, special care must be given to settlement, particularly differential settlement, risks.

4.2.2.14 - Cable staying mast

For bridges, whose maximum span distance exceeds approximately 100 m, it may be advantageous to use a cable staying mast in conjunction with a normal length launching nose in place of a very long launching nose. The aim is to minimise loads and especially deformations in the steel frame during launching.

A cable staying mast is a steel structure, which enables the front of the steel frame to be stayed (Figure 4.17). It usually comprises a portal frame fixed to the steel frame, if possible at a pier cross section and its top and bottom remain connected to the portal frame and the steel frame webs respectively.

In recent years, several large composite bridges have been launched with a cable staying mast, in particular the Verrières viaduct (maximum span 144 m) and the Centron downstream viaduct (maximum span 125 m).

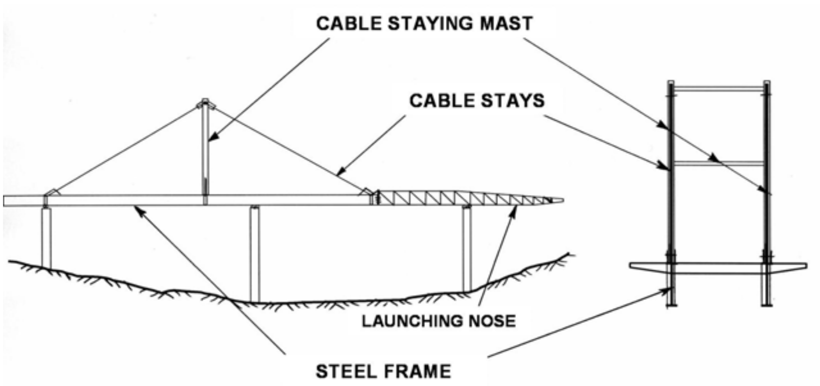


Figure 4.17. Outline diagram of a cable staying mast

4.2.2.15 - Special launching operations

Launching of variable depth bridge decks

In variable depth steel frames, this deck depth variation induces a local gradient in the bottom flanges, which effectively increases the gradient of the longitudinal profile. This local gradient, which should be limited to 10/12% to prevent jamming of the rocker mountings, results in high variations in the required pulling load. Furthermore, launching these steel frames requires a large number of jacking and load transfer operations, which become increasingly major, the greater the depth variation.

This type of launching is performed either “isostatically” on two supports, which then limits the bridge length (e.g. 3-span bridge launched from both sides) or “hyperstatically” on several supports, using the structure’s geometry to advance on the same direction gradients. Launched installation of very long, variable depth bridges can remain viable in this way.

Moreover, roller saddles are more suitable than sliding skids for launching highly variable depth bridges.

Launching of isostatic bridge decks

The main difficulty in launching an isostatic span is to prevent the steel frame overturning. For this purpose, we usually use separately or combined, a very long launching nose, a rear launching nose, a counterweight behind the framework or a temporary pier bent.

Furthermore, large steelwork deflection at the cantilever tip means that the launching devices at the assembly area need to to be adjusted to a steep upward gradient. We recommend accurately sloping the assembly area formation level to prevent excessively high (therefore dangerous) assembly wedges and stacks.

Launching of curved bridge decks

For launching purposes, the horizontal alignment of a steel frame must generally be a curve, which can be superposed on itself by rotation or translation (straight or circular).

However, launching curved bridges raises a number of specific problems. The pier launching supports are in fact not loaded in the same way beneath the deck internal and external girders. Thus, the pier internal support is highly overloaded, when the pier supports a deck with a major overhang. It is absolutely essential that this phenomenon be taken into account, when designing the launching devices. In practice, launching can be envisaged when the radius of curvature is greater than approximately 100 m and the angular range does not exceed 0.2 rad.

On the other hand, if the horizontal curvature is high, it may be very difficult to implement a launching nose because the latter does not correspond perfectly with the steel frame horizontal alignment. In this case, launching stability of the steel frame can also necessitate implementing an anti-torsion balance beam (Figure 4.18) or a temporary pier bent to reduce the angular range, as in the case of the Monistrol d'Allier viaduct.

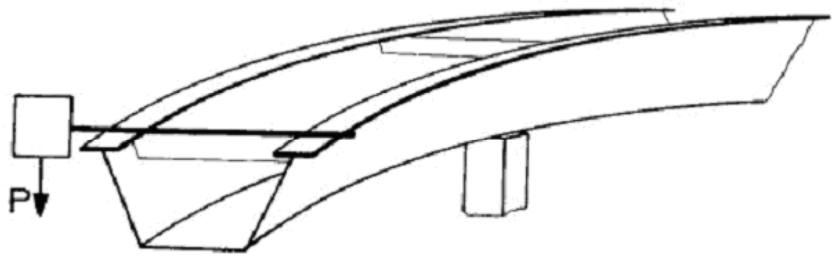


Figure 4.18. Principle of anti-torsion balance beam

In the case of a curved bridge deck, whose horizontal alignment features a point of inflexion, launching may be performed from both sides as long as the radii of curvature of the sections launched from each abutment are constant. If the alignment allows, the point of inflexion in this case should be placed at the centre of a span to balance the final cantilevers in the two launching stages, thereby more easily ensuring continuity of the opposing section tangents by jacking from the adjacent supports (if this continuity has not be adjusted by the camber).

However, variable curvature steel frames can be launched in some special cases. The launching saddles must be capable of crosswise movement to achieve this and they are therefore mounted on trolleys, which slide along a transverse shifting beam. The transverse positions of certain so-called fixed saddles are then fixed to restrict the launching trajectory, whilst the other saddles are completely free to follow the structure in its movement during a launching stage. But, as soon as a launching stage has been completed, all the saddles must be immediately locked in the transverse direction to allow the deck to resist wind action. It should be noted that these measures also enable variable width steel frames to be launched.

These launching saddles can also be moved using jacks anchored to a fixed frame; this was done for bridge OA1 on Lille’s eastern ring road (Figure 4.19).

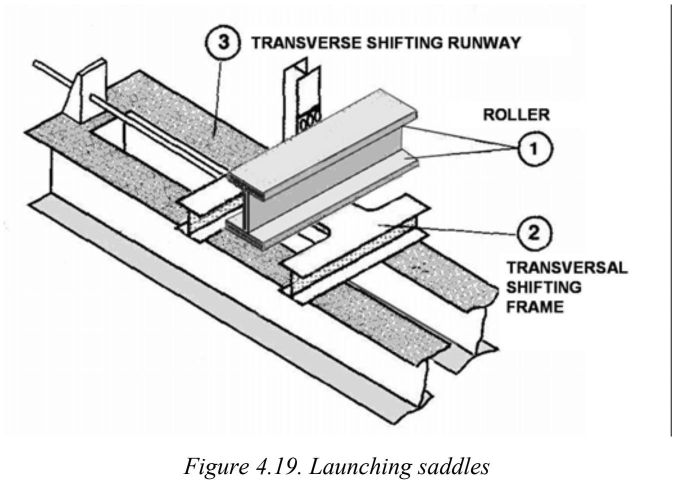


Figure 4.19. Launching saddles
with lateral translation system

The problem of variable curvature can also be solved by launching the deck from both sides and keeping each half-structure on two support lines only, as was done at the Urbino bridge in Italy.

In any case, if the steel frame horizontal alignment is ill-suited to the launching configuration, it may be helpful to ensure that its horizontal alignment is regular and to compensate for differences by playing on the slab overhangs, if the corresponding variations remain of the order of +/-50 cm.

Launching of skew bridge decks

In the case of skew bridges, for which the support lines are all mutually parallel, the most important precautions involve positioning the launching nose and the support lines on the launching formation according to the bridge skewness.

Matters are more complex for skew bridges with support lines that are not always parallel because their spans are then different from one girder to the other. Thus, in keeping with a curved twin girder deck:

- the deflections of the two beams at the end of the launching nose are different,
- one launching nose beam docks at a pier before the other,
- the girders require different cambers,
- the bearing reactions on the launching devices in a common line can be very different.

Launching with a pier bent on a barge or floating pontoon

Principle

Self-propelled barges or floating pontoons may need to be mobilised for launching isostatic spans across certain waterways. The front end of the steel frame is then supported by the barge or pontoon through a pier bent, whilst the rear end rests on a trolley, which rolls over the approach embankment (Figure 4.20).

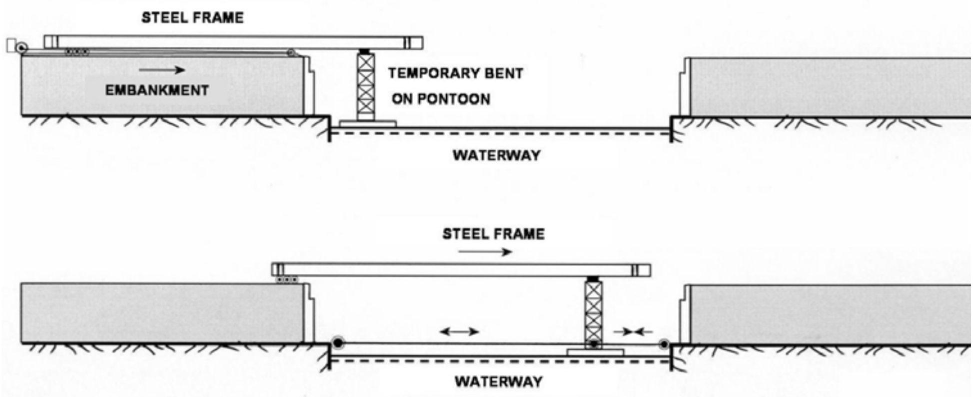


Figure 4.20. Principle of launching over a barge or pontoon (pre-launching and crossing stages)

Kinematics

After temporarily interrupting navigation, the barge is displaced transversely, by to-and-from movements generated by winches and cables, from its initial position parallel to the banks to a position along the launching axis.

Before starting actual launching, the steel frame is positioned vertically above the floating support by means of a preliminary launching operation. Its load is then taken up by deballasting the barge or jacking from the rear support used during preliminary launching, if the latter operation has been performed isostatically. The steel frame is then solidly fixed to both the barge and the pier bent, then the barge is displaced across the waterway.

Once the end of the steel frame arrives above the far abutment, the deck is placed on temporary supports by ballasting the barge or jacking beneath the directly supporting cross-beam or end cross-beam.

Launching on a pontoon pier bent should only be performed on fairly calm canals or rivers, for which a bathymetric survey is recommended. Moreover, the frame-bent-pontoon combination must be solidly fixed by efficient lashing, which usually involves connecting the pontoon to the steel frame by cables (Figure 4.21), the pier bent also being solidly fixed to the pontoon. These precautions effectively prevent relative movements between the steel frame and the pontoon, in particular unwanted listing of the latter.

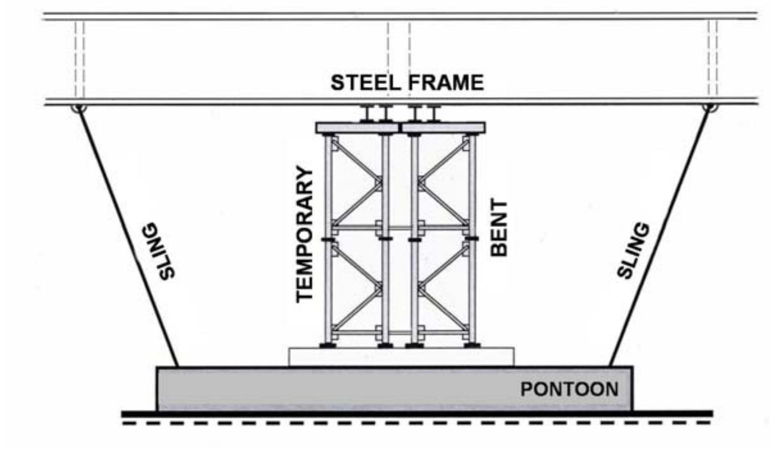


Figure 4.21. Lashing of pier bent to pontoon

During launching, the structure remains in bearing according to its permanent structural design, which is particularly advantageous for launching steel frames integrating their slab or ancillary equipment because this process introduces no tensile stress in the slab.

Launching on a runway or with self-propelled trailer modules

A variation to the method described above can be used for launching isostatic spans across very busy roads. In this method, after a similar preliminary launching stage, a pier bent at the front takes up the steel frame load and is displaced either along a runway installed on the road to be crossed or by self-propelled trailer modules fitted with hydraulic jacks on each axle (Figure 4.22).

Launching operations and, if necessary, runway installation and removal do not usually take more than one night, which reduces traffic disturbance to a minimum.

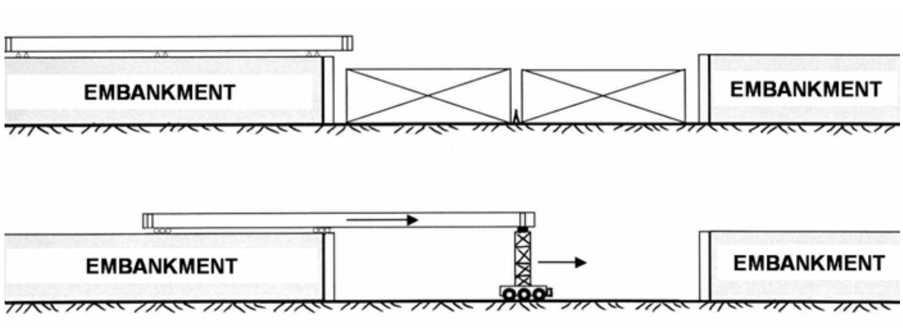


Figure 4.22. Outline diagram of launching on self-propelled trailer modules

Installation without launching nose

For small spans, the steel frame material distribution sometimes allows us to avoid using a launching nose; in this case and as in launching with a nose, the steel frame must be launched at a high enough level to allow straightforward docking of the next bearing by taking into account:

- the steel frame deflection at the cantilever tip,
- level adjustment due to the longitudinal profile,
- possible support vertical adjustments,
- clearance required at bearing plinths, if these have been installed before launching.

For safety reasons, stacks must be limited in height to 2 m and, as for lower wedging used when launching with a nose, it must be anchored on the supports and inter-braced.

Launching with the slab reinforcement

Launching the steel frame with its slab reinforcement can prove advantageous in certain special cases, in which reinforcement cage handling is to be avoided, especially above traffic routes in service such as electrified railway lines or very busy roads. Moreover, if the slab concreting-related constraints are acceptable, launching with the slab reinforcement can represent an attractive alternative to launching with the slab, which is considered below.

All the same, this option should not be inappropriately adopted because the weight of the slab passive reinforcement can reach nearly 50% of the steel frame dead weight in certain cases. Launching device capacity shall therefore be significant increased and horizontal loads during launching are proportionally greater, leading to higher bending stresses on both supports and foundations during construction. Furthermore, the presence of reinforcement on the steel frame makes it impossible to use mobile working platforms travelling on the girders to dismantle launching devices on the piers and complicates removal of launching equipment on these supports.

This method must therefore be reserved for cases really requiring it and should never constitute a standard solution. In any case, if it is implemented, we recommend not fixing reinforcement to the first span to avoid overloading the cantilevered part of the deck.

Launching with concrete slab

Area of usage

A steel frame is only rarely launched with its concrete slab. Major, frequently expensive precautions need to be adopted under these conditions, so this launching method remains specific to crossing very busy roads or railways, above which slab formwork erection and concreting are prohibited by the road or railway operator. In fact, this method requires that traffic interruptions essential to building the whole deck be generally limited to the few hours required for launching operations. Installation of edge superstructures on the slab sections already cast and steelfixing for the remainder of the slab can even be envisaged.

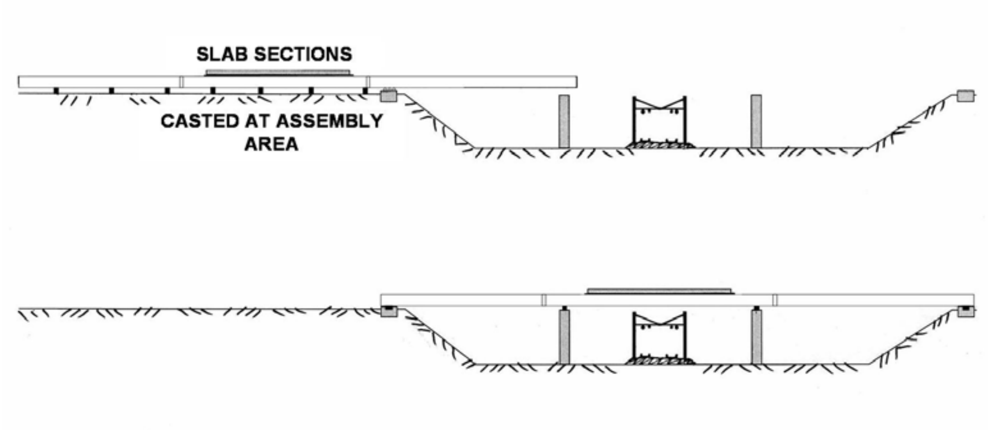


Figure 4.23. Launching with slab sections cast at assembly area

Few bridges have been built using this method. Amongst the most recent, we can quote the Avre viaduct in 1996, the Yonne viaduct in 1997, bridge SD at the Palays interchange in Toulouse in 2005 and, more recently, the French-German Erich Dilger bridge between Fessenheim and Hartheim.

Technical and technological considerations

In general, launching a steel frame with its concrete slab is a much more difficult, expensive operation than conventional launching. The weight of the structure moved is much higher, so precautions must be taken to curtail bending in the steel frame, prevent web elastic instability phenomena and control cracking of the slab, which is usually connected to the steel frame prior to launching. In particular, the end of the slab that extends as a cantilever during launching should not be concreted.

Very high capacity, oversize launching saddles have to be used. Moreover, the combined structure is much stiffer than the steel frame alone, which makes the deck more sensitive to bearing geometrical inaccuracies and increases the risk of load differences between the saddles on the same support.

When launching a steel frame with a full concrete slab (which can be justified only for small-span distances) is desired, the permanent steel frame should be used as a launching nose and a deck length of approximately 20 m should not be concreted. This curtails the length of cantilevered steel frame and slab, whilst preventing the need for a very long launching nose. A small launching nose for docking purposes can then simply be mounted on the end of the steel frame.

Launching with the concrete slab can be performed using roller saddles or sliding skids. When skids on stringers are used, the vertical adjustment height for setting on the casting supports is much smaller than with roller saddles. On the other hand, the roller saddle method is 3 to 5 times quicker.

Finally, temporary pier bent installation can prove necessary to reduce loads in the part of the steel frame combined with the slab during launching.

Launching on an existing steel frame

Sometimes, a section of new steel frame is rolled over existing sections on bridges, whose horizontal alignment or deck depth variation is complex, or to avoid bringing very large lifting equipment to site (Figure 4.24).

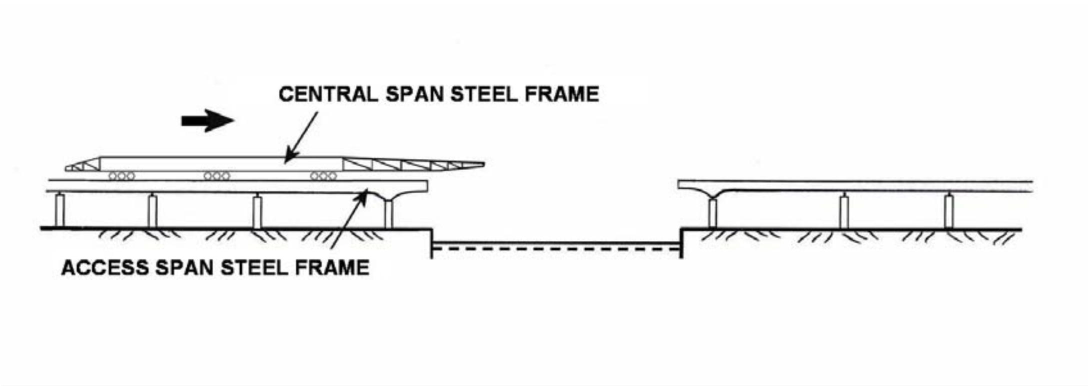


Figure 4.24. Principle of launching over existing steel frame sections

A famous, yet fairly rare, example of applying this method is the bridge on the Nive at Bayonne, for which the main span was completely assembled on the approach embankments on one bank, then launched supported by the steel frame for the land spans, which were previously installed by crane with the starters for the main span.

For this type of launching operation, the front bearing can be a roller bearing comprising an upturned roller saddle welded beneath the bottom flange of the supported girder. The steel frame top flange, used as a runway, must be fitted with guidance devices positioned between the rows of connectors. Once the river span has arrived vertically above its final position, the front and rear launching noses are dismantled and the two span ends are suspended from shear legs for lowering them to their final position.

Launching without an assembly area

Launching steel frame sections can be envisaged even when there is no assembly area behind the abutments. In this case, the sections to be launched can effectively be installed by crane, for example on the first piers and possible temporary piers; all these supports will have previously been equipped with launching devices. The section assembled in this way can then be launched from these supports using main winches anchored at the foot of the piers, for example (Figure 4.25).

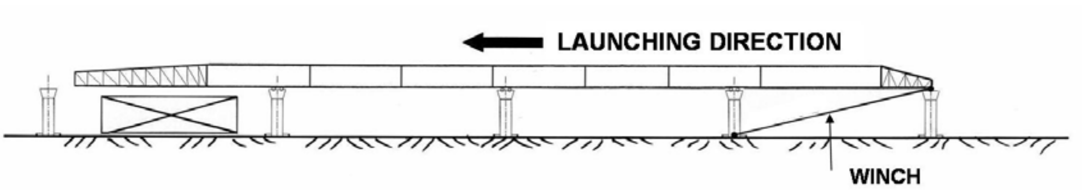


Figure 4.25. Principle of launching without assembly area behind abutments

This method, which closely combines cranage and launching, is particularly advantageous in cases in which the bridge crosses roads, railways or waterways, for which full crane installation would be impossible because of the operating constraints imposed.

Launching multi-girder bridge decks

When launching multi-girder decks, it would be an illusion to believe that it would be possible to achieve uniform distribution of the support reactions over the launching devices, if these were installed beneath each girder. We therefore recommend launching 3-girder or 4-girder decks on only 2 girders as long as the transverse rigidity of the steel frame allows the intermediate launching saddles to be effectively excluded. Three-girder decks must be supported beneath their external girders, whilst 4-girder decks can be supported beneath either the external or internal girders, the latter option usually being retained when the bridge is very skew. Four-girder decks can also be installed by launching two twin girder parallel decks in the conventional way. These will then be connected by cross-beams welded after installation. Moreover, transverse sequencing can be envisaged for launching a multi-girder deck featuring more than four girders.

Finally, it should be stated that, in the very special case of a twin box girder deck, launching can be performed with the deck supported beneath the two internal webs based on the fact that the two box girders will be temporarily or permanently interbraced.

4.2.2.16 - Launching speed

Construction time for launched installation of a conventional twin girder composite bridge deck can be roughly broken down as follows:

- Reconstruction of steel frame at the assembly area: for twin girder cross-beam decks, one week on average per steel frame section for welding the two girder butt joints and cross-beam installation. For twin girder directly supporting cross-beam decks: 1.5 to 2 weeks for welding the two girder butt joints and directly supporting cross-beam assembly.
- Launching: one week per launching cycle, although actual launching operations only take one day for a 50 to 70 m long single span bridge and two days for a 100 to 150 m long 3-span bridge. However, the time taken to set up the winch, adjust the saddle levels and perform intermediate jacking operations must also be taken into account.
- Lowering onto concreting supports: this time obviously depends on the number of supports and the height of the launching stacks because the deck must be lowered in successive, alternate stages from one support to the next. The time required for this operation can be estimated in days worked by dividing the cumulative lowering height for all the support lines by 50 cm. Launching devices must be removed and replaced by wedging before lowering and this requires an additional time of approximately one week per support line.

Under steady conditions and excluding specific manoeuvres, the launching speed is approximately 50 cm/minute, i.e. 30 m/hour, although the instantaneous speed can reach 45 and even 60 m/hour. However, the average speed is much lower because of difficult stages including docking at piers, releasing a saddle at the back, intermediate support vertical adjustments or resetting the main winch pulley block.

4.2.2.17 - Special precautions to be adopted

For safety reasons, a bridge deck can only be launched in very light winds with mean and maximum speeds of 36 and 50 km/h respectively, and this requires working under meteorological control.

Deck installation study

At project and, even more so at construction, design stage, deck launching impacts on the steel frame must be checked, ensuring especially that this operation does not upset the steel frame material distribution.

During launching, the steel frame will in fact be subjected to a succession of structural conditions completely different from those for which it has been designed for the bridge in service. Thus, even though the steel frame is usually only required to support its dead weight during launching, certain stages of this operation may prove to be design significant, especially for lightly stressed cross sections under bridge operating conditions.

In relation to the steel frame, the detailed launching study must specifically identify, right from design stage, the support uplift effects likely to occur and must include checking of:

- its static equilibrium,
- its overall resistance to longitudinal bending and its stability (no tilting of main girders for a girder bridge and no buckling of bottom flange for a box girder bridge),

- web resistance to local effects of launching devices (crushing, local punching, elastic instability under concentrated loads).

At project design stage, the designer can normally check the steel frame fairly easily by considering maximum cantilever phases based on realistic assumptions for launching nose length and weight.

At construction stage, the contractor is responsible for the implementation method and, during relevant design work, the deck installation study must also allow any special equipment and temporary works to be very accurately designed and checked.

Despite its apparent simplicity, deck launching must only be authorised if the contractor submits a detailed calculation and drawing showing the steel frame levels, expected ranges of support reactions, horizontal, vertical or angular movements to be generated and all monitoring and inspections to be performed. This information is especially important for skew or curved bridges, for which the difference between support reactions on the same line of supports can be very significant.

Furthermore, support stability must be ensured by not underestimating the horizontal loads likely to be transferred to the pier heads during launching. These loads depend not only on the friction coefficients of the launching devices, but also the inclination of the tangent to the girder underside, which varies constantly (depending on the final longitudinal profile), fabrication cambers, possible girder depth variations, elastic deformations during launching and re-adjustment procedure.

Finally, on large span (exceeding 90 m) bridges, special attention must be given to wind effects during launching because these can be very significant. If necessary, an anemometer should be installed on site.

Levelling of launching supports

Launching support levels must take into account:

- permanent longitudinal profile of the deck underside and the fabrication cambers,
- general bending or steelwork (at front, to allow docking at next support; at rear, to ensure that steel frame end does not touch the ground after releasing a line of supports).

Support levelling must always be designed to limit re-levelling operations on the launching devices as much as possible. These long, difficult operations must effectively be avoided, especially when crossing very busy traffic routes. In practice, it is often essential to introduce support levelling operations at certain launching stages, either for geometrical reasons or to curtail structural loads. The launching support levels least constraining for the steel frame should then be sought.

Inspection

Temporary steelwork elements (launching nose, rear launching nose, pulling end, pulling cable attachment systems, etc) and installation equipment (winches, cables, pully blocks, roller saddles, sliding skids, etc.) must be checked within the scope of both the contractor’s internal and external inspection obligations. External inspection, performed by a competent independent body, must be based on the design and fabrication of the above elements and must form the subject of a report required by the Engineer.

Moreover, the Engineer must ensure that the Installation Operations Manager has submitted all documents required for lifting any launching-related hold point.

4.2.2.18 - Advantages and drawbacks

Advantages

Launching is the most frequently implemented steel frame installation method because it has multiple advantages and its range of application is extremely wide, as we have seen.

Its main advantage is that it allows gaps to be bridged without special installations, except on the permanent pier heads and behind the abutments. Launching is therefore particularly recommended for crossing:

- poorly accessible gaps such as deep valleys, aquatic sites, etc.,
- fairly unstable or compressible ground unsuitable for supporting lifting equipment,
- traffic routes with very small possibility of interrupting traffic and at which crane installation is unfeasible.

Launching allows steel frame assembly and geometrical adjustment under optimum safety conditions because a maximum number of welds can be run “on the ground”.

Drawbacks

Launching requires extensive technical capability and multiple specific equipment items (main winches, launching nose, roller saddles, etc.), which can be expensive to mobilise, deploy and remove.

The process also requires steel frame jacking operations, which are difficult and labour intensive.

The time to install the steel frame is longer than for other installation processes.

Launching allows no overlapping of steel assembly and concrete slab operations, except for cases in which the steel frame is launched with all or part of the slab, formwork or reinforcement, or if the slab is precast.

Finally, launching phases can, under certain circumstances, turn out to be design significant compared with structure serviceability phases and this may require slight additional strengthening of the steel frame.

4.2.3 - Crane installation of steel frame

4.2.3.1 - Principle of crane installation

The crane installation principle involves lifting the steel frame and placing it on its permanent bearings using one or more lifting machines. The elements to be lifted must be as large as possible to limit the number of cranage stages, whilst ensuring a good compromise between the number of elements, their weight and the power required for the lifting equipment.

Crane installation is possible on both a land site, using mobile crane on lands, and an aquatic site, using floating derricks (barges equipped with a lifting system) (Figure 4.26).

Depending on the site, each deck section is installed by one or two cranes positioned in extension of the bridge or on the road or railway to be crossed.

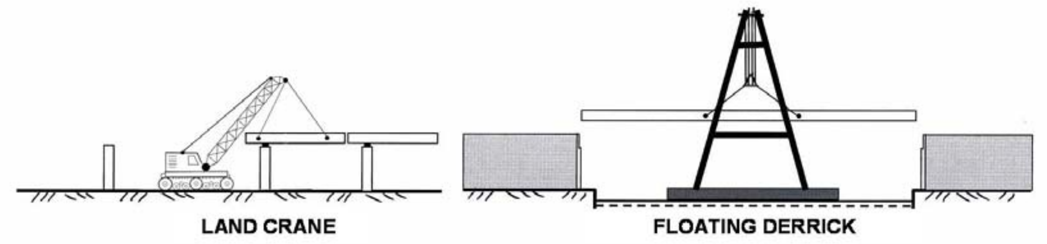


Figure 4.26. Principle of steel frame installed by crane

Depending on its dimensions, transport conditions and lifting equipment capacity, the steel frame can be lifted in whole pre-assembled sections (in the transverse direction), i.e. in one piece, or in elements (main girders, cross-beams, directly supporting cross-beams, etc.), which are then assembled at high level. Steel frame sections are placed on temporary supports.

4.2.3.2 - Lifting equipment and systems

Cranes for land sites

Mobile crawler cranes fitted with lattice jibs (Figure 4.27, right), which can lift heavy loads to a significant height are often used for lifting heavy elements. Their crawler tracks ensure their great stability in relation to overturning, including on low bearing capacity ground. Main disadvantages are their transport and erection, which can last several days, and the most powerful of these cranes need to be reserved far ahead of their usage.

Mobile pneumatic-tyred cranes with telescopic jibs (Figure 4.27, left), which are easily available and whose transport and erection are quicker, can be used when loads are moderate and a free, stabilised track is available for their movements. Given the importance and concentration of loads exerted on the ground by crane outriggers, extreme care should be given to drainage and utility networks likely to be found under the outriggers. Very thick steel plates are therefore very often placed beneath the outriggers to reduce the vertical stress on the underlying ground.

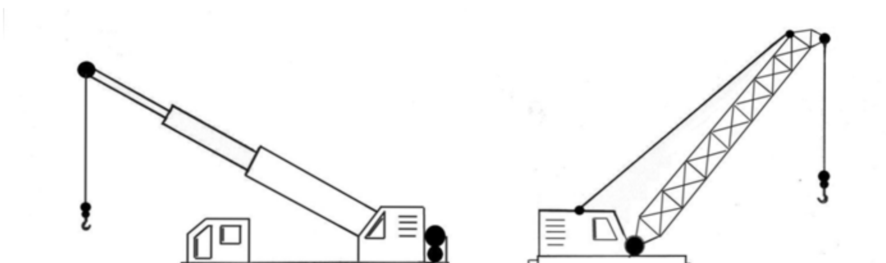


Figure 4.27. Land cranes: pneumatic-tyred with telescopic jib at left;
crawler-mounted with lattice jib at right

The table below provides the main characteristics of a few self-propelled cranes that can be used to lift steel frame sections:

Type	Maximum capacity	Maximum lifting height	Standard capacity(*)	Horizontal dimensions L x l	Weight (excl. ballast)
Mobile crane with telescopic jib	100 t	50 m	10 t	12.5 m x 7.5 m	50 t
Mobile crane with telescopic jib	150 t	50 m	20 t	15 m x 8 m	60 t
Mobile crane with telescopic jib	200 t	60 m	30 t	15 m x 9 m	60 t
Mobile crane with telescopic jib	300 t	60 m	40 t	16 m x 9 m	70 t
Mobile crane with telescopic jib	500 t	50 m	70 t	21 m x 10 m	100 t
Mobile crane with lattice jib	500 t	50 m	80 t	20 m x 14 m	110 t
Crawler crane with lattice jib	500 t	100 m	90 t	11 m x 9 m	175 t
Mobile crane with telescopic jib	800 t	60 m	100 t	18 m x 15 m	100 t
Crawler crane with lattice jib	1250 t	60 m	350 t	15 m x 12,5 m	400 t

Table 4.5. Example of different crane types capacities

(*) *Rough value for a load lifted 15 m high and at approx. 20 m range.*

It should be recalled that the maximum capacity of the cranes quoted in the above table is the maximum load that they can lift in their standard configuration and in their most vertical jib position. In practice, this load is never reached during a lifting operation because elements are always placed with the jib in a more inclined position.

Cranes for river or maritime sites

A floating derrick is a self-propelled barge carrying a lifting device comprising a frame, hinged at the bottom to ensure its variable inclination, that carries a tackle at its top end (Figure 4.28). This type of crane is very powerful and can lift elements weighing up to several hundred tonnes and measuring approximately 100 m long.

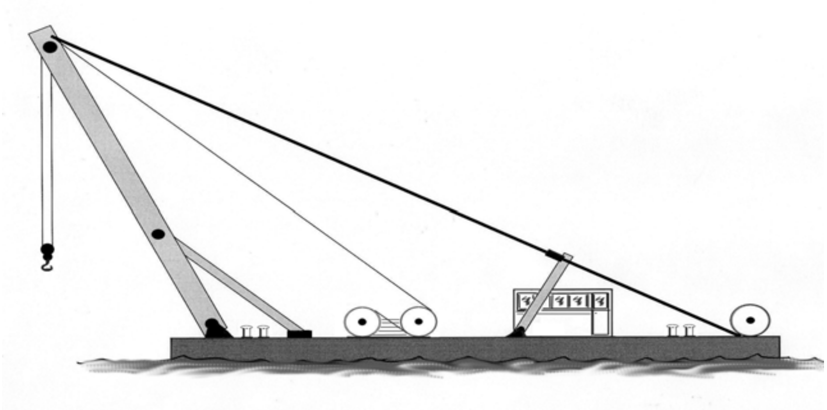


Figure 4.28. Outline diagram of floating derrick

4.2.3.3 - Details of steel frame installation using a mobile crane on land

Area of usage

Installation using a mobile crane on land can be undertaken for steel frame sections up to approximately 60 m long and for bridges of moderate height. If the crane is positioned at the foot of the structure, its height is in practice limited to approximately 15 m due to crane jib length and capacity, given that loads to be lifted must be all the lighter when lifting height is great.

The steel frame sections are placed directly on the concreting supports.

The table below shows a few examples of composite bridges, whose steel frames have been installed by cranage and details the maximum lengths and weights of the sections along with the number and type of cranes used:

Bridge	Length of sections	Weight of sections	Number and type of cranes used
River Rhône bridge at Pont-Saint-Esprit	53 m, one span	54 tonnes	2 lattice jib cranes
5th. bridge on River Nive at Bayonne	48 m	110 tonnes	1 high-capacity telescopic jib crane
Trans-Val-de-Marne western extension	50 m	140 tonnes	1 x 800 t maximum capacity grue
Schengen viaduct	-	380 tonnes	1 high-capacity lattice jib crawler crane

Table 4.6. Examples of bridges installed by cranage

Longitudinal break-down

Different installation sequences are possible depending on whether the section transportable length is equal to or shorter than the bridge main span distances.

In the case illustrated by Figure 4.29, the steel frame sections are roughly the same length as the main spans. In this case, the first section is placed on the abutment on the left and on the first pier on the right. The next section is positioned against the cantilever end of the span in place and on the second pier on the right, etc. The sections are therefore placed as construction progresses and without temporary pier bents.

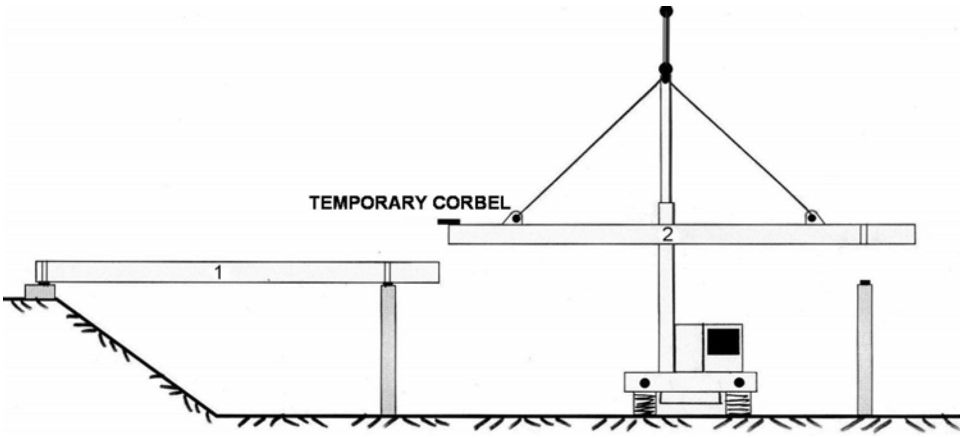


Figure 4.29. Installation sequence using mobile crane for sections roughly same length as spans

In the cases illustrated by Figures 4.30 and 4.31, the steel frame sections are shorter than the main spans. In this configuration, after installing the edge span – using a temporary pier bent if necessary – the next section can be installed as construction progresses by positioning its left end against the right end of the previous section and its right end on a pier bent. Installing the following section allows the first large span to be closed by bearing on the previous section and the first permanent pier (Figure 4.30).

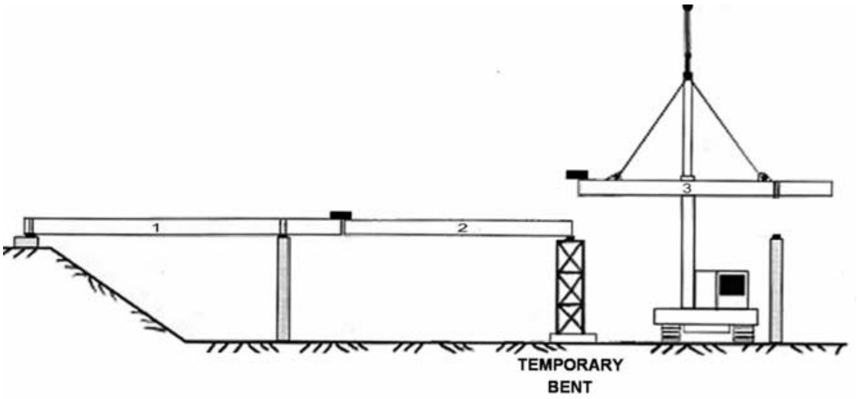


Figure 4.30. Installation sequence using mobile crane for sections shorter than spans / Case 1

The temporary pier bent can also be positioned such that it stabilises the section bearing on the following permanent pier and close the span with a centre section bearing on the ends of the two sections on either side of it (Figure 4.31).

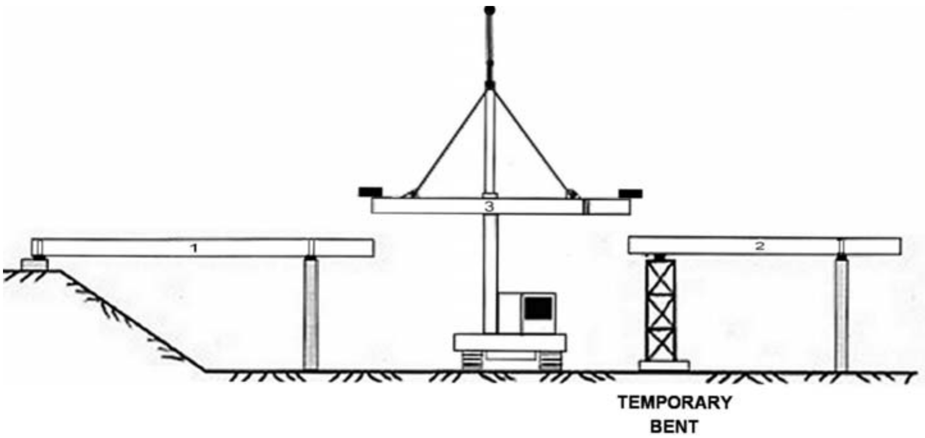


Figure 4.31. Installation sequence using mobile crane for sections shorter than spans / Case 2

Whatever the method retained, the end of a section to be positioned against another section must be shop-fitted with a temporary assembly corbel (Figure 4.32).

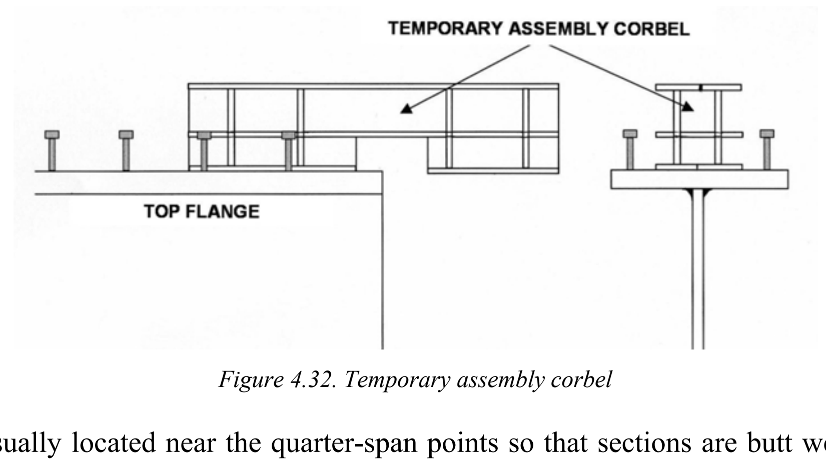


Figure 4.32. Temporary assembly corbel

Section ends are usually located near the quarter-span points so that sections are butt welded in situ (inside cabins suspended from the steel frame) in areas subjected to low stress, when the bridge is in service.

Transverse break-down

When the steel frame is a box girder, the sections referred to above are full steelwork sections.

When the steel frame is twin girder or multi-girder, installing full sections should also be attempted in the transverse direction, i.e. with their cross-beams or directly supporting cross-beams already welded. This provision in fact allows a maximum number of welds to be run on the ground, under good safety and ergonomic conditions, followed by installation of an independently stable structure. However, for large span and/or width girder bridges, this provision is not always possible because it would require very powerful cranes. In this case, we have to resort to installing individually the girders, temporarily bracing them on supports, butt welding them to sections already in place, then interconnecting them by their cross-beams or directly supporting cross-beams. All these operations have to be performed at height.

Installation details

Lifting lugs are temporarily welded to the top flanges of the steel frame sections for handling purposes. These lugs, whose strength and fixing to the structure at this temporary stage must be closely controlled, are usually removed after installation of the bridge deck and their weld seams must be properly ground. Furthermore, non-destructive tests (dye penetrant or magnetoscopic) must be systematically conducted to ensure there are no crack initiations in the top flanges.

Steel frame sections must be steadied to facilitate their docking during the actual lifting operation. The operatives pull on ropes normally attached to the section ends to steady the suspended sections.

When the steel frame is sensitive to elastic instabilities, a lifting beam is introduced between the crane hook and the steel frame to distribute properly the loads and prevent excessive compression in the lifted section. Conversely, lifting can be performed only with slings.

When part of the crane-lifted steel frame has to be butted against a section already installed, one of the opposing ends can be left intentionally too long and final adjustment made on site (butt ends trimmed after surveying the launched section end).

Access track

In the great majority of cases in which a steel frame is installed with a mobile crane, it is essential to first construct an access track over the whole length of the bridge, except if crane lifting is performed “from above”. The access track must be at least 5 m wide, with an extra width of 5 m at places where the crane will be positioned, to allow lifting equipment movement, installation and manoevering. The bearing capacity of the access track must enable it to support the weight of the lifted loads and its surface must allow it to be used by traffic in all weathers.

Installation drawings and programme

For each lifting operation, the installation drawings and programme must specifically show:

- number and type of cranes used,
- crane positions and movements,
- crane jib lengths,
- crane stabilisation systems,

- steel frame section storage location and centre-of-gravity positions,
- maximum range of cranes during lifting and allowable loads w.r.t. inclination,
- load to be taken up.

4.2.3.4 - Details of steel frame installation using a floating derrick

The steel frame section lifting procedure must be very carefully designed to establish the lifting lug positions and determine the floating derrick position for each load picking up and setting down operation.

Lifting lug positions generally result from establishing the best compromise between the length of the slings beneath the lifting beam, the free height beneath the hook and the allowable cantilever bending strength of the steel frame section.

The floating derrick position must be determined to facilitate as much as possible the lifting operation, but interactions with river traffic should also be considered and sufficient water depth should be checked, if necessary by organising a bathymetric survey.

Times for setting down the sections are usually dependent on high tide conditions, when the floating derrick is moved in a river tidal area. In addition to the draught available for floating derrick approach and the height beneath the lifting beam for raising the section, it is advantageous for the floating derrick final approach and setting down of the load to be performed during slack water because this tidal condition facilitates accurate movements of the floating derrick and its suspended steel frame section.

Lowering of the steel frame section onto the bridge temporary supports is generally performed by ballasting the pontoons at the end of the operation or, if not, by the natural movement of the falling tide.

Fine adjustment is then performed using jacks positioned on the supports.

Installing a section usually takes between a half and a full day. It is essential to concentrate operations and minimise their duration, given the constraints involved in deploying a floating derrick (mobilisation, rental, demobilisation, availability of qualified personnel for such operations).

4.2.3.5 - Special precautions to be adopted

General precautions

Prior to undertaking crane operation, the static equilibrium of both the structure and the supports at installation stage must be checked, given the possible positioning errors.

For girder decks, the elastic instability risks at each lifting stage (suspension, setting down, placement on temporary supports, etc.) must also be examined as carefully as possible. If necessary, measures should be taken to increase the transverse rigidity of the steel frame sections.

For box girder decks, these instability risks are much more limited because of the large torsional stiffness of this type of structure (previously fitted with its bracing system).

The stability and resistance of permanent and temporary supports, including their foundations, should also be checked if installation-related conditions will induce loads different to those induced when the bridge is in service.

For safety reasons, cranage can only be implemented in very light winds, usually with mean and maximum speeds limited to 36 and 50 km/h respectively. This requires working under meteorological control.

Special precautions at certain sites

In general, crane operations should be avoided near dangerous aerial obstructions such as electrical power lines, railway catenaries and light overhead structures (walkways, industrial bridges, etc.).

When cranage is unavoidable, the possibility of interrupting operation of these structures or networks for a few hours, so that crane operations can be performed in safety for third parties and site personnel, should be ascertained as early as possible.

4.2.3.6 - Advantages and drawbacks

Advantages and drawbacks of crane installation (mobile crane on land or floating derrick)

Crane-based installation is possible for all bridge geometries, in particular their horizontal alignments.

This represents the installation method that applies the least stress to the steel frame, which avoids modifying the material distribution designed for the phases when the bridge is in service.

The method allows steel frame installation in usually less than one day, which is especially appreciable when the operation requires total or partial closure of the road or railway crossed.

It requires no launching area.

When the bridge is long, section installation and girder welding is faster than concreting the slab, so these two operations can be overlapped as long as the steel frame is maintained sufficiently in advance (approximately 4 spans) for slab concreting-related deformations not to disturb steel frame joint welding.

In the other hand, post-installation operations (butt welding of sections, welding of transverse elements, etc.) are difficult and must effectively be performed at height and under less favorable conditions than at an assembly area. Similarly, inspections on these welds are more difficult to conduct.

Finally, if the natural ground is not flat or of poor quality, the crane and transport convoy movement areas can represent large zones to be prepared and this may significantly increase the construction cost.

Advantages and drawbacks specific to installation using a mobile crane on land

Steel frame installation using a mobile crane on land often represents an economic solution, when the structure is light and easily transportable, the supports are not too high and the operation can therefore be performed with standard capacity cranes.

Its main drawback lies in the requirement to build the necessary access tracks for bringing in, handling and possibly pre-assembling the elements.

Advantages and drawbacks specific to installation using a floating derrick

Floating derrick-based installation is very attractive above waterways because it allows installation of large dimension elements brought in by river. Far fewer welds are therefore run on site, which significantly curtails construction time.

However, installation using a floating derrick has several drawbacks. This method is therefore mainly limited by the very high cost of floating derricks, by the possibility of bringing this equipment to site by river and by the break of load, which is almost inevitably required near the site for a very large element. Moreover, floating derricks are usually difficult to move because of their size and they often require an interruption of navigable waterway traffic. Finally, there are few very high capacity floating derricks in the world and these need to be brought in from far away (from North Sea ports, even from Japan).

4.2.4 - Installation by shifting

4.2.4.1 - Principle and area of usage

Bridge deck installation by shifting involves assembling the steel frame at a location parallel to its final position, at a level very near to its final level, and then sliding or shifting it sideways using cables or jacks (Figure 4.33).

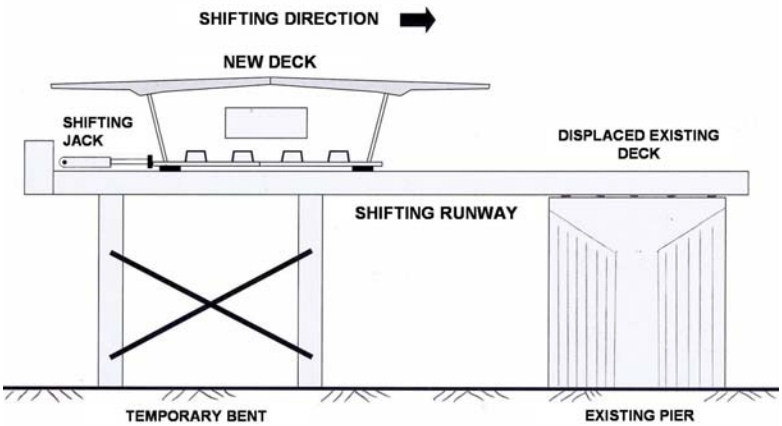


Figure 4.33. Principle of installation by shifting

This method is little used, but is particularly suited to replacing an existing bridge deck, for which we wish to both maintain the location and disrupt traffic as little as possible.

It should be noted that it is not always possible to build the deck simply before shifting it. When the structure spans very busy roads or railways, the Owner may effectively prohibit its construction on a temporary supporting structure and impose launching prior to shifting.

4.2.4.2 - Details of steel frame installation by shifting

Installation of a bridge deck by shifting begins with construction of the temporary supports parallel to the permanent supports of the existing structure (most common case of a bridge to be rebuilt).

The new deck is then fully built (steel frame, slab and equipment) above the temporary supports and roughly at its final level. Traffic is then transferred to this new deck, allowing demolition of the old deck and reconstruction of its supports, if need be.

A continuous shifting runway is built between the temporary and permanent supports and this will be used to support the deck during shifting. The top surfaces of this runway are greased to reduce friction during this operation.

Once this preparatory work has been completed, the new deck bears on the shifting runway through bearing devices, which are most often shrink fit elastomer with their bottom faces clad with a PTFE sheet. These slide on stainless steel skids positioned along the top face of each shifting beam.

The new deck is only closed to traffic during its transfer to the shifting runway, the shifting operation itself and the subsequent finishing work. This limits traffic interruption to approximately one day.

Deck shifting movement is produced either through pushing by jacks, which bear against an abutment and themselves advance with the steel frame so need to be moved each time they reach their end of travel, or through pulling by cable launching jacks.

During these transfer operations, jacking forces must be carefully monitored to ensure the deck moves exactly parallel to the lines of support. Horizontal deformations are also closely checked using reference marks on each shifting runway.

After checking the position of the deck on completion of the shifting operation, it is set down on the permanent support bearings and the shifting installations are removed.

Finally, the temporary supports are demolished.

It should be noted that the sequencing suggested here is only an example, other kinematics may be better suited to the site operating constraints.

4.2.4.3 - Advantages and drawbacks

Advantages

Installation by shifting has a number of advantages:

- very brief interruption of traffic on the supported road (approximately one day in standard cases),
- little or no work at height,
- no steel frame weight limitation because of low friction coefficient (5%), allowing shifting of both steelwork, slab and possible deck equipment,
- no formation required behind the abutments,
- no structural strengthening required by assembly, the structure is always in its final structural configuration.

Drawbacks

The main drawback of this method is its high cost, which is burdened both by the high construction and demolition costs of the temporary supports and by the cost of shifting operations. It is also sometimes difficult to find a sufficiently wide area along the bridge to be replaced.

4.2.5 - Installation by hoisting

4.2.5.1 - Principle and area of usage

Installation by hoisting involves bringing a whole section of steel frame vertically beneath its pemanent position, then hoisting it up to its final level (Figure 4.34).

Hoisting is a complex method and its only rarely used because it is only ecomomical in very special cases. In practice, it is mainly used for installing the central part of a span crossing a waterway requiring a large clearance; this central span is generally larger than the others and is not repeated. Launching is unsuitable under these conditions, as is floating derrick installation, which would be too expensive, even impossible. On the contrary, hoisting allows us to lift a section weighing several hundred, even several thousand, tonnes in a few hours; this significantly curtails traffic disruption on the waterway.

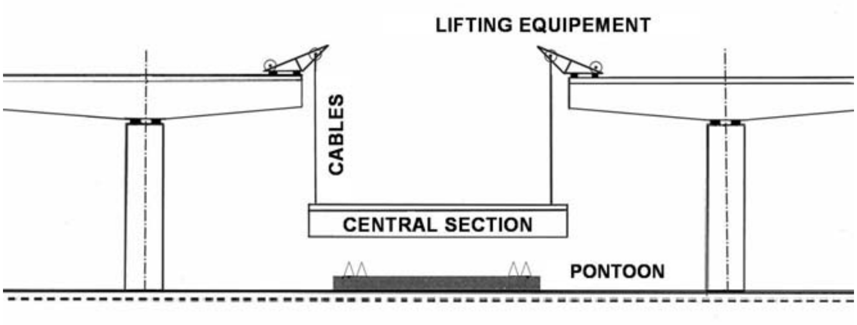


Figure 4.34. Hoisting principle

4.2.5.2 - Installation by hoisting

After installing the adjacent spans, which are often on land, using a more common method than hoisting (cranage, launching, a combination of both, etc.), the central part of the span crossing the waterway is brought in on a pontoon. It is then hoisted by machinery positioned at the ends of the cantilevers extending the spans already in place, as performed for the main span of the Aveyron viaduct on the A20 motorway.

The pontoons can be positioned by winches anchored in the waterway banks and a hauling winch located on the bridge axis can be used for fine adjustment beneath the waiting cables to be used for lifting the central section.

The hoisting operation is performed by cables either drawn by main winches with pulley blocks or by cable launching jacks. Lifting equipment is plated on temporary steelwork fixed to the cantilevers extending the sections already installed, the latter being designed and dimensioned to receive this additional steelwork and to support the loads induced by the hoisting operation itself.

To allow safe hoisting of the central section between the cantilevered parts of the steel frame and ensure proper adjustment of the welded joints, the gap between the ends of the cantilevers can be designed to be a few centimetres wider than the length of the section to be hoisted. To bring the ends facing each other together after hoisting, either the support bearings adjacent to the main piers must be raised or one of the previously retracted adjacent spans must be moved to its final longitudinal position.

As an alternative to this method, the hoisted section can also be designed slightly shorter than its theoretical length and an approximately 50 cm long piece, whose dimensions must be very accurately determined taking special account of thermal effects, can be butt-welded after the hoisting operation.

After adjustment of the joints, the hoisted steel frame section is butt-welded to the starter sections on each side.

4.2.5.3 - Advantages and drawbacks

Advantages

When steel frame installation is performed by hoisting, the main assembly work is undertaken on the ground or at the fabrication shop, thus under optimum safety and quality conditions. The bridged area is usually a very busy navigable river, so the opportunity for assembling the steel frame elsewhere and only bringing it site at the last moment limits interruption of river traffic to one day or even two half-days. Finally, the hoisted elements can be very heavy because the cables and jacks can be multiplied to adapt to the loads to be lifted.

Drawbacks

The elements to be placed are very large, so the equipment implemented is often custom-designed – making its reuse very uncertain – and it must integrate emergency gear. Furthermore, the large size of the elements to be moved imposes very severe safety conditions, which impact directly on the installation cost.

Moreover, hoisting operations – both very complex and requiring particularly skilled work teams - must be performed under meteorological coverage; the wind speed, in particular, must be very low (less than 5 m/s).

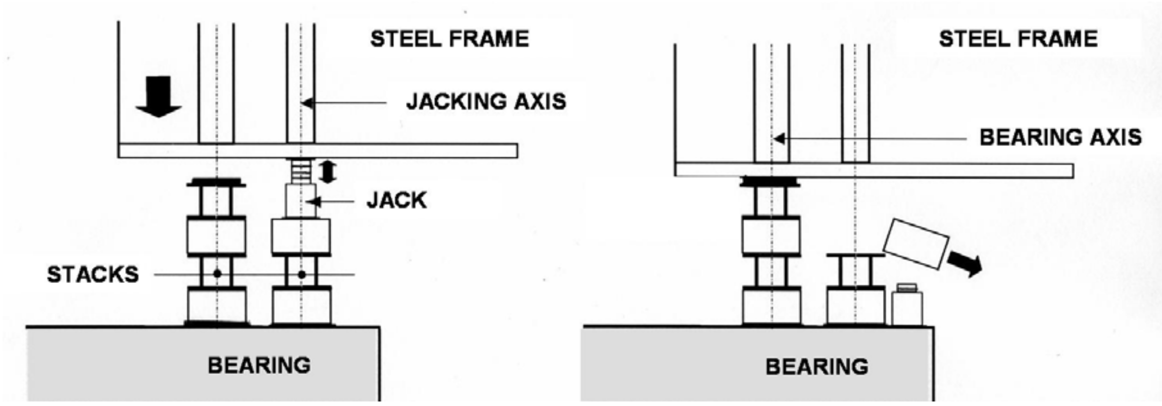
4.3 - Placement on temporary concreting and permanent support bearings

The steel frame is generally placed on temporary support bearings before casting or installing the slab to prevent introduction of unwanted rotations in the permanent support bearings, which would be likely to adversely affect their proper operation or damage them even before they are put into service.

Lowering the steel frame

When handled with a crane, the steel frame is placed directly on temporary concreting support bearings. On the other hand, when the steel frame is launched, it must be first transferred from the launching devices to the temporary concreting support bearings. The steel frame is therefore taken up by jacks, which move it away from the launching devices. It is then placed on stacks before being lowered towards the temporary concreting support bearings.

Lowering is undertaken by a successive dejacking operations and placement on load take-up stacks (Figure 4.35). The dejacking operations are performed in stages, each stage corresponding to the height (15 to 20 cm) of the elementary sections making up the stacks and are identical on the same support because the jacks are interlocked to produce uniform displacement. The required differential levelling deviation between the girders must be computed on a case-by-case basis because it depends on the torsional stiffness of the steel frame: the stiffer the frame (twin girder/box girder), the smaller the levelling deviation. A value of 25 mm can be considered a usual order of magnitude, but this cannot be generalised.



Steel frame lowering by jacks bearing at jacking axis for bridge in service

Deck now bearing on stack, allowing one layer of sections to be removed beneath jacks and start of a new cycle

Figure 4.35. Lowering of steel frame after launching (diagram shown on abutments)

Sequencing of dejacking between the different lines of support must be followed based on the measures foreseen by design calculation. The movements imposed at one line of support will modify the bearing reactions for the whole structure, so it must be especially ensured that these bearing reactions all remain positive.

Temporary support bearings

The temporary support bearings are shimmed to ensure that the deck is virtually at its final level. They are often embodied by laminated elastomeric support bearings, which have the advantage of being fairly tolerant to deformations and allow significant rotations. They are fitted with sliding devices to give the degrees of freedom necessary to steel frame operation during construction.

When the permanent support bearings are pot bearings, they should not be used as temporary support bearings, especially on abutments subjected to relatively large rotations. Pot bearings in fact have a smaller rotation capacity than laminated elastomeric bearings and damage possibly caused during construction is much more difficult to detect inside pot bearings.

Use of laminated elastomeric bearings can be envisaged during slab construction, if the deck is to be supported by this type of bearing when the bridge is in service. In this case, it is essential to perform support bearing relaxation jacking at the end of construction, at least on the abutments, to release unwanted deformations sustained during the concreting stages. At piers, this operation should be envisaged on a case-by-case basis because deformations are less.

Placement of bevel wedges

Bevel wedge welding is undertaken after casting the slab and before setting the deck on its permanent support bearings. Wedge bevel is determined for the observed rotations, such that the bearing surface is perfectly horizontal. The bevel wedges can also be temporarily fixed beneath the bottom flanges by tack welding them after the launching operations and before the deck is set on its temporary concreting support bearings. They would then be permanently welded after casting the slab as long as their horizontality and flatness has been carefully checked. In this case, the theoretical wedge bevel will have been previously calculated based in particular on the rotations generated by slab concreting.

Launching of the steel frame already fitted with its bevel wedges is not advisable. This configuration in fact requires inclusion of bevelled steel inserts for correcting the level difference between the bevel wedge and the bottom flange, which demands development of greater pulling or pushing force due to the resulting local increase in gradient. This method is particularly unsuitable if the launching operation has, in addition, to be performed with a slab section. Finally, it is especially difficult to remove previously welded bevel wedges, if their horizontality does not comply with the design tolerance ($3°/_{oo}$) after installing the steel frame and casting the slab.

The steel frame is ultimately placed on its permanent support bearings after the slab has been completely cast.

4.4 - Possible support vertical adjustments

Possible support vertical adjustments are made after slab construction has been completely finished.

The bridge longitudinal profile before support vertical adjustment must naturally integrate the design level adjustments, i.e. support level adjustments must be taken into account when calculating the fabrication cambers.

Support vertical adjustments are implemented by successively jacking the different supports, whilst adhering to stages whose amplitude is calculated to prevent cracking of the slab, in particular. These operations must be conducted by taking into account the maximum allowable transverse level differences on the same support. To achieve this, given the torsional stiffness acquired by the newly composite structure, the jack movements must be controlled in such a way as to limit this transverse level difference to a few millimetres, the exact tolerance being determined on a case-by-case basis.

As an alternative to support vertical adjustment on piers, it is possible in some cases to over-elevate the abutment bearings as was done at the Monistrol d'Allier viaduct in France’s Haute-Loire department.

4.5 - Maintaining a fixed point during construction

A fixed point must be maintained for checking the deck final position, during each stage of deck construction (steel frame assembly, slab construction, etc.). Subjected to thermal variation effects (daily day/night, seasonal summer/winter), the steel frame tends to move by sliding on its temporary concreting supports and the deck position can no longer be corrected, once the slab has been cast over a certain length.

This requirement, which is especially important for large bridges, imposes implementation of special measures at the temporary supports and design calculation of not only the equipment used for the fixed point, but also the supports themselves and their corresponding foundations.

4.6 - Related bibliography

Launching

RT [MON 96] [VIL 96] [POI 97] [LEB 98] [CHA 00] [ARG 00] [AVR 01] [CHA 01] [POU 01] [DEM 02] [CAL 02] [MAN 02] [CHA 03] [DUB 04] [DUM 06] [MAR 07]

BOA [FON 95] [VIL02 96] [BAR 06]

OTUA [VIL01 96] [ABI 96] [VIL 99] [DEZ 03] [TAV 04]

Launching with a cable-staying mast

RT [GIL 01]

BOA [BOU 01]

OTUA [GIL 04]

Launching with slab

RT [ROU 98]

BOA [BAR 06]

Installation using a mobile crane on land

RT [COU 95] [MEU 96] [LEN 98] [BOR 03] [HAU 07]

BOA [VIO 08]

Installation using a floating derrick

RT [CAR 00]

401: Transporting a twin girder by road. 402: Transporting a box girder by road. 403: Unloading gantry for box girder transported in 2 halves. 404: Mobile crane unloading box girder transported in 2 halves. 405: Transporting a full twin girder with directly supporting cross-beams by river.

411: Trial assembly at fabrication facility. 412: Locating device. 413 and 414: Assembly and launching areas. 415: Stacks made of bolted steel sections. 416: Stacks made of timber blocks (cribbage).

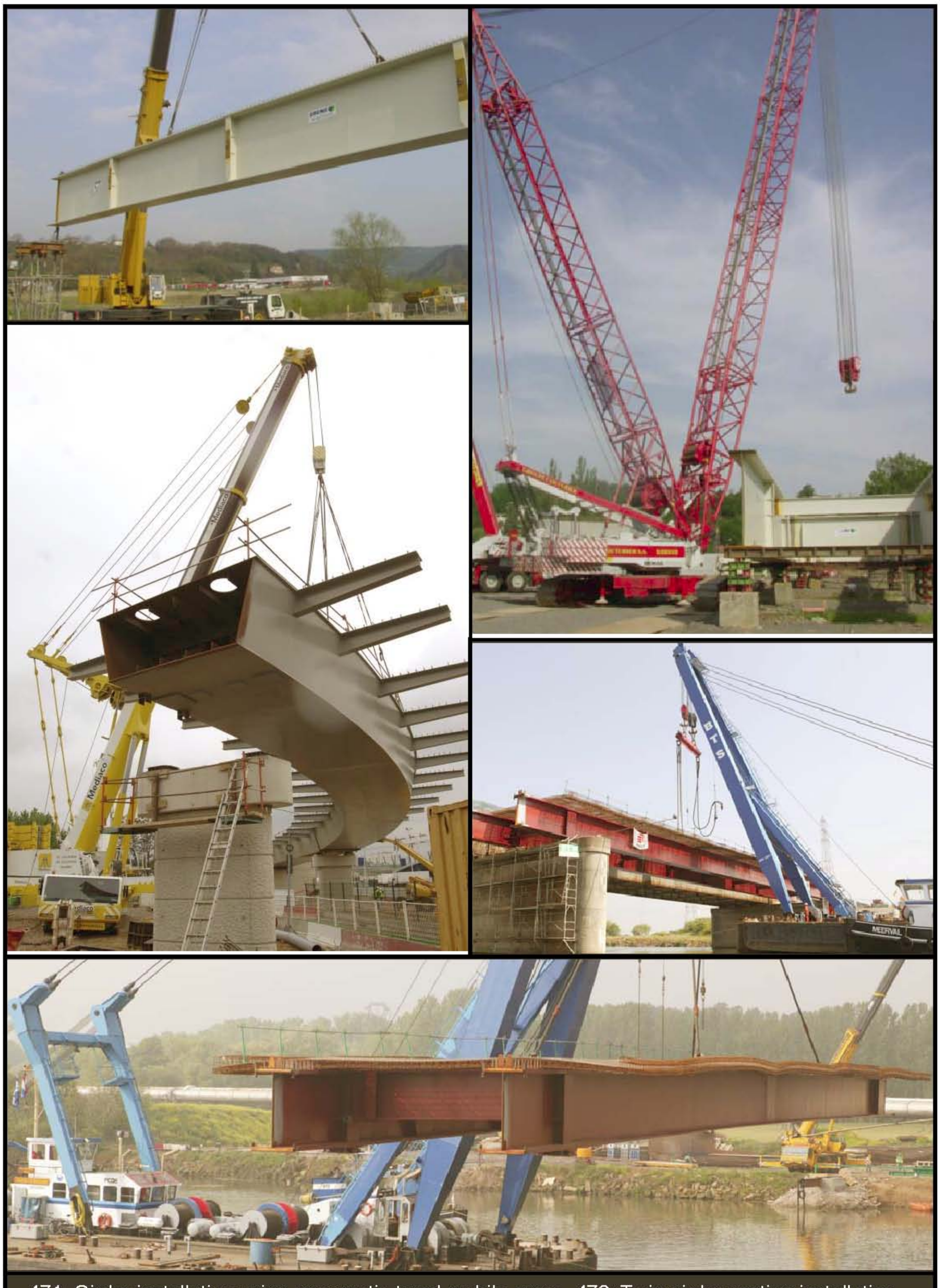
421 and 422: Roller saddles with rockers. 423: Simple roller saddles.
424 and 425: Sliding skids. 426 and 427: WinchesPHOTOS

431 and 432: Pulley blocks. 433: Cable launching jacks. 434 and 435: Pulling ends.
436 and 437: Pushing frame. PHOTOS

441 and 442. Launching noses with solid webs. 443 and 444. Lattice launching nose. 445: Rear launching nose. 446: Precambered combined (lattice/solid web) launching nose.

451 and 452: Temporary bracings. 453: Lateral guides integrated into roller saddle
454: Steel section lateral guide. 455 and 456: Lateral guides and horizontal jack
457: Launching counterweight.

461 and 462. Cable-stay masts.
463 and 465: Temporary stay anchorages on permanent steel frame.



471: Girder installation using pneumatic-tyred mobile crane. 472: Twin girder section installation using crawler crane. 473: Box girder section installation using a pneumatic-tyred mobile crane. 474 and 475: Full twin girder span installation using floating derricks.

481 and 482: Standard temporary pier bents.
483: Very high temporary pier bent for installing a box girder with props.
484 and 485: Assembly brackets. 486: Transverse shifting system.

5 - Slab construction

This section details construction of a composite bridge slab. The first part describes slab construction by in-situ casting using mobile formwork. The second part is dedicated to the use of closed precast slabs and the third part introduces less common methods, such as construction entirely on pre-slabs, installation by pushing slab segments cast in situ behind an abutment or, again, conventional formwork-based construction.

5.1 - Preamble

In this section, the main composite bridge slab construction methods are consolidated into two major families: casting in situ and precasting.

Casting in situ is a method that offers very many advantages. It effectively minimises the number of joints in the slab, allows steel frame geometrical imperfections to be corrected and optimises both the slab reinforcement tonnage and the frame steel consumption.

Precasting also has advantages. It reduces shrinkage effects, which contribute greatly to slab cracking, and allows not only industrialised casting, a priori ensuring better quality, but also quicker slab construction in general. Precasting nevertheless has a number of major drawbacks: reduction in the monolithistic character of the slab, multiplication of potentially weakening closing joints, often delicate installation of cast slab sections, pouring of closing concrete difficult because of reinforcement lapping bar and connector congestion, less good control of final deck geometry, increase in passive reinforcement ratios because of large number of closing volumes, slight increase in steel frame tonnage due to later composite operation.

Given the above factors, casting in situ must be favoured and precasting should only be envisaged in very specific cases: complex geometry steel frames, difficult environments (severe frost areas, nearness to catenaries, etc.), slab construction time to be reduced to a minimum (bridges over very busy roads, railways and waterways, bridges to be urgently rebuilt, etc.). Morever, for these special cases, we recommend drawing up a DCE based on the precasting construction method and providing for all the detailed measures required by this method.

5.2 - Slab construction using mobile formwork

5.2.1 - General principle of method

Slab casting in situ with mobile formwork travellers involves building the slab in situ, by 8 – 20 m long sections (or segments), using equipment that travels on the steel frame.

This method is used to build the great majority of composite bridges because it is ideally suited to structures with simple steel frames, such a twin girders and cross-beams, and these are by far the most frequent.

5.2.2 - Break-down into slab casting sections

For a bridge without directly supporting cross-beams, the slab is broken down into casting sections by considering not only the span lengths, the need to cast slab sections at the piers after those within the spans, but also the construction equipment itself (reuse of existing moving formwork, control of deformations and weight, etc.). 8 to 20 m long slab casting sections have been used in the past, but the most common length is 12 m.

For a bridge with directly supporting cross-beams, the latter elements represent a very major constraint in relation to mobile formwork travellers, so slab casting section lengths are 2 or 3 times the directly supporting cross-beam centre-to-centre distance, i.e. approximately 8 or 12 m.

The DCE [contractor consultation package] usually includes data on the main assumptions adopted by the Engineer (weight of mobile formwork, launching nose characteristics, etc.) as well as the design break-down of the slab into casting sections and the construction sequencing. However, this information is not normally contractual and can often be amended by the contractor as long as slab durability is preserved.

5.2.3 - General design of mobile formwork

A mobile formwork traveller is a temporary structure comprising a steel frame and shuttering platforms, which bears on the deck steelwork.

A full mobile formwork traveller with three shuttering platforms, one for the central section and one for each cantilever section, is implemented for forming the section between the top flanges of the steel frame using this equipment because we do not wish to use permanent formwork, for example (Figure 5.1).

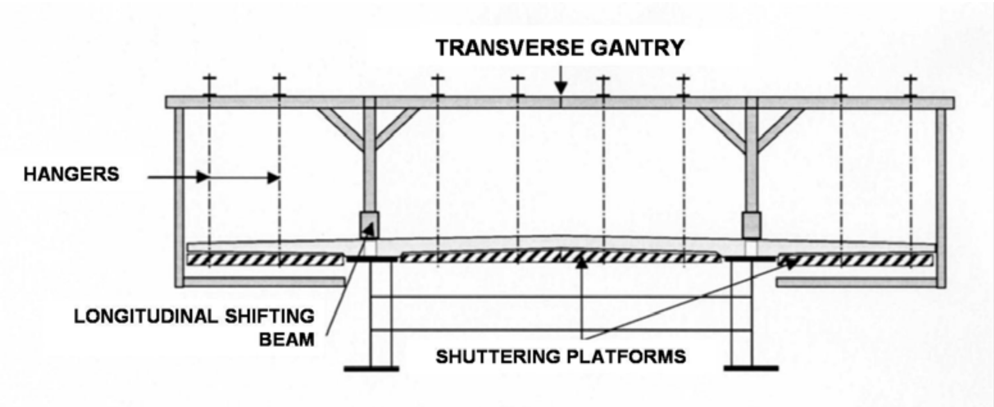


Figure 5.1. Full mobile formwork

Conversely, this central section does not need to be formed using mobile formwork, when it is formed by a pre-slab or the top flange plate of a box girder, for example. In this case, a partial mobile formwork traveller is implemented to form only the slab cantilever sections on its two lateral shuttering platforms (Figure 5.2).

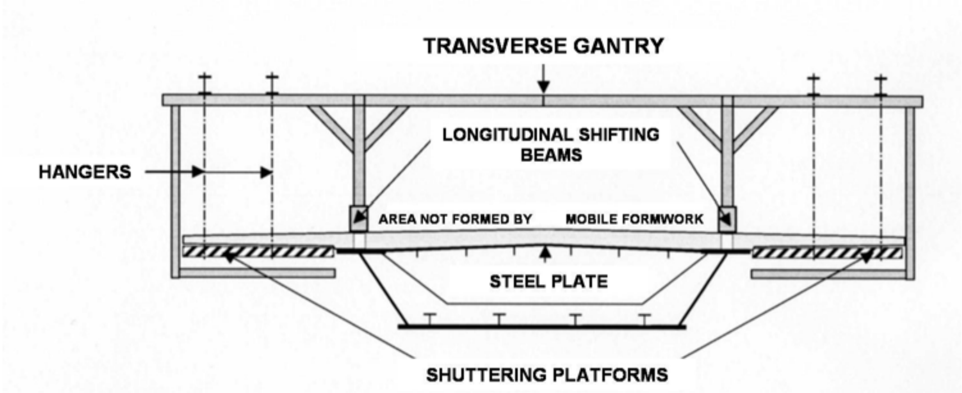


Figure 5.2. Partial mobile formwork

Mobile formwork for twin girder cross-beam bridges

In the case of a simple twin girder cross-beam bridge (constant depth deck without directly supporting cross-beams), the mobile formwork traveller usually comprises longitudinal shifting beams bearing on the girder top flanges, transverse gantries supported by the shifting beams and longitudinal stringers interconnecting the gantries. Hangers extending down from this framework support shuttering platforms between the slab soffit and the cross-beam undersides (Figures 5.1 and 5.3).

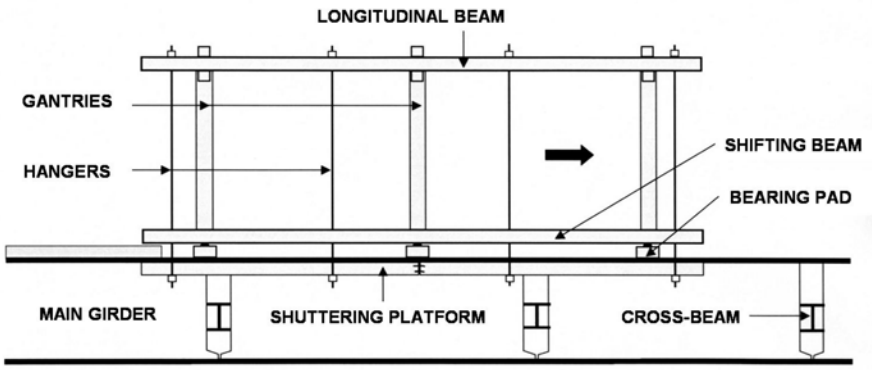


Figure 5.3. Longitudinal view of mobile formwork for a twin girder cross-beam deck (reinforcement for casting section under construction not shown)

Under static conditions, the longitudinal shifting beams supporting the entire formwork traveller bear on steel bearing pads, which cross the reinforcement of the slab casting section to be cast. The shuttering platforms are suspended from the transverse gantries by hangers, which penetrate right through the concrete.

Under dynamic conditions, the slab central section shuttering platform is released from the hangers and lowered onto the cross-beam top flanges, on which rollers or temporary stainless steel plates are positioned. The shuttering platform can then be easily moved by one concreting casting section length by a come-along or puller (Figure 5.4).

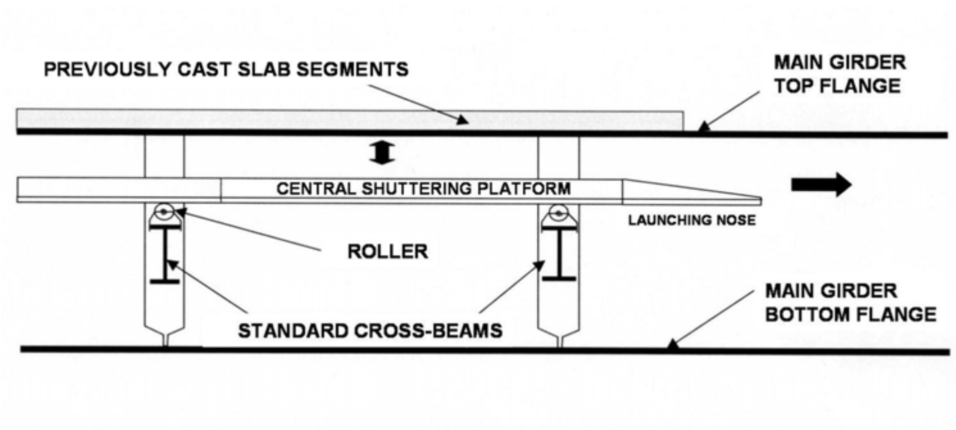


Figure 5.4. Translation of the central shuttering platform

Once this operation has been completed, the mobile formwork superstructure can be advanced along with the shuttering platforms for the cantilevers, which must be previously lowered onto lateral C-sections. This second operation is performed using come-alongs, the shifting beams rolling on the bearing pads.

Actions involving hanger release and shuttering platform lowering require human intervention and visual checking, so access to the underside of the structure is necessary. The areas beneath the cantilevers are thus accessed from static walkways fixed to the mobile formwork. Conversely, the area between the main girders is accessed from a mobile walkway totally independent from the mobile formwork (Figures 5.1 to 5.6 do not show this walkway for clarity).

The table below gives slab casting section dimensions and full mobile formwork weights for a few recent girder cross-beam composite bridges.

Name	Width	Slab casting section length	Formwork traveller weight
Aubenas bridge	10.70 m	10.60 m	45 t
Six Mariannes viaduct	11.00 m	9.75 m	35.5 t
Alse viaduct at Foix	11.25 m	12 m	52 t
Lapalisse viaduct	11.30 m	varying 10.20 to 10.96 m	~ 25 t
Alagnonnette viaduct	11.32 m	9.765 m	20 t
Cambrai viaduct	12.40 m	12.50 m	36 t
Cher viaduct	14.80 m	11 m	43 t
Loire bridge	varying 11 to 12.60 m	10 m	22 t
Rieucros viaduct	varying 12.7 to 13.80 m	varying 11.44 to 12.47 m	36 t
Triel bridge	varying 12 to 13.50 m	varying 11.40 to 13.50 m	48 t

Table 5.1: Slab casting section dimensions and mobile formwork weights

At preliminary design stage, the weight of full mobile formwork can be taken as equal to the area of the sections to be cast multiplied by a value between 0.2 and 0.4 t/m^2.

Difficulties specific to some twin girder cross-beam bridges

When the deck is of variable depth, the cross-beams are often located lower and lower as they approach the piers.

This situation does not usually cause any major problems, especially when moving the central shuttering platform, which takes on a very slight gradient even when placed, without particular precautions, on two successive cross-beams.

However, there are situations in which the gradient taken on by central shuttering platform is too large to ensure safe movement. In this case, raised chairs must be introduced between the top of the standard cross-beams near the pier and the rollers to compensate for the difference in height and allow the central shuttering platform to remain fairly horizontal (Figure 5.5).

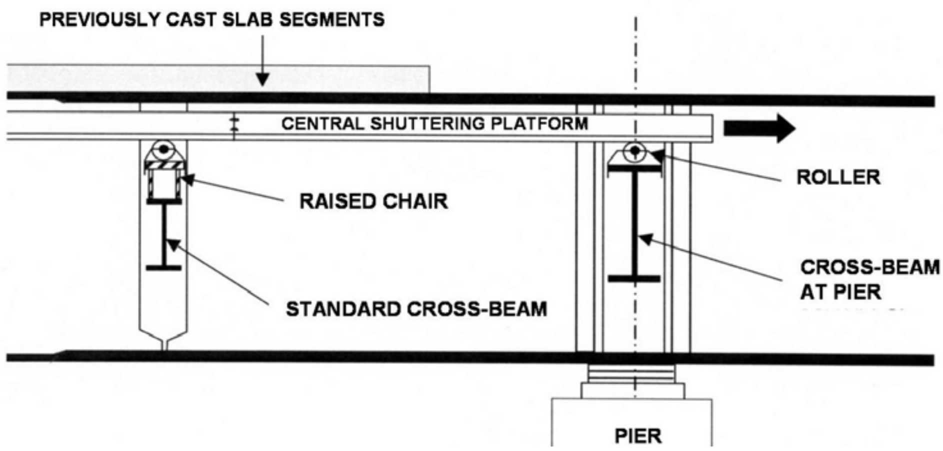


Figure 5.5. Raised chair for crossing pier cross-beams

The central shuttering platform can no longer be moved as explained above, when the deck features directly supporting cross-beams at the piers (an increasingly ununusal configuration). It must then pass beneath the directly supporting cross-beam either by lowering it to the ground or by placing it on a traveller, which rolls along the main girder bottom flanges. In both cases, this operation requires it to be dismantled into sections short enough to pass between the cross-beams and the pier directly supporting cross-beam (Figure 5.6).

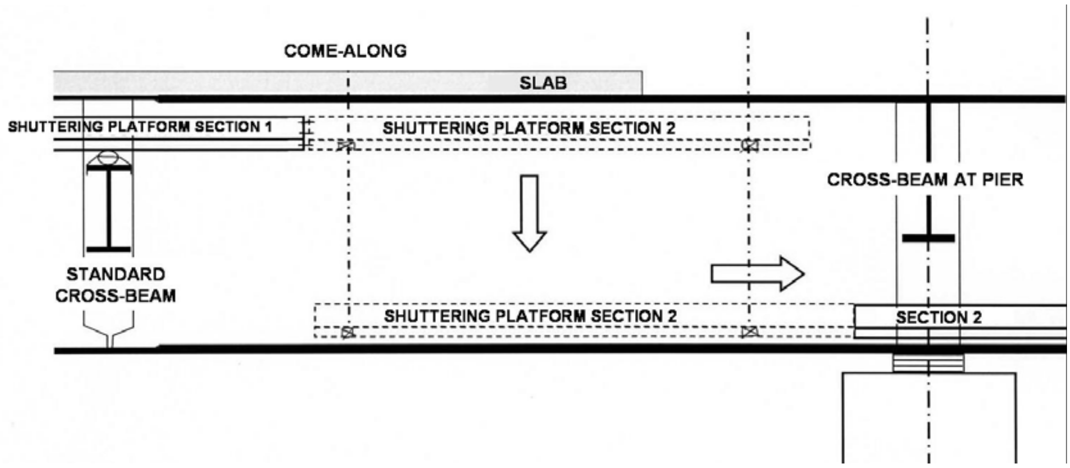


Figure 5.6. Moving/Dismantling central shuttering platform at pier directly supporting cross-beam

Mobile formwork for twin girder bridges incorporating directly supporting cross-beams with cantilevers

Design of mobile formwork for decks incorporating directly supporting cross-beams with cantilevers is much more complex than that of equivalent travellers for decks with cross-beams.

The directly supporting cross-beams are in contact with the slab, thus to allow shuttering platform movement, the platforms must be lowered to a level well below that of the bottom flanges of the directly supporting cross-beams, onto a temporary structure that rolls beneath the directly supporting cross-beams.

This leads to dividing the shuttering platforms into elements, whose length is equal to the distance between the directly supporting cross-beams, thereby increasing their number. For example, a mobile formwork traveller used to cast 12 m long slab casting sections for a bridge with directly supporting cross-beams and cantilevers spaced at 4 m will incorporate three shuttering platforms, each 4 m long.

Figure 5.7 illustrates the principle of a mobile formwork traveller incorporating central shuttering platforms, which can be moved using a shuttering support platform rolling on the main girder bottom flanges and a scissors chair. This traveller also incorporates lateral shuttering platforms, which can be moved using two shuttering support platforms rolling on both the main girder bottom flanges and temporary rails fixed at the end of the directly supporting cross-beam cantilevers.

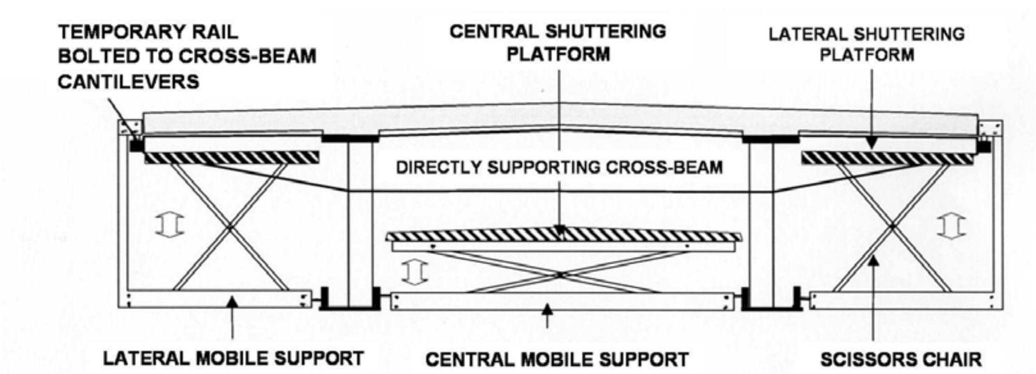


Figure 5.7. Mobile formwork for a deck incorporating directly supporting cross-beams with cantilevers / Example 1

Figure 5.8 illustrates another principle of a mobile formwork traveller based on a frame, which rolls by means of two large U-sections along the main girder bottom flanges and supports all three sets of shuttering platforms. Unlike the previous example, this mobile formwork must be removed to cross the piers.

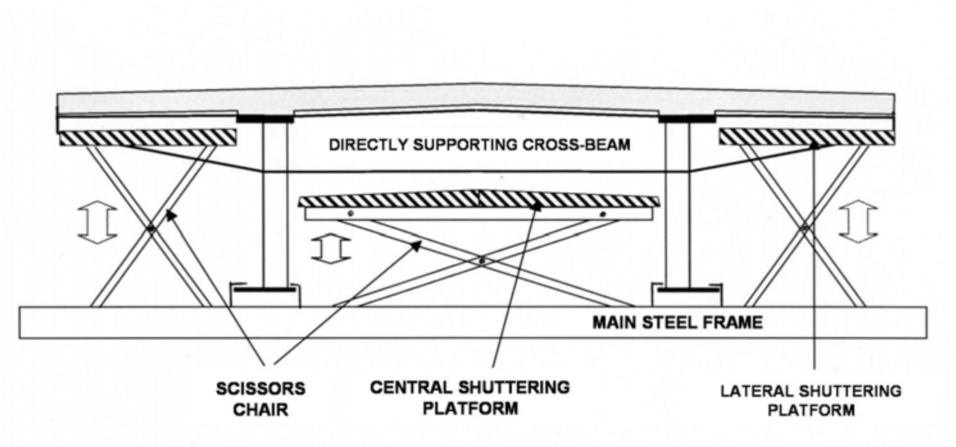


Figure 5.8. Mobile formwork for a deck incorporating directly supporting cross-beams with cantilevers / Example 2

Whatever the mobile formwork principle, once the shuttering platforms have been positioned for steelfixing and concreting the slab, they bear on the bottom flanges of the directly supporting cross-beams, ensuring freedom of the shuttering support platforms and scissors chairs and avoiding the need to design the latter “transfer” equipment for the fresh concrete load (Figure 5.9).

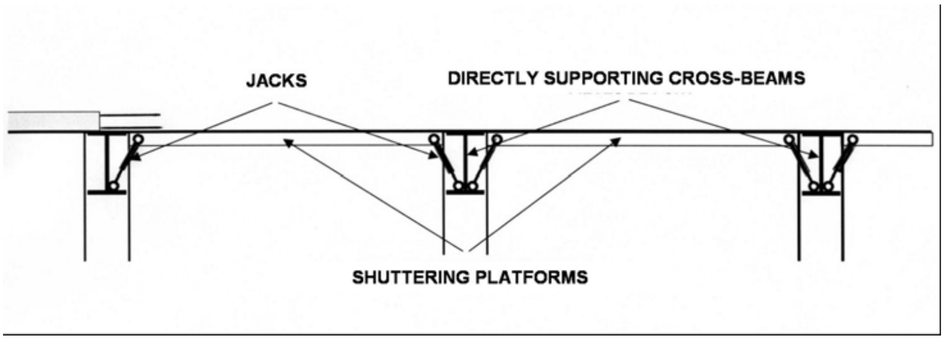


Figure 5.9. Shuttering platform bearing conditions in steelfixing/concreting position (reinforcement of casting section under construction not shown)

The table below gives dimensions of slab casting sections and weights of typical mobile formwork travellers used on a number of recent directly supporting cross-beam and cantilever bridges.

Name	Width	Slab casting section length	Mobile formwork weight
Downstream bridge Durance river at Avignon	21 m	12 m	42 t
Centron downstream viaduct	13.50 m	8.25 m	50 t
Maine viaduct	20.90 m	11.80 m	

Table 5.2. Slab casting section dimensions and mobile formwork weights

It should be noted that the directly supporting cross-beams and, in particular, their cantilevers can be designed based on slab concreting conditions. At this concreting stage, the directly supporting cross-beams, in fact, do not yet benefit from the structural contribution of the slab, so this represents a particularly unfavorable case.

Mobile formwork for twin girder bridges incorporating directly supporting cross-beams without cantilevers

To form the slab of a twin girder deck incorporating directly supporting cross-beams without cantilevers, we can implement combined mobile formwork travellers. These effectively combine the superstructure and lateral shuttering platforms of a mobile formwork traveller for a twin girder cross-beam bridge and the central section shuttering platform, scissors chair and mobile support platform of a mobile formwork traveller for a twin girder bridge incorporating directly supporting cross-beams with cantilevers (Figure 5.10).

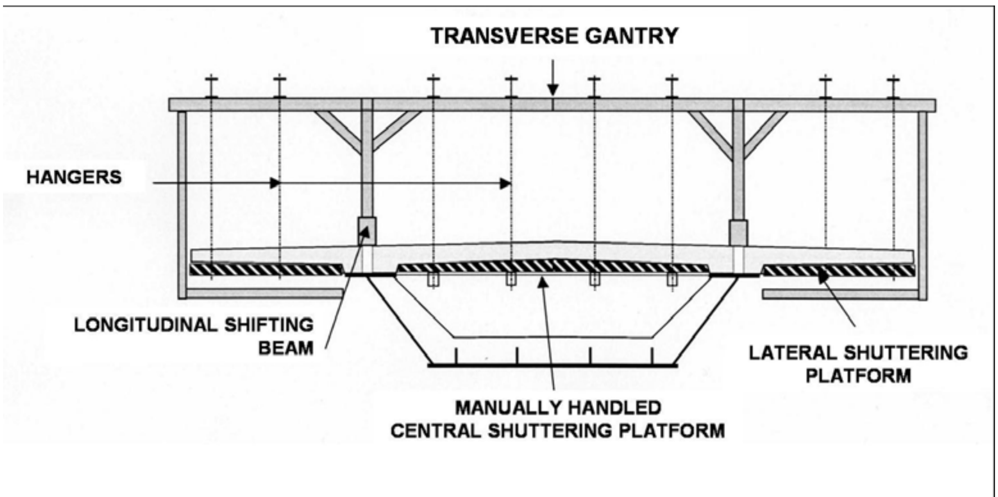


Figure 5.10. Mobile formwork for a twin girder bridge incorporating directly supporting cross-beams without cantilevers

Mobile formwork for box girders without directly supporting cross-beams

To form the slab of a box girder deck without directly supporting cross-beams, we can use another "hybrid" mobile formwork traveller. This effectively integrates the superstructure and lateral shuttering platforms of a mobile formwork traveller for a twin girder directly supporting cross-beam deck. However, in the central section between the box girder top flanges, the bulkheads greatly obstruct the lowering and subsequent longitudinal rolling of the central shuttering platform. Thus, a suspended shuttering platform composed of manually handled beams can be installed, dismantled and moved at each stage of the slab construction cycle (Figure 5.11).

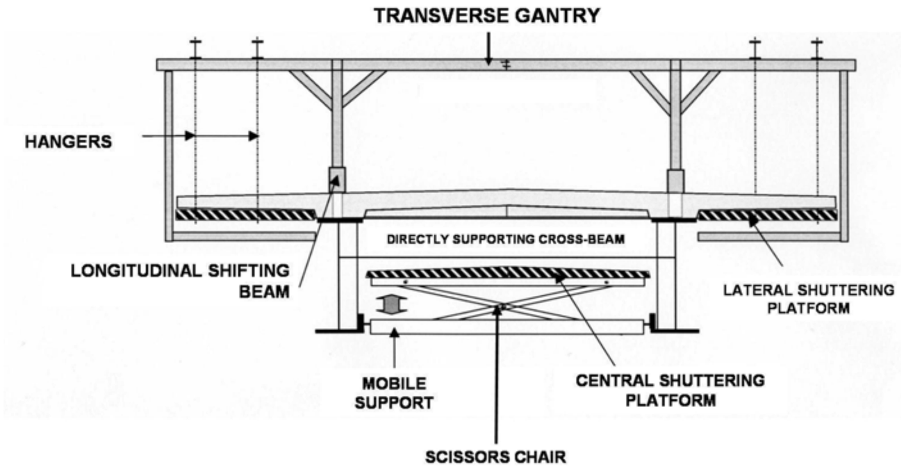


Figure 5.11. Mobile formwork for box girder without directly supporting cross-beams

5.2.4 - Practical details of slab steelfixing

Implementation in mobile formwork

Twenty or so years ago, concrete reinforcement for composite bridge slabs was most often fixed right in the congested space amongst the mobile formwork travellers.

Nowadays, this steelfixing operation – usually performed independently and in advance of the casting sequence – often involves positioning and fixing reinforcement cages prefabricated outside the mobile formwork. This system enables this operation to be removed from the construction critical path and facilitates implementation.

This means that the slab reinforcement is now only fixed inside the mobile formwork in special cases such as a complex geometry steel frame, no space near the bridge for assembling the reinforcement cages, very limited lifting equipment, etc.

Installing prefabricated cages in final positions using a crane

When the deck is less than 15 or so meters above the natural ground, the reinforcement cages can be prefabricated on the ground and installed directly in their final positions on the steel frame in place using a mobile or tower crane.

Cage prefabrication followed by translation

When the prefabricated reinforcement cages cannot be placed using a crane, they can be installed from one of the abutments using trolleys or buggies rolling on the steel frame top flanges between the two rows of connectors (Figure 5.12). These devices comprise steel sections mounted on wheels or rollers. There are usually 8 or 10 of them interconnected by a longitudinal section, depending on the cage length. They are often fitted with guide skids at the front and rear because the risk of jamming is very high.

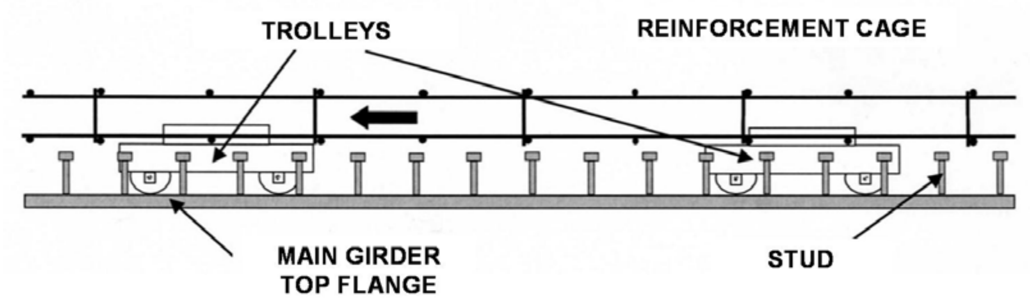


Figure 5.12. Reinforcement cages moved on trolleys or buggies

Once the cage has been transported to its final location, it is raised by screw jacks or lifting chairs (Figure 5.13) and the trolleys are removed one by one.

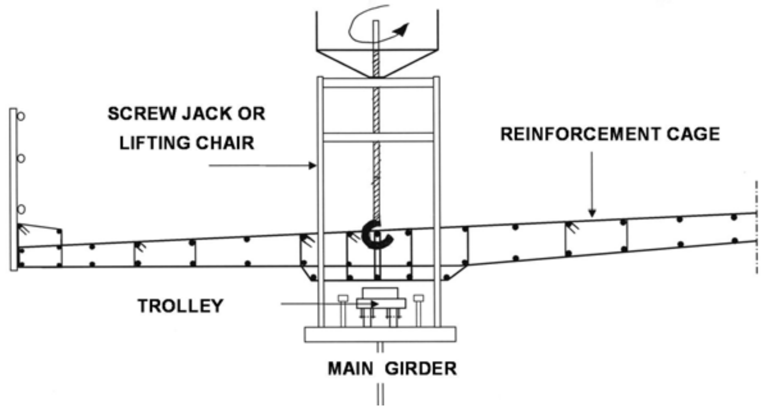


Figure 5.13. Raising of reinforcement cages by screw jacking and clearing of trolleys

More sophisticated means, such as motorised transfer carriages and even motorised cage installation gantries, are sometimes used on very large bridges. These offer a number of advantages including high speed, possibility of moving the cage horizontally before placing it, etc.

Fixing reinforcement on steel frame before launching

A steel frame is almost always launched, when the deck is more than 15 or so meters above natural ground. In this case, it may be advantageous to launch the steel frame with the slab reinforcement fixed to it. Reinforcement cages can be assembled directly on the steel frame from a working platform constructed at the steel frame assembly and launching area. They can also be prefabricated on the ground, then lifted by a crane onto the steel frame prior to launching.

The main advantages of this method are the quality ensured by prefabrication, the absence of cage movements and the reduction in operations to be performed once the steel frame is in place. Its principal drawbacks are the need to use suitably designed launching equipment, the inconvenience caused by the reinforcement during roller saddle removal operations and, in some cases, the need to strengthen the steel frame just for the launching stage.

Precautions associated with prefabricated cage usage

Use of prefabricated reinforcement cages reduces the operations to be performed within the mobile formwork traveller and hence the time required to complete a slab casting section. However, it does demand implementation of a number of precautions.

Firstly, the cages must be very carefully designed and built because geometrical clashes can occur between the passive reinforcing bars and the slab connectors, when the cages are set down on the steel frame top flanges. In relation to this point, use of templates integrating connector positions and odd and even cage masks by the steelfixing contractor is strongly recommended. The cages can only be lowered vertically onto the steel frame, so design of the junction between two adjacent cages also requires very careful thought (Section 3).

Furthermore, the cages are inherently flexible, so their rigidity must be increased by fitted them with stiffening bars (to be recovered or abandoned in the concrete) or by implementing systems to reduce their deflections during stages, at which they are not supported by the mobile formwork. If these measures are not adopted, the cages are subjected to very high deformations with deflections reaching 10 or so centimetres and these can be hazardous for personnel in particular (risks of falling, mobile formwork jamming when travelling, etc.).

Finally, for very long bridges, the time during which a reinforcement cage is in place on the steel frame prior to concreting can last several weeks. If this time exceeds 3 months, it may be necessary to takes measures to prevent pollution of the steel frame paint and abutment facings by rust running down from the cages, especially in a tropical atmosphere.

5.2.5 - Number and displacement sequence of mobile formwork travellers

Slab construction sequencing

Slab casting sections located at piers must be concreted last to reduce as much as possible the tensile stresses to which they are subjected. This is ensured by adopting a non-continuous sequencing technique, which involves either casting all the slab casting sections within the spans then all the casting sections at the piers (Case 1, Figure 5.14,) or casting all the casting sections at the supports immediately after all the casting sections in their adjacent spans have been cast (Case 2, Figure 5.14,). The second solution is often called “pas du pèlerin” [pilgrim’s step] in France. The mobile formwork displacement sequence can be deduced from the above sequencing.

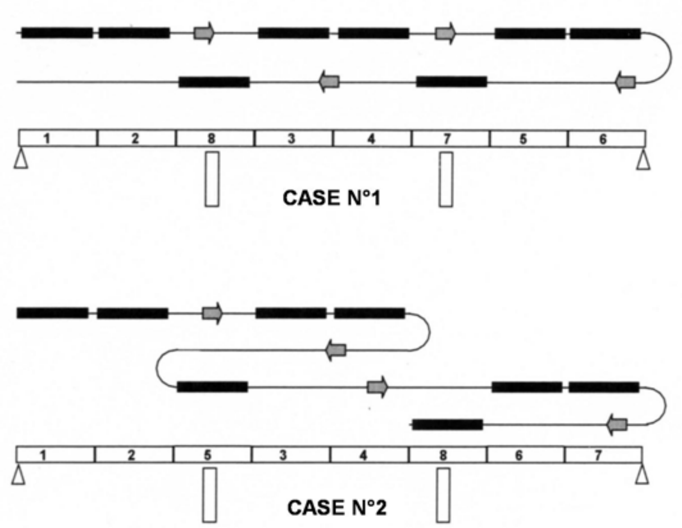


Figure 5.14. Principle of non-continuous sequencing

Number and displacement sequence of mobile formwork travellers

One or two mobile formwork travellers are used on a project. These are installed once the whole steel frame is in place and is supported on its temporary bearings.

When a single mobile formwork traveller is used, it most often moves from one abutment to the other, whilst adhering to the non-continuous sequencing conditions described above (Case 1, Figure 5.15).

When two mobile formwork travellers are used, these can start out from each abutment and meet at the centre of the deck (Case 2, Figure 5.15) or, conversely, be set up at the centre and move outwards towards each abutment. The two formwork travellers can also move in the same direction, the first used for casting the casting sections within the spans and the second for casting the pier casting sections (Case 3, Figure 5.15). In every case, the mobile formwork travellers adhere to the non-continuous sequencing detailed above.

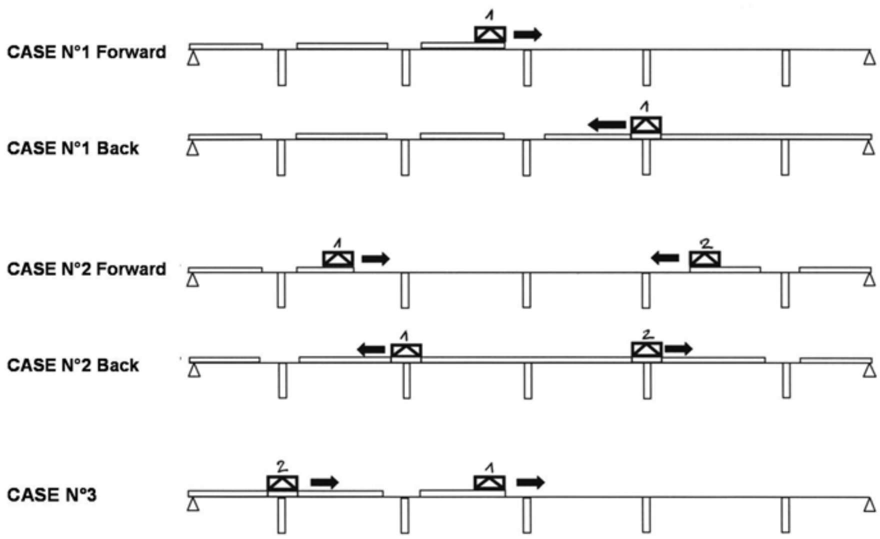


Figure 5.15. General mobile formwork advance directions for different sequencing

It should be noted that in many cases, the mobile formwork travellers have to pass over prefabricated reinforcement cages already in place, which means that temporary longitudinal U-sections must be installed on the cages.

Special cases

Deck casting section concreting that does not adhere to non-continuous sequencing may be accepted when the bridge to be built crosses very bust roads or railways and it is considered that the "to-and-fro" movements of the mobile formwork travellers may generate excessive constraints and risks for users of the road or railway beneath. However, exemption from non-continuous sequencing must only be granted if all other envisaged alternative solutions (launching with slab section on board, protection of users by decking or other means, etc.).

5.2.6 - Construction rate

The construction rate for composite bridge slab casting sections of course depends on financial criteria and concerns aimed at guaranteeing the required durability (minimum concrete strength at form stripping, concrete curing time, etc.). We recommend waiting a minimum of 24 hours before stripping the forms. Despite this necessary curing time, two casting sections, i.e. 16 to 25 m of slab, can be cast per week and per mobile formwork traveller.

5.2.7 - Adaptation to road geometry

Most road geometries are compatible with concreting using mobile formwork. For variable width bridges, one must nevertheless plan on adjustable platforms, which are expensive and their construction rate is much lower that the figure quoted above.

5.3 - Slab construction by precasting

5.3.1 - General Principles Of Method

Slab construction by precasting involves building - at the precast yard or in an area near the abutments – 2.5 to 4 m long, full depth slab sections and then installing these units on the steel frame prior to finally concreting the closing joints designed between the precast slab sections.

Whilst less widespread than construction using mobile formwork, precasting offers construction advantages, which make the method totally relevant to bridges:

- comprising a complex steel frame or geometry (variable width, skew, etc. twin girder or box girder composite bridges),
- for which the construction time is very short,
- for which site conditions are difficult (areas subject to severe frost, sites very far away from concrete batching plants, crossing of busy roads, railways, waterways, etc.).

From the structural standpoint, precasting also has the advantage of curtailing concrete shrinkage effects, which contribute significantly to slab cracking. Short-term shrinkage – mainly thermal and endogenous – in fact no longer occurs. Furthermore, when installing the slab sections, approximately 50% of the long-term shrinkage, due mainly to dessiccation, has already taken place.

On the other hand, slab precasting demands extreme care in terms of both design and construction.

5.3.2 - Full width precast units

For twin girder cross-beam decks or box girder decks without directly supporting cross-beams, the precast unit width is that of the slab (Figure 5.16), which raises the problem of their connection to the girders. To overcome this, the connectors are concentrated at slab areas called slab connection recesses, which are not concreted at the precast yard. These recesses are then filled in situ with low shrinkage closing concrete prior to cement grouting the space between the precast units and the top of the flanges. The volume of this space is in fact too small to be properly filled by the closing concrete.

These precast slab units are usually 2.50 m long, which corresponds to the maximum possible width for road transport. It also gives a unit weight of 25 to 30 tonnes, which is considered the maximum compatible with "reasonable" lifting equipment. Finally, the 2.50 m length corresponds to the usual spacing of the most common crash barrier posts for BN4 and BN4-16t barriers.

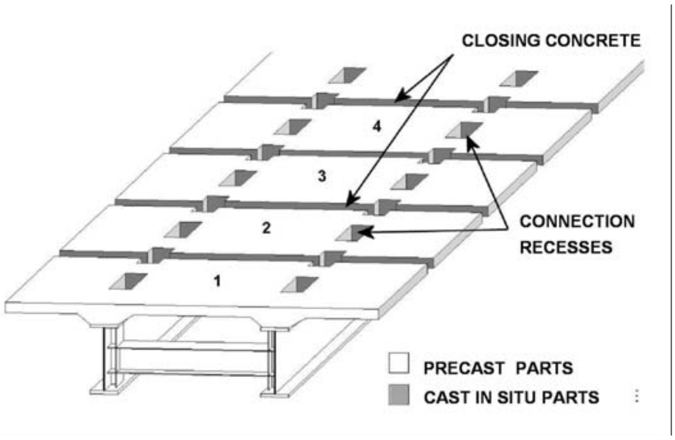


Figure 5.16. Precasting/In-situ casting distribution for a twin girder cross-beam deck

Slabs of many bridges have been built using this method, in particular the Monestier-de-Clermont viaduct on the A51 motorway, the Sauldre viaduct on the A85 motorway, the Dumbéa bridge in New Caledonia, etc.

5.3.3 - Partial precast units

Several situations may be encountered for bridges – twin girder or box girder – with directly supporting cross-beams.

If the part of the slab between the top flanges of the steel frame cannot be easily cast in situ, the slab must be broken down into two precast cantilever units and one (sometimes two for very wide decks) precast central unit (Figure 5.17).

Conversely, if the part of the slab between the top flanges of the steel frame can be easily cast in situ, which is effectively the case when a pre-slab, steel plate or even a trough section can be used as shuttering, then only the cantilevers need to be precast (Figure 5.18).

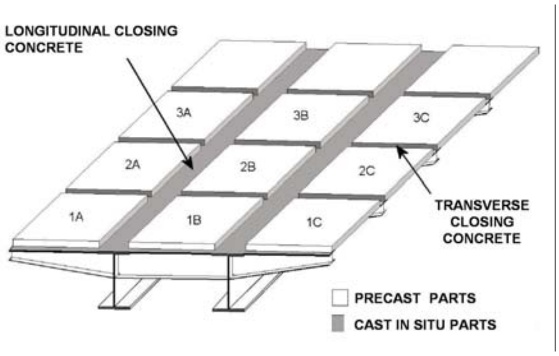


Figure 5.17. Precasting/In-situ casting distribution for a twin girder deck with directly supporting cross-beams and cantilevers

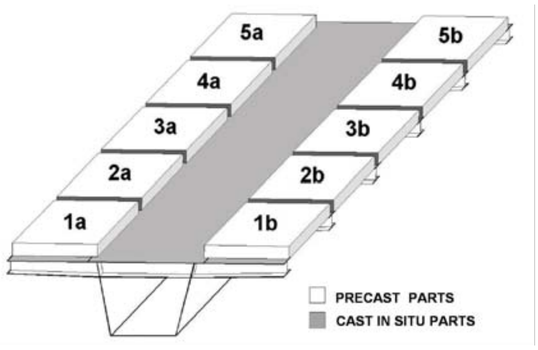


Figure 5.18. Precasting/In-situ casting distribution for a closed box girder deck with directly supporting cross-beams and cantilevers

In both cases, the slab areas above the top flanges do not require precasting, which eliminates the need for slab connection recesses and grouting of the space between the flanges and the precast parts.

With partial precasting, the slab unit length is directly deduced from the centre-to-centre distance between the directly supporting cross-beams. This centre-to-centre distance is close to 4 m and the precast unit length is approximately 3.60 m (excluding reinforcement starter bars). Moreover, the slab thickness is around 25 cm, so the weight of a precast unit is usually between 10 and 20 tonnes.

The slabs of the Frocourt bridge, the second River Rhône bridge at Valence and the Verrières viaduct were all built using these methods.

5.3.4 - Precasting and storage

Slab units can be cast at a permanent precasting facility or at an itinerant installation.

Slabs are almost always cast horizontally in steel forms integrating all provisions (steps, keys, etc.) required for creating high friction between the parts cast in situ and the precast parts.

Given the shortness of the closing concrete volumes and the presence of connectors, if required, their very dense reinforcement must be designed and fixed with the greatest possible care by means of templates and masks for even and odd units, controlling starter bar lengths, etc.

Great importance must also be given to slab unit storage conditions and precautions must be taken to ensure that passive starter bars are not damaged. Untidy stacks must be also avoided because these can lead to major slab unit deformations. Insufficiently compacted storage areas must also be avoided because these can sustain significant settlements.

As with all precast concrete elements, slab units must incorporate lifting devices (anchors, lifting rings) allowing them to be safety handled during storage and by the installation equipment.

5.3.5 - Precast slab unit installation

Precast slab units are generally installed on an advancing basis either to facilitate movement back and forth of the machine positioning the units on the steel frame or to optimise casting of closing concrete volumes.

The precast slab units can be installed by a crane positioned next to the bridge under construction, when the deck is near to the natural ground and a traffic route.

When this is not the case, the precast slab units can be installed by machines moving back and forth between the precast unit delivery area and the effective installation area. These machines must be capable of moving on both the precast units already installed and the steel frame top flanges. They can be mobile cranes or dedicated machines custom-built for the project.

Figure 5.19 illustrates the principles of a fairly simple machine for installing full width slabs without rotating.

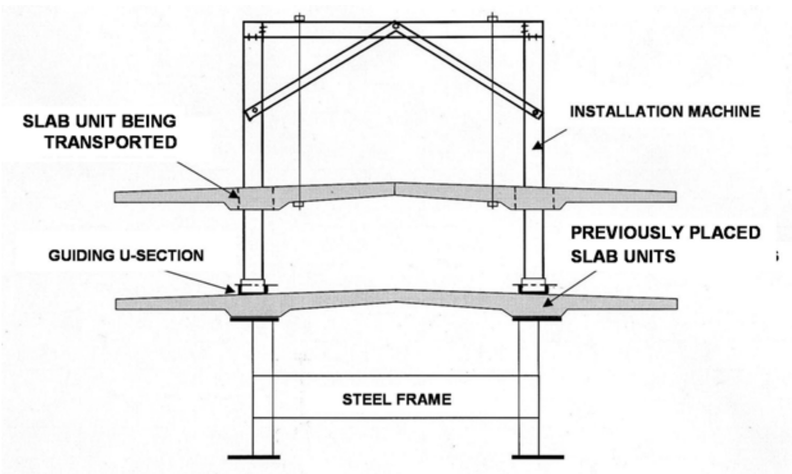


Figure 5.19. Principle of machine dedicated to installing full width precast units (here, rolling on previously placed units)

In common with placing prefabricated reinforcement cages by rolling them on trolleys, the precast slab unit installation operation requires that all steel parts (except for studs of course) protruding from the steel frame top flanges be eliminated, if they potentially obstruct movement of the installation machine.

To prevent any accidental movement of the precast slab unit installation machine, which could cause major damage, it is essential that it be guided by at least one U-section fixed to the precast units already in place, above one of the steel frame top flanges.

The installation machine is more complex for partial precast slab units because it must be equipped with a rotating crane to be able to install the cantilever slab units.

The table below gives the weights of slab unit installation machines used on several recent bridges with precast concrete slabs.

Name	Type of precasting	Maximum weight of units	Weight of machine
Monestier-de-Clermont viaduct	Full width	17,5 t	45 t
Boulogne-sur-Mer viaduct	Full width	12,5 t	11 t
VRL viaduct on Lille eastern ring road	Full width	70 t	
Second River Rhône bridge at Valence	Partial width	18 t	
River Loing bridge	Partial width	11 t	26 t
Access viaducts to 6th Rouen bridge	Partial width	18 t	22 t

Table 5.3. Weights of precast slab unit installation machines

In the case of full width precast slab units, the installation loads must be carefully investigated because, if the unit is subjected only to its dead weight, its resisting cross section is much less in the main girder flange axis because of the slab connection recesses.

5.3.6 - Details of precast slab unit closing

General

There are two types of closing concrete volume between the precast slab units.

The simplest case is that in which the two precast units to be closed are installed on the top flange of the steel frame or a directly supporting cross-beam because this flange can be used as a bottom shutter for the second stage concrete. Figure 5.20 illustrates the system implemented on the Verrières viaduct and this was re-implemented on the second River Rhône bridge at Valence.

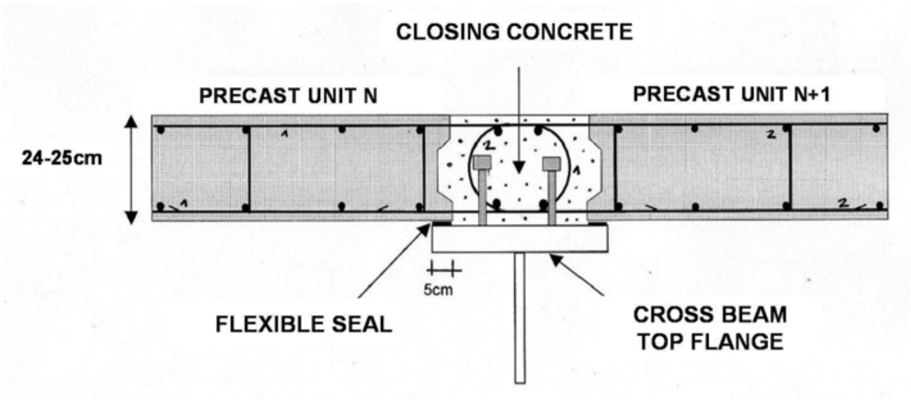


Figure 5.20. Principle of closing over directly supporting cross-beam flanges

The case, in which the closing volume is cast "in space" between flanges, is of course more complex. Theoretically, this type of operation can be performed in several ways, including:

- using conventional formwork,
- extending the precast units with steel consoles locally thickening the slab,
- extending the precast units with reinforced concrete corbels locally thickening the slab,
- extending the precast units with reinforced concrete corbels created within the slab thickness.

The conventional formwork solution ensures total continuity of the slab, i.e. continuity of all the longitudinal passive reinforcing bars, especially those in the bottom reinforcement layer. However, it requires formwork to be installed beneath the closing concrete volume, which can effectively be supported by fixings crossing the precast parts (Figure 5.21). This type of procedure, applied on the RN125 bypass bridge at Fos for example, can be used without reservations.

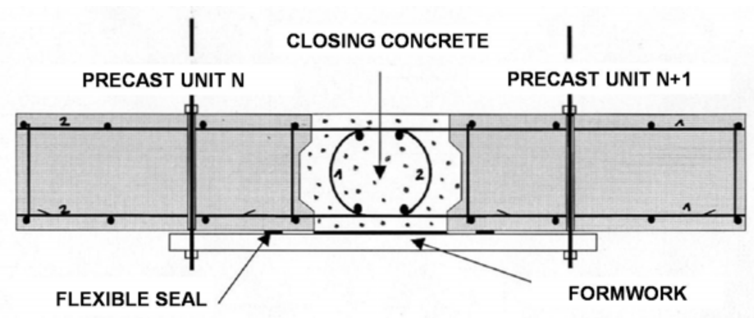


Figure 5.21. Conventionally formed closing concrete

The second solution, which is also highly favorable from the slab integrity standpoint, raises the problem of anti-corrosion protection of the steel consoles and, to our knowledge, has never been applied.

The solution involving low-level concrete corbels locally thickening the slab (Figure 5.22) has been used on the Sauldre viaduct and on the Dumbéa river bridge. It is less attractive than the first two solutions because it is rather difficult to form and steelfix and the cantilever bottom faces, with ribs approximately every 2.50 m, is aesthetically less satisfactory. On the other hand, it does ensure continuity of the top and bottom longitudinal passive reinforcement and does not reduce the effective slab thickness.

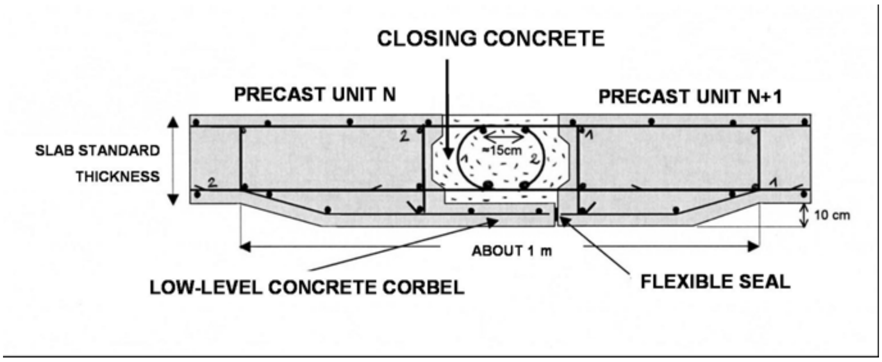


Figure 5.22. Closing volume cast over low-level concrete corbels

The solution based on reinforced concrete corbels integrated into the slab thickness (Figure 5.23) has effectively been applied on the Monestier-de-Clermont and Boulogne-sur-Mer viaducts. Its appearance is satisfactory and it is relatively easy to implement. Its main drawback resides in the sudden discontinuity of the slab effective thickness, which introduces at each joint and in the reduced lever arm of the longitudinal passive reinforcing bars within the closing volume. Furthermore, this solution causes discontinuity of the bottom longitudinal passive reinforcing bars and places them very near to a construction joint parallel to them. It is therefore clearly less safe than the preceding solutions and should therefore not be retained.

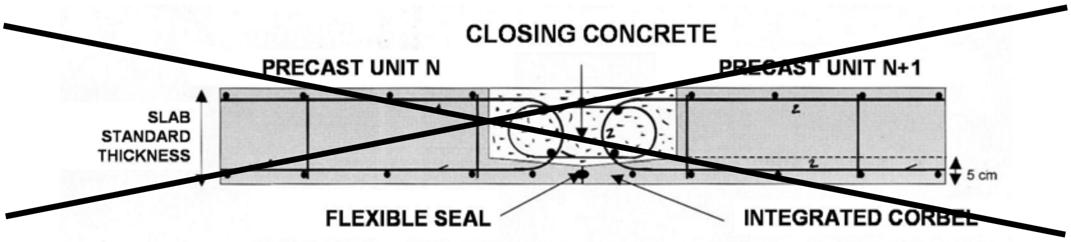


Figure 5.23. Closing volume cast over concrete corbels integrated into the slab thickness
(prohibited solution)

On the same bridge, it may prove necessary to cast closing volumes both over the girders or cross-beams and between them (in space). This is especially the case for twin girder bridges incorporating directly supporting cross-beams without cantilevers and for box girders with bulkheads.

Closing concrete dimensions

Whatever the planned method, utmost care should be given to the design and construction of the closing concrete volumes.

Deck closing must enable the reinforcement of adjacent precast slab units to be structurally interlinked by hoops; these reinforcing bars being effectively linked by four transverse bars. Minimum closing concrete thickness is therefore determined by the minimum diameter of the passive reinforcing bar bending mandrel defined by Eurocode 2. Moreover, the closing volume length must allow correct lapping of the slab reinforcing bars.

In the case of a closing volume similar to that shown in Figure 5.21, its minimum dimensions can be determined in the following way:

- minimum closing concrete thickness: diameter of bending mandrel + 2 longitudinal bar diameters + concrete covers,
- minimum closing concrete length: slab thickness plus approximately 15 cm (*).

These dimensions are given on the assumption that the interlinked passive reinforcing bars are located outside the secondary reinforcement. Their diameter must be strictly less than 20 mm in compliance with the construction conditions laid down in Eurocode 2 (cf. Section 3).

When two prefabricated slab units are installed on the top flange of the steel frame or directly supporting cross-beam (case shown in Figure 5.20), the minimum width of this flange is equal to the closing concrete length determined as indicated above plus the 2 x 5 cm bearing length. It should be noted that, for heavily reinforced slabs, the closing conditions may require a significantly wider directly supporting cross-beam upper flange than that required for a solution based on casting the slab in situ.

(*) This value has been arbitrarily chosen to prevent crushing of the concrete inside the interlinking hoops.

Construction joint surfaces

Special attention must be given to the quality and structural roughness of construction joint surfaces.

For surfaces parallel to the bridge axis, a good solution is to incorporate vertical keyways, which contribute to resisting slip between the precast parts and those cast in situ.

For surface perpendicular to the bridge axis, incorporation of keys resisting all vertical movement is recommended.

Precast slab unit bearing conditions

Conditions governing bearing of the precast slab units on the steel frame must be considered as early as possible.

Flexible seals must always be incorporated between the precast units and the top flange surfaces to prevent leakage of laitance during placement of closing concrete (Figure 5.24).

In addition, for full width precast slab units, provision of a centring device, which effectively positions the unit in line with the steel frame webs and, in some cases, allows unit level adjustment is recommended (Figure 5.24). This device can take several forms: a slab longitudinal rib, a steel square section tacked to the top flanges, level adjustment bolts, etc.

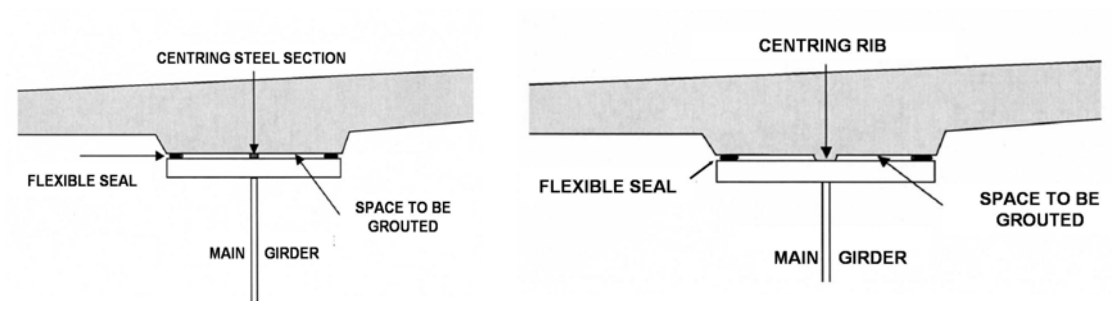


Figure 5.24. Full width precast slab unit bearing conditions

The flexible seals and the centring device height must be selected such that, once the slab unit has been installed, the space to be grouted between the top flanges and the precast units is 10 to 15 mm deep.

Whatever the type of slab, the flexible seals can never resist horizontal loads, especially before compression by the weight of concrete, so a precast unit must never be moved horizontally to correct its transverse position without initial lifting.

Closing concrete reinforcement

Closing concrete volumes are very congested parts of the slab, so the reinforcement details for the closing concrete itself must be properly designed (transverse bars, links, bars anchoring the restraint systems). Certain special requirements may indeed be necessary.

Closing concrete construction sequencing

Closing concrete volumes can be steelfixed and cast:

- either after installing all the slab precast units (case for short bridges),
- or by sets as the spans are completed (case for long bridges).

In every case, the order of steelfixing and casting the closing concrete volumes must be confirmed by the design office and Engineer because it affects the loads in both the steel frame and the slab.

Closing concrete mix

The closing concrete mix must be designed to limit its shrinkage to approximately 1×10^{-4} at an ambient relative humidity of 70 to 80%. Moreover, its characteristic strength must be higher because this concrete resists high concentrated loads at the connectors.

Grouting

In the case of full width precast slab units, the remaining gap between the girder flange top surfaces and the underside of the precast slab units must be grouted with extreme care because it effectively constitutes the anti-corrosion protection for the flange top surfaces. In particular, vents must be incorporated to allow the air to escape from the volume to be grouted and to monitor the grouting operation. These vents are usually located above the girder flanges prior to installing the slab units and their outlets are positioned inside the slab connection recesses.

In the case of partial width precast slab units, no grouting operation is required because the parts of the flanges beneath the precast units are very small and most of their area is protected by the 50 mm anti-corrosion protection return (cf. Section 6).

Transverse prestress

We recommend applying transverse prestress to the slab, when it has been constructed using partial precast units.

5.3.7 - Construction rate

When the slab comprises full width precast units, approximately 10 of the 2.50 m long units (including closing concrete volumes) can be installed and concreted per day and per installation machine, i.e. approximately 125 m of slab per 5-day week.

When the slab is made up of partial width precast units, its construction rate is between 135 m and 180 m per 5-day week.

5.3.8 - Other points

Precast slab units can be any shape, so this method is compatible with most road or railway geometries.

Moreover, it is theoretically possible to integrate restraint system anchoring stringers into the precast slab units. However, we do not recommend this measure in practice because it creates more joints in these stringers and does not provide them with a regular longitudinal profile, which can correct slab casting and installation tolerances.

5.4 - Other slab construction methods

5.4.1 - Casting in situ with permanent formwork

General

All or part of a composite bridge slab soffit can be formed using permanent or sacrificial shutters. This method is particularly attractive, when handling and moving mobile formwork travellers is difficult.

When the underside of the permanent formwork is in an open volume, which is the case for both the cantilever and central parts of twin girder and multi-girder decks and the cantilever parts of box girder decks, only concrete pre-slab type formwork is used because other methods present a corrosion risk and do not offer a uniform appearance of the slab soffit. These pre-slabs can be structurally independent, when the spans are small (Figure 5.25). On the other hand, it is preferable to incorporate structurally contributing or co-operating pre-slabs, which do not increase the weight of the slab, when the spans are large (cf. Sub-section 5.4.2).

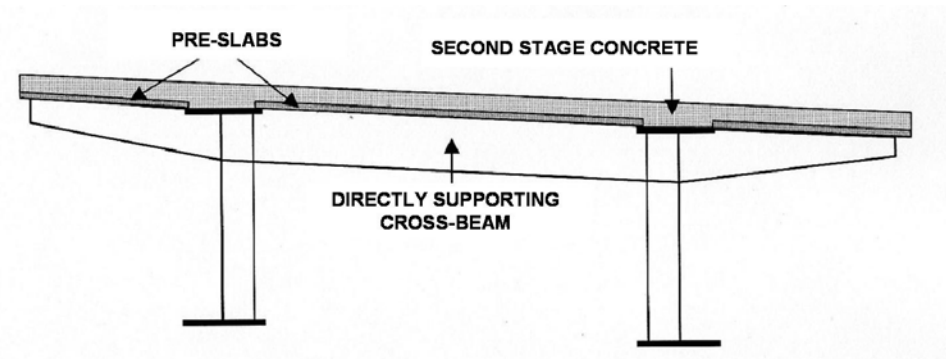


Figure 5.25. Twin girder directly supporting cross-beam deck formed by pre-slabs

When the underside of the permanent formwork is in a closed volume, which is the case for the central part of box girder decks, structurally independent flat plates, which are in fact very light, can be used as bottom shutters. Steel ribbed trough sections are sometimes used, but this method raises a number of problems (difficulty of steelfixing the soffit due to the trough profile, sealing around the flanges, fixings, large unpropped deflections, etc.) and these lead us to advise against its application in civil engineering structures.

Whatever the technology retained, the permanent formwork deflections must be curtailed during the concreting at stages. CCTG Fascicule 65 allows a straightness tolerance (in elevation) of max [$0.05\sqrt{L}$, 1 cm], in which L is the distance in centimetres between the directly supporting cross-beams on a deck supported by these transverse members. This condition imposes strict requirements for the formwork elements and often restricts their usage to bridges with directly supporting cross-beams or small box girders.

Inspection of the underside of the slab is impossible when permanent formwork is used.

Structurally independent concrete pre-slabs

Sétra Information Memorandum No.14 of February 1991, specifically dealing with permanent formwork, lays down multiple conditions for designing, calculating and implementing concrete pre-slabs (Figure 5.26). It is no longer up to date since it was drafted in compliance with the so-called French BAEL91 regulations, but it may be transposed into the Eurocodes.

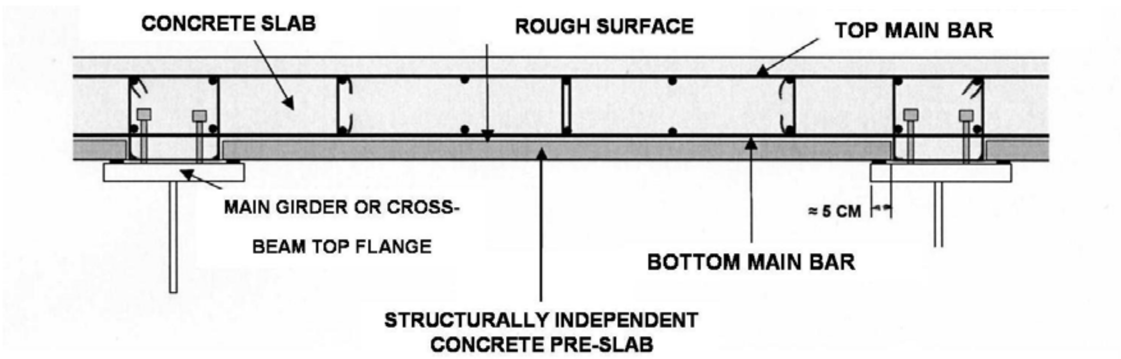


Figure 5.26. Slab formed by structurally independent pre-slabs (pre-slab reinforcement not shown)

Reinforcement of structurally independent concrete pre-slabs is only required to resist loads exerted during casting of the slab (pre-slab self-weight, slab reinforcement and fresh concrete self-weights, construction live loads) and is not continuous.

Watertightness of the top flange/pre-slab interface should be ensured as with the precast slab units. In this respect, concrete trial mixes or suitability samples are almost always necessary to confirm the measures to be implemented. Moreover, roughness of the pre-slab surfaces must be sought. It should also be recalled that pre-slab must overlap the flange by approximately 5 cm.

In keeping with the methods described at the start of this section, the slab sections must be installed in compliance with the principle of non-continuous sequencing defined in the sub-section dealing specifically with installation using mobile formwork.

The pre-slabs can be any shape, so this method is compatible with most road or railway geometries.

5.4.2 - Casting in situ with composite acting pre-slabs

In the case of a bridge with large spans, for which permanent formwork must be used, we need to resort to composite acting pre-slabs.

Compared with structurally independent pre-slabs, composite acting pre-slabs are often thicker – approximately 50% of the total slab thickness – and therefore heavier. Installation is therefore more difficult, frequently requiring design and construction of a true installation gantry.

Another major difference between a structurally independent pre-slab and a composite acting pre-slab is the reinforcement: for the former, this only has to take up loads exerted during concreting of the slab and is non-continuous and, for the latter, this is much more complex because, once the second stage concrete has been placed, it must take up loads from deck superstructures as well as road imposed loads. In this case, the reinforcement must be continuous both longitudinally and transversely and must comply with the requirements of Eurocode 2 in relation to shear at construction joint surfaces between the pre-slab and the second stage concrete (Figure 5.27).

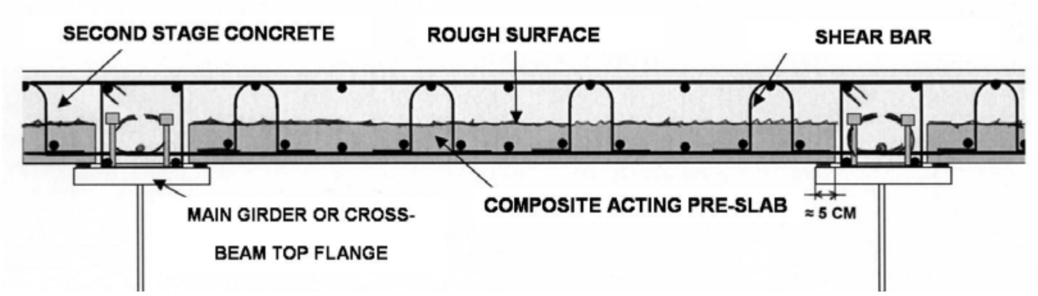


Figure 5.27. Slab formed by composite acting pre-slabs

Presence of slab connectors and reinforcement density mean that composite acting pre-slabs need to be cast and stored subject to the same precautions as those described for precast slab units: use of templates, control of starter bar lengths, upkeep of passive reinforcing bars, quality storage areas, etc.

As with structurally independent pre-slabs, due attention must be given to the watertightness of the top flange/pre-slab interface and the roughness of the pre-slab surfaces. We also recall that the pre-slabs must overlap the flanges by approximately 5 cm.

In recent years, slabs for many composite bridges incorporating directly supporting cross-beams with cantilevers have been built entirely with composite acting pre-slabs, in particular the Laize viaduct in Calvados and the Elle and Ribeyrol viaducts on the A89 motorway. Use of composite acting pre-slabs for these structures avoided the need for the relatively complex mobile formwork described in Sub-section 5.2.3 of this guide. Compared with partial width precast slab units, composite acting pre-slabs allow much less powerful lifting equipment to be used and significantly curtail the number of construction joints, although they do imply a slower construction rate (installation can reach a rate of 4 sets of 3 pre-slabs (1 central and 2 cantilever) per day, but slab steelfixing must then be completed in situ and its top section must then be cast based on non-continuous sequencing.

Composite acting pre-slabs are also used in the centre of twin girder composite decks incorporating directly supporting cross-beams without cantilevers or to facilitate formwork for variable width areas.

5.4.3 - In-situ Casting on conventional formwork

The slab is sometimes conventionally formed for bridges that are short or located very far away from the contractors' equipment yards.

For directly supporting cross-beam bridges, full width slab formwork can be supported by small beams bearing on the secondary steelwork.

For simple cross-beam bridges, central section formwork can be supported by props installed on decking, which bears on the main girder bottom flanges. Cantilever formwork can be supported by brackets fixed to the main girder top and bottom flanges (Figure 5.28).

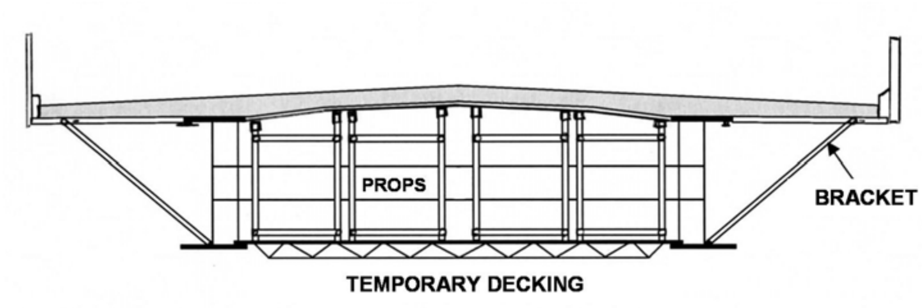


Figure 5.28. Conventional slab formwork for a twin girder cross-beam composite deck

5.4.4 - Installation by pushing slab cast sections from behind an abutment

Early in the 1980s, a French civil engineering contractor developed and patented a slab construction method for composite bridges based on the pushing method for prestressed concrete bridges. This method involves casting slab casting sections on a temporary steel frame welded or bolted to the permanent steel frame behind one abutment and then moving all the cast slab casting sections forward by horizontal jacking (Figure 5.29).

Between 1990 and 2005, this method was implemented at 10 or so bridges including the Varennes-lès-Mâcon viaduct and the Brioude viaduct on the River Allier. During this period, because of not only geometrical clashes between slab and connectors, but also tolerance problems, the parts vertically above the main girder top flanges were not constructed until completion of the pushing operation. This meant integrating heavy transverse sections into the slab under construction and transverse prestress into the deck of the bridge in service (Figure 5.30).

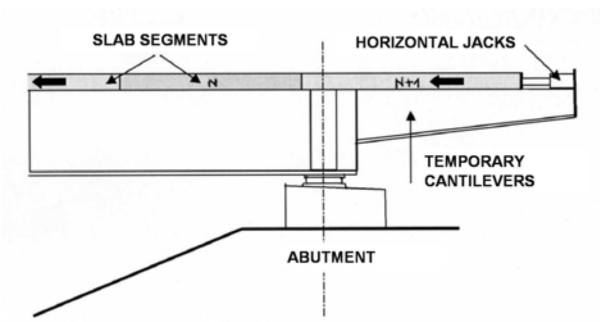


Figure 5.29. Principle of slab casting section pushing system

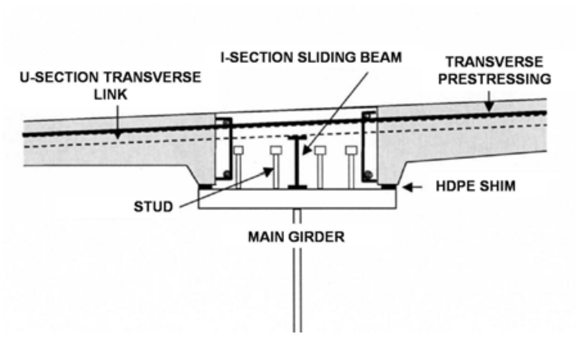


Figure 5.30. Connection conditions during construction for cantilevers and central section adopted up to 2005

Since the end of 2005, this contractor implements a new method based on the same main principles, but allowing elimination of the transverse prestress in service and connecting sections during construction. Applied in particular to the arch bridge on the River Tech at Vila and on the Route des Tamarins bridge crossing the Bras Mouton ravine on the island of La Réunion, this method involves casting the top half of the slab above the main girder flanges on the temporary steel frame and then casting the remaining enclosed volume at the end of construction. The latter volume, called the connection tunnel, is filled with self-compacting (therefore very fluid) concrete poured into open pockets in the slab extrados (Figure 5.31).

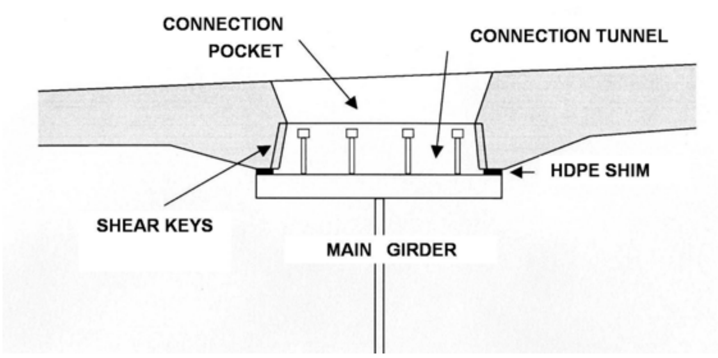


Figure 5.31. Central section-cantilever connection conditions during construction, adopted since 2005

In this new method, the steel frame is conventionally connected to the connection tunnel infill concrete by studs and the rest of the slab is connected to this infill concrete by both multiple shear keys, arranged continuously along the subvertical faces of the tunnel, and connection pockets acting as locking lugs.

Linking of the connection tunnel and surrounding slab concrete is complemented by reinforcement fixed around the connectors prior to pushing the slab and embedded in the self-compacting concrete at the end of construction.

During construction, the rear of the slab is guided by a system fixed to the temporary steel frame and its front is guided by small temporary steel sections. These sections are temporarily fixed to the slab on both sides of each top flange and effectively force the slab to follow the main girder horizontal alignment.

Both forms of this method, which cannot constitute a tender basic solution, offer the advantages of prestressed concrete deck pushing, including working locations with no risk of falling, joint reliability by passive bar lapping, high construction rate, etc.

Their main drawbacks are the difficulty of controlling the geometry, especially the longitudinal profile, the magnitude of the horizontal loads to be applied to the last slab casting sections (these internal loads have nevertheless no impact on the supports) and the cantilever/central section connection conditions, especially during construction. It should also be noted that the transverse reinforcement installed at the slab connectors does not strictly comply with Eurocode 4 requirements since no tranverse bar crosses the standard/self-compacting concrete interface. This imposes test-based confirmation that the connection between these two concrete volumes is operating properly.

Unlike the other methods explained in the section, pushing is only suitable for building decks, whose horizontal alignment and longitudinal profile are each only formed by a single element (straight or circular alignment).

Construction rates of approximately 10 m of slab per day can be achieved using the pushing method.

5.4.5 - Delayed connection and total precasting combined with prestressing

Later connection

At the end of the 1980s, another French contractor patented a so-called “delayed connection” principle. This involves:

- delivering the steel frame to site without its studs,
- building the slab, leaving separate 80 mm diameter cylindrical pockets for the future studs (Figure 5.32),
- welding the studs on the main girder top flanges using a gun with an extension,
- grouting the connector pockets and possible voids remaning between the top flanges and the slab.

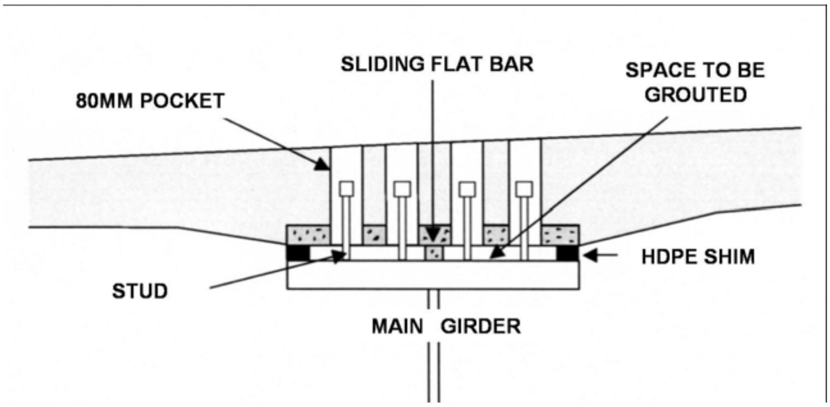


Figure 5.32. Slab design details at main girders for delayed connection

The advantages of this principle are the possibility of constructing a slab without strengtheners, connection recesses (only pockets) or stud concentrations. Its main drawbacks are high cost and impossibility of checking the welds at the base of the studs.

This principle was successfully implemented at overpass PS13 (2-span bridges) on the A85 motorway and at the River Yonne viaduct, where a 27 m long slab casting section was launched with the steel frame but unconnected to it.

Total precasting combined with prestressing

The slab on a few composite bridges (Manosque bridge on A51 motorway, PS13 overpass on A85 motorway) has been built based on the glued precast concrete segment method developed for precast prestressed concrete decks (Figure 5.33).

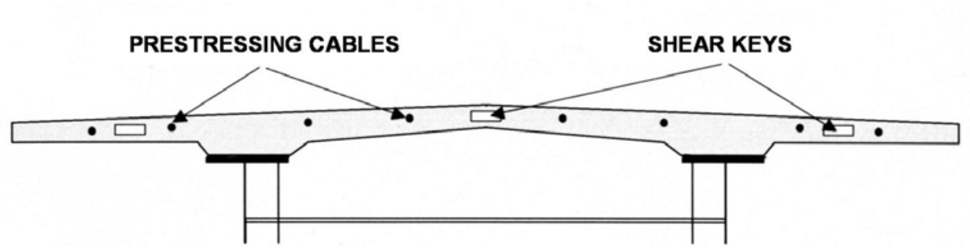


Figure 5.33. Slab comprising glued composite precast segments

Unlike the slab construction method described in Sub-section 5.3 above, this method involves precasting the whole deck slab without the need for cast-in-situ closing concrete. After installing the precast slab units on the steel frame, they are assembled by applying longitudinal prestress inside the slab concrete to compress the joints prior to finally connecting them to the top flanges of the steel frame.

As with all prestressed decks, the ends of the precast units have keys to ensure proper their vertical and transverse positioning, as well as shear load transmission. Epoxy glue is applied to them before final positioning.

This method is only attractive for 2- or 3-span bridges. Its main advantages are:

- slab durability because of its construction conditions and longitudinal prestressing,
- speed of slab unit installation, representing a major advantage for bridges to be built over roads or railways in operation.

The main drawbacks of this method are the costs of precasting and transporting the slab units, the cost of implementing the prestressing cables, which is all the higher since the cable unit power is low, and the need to perform a scientific creep calculation.

In the case of the PS13 overpass on the A85 motorway, the delayed connection principle described above was used because it was especially suitable.

Dans le cas du PS13 de l'autoroute A85, le principe de connexion différée présenté ci-avant a été utilisé car particulièrement bien adapté.

5.5 - Related bibliography

Construction using mobile formwork

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RT [GIL 01] [DUB 04]
OTUA [TAV 04]

Permanent formwork and composite acting pre-slabs

RT [MON 96] [JOL 08]
OTUA [MOS 09]

Pushing

BOA [PET 95]

Delayed connection

RT [ROU 98] [CHE 01]
BOA [BAR 00]
OTUA [CHE 01]

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501 to 503: General views of various mobile formwork travellers for twin girder cross-beam bridges. 504: Traveller movement jacks. 505: Traveller central shuttering platform for twin girder cross-beam ridges. 506 and 507: Travelling platforms for operating on slab soffit.

511: Traveller movement roller. 512: Mesh for construction joint between slab sections. 513 and 514: Casting a slab section. 515: Partial mobile formwork for single box girder. 516 and 517: Shuttering support platform with scissors chair for installing central section shutters on a twin girder directly supporting cross-beam deck.

521: Twin girder cross-beam bridge launched with slab reinforcement. 522: Cage installation by crane & lifting beam on a twin girder directly supporting cross-beam bridge. 523: Prefabricated reinforcing cage rolling frame and trolleys. 524: Rolling prefabricated cage over steel frame using frame. 525: Screw jacks or lifting chairs. 526: Excessive prefabricated cage deformation for a twin girder cross-beam bridge.

531: Slab unit precasting area. 532: Steelfixing with templates for precast slab units. 533 and 534: Full width precast slab units awaiting closing concrete and connection. 535: Full width precast slab unit storage. 536 and 537: Installation machines for full width precast slab units.

541: Closing concrete between full width precast slab units. 542: Centring square sections for full width units. 543: Conventional formwork for closing concrete between full width slab units. 544: Partial width slab units. 545: Details of partial width slab unit construction joint surfaces. 546 and 547: Partial width slab unit installation machines.

551: Closing concrete node at directly supporting cross-beam/girder junction. 552: Slab connection recess. 553. Steel trough formwork of box girder central section slab. 554: Trough fixing & waterproofing details at a girder top flange. 555: Box girder central section formwork using sacrificial plate. 556: Structurally independent pre-slabs. 557: Composite acting pre-slab. 558: Installation machine for composite acting pre-slabs.

561: Composite acting pre-slabs showing part of slab top reinforcement. 562: Jacks for pushing slab sections cast on site behind an abutment. 563: Slab being pushed, with connection tunnel. 564: Detail of pushed slab connection pocket showing connection tunnel shear keys.

6 - Measures contributing to durability and maintenance

▶▶ This section details the measures right from the design phase for a composite bridge to ensure its durability and easy maintenance.

6.1 - General principles contributing to maintenance

All measures required for examining and maintaining (without the need for implementing heavy equipment) all the bridge parts, including the interior of its hollow parts if necessary, should be foreseen during its design and construction phases.

This being ensured, the bridge must be equipped in such a way that management and monitoring operations may be carried out in compliance with labour laws and in particular French Law No. 93-14-18 of 31st. December 1993, which instigated the idea of compiling a DIUO (Dossier d'Intervention Ultérieur sur Ouvrage) or set of measures designed to facilitate subsequent work on the bridge.

Lastly, these access facilities must not subject the bridge to the risk of vandalism.

6.2 - Steelwork anti-corrosion protection

6.2.1 - General

Steel tends to oxidise naturally in the open air and when submerged or buried. So-called atmospheric corrosion (in open-air environments) is due to moisture on the surface of the steel and is accelerated in the presence of water and pollutants. Anti-corrosion protection treatment must be applied to the surface of metal parts to counter this phenomenon.

6.2.1.1 - Corrosiveness of the site

Not all sites have the same aggressiveness in terms of steel corrosion. Standard NF EN ISO 12944-2 defines several categories of corrosiveness that can be observed by monitoring standardised blank samples during the year and measuring any loss of mass. These categories are:

- C1 very weak,
- C2 weak,
- C3 average,
- C4 high,
- C5I very high (industry),
- C5M very high (marine),
- Im1 submerged in freshwater,

- Im2 submerged in seawater,
- Im3 buried in the ground.

Only six of these corrosiveness categories have been retained in CCTG Fascicule 56, which deals with anti-corrosion protection of metal structures. These are C2, C3, C4, C5M, Im1 and Im2, also called environment classes.

6.2.1.2 - Choice of anti-corrosion protection systems

There are three families of anti-corrosion protection systems:

- paint on stripped steel,
- hot galvanisation followed by painting,
- metal coating followed by painting.

Painting systems are by far the most frequently used for bridges. Galvanisation, or metal coating, followed by painting are anti-corrosion protection complexes that certainly offer a better durability but are relatively expensive and therefore reserved for very special cases (parts in very corrosive atmosphere, parts that are very difficult to repaint, etc.). Furthermore, galvanisation requires the parts to be fully immersed in a zinc bath, which limits the dimension of the parts to be treated to about 15 m long, 2 m wide and 2 m high. If this method is necessary, the steel must be suitable for galvanisation (the description of the steel must refer to one of three suitability classes defined in Standard NF A 35503 based on phosphorus and silicon contents).

In France, CCTG Fascicule 56 requires that the anti-corrosion protection systems used on bridges be certified by the ACQPA (Association pour la Certification et la Qualification en Peinture Anticorrosion) or association for the certification and qualification of anti-corrosion protection paint.

6.2.1.3 - Reminder of the description of anti-corrosion protection systems

Anti-corrosion protection systems are named by the ACQPA using five or six characters (C3ANV, C4GNV, Im2ZMV, etc.):

- The first two or three characters (C3, C4, Im2,…) denote the certification class, i.e. the category of maximum corrosiveness to which the system can be exposed while offering the durability guarantees defined by CCTG Fascicule 56 and CCTP (market performance specifications) such as appearance, blistering, etc.
- The following letter denotes the type of substrate, i.e., support: A for stripped steel, G for galvanised steel and Z for coated steel.
- The second last letter denotes the type of work: N for new work, M for maintenance work (not necessarily requiring the steel to be stripped).
- The last letter denotes the visibility of the surface to be painted: V for seen surface, I for unseen surface.

6.2.1.4 - Choice of certification class

The certification class must at least be equivalent to the corrosiveness category. In practice, the following is generally retained for a bridge deck:

- C5 in coastal areas located less than 5 km from the sea,
- C4 in urban areas, near polluting industries, located less than 20 km from the sea or in locations where repainting is difficult (e.g. above railway lines or main trunk roads),
- C3 in all other cases.

6.2.1.5 - Choice of a certified finishing colour

In metropolitan France, selection of the finishing coat from among the 23 colours certified by the ACQPA is recommended because these colours benefit from a 3-year colour stability guarantee.

In the majority of French overseas administrative Departments and Territories, no colour has this type of guarantee because the atmosphere there is tropical, but opting for one of the 23 colours on the ACQPA card is strongly recommended because they are more stable than other colours.

6.2.2 - Special cases for box girder's

External surfaces need to be distinguished from internal surfaces for box girders. Treating external surfaces of box girders falls under the general case described above and does not pose any particular problem. On the other hand, treating box girder internal surfaces is much more delicate. We have highlighted four typical cases.

Case 0: Box girder is watertight and cannot be accessed.

When the box girder is watertight, which pre-supposes all six sides are made of steel plates and that its interior cannot be accessed under any conditions (no trap door), no inner anti-corrosion protection is provided for. This is the case of very small closed box girders.

Case 1: Box girder is watertight but can be accessed.

When the box girder has six sides of welded steel but remains accessible in certain conditions (trap doors with seals, which can be opened or closed taking special precautions), in theory no treatment need be designed. In practice, the metal must be totally stripped and a light coloured primer applied to help detect any cracks. For bridges with small transverse dimensions, paint maintenance operations would indeed be very difficult.

Case 2: Box girder is not watertight but is subject to very regular inspection.

When the box girder is not watertight but is subject to very regular inspection, which is the case for a movable or toll bridge, a dryer could be used, but the plate internal surfaces must be totally stripped and a light coloured primer applied because, if the Client changes maintenance strategy, paint maintenance operations would be very difficult to carry out with the box girder in service.

Case 3: Box girder is not watertight and is not subject to very regular inspection.

If the box girder is not watertight and is not subject to very regular inspection, which is the most frequent scenario, conventional protection is implemented with a light colour designed for the relevant corrosiveness category (in general, a C3 system is designed but a C4 system may be required for a box girder near the sea). The system of course does not need to be UV resistant.

Remark

The reader's attention is drawn to the fact that, if pipes are fixed or are likely to be fixed inside the deck volume, internal anti-corrosion protection must be selected based on the fact that the steel box girder is not airtight.

6.2.3 - Implementation-related remarks

Treated surfaces

Anti-corrosion protection systems are applied to all steel surfaces except those in contact with the concrete slab. For the latter, a 50 mm return from the protection system is implemented to prevent oxidation of this critical area (triple air/concrete/steel interface).

Grinding sharp edges

It should be remembered that to obtain sufficient adherence of the system, sharp edges must be ground as per the conditions laid down in Standard NF EN ISO 12944-3.

Problems inherent to anti-corrosion protection of certain structures

Paint systems usually adopted on recent bridges, in particular systems certified by ACQPA, provide excellent anti-corrosion protection.

However, on certain recent bridges examined during preparation of this guide, separation of the finishing coat was noticed at certain locations on the underside of the bottom flange. These paint coat separation areas seem to come from either an excessive time lapse between application of the finishing coat and the previous coat or an excessive degree of humidity, when the finishing coat was applied. It is therefore important to remember the need to implement the protection system in total compliance with the recommendations provided in Section 3 of CCTG Fascicule 56, especially those concerning both the maximum time between two coats and the maximum degree of humidity.

Applying the finishing coat

For bridges crossing main trunk roads, whose steel frame is installed by a single launch, it may be worth applying the finishing coat at the launching area, well before deck construction is completed. This technique obviously imposes special precautions during launching then during slab construction, as well as probably touching up this final coat, but can mean it is applied under much better conditions.

6.3 - Deck

6.3.1 - Measures common to all structures

6.3.1.1 - Detailed design facilitating welding and anti-corrosion protection system

Stiffener centre-to-centre distance

In some very stiff areas, stiffener height and centre-to-centre distance may contribute to problems in implementing anti-corrosion protection.

Appendix C of Standard NF EN ISO 12944-3 imposes for stiffeners of a given height h,

- their minimum centre-to-centre distance a_{mini}, if they are quite far from a wall (see Case 1, Figure 6.1),
- their minimum distance to the wall a_{mini}, if they are near a plate or a wall likely to hinder the painting contractor (see Case 2, Figure 6.1).

Case 1. Standard stiffener *Case 2. Stiffener near plate perpendicular to stiffened plate*

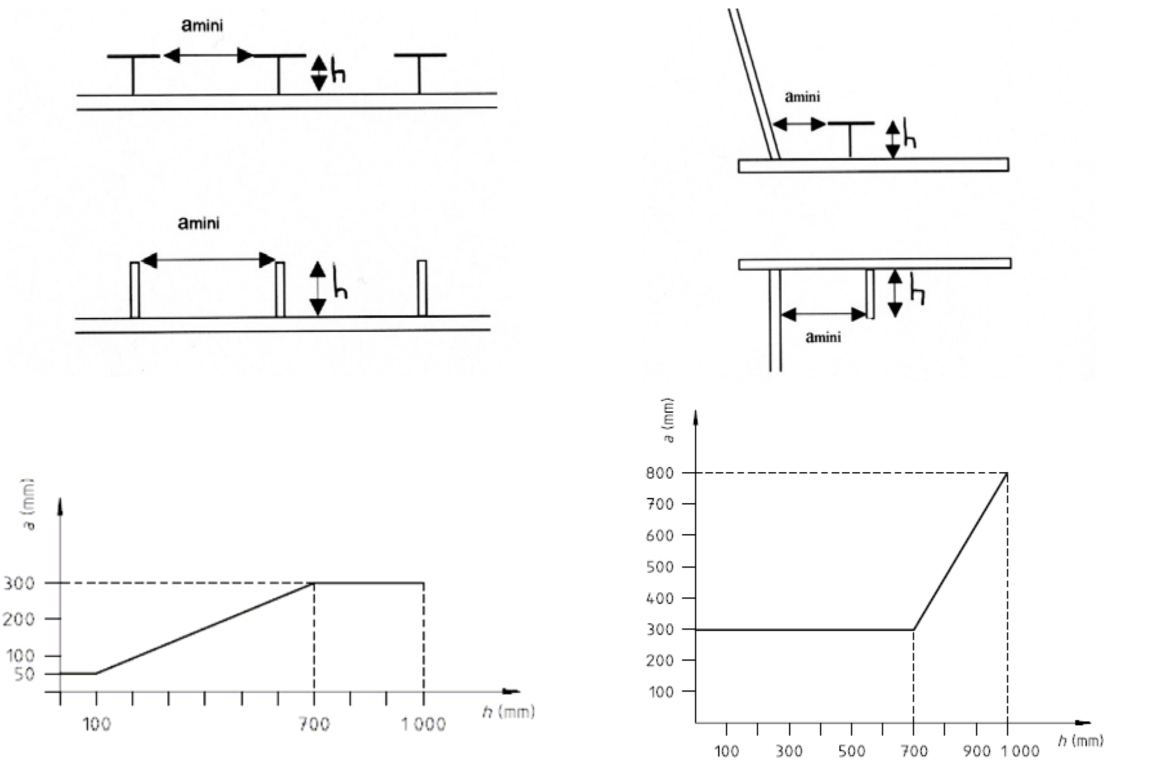

Figure 6.1. Minimum stiffener centre-to-centre distance based on Standard NF EN ISO 12944-3, Appendix C

These minimum distances are not a deciding factor in the majority of cases. However, their compliance should be checked in areas supporting large span box girders. Their bulkheads, in particular, feature stiffeners on very high support bearings whose centre-to-centre distance may come close to minimum values.

Ends of built-up welded girders

At the ends of built-up welded girders, the web either has to be bevelled or a 10 – 15 mm flange extension has to be provided to overlap properly the weld and prevent oxidation (Figure 6.2).

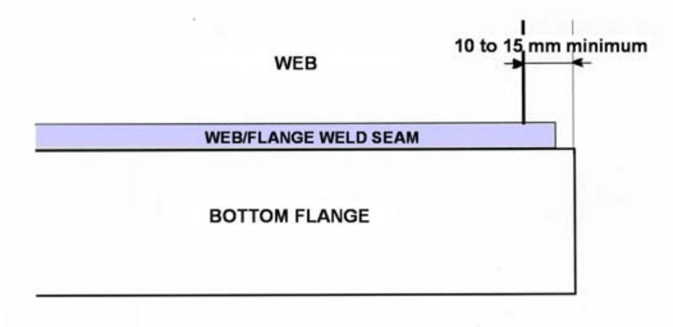

Figure 6.2. Flange extension required for web-flange weld overlap

Nose of triangular gusset plates and longitudinal stiffeners

The nose, in other words the end of certain bevelled edge parts (triangular gusset plates, web longitudinal stiffeners, etc.), has to be removed to ensure integrity of the corner weld fixing these ancillaries to the main section and to ensure proper continuity of anti-corrosion protection. A nose height equivalent to the weld seam plus 2 or 3 millimetres is generally considered satisfactory (Figure 6.3).

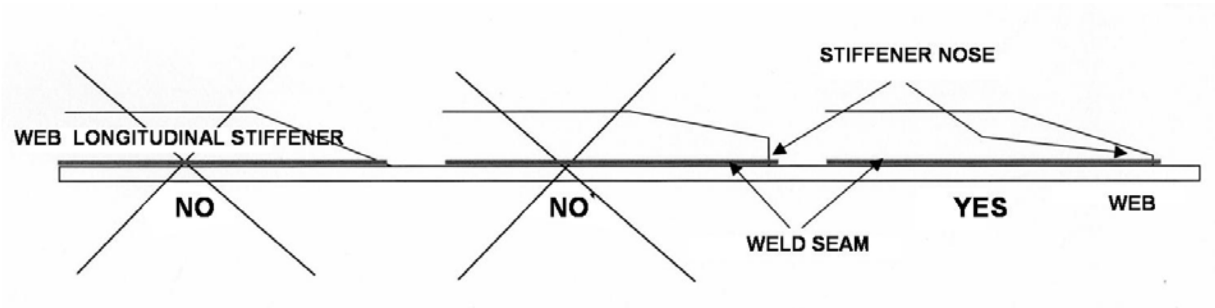

Figure 6.3. Nose design of the triangular parts (web longitudinal stiffeners and gusset plates)

Bottom of cross-beams and directly supported cross-beams

As previously indicated in Section 3 of this guide, a quarter-round cut-out at the bottom of the web on the cross-beams and directly supported cross-beams is required on the side on which it is welded to the main girder web, when the main girder bottom flanges are sloping significantly downwards. This measure prevents rainwater and therefore dirt from building up at the bottom of the posts.

6.3.1.2 - Mobile walkway for inspecting bridge intrados

For some times, site walkways under a bridge intrados have been recommended for facilitating deck, and in particular steel frame, inspection and maintenance.

In practice, these walkways prove difficult to use because of the very long periods of inactivity that they are subjected to and even acts of vandalism at times.

Furthermore, many truck-mounted mobile walkways are now available for inspecting parts of the bridge intrados.

Installation of permanent walkways is therefore only recommended now for bridges, which cannot be thoroughly inspected by a truck-mounted platform positioned under the bridge, because it is too high or by a truck-mounted platform travelling along the concrete slab because the deck width exceeds 30 m or there are insurmountable lateral devices (noise barriers, truck loading restraint systems, etc.).

6.3.2 - Special measures for box girder decks

6.3.2.1 - Minimum deck depth

The allowable internal depth of a box girder must not be less than 1.50 m for inspection purposes. If this depth is not feasible, it is better to opt for a totally closed and watertight box girder design, requiring higher grade plate and fully closed ends.

6.3.2.2 - Access through transverse frames and bulkheads

Box girder composite bridges have transverse frames and bulkheads that must allow free movement of construction and supervision personnel by their shape and manholes respectively. If internal access is to be ensured without risk or difficulty, there must be 60 cm minimum diameter imaginary circular clearance at the centre of transverse frames and bulkhead manholes.

6.3.2.3 - Electrical installation

Steel box girder composite bridges must be fitted with an electrical installation enabling both lighting of the deck interior and power for tools required during maintenance operations (floodlights, electric drills, etc.). Box girder lighting must be sufficiently powerful for people to move around in total safety. It is preferable to place the main switch outside the box girder along with the necessary safety systems. Earthing of the steel structure should also be designed, especially near high voltage networks or railway electrified catenary systems. The power outlet distribution circuit must invariably be independent from the lighting circuit to ensure that lighting inside the box girder is not suddenly lost, if a problem occurs with a tool.

The electrical system must comply with Standard NF C 15-100 for low voltage electrical installations and must be approved by an authorised body after installation.

6.3.2.4 - Steel grating accessway

A box girder bottom flange features longitudinal stiffeners, which often make it difficult to move around inside the deck, particularly when their characteristics (centre-to-centre distance, depth, etc.) are not constant throughout the length of the bridge. This problem can be solved by installing an accessway made up of steel grating components (see Figure 6.4). When the accessway level is correctly set, stepping between transverse frames and crossing both these and bulkheads is greatly facilitated.

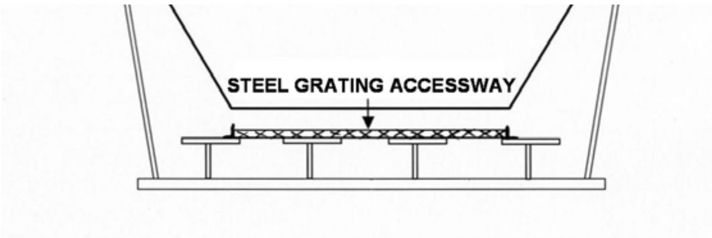

Figure 6.4. Steel grating accessway

The system of securing a walkway of this type must be carefully designed to prevent damage to the anti-corrosion protection. Furthermore, it should be removable to allow close inspection and repainting operations.

6.4 - Piers

6.4.1 - Space between deck underside and top of piers

To facilitate inspection and maintenance operations, minimum clearance of approximately 40 cm and 60 cm should be maintained between top surfaces of pier heads and undersides of steel frames for girder composite bridges and box girder composite bridges respectively.

6.4.2 - Pier head design

6.4.2.1 - Access and inspection pit

Large prestressed concrete bridge pier heads often incorporate a support bearing inspection pit (“bath”) to "comfortably" examine the support bearings, despite the limited space between the top of the pier head and the underside of the deck.

On girder composite bridges, the support bearings can be inspected in relative comfort by gaining access between the main girders on each side of the cross-beam or directly supporting cross-beam. Inspection pits are therefore generally not incorporated.

Inspection pits should be incorporated in box girder composite bridge pier heads featuring hollow piers over 15 m high .

Inspection pit depth is commonly 0.80 to 1.00 m and width 1.0 m. Its length varies based on the support bearing centre-to-centre distance. When the pier shaft is accessible (usually the case), the pit is linked to the shaft interior by a generally cylindrical access manhole (Figure 6.5), closed at the top by a steel trap door, which is also used to drain rainwater, especially during construction.

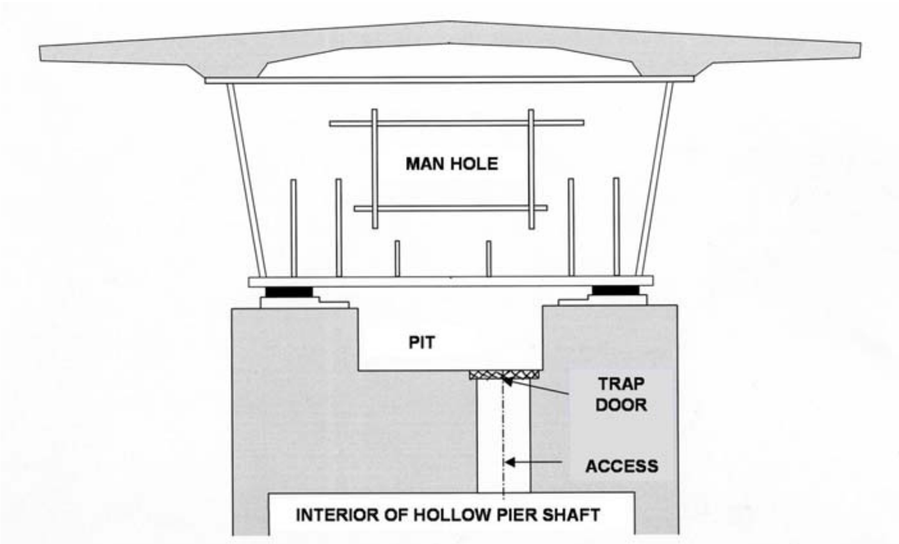

Figure 6.5. Example of pier head with a pit and access to hollow pier shaft

6.4.2.2 - Deck jacking locations

As we have already seen, it must be possible to jack bridge decks after putting the bridge into service for changing their support bearings, resetting their sliding bearing plates or even correcting the effects of accidental support subsidence or a geometry fault.

Figures 3.13 and 3.14 in Section 3 illustrate the most common jacking locations: under the main girders (two locations per girder located on either side of the support bearing), under cross-beams or directly supporting cross-beams on the pier (one location per cross-beam) or under bulkheads on piers.

Jacking locations are embodied on the pier head either commonly by concrete plinths or less commonly by permanent markers (studs, etc.).

6.4.3 - Hollow pier inspection

For the hollow pier inspection and equipment, we refer the reader to Section 9 of the Sétra guide entitled "Ponts construits par encorbellements successifs - Guide de conception" [bridges built by successive cantilevering – a design guide) published in June 2003.

6.5 - Abutments

6.5.1 - Space between deck butt end and abutment retaining wall

A minimum 50 cm gap measured from the butt end abutment side must be maintained between the steel frame (girder or box girder) and the abutment retaining wall to allow access to the abutment retaining wall and deck butt end during inspections and repainting of the steel frame faces on the abutment retaining wall side (Figure 6.6).

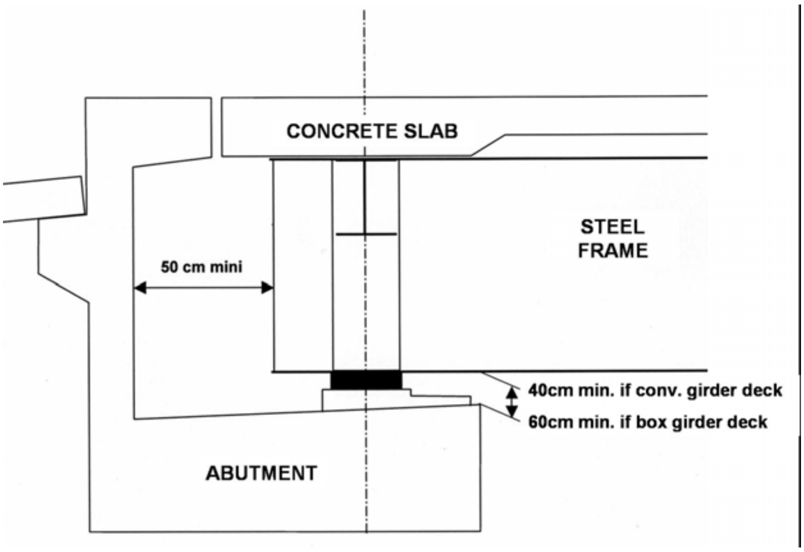

Figure 6.6. Minimum spaces on abutments under or behind the steelwork

6.5.2 - Space between deck intrados and top of crossheads

In common with piers, a minimum 40 cm gap between the top of the abutment crossheads and the underside of the steelwork on girder composite bridges and an equivalent 60 cm gap on box girder composite bridges should be maintained to facilitate maintenance operations (Figure 6.6).

6.5.3 - Deck jacking locations

Both abutments and piers, must incorporate deck jacking locations. For a girder composite bridge, these locations are beneath the girders, between the support bearings and the front face of the crossheads or beneath the abutment directly supporting cross-beams. For a box girder composite bridge, these locations are beneath the abutment bulkheads between the two support bearings.

6.5.4 - Water collection under pavement expansion joints

Despite the care generally given to designing and installing pavement expansion joints, these are never fully watertight throughout their life. The water that runs through them has to be collected or else the steel frame, support bearings and abutment crossheads will be dirtied.

Figure 6.7 shows the most satisfactory measures, involve fixing a steel channel section gutter directly beneath the expansion joint. Supported by galvanised steel brackets, this gutter must be centred beneath the pavement expansion joint, thereby requiring a 20 - 30 cm corbel on the deck side. To prevent splashing, the water should be channelled by vertical elastomeric flaps, which effectively close the gap between the structural joint and the gutter. These flaps, which are usually different to those delivered with the expansion joint, must be weighted to maximise their insensitivity to air pressure from trucks.

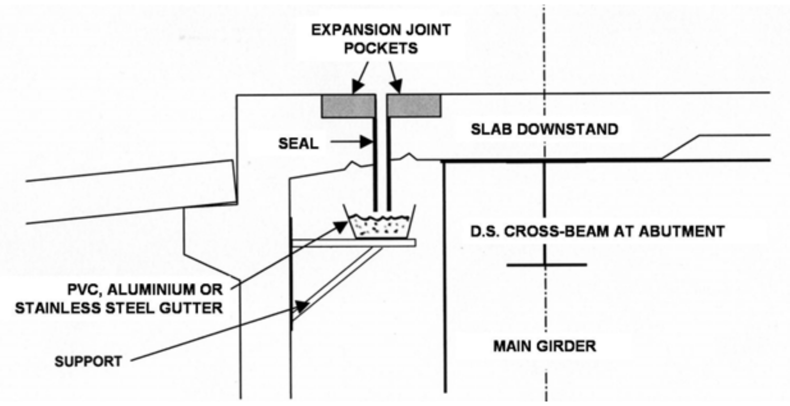

Figure 6.7. Collection of water coming from the pavement structural joint

6.5.5 - Electrical installation

Bridge abutments must be provided with the same electrical facilities (lighting, power outlets) as the deck of a box girder composite bridge. In particular, a switch must be installed in the immediate vicinity of the abutment access door.

6.5.6 - Restricting access to abutments

Closing the abutments of large bridges is usually recommended to prevent malicious acts on support bearings and expansion joint gutters as well as to prevent intruders entering a box girder deck. This measure also has aesthetic advantages because the closing walls do conceal the abutment interior, which is seldom attractively finished.

In the case of a composite bridge, closing the abutments with concrete walls is not an ideal solution because they are permanent and this can make repainting of certain parts of the steel frame very difficult.

A better solution would involve closing abutments with very rigid galvanised steel panels, e.g. steel grating panels, which could be removed during repainting operations.

6.5.7 - System for preventing people walking along main girders

On a long span or wide twin girders composite bridge, the main girder bottom flanges are often very wide and may even form unintentionally a walkway along their webs.

An effective solution to totally prohibiting this practice would be to obstruct the 2 or 3 metres of bottom flange with highly inclined (45° if possible) removable plates, which are impossible to walk on (Figure 6.8). Nevertheless, it should be noted that these plates must be removed during close inspections because they prevent rigorous examination of the main girder bottom flanges.

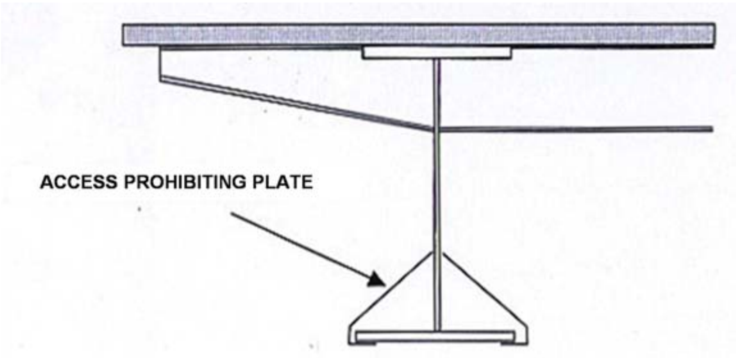

Figure 6.8. Inclined access prohibiting plate

Less effective, but simpler and less penalising solutions, can be implemented for prohibiting access to the steel frame. These generally comprise preventing access from the abutments to the main girders by installing impenetrable devices (wire fences, architectural components, etc).

6.6 - Related bibliography

Anti-corrosion protection

BOA [AND 97] [MAI 99] [BIN 01]

OTUA [AND 00] [MAT 04] [BIN 04]

■ ■ ■

601: Detail of a gusset plate nose. 602-604: Lighting inside box girders. 605: Seal for bulkhead closing door on abutment. 606: Steel grating walkway installed at box girder transverse frame crossing points. 607: Dryer.

611: Inspection platform. 612: Access prevention device on main girder bottom flange near abutment. 613: Support and jacking plinths on pier. 614: Support and jacking plinths on abutment. 615: Abutment closure. 616: Bottom access to hollow pier. 617: Local separation of finishing coat.

7 - Recommendations for DCE preparation

This section contains a number of recommendations for preparing the DCE (Dossier de Consultation des Entreprises) [contractor consultation package] for a composite deck bridge, particularly for the written documents in this package. As it is impossible to be exhaustive, these recommendations focus on the clauses heavily influenced by the structure of the deck and construction methods pertaining thereto.

7.1 - Type of consultation

The Owner decides on the conditions for launching the call for tenders. It may be decided to divide the scope of work into individual work packages or technical work items. In this context, the tenderers are most often contracting groups comprising a civil engineering contractor and a steelwork contractor.

For very simple bridges, the call for tenders may not foresee any division of the work. Under these conditions, tenderers may be single contractors rather than contracting groups. The Owner may thus be led to sign a contract with a civil engineering contractor without knowing which steelwork will build the steel frame; this is undesirable, given the importance of this portion of the structure.

In either of the above cases, the call for tenders may be open or restricted.

7.2 - DCE composition

Except, in very special cases, DCEs are made up of three sub-packages or schedules.

Sub-package 0 is limited to tender terms and conditions (RC) and the public tender notice (AAPC).

Sub-package I contains the documents that will form the contract. It includes framework documents for the tender (AE), price schedule (BP), detailed cost estimate (DE), the CCAP (particular administrative conditions of contract applicable to French government contracts), CCTP (particular technical conditions of contract applicable to French government contracts), SOPAQ (quality assurance plan organisation package), SOSED (construction waste control package) and sometimes price break-down and sub-detail framework documents. This sub-package also contains a series of documents appended to the CCTP, notably:

- an existing site drawing,
- operational cross sections of the supported road (1),
- a longitudinal profile and a horizontal alignment of the supported road (2),
- a plan view of the bridge,
- a longitudinal cross-section of the bridge,
- deck typical cross sections,
- a drawing of deck superstructure details,

- a drawing of steel frame details (post, stiffeners, deck/steelwork connections, directly supporting cross beam at abutments, etc.),
- formwork drawings for piers and abutments,
- contract part of the geotechnical survey, i.e. usually the site investigation results,
- a land survey drawing for areas able to accommodate site installations and their possible access roads.

If required, sub-package I will also contain specific studies directly affecting bridge design: hydraulic, wind effect studies, etc. A part of the architectural study may also be included in sub-package I to give a contract value to information only contained in these study documents (reference of formwork panel mould, special aggregates, etc.). If the bridge is to be built above or near a busy road, sub-package I will also include a document listing road, rail or river operating restrictions, with which the contractor will comply during the work. In urban environments, drawings of concessionary service networks, which may affect the work, are frequently included.

Sub-package II only contains documents of an informative nature. For a composite bridge, this sub-package generally includes:

- a drawing of the launching area, if required,
- a drawing of steelwork material distribution,
- a prestressing cable layout (for transversely prestressed bridges),
- construction kinematics detailing steel frame launching or crane installation phases and various concreting phases,
- a preliminary bill of quantities,
- the architectural study,
- non-contract part of the geotechnical survey, i.e. foundation preliminary design conducted by study geotechnical laboratory.

Sub-package II may also include deck reinforcement preliminary design drawings, calculations and even miscellaneous studies made available to contractors.

(1) These data may be shown on the deck cross sections.

(2) These data may be shown on the bridge longitudinal section and plan view.

7.3 - Consultation rules

7.3.1 - Additions to cctp / technical proposals

Technical proposals are details complementing the basic solution, which contractors are required to provide with their tenders. The sub-clause entitled "Compléments à apporter au cahier des clauses techniques particulières" [additions to particular technical specifications] within Clause 2 of the consultation rules document details the bridge segments to be covered by tenderer technical proposals. For a composite bridge deck, these proposals generally apply to:

- material origin, concrete mix design and placement,
- paint system,
- support bearings,
- waterproofing course laying procedure,

- pavement expansion joints,
- slab transverse prestressing procedure, if relevant.

7.3.2 - Alternatives

To encourage competition, the Owner may allow Contractors to offer alternatives, i.e. to modify certain characteristics quoted in the DCE contract documents. Unlike technical proposals, these alternatives may require adjustments to the price schedule, cost estimate and, of course, both CCTP and drawings.

When the slab construction or steel frame installation method has been detailed in one of the Sub-package I documents, the Owner may sometimes allow alternatives for these aspects. However, these alternatives will be likely to cause variation in certain major quantities, so the Owner must require the tenderer to submit documentation featuring both detailed justification of these quantities and design calculations.

Conversely, when the slab construction or steel frame installation method has not been made contractual, this major aspect of the project must imperatively be detailed in the technical memorandum submitted by the Contractor in support of his tender or, better still, be considered additions to the CCTP to be provided by the Contractor.

When in doubt, it is preferable to make contractual all project major provisions including those concerning construction methods and to allow alternatives on certain points only. This approach requires the Contractor to provide the necessary details for in-depth examination of his alternatives and allows all project major provisions to be made contractual.

7.4 - Act of engagement

7.4.1 - Tender validity period

For simple composite bridges, for which no alternative is permitted, the tender appraisal phase may be very brief. In this case, the tender validity period laid down in the first clause of the act of engagement may be quite short (90 days).

When the bridge is more complex or likely to be subject to major alternatives, a much longer bid validity period should be laid down (180 days or more) to enable the Owner to perform all necessary analyses and checks.

7.4.2 - Preparation period

To prevent site operations being delayed by excessively slow construction studies, it is essential to specify a preparation period, during which designers will gain a significant advance on the site progress. For simple bridges, a design lead period of one or two months is generally sufficient. For more complex bridges, a much longer design lead period of up to 6 months may be necessary. Irrespective of the size of the structure, this lead period is also used to order and sometimes procure steel plate for the steel frame.

7.5 - Particular administrative conditions of contract (ccap)

7.5.1 - General documents forming the contract

For a composite bridge, the general documents forming the contract include at least the following CCTG constituents:

- Fascicule No.2 "Terrassements généraux" [general earthworks];
- Fascicule No.4 title III "Aciers laminés pour construction métallique" [rolled steel for steelwork] supplemented by the Sétra informative memorandum entitled "Approvisionnement en tôles d'acier pour ouvrages d'art" [procurement of steel plate for civil engineering structures], published in March 2007, in relation to the "NF-Acier" quality mark;
- Fascicule No.56 "Protection des ouvrages métalliques contre la corrosion" [anti-corrosion protection of steel structures];
- Fascicule No.65 "Exécution des ouvrages de génie civil en béton armé ou précontraint" [construction of civil engineering structures in reinforced or prestressed concrete]; 2008 version;
- Fascicule No.66 "Exécution des ouvrages de génie civil à ossature en acier" [construction of steel frame civil engineering structures];
- Fascicule No.68 "Exécution des travaux de fondation des ouvrages de génie civil" [construction of foundations for civil engineering structures].

Circular No. R/EG3 of 20th. July 1983, entitled "Transports exceptionnels, définition des convois types et règles pour la vérification des ouvrages d'art" [abnormal transport, definition of standard loads and rules for checking structures] published by the French roads directorate for structures supporting this type of vehicle, should also be mentioned for bridges subjected to abnormal convoys.

Furthermore, the following specific documents should be added, when in an earthquake zone(*):

- Standard NF EN 1998-2, its national appendix and Standard NF EN 1998-2/NA on earthquake design;
- Decree on seismic risk prevention and Order on classification and rules governing earthquake-resistant construction of bridges in the "normal risk" category(*).

(*) At the completion date of this guide, i.e. mid-July 2009, these documents were still not available.

7.5.2 - Monitoring period for construction studies

Adherence to approval times is very often a source of dispute between the Contractor, Engineer and Owner. We therefore recommend including wording in the CCAP sub-clause entitled "Etudes d'exécution des ouvrages" [construction studies], which stipulates clearly:

- the documents that the Engineer considers to form an indivisible set,
- the appraisal times that the Engineer undertakes to satisfy in terms of examining the initial documents, then subsequent documents.

The following text could be a drafting example:

Appraisal and approval times for construction documents

The Contractor shall submit construction studies to the Engineer for approval on a structural segment basis and in the form of standard document groups (e.g. formwork drawings, reinforcement drawings and design calculations for the structural segment considered). The relevant construction procedures shall be attached.

The Engineer will inform the Contractor of his observations in writing, within thirty (30) working days for initial first examination of the "longitudinal bending of deck" and "transverse bending of deck" document groups and fifteen working (15) days for initial examination of the other document groups. These periods will be reduced to fifteen (15) and eight (8) working days for subsequent examinations of these document groups.

It should be noted that in the event of sequenced arrival of documents in the same group, these periods would start at the date of receiving the last document.

7.5.3 - Constraints associated with operating in the public domain

For bridges built very near to or even above busy roads, these roads usually need to be closed to traffic before certain work can be performed. These interruptions are only possible for very limited periods during the day or at night, resulting in major hindrance to the construction progress. under these circumstances, care must be taken to give due notice of such restrictions in a sub-clause of CCAP Clause 8 entitled “Sujétions résultant de l’exploitation du domaine publique ou privé” [constraints associated with operating in the public or private domain], by either directly listing the constraints in this sub-clause or referring to a special safety memorandum or any other DCE contract document prepared in conjunction with the relevant road operator.

7.5.4 - Lifting the hold points

The sub-clause of CCAP Clause 9 entitled “Essais et contrôles des ouvrages en cours de travaux” [testing and inspection of structures during construction] recalls the main construction hold points and the time required by the Engineer for lifting them. The table below provides a non-exhaustive list of construction hold points that may apply to a composite bridge deck and the average time required for their removal.

Hold points	Time
Steelwork	
Authorisations to perform shop or site welding (acceptance of QA plan, DMOS (welding procedure description), QMOS (welding procedure approval), weld quality, material certificates, and welding product acceptance certificates).	5 days
Authorisation to ship shop-fabricated segment to site (acceptance of shop welding and trial assembly inspection and part dimensional test reports).	2 days
Authorisation to weld on site.	
Authorisation to weld box girder longitudinal stiffener continuity segments.	1 day
Acceptance of shop and site welds.	1 day
	1 day
Steelwork anti-corrosion protection (civil engineering process)	
Acceptance of documents prior to shop fabrication (shop QA plan).	
Acceptance of shop suitability test.	5 days
Acceptance of shop painting system before shipping segments to site.	1 day
Acceptance of documents prior to site fabrication (site QA plan and environmental precautions).	2 days
Acceptance of site suitability test.	
Acceptance of finished painting system before removing scaffolding	5 days
	1 day
	2 days
Steel frame installation operation	
Authorisation to start a crane installation phase.	2 days
Authorisation to start a launching phase.	2 days
Slab concreting and formwork removal operations	
Acceptance of mobile formwork.	1 day
Acceptance of suitability control segment.	1 day
Authorisation to concrete a pad or a slab segment.	1 day
Slab prestressing (if relevant)	
Authorisation to start prestressing slab.	1 day
Authorisation to stress before cutting reinforcement.	1 day
Authorisation to inject prestressing ducts.	1 day

Table 7.1. Times for lifting hold points

7.5.5 - Specific guarantees

The CCAP Clause 9 sub-clause entitled “Garanties particulières” [specific guarantees] must recall that the Contractor guarantees certain bridge parts against all defects for a given time. For a composite bridge, aside from deck waterproofing and possible anti-graffiti painting, these guarantees must include the anti-corrosion protection system applied to the steel frame and must be established with reference to the times and defects listed in Clause 1.5 of CCTG Fascicule 56.

7.6 - Particular technical conditions of contract (CCTP)

7.6.1 - Preamble

We include below a number of points to be exhaustively detailed in the CCTP of the contractor consultation package (DCE). These points can be classified into two categories; the first of which comprises documents featuring additions to the CCTG fascicules and applicable standards because either the latter documents are incomplete or none of them covers the relevant area and the second of which comprises documents impacting on options proposed by these standard documents.

It is should be recalled that the general requirements provided by CCTG fascicules and standards should not be “copied” into the CCTP because they can be opposed by the Contractor as soon as these documents are approved in the contract CCAP and CCTP.

7.6.2 - Programme of bridge construction studies

Utmost care should be given to construction study quality and programme. With respect to this last point, a clause should be included in the CCTP entitled “Programme des études d’exécution” [construction study programme] and drafted as follows:

Construction Study Programme

The Contractor shall provide a construction study programme including a list and preliminary timetable of documents to be prepared.

The list shall include documents required for both temporary and permanent works. It shall be drawn up in compliance with the design framework laid down by the contract.

The preliminary timetable shall include the document submission timetable and the forecast or required dates for obtaining the Engineer’s approval in compliance with the minimum times laid down by Clause 8.2 of the CCAP. Its format shall be a bar diagram clearly highlighting critical tasks and leeways.

7.6.3 - Bridge construction studies

7.6.3.1 - Actions

The CCTP must detail all actions to be considered in the structural design calculation checks. The majority of these actions are specified Standards NF EN 1990 and 1991, and their national appendices:

- deck dead weight (1),
- deck equipment weight,
- concrete shrinkage and creep (2),
- general thermal effects (gradient and uniform variation),
- road traffic and pedestrian live loads and fatigue loads (3),
- impact loads on restraint systems,
- slab prestress, if relevant,
- wind load when in service (4),
- wind load during construction (5).

These actions must be complemented by a number of construction-related actions, in particular the dead load of temporary structures and special equipment (launching nose, mobile formwork, etc.).

Depending on the case, it may also be appropriate to specify certain additional actions:

- abnormal convoys or military loads (6),
- earthmoving machines not complying with the highway code,
- impacts on certain supports or even the deck,
- loads due to water or ice,
- earthquake loads.

(1) Specify in particular the concrete mix density.

(2) Specify the level of humidity.

(3) Specify the fatigue class for traffic and trucks to be adopted in relation to Standard NF EN 1991 Part 2.

(4) Specify the reference height Ze, reference velocity Vb,o, direction coefficient Cdir, ground category, orographic coefficient Co(Ze) and force coefficients.

(5) Specify the return period, season coefficient and if reduced wind velocities can be adopted for installation.

(6) It should be remembered that Eurocodes do not cover military loads. If these are to cross the bridge, they should be described in detail in the CCTP.

7.6.3.2 - Combined actions

In general terms, the CCTP recalls the various combined actions to be foreseen. These stem from Standard NF EN 1990 and its national appendix.

7.6.3.3 - Deck design check

The CCTP must stipulate the following:

- exposure classes to be used when determining allowable slab crack opening (Standards NF EN 1992) and passive reinforcement covers,
- main design assumption adopted by the Engineer (length and weight of launching nose if deck is to be launched, deck sectional break-down, slab construction methods, etc.),
- method to be retained for determining cracked areas in overall analysis (Standard NF EN 1994-2),
- opportunities for upgrading certain sections from Class 3 to Class 2 (Standard NF EN 1994),
- rules governing addition of certain passive reinforcing bars.

Depending on the bridge, the CCTP may also impose:

- taking bridge horizontal curvature into account in longitudinal bending calculations,
- finite element design of one deck segment or assembly,
- second order design of very slender piers (in this case, CCTP must specify design assumptions for pier head horizontal loads).

Lastly, for a bridge whose span distance exceeds 100 m, the CCTP must specify whether dynamic analysis of the structure is required or not.

7.6.4 - Structural steel grades

Reference documents for steelwork materials are Standards NF EN 10025-1 to 4 (steels), Standard NF EN ISO 13918 (stud connectors), Standard NF A 36-270 (variable thickness plates), when required and CCTG Title III Fascicule 4,

The CCTP must specify the steel grade for each steel frame element (1), whether Z quality plates are required and for which steel frame components.

(1) Main structural steel grades are S355K2+N (e ≤ 30 mm), S355N or S355M (30 mm < e ≤ 80 mm), S355NL or S355ML (e > 80 mm). S460 ML steel is also used on occasion for certain sections on large span bridge piers. Standard grades for rolled steel joists are S355K2+N, N, NL, M or ML.

7.6.5 - Steel frame fabrication

Reference documents for steel frame fabrication are Standard NF EN 1090-2, its national appendix and CCTG Fascicule 66.

In essence, the CCTP must provide the additional data foreseen in Appendix A1 of Standard NF EN 1090-2 and decide on the optional requirements listed in Appendix A2 by detailing in particular:

- the assembly fabrication classes (1),
- whether a quality plan is required (2),
- whether impact tests are foreseen (3),
- whether specific requirements for dimensional tolerances, plate supply and surface conditions are necessary (4),
- whether traceability for each product is specified (5),
- whether specific welding quality levels are required (6),
- any parts requiring full penetration welds (7).

The CCTP must also state whether a trial assembly is required and, if so, which parts of the structure are concerned by this assembly (8).

(1) Recommended fabrication class is EXC3 for all assemblies except tension flange butt joints, which are subject to fabrication class EXC4.

(2) A quality plan is an essential requirement.

(3) Welding procedure rating complies with recommendations given in Standard NF EN ISO 15614-1 with grade and quality equivalences satisfying the following requirements: minimum fracture energy for impact bending tests less than or equal to that of the rating fabrication steel; impact bending test temperature greater than or equal to that of the rating fabrication steel.

(4) Class C2 requirements given in Standard NF EN 10163 should be imposed for long product surface condition, Classes S1 for supplied plate centre, E1 for plate edges and dimensional tolerance Class B in compliance with Standards NF EN 10160 and NF EN 10029 respectively.

(5) Individual traceability requirement is recommended.

(6) Assemblies foreseen within fabrication class EXC4 should be subject to required quality level B+ for full penetration corner welding and for full penetration welding of transverse member flanges to main girders.

(7) This usually involves butt welding of main girder webs and flanges, butt welding of directly supporting cross-beam top flanges on main girder flanges and longitudinal stiffener butt welds.

(8) Trial assembly should be required for all decks other than straight twin girder composite decks of constant depth and width.

Finally, we strongly recommend reiterating in the CCTP that drill holes not shown on the steelwork fabrication drawings (approved by the Engineer), are strictly prohibited.

7.6.6 - Assembly area

The CCTP must define areas of land made available to the Contractor for steel frame assembly operations and detail possible easements to which these areas are subject.

For long-term or urban area sites, the CCTP may impose the use of surfacing material (clinker, cement-bound graded aggregate, etc.) allowing construction vehicles to circulate in all seasons and maintaining site access road cleanliness.

When the assembly area is located beneath a retaining wall, whose construction will be interrupted during steel frame assembly, the CCTP must state clearly the precautions to be taken by the Contractor during this period.

7.6.7 - Steel frame installation

On some sites, the CCTP may impose specific precautions during steel frame installation: closure of roads crossed, obligation to use a holding winch, etc.

7.6.8 - Steel frame anti-corrosion protection

In terms of anti-corrosion protection, for which the CCTG Fascicule 56 is the reference document, the CCTP must specify in particular:

- whether the site is subject to a tropical atmosphere,
- the site environmental class based on Standard NF EN ISO 12944-2 (1),
- the type of anti-corrosion protection (2),
- the required certification class (3),
- the extent of surfaces to be painted (4),
- the surfaces considered to visible (5).

Requirement for the “ACQPA-Systèmes anticorrosion par peinture” [ACQPA painted anti-corrosion systems] quality mark or equivalent certification quality mark is strongly recommended.

In relation to applying anti-corrosion protection guarantees, the CCTP must state in which category, specified in CCTG Fascicule 56 Clause 1.3, the steel frame elements (6) are classified and must describe overall visual perception areas (ZPVG) as defined in Sub-clause 1.5.2.3.1 of the above fascicule (7). Furthermore the CCTP must state whether the colour stability guarantees, specified in Fascicule 56 Clause 1.5, are required (no guarantees can be required in a tropical atmosphere; only the colours shown on the ACQPA card are subject to this guarantee in a non-tropical atmosphere).

If the temporary bracing can stay in place, which may the case for shallow box girders, the CCTP must clearly indicate that these braces must receive the same anti-corrosion protection as the interior of the box girder itself.

Finally, the various layers of the paint system should be of different colours because this will facilitate checking of their thicknesses.

(1) Possible environmental classes, also called corrosiveness categories, are C2 (low-level pollution, particularly in rural areas), C3 (urban and industrial atmospheres subject to moderate sulphur dioxide pollution; coastal areas subject to low-level salinity), C4 (industrial areas and coastal areas with moderate salinity), C5M (coastal and maritime areas subject to high salinity).

(2) Most common protection is paint on bare steel.

(3) Possible certification classes are C3, C4 and C5Ma, the required class is usually the environmental class plus one point.

(4) All steelwork surfaces must be painted except areas in contact with the slab, for which only the 5 cm lateral returns are painted.

(5) For a twin girder composite bridge, all painted surfaces are considered visible surfaces. For a box girder composite bridge, internal surfaces are considered invisible.

(6) In general, all steel frame members is considered to belong to Category 1.

(7) For a twin girder cross-beam composite bridge, the overall visual perception areas (ZPVG) are the external faces of each girder, including the flange bottom faces, and the internal faces of each girder, including the flange top faces and the cross-beams seen from each abutment, i.e. one ZPVG per girder or one ZPVG per abutment. For a twin girder directly supporting cross-beam bridge with cantilevers, the ZPVGs are the external faces of each girder, including the flange bottom faces and the directly supporting cross-beam cantilevers, and the internal faces of each girder, including the flange top faces and the directly supporting cross-beams seen from each abutment, i.e. one ZPVG per girder or one ZPVG per abutment. For a box girder composite bridge without directly supporting cross-beams or props, the ZPVGs are the external faces of each web, the bottom face of the box girder bottom flange and internal face of the box girder, i.e. four ZPVGs.

7.6.9 - Slab concrete

Reference documents for all concrete mixes are CCTG Fascicule 65, Standard NF EN 206-1 and LCPC documents entitled "Recommandations pour la prévention des désordres dus à l'alcali-réaction" [Recommendations for preventing alkali reaction damage] and "Recommandations pour la prévention des désordres dus à la réaction sulfatique interne" [Recommendations for preventing internal sulphate reaction damage] published in June 1994 and August 2007 respectively.

The CCTP must state the level of alkali reaction prevention and whether the specifications provided in of the LCPC technical guide entitled "Recommandations pour la durabilité des bétons durcis soumis au gel" [Recommendations for durability of hardened concrete subjected to freezing] published in December 2003 apply. If so, the relevant concrete mixes must be specified. Standard NF EN 206-1 has been drawn up for a 50-year structural life, so it is therefore essential to state that the structural life of the bridge to be built is 100 years.

For each concrete mix or structural part, the CCTP must also stipulate:

- exposure classes (1)
- chloride classes (2),
- internal sulphate reaction classes (3),
- concrete strength class based on Standard NF EN 206-1 (4),
- minimum equivalent binder content (5),
- Eeff/Leq ratio (6),
- cement type and possible additional characteristics (7),
- concrete possible additional characteristics (8).

(1) The most frequent exposure classes are XC2 and XC4 for carbonation, and XH2 and XH3 for internal sulphate reaction. Depending on the bridge location, classes XF1, XF2 and XF4 (freeze/thaw), XS1 and XS3 (seawater spray), and XA1 (chemical attacks) are also foreseeable.

(2) Most common chloride classes are Cl0,2 and Cl0,4.

(3) Internal sulphate reaction classes are XH2 and XH3.

(4) Most common concrete strength classes are C35/45 and C40/50.

(5) Minimum equivalent binder content is generally between 330 and 385 Kg/m^3.

(6) This ratio is generally between 0.40 and 0.45.

(7) Additional characteristics for cement based on Standard NF P15-301 may be PM (seawater hardening) or ES (sulphate water). Additional characteristic CP (cement for prestressed concrete) is essential for prestressed slabs.

(8) Slab concrete additional characteristics, which must always be selected, are RAG (alkali reaction prevention) and LRE (limited endogenous shrinkage). Depending on conditions, characteristics such as EQP (specific requirements for facing quality) and LCH (limited heat of hydration) can also be retained.

7.6.10 - Slab passive reinforcement

Reference documents for passive reinforcement are CCTG Fascicule 65 along with Standards NF A 35-015, NF A 35-016-1 and 2, NF A35-019 and NF A35-028. For reinforcement covers, the reference document is Section 4 of Standard NF EN 1992-1-1 and its national appendix, Standard NF EN 1992-1-1/NA.

The CCTP must specify steel grades to be used (1) and state whether they are subject to particular specifications (fatigue resistance, ductility, etc.). The CCTP must also specify the design reinforcement concrete covers (2); these may vary slightly with respect to those strictly specified in the above European standards.

(1) Steel grades most often used are B500B (high-strength steels) and B235C (low-carbon steels).

(2) Passive reinforcement concrete covers are generally between 30 and 50 mm.

7.6.11 - Slab prestressing

The reference document for prestressing is CCTG Fascicule 65.

If the slab is prestressed, the CCTP must accurately specify the number, type, strength and relaxation classes of strands making up the deck transverse prestressing cables. It must also specify the type of grout (1) and prestressing duct (2) as well as the Owner's requirements for final anchorage protection (3).

(1) Prestressing grout is either cement grout or grease.

(2) Prestressing ducts are sheet-steel formed, if protection is ensured by cement grout, or PEHD (high density polyethylene), if grease is used (greased sheathed single strands).

(3) Use of sealed anchorages is strongly recommended.

7.6.12 - Slab construction

The reference document for facings and, more generally, slab construction is CCTG Fascicule 65. Standard NF EN 13 670 had not yet been applied at the time of guide completion but should also have an important role in the future.

7.6.12.1 - Facings

For slab bottom and lateral faces, the CCTP must specify in particular the class of facing retained on the basis of CCTG Fascicule 65 Clause 62; possible classes are "soignés simples" [standard finish] and "soignés fins" [fine finish].

7.6.12.2 - Control panels, segments or concrete mixes

The CCTP must specify whether the Contractor is required to perform a complete trial segment for the concrete suitability tests or whether these can be reduced to producing control segments or panels. In every case, the CCTP must specify the dimensions of these segments and the passive reinforcement or even the ducts incorporated in them, if the slab is prestressed.

In addition to these requirements, the CCTP must lay down acceptance conditions for these tests: inspection of facings, coring in most congested areas, flatness of facings, etc.

7.6.12.3 - Formwork removal

The CCTP should specify a minimum concrete strength at the time of removing formwork. This must not be less than 18 MPa. No formwork should be removed less than 24 hours after completion of slab concreting.

7.6.12.4 - Construction kinematics for in-situ concrete pours

If the slab is cast over mobile formwork, the CCTP should clearly specify:

- either that non-continuous sequencing is compulsory, which is the usual case,
- or, conversely, that progressive continuous construction of all or part of the slab has been foreseen and should be complied with because of the risks to which crossed road users are subjected.

7.6.12.5 - Precast slabs

If precast slabs are used, the CCTP must clearly specify:

- dimensions of slab segments and closing concrete volumes,
- installation conditions for the segments adopted to allow preliminary design of the steel frame, in particular the weight of the slab segment installation machine, if used,
- slab segment-steelwork bearing conditions (minimum flange overlap of segment, joints waterproofing details, etc.),
- for full width segments, mix design and conditions for grouting cavity between slab segment undersides and steel frame top flanges,
- provisions to be implemented at concrete construction joint surfaces.

The CCTP must also require submission of a construction procedure detailing:

- slab segment precasting and storage,
- installation (design of installation equipment, movement procedure, segment adjustment details, construction measures for segment connection, etc.),
- closing concrete volumes (concrete mix design, waterproofing measures, method of grouting gap between haunching concrete underside and steel frame, formwork if required, etc.).

We also recommend that the CCTP requires construction of a control slab segment to check and test conditions governing this procedure.

7.6.12.6 - Support vertical adjustments

The CCTP must state whether post-installation support vertical adjustment is foreseen and, if so, it must specify the supports concerned and the vertical adjustment heights.

7.6.13 - Maintenance equipment

Maintenance equipment is not always well defined in DCE drawings, so the CCTP must describe as accurately as possible (capacities, components, anti-corrosion protection, installation tolerances, etc.) the maintenance equipment that the Owner wishes to have installed on the bridge. This equipment should specifically include:

- fixing rails sometimes embedded in the slab to allow immediate or future installation of services under it,
- an inspection platform, if this equipment is foreseen,
- support inspection and closing equipment, if piers are hollow.

If the deck is a box girder, its internal electrical installation should also be specified, particularly lighting, possible dryers and box girder internal access systems.

7.6.14 - Site supervision

Supervision of composite bridge construction is a major, complex operation to be organised somewhat in advance by the Engineer.

Both the CCAP and the CCTP must detail site supervision and inspection operations, which will frequently represent Contractor hold points.

7.6.15 - Load tests

The CCTP must detail the load test programme.

7.7 - Price schedule

The price schedule included in a composite bridge DCE may be compiled based on standard price schedules and those contained in CCTG fascicules, in particular Fascicules 56 "Protection anticorrosion des ouvrages métalliques" [steelwork anti-corrosion protection], 65 "Exécution des ouvrages en béton" [concrete work], 66 "Exécution des ouvrages métalliques" [steelwork] and 68 "Exécution des travaux de fondations" [foundations].

Listing all prices required for building a composite bridge would be excessively tedious. But, we should recall that the above provisions do prompt provision - for a composite bridge with a launched steel frame and slab cast over mobile formwork – of the following specific prices for composite bridges:

7.7.1 - General prices

Steel frame assembly platform (F)

Composite deck steel frame launching equipment (F)

Mobile formwork for casting composite deck slab (F)

Installation on permanent supports / Final jacking (F)

7.7.2 - Steel frame and anti-corrosion protection prices

Steel for deck frame (kg)

Studs for steel frame-concrete deck connection (kg)

Steel frame assembly and installation on site (F)

Anti-corrosion protection suitability test (F)

Anti-corrosion protection using paint on bare steel (m^2)

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MAIN COMPOSITE ROAD BRIDGES BUILT IN FRANCE FROM 1995 TO 2005

Bridges are classified here by decreasing span distance.
FW precast = Full width precast slabs

A – Twin girder cross-beam composite bridges (Total length Ltot > 200 m)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Seine bridge at Triel	Liaison RD1/RD154	78	635m	124m	11.5 - 16.93m	4.5m	6,300t	Launching	Mobile formwork	2002		Seine bridge at Triel
Monestier-de-Clermont viaduct	A51	38	860m	110m	11.85m	3m	4,000t	Launching	FW precast slabs	2005		Monestier-de-Clermont viaduct
Brioude viaduct	RN102	43	242m	110m	12.5m	2.5 - 4m		Launching	Pushing			Brioude viaduct
Vilaine viaduct	RD773	44	672m	105m	14m	1.9 - 4.4m	2,350t	Launching	Mobile formwork	2002		Vilaine viaduct
Vézère viaduct	A89	19	973 and 1002m	105m	2 x 10.97m	2.6 - 5.3m.		Crane installation and partial launching	Mobile formwork	2004		Vézère viaduct
Vézère viaduct	A20	19	360m	104m	2x 12m	3.15 - 4.25m	2,250t	Launching		1995		Vézère viaduct
Marmande bridge	RD933	47	250m	104m	11m	2.4 - 4m	850t	Launching		1998		Marmande bridge
Loir viaduct		72	340m	100m	2 x 10m	3.95m	2,277t	Launching		2004		Loir viaduct
Mascaret viaduct	A89	33	540m	95m	2 x 12.1m	2.25 - 4.6m	3,500t	Launching and floating derrick	Mobile formwork	2000		Mascaret viaduct
Viaduc de Crivilliers	RN59	54	322m	91m	20.5m	3.3 - 4.3m	1,475t	Launching	Mobile formwork	1995		Viaduc de Crivilliers
Risle viaduct	A28	27	1320m	90m	15m	3.5m	5,295t	Launching from 2 sides	Mobile formwork	2003/4		Risle viaduct
Vienne viaduct - Aixe	RD2000	87	255m	85m	11.65m	2 - 3.35m	820t	Launching				Vienne viaduct - Aixe
Dordogne bridge - Sainte Foix	RD936	33	203m	84m	12.7m		550t	Launching	Mobile formwork	2002		Dordogne bridge - Sainte Foix
Adour bridge - Bayonne		64	206m	84m	22m	1.8 - 3.8m	1,100t	Launching	Mobile formwork	1995		Adour bridge - Bayonne
Yonne viaduct	A19	89	580m	83m	2 x var de	0.95 - 3.05m	2,650t	Launching with 1 part of	Mobile formwork and	1997		Yonne viaduct

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
					12.42 - 14.92m			slab and crane installation	delayed connection			
Roumer viaduct	A85	37	249m	82m	15m	2.8m		Launching	Mobile formwork	2005		Roumer viaduct
Lavedan bridge	RN21	65	249m	81.5m	2 x 10.9m	2.9m	1,350t	Launching	Mobile formwork	1997		Lavedan bridge
Bec viaduct	A28	27	690m	80m	15m	3.5m	2,526t	Launching	Mobile formwork	2004	Curve 2700m	Bec viaduct
Aveyron viaduct	A20	82	273m	80m	2 x 12m	2 - 4.5m	1,350t	Crane installation	Mobile formwork	1997		Aveyron viaduct
Marnaval viaduct	RN4	52	591m	76m	12m	3m	1,700t	Launching	Mobile formwork	1998		Marnaval viaduct
Cher viaduct	A85	37	499m	74.8m	14.8m	1.8 - 3m	1,200t	Launching	Mobile formwork	2005		Cher viaduct
Garrigue viaduct	A75	12	340m	74m	2 x 10.85m	2.65m	1,750t	Launching	Mobile formwork	2001		Garrigue viaduct
Chatelles viaduct	RN59	88	386m	73m	2 x	2.5m	1,800t	Launching	Mobile formwork	1997		Chatelles viaduct
Mirville viaduct	A29	76	310m	72m	2 x 12m	2.85m	1,650t	Launching	Mobile formwork	1995		Mirville viaduct
Réole bridge		33	400m	70m	12.2m	2.5m	850t	Launching	Mobile formwork	1995		Réole bridge
Ingrandes de Touraine viaduct	A85	37	552m	67m	16m	2.8m		Launching		2005		Ingrandes de Touraine viaduct
Sèvre Nantaise viaduct	A87		285m	65m	2 x 10.97m	2.5m	1,400t	Launching	Mobile formwork	2002		Sèvre Nantaise viaduct
Pont de l'Arc viaduct	A43	73	206m	64m	2 x ?	2.3m	1,050t	Launching and crosswise shifting		1997		Pont de l'Arc viaduct
Rocher de l'Escalade viaduct	A43	73	240m	64m	2 x 10m	2.2m	1,150t	Launching		1997		Rocher de l'Escalade viaduct
Avre valley viaduct	Rocade Sud d'Amiens	80	592m	61.5m	2 x var. from 11.7 - 16m	2.4m	3,550t	Launching with part of slab and crane installation	Mobile formwork	1997		Avre valley viaduct
Alses viaduct	RN20	09	500m	60m	11.24m	2.15m	1,350t	Launching	Mobile formwork	1999		Alses viaduct
OA8 bridge - Nancy	A330/RD2bis /RN74	54	200m	60m	14m	2.4m	500t	Launching	Mobile formwork	1996		OA8 bridge - Nancy
Saint-Privas viaduct		07	256m	60m	12.2m	2.15m	600t	Crane installation		2001		Saint-Privas viaduct
OA1 bridge	BP Est de Lille	59	290m	60m	12m	2.20m		Launching	Mobile formwork		Low-level bracing	OA1 bridge

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Blazy viaduct	A20	46	363m	58m	2x 12m	2.25m	1,750t	Launching	Mobile formwork	1998		Blazy viaduct
Musson viaduct	A83		303m	55m	2x 9.85m	2.2m	1,375t	Launching	Mobile formwork	2000		Musson viaduct
Approuage viaduct	RN2	973	350m	55m	10.10m	2m	750t	Launching	Mobile formwork	2002		Approuague viaduct
Creuse viaduct	RN145	23	204m	54m	12m	2m	800t	Launching		2000		Creuse viaduct
Charente viaduct	A837	17	856m	52m	2 x 12m	2m	3,125t	Launching and crane installation	Mobile formwork	1995		Charente viaduct
Somme viaduct	A29	80	460m	52m	15.1m	1.85 - 2.5	1,050t	Launching and crane installation	Mobile formwork	2000		Somme viaduct
Douime viaduct	A89	24	290m	52m	2 x 9.25m	2m	1,265t	Launching	Mobile formwork	2003		Douime viaduct
Aiton viaduct	A43	73	295m	50m	2 x 9m	1.85m	1,000t	Launching	Mobile formwork	1995		Aiton viaduct
Hyères bridge	RN64	29	230m	50m	2x 10.95m	1.80m	1,000t	Launching	Mobile formwork	2004		Hyères bridge
Sinnamary bridge	RN1	Guy.	225m	50m	9.90m	1.9m	425t	Launching	Mobile formwork	1997		Sinnamary bridge
Saussaz viaduct	A43	73	210m	50m	2 x 9.85m	1.8m	950t	Launching		1999		Saussaz viaduct
Egray viaduct	A83	79	330m	50m	2 x 9.85m	2m	1,250t	Launching	Mobile formwork	1999		Egray viaduct
Crempse viaduct	A89	24	307m	49.5m	2 x 12.1m	2m	1,300t	Launching	Mobile formwork	2000		Crempse viaduct
Sèvre Niortaise viaduct	A83	79	480m	46m	2 x 9.85m	1.75 m	1,775t	Launching	Mobile formwork	2000		Sèvre Niortaise viaduct
OA 85 bridge	A77		238m	45.5m	2 x 15.32m	1.6m	1,000t	Crane installation		1999		OA 85 bridge
Rive Gauche viaduct	A83		285m	45m	2 x 12.1m	1.8m	1,175t	Crane installation		1996		Rive Gauche viaduct
Roselière viaduct	A29	76	320m	43m	Var from 10.7 - 14.5m	1.8m	700t	Launching and Crane installation		1995		Roselière viaduct
Nièvre viaduct	A16	80	207m	40m	2 x 9.97m	1.5m	750t	Crane installation		1996		Nièvre viaduct
Maye & Pendé viaducts	A16	80	1000m	38m	9.75m	1.35m	1,500t	Launching		1996		Maye & Pendé viaducts

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Maye viaduct	A16	80	350m	20m	9.75m		1,500t	Launching		1996		Maye viaduct
Dumbéa bridge	Airport road	New Caledonia	244m	34.5m	9m	1.6m		Launching	FW precast slabs	2005		Dumbéa bridge
Intermediate viaduct	A85	37	450m	31m	14.8m	1.35m	1,000t	Crane installation	Mobile formwork	2005		Intermediate viaduct
Charmes-sur-Rhône bridge		07	239m			1m	200t	Launching		1999		Charmes-sur-Rhône bridge
Schwalb viaduct		57	258m		12.14m	2m	700t	Launching	Mobile formwork	2000		Schwalb viaduct

B – Twin girder directly supporting cross-beam composite bridges (Ltot > 200 m)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Jassans bridge	RD131	01	310m	130m	14.7m	De 3.25 - 5m	1,500t	Launching	Mobile formwork	2000	No cantilevers	Jassans bridge
Centron downstream viaduct	RN90	73	450m	125m	13.5m	De 1.9 - 4m	2,100t	Launching	Mobile formwork	2005		Centron downstream viaduct
Saulières viaduct	RN89	19	468m	106m	10.9m	De 3.4 - 5.20m	1,925t	Launching	Partial mobile formwork + composite acting pre-slabs	2005	No cantilevers	Saulières viaduct
Chadon viaduct	A89	19	530m	100m	19.5m	4.4m	3,925t	Launching	Mobile formwork	2002		Chadon viaduct
Saultbesnon viaduct	A84	50	345m	81m	23.5m	3.5m	2,250t	Launching	Mobile formwork	2001		Saultbesnon viaduct
Dordogne viaduct	A20	46	1070m	80m	21.3m	3.2m	8,150t	Launching and crane installation	Mobile formwork	2001		Dordogne viaduct
Achard viaduct	A43	73	374m	77.5m	2 x 9.85m	3m	2,300t	Crane installation	Mobile formwork	1998		Achard viaduct
Valenton rail viaduct	VDO du Val-de-Marne	94	216m	72m	21m	3.15m	975t	Launching and crane installation		1995		Valenton rail viaduct
Lot viaduct	A20	46	535m	70m	22.47m	1.9 - 3m	3,250t	Launching and crane installation	Mobile formwork	2002		Lot viaduct
Sèvres Nantaise viaduct - Clisson	RN149	44	210m	67.5m	12m	2.25m	800t	Launching	Precast slabs	2003		Sèvres Nantaise viaduct - Clisson
Langeais viaduct	A85	37	653m	67m	16m	3m		Launching	Mobile formwork	2005		Langeais viaduct
Access viaducts to Rouen mobile bridge	RN338	76	407m	64m	2 x 15.58m	2.2 - 3.5m	7,600t	Launching	Precast slabs	2005		Access viaducts to Rouen mobile bridge
Laize viaduct	RD562	14	352m	60m	21.2m	3m	2,150t	Launching	Composite acting pre-slabs	2004		Laize viaduct
Coteau viaduct	A85	37	250m	57.5m	19.4 m	2.6m	975t	Launching	Pre-slabs			Coteau viaduct
Sauldre viaduct (*)	A85	41	162m	50.5m	14.8m	1.9m		Launching	FW precast slabs		No cantilevers	Sauldre viaduct (*)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Maubeuge viaduct	RN49	59	304m	51m	21.2m	2.3m	1,500t	Launching	Mobile formwork	2000		Maubeuge viaduct
Charente bridge. Jarnac (South leg)	RN141	16	384m	36m	19.5m	1.6m	1,500t	Crane installation	Precast slabs	2003		Charente bridge. Jarnac (South leg)
OA3 and OA4 bridges	Avenue de France	75	510m	15m	38m	2m	5,000t	Crane installation		2002		OA3 and OA4 bridges

(*) Total length of this bridge is less 200 m, but listed because quoted in text in relation to some of its characteristics.

C – Single box girder composite bridges (Ltot > 100 m)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Doubling of Ante bridge	RN158	14	170m	78m	10.75m	1.80 - 3.30m	525t	Crane installation	Mobile formwork	2005		Doubling of Ante bridge
Bonneville viaduct	RN205 / RD19	74	115m	71m	15.3m	1.61 - 2.87m	410t	Launching		2004		Bonneville viaduct
Monistrol d'Allier viaduct	RD589	43	168m	70m	10m	2.3m	500t	Launching	Mobile formwork and steel troughs	1998		Monistrol d'Allier viaduct
Loire bridge at Rivas	RD101	42	159m	59.1m	10.8m	1.6m	400t	Launching	Mobile formwork and steel troughs	1999		Loire bridge at Rivas
OA3 bridge on Jenlain bypass	RN49	59	134m	57.2m	11m	1.55m	400t	Shifting	Mobile formwork and steel troughs	1999		OA3 bridge on Jenlain bypass
Vienne bridge at Nouâtre	RD108	37	195m	55m	12m	1.2 - 1.9m	550t	Launching	Mobile formwork and steel troughs	2005		Vienne bridge at Nouâtre
OA4 bridge at Embrun	RN94	05	255m	55m	12m	-	650t	Launching	Mobile formwork and steel troughs	2004		OA4 bridge at Embrun
SD bridge	Palays interchange	31	378m	51.4m	9.5m	1.4m	1,325t	Launching with part of slab	Mobile formwork	2005		SD bridge
Gardon bridge at Ners	RN106	30	189m	44m	21m	2.28m	800t	Launching	Mobile formwork	1995		Gardon bridge at Ners
Boulogne-sur-Mer viaduct	RN1	62	190m	40m	2 x 9.15m	1.3m	850t	Launching	FW precast slabs	2005		Boulogne-sur-Mer viaduct
Volèsvre viaduct	RN79	71	208m	52m	12.2m	1.75m	500t	Crane installation	Mobile formwork and steel troughs	1998		Volèsvre viaduct

D – Box girder composite bridges with directly supporting cross-beams (Ltot > 100 m)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Lille ring road viaducts	BPL	59	445m	96.7m	13.1m	2.75m		Launching	FW Precast slabs	1997		Lille ring road viaducts
Charles de Gaulle bridge	rue Van Gogh	75	208m	84m	34.9m	2.5m	3,000t	Launching	Steel troughs	1996		Charles de Gaulle bridge
Freney viaduct	A43	73	207m	79m	18.2m	2.7m	1,150t	Launching	Mobile formwork	1998		Freney viaduct
Pont des Chèvres viaduct	A43	73	354m	78m	13m	2.7m	1,600t	Launching	Mobile formwork	1998		Pont des Chèvres viaduct
Moselle bridge at Custines	RD40e	54	125m	62.7m	10.3 - 18.93m	1.8m	375t	Launching	Mobile formwork	1998	Closed box girder	Moselle bridge at Custines
Roche Bernard viaduct	RN165	56	376m	36m	20.8m	1.7m	1,550t	Launching	Mobile formwork	1995	Closed box girder	Roche Bernard viaduct

E – Box girder composite bridges with directly supporting cross-beams and propped cantilevers (Ltot > 100 m)

Bridge name	Road carried	French Dept.	Ltot	Max. span	Deck width	Girder depth	Steel tonnage	Steel frame installation	Slab construction	Steel frame year	Comments	Bridge name
Verrières viaduct	A75	12	720m	144m	23.5m	4.5m	6,250t	Launching	Composite		Closed box girder	Verrières viaduct
Rhône bridge at Valence	RN7/ RN86 link	26	526m	125m	22.1m	4m	3,800t	Launching	Composite	2001	Closed box girder	Rhône bridge at Valence
Frocourt viaduct	RN31	60	284m	60m	13.3m	2.5m	1,150t	Launching	Composite	2005	Closed box girder	Frocourt viaduct

This guide describes very precisely the design and the building methods of steel-concrete composite bridges.

Each of its seven chapters concerns one step of the elaboration of such a bridge: general design, detailed design, steel frame installation, concrete slab building, maintenance, preparation of the call for tenders

Anyone involved in the elaboration of a steel-concrete composite bridge (owners, engineers, checkers, architects) and teachers specialized in civil engineering will be very interested by this guide.

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