

Bulletin 82



Precast segmental bridges

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Precast segmental bridges

Guide to good practice
Task Group I.7

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Back cover: (From left to right) Cieza viaduct, Santander, Spain (see case study on page 167; photo courtesy of Grupo Puentes/Pondio Ingenieros); erection of a precast pier segment (Fig. 5-80; photo courtesy of Ferrovial-Agroman); Ramat viaduct, Torino–Bardonecchia highway, Italy (built 1990-1992; photo courtesy of Giuseppe Mancini)

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Foreword

The concept of precast segmental bridges is not new: the first application documented was from the mid-1940s, designed by Eugene Freyssinet and built over the river Marne near Luzancy in France, between 1944 and 1946. Although innovative, it also contained traditional wet concrete joints between the members. The impressive breakthrough came slightly later with the introduction of match-cast joints by Jean Muller, first for a bridge near Buffalo (USA) in 1952, and later for a bridge across the River Seine at Choisy le Roi near Paris in 1962. This opened the way for a large number of new developments in terms of design, production approaches and construction techniques, and precast prestressed concrete segmental construction became rapidly one of the most efficient and successful bridge construction methods all over the world. These developments are still evolving, but the interaction between design, production and construction is a critical factor for success: the interaction creates opportunities to optimise the scheme, but at the same time is crucial to ensure safety, especially during construction, when large weights are moved, placed and secured, frequently at substantial heights. Engineers of all disciplines involved should interact during the development and realisation of precast segmental bridge (PSB) schemes, to conclude the optimum method statement and consequently check all the intermediate steps of the method statement in terms of stress, stiffness, stability, production and constructability. With the ongoing development of the PSB concept, and consequently moving limits in terms of dimensions, it was concluded to be appropriate to develop a Guide to good practice for the PSB construction method. The present report was developed by an integrated team of engineers with roots in design, structural engineering, production and construction, and provides a valuable source of knowledge, experience, recommendations and examples, with particular emphasis on the *fib* Model Code for Concrete Structures 2010 and *fib* Bulletins 20, 33, 48 and 75.

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1 Scope

This report is devoted to beam bridges that are not fully or are partially cable supported.

The emphasis of this report is on the specific features of precast segmental bridges (PSB), particularly in comparison with cast-in-situ bridges.

The report gives an historical overview of the development of this type of bridge, describes the available techniques for production of the elements and construction of the bridge, addresses the strong interaction between design, production and construction, addresses concept specific aspects of design, production and construction, addresses maintenance, repair and demolition, and presents characteristic case studies.

The report concludes with a Bibliography and examples of projects.

2 Definitions

Anchorage block – Thickening of the web, flange or web-flange junction to provide an area for one or more tendon anchorages.

Balanced cantilever method (BCM) – An erection method where segments are placed alternatively at either side of the pier cantilever up to the point where a closure is made with the previous cantilever or side span structure.

Bar – Post-tensioning bars are high-strength, high-yield bars, used for temporary or permanent stressing.

Beam and winch – See *lifting frame*.

Box girder – Commonly used box shaped structural member used for bridge superstructures.

Box pier – Box shaped structural member used for bridge piers.

Bulkhead (also end mask) – The fixed-end formwork of the casting cell or mould. This has fixed openings for the tendon locations.

Cantilever tendons (also continuity prestress) – Longitudinal post-tensioning in the top slab of bridges, built with the balanced cantilever method.

Closure or closure pour – Cast-in-place concrete segment or segments used to complete a span.

Continuity tendons – Longitudinal post-tensioning installed in the bottom slab of balanced cantilever bridges to resist mid-span bending; external prestressing installed in a span, or between spans for balanced cantilever or span-by-span construction methods.

Coupler – A device used to transfer the prestressing force from one length of tendon to another. Strand couplers are not generally allowed.

Deviator or deviation saddle – A concrete block in the web, flange or web-flange junction to control tendon geometry or to change the direction of an external tendon.

Deviation trumpet or trumpet – Detail for tendon connection through deviators or diaphragms to provide tolerance for angle of tendon entry.

Diabolo – Detail of duct forming through deviators or for connection at diaphragms.

Duct – Material forming a conduit to accommodate prestressing steel installation and provide space for grout for internally bonded tendons.

External prestress or externally bonded tendons – Provision of prestressing tendons outside the flanges or webs of a member and generally within the internal void of a box girder. The tendon polymer duct is generally exposed and connected to the deck only at end anchorages and intermediate deviators.

Field segments – Those segments within the span of a bridge built by the span-by-span method.

Hammer – Portion of a balanced cantilever deck comprising the pier head and cantilever segments either side. At completion a hammer is connected to the adjacent hammer with a closure pour.

Hammerhead – Starting segment for a precast segmental balanced cantilever bridge.

HDPE – High-density poly-ethylene/PE – poly-ethylene UV resistant. Commonly used for external post-tension tendon ducts respectively post-tension internal ducts.

Internal prestress or internally bonded tendons – Provision of prestressing tendons placed within the flanges or webs of the superstructure and continuously bonded to the concrete with grout.

Erection gantry or erection truss – framework supported on previously built structure or piers, which carries deck segments for erection by balanced cantilever or span-by-span methods. The gantry takes delivery of segments, lifts, moves and places the segments.

Lifting frame – Erection equipment, often bespoke, consisting of a longitudinal frame or beam attached to the end of a cantilever to lift up segments using lifting winches, cables and pulleys attached to the frame. After erecting a segment, the frame is advanced for the next segment. Lifting frames are most often used with balanced cantilever or progressive cantilever erection methods.

Long line casting – A method of casting segments on a long casting bed which makes up the entire cantilever or span between site closures.

Match casting – Method of casting segments where a segment is cast against an existing segment to produce a matching joint.

Permanent post-tensioning – Post-tensioning that is required as part of the completed structure.

Pier Segment – Precast segments located at the piers, typically the heaviest segments with diaphragms (anchorage and/or deviation of external tendon).

Post-tensioning – A method of prestressing where tensioning of the tendons takes place after the concrete has been cast and reached a specified strength.

Post-tensioning scheme or layout – The pattern, size and locations of post-tensioning tendons.

Precast segments – Precast concrete elements, typically box shaped, which can be assembled to form a bridge superstructure or pier.

Precast segmental bridge – A bridge constructed with precast segments.

Progressive cantilever erection – Segments are erected in cantilever in one direction only using temporary intermediate piers or cable stays.

Segmental construction – The fabrication and erection of a structural element using individual elements. These may either be precast or cast in-situ.

Short-line casting – A method of casting segments using a special form with a fixed bulkhead at one side and the previously cast segment at the other.

Span-by-span method (SSM) – An erection method where all of the segments for a span are placed, aligned and longitudinally post-tensioned together in one operation to make a complete span.

Strand – An assembly of several high-strength steel wires wound together (usually 7 No.).

Temporary post-tensioning – Post-tensioning that is required only in construction and not as part of the completed structure.

Tendon – A single or group of prestressing steel elements (strands) and their anchorage assemblies giving prestress forces to a member. These components of the tendon also include ducts, grouting attachments, grout, and corrosion protection coatings or fillers.

Transverse tendons – Post-tensioning installed in the top slab and perpendicular to the spanning direction of the bridge. Typically used for wide decks or to stiffen deck cantilevers.

Wire – A single, small diameter, high-strength steel member and the basic component of strand.

3 General introduction

3.1 Description of precast segmental bridges

3.1.1 General description

This chapter presents a brief description of precast segmental bridges and related methods of construction.

In precast segmental bridge (PSB) construction, the structure is built by post-tensioning together precast elements (see Figure 3-1). The bridge is divided transversely into ‘small’ segments. This method is in contrast to precast beam bridges, where the length of the girders and the span length are identical. External as well as internal longitudinal tendons can be used. The glued or dry joints between adjacent precast elements are perpendicular to the longitudinal axis of the structure. Generally no continuous reinforcement is provided across the joints between the elements, except where it cannot be avoided for structural reasons. In-situ concrete sections instead of joints are needed if lapping of rebar between adjacent segments is required. This results in a great decrease in construction speed and the loss of one of the mayor advantages of PSB. The superstructure is composed of typically two to three different types of precast segments, depending on the post-tensioning system used. Single as well as multiple-span bridges have been built using PSB technology. Decks are most commonly box girders.

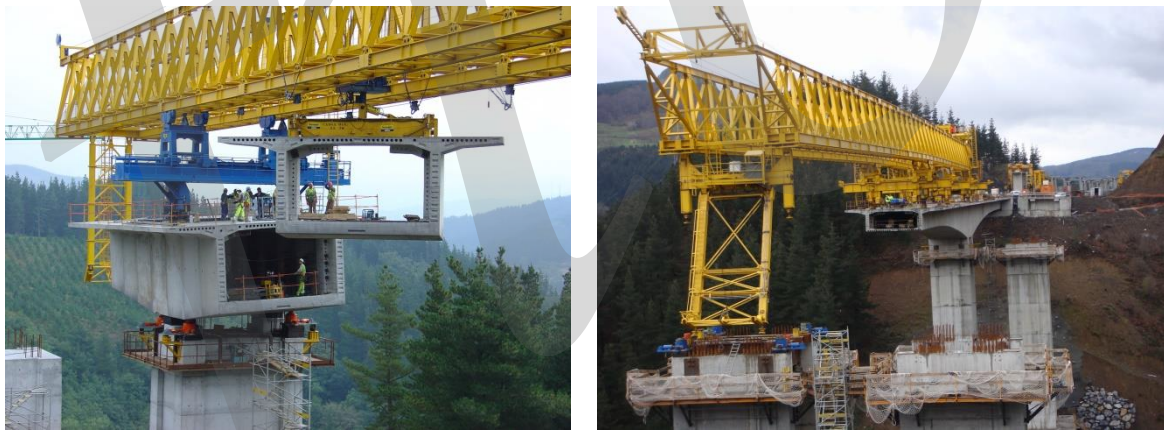


Figure 3-1: A typical precast segmental bridge under construction

Many design decisions depend on the method of construction, such as shape of segments, weight of precast elements and arrangement of the prestressing cables.

Longitudinally, three types of segments can be identified, regardless of the construction method: the pier segment, in span segments, and deviator segments if external prestressing is used.

As a general principle, the pier segment is used for anchorage or deviation of longitudinal external post-tensioning cables, if present, and to transfer the loads to the substructure. Therefore, a diaphragm is needed to stiffen the deck, which makes it the heaviest precast element. Large, concentrated forces act on this segment, which requires a careful design of the details to avoid cracking due to local tensile stresses in the concrete.

If external prestressing is used, the tendon's angle will change at the deviation points, and then ribs, corbels or diaphragms are used to transfer the resulting vertical forces into the webs. The detail for deviation of the tendons must provide a smooth change in angle, with a minimum radius given in the technical approval of the prestressing system. External cables or the polyethylene duct should not be damaged by a sharp corner within or at the end of the deviator.

The rest of the span segments are composed of standard segments without deviators or anchorage blisters, which are characterised by thin webs, in order to minimise weights.

3.1.2 Construction methods

Generally there are two principal ways to construct precast segmental bridges: the span-by-span construction method and the balanced cantilever method.

The span-by-span construction method (SSM) is similar to the span-by-span method for cast-in-place bridges, but with some particularities which are described below.

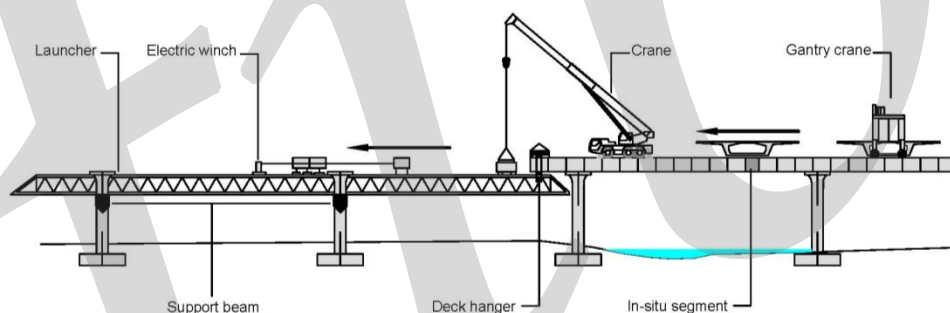


Figure 3-2: Sketch of the span-by-span construction method

The balanced cantilever method (BCM) is similar to the free cantilever construction method with travellers and cast-in-place segments.

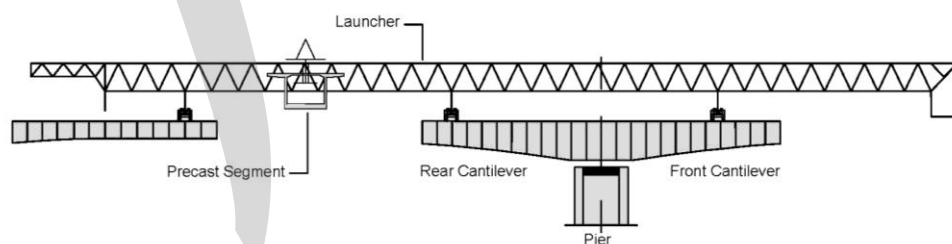


Figure 3-3: Sketch of the balanced cantilever construction method

Similarly to an in-situ balanced cantilever erection method, it is possible to erect the segments by using bespoke machines.



Figure 3-4: A variant of balanced cantilever erection by bespoke erection machine –integrated walkway for access around the segment for gluing (Quarashia Viaduct, KSA) [Srinivasan, 2004]

A third, less frequently used option is the progressive placement method. In this case, the precast segments are placed from one end of the structure to the other in successive cantilevers on the same side of a given pier, rather than by balanced cantilevers on each side of a pier. Sometimes the help of temporary stay cables is needed to reach the next pier.

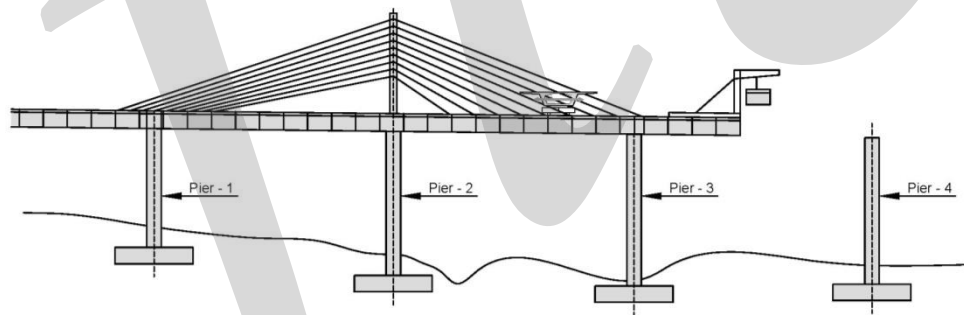


Figure 3-5: Sketch of the progressive placement construction method

In both the span-by-span method and the balanced cantilever method, the erection of the segments can be made from the ground or water below, or from the existing structure.

The benefit of crane erection is the low cost of equipment and use of existing roads. This method is particularly suitable where pier heights are low and there is ready site access

such as in an urban location, where the existing road network can be utilised for delivery of precast segments. A crane located next to the deck shadow places the segments adjacent to the previous installed deck units while prestress is installed (when necessary). This tends to be the most economical way of erection due to the lower cost of the equipment needed, and also because the construction loads do not govern the design. Variants to this method are the use of beam and winch or bespoke lifting frames to erect segments from the ground (see Figure 3-4).

The primary benefit of an erection method lifting segments from the ground, as opposed to using structure-mounted equipment, is that there are no erection loads on the structure so that the erection method does not govern the design of the deck and piers, which can be the case for other erection methods.

Often, access from ground level is not possible or the height of the bridge is such that the lifting of segments is not feasible. In such cases, for example when the bridge crosses a river or deep valley, the deck or piers need to support the erection equipment, such as underslung or overhead erection gantries (see Figure 3-5).



Figure 3-6: Underslung truss (Miño viaduct)



Figure 3-7: Overhead gantry (Alcalá viaduct)

3.1.2.1 Span-by-span method (SSM)

This construction method has two principal types of erection equipment: underslung trusses or overhead gantries. The difference between the two methods of erection is how the segments are transported to the location of placement. In the first case, the segments are hung below the gantry, which is positioned over the deck. In the second case, they are supported by a truss located under the theoretical deck position.

In addition to the points mentioned below, in section 4.4.1 Bridge alignment, headroom restrictions influence the selection of the underslung or overhead erection options. While underslung trusses generally do allow for the passage of traffic below, their potential projection below the permanent bridge soffit reduces the available headroom, which is not always permissible. Overhead gantries are advantageous in these situations. However, there are underslung gantries that support deck cantilevers and hence can operate within the depth of the section.

The launch of underslung trusses from one span to the next can be executed with the support of cranes. Where access is restricted, or where it offers economic or programme benefits, the trusses can be self-launching. This can lead to underslung gantries being quite simple and low-cost. Typically, overhead gantries are self-launching, which involves more complexity and hence, greater cost. Overhead gantries are typically produced by specialised suppliers and their cost leads to reuse over a number of projects. Underslung trusses are much simpler and so can be fabricated by less specialised suppliers and can be sufficiently inexpensive to be used on a single project economically.

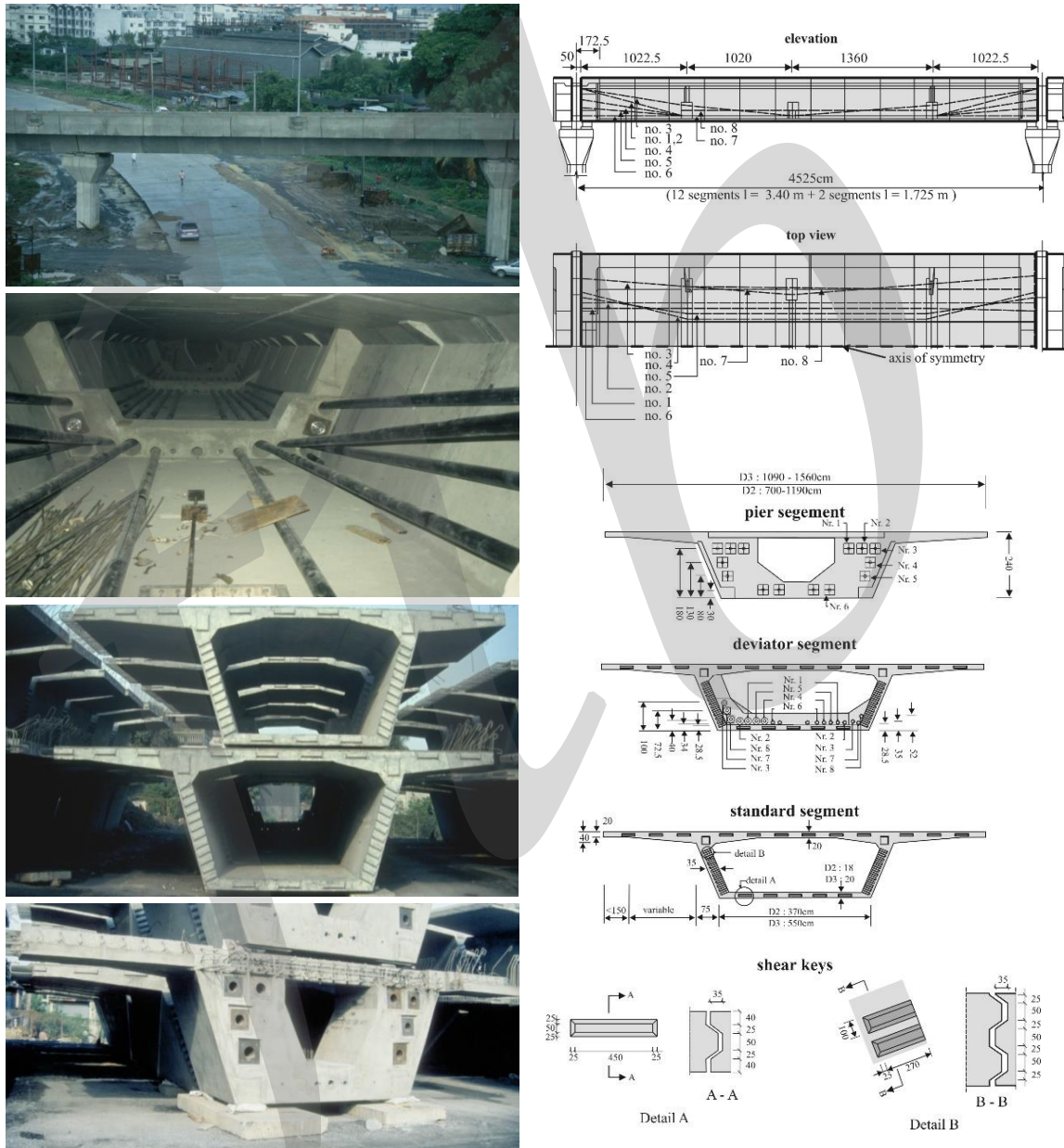


Figure 3-8: Typical span of a segmental bridge (SES, Bangkok)

Another possible method, generally only used in special circumstances, is to support the segments by means of conventional ground-supported falsework. This method is only feasible when there is a possibility of the ground having an adequate capacity and planarity. This is a more common approach in urban locations for relatively short roads, access ramps to viaducts and rail viaducts of 1-2 km length or shorter. This is especially the case for a family of short viaducts which share the economy of set up of the casting yard, but need low construction costs.

3.1.2.2 Balanced cantilever method (BCM)

For this method, there are three principal choices of erection equipment: ground- or barge-based cranes, deck-mounted lifting equipment, and overhead gantries. The cost of the erection equipment must be balanced against the construction speed. The selection of the appropriate equipment depends primarily on access conditions, number of spans to be constructed, programme requirements and characteristics of the structure.

Ground- or barge-mounted cranes might be the best choice for relatively small projects with good access from below and medium-height piers. The placement of segments using this method is usually more time consuming, but its flexibility and low capital expenditure make it an attractive option. Another advantage is that the design of the structure can be optimised independently of the erection method, as, unlike for other erection methods, no additional loads are acting on the structure during segment installation.

Another flexible method, though typically associated with increased equipment cost, is the installation of segments using deck-mounted lifting equipment, such as beam and winch methods or derrick cranes. This requires alternative methods for the installation of the pier segments as a base for the equipment is required. Accordingly, this will impose additional erection loads onto the bridge structure. Typically, these loads are relatively small; nevertheless, they should be considered in the design, as they may cause temporary overstress during construction.

Overhead gantries are usually the only feasible option where, due to access restrictions, the segment delivery cannot happen from below. They offer a very fast and efficient erection method, but require the segment installation to proceed in a linear fashion from one end of the bridge structure to the other. Overhead gantries are typically sophisticated units customised for a particular project, which results in a high level of capital expenditure. This is usually only advantageous where the aforementioned access restrictions apply, or for large scale projects (i.e. many repetitions) where speed of erection is very important. Depending on the type, and particularly the length, of overhead gantry used, significant loads will be imposed on the permanent structure during segment installation and launching.

The arrangement of tendons is similar to a bridge built by the free cantilever method in place, with some particular features:

- Temporary prestressing, usually by prestressed steel bars, is needed to squeeze the glue of the joint and to fix the adjacent elements together prior to the installation of the permanent prestress.
- The cantilever tendons are anchored in ribs or blocks, or in the free face of the segment. This facilitates the assembly and installation of cables.
- A rubber ring around the cables at the joint location is used to guarantee the tightness of the duct.

3.1.2.3 Progressive placement method

A third method, although less common, is based on a temporary stay progressive cantilever construction method. It has been used for over thirteen bridges, mainly in France. This method fully benefits from the precasting of a bridge deck on a large scale. The structure is built continuously, from one abutment to the other, by regularly placing precast segments one after the other, similar to span-by-span construction as described in 3.1.2.1. The temporary stability of the deck is ensured by staying. Lifting equipment situated at the end of the cantilever lifts the segments into position.

For example, this technology allowed the easy realisation of a three-lane box-girder bridge, with a span length of up to 60 m and a plane radius of curvature of down to 350 m. The weight of the segments was up to 50 tonnes. The segments are driven along the deck from the storage area to the end of the cantilever with a self-propelled multi-wheeled trailer.

The lifting equipment consists mostly of a self-moving purpose-made swivel crane of some 50-tonne capacity that is anchored on the last erected segment.

The temporary staying mast, which lets the segments through, consists of two vertical, transversely braced steel legs, on which regularly spaced hydraulic jacks allow the tensioning of the cable stays by moving back their anchor plates. The mast is transferred from one pier to another on a trailer drawn by the multi-wheeled trailer used for segment transportation.



Figure 3-9: Progressive placement method: staying mast and swivel crane

A typical span erection cycle is as follows:

As the final prestressing of a span is progressively applied, the construction of the following span starts with the erection of a few segments in free cantilever to make room for the staying mast on the pier. After post-tensioning the complete previous span, the staying system is released, moved forward to the next pier and the first two pairs of cable stays are tensioned. The next segment is transferred from the storage area, lifted into position by the swivel crane and bound fast to the deck with prestressed bars. Then the swivel crane moves onto this segment and the next two pairs of temporary cable stays are tensioned. Before placing the pier segment, the level of the deck can be adjusted, if necessary, with hydraulic jacks, while the permanent bearings are placed. Afterwards the whole cycle can be repeated.

The advantage of this method is the ability to deliver the segments via the previously erected continuous superstructure and to span waterways, railways and main roads, without disturbing the traffic. Moreover, bridges with double curvature and varying width are easily erected. As far as the organisation of the works is concerned, the management of precasting is simplified, and the placing of the segment is continuous (a whole 60 m span can be built every week) and safe, since everything takes place on the erected deck. Furthermore, prestressing is minimal, as in a cast-in-place bridge, and construction costs are reduced, thanks to light and easily reusable equipment.

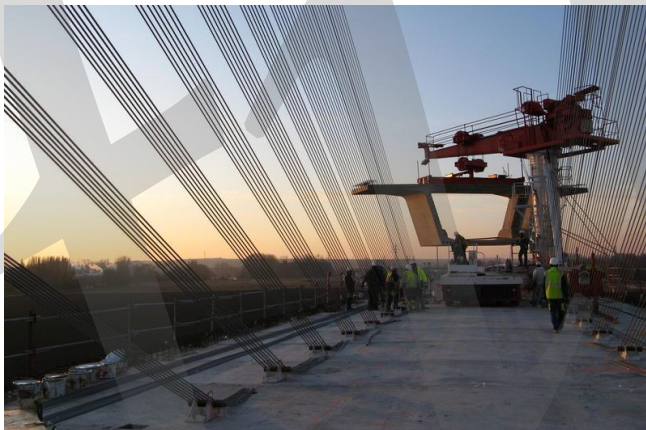


Figure 3-10: Progressive placement method: segment placing and staying

3.1.3 Advantages and disadvantages of PSB

The development of the concept for a precast segmental bridge (PSB) has a similar process as for other types of bridge structures. However, to benefit most from the advantages of the precast segmental bridge solution, it is of paramount importance to develop the design and construction concepts in such a way that it lends itself to the selected construction method.

3.1.3.1 Advantages of PSB

The main advantage of PSB is the speed of construction. The deck segments can be precast before the commencement of the pier construction, or both operations can be performed in parallel. This is especially important if problems arise during the land acquisition process or when the bridge is the critical path of the construction programme. The bridge deck can be mostly built without any contact with the ground. This makes it possible to build structures over rivers, which may be subject to severe flooding, or above very deep and rugged valleys, or in a congested urban location. As a result, PSB construction can have a significant advantage in the reduction of environmental disturbance.

In spite of being a precast solution, this method has remarkable flexibility to adjust its geometry to any sort of alignment. The method is extremely forgiving with regard to the geometry of the roadway supported by the bridge, because, in contrast to incremental launching and pushing techniques, many variations of horizontal and vertical alignments can be built without difficulty.

It is possible to design decks of a constant or variable depth. For the latter, parabolic, cubic or linear variations are all possible.

Precasting also gives advantages in quality control of:

- geometry,
- materials,
- workmanship,
- review.

3.1.3.2 Disadvantages of PSB

To use this technique, it is necessary to make a substantial investment in equipment, except in cases where a relatively simple erection method is being used, such as crane erection. Furthermore, specific training is required for everyone involved in the site work.

Precasting also involves multiple handling of the segments and storage. Speed of erection at the working site is dependent on a remote casting yard and also on transportation.

Very good design-construction integration is essential for this process. During the design stage, the designers involved must be very knowledgeable of the equipment required for construction and how said equipment interacts with the structure, i.e. induction of forces.

Another major disadvantage of this method is the large number of tasks, which have to be carried out away from the bridge footprint. Considering firstly the actual casting of the deck segments and secondly the development of the access routes for delivery of the segments from the fabrication location to the site, both of these are considerable undertakings that are not found in cast-in-place construction. Design is also early in the overall project programme, so that design changes are hard and hence it is difficult to respond to site changes.

3.2 Historical overview

The first segmental bridges, designed by Eugène Freyssinet, were built in 1944-1946 over the Marne River near Luzancy in France (Figure 3-11) [Lalande, 1946]. The structural system is a two-hinged portal frame bridge with multiple hollow box cross-section and adjustable hinged bearings. The joints between the segments were filled with mortar to equalise the tolerances between adjacent elements. The bridge is stressed by internal bonded tendons in transverse, longitudinal and vertical direction.

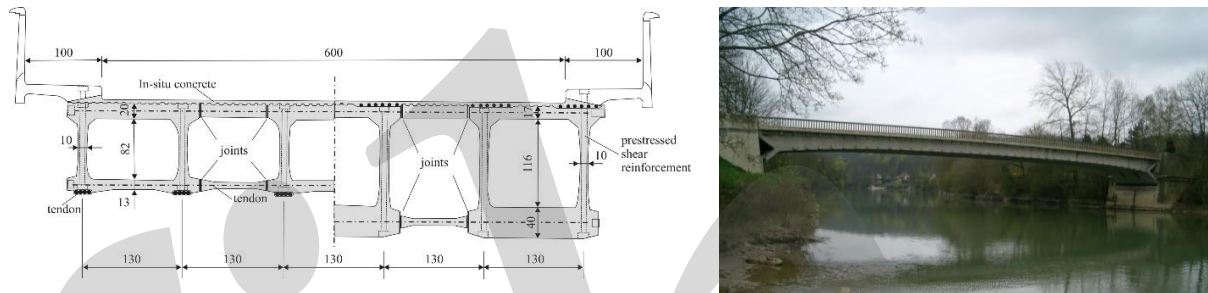


Figure 3-11: Frame bridges over the Marne River at Luzancy, France [Lalande, 1946]

The erection speed of segmental bridges with mortar or concrete joints is comparable to in-situ construction due to the time needed for the hardening of the infill material. Therefore segmental construction showed few advantages in relation to standard erection methods and was rarely used in the following years. The breakthrough of precast segmental bridges came with the use of thin glued joints or dry joints without any infill. This method required the development of special adhesive materials and a new production method for the segments. A perfect fitting of adjacent joint surfaces is needed due to the small width of the epoxy or dry joint, which can only be guaranteed by the match-cast method.

The long-line match-casting method was first used in 1952 for a small single span bridge in the New York State near Sheltonne, designed by Jean Muller [Muller 1975] who mostly invented the precast segmental bridge construction. The superstructure was divided longitudinally into three precast reinforced concrete beams which were cast end-to-end (match casting). The elements were assembled on site and stressed together with post-tensioning tendons. Dry joints were used in this project.

The bridge over the Seine River near Choisy-le-Roi, France (Figure 3-12) built in 1962 was the first mayor application of precast segmental bridge construction [Mathivat 1966]. The superstructure (span lengths $l = 37.5 - 55.0 - 37.5$ m, $l/h = 27.5$), consists of one or two standard hollow boxes joined by link slabs. Each element has three shear keys per segment surface. The precast elements were assembled by means of the free cantilever method. For the first time, epoxy glued joints were used. Segments were produced by the long-line match-casting method to achieve a perfect fitting of adjacent segments.

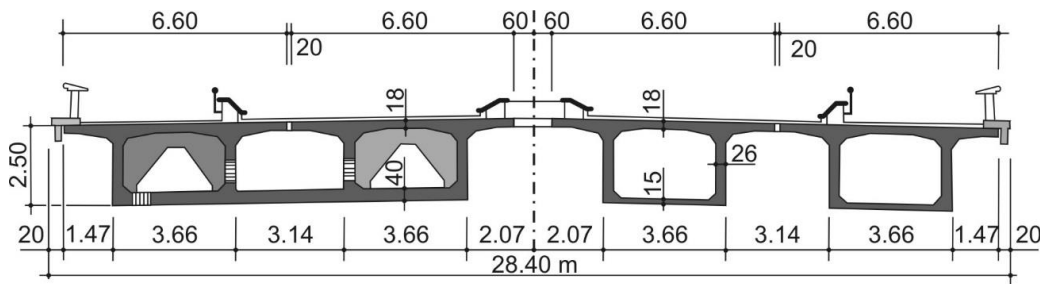


Figure 3-12: Cross-sections of the bridge over the Seine River at Choisy-le-Roi, France [Podolny 1982]

In the sixties, segmental bridges made of precast elements had become a standard construction method for many highway and railroad bridges all over the world. All mayor developments in the last decades are focused on special design aspects, the optimisation of segment production (short- and long-line match-casting method) and erection process (self-launching gantries).

The bridge over the river Rhône at Pierre-Bénite (1962-1965) was the first structure for which the precast segments were produced by the short-line match-casting method, which uses a stationary formwork [Muller 1966]. This system is very flexible with regard to variable bridge geometry and alignment and requires less space in the precasting yard, but needs detailed and reliable geometry control.

The construction speed is dominated by the number of joints per span. Therefore it becomes necessary to increase the length of the segments, which is limited by the permissible loads of the trucks and the lifting equipment. Large precast segments with a length of 12.5 m each and weighting up to 600 tonnes were used for the free cantilever construction of the 95 m spans of the Oosterschelde Bridge in the Netherlands. The 5 km long bridge was completed in 1965. Concrete joints with a width of 40 cm were used.

The time for transportation and assembling of the segments can be reduced by using a self-launching gantry; this was done for the Viaduc d'Oléron, built in 1966 [Mathivat 1976]. With a total length of 2862 m it was at that time the longest prestressed bridge in France. This project demonstrated the big potential of segmental construction. For the first time, an overhead gantry was used to assemble the segments. This project marks the start of industrialised prefabrication of segments. The hollow box cross-section had a width of 10.62 m and a height of 2.50 to 4.50 m (Figure 3-13). This gives a slenderness ratio of $l/h = 79/4.5 \approx 18$. The segments were produced on a long bench (long-line method). The balanced cantilever method and epoxy joints were used in this project. The precast segments were transported on the already finished superstructure and erected by a self-launching overhead gantry. Each segment had three shear keys per surface, two at the webs and one at the top slab to simplify the construction.

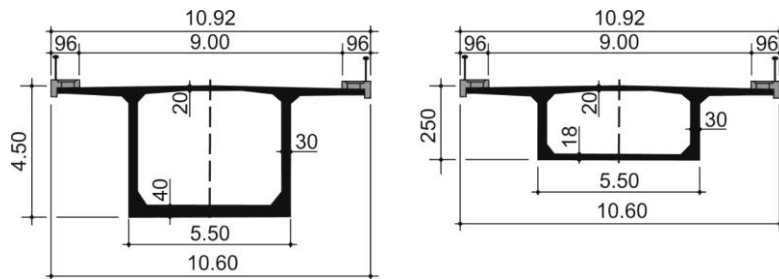


Figure 3-13: Cross-sections of Viaduc d'Oléron [Podolny 1982]

The same construction method was used for the parallel Chillon bridges in Switzerland (total length of over 2000 m, completed in 1969) [Piguet 1969]. Typical spans were 92 m long. The girder depth varied between 2.2 m and 5.0 m. Several shear keys were arranged in the webs, whereas one single key had been used in the projects before to transfer the shear forces.

Cantilever construction is uneconomical for medium span bridges (span length < 65 m). Here stationary falsework (e.g. Hammersmith flyover, London, 1961), temporary erection girders (e.g. Long Key bridge and the Seven Miles Bridge, both completed in 1980) and self-launching girders (e.g. First and Second Stage Expressway System in Bangkok, completed in 1996 and 2000) are used. All units of a span are assembled in place on an erection girder (span-by-span-method). Then the segments are stressed together by internal or external tendons.

A significant reduction of the time needed for erection and assembling of the segments can be achieved by using dry joints. This system has been successfully used for many major road projects in South East Asia (e.g. SES and BBB, Bangkok [Rombach 2003]).

Segmental construction is not only applicable for beam bridges but also for cable-stayed bridges. In 1978 the first cable-stayed bridge built in the United States utilising precast, prestressed segments was completed. The Ed Hendler Bridge (Pasco-Kennewick-Bridge) was built by the cantilever erection method.

One of the most remarkable segmental bridges is the Bang Na - Bang Pli - Bang Pokong Expressway in Thailand, the longest highway viaduct structure ever built (length appr. 54 km) [Rombach 2003, Borkenstein et al. 1999]. It was completed and opened to traffic in 2000. More than 21,000 segments were produced by the short-line match-casting method for this project. 18 segments were needed for a standard span (length $l = 42.6$ m between the supports, 44.4 m between centreline of H-shaped columns; Figure 3-14). The span units were assembled on a self-launching girder underneath the superstructure. Then the segments were stressed together by external tendons. Dry joints were used.

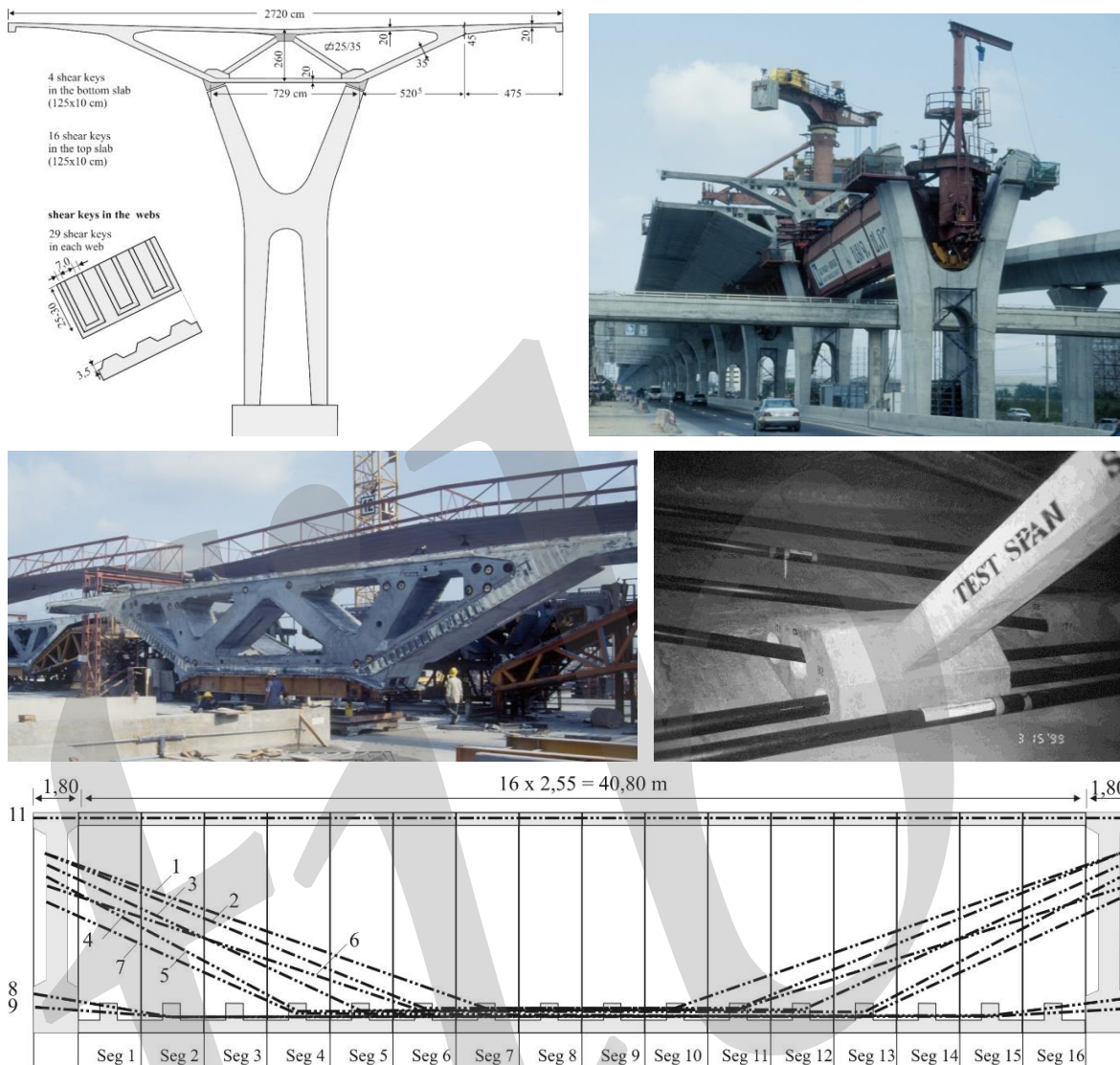


Figure 3-14: Bang Na – Bang Pli Bang Pakong Expressway [Rombach 2003]

Due to the width and slenderness of the D6 segments (6 lanes of traffic), an extensive full-scale testing programme (load-bearing behaviour, bowing effect, etc.) including in-depth theoretical investigations was performed prior to commencing the erection of the superstructure; refer to [Fischer 1998].

3.3 Field of application

Precast post-tensioned segmental construction is a flexible system with regard to variable bridge geometry and alignment. It is applicable for most bridges. Nevertheless, there are some limitations due to the cost for the precast yard and the erection equipment. The span length should range between 30 m and 50 m when the span-by-span erection method is used. Bigger spans can be built by means of the precast cantilever construction method

suitable for 65 m to 150 m span length (depending on the launcher capacity, use of lifting frames) or progressive placement, where the segments are fixed by temporary stay cables. The total length of the bridge should be more than 400 m in order to take advantage of the first investment that is needed. A tight plan radius ($R < 150$ m) could increase the cost for the erection truss. Erection equipment load limitations, permissible transportation loads and space and access to site may limit the weight and the size of the segments.

4 Conceptual design

4.1 General principles

The decision to use precast segmental methodology for a bridge is generally due to the large benefit to the contractor regarding the speed of construction and to the cost savings from the optimisation and extensive use of a strong industrialisation and mechanisation of the construction processes. The high quality achieved in an adequate plant producing the segments is also a considerable benefit given by this type of construction method.

But, all these advantages can be dramatically brought to nothing if the complete design, production and erection line are not perfectly interconnected and controlled by the precast specialist, the main contractor and the designer.

The description and recommendations relative to the prefabrication yard and the production line will be given in section 5.6. This part will emphasise some general rules showing the interaction points between design and construction and describe in general the major erection methods with their specificities.

The major goals of the design must be oriented towards safety, functionality, efficiency and elegance.

During the conceptual design, all the main aspects and elements of the bridge are defined. Definition of the structure will be given by:

- the longitudinal configuration, i.e. the distribution of supports and the corresponding distribution of spans; the structural type (beam, arch, cable-supported bridge, etc.); the main dimensions (structural depth of the different bridge elements);
- the definition of the cross-section of the different structural members;
- the construction material or materials; and
- the erection technique, including the definition of the major steps of the construction sequence.

All these basic elements of the conceptual design are deeply interlinked: the dimensions of the deck and its shapes depend on the construction material; the structural type is directly and strongly dependent on the material and the erection method. This is why the

preliminary design and the selection of the structural solution are based on the experience and intuition of the designer, who should know all the criteria and the typical configurations which lead, for example, to a specific choice of spans and structural systems. A good conceptual design is the result of knowledge, experience, intuition and mature judgment combined with a clear view of the flow of forces, a preliminary evaluation of the main loads, a global perception of all engineering aspects of the structure and a great feeling for aesthetic forms and proportions as well as for integration into the site.

4.2 Integration of design and construction

The construction and erection method and, through it, the construction equipment are major parts of the preliminary design, with a decisive impact on the project economy. However, they must not dominate the design and take such an importance that safety, functionality and elegance could be affected.

There is often no detailed analysis made to support the different solutions that are compared unless a very specific problem is foreseen; the selection is based on an engineering judgment supported by experience and competence.

Alternatives and modifications proposed by the contractor to adapt the design to a different erection technique, or to its existing construction equipment, must not change the bridge equilibrium defined above, the operation condition, its functionality, or its elegance. For example: when the bridge alignment has to be curved and different from a pure circle with a constant radius, it is not acceptable to design a box girder with its webs following a perfect circle to allow the construction of regular and constant precast elements. Slab overhangs of very variable widths on both sides would give a very poor visual appearance to the structure.

Generally, a completed bridge should not show obvious signs of its construction process. The bridge is designed for serviceability and optimised for its environment. The construction method is a tool to reach the final goal with respect to the initial spirit of the bridge or the “good practice bridge design rules”.

The development of design-and-build contracts gives a major responsibility in design to contractors, with obvious advantages regarding the selection of the construction method and the resulting economies. In fact, precast segmental bridge techniques help contractors to achieve their goal, but they must be applied when the alignment and the geometry of the bridge allow for such techniques without changing or affecting the general structural form and elegance of the original concept. As an example, the longitudinal profile and the bridge alignment must be selected considering the elegance of the bridge line in its environment and not only to reduce the cost of piers by lowering the deck as much as possible, or when the shape of piers has to be adapted to fix a gantry or to allow for an easy passage of a launching girder. This must be done with elegance, even if this leads to a slightly increased material cost. Similar types of considerations will lead to the optimum in design and geometry of the precast segments. In addition to these general interactions between design

and construction, some more specific points, where precast techniques could influence the design or vice versa, are set out in the following sections.

4.3 Erection method

As indicated in section 4.1, the erection method is heavily interlinked with the rest of the parameters involved at conceptual design. Consequently this is one of the first decisions to make at the beginning of the design process.

From the three main erection methods described in section 3.1.2, span-by-span and balanced cantilever are the most common systems used.

The following parameters are considered when comparing these two methods: suitable span, construction speed, stability during construction, flexibility of using equipment, load capacity requirements, segment delivery, temporary prestress and geometry accuracy and limitations.

4.3.1 Span-by-span construction method (SSM)

Suitable span length: For the SSM, the maximum span length is limited by the need to support the whole span on a gantry. Depending on deck width, a span of approximately 50 m is typical. This limit derives from the capacity of the gantry. In the last few years, the capacity of equipment has increased, thus allowing longer spans for given deck widths.

Construction speed: For regular span lengths, this method is recognised to be the fastest. It is assumed that an erection ratio of two spans per week is possible for simply supported schemes. Typical erection times for 35 m to 40 m spans are 3-4 days/span.

Stability aspects: With large elements working at great height and often supported by the slender piers of the deck structure, overhead gantries are potentially susceptible to instability, particularly when heavily loaded or under high winds. Care is needed to ensure a safe working methodology; if there are particularly windy conditions, construction activities may need to be temporarily halted, with delays to the construction programme and requests to temporarily restrain the launching girder on the deck or piers.

Flexibility in using equipment: Compared with balanced cantilever erection, there is a larger range of types of available erection equipment, such as overhead, underslung or falsework systems. This can be an advantage in that it allows the reuse of already available equipment or adaptation to varying site conditions.

Load capacity: As just before permanent stressing, the whole deck is supported by the erection system, it needs to have a larger carrying capacity than a balanced cantilever system.

Delivering segments: The delivery of each segment can take place above or below the deck. This issue needs to be taken into account during the equipment selection process.

Temporary prestress: Temporary prestress quantities are lower than for balanced cantilever erection. In some cases, this temporary prestress can be a part of the permanent prestress. In other cases, where glued joints are used, extra temporary prestress is necessary.

Geometrical accuracy: During erection, the geometry control is more precise than with the BCM, as the whole span is held, then adjusted before stressing. There is also less of the effect of creep on the geometry, as built-in stresses can be reduced. The SSM offers the advantage that the errors are not cumulative between spans, which is not always the case in BCM.

Geometrical limitations: For smaller plan radii, the erection difficulty increases. However, the SSM is less sensitive to this condition; again because the span length is the primary factor. Special articulated launchers have been designed to provide a better adaptation to curves.



Figure 4-1: Temporary stressing arrangement of segments in a span-by-span method – here, blue temporary anchorages are attached through the top flange of the deck. Temporary stressing bars are threaded through pairs of temporary anchorages. The bars are reused for the subsequent spans. (Sirsi Circle Viaduct, India) [Srinivasan, 2004]

4.3.2 Balanced cantilever construction method (BCM)

Suitable span length: As the erection machine does not have to support the segments of a whole span, greater span lengths are achievable with this method than with the SSM. The governing factor of the BCM is usually the self-launching capacity of the traveller. Typical economical span lengths for this erection method are 50 m to 120 m, although longer spans have been completed up to 160 m and planned for up to 200 m. As well as gantries, there are others methods for the BCM.

Construction speed: This erection method is slower than the SSM. While the launching and assembling of segments may be rapid, time-consuming operations are needed at the end of each span (closing segment, continuity prestress, geometrical adjustment). This is a largely fixed time, not directly related to the span length. For this reason, this method is more suitable for larger spans.

Stability aspects: The stability control becomes decisive in each stage of the design and erection. High overturning loads during construction can be a controlling factor in the foundation design (overturning moment). The overturning moment needs to be absorbed by the pier and can sometimes govern its design, especially for the upper portion of the pier and pier head. These loads are determined by the span, the geometry of the bridge and the loads coming from the erection equipment (gantry, beam and winch, etc.). As launchers are more slender than those for the SSM and spans are larger, launcher stability, particularly during launching, requires consideration. Moreover, segments on an SSM gantry can be arranged to give a reasonably even load on the gantry. In the BCM the geometry and sequence of operations can lead to out-of-balance forces on the gantry. There may also be changes in geometry, such as false cantilevers, giving different temporary stability conditions during erection compared with the permanent condition.

Flexibility using equipment: the BCM frequently uses erection gantries resting on piers or the previously erected structure. Lifting frames, cranes and hybrid bespoke systems have also been used more often in a situation where segments can be delivered from below.

Delivering segments: The delivery of each segment can take place above the deck, usually by means of a launcher, or below, usually by means of a lifting frame.

Temporary prestress: As glued joints are frequently used, temporary prestress is usually placed with bars. Sometimes permanent prestress is governed by the temporary self-launching condition of the gantry, governing both local loads and deflections. The increasing capacity of the launcher for longer spans makes them heavier. This could result in a controlling situation during the advancing of the launcher, sometimes also known as jumping.

Geometrical performance: During the segment erection, the longer the span length, the lower the geometrical accuracy is likely to be. For this reason, the common practice is to build the balanced cantilever over jacks that allow for three-dimensional geometrical corrections at the completion of each balanced portion. This issue directly correlates with the precast works, and the precision thereof.

Geometrical limitations: The allowable plan radius is related to the span length due to the geometrical and structural implications. Longitudinal and transversal slopes are also limited. The limitations are often practical, as the ability to correct construction errors becomes less viable for small radii or large slopes.



Figure 4-2: 131-m span balanced cantilever erection (Al-Khaleej viaduct, Riyadh, KSA)

4.3.3 Loads during construction

Depending on the system considered and as a consequence of the evolution of the capacity of launchers for longer spans and more complex geometries, the weight and influence of gantries has increased significantly. For this reason, the following considerations must be taken into account:

- the final prestress can be governed by temporary conditions, particularly the self-launching situation in balanced cantilever construction;
- a bigger elastic deformation than expected in a structural scheme, resulting in non-negligible permanent deformations; and
- residual nonlinear deformation due to temporary situations with concrete working slightly out of elastic range. For example, in the case of launching with a heavy machine and more than 40 m of cantilever, a small plastic rotation can be produced around the pier that, when multiplied by the big lever, can produce a significant deflection.

During construction the safety factor for wind or seismic actions must be considered to have a lower value that takes into account the lower probability due to the short period of construction. Some of these actions can be limited if they are produced in controlled operations, for example, if a temporary situation due to a specific operation can only be done when the wind is lower than a fixed value.

However, the designer and contractor should also be aware of robustness and consequential effects of a failure during construction, which can be more onerous than in the permanent case.

Loads under construction and particularly seismic action can produce large loads in a discrete number of elements, but also it is advisable to pay special attention to the connection between deck and pier, and substructure loads.

4.4 Bridge geometry

4.4.1 Bridge alignment

The deck alignment is usually driven by many different constraints, from environmental and geographical to construction limitations such as:

- the shape and the environmental characteristics of the valley or crossing;
- whether the access from the ground is possible or not;
- the available construction equipment; and
- the particular constraints of the road or railway that the bridge carries.

The geometrical profile of the road or railway and other geometrical characteristics are usually fixed before the design of the bridge begins, or when a PSB is considered as an alternative to another method of construction. Commonly, this situation requires finding solutions that fulfil specified geometrical requirements.

When the bridge position is completely defined by the road geometry, the bridge design and construction must adapt their shape to the road. Precast segmental bridges are very suitable to this philosophy of bridge design and construction since they are able to accommodate their geometry to the defined horizontal and vertical alignments.

Some issues must be taken into account:

- **Small radius alignment:** The achievable span lengths are limited by the values of the horizontal radius of the road. As the span length of a bridge increases, so does the length of the launcher. Since the equipment is generally straight, plan curvature must be low enough so that the piers can receive the equipment support while remaining within the permissible eccentricity limits. Transverse eccentricities of the loads supported by the launcher can be increased when the radius in the plan decreases, giving a reduction in the gantry load capacity and a reduced stability, although this can be mitigated by the use of temporary intermediate piers.
- **Large transverse or longitudinal slopes:** Longitudinal gradients larger than 8% can limit the capacity of the launcher supports. In some cases, extensions need to be designed to increase the height or level the gantry. In addition, large slopes give obvious additional problems for the movement of self-propelled systems, with additional force required for uphill movement and controlled movements on descents.

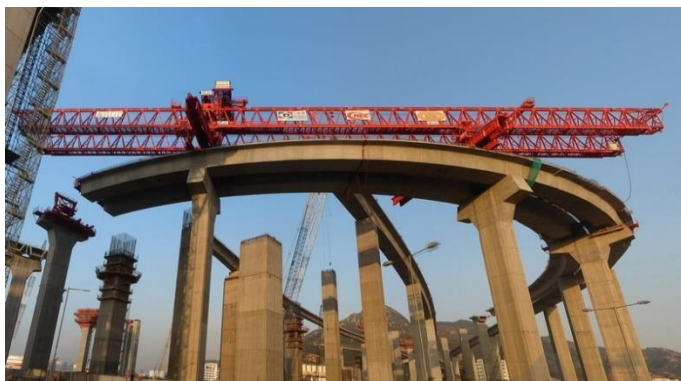


Figure 4-3: Launcher in small radius with balanced cantilever method, using transverse beams to adjust location and eccentricity

4.4.1.1 Span-by-span method

Underslung trusses are usually the most economical erection machine for the span-by-span method. However, due to their geometric constraints, they have relatively tight limitations in relation to the minimum possible radius. Overhead gantries, in particular articulated gantries, are more flexible but generally more expensive.

A system to allow for plan curvature for a span and for transverse adjustments is needed. This system must be able to move each segment independently from the position of the gantry on the chord line between the piers to the required location. The gantry needs to have the transverse movement capability as well. The range of both movements limits the minimum radii of the deck. The additional eccentric temporary loads also need to be taken by the substructure.

4.4.1.2 Balanced cantilever method

While a bridge with a straight horizontal alignment or a large radius is preferred for the balanced cantilever method, small radii geometries can be constructed as well, using the high torsional stiffness of the hollow box-girder section. Whereas deck-mounted lifting equipment and ground-based cranes can install segments for virtually every alignment, the use of overhead gantries does impose some restrictions on deck plan curvature.

The maximum achievable radius depends on the span, but also on the movements the launcher has to do and on the distance between legs. A tight curvature means transversal displacements to carry the segment in the back side of the launcher and put it in its position in the deck. These transversal displacements also have maximum allowable values.

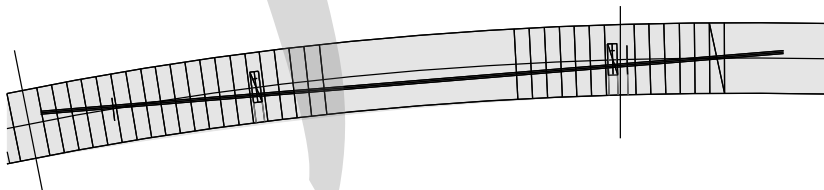


Figure 4-4: Launcher in small radius with balanced cantilever method



Figure 4-5: Launcher leg in transversal slope with balanced cantilever method

Table 4-1 gives an indicative illustration of the order of magnitude of the relationship between span length and radius for erection using a gantry. There are other factors, but this is good guidance at the early stages of design. If the project requirements require smaller radii, then consideration should be made to mitigation measures which could add complexity to a project such as articulated gantries or temporary towers.

Table 4-1: Span length and minimum suitable radius for erection using a gantry

Span length (m)	Minimum suitable radius (m)
65	400
85	500
100	700
120	780

When the radius is small, the structural capacity of the launcher can limit the operation. Eccentricities are produced during the assembling of segments and this may overload one of the trusses of the launcher.

4.4.2 Maximum span length

In general, for span lengths below 25 m, construction methods other than precast segmental bridges are more attractive. Unless special conditions apply, (refer to section 4.4.1 Bridge alignment) the span-by-span method is typically more economical than the balanced cantilever method for span lengths between 25 m and 50 m. For spans over 50 m in length, the balanced cantilever method is usually more suitable.

4.4.2.1 Span-by-span method

The erection truss, underslung or overhead, usually works as a simply supported beam structure. For such a system, the economic limit, as stated above, can typically produce an approximate span length of 50 m.

4.4.2.2 Balanced cantilever method

The concreting of the closing segment and the installation of continuity prestress tendons are time-consuming operations that decrease the construction efficiency ratio. However, the longer the span length, the fewer closing operations there will be. In practice, the minimum span length is approximately 50 m. However, a slightly longer span length of about 65 m allows a good balance between the constraints. The balanced cantilever method has been used for bridges with a span length of up to 160 m, although spans of up to 200 m have been planned. As the segments with deep sections at or near the piers are launched, these long spans have segments that become very large and heavy, and the quoted maximum span length can usually only be achieved if these segments can be lifted from below using lifting frames instead of gantries.

If a launcher is used, approximately 130 m is generally the longest economically achievable length. Especially in this case, the weight of the launcher needs to be limited. The higher the capacity of a launcher, the heavier it is. With long spans, the self-launching operation must be done from the end of the cantilever. Sometimes this situation governs the hogging prestress.

4.4.3 Span arrangement

4.4.3.1 Span length distribution

Both structural and construction considerations need to be taken into account when span distribution is being determined.

Span-by-span method (SSM)

A regular sequence of spans is important to achieve an efficient system. Significant differences in span lengths might require a more complex erection truss.

Balanced cantilever method (BCM)

The BCM can be adapted to different span lengths, as long as the cantilevers can be made reasonably symmetrical around the piers. On the other hand, due to what is usually a variable depth of a longer span deck combined with variable span lengths, a higher number segment types may be required. This needs to be considered when developing the formwork concept as it can increase construction complexity considerably.

From a construction point of view, a sequence of identical cantilevers is the most desirable solution. This simplifies the design of the erection method and the concrete casting equipment used, and the result is a simple and economically efficient structure. However,

gradients, physical obstacles, or environmental requirements may impose span lengths that do not allow for the ideal arrangement. It should be noted that, if the valley is very deep, then fewer tall piers are preferred, which can then lead to longer span lengths.

As a continuous structure, the general arrangement of a PSB using the BCM should consider the cantilever sequences rather than just the span lengths. Each span is built up from two consecutive standardised cantilevers. For instance, if a bridge has standardised cantilevers of 85 m, 100 m and 120 m, it is possible to arrange the spans as follows:

$$\text{First span } l_1 = \frac{85}{2} + l_{1,add} = 60.125 \quad l_{1,add} = 0.65 \cdot l_2 - \frac{85}{2} = 17.625$$

$$\text{Second span } l_2 = \frac{85}{2} + \frac{100}{2} = 92.5$$

$$\text{Third span } l_3 = \frac{100}{2} + \frac{120}{2} = 110$$

Note: The fourth, fifth and sixth spans are symmetrical.

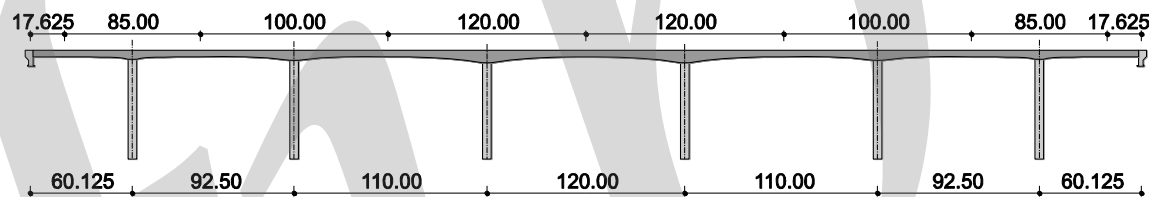


Figure 4-6: Span arrangement in balanced cantilever method

The end-spans are usually longer than a half-cantilever, with an optimal relationship between end and main span being 0.6 to 0.7. This ratio is lower than the conventional 0.8 taken for continuous beams because the construction process is in cantilever, so just after the completion of the span, there is no sagging moment in the end-span. For longer end-spans, additional bottom prestress may be required, which is an additional construction activity. However, this can be used to build a moment into the adjacent pier and deck to balance other construction loads.

The remaining length from the half-cantilever is usually built on falsework, as this is generally the most economical method. If the ratio is lower, smaller support reactions on the abutments will appear. If it is higher, the distribution of bending moments becomes unbalanced.

A maximum length of about 10 m can be built as an extra end-span without the need for temporary piers by cantilever erection from the abutment.

Another issue to take into account is the suitable relationship between the height of piers and span length. A large number of piers can be economically unacceptable if the ground

below has low levels of bearing capacity, as the foundations would be too expensive. Otherwise, as bending resistance is more expensive than axial resistance, more piers and smaller spans can produce economic advantages. Fewer piers, on the other hand, can give better aesthetic results.

4.4.3.2 Types of segments within a span

All precast segmental bridges have four different types of segments along the cantilever or the span:

- The standard segment.
- The pier segment. Precast segments are placed over the piers.
 - During construction, these elements can be placed over hydraulic jacks to allow the cantilever to be moved (reoriented). Steel plates have to be present on the bottom face to allow temporary support over orientation jacks. Likewise, holes to host temporary or permanent prestress bars or cables are made in the bottom plate.
 - For the permanent situation, pier elements may be supported by elastomeric bearings, Teflon® bearings or by rigid connection with the pier. In this case, the diaphragm and one pier face can be connected by a group of prestressed bars or permanent post-tensioning.
- The abutment segment. This is placed over one abutment and is supported by elastomeric or Teflon® bearings. A diaphragm has to be cast in place before or after the delivery of the segment. Like pier segments, these should have special elements, such as steel plates and holes for similar purposes as stated above.
- The closing segment (or stitch). This is the only segment cast in-situ and joins two cantilevers or spans. Some permanent prestress must cross this segment.

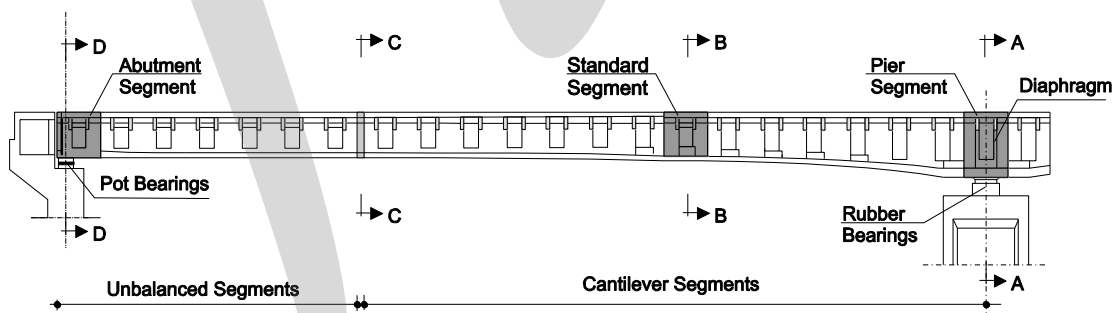


Figure 4-7: Segment arrangement

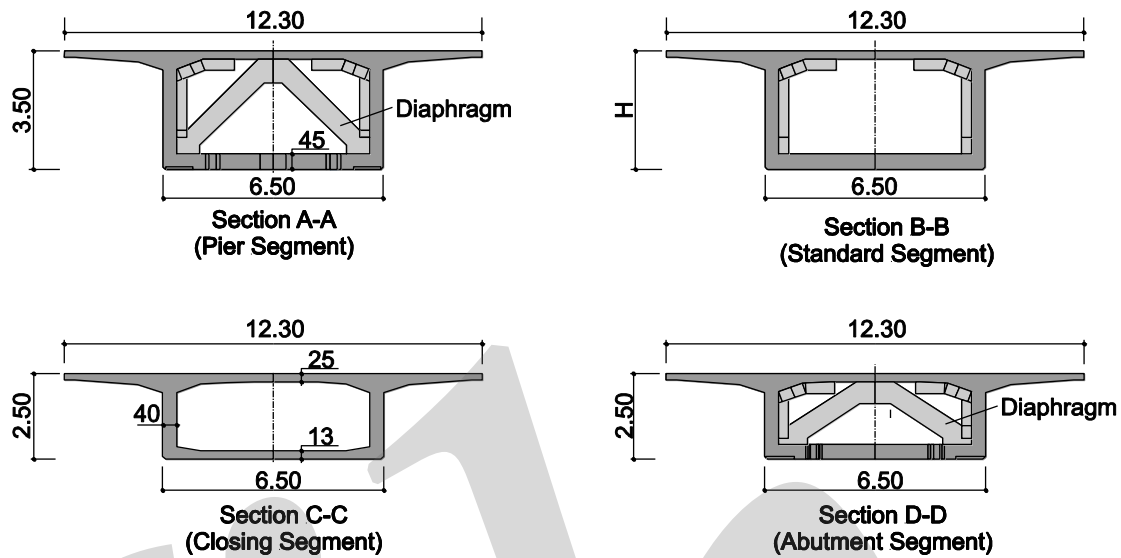


Figure 4-8: Segment types

Special remarks on segment types:

Composite segments

Special cases of segments with webs in steel have been built. Trusses and plates are also possible as web elements. In this case, external prestress is required. Solutions with webs of corrugated plates have been built to avoid the problem of the transmission of part of the prestress forces to steel because the creep of the concrete.



Figure 4-9: Segment with webs in steel

Precast pier segments

For a precast pier segment there are two options for the location of the joint.

- Single pier segment with joints either side of the pier. To allow for adjustments in a BCM, the segment is often temporarily supported by jacks whilst the spans are being erected. In an SSM, a single pier segment is common for a continuous deck

with pier deck continuity. In this case tolerance can be adjusted in the span. If a single line of bearings are needed, temporary jacks may be needed to keep stability during erection.

- Two pier segments with a joint coincident with the pier centreline. This is a common approach for the SSM when spans are simply supported with pairs of bearings. In a BCM case, having two segments is more complex. Firstly, one of the two pier segments is placed over four jacks. Next, the other pier segment is placed over two jacks and then joined to the initial pier segment with temporary prestress. The last step is to remove the two central jacks, so the two-segment-pier element is now supported by four jacks, and the balanced cantilever process can begin.



Figure 4-10: Pier head with balanced cantilever method

In-situ pier segments

In-situ segments over piers have been built on some occasions. In this case, it is very important to consider the position of both of the neighbouring precast segments. The orientation of both segments determines the position of the end of each cantilever. A slight error in the angle can produce a large deviation in plan or elevation at the end of the cantilever end, particularly for long spans.

The effect of creep and shrinkage on the deformations is less easy to establish and mitigate for an in-situ structure than for a precast one, as a large amount of the early age effects have been completed by the time a precast segment is erected. This is complicated if there is a mixture of precast and in-situ works. Precast elements also need to have this early age behaviour modelled and accounted for to enable the final geometry to be achieved. Moreover, precast elements have restrictions on lift-weight and handling. The options have different construction risks and the chosen variant needs to be considered as part of the overall construction methodology.

An intermediate approach used particularly when deck-pier continuity is required, or for high accuracy, is to adopt a precast shell segment with an in-situ filling. This gives light weight and geometrical control.

4.4.4 Segment geometry

4.4.4.1 Construction considerations

The shape of the segment shall take into consideration both design and construction method requirements. Some minor changes in proportion or shapes can be proposed by the contractor to fit in an existing production line, which can dramatically reduce the costs. But these changes should not affect the quality of the structure, its safety or its durability. As an example, a straight cantilever box-girder bridge is characterised by its variable height. To lighten the structure and to retain an elegant profile, in spite of a very high section on the pier, many designers would propose to have a cross-section with inclined webs. Precast contractors would usually prefer to have a vertical webs box girder, in order to ease the production of the elements by reducing the number of variable sizes from one element to the other, to the vertical dimension only. The cost of increased concrete used would certainly be largely balanced by the speed of form changes after each casting. But the global visual and static stability of the bridge would be changed. Owners and designers should be aware of these changes and those proposals should take place from the very beginning of the process, as they influence the aesthetics of the bridge and the design.

The challenge of precasting segments is to produce good quality and well-designed elements in the minimum time cycle and at minimal costs. In that context, some basic rules can be proposed which can be considered as a good starting point for the design of efficient and durable precast bridge segments.

In general, and depending on the span length and load type, the equivalent solid concrete thickness of a segment is of the order of 600 mm for a span-by-span erected bridge and between 450 mm to 650 mm for a balanced cantilever bridge segment.

The length of the precast segments is governed by their weight and by the possibilities of transportation and lifting to be used. Lengths are usually in the range of 1.8 m to 4.0 m, depending upon their width and depth. With a cantilever erection method, it is important that the segments not be too long, in order to limit the bending moments during construction and to avoid having to anchor a large amount of prestress at one location.

Because of its greater size and weight, the segment over the pier may have to be divided and delivered to the site in sections and subsequently prestressed.

Straight bridges with a constant box section are of course the most common structures to be built with precast elements. A single external form can be used for all elements, with variable thicknesses (webs and bottom slab provided on the inside part of the box girder) as required. The transition between thicknesses can then be introduced step-wise between two elements or along the element length.

When variable dimensions are necessary along the bridge (most frequently the depth has to be varied), it is preferable to be more efficient and to avoid this by changing only one dimension at a time along the element. As mentioned before, in the case of variable bridge height, precast subcontractors will prefer to have vertical webs, in order to change only the bottom part of the mould from one element to the other. If the webs are inclined, the variation in depth will implicitly result in more than one change in form at a time.

Dimensions of the element parts that are subjected, after demoulding, to being directly supported on their own as cantilevers, like lateral wings, should be limited in length so as to avoid deformation due to early creep of young concrete. This has an impact on the proportions between the box and upper slab width and will therefore also influence the width of the bottom slab. The best compromise should be found between lateral and central width to minimise risks. A cantilever length of less than 2.50 m with a minimum thickness of 240 mm at the junction web/cantilever will usually be sufficient, although it may be necessary to put some temporary supports under the wings during storage. Beyond those dimensions, increased length/thickness ratio or additional transversal prestressing should be used. Other solutions such as ribs or props are possible but increase the precast time and the difficulties. In any case, it is preferable not to cast parapets or edge girders simultaneously with the element, to limit the extra eccentric weight and limit the early creep deformations.

Sufficient minimum thickness of the concrete should be maintained to allow for the required cover of reinforcement and post-tensioning ducts, consistent with the service environment and to be able to correctly shape the reinforcement bars. A global ratio of 180 -200 kg/m³ reinforcement remains a usual value. The concreting method will also influence the thickness and the connection geometry (web/slabs) of the elements, as the compaction of concrete by vibrators needs to be done inside the reinforcement cage. By the use of self-compacting concrete, minimal thicknesses should allow for non-congested reinforcement details and thereby ease the progression of the concrete mix without segregation.

The choice of internal or external prestressing has also a large impact on the web thickness. For internal tendons, enough space should be allowed around the prestressing ducts to allow for concreting and vibration. With external tendons, deviators should preferably be placed regularly, at a constant location within the element, in order to standardise and optimise the construction of the internal part of the mould. Here the usual prestressing ratio is 40 - 60 kg/m³.

The economic success of prefabrication requires an efficient moulding system, an adapted and well-controlled concrete mix, a performance-based curing system allowing for speedy production cycles and geometry changes. But economy is also achieved by minimising repairs and clean-up after demoulding. For that, not only a well-engineered mould system is needed, but the shapes, angles and dimension variations of the element must be well designed for easy reinforcement construction, trouble-free concreting and especially for smooth and easy mould removal. These are the major factors for production speed combined with good quality results: no sharp angles, preferably a chamfer and open angles,

reduce rapid direction changes by limiting the number of mechanical inserts and any cast-in fixings, try to eliminate openings going through the formwork.

4.4.4.2 Cross-section

There is a range of possible deck types for precast segmental bridges. The choice is dependent on a variety of issues, but is primarily span- and deck-width-dependent.

Whilst these choices are similar for all concrete bridges, in the case of precast segmental bridges, temporary stability and low weight are of particular importance. Hence, box sections are most common, but for shorter spans of up to approximately 40 m there are alternatives such as hollow slabs, or 'double tee'.

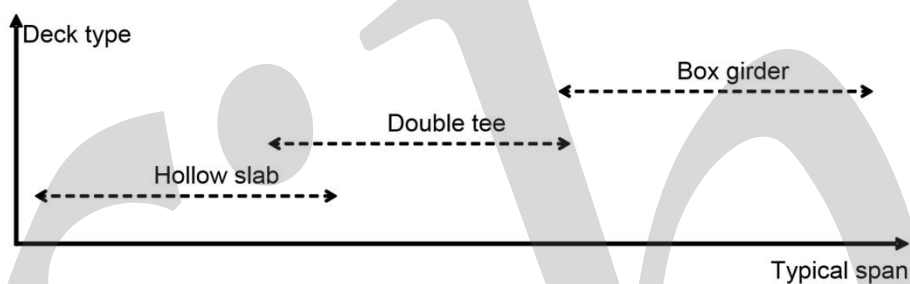


Figure 4-11: Relationship of typical span with deck types

Tentatively, hollow slab could be an appropriate cross-section type for shorter spans, up to 18-22 m while double-tee cross-section could be an alternative to box girder for spans between 20 and 30 m approximately, as shown in Figure 4-11.

A segmental erection method can have transverse out-of-balance forces during erection. The torsional stiffness of a box section has benefits in this regard to give some tolerance in the construction.

There are a number of reasons why box sections are particularly common in precast segmental construction:

- the torsional stiffness of a box girder allows for temporary out-of-balance forces during erection;
- torsional stiffness is beneficial for mitigation of lateral torsion deformation and buckling effects for balanced cantilever erection;
- segmental bridges are often used on curved alignments where a box section is needed for permanent loads;
- launching gantries can impose significant hogging loads at the piers during erection; a box section with a large bottom slab allows the neutral axis to be lowered to make the prestressing efficient. This has additional importance for segmental erection, as inefficient structural design of the pier section leads to a larger, heavier

section which then leads to a larger lifting weight, which may govern the erection methodology;

- use of external prestressing can allow thin webs, which reduce the weight of a section;
- a reduction of segment sizes to limit lifting weight is particularly important for the pier segments in segmental construction. A box girder gives a lower neutral axis than an open section, which allows an efficient use of prestressing and high moment capacity in hogging at the pier. This hogging capacity can be needed to allow for additional hogging moment effects from a launching girder.



Figure 4-12: Box section



Figure 4-13: Box with struts



Figure 4-14: Box with ribs



Figure 4-15: Hollowed slab with transversal cantilevers



Figure 4-16: Two-stage construction — multi-cell spine box and precast cantilever (Quarasiah Viaduct, KSA) [Srinivasan, 2004]

Examples of typical box-girder decks are shown in Figure 4-112 to 4-14. These include typical single-cell sections and variants with ribs and struts.

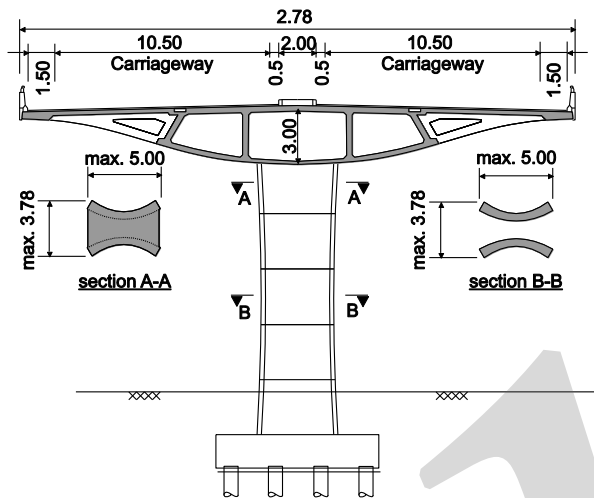


Figure 4-17: Multi-cell spine box and precast cantilever (Sungai Prai Bridge, Malaysia) [Srinivasan, 2009]



Figure 4-18: Parallel boxes with longitudinal closure pour (Deep Bay Link, HK)

Single cells vs. multi cells

Typically the flexural and shear capacities of the top slabs and webs define the maximum achievable width for a single-cell segment, which is directly linked to the weight of the segment. The structural capacity of the top slab can be increased by transverse post-tensioning or by transverse ribs. Where the roadway width is greater than the achievable maximum for single-cell cross-sections, alternative solutions may be:

Multi-cell boxes

- single or multi-cell boxes with transverse ribs;
- single or multi-cell boxes with struts; or
- parallel single-cell boxes connected by longitudinal closure pours.

The choice of single- or multi-cell box section decks is similar to that for other forms of concrete bridge. Factors to consider include the indeterminate loads in multiple webs and the complexity of formwork design. Examples are shown in Figure 4-15 to Figure 4-17.

The use of internal or external prestressing also has an effect. Internal prestressing gives a minimum web thickness for ducts and associated reinforcement. External prestressing does not have such a constraint and webs can be thinner. The designer will use these constraints, along with the minimum web thickness needed for shear and develop a cross-sectional design.

A single cell is often favoured as it minimises the number of forms needed and allows faster stripping and hence turnaround of the mould. As the width of the cell increases, the local top slab bending becomes important. This leads to either more webs or to methods of stiffening the top slab, by ribs, struts or transverse prestressing. Single cells are commonly used for deck widths of up to 20 m; wider single-cell decks are possible but less efficient. Two- and three-cell boxes of 35 m to 40 m wide are possible.

Further details of the design of these deck sections are presented in Chapter 5.

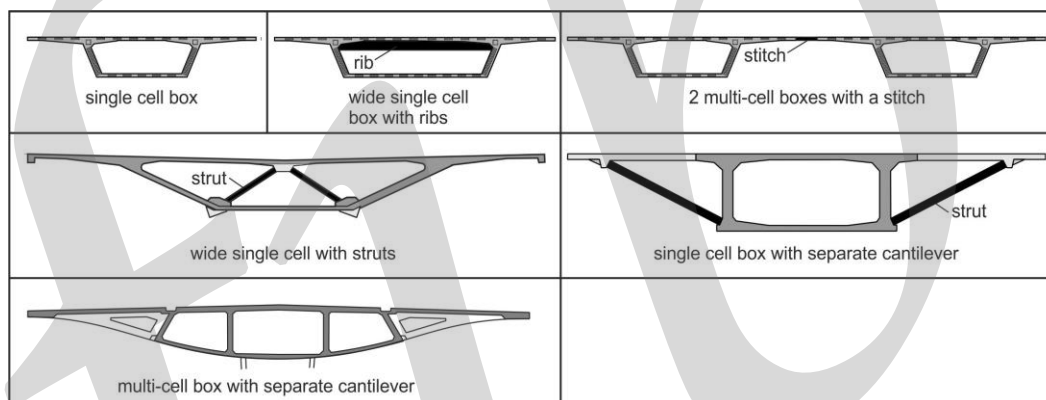


Figure 4-19: Box cell types

Transverse prestress

Transverse prestressing has advantages and disadvantages both in structural behaviour and constructability.

Some of these are set out in Table 4-2. The designer of a segmental structure has to balance the particular demands of light weight for the segmental construction method with multiple construction activities.

Large width cross-sections have two main problems: their high weight and the bow effect. These are also a function of deck type. The bow effect is a distortion due to thermal gradient effects in a segment being cast. It is a function of in-plane stiffness and becomes increasingly important as deck width increases.

These effects and mitigation measures are discussed in more detail in Chapter 5.

Table 4-2: Advantages and disadvantages of transverse prestressing

Advantages	Disadvantages
Allows a thinner top slab and hence lower weight	Additional cost due to many anchorages
Allows a longer slab top slab and hence wider single-cell boxes	Additional work activity
May be needed if long cantilevers are used with a spine box	Transverse stressing may delay erection of spans
	Anchorages can cause congestion at deck edges with parapets and drains.

4.4.4.3 Depth/span ratio

For constant depth, a ratio of $1/22$ is commonly used (between $1/20$ and $1/25$). When external prestress is used, mechanical depth decreases, so greater depths must be provided, and ratio of $1/18$ is used. Previous values are for roads, for railways the ratios become $1/15$ and $1/12$ respectively.

When a variable depth deck is designed, the ratio over the pier is $1/17$ to $1/20$ with a minimal depth at the mid-span of $1/40$ to $1/50$, but usually not less than 2.5 m. A minimum of 2.2 m is common in order to allow movement inside the box. In the case of very long spans (more than 130 m), it is convenient to increase the depth over the pier, while maintaining the depth at mid-span, emphasising the variations of the depth, but this depends on the actions during construction.

For a pre-design of the cross-section, see point 2.4 of “Prestressed concrete bridges built using the cantilever method – Design guide” by SÉTRA [SETRA, 2007].

4.4.4.4 Variable depth

For larger span lengths, it is usually more economical to have variable depth segments, i.e. deeper segments at and near the piers and less deep segments at mid-span. The additional casting effort is offset by savings for both the permanent structure as well as for erection and lifting equipment. An alternative is the use of segments with thicker webs and bottom slabs in the support zones, which allow the use of one set of outer formwork throughout, but have an enhanced capacity with respect to vertical shear and hogging moments. This is structurally less efficient than deep segments, but potentially economically viable, especially for the SSM.

From the prefabrication point of view, box sections with variable depth are a little more complicated than with a constant one, but the production efficiency ratio can be more or less the same.

Inclined webs enable wider decks with lower weight, but the combination of inclined webs with variable depth can lead to a more complicated formwork solution. Therefore, inclined webs are more frequently used in constant depth and vertical webs in variable depth.

Handling of precast elements must be taken into account since it can lead to a reduction of height compared to cast-in-place bridges; it's a simple question of equipment capacity.

Usually second grade parabolic variation is more convenient from the structural and aesthetic point of view. In some cases, a third grade parabolic shape is selected, especially if the actions of the launcher are not very big. From the aesthetic point of view, this solution seems to vary linearly with constant depth in the centre of the span. Because the strong curvature of the bottom slab close to the pier, it is important to take into account the deviation forces as a consequence of the compression stress in that slab. This effect is of paramount importance in case of linear variation, at the point of change of inclination.

Conventionally, it is assumed that for spans smaller than 60 m, constant depth is more convenient.

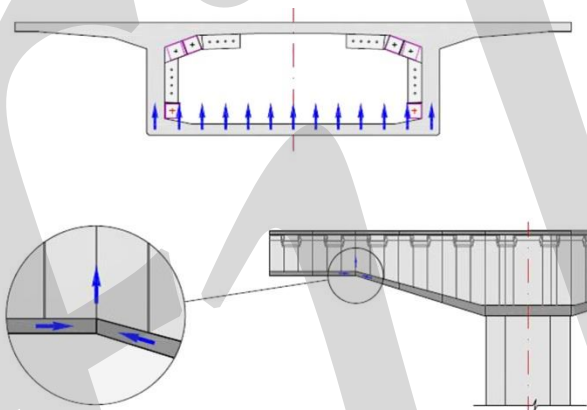


Figure 4-20: Deviation forces in lower slab

The typical height to span ratio of the profile for variable and constant depth can be obtained from the statistical study in the aforementioned SETRA document.

4.4.4.5 Segment length

The fundamental principle in the choice of segment lengths is to reduce the number of joints. This is predominantly a trade-off between speed of erection (minimum number of segments) and cost of erection equipment (minimum weight of segments).

If segments have to be hauled on public roads, this typically provides a further constraint for the permissible segment length (usually 2.5 m). Typical segment lengths range from approximately 2.0 m to over 4 m.

For a deck with a small plan radius, it may be necessary to use smaller segments. This is only a problem if there are very small radii. It should be noted that longer segments benefit

erection but also reduce the cost of prefabrication of reinforcement: fewer larger, more standard cages are faster and easier to produce than a larger number of smaller cages.

Usually the segment length can be varied relatively easily without the need for significant modifications to the formwork so that more than one segment length can be used for a project. In particular special segments, such as pier segments, deviator segments or segments with thicker slabs or webs, are sometimes constructed with different lengths than standard segments.

Where internal prestressing is used it should be checked that the tendon anchorage blisters can be accommodated wholly in a single segment. Considering the location and length of the tendon anchorage, as well as the limitations of the tendon geometry (e.g. minimum permissible curvature radius), shorter segments can result in a less smooth tendon geometry, which would lead to higher reinforcement concentrations to take the greater deviation forces. It is also beneficial to optimise the geometry so that the blister or deviator segments are standardised.

4.4.4.6 Weight limitations

The reduction in weight is more important in the case of precast segmental bridges than for cast-in-place bridges. It is limited by transportation of the segment and the capacity of cranes and winches of the launcher. There are several ways to reduce weight in the cross-section:

- reducing webs (even solution with trusses),
- reducing thicknesses of lower slabs in segments over piers,
- two-stage construction for some parts of the segment,
- transverse prestressing,
- utilisation of high-strength concrete.

This is in addition to the ability to change the length of the section.

As an alternative to an all-concrete segmental beam bridges, composite solutions have been used on a limited number of structures where webs are formed as corrugated steel, or steel or concrete trusses.

4.4.4.7 Other parameters

Deck width can also be influenced by the need to accommodate ramps or bifurcations, and also to make best use of previously existing equipment and formwork. The designer needs to be aware of these issues to enable a design to be sufficiently flexible and optimised with respect to the needs of the construction process.

In some countries, particularly in North America, it is common practice that the traffic runs directly on the surface of the precast segments. To allow this, sacrificial thickness for

planing and grooving is often provided. This should be considered in the concept design as it affects the lifting weight of the segment. Whilst a waterproofing membrane and asphalt pavement are common in many parts of the world, both approaches have advantages and risks that depend on location, climate and environment.

The benefits and disadvantages of these approaches are discussed in more detail in Chap. 5.

4.4.5 Pier and pier head geometry

The geometry of the pier and the pier head are directly related to the structural arrangement as described in section 4.5.2 and the requirements of the erection method.

4.4.5.1 Span-by-span method

Typically the erection truss does not impact on the principal layout of the pier when employing the SSM. Underslung trusses are usually either supported by temporary brackets, cantilevering of the piers, or the vertical truss forces are transferred directly into the pier foundation by means of a temporary falsework structure. The auxiliary front leg of overhead gantries, often required to place the pier segment, is usually supported by temporary brackets. The main legs of overhead gantries rest on top of the box segments.

4.4.5.2 Balanced cantilever method

At the pier locations, structural elements are required to transfer the cantilever moments that develop during segment installation into the ground. Either the permanent piers can perform this function alone or additional temporary falsework frames will be required. When developing a concept for the BCM, the pier and pier head layout should be an integral part of this. Two principal solutions are possible: either a monolithic deck-pier connection or a pier head with bearings supporting the superstructure.

The monolithic connection is typically created by casting an in-situ pier segment in advance of the cantilever installation. This solution offers advantages both during construction (relative simple transfer of cantilever moments from deck into piers, typically without additional temporary measures; with a platform available should deck-mounted lifting equipment be chosen) and during the operational period (no need to maintain bearings at pier locations). However, it is not always structurally viable, primarily as the longitudinal expansion and contraction of the deck will cause bending moments in the piers. The level of these bending moments depends on the stiffness of the piers. Although the pier stiffness can be influenced by choosing different shapes and/or foundations for piers with different heights, unless the piers all have a similar reasonably low stiffness (e.g. all piers are very high and slender) monolithic deck-pier connections may not present the best solution. Other operational issues, such as accessibility of pier locations, cost for in-situ formwork, impact of in-situ pier segment construction on the programme, and mainly the impossibility of making orientation movements, may also affect the viability of this option.

The solution using bearings is more flexible and widely used. The cantilever moments occurring during segment installation are taken by pairs of temporary supports, which can

be either supported by heavy falsework (stability frame) or by the permanent pier. In the latter case it should be checked that the pier head is wide enough to accommodate the temporary bearings. Their spacing in the longitudinal bridge direction and their size should be chosen such as to not to exceed the permissible bearing pressure. Depending on the eccentricity (M/N ratio) it can be efficient to employ additional prestressed holding down bars to control uplift forces and to optimise the distribution of forces between the supports. For high piers, temporary stability frames can become expensive. Here, single shaft columns with large pier heads or alternatively columns consisting of two walls spaced in longitudinal direction and connected at the top, can present the better solution.

Pier head design is guided either by permanent design only or by permanent and temporary works.

Permanent works:

- permanent loadings;
- bearings;
- bearings maintenance (jack positioning);
- access to pier head for maintenance.

Permanent and temporary works:

- same as above;
- temporary jacks positioning for vertical and cantilever stability loadings;
- vertical prestress bars (vertical for cantilever stability or horizontal for horizontal stability for wind and seismic cases, for example) structural system.

4.4.6 Deck continuity

Segmental bridges can be designed as either simply supported or continuous structures. The construction method has a significant impact on this choice, as described in section 4.3.

4.4.6.1 Span-by-span method

Either simply supported spans or continuous spans can be constructed using the SSM.

As with all bridges, in general, continuous spans are more structurally efficient, as the hogging action over the piers reduces the sagging moments at mid-span. Multi-span concrete bridges are typically used for road and rail viaducts and continuity also gives benefits in lower capital and long-term maintenance costs by the reduction in the number of bearings and expansion joints required. To achieve continuous beam action, the newly erected span will be connected to the previous span.

This can be done in a number of ways, but there are two fundamental approaches. The first approach constructs a span between piers. The continuity is made with an in-situ stitch.

The second approach is to build segments beyond the pier, to the quarter point or similar of the next span.

The key feature is that continuity prestress needs to be placed over the pier. The details of the options for this are set out in Chapter 5.3 for details of erection method and Chapter 6 for detailed design considerations, but it is important to understand this requirement at concept design in order to give a feasible geometry that allows for secondary prestressing operations, or prestress tendon couplers, as well as additional construction activities.

Continuity of deck structure and prestressing are additional operations for a simply supported arrangement. Thus, programme requirements may drive the use of a series of simply supported beams, which might be beneficial even though this often results in deeper segments or a higher amount of prestressing.

An intermediate solution can be implemented. A simply supported arrangement is erected in such a way that the construction front moves quickly. As a later activity, continuity is provided both for deck and prestressing.

This leads to three structural arrangements:

- simply supported span arrangement during erection and in service;
- continuous span arrangement during erection and in service;
- simply supported span arrangement during erection and continuous span arrangement in service.

These arrangements have different design issues, many of which are independent of the construction method.

However, there are some design issues and features which particularly influence segmental construction:

- Simply supported (statically determinate or isostatic) spans have the same behaviour in construction and in service. In this case, closing segments are not necessary, but construction tolerance needs to be taken up within each span. The structural behaviour does not change for internal or end-spans, so all span lengths can be equal. For a segmental erection method this gives greater simplicity for segment geometry and launching gantry operation. Each span needs to be individually stable and carry longitudinal and lateral load effects through the bearings. This can lead to temporary works during movement of the launching girder or for the launcher loads to govern the bearing loads, which is not good practice.
- Continuous (hyperstatic or statically indeterminate) spans in construction and in service require a staged analysis to account for the evolution of the structural configuration during erection. Changes are produced during the life of the bridge due to the time-dependent effects of the behaviour of the concrete, although this is

reduced for a segmental structure compared with in-situ because of the precasting of segments. End-spans are typically smaller than internal spans, typically 0.7 to 0.8 of the internal span, to mitigate uplift at the end of the module. This may require greater complexity in a segmental method for segment size and also adjustment of the launching gantry. To make the segmental construction method more efficient, for example to have more equal spans, or spans comprised of standard length segments, the structural design may need to be adapted by variation in deck depth or by thickening of segments or uplift bearings. This is an example of the interaction of permanent works design and construction method in PSB.

- Simply supported span arrangement during erection being made continuous in service is an approach to maximise use of the erection machine. It is thus driven by construction considerations which are rarely critical for non-segmental work, but are intrinsic to efficient, economical segmental construction.

This multi-phase working is not always ideal and is used if erection gantry use is the critical path in the programme and hence, construction influences the structural design. It should be noted that in using this approach, continuity prestressing can be done using external tendons running the full length of the module between expansion joints. This can be an efficient system compared with individual continuity prestress over each internal pier.



Figure 4-21: Simply supported spans with span-by-span-method



Figure 4-22: Monolithic connection between pier and deck in span-by-span-method (Deep Bay link, HK)

4.4.6.2 Balanced cantilever method

Due to the nature of the construction method, the superstructure will have to be continuous over the piers, but the deck is not necessarily monolithic with the pier and can be supported on bearings.

This method also has a direct impact on the span arrangement as described in 4.3.3. For the structural optimum, the spans on either side of these intermediate piers (and the end-spans) should be approximately $0.65 \times$ the regular span length. Without additional variations in the span arrangement leading up to the pier, the cantilever reaching over from the adjacent pier will make up for only $0.5 \times$ the regular span lengths without out-of-balance moments on piers and deck. Alternative means of construction may have to be considered for the remaining length of these spans (false cantilever from the abutment, using temporary piers, falsework, designing a temporary cantilever, on drop-in segments).

4.4.7 Pier-deck connection

The connection of deck to pier has many aspects in common with a cast-in-place solution. The choice between a monolithic pier-deck connection and a deck supported by bearings over the pier depends on many structural factors, such as the span length, total bridge length, pier height and flexibility requirements (Figure 4-22 to Figure 4-25). It is important to provide enough capacity to allow for horizontal movements, wind and seismic activity, or rather, the probability thereof, must be taken into account.

During construction, the deck can be built on jacks to provide construction tolerance and adjustment of geometry, in which case an extra operation must be done to provide permanent fixity. Support on a jack may not give sufficient stability for construction operations, such as gantry movements and reactions on different parts of the structure. In this case, additional temporary works may be needed.

Alternatively, if a monolithic pier segment is used, this may need to be stiff and hence heavy to carry the moment and allow connection to the pier. As already noted, the weight of this pier segment can be a limit on the construction equipment and hence construction method and equipment availability may influence the intended structural design, if the required equipment is not available.

As on non-segmental bridges, to give longitudinal flexibility whilst providing a stable construction platform, twin leaf piers can be used. Operating in push-pull they can be sufficiently slender to flex longitudinally, and materials are used efficiently. Stiffness can be tuned by plan curvature and by partial connection of the leaves at the top or the base. This gives the designer the opportunity to optimise the structural design to suit the global behaviour. In a segmental context, the twin leaves also allow the use of twin segments to reduce segment lift weight.



Figure 4-23: Examples of twin leaf piers continuous with the deck. The use of taper and curvature and connection between the leaves allows pier stiffness to be varied [Srinivasan, 2004]



Figure 4-24: Simply supported deck over pier

Figure 4-25: Fixed deck over two separated shafts

4.4.7.1 Rotation of piers

A particular issue with segmental construction which is sensitive to geometry is the rotation of piers, which has significant influence on the geometrical control during the construction process. In these cases, it is recommended to calibrate the model of analysis with real measurements or to adopt a structural scheme, which is insensitive to pier rotation. Alternatively, additional temporary supports can be introduced to mitigate and control the rotation. Care is needed if the rotations are permanent and subject to creep and shrinkage, as this is less easy to evaluate accurately *a priori*.

4.4.8 Concrete long-term effects

In most of the cases described above, the longitudinal structural system endures a huge change during the construction process. In the short term, the total efforts in each section are the result of the addition of all the stages the structure has experienced until its completion. In the long term, creep effect produces additional changes in the loads on the structure. These effects need to be taken into account, as described in Chapter 6.

4.5 Materials

4.5.1 Concrete grade

For segmental bridges there are particular benefits in the use of concrete with higher strengths. In the permanent situation, high compressive strengths allow some reduction in deck cross-section, and hence segment weight, especially in areas such as near piers. Where design codes allow increases in allowable shear stress, there is some benefit in the reduction of web thickness, although geometry of reinforcement or internal tendon ducts may govern web thickness.

A particular feature of segmental construction is the need for high early age strength to allow early stripping of the forms and allow lifting and stacking. High early strength has a significant effect on the casting programme and hence the cost.

It should be noted that for projects located in developing countries or with very restrictive codes, consideration must be given to grades of concrete that are locally available. These may vary significantly by geographical location.

Also of notable importance, strong consideration should be given to the concrete's consistency. With the use of polycarboxylates, the concrete becomes self-compacting. Self-compacting mixes are highly flowable, yet they do not suffer the effects of aggregate segregation. The use of this type of concrete has made the normally complex and tedious process of concreting precast segments significantly less cumbersome.

If non-self-compacting concrete is used, care is needed to avoid local honeycombing, particularly at the bottom of box girders. This may lead to the need for segments to be cast in various phases with differing concrete consistencies. These issues can be mitigated with the use of self-compacting concrete.

There are other behavioural characteristics of self-compacting concrete that require emphasis and must be considered in the design and construction phases. They are as follows:

- The fluid-like behaviour of self-compacting concrete and the forces thereof, which act upon the formwork, are much greater than when compared with conventional concrete. For this reason, formwork design requires special attention, especially in floating effects of internal forms and deflections in panels.
- The performance of the concrete suffers serious variations in relation with the total amount of water. As such, the humidity of aggregates must be controlled.
- In regards to concrete strength and the performance thereof, it must conform to another, usually more restrictive, condition than that of the 28-day resistance specified on drawings. For productivity reasons, the concrete needs to reach a strength of around 14 MPa within 10 hours of placement. After 10 hours, the forms can be struck without the risk of cracking on cantilevers. It is particularly important

to have the suitable tension strength to avoid those cracks, but it is easier to control it by means of compression strength; therefore specific strength tests must be done before striking.

- Typically grades from 35 MPa to 65 MPa are used.
- Increased heat of hydration due to high early strength mixes.
- Increased risk of alkali-silica reaction for some rapid strength rise mixes.

4.5.2 Prestressing

4.5.2.1 Permanent prestress

Generally, the permanent post-tensioning of precast segmental bridges may be applied either by external tendons or internal (bonded) prestressing, or by a combination of these systems.

In some countries, there are tight restrictions regarding the crossing of bonded prestress cables through joints between precast segments, or, even if allowed in the design codes, some demanding requirements to guarantee the watertightness of cable ducts. Frequently, these restrictions make the use of bonded tendons economically impractical, so external post-tensioning has to be implemented.

In most countries, the common solution is an implementation of bonded tendons or a combination of external and internal.

In general, it is assumed that external prestress is more expensive than internal, although this depends on several circumstances: local codes, construction process, and design of external prestress (replaceable, ability to be re-stressed, type of ducts and filling). Aside from the maintenance advantages, external post-tensioning (PT) offers the possibility of tendon replacement or can provide additional reinforcement to the structure, but the cost increases significantly. External prestressing is also slightly less structurally efficient.

Span-by-span method

Typically external post-tensioning is used for precast segmental bridges installed by the Span-by-span (SSM). This is advantageous, as external tendons can be installed very quickly and a nearly ideal vertical PT arrangement can be produced without affecting the required web thickness. They also offer benefits with regard to durability, as the corrosion protection is pre-applied under a controlled environment and can be inspected during the operational period of the structure. These benefits must be balanced against the need for additional anchor blocks and deviators, as well as the reduced available internal lever arm (distance between centre of compression zone and tendon axis), which can lead to a higher amount of prestressing. The behaviour of external PT for ultimate limit state (ULS) is less efficient, as the tension in the cables increases due to rotation of the section.

Inherent to the design of external prestress, many specific details such as anchorage blocks, deviators, protection measures, and so on are required. This is not specific to PSB, so this is not the scope of the present document; however it is noted that the constraint on weight of the segments can lead to segments with these elements governing the temporary works.

Internal post-tensioning or a combination of internal and external PT can be used as well. A 50/50 distribution has been used several times, but this depends on the construction process and is not recommended.

The use of internal post-tensioning in combination with dry joints is not recommended due to concerns about tendon corrosion at the non-watertight joints. This problem can be solved by sealing the segment perimeter. Dry joints can also produce very high stresses in certain points. To account for these concentrations, a decrease of the design concrete resistance is the typical adopted approach.

In the design for span-by-span erection, the designer needs to ensure that there is a feasible and practical method for placing and removing stressing jacks, which may require temporary openings in the box girder.

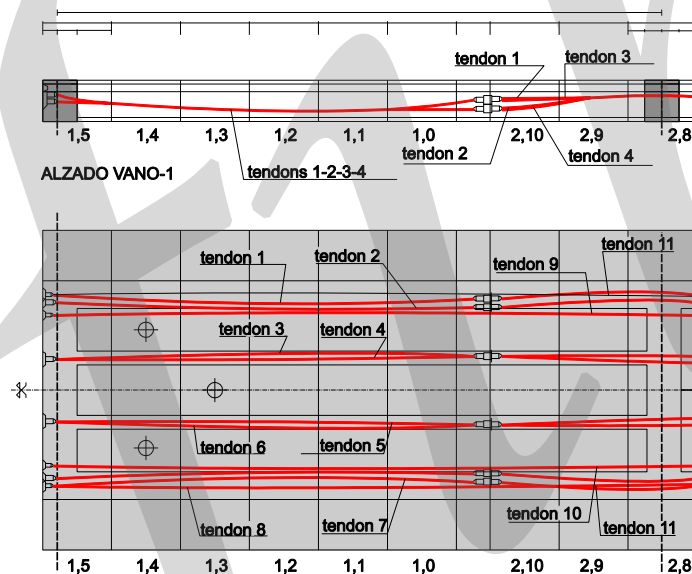


Figure 4-26: Prestress arrangement for SSM

A particular conceptual design issue with the PSB SSM regards the continuity prestressing tendons over and adjacent to the pier. Continuity prestress can be done in a number of ways:

- long tendons over multiple spans after spans constructed,
- cables terminating at or near $\frac{1}{4}$ span,
- crossing cables close to the pier segment,

- cable couplers (Figure 4-26), and
- straight cables adjacent to the pier.

The variety of methods have advantages and disadvantages, but also in the particular context of a PSB have an effect on the construction methodology and hence, erection equipment. There is further discussion of the details in Chapter 6.

Balanced cantilever method

The PT layout is largely influenced by the balanced cantilever construction method. In particular the requirement to resist the large negative moments due to cantilever dead load and erection load typically results in the use of internal tendons arranged in the top slab ('face anchors'). They are usually anchored at the intersection of the web and top slab, but anchorage at a blister inside the box is also an option.

The advantage in this last case is the independence gained for the assembling activity and installation of prestress. When arranging the anchors and angles, one needs to take into account the dimensions of the jacks so as to avoid any interference with webs and slabs. This constrains the layout of the PT. Another important condition is the need to fix the holes in the end bulkhead for the form unit, which has a fixed array of openings for all cables to pass through.

The required mid-span continuity PT across the closure pour (both top and bottom) is also typically realised using internal tendons in the respective slabs. However, external tendons could also be used for the continuity post-tensioning, in particular where the number of tendons in the hogging zones, at and near the piers required for the cantilever construction, is insufficient for the permanent situation. Here the additional post-tensioning can be efficiently created using continuous external tendons with a high point over the piers and low points at mid-span.

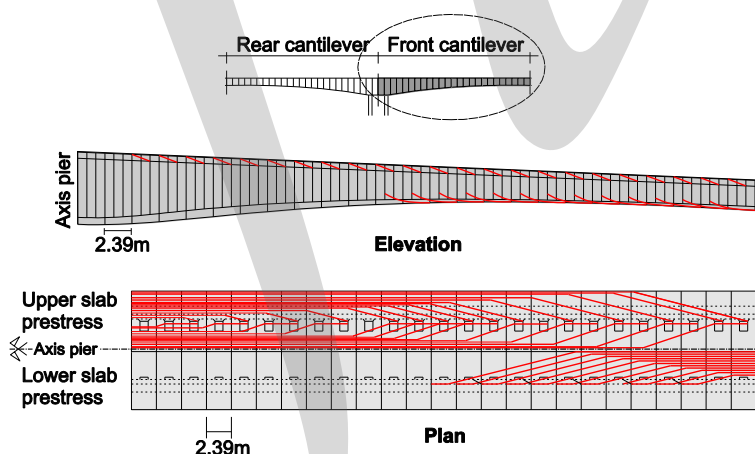


Figure 4-27: Prestress layout in BCM

4.5.2.2 Temporary post-tensioning

During construction, two types of temporary prestressing can be required: between segments and vertically between the deck and the supports to provide stability. This temporary prestressing is heavily dependent on the erection method.

Temporary post-tensioning between segments

Temporary post-tensioning secures the segments in the correct position until the permanent prestress is installed and stressed. It is required for structures with epoxy glued joints, but may be used to ensure perfect matching of dry joint segments.

The temporary prestressing system between segments has three main purposes:

- To provide capacity to assemble each segment in a very fast process without the help of the permanent prestress. Segment assembly and installation of permanent prestress can then be independent activities.
- To squeeze out excess resin ensuring tightness. The smaller the thickness of resin remaining between segments, the better the final geometry of the cantilevers will be. This thickness must be uniform in order to avoid undesirable angles in geometry. For this reason, the total action of this prestress will have a small eccentricity.
- To provide the joint between segments with an initial minimum compression and keep this minimal safety value during the construction process. This depends on local codes.



Figure 4-28: Example of temporary prestress with anchorages in ribs

The number of segments held on temporary prestress is limited, usually to two on either side of a pier, in order to avoid a single bar failure inducing a progressive failure. Alternatively, an arrangement alternating bars can be made to give more robustness.

Typically, high-strength threaded bars are used, which can either be anchored internally to the segment cross-section or in small anchor blocks or ribs inside the box. Generally the bars will be removed after installation of the permanent post-tensioning and reused. Some design codes may specify particular requirements, but it is recommended that, in general, the bars should not be stressed to more than 50% of their ultimate capacity. If higher stresses are required, specific testing/marketing limitation of the number of cycles of reuse of the bars should be carried out. Alternatively, internally anchored bars can be left in place and used as part of the permanent post-tensioning.

Regardless of whether the bars are left in place or removed, the ducts, when set within top and bottom slab, of internally anchored bars must be grouted.

Temporary prestressing can be inside or outside the deck box girder. External prestressing may require temporary blisters with connection to the deck. These connections need to be coordinated with prestressing in the top slab.

Vertical temporary prestress to provide balanced stability

This prestressing is only required in balanced cantilever construction.

In general this question is similar to the cast-in-place method, but some specific issues have to be addressed:

- This prestress must be designed to allow easy removal during orientation operations.
- Part of the system is installed over the pier and the segment over the pier has holes in it to allow the passing of the bars/cables. Careful geometry control must be implemented to ensure both ends match.
- A geometrical tolerance must be provided, not only for the compensation of mistakes, but also to allow capability of movement during orientation operations.
- To control the overturning moment, it is good to monitor the stresses within this prestress and its variation during construction.
- In some designs, this temporary prestress becomes permanent to give continuity for piers. In this case, elements of protection against corrosion must be implemented.
- The solution for this temporary prestress comes from threaded bars or cables with adjustment devices in anchorages.

Specific recommendations for the structural analysis of these elements are given in Chap. 6.

4.5.3 Joints and shear keys

The connection interface between segments can be achieved using different types of joints. This element is of critical importance for the following reasons:

- geometric accuracy (the segments fit accurately with the alignment once erected);
- structural safety (load transfer between adjacent elements), and
- durability (e.g. watertightness).

Construction of joints must be simple and economical. A limited number of different joint types or configurations have been used in precast segmental bridges. The evolution of joints in PSB is closely related to the historical evolution of joint technology.

4.5.3.1 Type of joints

During the erection process, the precast segments are either simply stressed together without any additional treatment of the joint (i.e. dry joint) where the segments are glued together by means of an epoxy resin applied to the joint surfaces, or a concrete or mortar joint is placed between segments.

Hence, four different types of joints can be distinguished with regard to the size of the gap between adjacent end surfaces of segments and the type of material between them:

- Concrete cast-in-place or mortar joints. A significant gap is placed between segments. If used on each segment, this eliminates the need for match casting and high accuracy of the segment geometry, but then falsework and a long period for hardening of the concrete are needed, thus negating one of the advantages of segmental construction. Depending on the spacing between segments, they may require reinforcement. Their use is more common only on special locations such as mid-span closure of long single spans.
- Epoxy or dry joints. There is a negligible gap between segments. They require a perfect fit between the ends of adjacent segments and a high accuracy of the segment geometry as tolerances cannot be compensated on site. This is achieved by match casting.
- Epoxy glued joints are a common type of joint in modern segmental construction. Depending on the specific durability requirements of the country, they can be used on bridges with internal tendons. Resin must be free of sand so that it can be squeezed, retains a small thickness and maintains the geometry. For resin requirements, see the FIP publication “Proposal for a standard for acceptance tests and verification of epoxy bonding agents for segmental construction” (FIP 1978).
- Specific environmental conditions are needed for application of the epoxy and there are practical restrictions when working in environments with high humidity or low temperatures.
- In some conditions, it is possible to use dry joints with no material placed between segments. Their use depends on specific durability requirements, and in some countries they are not permitted, particularly when combined with internal bonded prestressing. They provide a lower level of durability, since watertightness through

the joint is not guaranteed but allows for faster construction. The *fib* does not recommend the use of dry joints without duct continuity (see *fib* Bulletin 33, p. 28).

Part of the choice of the type of joint is a function of flexibility of the design for construction tolerance, durability requirements, and type of prestressing used and environmental conditions on site.

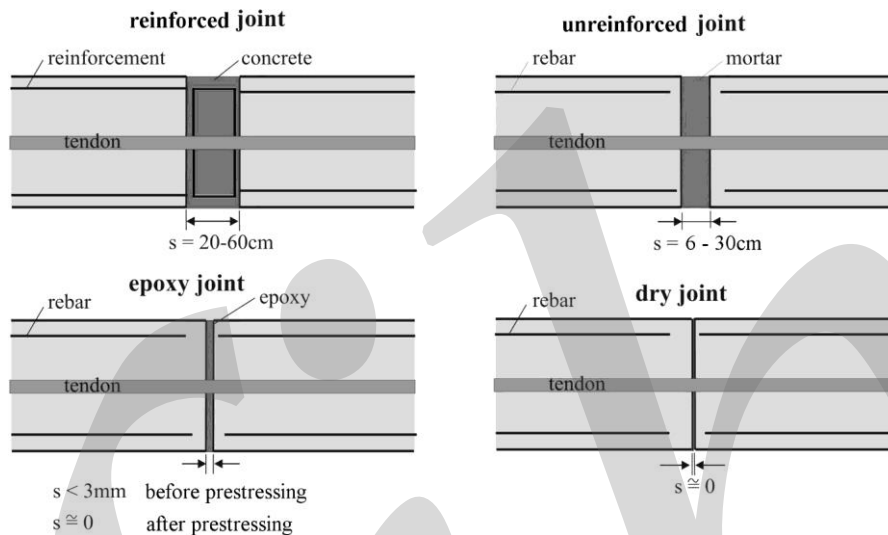


Figure 4-29: Joint types

If no deck waterproofing is used, then more care is needed for the waterproofing of the joints.

As stated above, the use of dry joints is not recommended in combination with internal tendons due to durability concerns and especially in an aggressive environment. However, additional measures from a design point of view can be implemented to guarantee the durability when local codes allow it. Duct couplers that provide a watertight seal should be provided in this case (see *fib* Bulletin 75, *Polymer-duct systems for internal bonded post-tensioning*).

Dry joints have been observed to permit percolation of water due to shrinkage as well as creep and temperature induced warping of segments. As a result, some local standards prohibit the use of dry joints in regions where de-icing salts may be applied regardless of the type of post-tensioning.

4.5.3.2 Considerations about the epoxy glue

The purpose of epoxy glue in joints is:

- to help in assembling and a perfect cast, lubricating both surfaces.
- to seal the joint, ensuring tightness and preventing:

- the injected grout within prestressing ducts from going out through the joint
- water and ice from going into the joint for the service live
- to even out contact stresses. The first contact of concrete against concrete is made with a minimal portion of the prestress. When the epoxy glue has hardened, the rest of the prestress and loads are applied over a solid surface. In the case of dry joints, some authors recommend to take into account a decrease of the strength of the concrete due to the concentration of stress in some points.

4.5.3.3 Shear keys

Shear keys are used to provide locations for joints of adjacent segments and to carry some shear load. The size and shape of the shear keys should be such that they are minimised to simplify the demoulding.

Shear keys should be positioned so as to simplify the stop end formwork geometry (with a constant position in as much as possible). Commonly, the shear keys are within the width of the webs and flanges. However, shear keys can be formed across the full width of the web giving a jagged joint. This has benefits in ease of casting and reinforcing, but is less visually pleasing.

Section 5.2.5.2 discusses joints in further detail.

4.5.4 Closing stitch

For speed and economy, the closing of segments on prefabricated structures is carried out using the simplest possible system. This closure is thus reduced to its simplest form, not exceeding 250 mm to 400 mm in length if possible. Its shortness makes it impossible to tighten the cantilever tendons that are usually attached to the last standard segments. Therefore, temporary bars provide the only form of prestressing before the jointing takes place.

It is important to take into account the vertical gradient when the closing section is being cast, as this can affect the final geometry and other elements of the design.

4.6 Durability

4.6.1 Deck drainage

It may be required for drainage from the carriageway to be fed into a collector pipe inside the box, from which it would run across to a vertical outfall at the next pier location. Depending on the position of the kerbs relative to the webs of the hollow box section, the gullies, and even more so the pipes connecting the gullies with the collector pipe inside the box, might conflict with the prestressing tendons in the top slab. The conflict scenario is particularly likely for balanced cantilever bridges where numerous internal tendons run side by side in the top slab, potentially making angular protrusion very difficult. Therefore, it is recommended to verify the compatibility of the PT layout with the proposed gully

arrangement at an early stage. Potential clashes also need to be considered if transverse prestressing of the deck is used, as drainage and parapets may be in close proximity to the tendon anchorages. Longitudinal runs of the collector pipe also need to be space proofed with external post-tensioning tendons and any other internal fixings, such as electrical or communications cable trays, which may also require clearances to water. As in any other box-girder bridge, the potential water ingress within the box and the treatment of the drainage requirements of the bottom flange might require specific considerations.

4.6.2 Durability of internal bonded prestress

Corrosion of bonded tendons crossing precast joints, or rather the protection from corrosion, is a key issue in PSB technology, and is sometimes the object of controversial discussions. In dealing with bridges built with precast segments, the need arises to guarantee the sealing of the joints so as to obtain the watertightness (and hence, durability) as prescribed by the local codes.

Some details can be implemented to meet this requirement:

- Design requirements. Serviceability limit state conditions are usually more stringent than in other types of structures, as detailed in Chapter 6. This requirement points towards greater watertightness of the duct at the segment joints under normal loading scenarios of the structure; hence the deck will perform better from a durability point of view.
- Duct tightness testing. The sealing of the joint is usually tested first with air, but can also be checked by testing with injection pressurised water. Once clean water exits the port holes, pressure devices take readings to verify that the pressure is stable. If stable, it is assumed that there are no leaks. Then, the procedure is repeated, but this time with pressurised grout. If leaks appear at this time, the grout will seal those locations. Air testing is also used, but is less common than water testing.
- Use of glued joints. The structural capacity of the resin is not considered in the calculations. With the application of temporary prestressing, the excess of resin is squeezed out of the joint interface, thereby ensuring a proper seal as described in 4.5.2.2. This form of joint can be considered to be a waterproof system in combination with the appropriate design requirements and resin application that is sufficiently well controlled to stop the formation of voids or trapped air between the surfaces as the joint is stressed.
- Use of rubber gaskets. The watertightness of internal prestressing tendons can be improved by the use of rubber rings around the ducts at the joints between segments. Before casting of the segment, steel rings are placed around the ducts, leaving an imprint of half the thickness of the rubber rings. The rubber rings are placed in this recess, and are compressed when the temporary prestressing is applied. There are also proprietary prestressing systems which have couplers at the segment joint. Refer to *fib Bulletin 75*.



Figure 4-30: Rubber gaskets



Figure 4-31: Metallic rings to leave the room for gaskets

4.7 Specific aspects for railway bridges

General guidance on the initial design of precast segmental bridges was given earlier in this chapter. However, railway bridges impose specific requirements that, although applicable to all bridge types, have a particular impact on a precast segmental bridge.

4.7.1 Loads

Railway bridges are characterised by large traffic loads and severe constraints regarding deformation under service loads, in order to give the tolerances required by the track geometry.

The importance of live loads has following consequences:

- spans over 80 m to 100 m long are not usually economical;
- the slenderness of the deck is reduced, in comparison with road bridges (from 1:12 for rail bridges compared with 1:18 for road bridges); and
- the weight (per meter length) of the deck is high.

These features of rail bridges affect the segmentation and geometry of a precast segmental bridge, particularly the potential need to reduce the length of the segments (due to their weight) and often to increase the capacity of lifting devices.

Rail bridges have particularly high shear loads. A segmental bridge without reinforcement crossing the joints and designed only with external prestress has more uncertainties in shear mechanism (see 6.4.2). It is advisable that these railway bridges have at least 50% of bonded prestress tendons.

4.7.2 Articulation

The articulation of a rail bridge is different from that of a road bridge due to the expansion of the rails as well as the supporting structure. Modern railway bridges, especially on high-speed lines, are characterised by the fact that the track they carry is, as often as possible, continuous over the deck ends:

- When the length of the decks (including when the bridge is made of a succession of independent groups of spans) is less than 90 m to 100 m (in a European climate) then no rail expansion joints are needed;
- When the expansion length of continuous decks exceeds 90 m to 100 m, the track must be cut and a rail expansion joint must be incorporated. For high-speed lines these devices are expensive and need continuous maintenance.
- It is not usual, even when an expansion device is integrated into the track, for the expansion length to exceed 450 m. If the total length of the bridge is not compatible with this constraint, solutions consist either of adding an intermediate individual span to carry the expansion devices, or designing a cantilever bearing in an intermediate span.

Thus, the span arrangement and location of movement joints for a structure are different from those of a road bridge. The previously noted comparison of these configurations needs to be reviewed, as the economic analyses must not only be performed only on the bridge itself, but must also include considerations about the track (construction and maintenance), which are often governing.

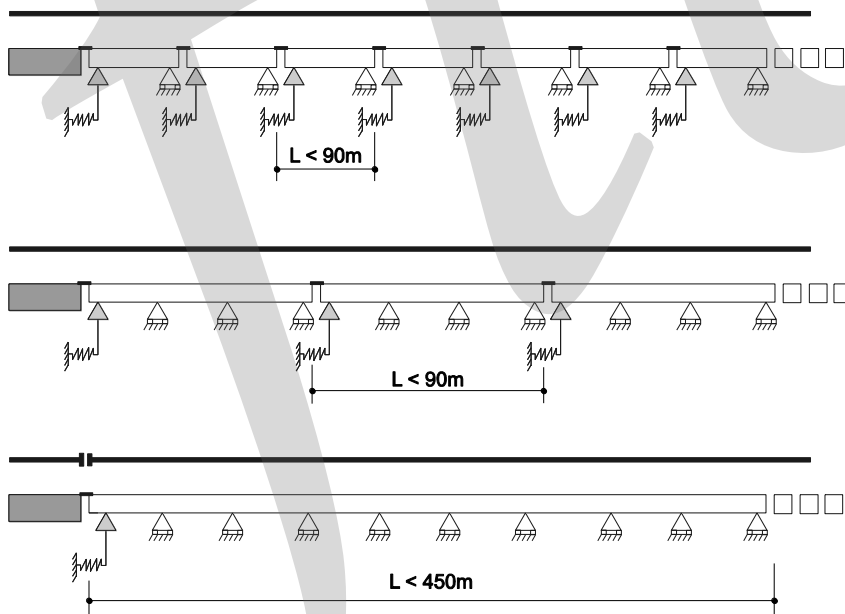


Figure 4-32: Different types of general design for railway bridges

4.7.3 Electrical induction

Sometimes electrical isolation may be required of the prestressing units in order to:

- prevent excessive corrosion phenomenon due to the electromagnetic environment of railway bridges under the catenaries or third rail (this can also occur if a bridge carries an HV cable); and
- ensure the safety of people who may come in contact with the tendons, particularly during maintenance.

For a precast segmental bridge there is a particular design and detailing issue in providing reliable electrical continuity across the joints.



Figure 4-33: Viaducts in Avignon

5 Construction

5.1 Introduction

Close coordination and integration of design and construction is vital for a successful precast segmental bridge. This chapter describes some of the main issues and parts relating to construction.

First the main overall issues are described. The process of production of segments in the casting yard is described, then erection by span-by-span and balanced cantilever methods. These give a good overall understanding of the construction process to assist with design and to promote safe, efficient construction.

5.2 Main issues in relation to construction

5.2.1 Introduction

This section sets out some of the more important issues relating to construction. Whilst these must be dealt with on site, it is also important that the designer be aware of and understands the issues, so that these are mitigated early in the design process to minimise delays and cost.

5.2.2 Construction cycle, process and temporary equipment

A fundamental requirement of a successful project is the coordination and consistent interpretation of the construction cycle by both designers and contractors. Moreover, if designers understand the constraints and drivers of the construction programme, they are better placed to develop schemes that assist the construction process.

An example of this is understanding the use of temporary equipment in the construction process, regarding both the duration of the temporary works and the need to allow time in the programme and physical space to both install and remove the temporary works. A good designer will also consider approaches to minimise the temporary works if this reduces the project cost.

There is a range of temporary works and equipment, some of which are particular to or more common on precast segmental bridges.

Orientation jacks and devices: Jacks are often used at pier heads to support the deck pier segment. This allows the geometry of the segment to be fixed accurately to ensure that the cantilever geometry starts in the correct place. The use of jacks often requires auxiliary elements to support them or additional falsework in piers, as well as scaffolding to give access.

Stability elements: The use of bars, cables and temporary supports to stabilise the structure of both piers and deck at various stages is common. It is important to consider the structural behaviour during erection with such supports or ties in place. It is not unusual for the structural behaviour to change during erection, for example from simply supported or cantilever to multi-span with continuity. In doing so, the temporary works, for example temporary ties, can lock permanent forces or deformations into the structure. Such forces can be detrimental to the structure, but by careful forethought can be used to produce beneficial effects such as precamber.

False Cantilevers: False or temporary cantilevers are another example of temporary stability works, but are particular to the erection of the end-span of a balanced cantilever bridge. At an abutment or movement joint pier the deck is temporarily fixed ('nailed') to the pier to allow cantilever erection of the shorter side span. After closure with the cantilever from the first pier, the temporary cantilever tendons are cut and a movement joint reinstated over the abutment.

Falsework in lateral spans: These are used when linking parallel adjacent decks, often by joining the box cantilevers. The time and erection sequence is important to ensure that the movements and deformations of the two spans are compatible before completing the stitch.

5.2.3 Temporary prestressing during erection

As noted in section 4.5.2.2, segments are first held in position by temporary prestressed bars. The purpose of the stress bars is to:

- maintain the segments in position;
- apply a minimum pressure on the joint to spread the glue on the joint evenly and not to alter the general geometry of the span. The minimum pressure on the joint shall be in accordance with the relevant codes and is generally in the range of 0.1 MPa to 0.15 MPa; and
- withstand certain temporary loads due to the erection equipment so as to minimise the final prestressing requirement.

Stress bars are reused from one pair of segments to the other. Checking of the integrity of the bars shall be done regularly and bars shall be replaced from time to time when required (if they are damaged or to be agreed with the bar supplier depending of the stressing force).

As no tension is allowed at joint location, the removal of stress bars shall be defined by calculation. Generally top temporary stress bars are removed after stressing of cantilever tendons. Bottom stress bars are sometimes left in place on a few joints to keep a minimum compression strength at the joint at the bottom chord (>0 MPa or according to relevant codes).

Top stressed bars can be placed either on the top of the deck using some special steel devices (Pos 1) or within concrete blisters inside the segment (Pos 2) (see Figure 5-1).

Bottom stress bars can be placed on concrete blisters along the webs inside the segment (Pos 1') or in the middle of the bottom slab (Pos 2') (see Figure 5-1). Very deep box girders may need blisters for stress bars part way up the webs. However, their use is not preferred as they are an additional complication to the segment casting, and hence their need should be verified by calculation to ensure that top and bottom bars alone can give minimum compressive stress over the full height of the webs.

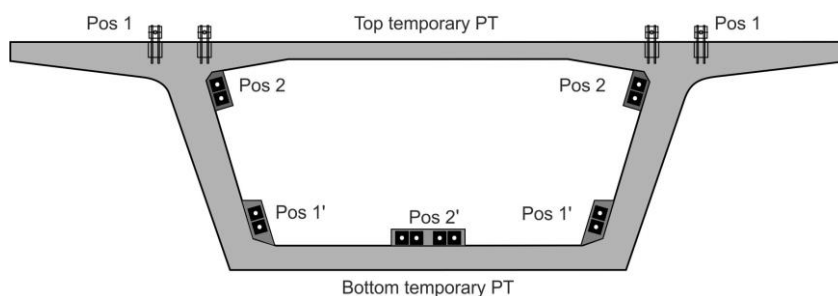


Figure 5-1: Typical temporary prestressing arrangement

A particular consideration needs to be made for curved decks, where the temporary prestress bars could be at different angles on either side of the segment. As it is preferable to reuse anchorages to avoid congestion within the box and to minimise additional works, the anchorages need to be detailed so that they are able to make allowances for variation in the angle of the bar. Measures to assist this include making the hole oversized and using washer plates, but care is needed to ensure that an even load is applied to the anchor blocks. A similar consideration is required for launched decks where the anchorages at the Pos 1 and Pos 1' positions follow the bottom slab profile. It should also be noted that care is needed where the deck has changes in web or flange thickness to ensure that the anchorage geometry makes allowance for the geometry changes and that there is no clash of bar with concrete segment at the step in thickness.

5.2.4 Early striking, handling of segments

As noted in section 4.5.3, a particular driver in segmental bridge construction is to minimise the number of moulds. To do this, the segment needs to be demoulded as quickly as possible. To achieve this early striking, the concrete mix design needs to be done such that high early strength is achieved as quickly as possible. As the segments are then stacked and stored until ready to be taken to the construction site, the design of the segments should allow safe stacking and mitigate the risk of permanent creep deformation of the section. Hence, long slender unsupported cantilevers should be avoided, and segments with curved soffits may require props.

5.2.5 Joints

A fundamental difference between segmental bridges and other types are the joints. There is a range of joint types for segmental bridges.

The most common are:

- in-situ (wet, or mortar) joints;
- glued epoxy joints; and
- dry joints.

The joints connect segments but also allow geometric adjustment and control of the deck geometry both longitudinally in the span direction and transverse to the span direction.

Joints allow construction tolerance for segments to be placed and accommodate small construction errors. They give the designer and builder some flexibility at the erection stage. In order to adjust the final geometry of a cantilever erected span, it is not unusual to allow for a slender jack at the joint. This requires a minimum space of approximately 180 mm. Once the deck geometry is adjusted, a stitch is made to fix the segments.

Whilst the joint is not a structural element, the deck segmentation, taking into account the number and location of joints, affects the geometry and buildability of the bridge.

- The use of dry joints, without glue, is practical in dry countries with external post-tensioning, but is not recommended where internal post-tensioning is used or where there is risk of water ingress through the joint.
- A logical joint-filling material is concrete. If a joint is over approximately 200 mm long it would usually be reinforced to carry the local shear and to avoid having to reinforce the adjacent segments for shear, as indicated in section 4.5.3. This is an awkward operation to form, place and properly compact the concrete. This is also time-consuming, and hence, wide wet joints are not commonly used as they negate many of the benefits of segmental bridge construction.
- Depending on the slab thickness, a joint of up to 200 mm to 300 mm can be made without reinforcement with mortar or high-strength concrete. Being unreinforced, the construction is greatly simplified, only requiring a form. This is a common route to allow some construction tolerance and geometrical control, but in discrete locations, with usually one or two per span at the most. The minimum spacing can be driven by the space required to connect the ducts.
- The joints between the majority of segments in a bridge are either glued or dry.

Major general differences as well as advantages and disadvantages of dry and glued joints are listed below.

Dry joints:

- simplified erection process (optimised construction speed), cost saving;
- defined load-bearing behaviour (no structural cracks but joint opening when there are loading situations exceeding the decompression); in addition: distribution of joint rotation to several joints in ULS (i.e. reduction of concrete compression/rotation);

- durability: generally durability of the segments should not be an issue, since all surfaces of the segments (including contact area in the joints) are provided with sufficient concrete cover (comparable to reinforced concrete segments in tunnels). However, it cannot be guaranteed that dry joints are watertight (this might be a problem with the top slab joints).

Glued joints:

- Adds an additional step within erection process, leading to slower construction works and higher costs.
- Glue may fill minor surface imperfections and irregularities; it acts as a lubricant when erecting the segments and provides the joints with a watertight seal.
- The glue layer may have a thickness of approximately 3 mm, but after prestress the gap between the segments becomes negligible.
- Epoxy resin generally represents a relatively sensitive component as it is a two-part mix that needs to be site mixed and quality controlled. The material is environmentally disadvantageous as well. However, most of the difficulties with glue are not with the material quality but with the time necessary for the glue to polymerise (harden). Even with the use of special admixtures (for winter or summer), in summer the glue tends to polymerise very quickly (sometime too quickly for the construction process) and in winter it can polymerise too slowly and can delay the erection process.
- In case of overloading, the bridge decompression of the joint and structural damage is unavoidable; further, joint opening/rotation will mostly be concentrated to one joint (this particularly applies for bridges with external tendons or more generally with unbonded prestressing).
- In cantilever construction, the need to have prestress for cantilevering gives good compression to the glue as a part of the construction process. However, in span-by-span construction, temporary prestress is needed specifically to provide compression across the joint as the glue polymerises. It is recommended that this prestress give a minimum 0.15 MPa stress across the joint.

A particular issue for a glued joint is the need to have compression of the epoxy resin during the course of assembly of the segments. The methodology is that during the installation of a new segment, the thin layers of epoxy resin spread both onto the ready segment and onto the one to be lifted. Due to the uneven pressure of the temporarily fixing bars as well as of the dead weight of the segment, there is a risk in this case that the compressive stress is not uniform along the height of the joint. This leads to a less than uniform transfer of axial force across the joint and the risk of voids and water ingress to the joint or a change to the desired erection geometry due to unevenness of the joint.

The management of the risk of uneven glue can be done by correcting the erection of the subsequent segment on the basis of field measurements. This correction is sometimes done using thin distance-keepers plates, or shims, of made of steel or glass-reinforced plastic (GRP). The use of shims is not recommended and is often not permitted by the project specifications. This is due to the risk of stress concentrations being produced at the shims. It has been argued that the use of GRP shims reduces this risk because after the epoxy resin hardens it is stiffer than the GRP and the only long-term effect of using a GRP shim is the reduced contact area. This is a matter of debate that has not been settled. Care needs to be taken on each project to ensure that any use of shims is compliant with local codes and project specifications.

Irrespective of whether the joint is glued or not, the match-cast joints are normally provided with shear keys (positive/negative pattern in match-cast and wet-cast segment). The main functions of the keys are to facilitate fitting of the segments during erection (“LEGO™ system”), to ensure the top slab load distribution at joints and to contribute to the transmission of web shear forces via the joint. Recommendations on arrangement and form of keys are given below.

5.2.5.1 Demoulding, treatment of joint surface

A demoulding agent is often used to assist in removal of the segment from the mould. Before erection, a cleaning operation over the contact surface might be done to remove the remains of the demoulding agent. This is commonly done either chemically or using sand blasting.

In general, patching on the match-cast face is not preferred, unless the following segment is not yet cast and remedial works could be carried out on the match-cast face before casting the next segment.

If patching is needed, it should be shallow: a few millimetres deep relative to the theoretical bulkhead face.

5.2.5.2 Shape of shear keys

As noted in section 4.5.3.3, shear keys are conventionally provided between segments. Early segmental bridges were built with a few single large reinforced keys in the web section. This section can be designed like a corbel. The main disadvantage of this joint type is the concentrated load transfer and the resulting high local stresses.

Multiple shear keys do not have the disadvantages of the single key and are preferable from a construction point of view.

The vertical component of the forces in internal draped tendons can be beneficially utilised in the assessment of shear capacity of the section.

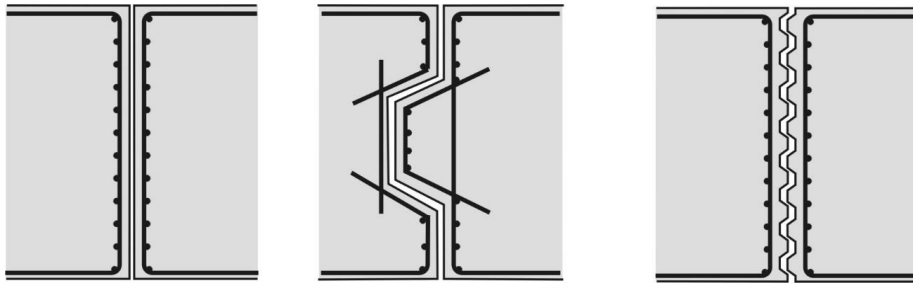


Figure 5-2: Joint with no, one and multiple shear keys

Theoretically, a joint with no shear keys could be considered, with the joint relying purely on the friction between faces. However, this option is clearly not recommended.

Current practice is to use of multiple shear keys distributed over the whole height of the webs and flanges. The fine indentation of the segment surface gives a smooth load transfer and avoids high, concentrated forces. A perfect fit of the end of adjacent segments is required.

The standard joint used in all types of precast segmental bridge structures is a perfectly profiled joint resulting from the match-cast production of the segments. Nowadays in-situ joints are typically only used as closure joints in continuous bridges so that an element is able to compensate longitudinal tolerances.

Regular distribution of keys around the whole interface between segments is common, mostly in the webs, since the upper and lower slabs have more difficulties in providing space for shear keys because of the interference with duct locations.

The angle of the shear keys must also take the forms into account, particularly the internal form and its direction of movement. For example, for an inclined web, the shear keys are often oriented perpendicular to the inside face of the web to facilitate easy stripping, although this is not quite the optimal structural orientation.



Figure 5-3: Example of joint with multiple shear keys

5.2.6 Method of post-tensioning

In most cases the serviceability limit state governs the design of prestressing, requiring a minimum compression at segment joints under serviceability limit state (SLS) conditions; this is dependent on the local code to be applied. However, if there is only limited tendon stress increase in the ultimate limit state (ULS), the ultimate conditions may govern the tendon quantities. This particularly depends on the applicable code, on the prestressing concept (bonded or unbonded) and the joint treatment (glued or dry) and layout. For example, in Germany only external post-tensioning is allowed with precast segmental construction and the maximum joint opening is limited to 1/3 of the construction height (at piers) and 2/3 at mid-span. With these criteria, the achievable ULS tendon stress increase is limited to only 30 – 60 MPa, whereas it has been proven by numerical analyses and several full-scale tests that at least 250 MPa is reached, also under unfavourable conditions and when there is external post-tensioning.

As noted above, a detailed programme of the order of work of cantilever cables, temporary prestress, and assembling of segments is necessary to take into account the analysis of temporary stages.

It is noted that space provision needs to be made early in the design stage to enable prestressing jacks to be brought into the deck, removed and manoeuvred within the deck. This can constrain the arrangement of prestressing in the top slab, to allow a sufficiently wide and long opening. The spacing of diaphragms and location of anchorages relative to the webs and flanges needs to allow the jacks to access the anchorages, stressing operations, and for strand to be threaded through the duct. If work is carried out inside the box, then the designer and contractor should ensure that there are sufficient vents and space inside the deck for safe working.

During stressing, the angle deviation of the tendons gives a resultant force that may tend to burst the concrete. This can be mitigated in design with bursting reinforcement, as in all prestressed bridges. However, in a segmental case, care is needed to ensure that the reinforcement required for bursting can be provided in a segment and does not cross a joint.

If the cable ducts are closely spaced, with only a small thickness of concrete between, then as grout is pumped at a pressure into the ducts there is a risk of by-pass during grouting, leaving some cables not fully grouted. This can be managed by grouting in groups, but even this is not fool-proof, as it is not viable to check whether the grouting mortar fills each cable equally. Practical management then leads to cast ducts with a greater separation from each other, or the use of more external cables.

It is necessary to distinguish between balanced cantilever erection and span-by-span. In the first case, the use of external prestressing can be problematic, especially in the case of top slab cables. In the case of span-by-span and longer span balanced cantilever, with more than 40 m span length, external prestress is more common, but often a good solution is to use 50% of bonded and 50% of external.

5.2.7 Geometrical control

Geometry is the most important issue in segmental construction, since during erection there is a small margin of manoeuvre to correct errors, which is a marked difference from cast-in-place methods. This is an issue for both balanced cantilever and span-by-span methods. Moreover, each erection method has slightly different geometry issues.

Since the deck is built away from its final position detailed geometry quality control is paramount to ensure a proper geometry, this is even more important if the short-line method is used as it is sensitive to accumulation of geometry errors.

5.2.7.1 Geometry control main operations

The sequence to be followed while casting segments should always be the same throughout the process:

1. First thing in the morning, having concreted a segment in a mould between the match-cast segment and the bulkhead the day before, the land surveyor must measure distances in plan with the ruler and levels with the levelling rod. This set of data will be put into the geometry-control software. The program checks these over-abundant data, and is able to detect non-allowable deviations.
2. Following the survey and geometric verification, all operations of stripping formwork, separating the old segment from the new one, transporting and stacking the match-cast segment to the storage area, sliding the “new” segment to match its cast position, cleaning forms and placing the prefixed rebar cage into the cell, and finally closing formwork into the precast cell, are carried out.
3. Subsequently, the match-cast segment “n-1” is adjusted in order to get the theoretical casting coordinates of segment “n”. Corrective movements in plan, elevation and superelevation are applied to replicate the target solid and get rid of the errors made before.
4. Finally, the match-cast segment “n-1” is blocked, and the segment “n” is concreted. The following day, the cycle starts again with the survey.

During the erection process, surveys should be done at regular intervals to compare the expected geometry (data from the precast as built survey) to the real one, after erection. Discrepancies should be identified and contingency measures shall be taken if there are major differences. There are different ways to correct the errors:

- For minor discrepancies, packers can be placed at joint locations to orientate the segment correctly (special procedures shall be followed to spread the load evenly when the permanent post-tensioning is carried out). For further information please refer to the geometric adjustment described in 5.8.4.
- For major discrepancies, an additional joint cast in-situ (around 150 mm) could be done.

- In the worst case, it may be necessary to cast a new segment, but this operation would have a major effect on the erection process and programme.

5.2.7.2 Geometry-control systems

An appropriate survey system to control the real geometry of each segment should be part of the design information.

The geometry control of the segments shall provide enough redundant information to ensure that potential survey-reading errors are detected immediately. The geometry-control points shall ensure that a full three-dimensional geometry of the segment can be measured, including warping.

The system should allow for adjustments to compensate for fabrication errors on each previous segment as the casting process advances.

A control system and special devices are used to mitigate this; these need to guarantee enough accuracy to ensure the final geometry under tolerances. Some desirable characteristics are:

- A control system is not an error free system. So it is very important to perform a redundant survey control, to know the shape of each just-hardened segment. This control should be computer controlled to avoid mistakes. This can be made by using controlled applications, which must be reviewed by experienced third persons the moment there is some incoherent data.
- Allowable error needs to be adjusted to reach the final geometry under tolerances and avoiding warnings on a daily basis.
- Mould stiffness, curing temperatures, movements at early ages, survey mistakes, etc., can cause some imperfections that must be known so that they can be corrected in the immediate next segment.
- Project specific concrete mechanical properties, such as Young's Module, creep and shrinkage coefficients need to be evaluated and, if necessary, included as part of the control system. Further, it is necessary to take into account future events, construction processes and programme to get the theoretical shape. General code-defined values are not likely to be appropriate.

The segments are cast so that their geometry takes into account the long-term deflection due to creep and shrinkage, the effect of the post-tensioning and the deflections due to construction phases.

The “match-casting” method used in PSB requires geometry-control software. The program to be used undertakes several levels of control. First of all, it checks that the input data is logical. Once confirmed that the data are correct, detection of other errors is attempted, such as:

- unacceptable distances between points of control at the joint and points of the bulkhead;
- misalignment between the bulkhead points, providing its angle.

Geometrical control may be carried out with three plates per segment joint, which represent the points of control, a steel ruler and a levelling rod. Once the segment is cast, elevations and distances between these points (alignments between points at the same joint, between points located at different joints including diagonals) are measured. All data surveyed are usually redundant, so internal checks can be undertaken to detect gross errors such as reading or data-recording errors.

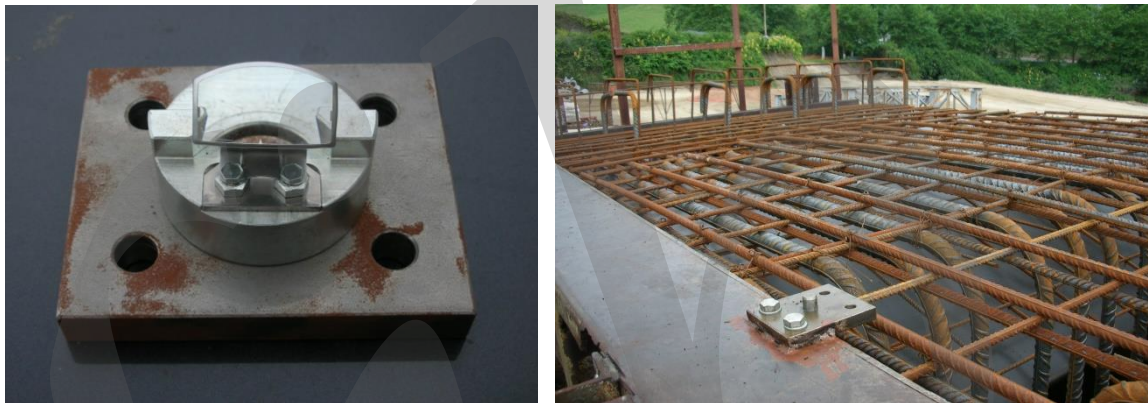


Figure 5-4: Plate used for topographical survey

Using a mathematical model implemented in the software, elevations and coordinates of the already cast segment are calculated. Errors are detected and corrective measures to be taken on the following segment are obtained. Output data from the software are utilised to accurately position the match-cast segment by means of the XYZ chariot acting on the supporting horizontal and vertical jacks.



Figure 5-5: Metal ruler and levelling rod used for the geometrical control

A topographical base station to be used for the geometrical survey must be installed within the precast yard adjacent to the moulds. It is recommended that it be placed on the extension of the cast segments centreline, to approximately equalise to the magnitude of the surveyed errors on the control points. It should not be more than 5 or 6 metres from the precast unit, in order to limit reading deviations. On top of the base station, a digital level may be placed, always at the same elevation, to serve as a level reference.



Figure 5-6: Survey base station close to precast cell

5.2.8 Extreme events during construction

In areas where extreme events such as earthquake or typhoon could occur during construction, the temporary works (gantry, propping) shall be checked for compliance with these severe conditions. The structure of the bridge shall also be checked at each step of the erection. At some construction stages, the bridge structure may be particularly sensitive to these extreme events, thus in some cases driving the choice of construction method.

The design level of a structure needs to be carefully considered under extreme events. The return period of the event needs to be considered, that is, the likelihood of the event occurring during erection, along with the acceptable damage to the key parts of the structure. Repairable damage may be acceptable for some elements but for others. The acceptable damage and return periods can be different between the construction stage and the permanent case, depending on the overall risk.

5.2.8.1 Earthquake

In active seismic areas, special attention shall be paid to the construction steps in which the segments are first held in the structure by use of temporary bars. Particularly when using the span-by-span method, the stage before tensioning the longitudinal tendons is critical under a major earthquake event. At that stage, the segments of the whole span are only tied with temporary bars and held on the temporary works. This temporary prestressing

cannot withstand a strong earthquake, which could cause major damages to the span and even the overall stability of the structure.

5.2.8.2 Typhoon

In an extreme wind, such as a typhoon, the permanent structure must withstand large wind loads. If the stability of the structure is reliant on temporary post-tension bars, then particular care should be taken to give a sufficiently high level of safety,

A segmental bridge also has the additional risk of wind loads on erection equipment. Equipment such as launching gantries can have large wind areas. They may also not be able to be moved to safe areas of the structure if there is insufficient prior warning. In this case, the structure may have to be able to withstand not only the direct wind loads, but also forces due to tie-down restraint of the gantry. Such loads need careful coordination between the erection contractor and the permanent works designer and owner. A site specific risk analysis is a useful way of clarifying risk, ownership and coordinating responsibilities.

5.3 Safety risks

The concept and construction methods of precast segmental bridges are reliable and do not have an increased risk compared to other bridge concepts, if properly addressed. Key elements are the structural safety, safe working methods and safety of site personnel.

Specific aspects for precast segmental bridges are:

- structural safety: check of strength, stiffness and stability of the permanent structure for all construction stages;
- safe working method: preparation of a method statement that describes all construction stages in terms of sequence, temporary (lifting and placement) equipment (what, when and how), robustness of the temporary provisions and equipment; and
- personnel safety: assessment of special safety provisions based on the method statement: exposure and mitigation of risk as well as providing proper staff training.

To develop adequate safety, a risk analysis should be performed to address the risks for each of the three aspects mentioned above. This is generally performed through a risk inventory list prepared by all disciplines involved. Each risk should get a mitigation measure.

The interaction of design and construction makes proper recording of risks and mitigations an important tool in ensuring that all project participants, including the owner, understand the risks and responsibilities for mitigation.

5.4 Quality control

The design and construction of precast segmental bridges requires a tight integration between several interfaces as part of an industrialised process, and requires specific measures to be taken into account in the quality control of the geometry, materials and erection and also specialised workmanship in different areas.

Detailed documents including method statements, fabrication and erection procedures and testing requirements need to be detailed in the contract documents to assist the relevant teams involved in the project to reduce the risks during construction.

In addition to the usual quality control requirements of conventional prestressed concrete bridges, specific attention should be drawn to the following areas.

5.4.1 Segment fabrication

The fabrication of segments, particularly in the short-line method, is a repetitive and time-intensive succession of specialised tasks that usually span a single day cycle. Appropriate methods of quality control, inspection and remedial actions need to be in place prior to the commencement of the works. The following points summarise some of the key areas where special attention needs to be placed:

- Formwork should be inspected prior to fabrication of the next segment (cleanness of faces, internal seals, adjustment to the bulkhead and cast segment previously concreted).
- Dimensional and compliance of prefabrication reinforcement should be verified prior to erection in the segment. Cover tolerances and spacer placing in the segment are also very important. Dimensional integrity of the cage must be kept during transportation.

Specific procedures allowing for local modifications of the reinforcement when clashes arise are highly recommended.

- Installation and leak tightness of ducts and prestressing anchors should be verified.
- Concrete pouring sequence and concrete mix testing parameters should be specifically indicated as part of the construction requirements, particularly if self-compacting concrete is not used. The concreting of each segment requires a specific pouring, sequence and vibrating within each segment to allow the cast of the full section without the formation of cold joints. This process is very sensitive to workmanship skills and the consistency of the concrete mix. Concrete consistency should be tested prior to the pour of every segment.

If self-compacting concrete is used, it is recommended to produce a test specimen that reflects the complexity of the segments in terms of rebar and duct congestion

and concrete slab thicknesses, to accurately adjust the additive mix for the specific concrete to be used in the bridge.

- Due to the short cycle of segment fabrication, curing methods, climatic requirements and minimum concrete strength prior to segment handling should be clearly specified in the quality plan for the project.

5.4.2 Segment storage and erection

A detailed description of the transportation and storage method of the segments, including segment handling procedures, support points, temporary support materials and type of stacking allowed, should be fully detailed in the appropriate documents and the personnel involved fully trained in their use.

The specific erection equipment should only be used by authorised personnel and a specific erection manual and procedures should be in place prior to any activities on site.

The deck erection method shall include measurements to adjust the final geometry of the bridge and an appropriate survey method should be in place during the erection.

If epoxy joints between segments are used, a detailed process of their application should be specified, including but not limited to: joint face preparation, method of mixing and application of the epoxy resin as specified by the supplier, conditions of use and curing process.

5.5 Remedial works for fabrication defects

5.5.1 Fabrication defects at precasting yard

There are some particularities that make the fabrication of precast segments sensitive to certain types of defects. Due to the short pouring cycle between segments (usually a day), the whole process of segment construction can lead to different type of construction defects if specific inspection and measures are not taken adequately. Some of the most common defects during the fabrication of segments are presented below.

5.5.1.1 Formwork leaks

The continuous reuse of movable parts of the formwork might affect its watertightness after several reuse cycles. This can lead to leakage of the grout component of the concrete mix, which can affect the durability of the reinforcement.



Figure 5-7: Grout leakage due to formwork leakage

5.5.1.2 Defects due to concrete compaction and concrete mix

The segment pour sequence, particularly if a self-compacting concrete mix is not used, is very sensitive to workmanship skills. Common problems that might arise in the compaction and pouring sequence of each segment include honeycombs, voids or cold joints due to improper spacer fixing.



Figure 5-8: (from left to right). Cold joint, honeycomb, and lack of concrete compaction in segments

5.5.1.3 Defects due to lack of cover

The placing of the preassembled segment reinforcement within the formwork and the difficult access for inspection and location of spacers can lead to lack of cover in some locations.



Figure 5-9: Lack of cover of reinforcement

5.5.1.4 Repairing of segment defects

The segments should be repaired prior to their erection, as long as the defects do not affect the contact faces between segments that should be match cast. These faces cannot be repaired prior to erection and prestressing of the segment without risking further damage.

After cleaning the affected area, conventional concrete repairing techniques, such as the use of micro concrete or grout mixes with low shrinkage, are common repairing strategies.

5.5.2 Fabrication defects during erection

When moving segments for storage or erection, damage such as chipping or breaking of shear keys and local cracking can arise from improper manipulation of the segment, if temporary lifting points and support points are not adequately designed or detailed.

As in the previous case, repairs should never affect the contact faces between segments and, depending on the type of damage, can be made after the segment erection.

5.5.3 Repair works during PT works

5.5.3.1 Local duct bursting

During the prestressing and grouting of segments, some additional problems can arise if the reinforcement detailing and duct placement within the segment present defects. The most common problems are sharp bends in the ducts that induce excessive local pull-out forces or improper reinforcement detailing to take into account the forces around the blister area.



Figure 5-10: Pull out concrete due to abnormal duct placement

5.5.3.2 Duct blockages

Duct blockages can occur due to imperfect application of the compressible seal or alternative systems to provide duct tightness at the joint when the epoxy resin is applied, or due to leakage from an adjacent duct that has already been stressed and poured. This can lead to unusable ducts or complex repair. Hence, this problem is best avoided as it is often impractical to remove blockages. Provision of additional spare ducts is also recommended if space allows.

5.5.3.3 Cracking due to excessive local stresses at the joint interface

As indicated in 5.6.3.6, thermal effects may cause bowing in the segments, leading to an uneven distribution of stresses at the segment interface. Local defects in the segment can also produce additional local stresses between segments that can then produce longitudinal cracking in the segments after prestressing.



Figure 5-11: Cracking due to local stresses at the segment interface

5.6 Fabrication yard

5.6.1 General principles

5.6.1.1 Introduction

The prefabrication yard is generally located close to the bridge to minimise transport. However, due to site or economic constraints, it may be located away from the construction site. In this case, transport may become a parameter that will influence the design (particularly the weight and dimensions of the segment).

5.6.1.2 Prefabrication process

“Match casting” is now the most commonly used system for the fabrication of PSB segments, and can be considered as the default method for PSB. If a wet joint detail is compulsory because of local requirements, a PSB loses some of its advantages.

In match casting each segment is individually cast using the adjacent segments as face forms, hence, this system guarantees a precise connection between adjacent segments once located in their final position. A span or a cantilever is incrementally cast segment by segment. Two primary methods exist for the match-casting system: long-line and short-line.

In the long-line method, the segments stay on the casting bed after concreting and the formwork is moved forward to cast the next segment. In the short-line method, it is the opposite, the formwork is at a fixed position and the completed segment is moved from the casting bed to make room for the next one. This is the main difference between the methods. Paradoxically, practical experience suggests that it is more efficient to move the segments than the forms, even if segments are much heavier. It is a longer operation to move the external forms, the internal form and the bulkhead from one segment position to the next one and then adjust them, than to slide a segment to the match-cast position. Moving and adjusting forms requires more labour hours than sliding a completed segment.

The advantages and disadvantages of both solutions are shown in Table 5-1.

Whilst the choice of casting method is the contractor’s decision, the simplicity of a long-line system vs. the flexibility of a short-line system and the effect of each of these methods on design should be taken into account when considering alignment and structural configuration.

Table 5-1: Advantages and disadvantages of long-line vs. short-line match casting

Long-line		Short-line	
Advantages	Disadvantages	Advantages	Disadvantages
Simple – fewer skilled workers needed.			More complex – more skilled workers needed.
	Not flexible for changes in segments	Flexible for changes in segments	
Good for multiple straight bridges	Cannot change plan curvature easily	Flexible for changes in plan alignment	
	A minimum space requirement of half a span is needed in the casting yard	Smaller space required in casting yard	
	As a minimum, a half span is cast so that geometry errors do not accumulate.		Sensitive to accumulation of geometry errors, so high-quality workmanship needed.

Long-line method

This method of match casting utilises a large formwork system that can take the shape of the entire double cantilever length over a pier (or “T” shape), including camber, which compensates for deflections of the installed segments. Similarly to the span-by-span method, a full span is cast. It should be noted that for a long-line system in balanced cantilever, the pier segment is often cast in-situ to enable control of the erection geometry.

This method’s primary advantage is its independence from the constant need for geometrical control adjustments. The primary disadvantage, however, is the need for a large area to locate the forming systems.

Unless the bridge is long, the reuse of formwork is limited compared to short-line.

Another inherent disadvantage of the long-line match-casting method is in regards to the interface between each of the cantilevers directly over the pier, or segment zero. If each side of the “T” is fabricated separately, the end faces of each cantilever (looking at the pier) are formed without match casting. This requires that the linking segment over the pier be cast in-situ. To construct this in-situ segment, geometrical data must first be gathered from each of the initial segments to each side of the pier. This method and the associated tolerances or uncertainties of data gathering may lead to unacceptable final positions of the cantilever extremities.

If, on the contrary, the double cantilever over the pier is formed together as a single unit, then segment zero will be enclosed by the initial segments of each cantilever. Due to this pinching effect, once the forms are stripped, the removal of this central segment can prove to be very difficult. To alleviate this problem, segment zero should be formed so as to have a trapezoidal section.

A problem encountered in the past has been damage to the segments during the striking operation. In spite of using products that facilitate the separation of each segment, serious problems have sometimes occurred.

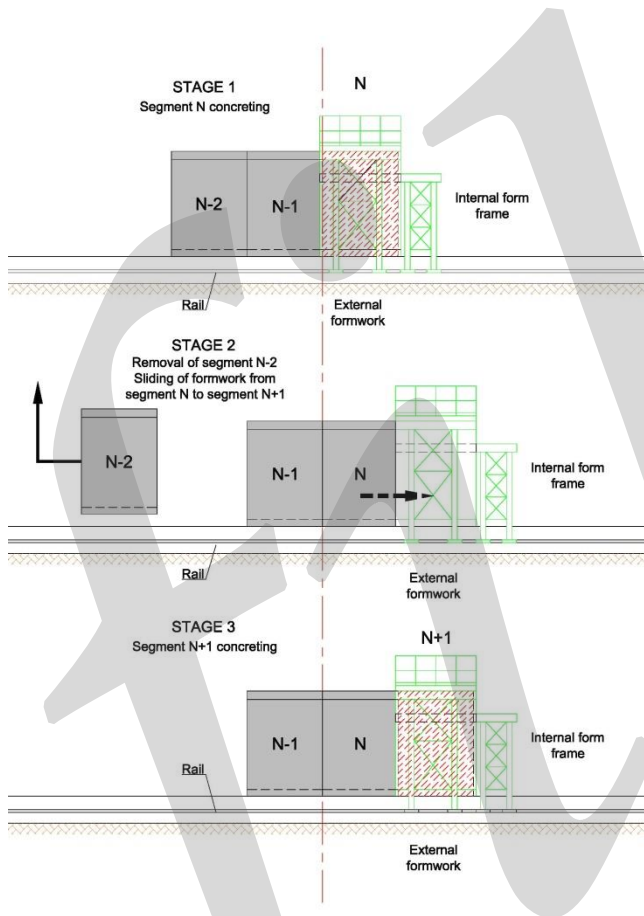


Figure 5-12: Long-line match casting

Short-line method

In this method, only one segment is cast at a time. The segment being fabricated has one face formed by the previous segments, thereby insuring that the joint interface is precise. Furthermore, the fabrication equipment allows for movement of the previous segment in order to establish a relative position. This unique positioning of each segment, throughout fabrication, is critical in guaranteeing that the final geometrical shape of the installed segments over the pier is correct. To do this, geometrical modelling calculations of the final

positions of the segments must be performed; then the segments must be positioned accordingly in the mechanism. Geometrical control system is the key issue for the short-line method, so a double checking (redundant) system must be implemented.

This permanent geometrical control must be done by experienced qualified personnel. Currently, commercially available bespoke computer programs are used to perform these operations, whereas in the past, only proprietary software existed. If for geographical or logistical reasons, skilled personnel are not available, it is possible to control the performance and evolution of the segment fabrication process from a distance. This is not preferred as it breaks the link between construction and design.

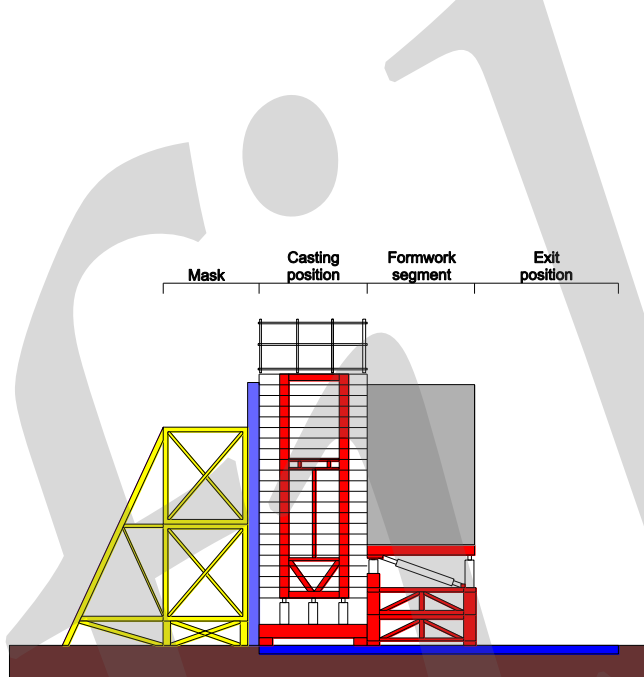


Figure 5-13: Scheme of precast unit for short-line method

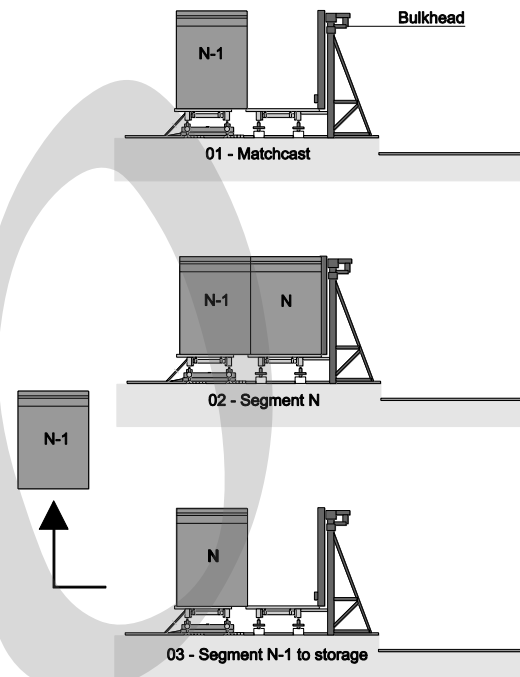


Figure 5-14: Short-line method

5.6.2 Yard organisation

5.6.2.1 General arrangement

A prefabrication yard consists of the following main facilities:

- production area including the formwork;
- segment storage area;
- rebar yard;
- concrete batching plant, if concrete is produced on the yard; and
- additional areas for workshops, stores and ancillary facilities.

The core of the prefabrication yard is the production area. The other areas—rebar yard, batching plant and segment storage—are located as close as possible to it in order to limit handling and transportation at the yard.



Figure 5-15: Production area with short-line bed and storage area

The total area required to house the facilities depends on the following parameters:

- type of formwork (long-line or short-line beds) and number of casting beds;
- rebar daily production; and
- number of segments to be stored and type of handling equipment (travelling gantry crane or straddle carrier).

The layout of the yard is determined by the type of equipment used to lift the segments. If a gantry crane is used to cover all the required areas for segment production and storage, then the yard is organised in a line along the ground slabs of the gantry crane with the rebar yard and batching plant located on both sides. If using a gantry crane, the yard requires a long rectangular area that could be a few hundred metres long (see Figure 5-16). If using a straddle carrier, it is easier to deal with any shape area. Refer to section 5.6.2.4, which describes commonly used handling equipment.

Level and soil conditions are deciding factors for the choice of the prefabrication site. Earthworks and deep foundations, if any, may increase drastically the installation costs of the yard, particularly if piled crane beams are required for the gantry crane.

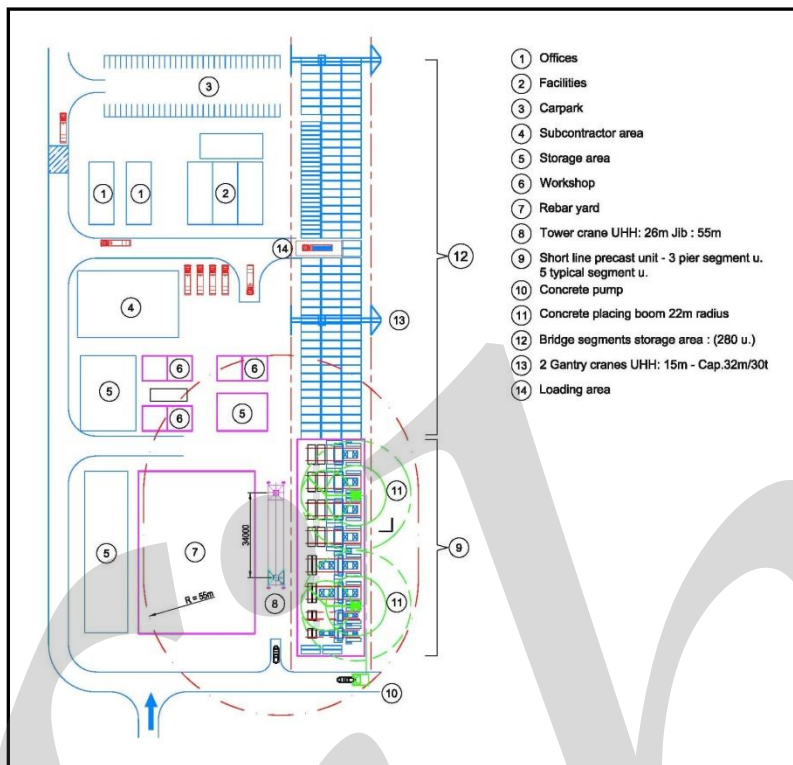


Figure 5-1 6: Illustrative prefabrication yard with gantry crane

If the prefabrication yard is a permanent facility, the production area could be covered by sheds. In a temporary yard, the production area could be open or covered by light shelters according to the weather conditions.

The yard is provided with circulation roads for rebar, concrete and segment transportation. The road construction (thickness and type of course) is adapted to both the foreseen traffic and the subsoil conditions.



Figure 5-17: Temporary prefabrication yard

5.6.2.2 Concrete supply chain

Concrete can be supplied from external batching plants. However, the volume of concrete for a segmental bridge is usually significant overall so there are benefits to an on-site batching place if space permits. Moreover, experience suggests that the concrete from on-site plant is more consistent as it is quality controlled and also only provides a limited range of mixes. The concrete supply is a constraint to casting speed and quantity, so on-site batching gives more programme control.

5.6.2.3 Storage area

The segment storage has two purposes:

- The segments are required be kept in the storage area until concrete reaches the required strength for erection.
- Segment prefabrication and segment erection activities have different speeds. To make these as independent as possible, a quantity of segments is usually stored in order to give programme margins in both activities. Usually the most convenient location to store these segments is the prefabrication yard.

The storage capacity is sized to meet both requirements.

The storage is arranged so as to avoid stress and deformation of the segments.

Thus, each segment is placed on three temporary bearings positioned under the segment webs so as to stay in isostatic conditions. Temporary shims—two under a web and one under the other one—are centred on the web axis. This is the typical arrangement for segments up to 4 m in length. For a longer segment, four elastomeric shims could be considered. The shims are sized to limit the local pressure on concrete.

Typically, the span segments are stacked on two levels. The shim arrangements are identical at both levels. The pier and end segments, which are shorter and often heavier, are stored on one level for stability.

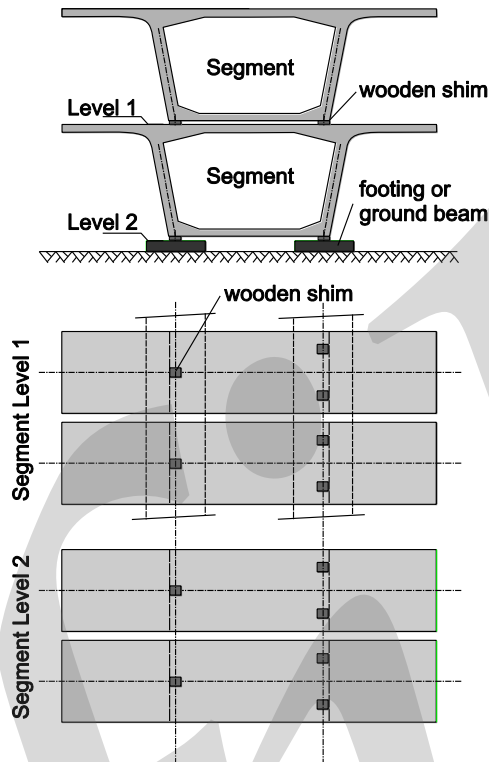


Figure 5-18: Principle of segment stacking

Stacking on three levels or more can be considered for segments with vertical webs. Segments with inclined webs should be checked, as stacking will induce bending moments in the webs and the top slab of the segment at the bottom level.

Segments with inclined webs and varying heights require specific arrangement as web spacing can be different for each segment.

The N segments of a half cantilever could be stacked on two levels within these conditions:

- segment N on segment 1,
- segment $N-1$ on segment 2,
- segment $N-2$ on segment 3, etc.

This reduces the risk of double handling. Handling at early age shall be also limited. The segments shall be stored in dedicated positions in order to properly manage the storage and avoid repeated segment handling. All the segments of a span or a cantilever shall be stored together according to transportation order. This is key to avoid multiple handling of segments.



Figure 5-19: Stacked segments in a storage area

The foundations for a stacking yard consist of longitudinal ground beams or isolated footings, depending on whether the handling system is by travelling gantry crane or straddle carrier. The settlement of the foundations is not critical as the segments are laid on isostatic supports. The stability of the stacked segment piles limits the settlement. Toppling of a stacked segment pile is the main cause of disorder in a storage area.

Depending on the type of segment—constant height or varying height with inclined webs—the foundations are arranged in parallel or non-parallel rows.

Robust working procedures are required to ensure safe conditions for site personnel in the storage area during all handling operations, especially when working on stacked segments to fix the lifting devices.

5.6.2.4 Handling and transport

As for storage, the aim during handling and transportation is to avoid segment deformation.

Equipment and plants used commonly for handling and transport are:

- travelling gantry cranes,
- straddle carriers,
- tower cranes,
- low bed trailers, and
- segment carriers.

The travelling gantry crane is the most commonly used equipment for handling segments on a prefabrication yard as it is robust and easy to maintain. Its movements are slow and accurate so that the spacing between stored pieces can be minimal (about 25 cm) without any risk of damaging the segments and the whole storage area is efficiently used. Its capacity to span above the production area makes it easier to arrange. It is used only for

segment handling as it is very convenient for that purpose. Additional lifting equipment shall be provided for smaller loads. The allowable longitudinal slope along crane tracks is about 1%, hence nearly horizontal ground is required, otherwise site preparation, excavation and backfill works need to be done. Movements are slow (longitudinal speed about 20 m/min). Gantry crane tracks above 300 m long are not recommended as this could significantly lengthen the segment handling cycle and make storage operations become critical. It may be difficult to increase the storage capacity during the project, if additional distance has not been provided to lengthen the crane tracks. If segments are transported on a barge, it is very convenient to load them using a gantry crane by adding a temporary jetty at the end of the storage area.



Figure 5-20: Travelling gantry crane

Straddle carriers may be used to handle and store segments. Their main advantage is the ability to manoeuvre and to climb ramps, which allows the use of almost any storage area regardless of the shape in plan. The carrier tracks usually need to be reinforced as the usual tyre pressure is about 1 MPa. The overall dimensions of the carrier can make it more difficult to use as they impose additional requirements on the construction methodology. The production area and the formwork may need to be adapted to give access to the carrier. The storage area may need to be widened to accommodate the access tracks. Depending on the carrier, it may be impossible to stack segments on two levels. Thus, the storage area per segment may be at least twice as wide as those required using a gantry crane. A straddle carrier is convenient to install an additional storage area on the erection site as no temporary works are required for its operation, especially when there is room available and ground conditions good.



Figure 5-21: Straddle carrier

Tower cranes are used to service the production area and the rebar yard. Their main task is to lift the prefixed rebar cages from templates or jigs and to place them in the formwork. As this is the heaviest load to lift, the capacity of the tower crane is rated accordingly. Auxiliary tasks are lifting small parts of formwork, unloading rebar trailers and eventually placing concrete. A tower crane is very rarely suitable for lifting concrete segments.



Figure 5-22: Prefixed rebar cage lifted by a crane, to be positioned in the cell

If the erection site is far from the prefabrication yard, the segments may be transported by road using low-bed trailers. The segments lay on the trailer on three bearings which are centred along the segment web axis, as on the storage area. The weight and dimensions of the segment may be limited or reduced to respect road regulations or clearances along the journey to the erection site, typically 2.5 m or 3.0 m wide.

If the prefabrication yard is close to the erection site, the segments may be transported using the site tracks, which means fewer limitations on the weight and dimensions of the segments. The segments may be transported on a purpose-made carrier. A self-propelled modular transporter (SMPT) may also be used. The speed of these carriers is very low: usually 5 to 10 km/h. Thus, the number of carriers shall be optimised so that segment transport is not critical for the erection operations. The carrier access tracks on the site may need to be reinforced as the usual tyre pressure is about 1 MPa and the ramp slopes shall be limited to 10% maximum.



Figure 5-23: Purpose-made segment carrier

5.6.3 Prefabrication yard

The segments are cast one by one, generally following the order of erection in the bridge. For spans erected using the SSM method, the segments are cast starting from the first pier segment to the next one. For cantilevers, the segments are cast from the pier segment in both directions to the middle-span stitches. In some cases of SBS, when different cells are used to precast the segments of one span (one for pier segments and one for typical segments), other schemes can be used to smooth the prefabrication time schedule. It is possible to begin the prefabrication of a span with one of the typical segments, so that the pier segments are precast at the same time as the typical segments.

5.6.3.1 Principle of long- and short-line casting

Using the long-line method, the construction sequences of a standard segment “n” are as follows:

- the external side forms are stripped from segment “n-1” and moved forward to the next segment “n”;
- the internal core form is stripped;
- the bulkhead is stripped and moved forward to the next segment “n”;
- the forms are cleaned and sprayed with stripping oil;
- the prefixed rebar cage for segment “n” is lifted and placed into the bed;
- the bulkhead is adjusted;
- the internal core form is moved forward to the next segment “n”, adjusted and closed;
- the external side forms are adjusted and closed; and
- the segment “n” is concreted in one pour from the bottom slab to the top slab.



Figure 5-24: Preparation of deck bottom formwork in long-line



Figure 5-25: Initial segment and bottom slab cast in long-line



Figure 5-26: Segments cast on a long-line bed

Using the short-line method, the construction sequences of a standard segment “n” are as follows:

- the external side forms are stripped;
- the internal core form is stripped and moved back on its translation gantry;
- the match-cast segment “n-2” is handled to storage;
- the segment “n-1” is slid on his bottom form to the match-cast position;
- a new bottom form is lifted and placed into the cell;
- all the forms are cleaned and sprayed with stripping oil;
- a soapy demoulding agent can be applied to ease release of the cast segment;
- the prefixed rebar cage for segment “n” is lifted and placed into the cell;
- the internal core form is moved forward on its translation gantry into the cell and closed;
- the external side forms are closed;
- the match-cast segment “n-1” is adjusted to get the theoretical casting coordinates of segment “n”;
- then, the match-cast segment “n-1” is blocked by use of struts;
- the segment “n” is concreted in one pour from the bottom slab to the top slab; and
- if necessary, the segment is cured and protected.



Figure 5-27: Segments cast on a short-line precast yard

A short-line precasting cell is erected at a fixed location and provided with two or three segment positions:

- the segment to be cast (segment n);
- the match-cast segment (n-1);
- a temporary storage position for the segment (n-2) to be lifted and handled to storage area; this position is optional.

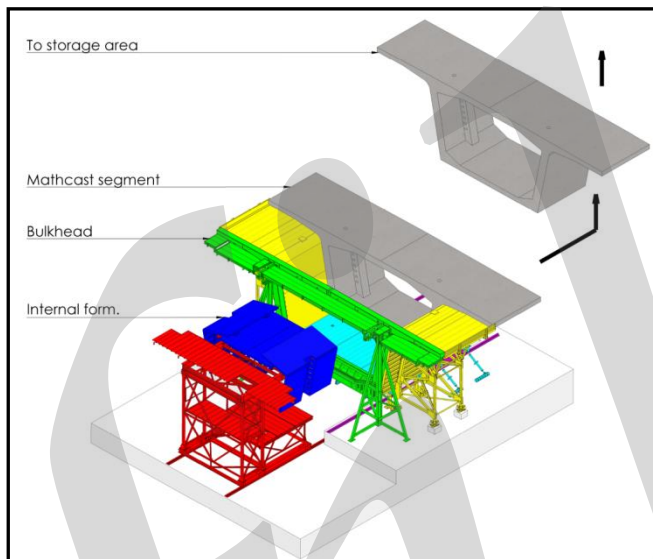


Figure 5-28: Short-line bed principle

Each cell consists of:

- two units of side formworks forming the external faces of the webs and the top slab wings;
- one unit of fixed bulkhead with support frame, provided with male shear keys;
- three units of bottom forms: one for the segment to be cast, another for the match-cast segment and a complementary one for the stored segment;
- one unit of XYZ chariot: a movable frame supporting the previously cast segment on its bottom form. This frame is provided with horizontal and vertical jacks allowing the accurate positioning of the segment. It is used also to transfer the completed segment on its bottom form to the match-cast position, avoiding lifting at early age;
- one unit of internal core form, which allows shuttering the inner faces of the webs, the bottom face of the upper slab and the vertical faces of the blisters;
- one unit of translation gantry supporting the internal core form.

The concrete pressure on vertical forms is balanced by means of tie bars linking both side forms under the bottom form and eventually at the upper part of the webs.

The cells are usually purpose-made and designed for multiple reuses and short cycles (one day for a span segment). Thus, all the parts are made of steel in order to resist heavy wear. The skin of the external and bottom forms shall be calculated so as to avoid buckling on these faces (5 mm thick steel is adequate). For internal forms, a 4 mm thick steel skin might be suitable. Following that, hundreds of reuses can be foreseen without noticeable damages of form skins. Wooden forms could be used to shutter inner surfaces of special segments (diaphragms or end segments) if there is variation and few repetitive uses.

The internal core form is a complex one. It needs to fold or retract to allow for stripping and removal without dismantling; hence, the shape and location of the blisters and ribs shall be adapted to be regular and suitable for the formwork.

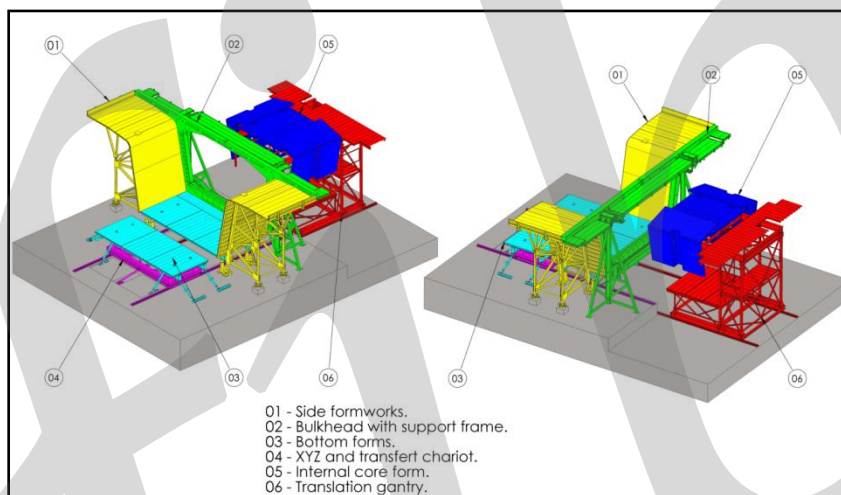


Figure 5-29: Short-line precasting bed

All movable parts and forms are handled by means of hydraulic and mechanical jacks. The level of mechanisation of the cell is adapted to the required productivity (concreting cycle) and the potential to amortise the equipment (number of segments).

The short-line bed is designed according to the following principles:

- Movement and adjustment of the forms are minimised. The geometry of the segment is given by adjusting the match-cast segment (n-1) in the three directions. It is then not necessary to adjust the side and internal forms, unless the segment geometry itself changes. They are closed in a fixed position at each segment. The stripping displacements of the forms are limited to a few centimetres.
- The bulkhead is the reference for geometry control. It is in a fixed position and shall be as rigid as possible. Accordingly, its support frame shall be also stiffened in order to limit the displacement during concreting. The adjustment of the bulkhead is limited to a few different fixed positions, which may be required (different lengths

of segment, short radius curves). This is key to successful short-line casting geometry control.

The precasting cells are provided with working platforms, handrails and access stairs to ensure safe working conditions and improve productivity.

The cell shall be founded on stiff foundations in order to avoid movements of the forms, inducing deformations of the segment (n) during concreting and relative movements of the match-cast segment (n-1) voiding the geometry-control adjustment. The foundation is subject to a vertical loading-unloading cycle during casting operations on a daily basis. There are two positions for a segment in a cell, which are alternatively loaded and unloaded. The design criteria for the foundations shall limit the relative settlement of the forms (1 mm of vertical relative settlement of the side forms) and the opening of the match-cast joint (1 mm of horizontal relative movement between bulkhead and match-cast segment at top slab level). A shallow foundation—a 400 mm thick concrete slab linking the ground bearings of the forms—and the match-cast segment may usually be used. On soft ground, deep foundations such as piles may be required.

After completion, a segment is transported to the storage area in two stages:

- The side forms and internal core form are stripped and removed; the completed segment is then transferred on its bottom form to the match-cast position without lifting it. At this stage, the concrete shall have the minimum compressive strength such that the top slab cantilevers are self-supporting. Usually 15 MPa is sufficient. For long top slab cantilevers, calculations shall be performed to define the required strength. Commonly, the segments are cast in one shift on a daily cycle. Thus, stripping of the form occurs 16 hours after the end of concreting.
- After another casting cycle, the segment in match-cast position is lifted from its bottom form and stacked in the storage area. Lifting occurs about 40 hours (16+24) after the end of concreting. At this stage, the segment has usually reached the required strength to be lifted.



Figure 5-30: Short-line precasting bed

There are three methods to determine the compressive strength of concrete at early age:

- Concrete samples. Currently the most-used method to determine the compressive strength at early age of the concrete in the structure consists of making informative concrete samples (cylinders or cubes) stored in conditions similar to those of the structure and crushing them to determine their compressive strengths at selected early ages. However, this method is only reliable if the samples are really representative of the segment concrete.
- Sclerometer (rebound hammer). A rebound hammer test can only measure the hardness of the concrete surface. The main defect of this method is that the measurement is superficial, whereas the compressive strength at early age is required inside the concrete (for example at the junction of the web and top slab cantilever).
- Maturity method. The maturity method is the state-of-the-art to verify concrete behaviour. It consists, based on the knowledge of some characteristics of the concrete mix (thanks to preliminary calibration tests) and on the recording of the temperature evolution within the concrete, of deriving the degree of concrete hydration corresponding to its hardening. It includes both the coupled effects of temperature and time on the concrete hardening process. Practically, the temperature of segment concrete is recorded by use of sensors embedded in concrete at the critical locations in the structure that will experience high levels of stress at a given maturity (when formwork stripping).

A cell is designed to cast segments with identical length, as the bulkhead is in a fixed position and the internal core form has a unique length. Practically, it is possible to vary the lengths of the segments in a range of 15-60 cm by adjusting the distance between the bulkhead and the match-cast segment longitudinally. For longer variations, the bulkhead shall be placed and fixed in a new position. It is not recommended to move the bulkhead; instead a movable external form accommodating the plan curvature of the bridge can be used.

5.6.3.2 Reinforcement prefabrication

The reinforcement of the segment is fixed in a dedicated yard close to the production area. The cut and bent reinforcement is delivered on site and stored on this yard. The reinforcement is prefixed using jigs.



Figure 5-31: Reinforcement cage prefixed in a jig

Prefixing of reinforcement in cages is required for two main reasons:

- **Productivity.** Typical segments are usually cast in a one day cycle. This can only be achieved if rebar placing is as short as possible.
- **Quality.** Steel fixing in the precasting cells shall be avoided in order to preserve the form skins from scratches and damage.

The rebar is assembled and fixed by element: bottom slab, webs and top slab, using separate templates. Then the elements are placed into a jig to complete the cages. The prestressing tendon steel ducts are placed in the jigs over welded supports.

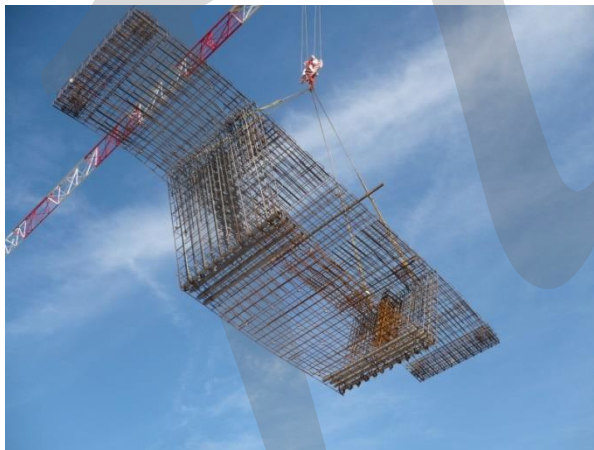


Figure 5-32: Prefixed rebar cage lifted by use of a tower crane

The completed rebar cages may be placed into the cells by means of a tower crane. The deformations of the cages shall be limited to avoid difficult modifications and adjustments in the cell while placing.

Measures to limit deformations are as follows:

- the prefixed cages are not stacked;
- the prefixed cages are handled by use of spreader beams with multiple hangers; and
- the cages are not transported on site by use of trailer. The jigs are located as close as possible to the production area in order to place the cages directly in the cells with the crane.

The jigs are purpose-made structural steel frames. Before use, the geometry of the jigs shall be verified to check the accurate fitting with the segment formwork. There are two types of jigs with horizontal axis or vertical axis. The vertical axis type improves the productivity of the fixing, but requires a tilting frame.

5.6.3.3 Special segments

Within a span, some segments have a non-standard geometry. This can be due to the use of external prestressing, diaphragm walls and transverse beams that are required to anchor and deviate the tendons, or blisters and anchorages for internal tendons. These have to be incorporated in the segments.

These special segments are of different types:

- pier segments,
- end segments, and
- deviator or diaphragm segments.

The pier and end segments have complex and thick diaphragms, flanges and webs. They are heavier than the span segments and, for that reason, usually divided into two or three shorter segments to maintain an even lift weight. As their shape can be very different from that of the span segments, they require special internal formwork and are cast in dedicated casting beds. In order to reduce their weight during handling and erection, it could be possible to complete them in-situ by a second stage of concreting.



Figure 5-33: Short-line bed dedicated to pier and end segments

Deviator segments are span segments including a diaphragm wall and a bottom transverse beam in which steel tubes are embedded for external tendon deviation. These segments are cast in the same cell as the span segments and are often concreted in two stages. The webs, top slab, bottom slab and transverse beam are concreted in the cell so that the internal core form is not modified and the segment is cast within the usual cycle. The diaphragm wall is poured later using additional forms. The alternative is to use inserts into a standard form for deviators and blisters.



Figure 5-34: Second stage of concreting of a diaphragm wall segment

5.6.3.4 Longitudinal post-tensioning ducts

In precast segmental bridges, the internal ducts for post-tensioning tendons are split up into as many parts as there are segments. It may be difficult to make all of these parts have a smooth and accurate geometry. The internal ducts are made with segments of steel or HDPE ducts, which are fixed in the rebar cages on the rebar yard. Once the rebar cage is placed in the casting cell, there are several solutions to opt for:

- Using inflated hoses inserted into the ducts to maintain their shape during concrete pouring. In the short-line bed, the bulkhead is provided with holes that coincide squarely with all the positions of the internal ducts. The inflated hoses are longer than the segments so that they can be maintained through the bulkhead at one end and into the match-cast segment ducts at the other end. The hoses, which are rubber tubes, are pressurised during concrete pouring in order to hold the ducts in position and to avoid buoyancy and crushing under concrete pressure.



Figure 5-35: Inflated hoses

- Using HDPE pipes, acting as forms, inserted in the ducts, with metal rings around them, placed at the joint before casting the following segment. The inner plastic couplers are used to ensure proper geometry of the ducts at the joint during pouring of the concrete; meanwhile the metal rings, removed afterwards, have left an imprint at the joint to be used for placing rubber rings. These devices will guarantee the water-tightness after applying prestressing. HDPE pipes and should be flexible enough to adapt to small differential settlements between segments, especially in order to avoid cracks in flanges due to a very rigid device.



Figure 5-36: Plastic couplers and washers placed at the joint before pouring the concrete

5.6.3.5 Separation of segments

After completion of a segment, the forms are stripped and both segments are separated carefully to avoid damaging the shear keys in the match-cast joint or edges of the segment. Before concreting, the joint face of the match-cast segment is coated with a stripping agent, preferably a soapy wax that can be easily washed away. After concreting, the joint shall be

opened progressively from top to bottom by rotating the match-cast segment using hydraulic jacks. Given that, the short-line method is more convenient than the long-line method as bottom forms are adjustable by use of jacks.

5.6.3.6 Bowing of precast segments

Adjacent segments must match perfectly to guarantee the load transfer of both shear and normal forces through the unreinforced joint between two precast elements. Thus, match-casting must be used as a segment production technique, where the new segment is cast against the previous one so that the faces in contact match perfectly. Nevertheless, the perfect fitting of two adjacent elements can be prevented by the so-called bowing effect. The heat of hydration and the ambient temperatures result in plan deformation and bow-shaped segments.

Figure 5-37 shows in plan the deformation of the top slab of a bridge deck slab produced by the short-line match-casting method. The bowing of both segments is actually formed by the thermal gradients in the match-cast segments. The previously finished segment is used as the removable bulkhead for a new segment and, due to a thermal gradient in the old segment, a curved face occurs between the two adjacent segments. After cooling to ambient temperature, the old segment returns to its original shape but the new segment has a permanent curvature.

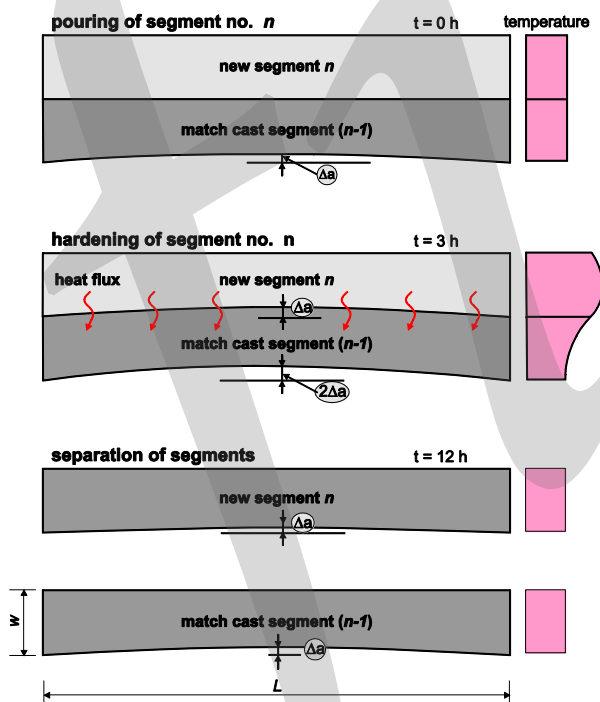


Figure 5-37: Segment production by short-line match casting

The main factors influencing the bowing effect are as follows.

- Width-to-length ratio (w/L) of a segment:

Special attention is advised if the slenderness ratio of a match-cast segment is greater than $w/L \approx 6$, particularly when the spacing of the webs is relatively low (i.e. small box with wide wings). This was the case in the San Antonio “Y” project, where it was reported for the first time that bowing caused problems in the erection process of precast segments effects; refer also to [Roberts et al. 1993].

- Heat of hydration:

This depends on the type and the amount of cement in the concrete mixture. Concrete with low heat of hydration should be used to reduce the bowing effect. Cooling of concrete is an appropriate method to limit the bowing effect in regions with high ambient temperature. However, by choosing an optimised concrete mix design it may also be possible to limit the bowing effect to uncritical values without the need for additional measures, even if there is an unfavourable w/L ratio, as was proven with the Bang Na Expressway project [Abendeh et al. 2006, Fischer 1998].

- Heat flow at the segment surfaces:

This depends on the heat transfer properties of the formwork, the ambient temperature and the wind velocity. Thus, curing blankets and plastic sheeting are recommended in cold and windy climates. Also, proper curing of both segments of match cast can reduce the problem. In a cold environment, the concreting should start only when the ambient temperature starts to rise. A fast setting of concrete can be advantageous as the heat flow in the match-cast segment is limited. For slender segments the heat of hydration should be measured to predict the thermal gradient before segment production. The creep of concrete plays an important role in closing the gap between adjacent segments.

5.7 Span-by-span erection

5.7.1 Introduction

5.7.1.1 General principle – description

The precast segmental span-by-span method (SSM) is defined as a bridge construction where an entire span consisting of precast segments (pier to pier) is erected with special equipment, namely overhead girder, underslung girder or falsework, then becomes self-supporting by applying post-tensioning.

The span-by-span method was developed and is most efficient for medium to long viaducts with relatively short and similar spans (30 – 50 m), as the dead load of the equipment becomes larger and thus less cost efficient for longer spans. SSM designed bridges usually

have a constant depth due to short spans, have a relatively large radius in plan, and have construction joints if the continuous spans are close to piers.

During the non-self-supporting stage of the span erection, segments are individually supported either by hangers connecting the deck top slab and overhead truss for the overhead girder, by point support below the segment wings (part of deck cantilevering out from the web) for underslung girder, or by point supports below the segment soffit for underslung girder and falsework.

Segments are usually epoxy jointed and some temporary post-tensioning is always applied to pull all segments together, which will eventually maintain minimum compression over the joint to squeeze out the glue and maintain a good geometry. The structure is then set with final permanent post-tensioning and the span load is transferred to the pier structure.

An SSM constructed bridge is one the most simple and cost effective types of segmental erection system. In most cases, this technique allows for traffic adjacent to and under the viaduct to be maintained except during movement of loads during lift, or movement of segments to or onto the truss.



Figure 5-38: Overhead girder over live train tracks

5.7.1.2 Typical structures

Typical bridge structures appropriate for SSM construction are shown in Figure 5-39.

It is noted that there is a further typical arrangement with the SSM, similar to that done for an in-situ continuous bridge, with the first span being built beyond the first pier to approximately 20% to 25% of the second span. The subsequent spans are also built with a cantilever. This is less frequently used as the deck geometry and load make the design of the erection equipment more complex.

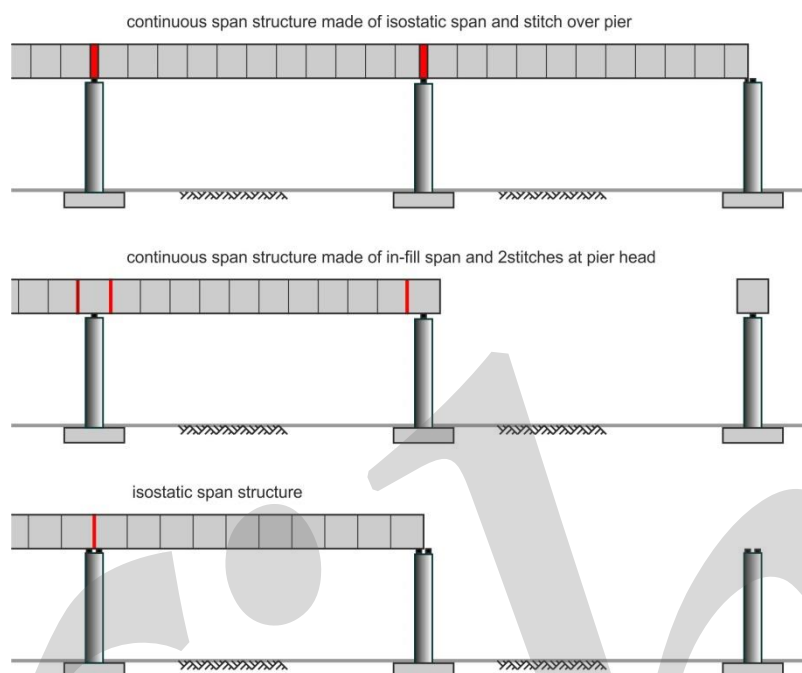


Figure 5-39: Typical bridge structures suitable for the SSM

5.7.2 Pier head geometry and arrangement

For SSM, the pier and pier head setup should allow for:

- access and working platform,
- support for erection equipment,
 - lateral side bracket for underslung girder and/or rolling device set on the pier head itself (in the case of a single T-shape pier head carrying multiple segmental decks). Fixing of side brackets to the pier is often made with high tensile stress bars set either on the side or through the pier head,
 - front support for overhead girder is set on:
 - bracket system in front of the pier, which could be set directly with the overhead girder or placed in advance by crane;
 - or heavy-duty temporary props (which is less common as additional preparatory ground works may be needed);
 - or directly over the pier if it is large enough to accommodate the length of pier segment and the front support;
 - or, in the case of a continuous span where the pier segment is already erected (usually cast in-situ), directly over the pier head.

It should be noted that in all cases the front support is temporarily fixed for uplift as well as downwards loads ('nailed') on the permanent works.

- temporary span jacks for isostatic spans, and
- permanent bearing for non-monolithic pier heads.



Figure 5-40: Top left: support bracket for underslung girder; bottom left: front support of overhead girder above pier head; right: access and front support for overhead girder

5.7.3 Span erection sequences

With reference to section 4.3.14.3.1, two main deck arrangements for the SSM are usually considered for the span erection sequences: isostatic and continuous span structures.

5.7.3.1 Isostatic span erection sequences

The span is composed of two pier segments and a number of standard or field segments. These segments are erected with construction equipment such as an overhead launching girder, underslung girder or by crane on falsework.

Segments can be delivered on the ground or from the already constructed deck and picked up by the construction equipment.

Typical steps of construction:

- Erection equipment in position (overhead, underslung, falsework).
- Delivery of segment: each segment is picked up and individually supported by the erection equipment. Delivery can be done from the ground below, in front of the erection equipment, or from behind the erection equipment on the erected deck.

- Usually pier segment + 1st field segment are glued and stressed together with temporary post-tension so as to have better geometric precision during alignment/levelling.
- Erection (and gluing) of remaining segments which remain individually supported on the steel truss. Segments are pulled together with temporary prestressing. Each segment remains individually supported by the erection equipment during this operation.
- If the deck is supported on bearings, span jacks are usually set in contact between pier and pier segment. These span jacks will be monitored and operated to give the load transfer between the erection equipment and the pier. Span jacks are set on a sliding device to complete the operation of geometry control.
- Stress permanent post-tension tendons to make the span self-supporting. The load transfer between the equipment and the pier is made during this phase of post-tensioning. Special care must be taken to ensure that there is no distress to the permanent or temporary structures (see section 5.7.4).
- After completion of load transfer:
 - all connecting parts between erection equipment and each of the segments are released,
 - all the temporary post-tensioning is removed.
- Erection equipment is ready to be launched/reset on the next span.



Figure 5-41: Individually supported precast segment glued and stressed with temporary post-tension (PT strands in this configuration)

The typical cycle is summarised in Table 5-2.

Table 5-2: Typical cycle for ISO span erected with span-by-span method using overhead or underslung girder

	Shift 1	Shift 2	Shift 3
Erection equipment move span N to N+1			
Erection segment			
Gluing and temporary post-tension			
Permanent post-tension + load transfer			

Note 1: “Shift” represents the team working either during daytime or nighttime, typically 10-12 hours.

Note 2: The number of “shifts” depends essentially on the erection equipment and number of field segments.

Depending on site conditions and equipment, the overall time cycle could be as small as 1.5 shifts to 4 to 5 shifts.



Figure 5-42: Isostatic span erected by underslung girder

5.7.3.2 Continuous spans erection sequences

For a continuous span, three typical structural layouts are considered (refer to Figure 5-42):

- isostatic span made continuous by stitching over the pier;
- infill span between already-erected pier heads (two concrete stitches at the pier head); and
- a typical span length with a cantilever (typically 20% of the span length, L) beyond the pier.

Isostatic span with continuity by stitching over the pier

All construction steps described in 5.7.3.1 apply.

Additional steps:

- cast stitch between span at the pier location, and
- install and stress the continuity post-tension cables.

The critical path of this construction method remains in the construction of the isostatic spans as a team that is independent from the erection team usually carries out the operations of stitching and setting out the continuity post-tensioning.

Infill span between already erected pier heads (two concrete stitches at the pier head)

The span is composed of pier segments already erected by an ahead working team (precast or in-situ) and a number of field segments. Those field segments are erected with construction equipment such as an overhead launching girder, underslung girder or by crane on falsework.

Segments can be delivered on the ground or from the already constructed deck and picked up by the construction equipment.

Typical steps of construction:

- Position the erection equipment (overhead, underslung, falsework).
- Delivery of segment; each segment is picked up and individually supported by the erection equipment.
- The first two field segments are glued and stressed together with temporary post-tensioning in order to have better precision during alignment/levelling when aligning this block, which is then secured against the fixed pier head.
- Erection (and gluing) of remaining segments, which remain individually hung/supported on the steel truss. Segments are pulled together with temporary prestressing.
- Stitches (150 mm to 250 mm) between in-fill span and pier head are closed with formwork and poured with concrete.
- Achieve concrete maturity.
- Install and stress post-tensioning tendons to make the span self-supported. During this operation of stressing, load transfer occurs. Special care must be taken to ensure that there is no distress to the permanent or temporary structures.

- After completion of load transfer:
 - all connecting parts between erection equipment are released and the relocation of the erection equipment can start; and
 - all the temporary PT is removed.



Figure 5-43: Continuous span erected as in-fill span

The typical cycle is summarised in Table 5-3.

Table 5-3: Typical cycle for “infill” span erected with span-by-span method using overhead or underslung girder

	Shift 1	Shift 2	Shift 3	Shift 4	Shift 5
Erection equipment move span N to N+1	■				
Segment placement		■			
Segment alignment, gluing and temporary post-tension			■		
Wet joint casting				■	
Curing (usually overnight)				■	
Permanent post-tension + load transfer					■

Note 1: “Shift” represents the team working either during daytime or nighttime, typically 10-12 hours.

Note 2: The number of “shifts” depends essentially on the erection equipment and number of field segments.

Typical span stretch with a 0.20 L long cantilever over the pier

Length of segments erected each stage is a typical span (length, L) with a 0.20 L long cantilever beyond the pier.

Ancillary equipment to deliver segments on site may be the same as for the aforementioned methods.

Typical stages of construction:

- Segments of first span plus 0.20 times the next span are erected.

- Each segment is glued and prestressed with the adjacent one, or with a set of already stitched segments.
- After this block is properly aligned and levelled, segments for the following stretch are placed. Usually that length is a typical span, in order to leave the stitch gap at a $0.20 L$ long cantilever over the pier.
- As per previous steps, each segment is glued and prestressed with the closer segments.
- Stitch gap between blocks pulled together with a temporary or percentage of final prestressing is poured with concrete.
- Wait for the concrete to achieve a minimum strength or pull against struts already placed in the stitch gap.
- Install and pull post-tension tendons to connect or lap with the section erected with the existing ones. During this operation of stressing, the same considerations as for the previous methods must be taken into account.



Figure 5-44: Typical span with a $0.20 L$ cantilever over the pier

5.7.4 Temporary post-tension during erection

For span-by-span construction the temporary prestress is similar to that used in cantilever construction and is used to fix geometry and squeeze out excess glue from the segment joint. It comprises high tensile strands or bars arranged to give a uniform stress distribution around the joint. However, the fundamental difference in this case from cantilever erection is that the amount of temporary prestress is much lower. As the self-weight of the segments is carried on the erection equipment, the temporary prestress only needs to maintain the minimum compression across the joint of approximately 0.2 MPa .

5.7.5 Erection equipment

There are several types of equipment used for of the span-by-span erection method. The three most common are:

- overhead girder,
- underslung girder; or
- falsework.

The choice of method is a choice for the design and construction teams. There may be overriding factors, such as previous experience and availability of existing equipment, but in general the choice of method is dependent on the following factors:

- segment delivery from below (ground) or behind (deck);
- influence of bridge alignment, ramps, single or twin decks;
- permanent works interface;
- local environment (urban, above water, over rail, over road);
- construction schedule and speed of erection;
- sustainability (reuse/adapt existing equipment);
- maximum capacity of the girder to jump related to span length;
- headroom below the deck to erect; and
- local safety regulations, risk assessment.

5.7.5.1 Overhead self-launching girder

This is essentially a temporary steel structure spanning two consecutive piers and designed to hang a whole span. To make it self-launching, the girder must be long enough and have a mechanised launching system to be able to move longitudinally to the next pier.

An overhead girder is usually a complex machine that needs to be used by trained and competent persons. Stability both during erection and movement are particularly important and the experience of the construction team in ensuring that there is robust fixity to the deck during erection and a safe method of launching are of utmost importance.

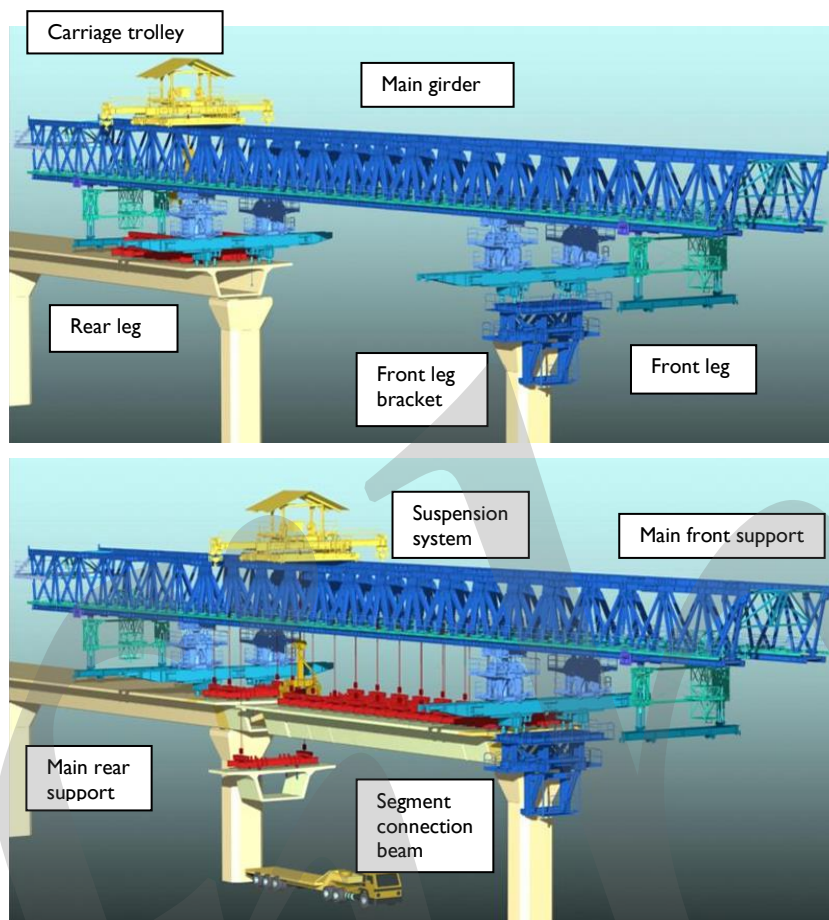


Figure 5-45: Description of main elements of an overhead girder for the span-by span erection method

The typical components of an overhead girder are:

- main girder (usually two but could be only one);
- carriage trolley able to pick up and handle segment, sometimes also used to launch the main girder;
- launching system (hydraulic jacks or chain + wire system, wire capstan system);
- main supports (front and rear);
- auxiliary supports/rear leg and front leg (extra supports required for movement);
- front leg bracket (in case there is no space over the pier head itself);
- hangers and segment connection beam able to handle the precast segment with the carriage trolley; and
- suspension system made by wire or high tensile bars to support the segment.



Figure 5-46: Overhead girder ready to start span erection for a U-shape precast segment



Figure 5-47: Infill span erected by overhead girder, twin decks



Figure 5-48: Temporary post tension suspension system for overhead girder



Figure 5-49: Transfer of front support during launching process

5.7.5.2 Design aspects for the erection equipment

As the full dead load (DL) of the span is supported by the girder, the reactions on the two main supports are in the range of 50% (DL steel girder + DL precast span). The main supports are usually over the pier or pier head but often with some eccentricity, especially for the rear support. It is therefore important to assess the construction load to determine interface and carry out checks on the permanent structure. It is noted that the checks on the permanent structure should include the loads during both segment erection and girder launching and should include dynamic as well as static effects.

Gantry dead load is mainly affected by issues such as the steel grade used, provision of delivery from the deck, suspension frame system and erection single of double deck.

The typical ratio between DL of overhead girder and span DL is between 0.5 and 0.7 and so it is not negligible in the substructure design checks.

5.7.5.3 Overhead gantries

Table 5-4: Advantages and disadvantages of overhead gantries

Advantages	Disadvantages
Gives headroom below deck	Load imposed on deck on rear support which could be offset from the pier centre
No need extra craneage for segment handling as equipped with segment lifting device such as winch system or strands lifting units depending on cost, speed	Cost
Minimal impact on ground, well suited for urban area or over water.	Complexity of equipment, higher engineering and higher labour qualification

Advantages	Disadvantages
Able to pick segments from already erected deck or from ground	High dead load of equipment
Usually self-launching	Need to load the truss girder with all segments before starting setting segments and gluing operations due to the flexibility of the steel girder.
Suitability for straight and large radius curved deck (over 300-500 m depending on span length)	Wind load on the overhead girder: stability of the girder itself + potentially adverse effect on the deck itself
Fast erection cycle	Double handling of segment: first time to load the girder and get its loaded geometry and last for gluing to adjacent segment.
	Some limitation in small radii curved decks (however hinged overhead girders can be used but are usually heavier and more expensive)
	Potential problem with overhead power lines.
	Overhead equipment takes longer to assemble.

Typical kinematics for an isostatic (simply supported) span

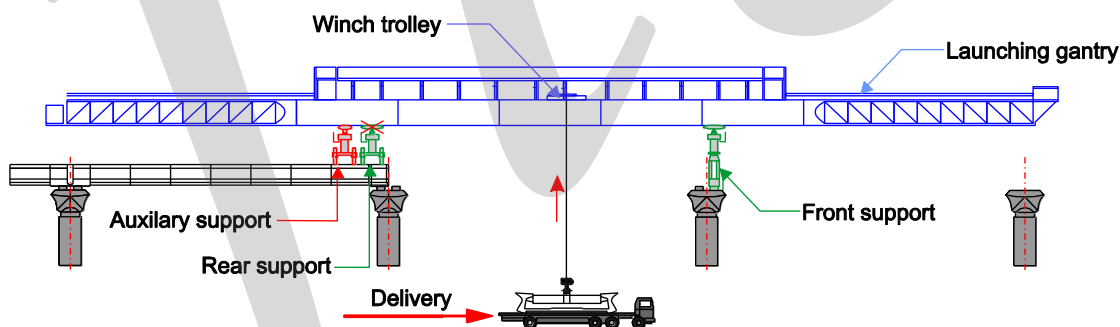


Figure 5-50: Stage I—pick up segment

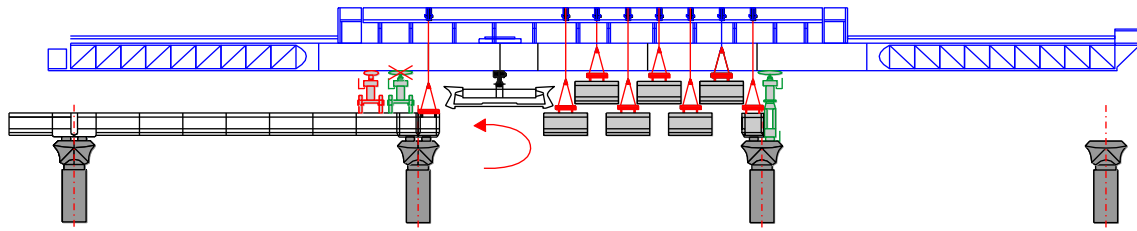


Figure 5-51: Stage 2—fully load girder with all precast segments

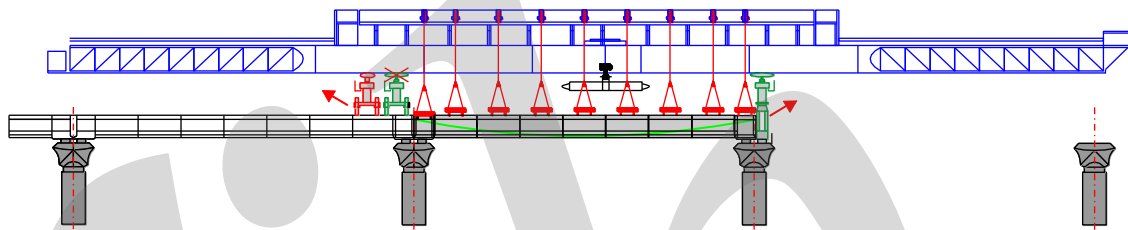


Figure 5-52: Stage 3—adjustment, gluing, temporary post-tension, permanent post-tension

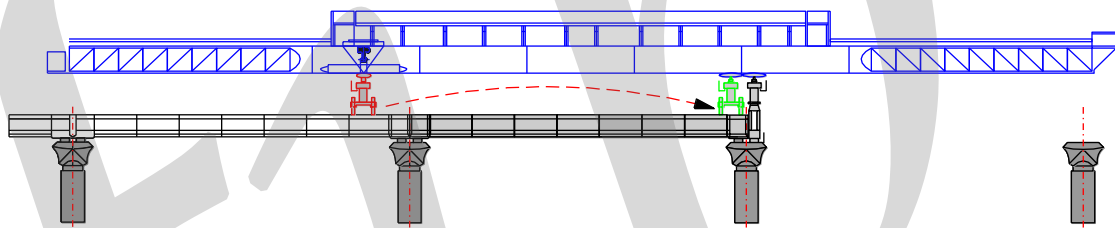


Figure 5-53: Stage 4—after span completion, start launching process by adjusting main truss position, set auxiliary support on erected deck and move rear support

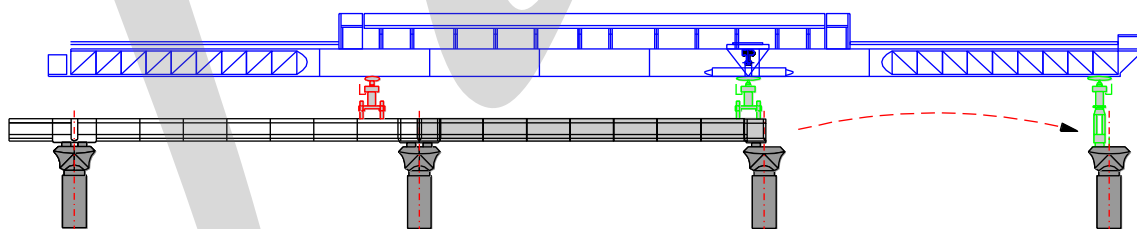


Figure 5-54: Stage 7—move front support

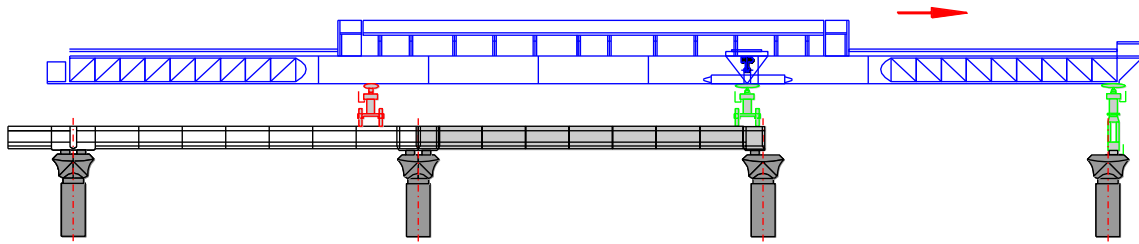


Figure 5-55: Stage 8—launch main girder forward then disconnect auxiliary support

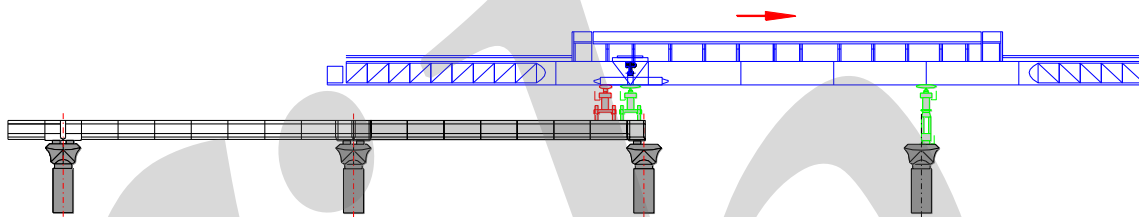


Figure 5-56: Launching process complete, ready to start span erection

Key areas of importance for overhead girder erection

The stages shown in Figures 5-50 to 5-56 are indicative and will vary according to the type of equipment. In all cases there are some particular areas of importance that need to be considered:

- Fixity of the overhead girder during segment erection and self-launching.
- If the front girder support is directly over the pier head and not the pier segment, a special front support design is necessary to allow deck stressing from the front pier segment.
- Load transfer between girder/permanent works: the erected span with the action of its dead load and post-tension will have less deflection than the girder itself.

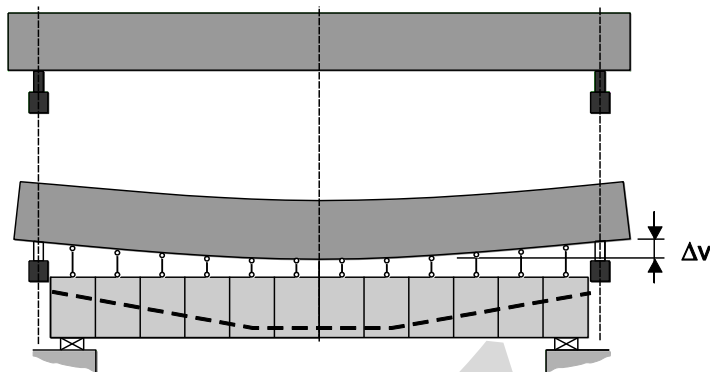


Figure 5-57: Load transfer between span and girder

The release of hangers during the load transfer operation should start at mid-span and move outwards to both supports. Therefore, hangers for the overhead girder close to the pier may be subject to excess tension due to the different stiffnesses of the post-tensioned span and the overhead gantry.

To mitigate this, preventive and monitoring actions are required. These are either based on the local control of loads on the hanger or support with the use of a hydraulic system, or by directly controlling a lowering hydraulic system set within the main support of the overhead girder. In the case of an isostatic span, the load transfer problem can be avoided by placing span jacks below the span that can be adjusted to take the deck load during the permanent post-tensioning operations as the span becomes self-supporting when the hangers go slack.

5.7.5.4 Underslung self-launching girder

As with an overhead girder, this is also essentially a temporary steel structure spanning two consecutive piers and designed to hang a whole span. To make it self-launching, the girder must be long enough and have a mechanised launching system to be able to move longitudinally to the next pier.

As opposed to the overhead gantry, the underslung girder main beams are directly supported over the pier brackets or tower set symmetrically transversely at the pier. Three sets of supports are required: two for erection and the third one for the launching to the next pier. Additional sets may be required depending on the time to relocate those supports.

The underslung girder directly supports the precast segments from either the segment cantilever (the most common type) or segment soffit. Three to four temporary supports are used to sit the precast segment over the main beam during the loading operation of the underslung girder. Two hydraulics jacks are then usually used to support each segment once glued and application of temporary post-tensioning.

Although it is a less complex machine than the overhead head girder, an underslung gantry still needs to be operated by trained and competent persons, as the requirements for stability and fixity are of utmost importance.

Description of main elements

Underslung girder typical components are:

- two main girders;
- carriage trolley set on each main beam able to shift the segment along loaded length of the main beam;
- launching system (chain + wire system);
- 3 No. twin pier brackets or tower at the pier location;
- stressing platform; and
- segment loader or external craneage.

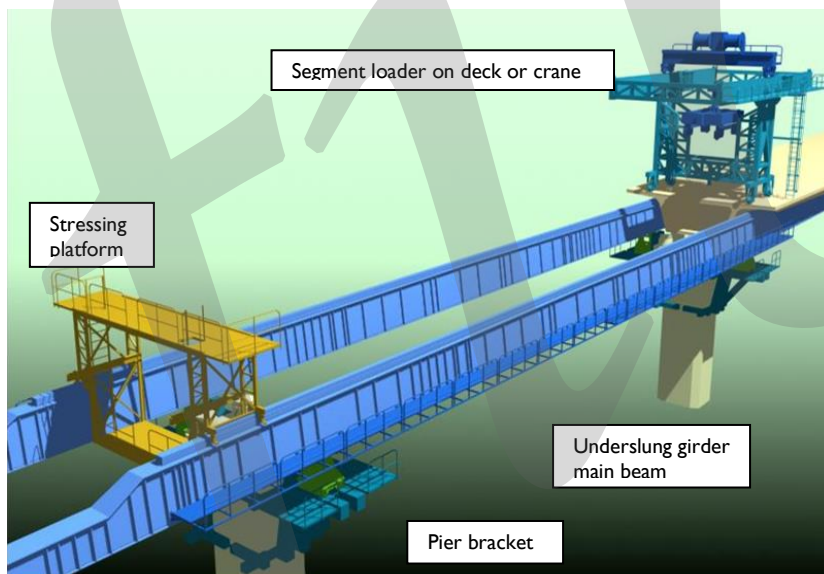


Figure 5-58: Different elements in an underslung girder

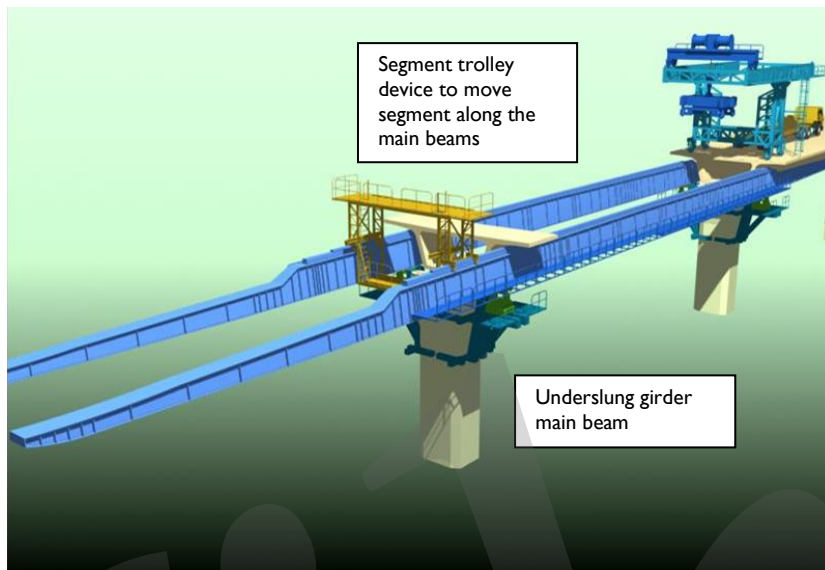


Figure 5-59: Different elements in an underslung girder

Design aspects for the erection equipment

As the full DL of the span is supported by the girder, the reactions on the two main supports are in the range of 50% (DL steel girder + DL precast span). The main supports are set over the pier bracket located on the side of the pier. The load is usually centred along the pier and the pile caps.

Underslung girder dead load is mainly affected by issues such as the steel grade used and sets of steel brackets/supports at the pier, but usually excludes the weight of the segment loader; hence, the dead load is usually less than that of an overhead girder.

The typical ratio between the DL of the underslung girder and span DL is between 0.4 and 0.6 (excluding segment loader), so it is not negligible in the substructure design check.



Figure 5-60: An underslung girder with its bracket system and segment loader from deck



Figure 5-61: Segment loading by external crane on an underslung girder



Figure 5-62: Front view of an underslung girder with bracket system



Figure 5-63: Placing segment above an underslung girder; tarpaulin used to catch excess glue and for safety over an urban area



Figure 5-64: Trolley system used for segment displacement

Table 5-5: Advantages and disadvantages of underslung gantries

Advantages	Disadvantages
Clearance above deck	Leaves less headroom below deck
Load centred on pier/foundation	Need extra craneage for segment setup over the main girder, either by external craneage or segment loader on the deck.
Less expensive than overhead girder	Access on ground / ground capacity required for the crane for underslung girder not equipped with loading frame on the previously erected deck.
Independent left/right beams	Compulsory two main beams system vs. possible one beam for overhead girder.
Setup of supports (bracket at pier/tower over pile caps) done in advance: speed increased on launching process.	Need extra craneage to relocate brackets/tower supports.
	Self-launching system not compatible with sideways shift when there is a parallel deck over T-pier shape or twin pier
	Difficulties for curved bridges (greater than for overhead gantries)
	Need to preload the steel girder with part or all segments before starting pulling/gluing segments together with temporary post-tensioning.
	Precast segment wings (cantilevering part of segment out from web) need to be checked if used for support during erection.

Typical kinematics for isostatic span

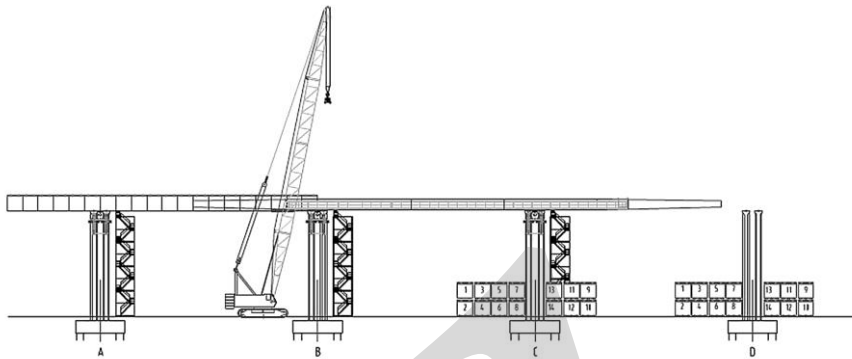


Figure 5-65: Launching completed; ready for span erection

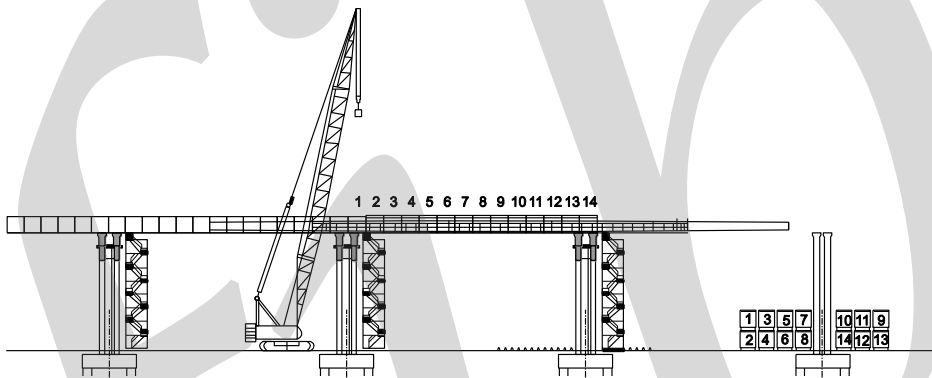


Figure 5-66: Segments erected by crane + adjustment + gluing + temporary post-tension

- Segments are loaded directly on the main beam of the underslung girder.
- Segments are moved along the main beam with a trolley system if there is no crane access along the span length.
- Segments are placed individually on three-four supports over the two main beams until the underslung girder is loaded and the final deflection achieved.
- Using the trolley system, segments are then adjusted, glued and tensioned to each other by temporary post-tensioning.
- Permanent post-tensioning is installed and stressed, setting the self-supporting span over its pier support.
- Connections between each precast segment and main beam are removed. The main beams are moved to the next pier using a launching system.

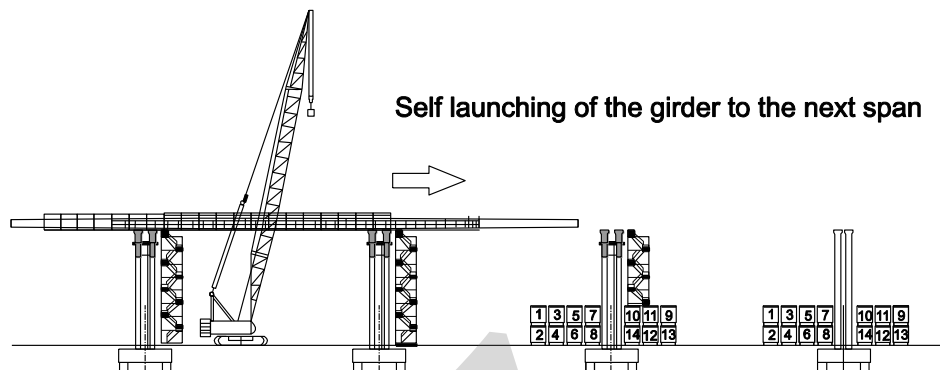


Figure 5-67: Permanent post-tension applied; load transfer; start launching on next pier

Key areas of importance for underslung girder and deck design

Load transfer operation between girder and permanent works:

- The erected span, with the action of its dead load and post-tension, will have less deflection than the girder itself.
- As load is transferred to the permanent supports, the deflected girder wants to come back to its initial resting shape, thus imposing reaction load on the individual supports below the segment wings. Preventive and mitigation actions are based on the local control of those jacks or by a lowering system set between the main beams of the underslung girder and its support.
- Check the precast segment wings, if used for support, as this is a load direction opposite to that of the permanent works design and may require consideration in the permanent works design.

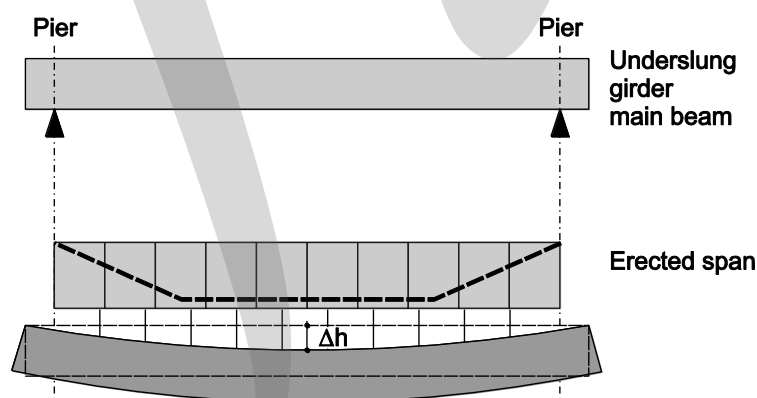


Figure 5-68: Load transfer between span and girder

5.7.5.5 Erection on falsework with crawler or mobile crane

This method of construction is used when the soffit of the deck is low compared to the ground (~ 6 - 8 m) or as a complement to underslung/overhead girder equipment in areas such as viaducts ramps or isolated small bridges.

Another advantage of this method is that it is less demanding in engineering and quicker to set up with local resources such as heavy-duty shoring and crane.

Because the falsework system is made up of a relatively short propping system from the ground, the system will have little deflection when placing the segment. Therefore, preload of the falsework is not required and the gluing operation/temporary operation can start as soon as segments are lifted. However, a precamber still needs to be considered when installing and levelling segments.

The main principles for this erection method are shared with falsework used in heavy construction; thus, for further information reference is made to *fib* Bulletin 48 “Formwork and falsework for heavy construction”.

Description of main elements

The falsework is usually composed of heavy-duty shoring, which offers the advantages of setting the modular tower. Standard scaffold systems can still be used, but need to be almost fully dismantled and reassembled.

The primary requirements are:

- hydraulics jacks to act as supports between falsework and soffit of segment;
- concrete pads to distribute the load (for heavy shoring system);
- good uniform ground preparation (approximately 200 kPa minimum bearing pressure).

Essentially, safety measures should be the same as those used for a regular shoring system.

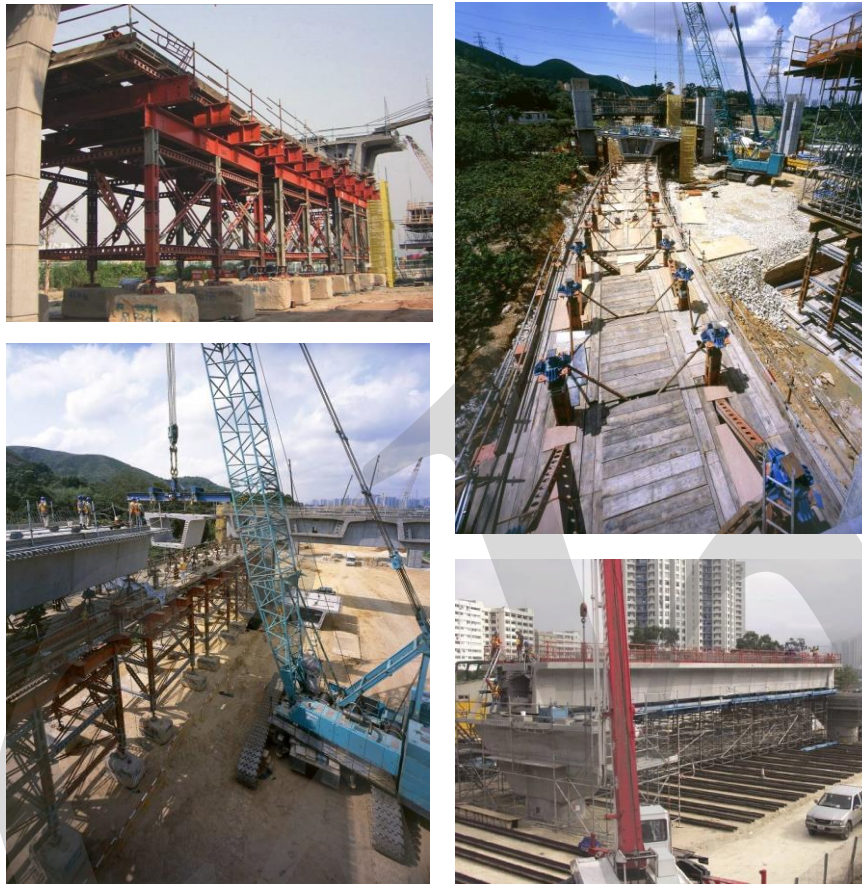


Figure 5-69: Main elements (crane, jacks, falsework, and concrete pad)

Table 5-6: Typical cycle for isostatic span

Typical Erection Cycle	Duration: 4-Day Cycle (Days)			
	1	2	3	4
Installation of Scaffold Support				
Erection of Segments				
Alignment and Gluing				
Post Tensioning Operations				
Load Transfer onto Permanent Bearings				

Table 5-7: Advantages and disadvantages of falsework erection

Advantages	Disadvantages
Easier setup made by proprietary equipment quicker for procurement	Longer time for erection / dismantling (more suitable for limited repetition)
No heavy engineering	Needs craneage
Investment in equipment limited	Amount of distribution beam to spread the load to each vertical elements composing the falsework
Modular shoring system easy to move /relocate.	Ground area to be well compacted to reduce the risk of settlement
Reduced problem of load transfer.	Additional care required for the geometrical control for infill spans
	Deck footprint on the ground. (however carriage way could be maintained with special portal frame)

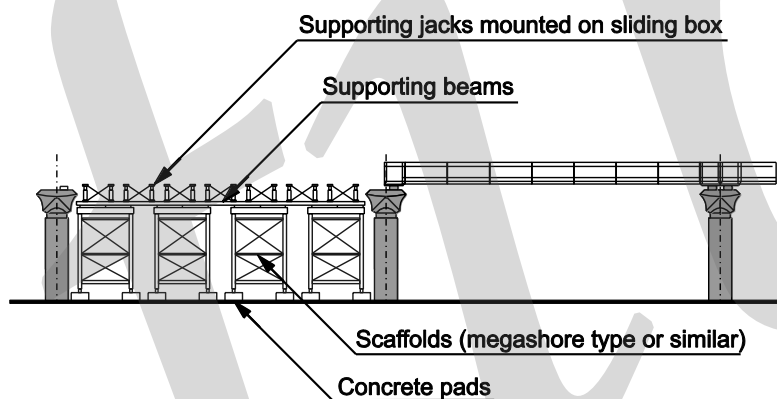


Figure 5-70: Falsework ready to start span erection

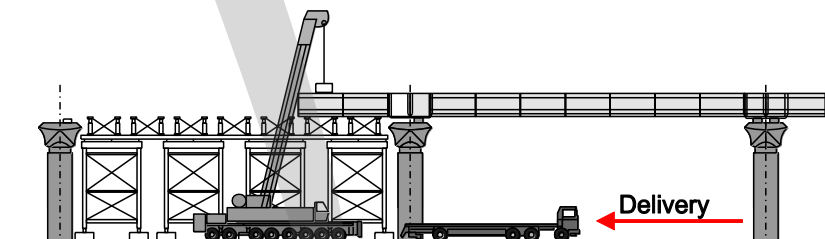


Figure 5-71: The bridge elements are placed on the falsework (by a crane)

Typical kinematics for isostatic span

- Segment placing is done using a crane.
- Adjustment then locks the first two (or three, depending segment length, ratio with span) segments against previous erected span (geometry control).
- Erection (and gluing) of remaining segments which remain individually supported on the falsework. Segments are pulled together with temporary prestressing and geometrical control (monitoring).
- Permanent post-tensioning – loads transfer with between temporary supporting structure and permanent support; and
- Relocate/dismantle shoring system.

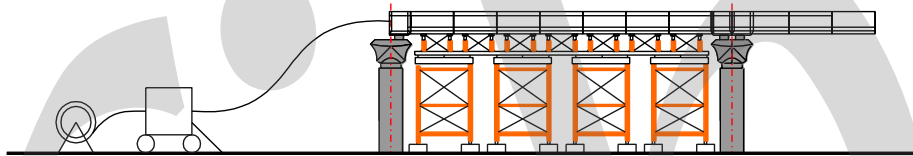


Figure 5-72: Segments are pulled together with temporary prestressing

5.8 Balanced cantilever erection

5.8.1 Introduction

5.8.1.1 General principle – description

The general principle of cantilever erection is to erect segments in pairs, generally symmetrically (that is, ‘balanced’) on either side of the pier, each of them being cantilevered from the preceding one. The erection of a pair of segments can be done either simultaneously or one after the other.

The discussion in the following sections will assume a symmetrical (‘balanced’) cantilever erection. If spans vary, then the cantilevering is not always fully symmetrical, and hence, out-of-balance loads, both temporary and permanently built-in, can occur in the substructure and deck. These need to be accounted for in both the temporary and permanent works.

A variety of lifting equipment can be used for segment erection, such as mobile or crawler cranes, lifting beams sitting on each of the last pair of already erected segments, launching girders or barges.

Many criteria need to be taken into account in the choice of the lifting equipment, such as:

- site location and environmental constraints;

- bridge geometric specifications (length, segment weights, deck geometry);
- construction programme;
- bridge design (precast or cast-in-situ pier segment, cantilever stability); and
- overall budget analysis.

5.8.1.2 General sequence of operations for the erection of a cantilever

The general principle for the construction of a balanced cantilever can be summarised as follows:

- erection (or casting in-situ) of the pier segment;
- adjustment in level and position of the pier segment then nailing to the pier head (when precast);
- erection of the first pair of segments including gluing of the diaphragm (if required) and setting up temporary prestressing;
- pushing/stressing of the permanent post-tensioning; and
- erection of the next pair of segments.

This sequence shall be repeated until the completion of the cantilever.

5.8.1.3 End-span erection principle

End-span erection can start when the adjacent cantilever is completed, and can be done in different manners depending on site conditions:

- erection on heavy-duty scaffolding or special shoring system;
- erection by ‘cantilever’ or ‘false cantilever’ method; or
- erection of the end-spans hung on a launching girder (‘drop-in segments’).

5.8.2 Pier head geometry and arrangement

In the case of a precast pier segment, the pier head geometry should preferably be designed for both permanent and temporary constraints.

The permanent constraints are:

- bearing plinths and pot/elastomeric bearing locations;
- hydraulic jacks location for maintenance/replacement of the permanent bearings;
- sump (when required);

- drainage (when required); and
- seismic devices (dampers or similar when required).

The construction constraints are:

- temporary cantilever support bearings (hydraulic jacks, concrete blocks, sand boxes or similar supports);
- bearing plates for span adjustment (when needed);
- nailing for cantilever stability (when required); and
- inserts for pier head platform fixing.

A special geometry for pier heads and pier segments must be set out to withstand temporary loads such as strong winds (typhoons) or earthquake loads during construction. Temporary nailing may ensure the stability against accidental loads, usually by means of temporary PT cables in a 3D-pattern, as shown below in Figure 5-73 to Figure 5-75.

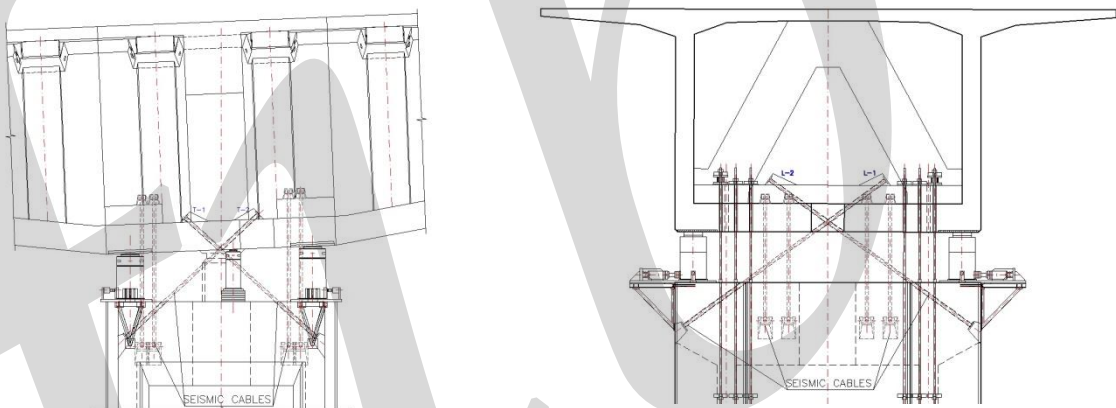


Figure 5-73: Layout of pier head and pier segment with temporary cables—elevation and cross-section

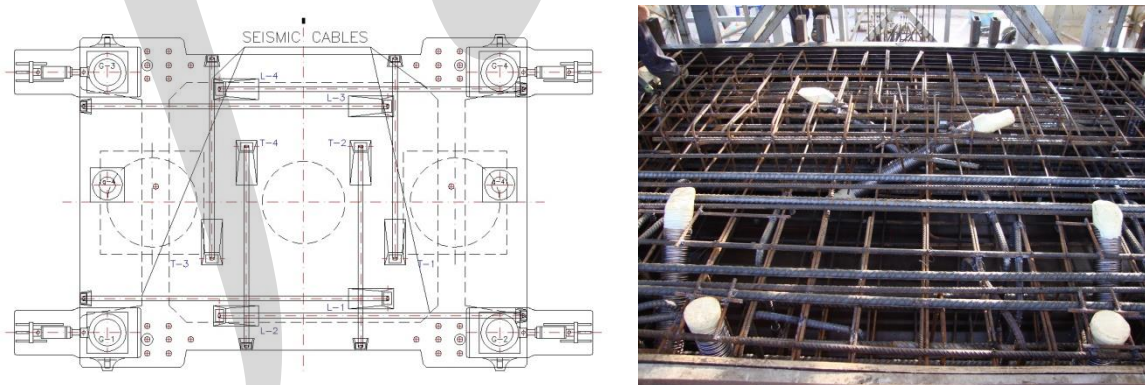


Figure 5-74: Layout of pier head and pier segment with temporary cables—plan and rebar cage



Figure 5-75: Temporary nailing to ensure the stability of both cantilevers

If temporary constraints cannot be taken into account, special methods of erection shall be defined (see section 5.8.3).

5.8.3 Cantilever stability

5.8.3.1 Construction principles

Stability made through the pier head

For precast pier segments sitting on jacks (or other types of shims such as concrete blocks and sand boxes), the general principle is to nail the pier segment to the pier head using post-tensioning cables or high-tensile steel bars stressed to the required value.

The general layout of cables is made up of loops, generally two crossing inside the pier head. The tensioning of the cable is done step by step to ensure that the load is evenly spread.

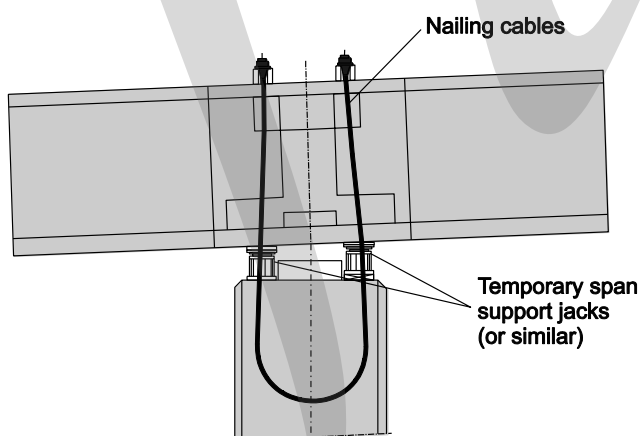


Figure 5-76: Example of cantilever stability by nailing

Since the longitudinal distance between groups of prestressed bars represents the lever arm used to compensate the unbalancing due to unstable forces, these groups of bars are set on the edge of the pier head to minimise their number.

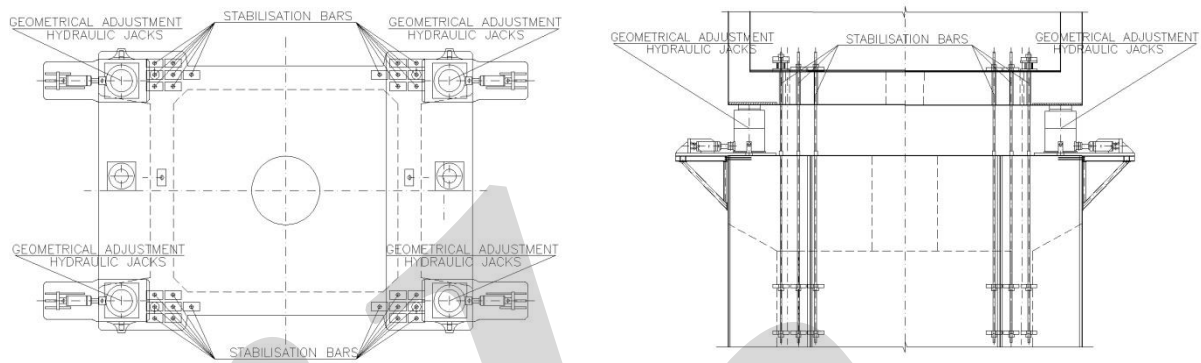


Figure 5-77: Example of cantilever stability by nailing with prestressed bars



Figure 5-78: Connection between the pier head and the bottom slab of the segment with prestressed bars

For cast-in-situ segments embedded in the pier, the cantilever stability is ensured by the rigid pier/pier segment connection, which shall be checked by the designer.

Stability made by a temporary structure

Cantilever stability is generally made by a temporary structure when:

- the pier head geometry is small and cannot cope with all the necessary temporary equipment;
- unbalanced loads are very high (such as with heavy segments, or a long cantilever). By increasing the lever arm of the temporary bearings, the nailing capacity can be reduced significantly; or
- pier/pier head is not designed to withstand construction loads due to unbalanced segments during construction.

The temporary structure supporting the temporary bearings can be made of:

- props sitting on the ground or pile cap and braced against the pier, or
- brackets sitting on the top of the pier head.

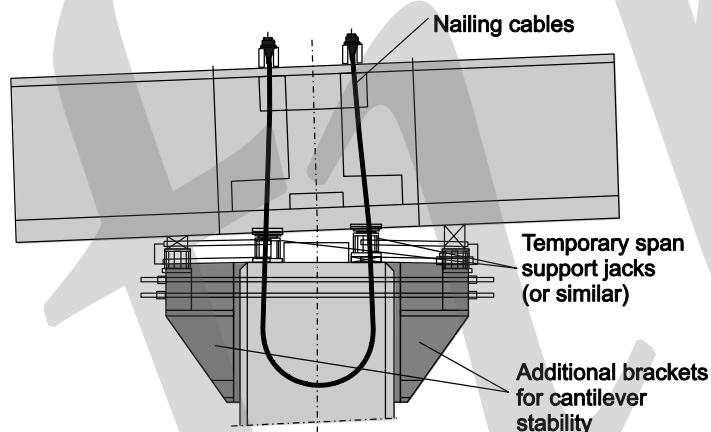


Figure 5-79: Example of cantilever stability using brackets

To reduce the load in the temporary bearings, the choice of a mixed solution can be made with a permanent vertical load taken by the permanent bearings, and unbalanced moment taken by the jacks.

5.8.4 Cantilever erection

5.8.4.1 Cantilever with precast pier segment

The detailed erection sequence of a typical cantilever can be described as follows:

- Erect precast pier segment on jacks or similar (concrete blocks, sand boxes).

- Adjust the pier segment in level and position. This operation is done using jacks sitting on sliding plates (generally stainless steel on Teflon®). Once the pier segment is adjusted, the load can be kept on jacks equipped with a safety ring, or transferred onto concrete blocks, sand boxes or permanent bearings.
- Push the temporary nailing system and stress it to the required value.
- Lift the first pair of segments (or first segment) close to its final position,
- Apply epoxy glue on the segment bulkhead face (in some particular cases, this operation is done before lifting of the segments, depending of the pot life of the glue).
- Erect segments by pair. The erection is carried out either nearly symmetrically (e.g. lifting frame, launching girder, etc.) or asymmetrically (e.g. mobile cranes, etc.). In either case, the nailing capacity shall be calculated taking into consideration the maximum unbalanced moment during segment erection and some accidental cases, such as dropping of a segment.
- Place and tension temporary prestress bars (top and bottom).
- Release the lifting appliance.
- Install prestressing working platform at both ends and push the post-tensioning strand within the ducts.
- Stress them to the required load while checking tendon extension. Once load and extension are checked and are within the defined value, cut the strands.
- Finally, remove the temporary prestress bars. In some cases, temporary stress bars shall be kept tensioned so that the joint is maintained in compression during the erection of segments $n+1$, $n+2$ as specified in the construction methodology.



Figure 5-80: Erection of precast pier segment

During segment erection, a span survey is done and span adjustment may be required in every direction. This operation is carried out using horizontal and vertical hydraulic jacks sitting on sliding plates. Bearing in mind that both cantilevers form a rigid body, the final geometry of the edges may be adjusted with rotations and translational shifts.

In plan, the final orientation can be achieved with the following movements:

- A translational shift, acting on two out of the four horizontal jacks and releasing the other two. Whilst the active cylinders are on one side of the centre line, the passive ones remain on the other.
- A rotation, acting on a horizontal jack and fixing the longitudinally opposed one. The stroke of the diametrically opposed cylinder should be monitored.

In elevation, the final adjustment can be achieved by raising or lowering the vertical jacks, but acting only on two out of the four cylinders in order to avoid excessive movements that could generate a loss of reaction in any jack. The aim is to minimise the number of operations to be carried out. A translational shift can be accomplished by applying the same displacement first on two out of the four jacks, and then on the other two. On the other side, a rotation can be achieved acting on just two out of the four jacks while maintaining the other two blocked.

As these hydraulic vertical jacks withstand high forces, it is necessary to secure loads mechanically by means of a safety lock nut. It is also advisable for these devices to be equipped with tilt grooved saddles to accommodate tiny errors in slopes.



Figure 5-81: Hydraulic jacks to be used on the geometric adjustment operations

Level adjustment must be done with extreme care, as it may affect the load in the nailing that ensures the overall stability of the cantilever during erection. To avoid such overloads in the temporary connection during positioning operations, the auxiliary equipment used to

install and erect segments is shifted and placed into a balanced position. Once it is crosschecked that negative reactions will not appear out on the temporary bearings at any stage, the nailing can be released and the rigid body of both cantilevers can be placed in its final location. Then, after checking the watertightness of each duct, the grouting of tendons can be done.



Figure 5-82: Erection by balanced cantilever method

5.8.4.2 Cantilever with in-situ pier segment

If the pier segment is cast in-situ or comprises a precast shell with in-fill, the two segments SI adjacent to it cannot be match cast and require some adjustment during erection. The general principle is described hereafter:

- bring the segments SI in position (around 200 mm from the pier segment) and hang them to a hanging beam;
- adjust the two SI in level and position then tighten to the pier segments using shims and stress bars;
- place a stitch formwork between the SI and the pier segment;
- cast the stitch (approximately 200 mm long);
- place and stress the permanent post-tensioning;
- release the hanging beam;
- continue the erection of the segments similar as section 5.8.4.1.



Figure 5-83: Erection of first field segment against in-situ segment on pier

5.8.5 Erection equipment

5.8.5.1 Overhead self-launching girder

An overhead self-launching girder is the most common equipment for the erection of a balanced cantilever precast deck. The main technical characteristics of this type of equipment are defined below:

- The structure is composed generally of lattice beams, one or two in parallel, with mobile main supports and secondary supports.
- The girder is able to move on the main supports by means of winches or rack.
- The main supports can move under the girder bearing on secondary support.
- One or two gantry cranes move on the girder beams to carry the segments from behind the launching girder to the final position under the front part of the launcher.
- The unloading of a pair of segments can be done either by direct reading of the load on the two gantries or by semi-automatic device control.



Figure 5-84: Launching girder used for the erection of the East Tsing Yi viaduct



Figure 5-85: Launching girder used on the Avignon project

Special procedures shall be put in place in case of strong winds/storms. These procedures depend on many criteria such as girder design, bridge design, and geographical location. As an example the following typical procedure can be set out:

- Movement of the girder is restricted depending of wind speed; the most critical operation being very often the launching of the girder.
- Nail the girder to the deck at a safe place using the winches and/or stress bars in the case of a high wind event.

The deck geometry may have an impact also on the design of the girder, for example:

- Maximum longitudinal slope of the bridge. Some girders are limited in terms of slope during operations such as launching and segment erection. In that case, some

special temporary supports shall be placed on top of the deck to reduce the girder slope during these operations.

- Maximum super-elevation of the bridge. Similar constraints as above.
- Minimum deck radius. This is a major issue and shall be checked as it may have some consequences on the bridge design. As an example, in tight curves, it may not be possible to find a girder position such that the segments can be erected simultaneously by pair.

Considering the complexity of the use of such equipment and the loading imposed on the permanent structure, it is necessary to have a good interface between construction methods, design and production to ensure the permanent structure is not overloaded by any temporary loads.

5.8.5.2 Erection using cranes

The use of cranes (mobile or crawler) for segment erection requires some special site conditions, such as the need for an access track able to withstand the load of a large crane.

Crawler cranes, being able to track from one position to the other with the load, are more flexible, while a mobile crane must be positioned such that it can pick up the segment from the delivery point to the erection point without any movement.

In term of design, the erection by crane is not done symmetrically and the impact on the permanent design shall be checked carefully.



Figure 5-86: Segment erection by crane

5.8.5.3 Erection using lifting beams

Lifting beams are a simple way to erect balanced cantilevers. The system is composed of a steel structure sitting on and fixed to the segments already erected. The precast segments are then delivered under the cantilever by trucks, barge or other equipment and lifted directly to the right position. The lifting operation can be carried out either by winches or strand jacks.

In terms of design, the lifting of the segments can be done either symmetrically or not. It must be noted that the self-weight of the lifting appliance at the extremity of the cantilever shall be added to the dead load of the segments, and the impact on the permanent design shall be checked carefully. Also, the effect on the permanent works of an accidental segment drop, including the dynamic factor, must be considered. Care is needed as this can govern the pier design.

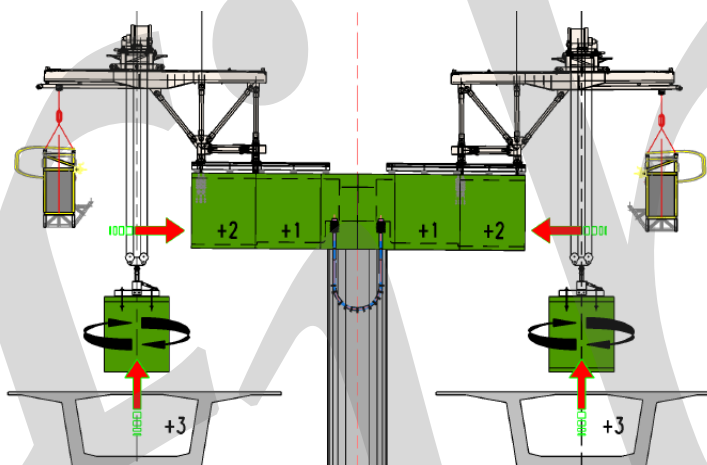


Figure 5-87: Segment erection by lifting beams

5.8.6 Stitch at mid-span

A stitch of varying length is kept between two adjacent cantilevers. The stitch is necessary to:

- erect the last segment of a cantilever (on the side of the already erected cantilever);
- join the prestress ducts without angles, if there is a non-smooth duct alignment.

The stitch is generally non-reinforced (depending on the length). Nevertheless, if it appears that the joint is wider than expected, it may be necessary to place some reinforcement, which is an awkward and time-consuming operation.

To adjust the ends of two adjacent cantilevers and avoid cracks in the stitch before prestressing due to span movement during concrete curing, steel ‘shear’ beams are often placed to connect the two structures adjacent cantilevers and thereby achieve equal displacement.

Another solution is to add a key segment between the two adjacent cantilevers. This key segment is adjusted in position and stitched to the two cantilevers.

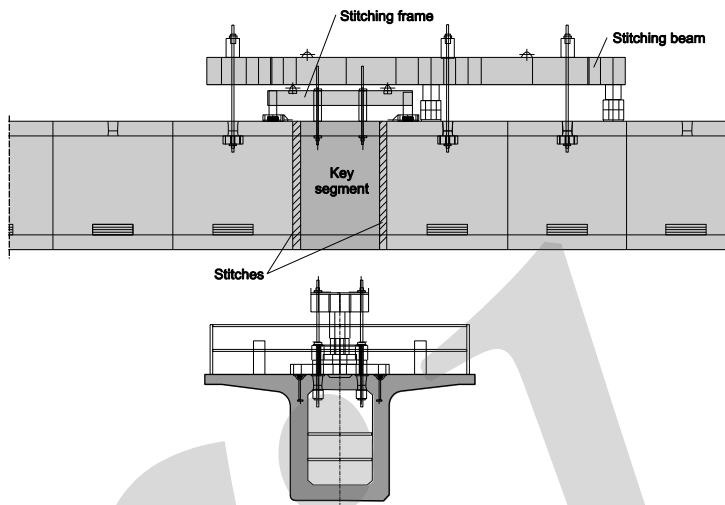


Figure 5-88: Example of key segment

It can be beneficial to get a special concrete mix with a high early strength for the stitch, as the stressing of the continuity cables can start only when the concrete reaches a certain strength (to be defined by the designer).

Detailed calculations are to be undertaken after pouring the concrete on the stitch. When it is hardening, shrinkage and prestressing applied on the concrete at a very early age to avoid unexpected cracks due to temperature gradients.

5.8.7 End-span erection

End-span erection can be done many ways, depending on the site conditions:

- Erection on heavy-duty scaffolding or special shoring system. This method requires good ground conditions or special treatments or foundations (if needed). In some cases the shoring can be design so that the load is transferred directly to the pile cap.
- End-span segments hung to the launching girder. The principle is to erect the segments one by one from the abutment to the pier, with each hung on the launching girder by stress bars or similar.
- End-span erected by “cantilever method”. In this case the end-span is built from the pier towards the abutment, asymmetrically on a single side of a balanced cantilever, with the other side already stitched to the adjacent span.
- End-span erected by “false cantilever methods” when two end-spans are on both sides of an abutment pier. The general principle is to temporarily connect two

adjacent abutment precast segments by post-tensioning to obtain a single element similar to a pier segment. This element is then nailed to the pier and the erection of the segments is done by the balanced cantilever method. Once the two stitches are cast and continuity cables tensioned on both end-spans, the post-tensioning cables connecting the two abutment segments can be cut off.

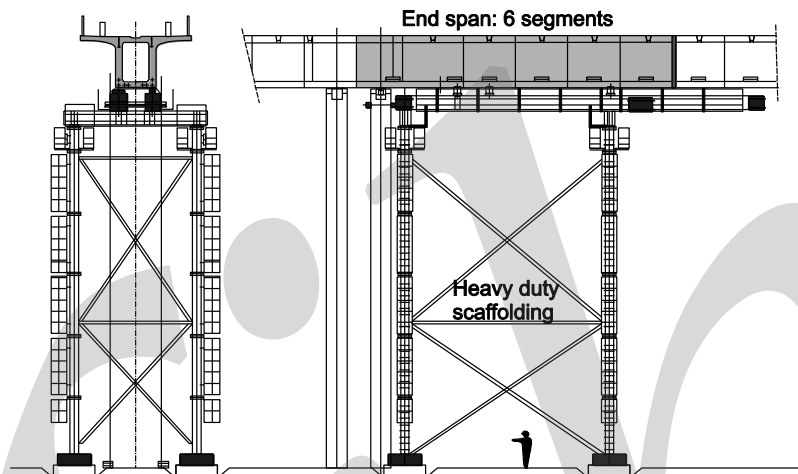


Figure 5-89: Example of end-span erection on heavy-duty scaffolding



Figure 5-90: End-span hung to the launching girder

5.8.8 Post-tensioning installation

As with other types of prestressed bridge, there are a number of types of post-tensioning cables:

- cantilever,
- span, and
- continuity.

5.8.8.1 Cantilever cables

Cantilever cables are the permanent cables used to hold each pair of segments together, forming a complete cantilever. They are usually internal tendons at the top of the segment. However, in unusual circumstances they can be supplemented with tendons dropping into the webs or with external tendons.

All strands of the tendons of a particular construction stage shall be threaded into the ducts after erection of the segments from a prestressing platform fixed to the segment end. Strands are delivered on the top of the deck in strand dispensers called coils.

Strand threading can be done either by hand or machine, depending on the length of the tendon. Strands are pushed one by one using a strand-pushing machine sitting as near as possible to the hole. The strand dispenser is generally set up on the top of the bridge deck within a reasonable working distance. Upon completion of the threading, the strands are cut to the required length. The protruding strand length shall be determined based on the required jack length to be used.

Hydraulic stressing jacks are then put in place and the stressing operation can start. Once the required stressing force is attained, the tendon is locked off by releasing the jacking force. Strand extension is checked during the whole process. The strands shall not be cut if the percentage of variation does not comply with the specification requirement.

5.8.8.2 Span cables

Span cables are by definition within the spans between piers. They can be internal or external tendons. If internal, they are usually within the webs or bottom slabs. For bridges constructed with the cantilever method, they are the permanent cables, usually located at the bottom of the box girder, used to ensure the continuity of the post-tensioning between cantilevers, and largely governed in their design by the permanent live load effects. For bridges constructed with the span-by-span method, the span cables carry both the dead and live loads.

The same equipment and similar procedure as for cantilever cables shall be applied. Special care shall be taken when casting the stitch to ensure the continuity and the watertightness of the cable ducts.

5.8.8.3 Continuity cables

Most of the external tendons are installed between the two ends or between the intermediate piers of a precast segmental bridge through the deviator tubes of some diaphragm segments. Generally these tendons are stressed after all the spans and end-spans are stitched and the internal continuity tendons are stressed. But in some cases, construction loads may require the installation of external temporary cables immediately after completion of a cantilever.

The anchorages are cast in the concrete at the precasting yard while HDPE pipes shall be installed when the span is already erected. All the individual HDPE pipes are connected together by welding. When necessary for long cables, temporary support shall be erected to support the tendon before threading the strands.

A grout inlet is installed at a low point close to the anchorages, while other grout vents are provided at high points of the tendon.

5.8.8.4 Grouting

After cutting off the protruding strands, a protection cap is placed at the casting surface to seal the anchor head. A water or air test is then carried out and all leaks are repaired. The grout mix and mixing procedures shall be approved by the client. The approval is generally linked to the acceptance of a grouting test.

Grout is then pumped through the inlet pipe of the lower end of the structure.

Grouting of each tendon should be carried out in a continuous operation and by family of cables to prevent any leakage from one duct to the other.

This operation should be done as soon as possible following tendon stressing so as to protect the strands, nevertheless generally:

- This operation is carried out so that it does not interfere with the erection process.
- This operation is preferably done after completion of the full cantilever because the grouting of tendons may result in a blockage of an empty duct if there is leakage from one duct to another. Nevertheless it is recommended to grout as soon as possible, which often means that unless other protection is applied, stage grouting of the cantilever tendons might be required before completion of the cantilever (see *fib Bulletin 33*, page 35, and *fib Bulletin 20*).
- Success of the quality grouting operation is vital to secure the integrity of strands against corrosion.

6 Detailed design considerations

6.1 Materials and general considerations

6.1.1 Introduction

In the design of a precast segmental structure there is a significant need for coordination between the permanent works design and the temporary works design, beyond the usual requirements in other types of bridges. These aspects overlap and interact and it is at this design interface that problems and errors can be introduced into the process, particularly if the various parties involved do not fully understand the roles, responsibilities and scope of the others. Most of the advantages of this type of construction will be lost if the interaction between all roles is not clear.

This is a particular issue in traditional engineering contracts, but can also occur in design-and-build works if there are subcontractors or sub-consultants involved.

It is vital that the permanent works design at least allow a viable approach for construction and that all parties ensure that errors do not inadvertently occur where there are design interfaces, due to lack of transfer of information, poor coordination or lack of clarity of design responsibilities.

Whilst designers of non-segmental bridges may well focus and optimise their designs for the permanent case, wise designers or design teams of segmental bridges understand that the effects of the temporary works and erection methods, as well as permanent cases, can govern the dimensions and materials of many elements in segmental construction.

6.1.2 Prestress systems

Prestress losses in internal bonded tendons may be higher in segmental cantilever bridges than in in-situ bridges, particularly if the detailing and execution at the precast joint, where the coupling system of the duct between segments is placed, is not done properly and kinks appear at segment joints. Curved tendon layouts through segment joints should be considered as much as possible.

In balanced cantilever construction, the prestressing of the first segment may lead to very short cable lengths (less than 10 m). Particular attention must be paid to the prestress losses and the uncertainty due to the pull-in of the cable anchorage.

6.1.3 Creep and shrinkage—longitudinal effects

In long-span concrete bridges, it is usual that the structural system changes considerably during the construction process. This is particularly important in balanced cantilever construction. The change in support conditions between the different temporary stages and the permanent stage produces a redistribution in the long-term stresses of every section of the deck under permanent loads. clause 7.2.4 of the *fib* Model Code 2010 gives guidance on how to consider this effect.

Whilst this is not different from a non-precast segmental structure, the construction method gives rise to some differences.

- The key features are due to the factory-produced nature of segments. This leads to similarity and reliability of elements with less variation in time-dependant properties. A more controlled curing, even potentially steam curing, and the likelihood of storage and stacking of segments for a period prior to their erection, reduces in general the long-term effects in the permanent structure.
- Compared to in-situ structures, precast segments generally have reached a higher degree of maturity during erection and application of permanent prestress. Hence the creep coefficient will be lower and more shrinkage will have occurred prior to the erection.

These assumptions and parameters used in the design need to be reviewed by the designer to ensure that they are not overly conservative.

6.1.4 Creep and shrinkage—transverse effects

If the cross-section is cast in more than one stage, for example cast-in-situ cantilevers supported by props are placed after a central box is erected using the segmental method (as shown in Figure 6-1), the evaluation of the transverse redistribution of stresses in the long-term analysis due to the differential shrinkage and creep between the different concrete elements must be taken into account. This analysis is usually critical for the longitudinal reinforcement of the concrete cast in-situ in a second stage and the prestressing of the box.

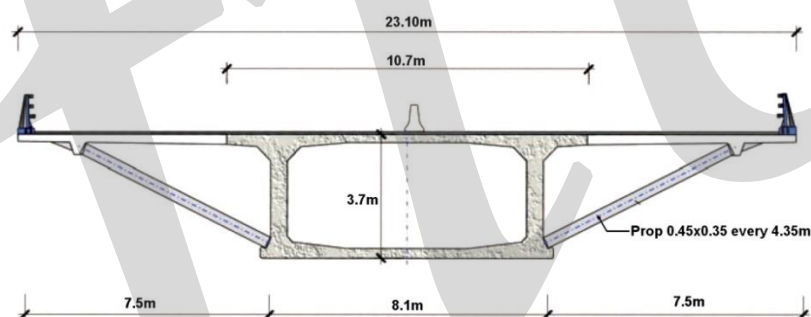


Figure 6-1: Example of segmental construction combined with transversal in-situ cantilevers

6.2 Construction

6.2.1 Launcher action

A difference between precast segmental bridges and other forms is the potential use of large erection equipment or a gantry to place the segments either in span-by-span or balanced cantilever construction.

The loads due to the launching gantry can be significant and it is very important to consider accurate values and locations of these loads, which can be governing scenarios in the overall design of the structure, including elements of the permanent works, such as:

- cantilever prestress quantities;
- additional diaphragms or deck stiffening where a launching gantry has leg positions away from pier diaphragms;
- tie-down loads effecting the sub-structure;
- bottom flange dimensions; and
- temporary prestress at pier and column.

It is noted that whilst assumptions can be made at early stages of design, there is a risk that they may build significant conservatism into the design, but also a risk that the design is non-conservative if some of the temporary works during construction change at a later stage. Hence, a close interaction and liaison between the temporary works designer and the permanent works designer of the structure, if different, is highly recommended.

The gantry action should be taking into account both instability and structural ultimate limit state (ULS) checks. Additionally the serviceability limit state (SLS) of deformations might need to be considered in order to avoid, or at least take into account, permanent (non-elastic) deflections that might affect the geometry control of the structure. This effect can be significant if the governing condition of the bottom flange of the deck is the self-launching of the truss during construction.

Particularly in long spans, where the use of very heavy launchers is more common, residual stresses due to the launcher can be a governing condition, not only during the temporary stages but also as permanent stresses due to creep if the structural scheme changes while these loads are applied.

6.2.1.1 Control of launching girder loads

In the case of balanced cantilever construction it is quite common that the launcher girder is an indeterminate structure (usually three supports). This means that the reactions on each of the supports is variable and relates to the relative stiffness of the launching truss and the deck itself; consequently, the theoretical values considered in the design need to be monitored on site by the gantry operator. The use of load-cells and testing is necessary

before construction starts, as well as using conservative values for the design and check of the bridge. The following safety values can be considered as indicative:

- The characteristic unfavourable value of the gantry action can be obtained using a load factor of $\gamma_q = 1.2$ in the worst leg for three leg gantries and $\gamma_q = 1.05$ in the others from the nominal values.
- The characteristic favourable value of the gantry action can be obtained using a load factor of $\gamma_q = 0.8$ in the worst leg for three leg gantries and $\gamma_q = 0.95$ in the others from the nominal values.
- The ULS verifications will require combining the effects of the load indicated above as part of the permanent loads.

6.2.1.2 Launcher structural check

The launcher is itself a complex structure that must be checked for operation on each bridge. It is important to check the stability of the launcher when moving in its different positions, especially if the bridge is curved, as the eccentricity of the elements could have an important influence.

6.2.2 Temporary prestress check between segments

Prior to the installation of the permanent prestress during construction, and particularly in the case of balanced cantilever construction, it is common to use temporary prestress bars to join the segments during a temporary stage.

An accidental loading scenario due a segment drop should be considered with the relevant dynamic amplification factor, commonly a factor of two.

If more than one segment is supported by temporary prestress, additional checks of potential progressive collapse of the segments should be considered prior to the introduction of the permanent prestress.

The temporary prestress is usually located in internal blisters and it is removed once the permanent prestress is installed. Consequently, the blisters should be checked under ULS. The durability and SLS checks are less relevant; however, excessive cracking should be avoided. Temporary prestress can also be applied using brackets on the top slab, but the local deck cracking does need to be reviewed as durability could be impaired.

6.2.3 Stitch segment design

The dimensions of the stitch segment at mid-span in balanced cantilever construction should be clearly determined, and when its length is larger than 300 mm, reinforcement should be provided.

Additionally, the concrete-to-concrete interface should be calculated taking into account the recommendations indicated in section 6.5.2 and the type of interface provided.

6.2.4 Temporary pier head connection

Depending on the construction sequence, temporary prestress might be required during construction to ensure the stability of the deck. This is particularly relevant in balanced cantilever construction, where this connection or another temporary type of connection is almost always required.

The temporary prestress should be checked under SLS and ULS (STR and EQU) load combinations.

If the bridge is in an area where significant horizontal loads could arise (such as seismic events or typhoons), the temporary prestress configuration should allow for the relevant horizontal force in an accidental load combination in ULS.

It is common practice in balanced cantilever construction to support the deck on jacks during the cantilever construction. In these cases, the maximum jack capacity should be considered by the designer using an SLS load combination taking into account the vertical prestress. The global geometry of the structure (vertical and horizontal alignment) should be taken into account as it will produce significantly different forces on each jack.

6.2.5 Lifting points

The segment lifting points should be clearly defined and calculated accordingly, both under SLS and ULS scenarios. Taking into account that the segments are usually heavy (30 to over 100 tonnes) and need to be lifted and manipulated at very early stages, specific consideration should be given to concrete strengths at early stages.

6.3 Serviceability limit state

6.3.1 Stresses

In general, design codes limit the tension stress on the extreme fibres of a section in the SLS case, particularly for positive values (tension/decompression).

This stress limit becomes more stringent for a PSB. The tension limit under some SLS combinations specified by many codes is zero, and compression is needed across the joints.

In the particular case of internal bonded prestress, the opening of the joint at SLS will compromise the durability of the structure. To avoid this situation, a certain level of compression is usually recommended; the following values can be used as guideline:

- 0.3 MPa of compression stress at the worst point of the cross-section under the SLS frequent combination. This limit should also be applied during construction.
- No tension at the worst point of the cross-section under the SLS characteristic combination.

As stated in section 6.1.2, creep and shrinkage induce changes in the stress state of the bridge and must be taken into account; hence, the SLS has to be checked during construction, at the beginning of its service life and after long-term concrete creep, in order to determine the worst case scenario.

Although a decompression requirement during construction is not necessary from a durability point of view, avoiding tensile stresses in the structure under construction loads enables uncertainties linked with the loss of stiffness to be reduced or removed when predicting the deflection of the tip of the cantilever.

6.3.2 Non-uniform transverse distribution of stresses

As described in Section 5.6.3.6, the effect of bowing can be significant if the width-to-depth ratio is important.

This phenomenon may affect the stress distribution in the deck across joints and could lead to reduced compression in the segments' centreline and to stress peaks at the tips of the segments' cantilever slab.

Local models to evaluate this effect could be required and its effect should be taken into account when calculating the minimum stress requirements at the joints.

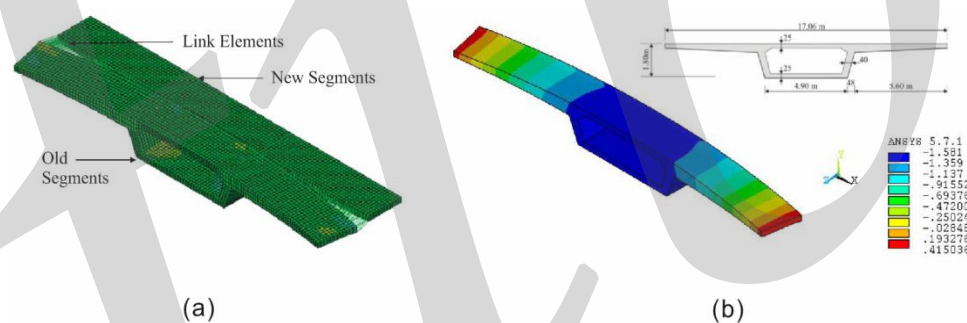


Figure 6-2: (a) Finite element mesh and (b) deformations from the numerical model of the type III match-cast segment of the San Antonneio 'Y' project

6.4 Ultimate limit state

Different considerations should be addressed depending on whether external unbonded prestress is present or not. This applies to both bending and axial checks and to shear and torsion.

6.4.1 Bonded prestress

6.4.1.1 Bending and axial

The ultimate capacity under normal stresses does not differ from the calculations for cast-in-place structures.

Since there is no reinforcement crossing the joints, it should be also verified that there is no load combination that produces a joint opening in ULS if prestress is not provided in the relevant face (top or bottom) at the location of the joint.

6.4.1.2 Shear and torsion

Since the longitudinal prestress is the only steel reinforcement crossing the joint, a suitable structural system should be provided to ensure that the shear and torsion can be resisted by the longitudinal prestress or the concrete in decompression when the accompanying axial and bending moments are also considered.

The Ritter-Mohr shear analogy, as defined in equation 7.3-34 of the *fib* Model Code 2010, is a possible system to carry out this check.

6.4.2 External/unbonded prestress

6.4.2.1 Bending and axial

The ultimate capacity under normal stresses does differ from the calculations in cast-in-place structures with external prestress, due to the joint opening effect and the lack of reinforcement crossing the joint.

In general, the joint opening leads to a redistribution of forces in the section for continuous structures and large deflections for simple supported structures. For this reason, non-linear analysis at ULS is recommended.

The approach in Eurocode 2 for ULS design is an alternative. Some codes, for example BS 5400, also have a requirement for overcapacity in longitudinal moment to avoid non-ductile failures. This is more difficult to achieve if the amount of external post-tensioning is high.

Some standards limit the maximum value for the joint opening under ULS.

The prestressing stress increases due to the increase in length between fixed points (anchors and deviators). An appropriate model to calculate this increase in stress should be considered.

This change in stress is heavily dependent on the geometry of the cable, and it must be taken into account to obtain the ultimate capacity of the section. Stress increases between 20 MPa and 250 MPa are usually expected.

The cable eccentricity within the section will also change due to the large global deflections. This change in eccentricity has to be taken into account when checking the ultimate capacity of the section, taking into account the effect of intermediate deviators, if present.

6.4.2.2 Shear and torsion

If the accompanying bending and axial loads to the maximum shear or torsion lead to a joint opening (concrete stresses higher than the design concrete tensile capacity), the shear and torsion has to be checked in ULS, taking into account the reduction of section depth once the joint is opened, assuming that the contact between segments is reduced to the compression block area.

An adequate system to transfer the shear and torsion through the joint has to be considered.

6.5 Detailing

6.5.1 Diaphragms, blisters and prestress deviators

Elements with concentrated loads and support diaphragms in segmental bridges are not different from those in concrete bridges built using other technologies and the general design rules for this type of structural element apply.

However, when detailing anti-bursting reinforcement at blisters (posttensioned end anchor outside the main box section), the length of the reinforced zone must lie within a segment. In general the blister geometry is conditioned (fixed) by the mould, which limits the flexibility in detailing this element.

If large tendons are used, particularly with thick bottom slabs, this may require ducts with small radii curvature to fit within a segment. This may cause increased cost and practical difficulties and so needs to be considered at an early stage.

6.5.2 Shear keys

The shear key should be treated as an unreinforced interface. Clause 7.3-50 of the *fib* Model Code 2010 provides the criteria to verify this element.

7 Maintenance, repair and demolition

It is incumbent on a designer to ensure that their structures are safe during construction, in service and during demolition, and that potential risks are highlighted and this knowledge is made available to others. This often takes the form of risk registers and manuals.

This chapter examines some particular safety and reliability considerations during the whole life of a segmental bridge structure.

7.1 Robustness

Robustness is a quality of design that reflects the behaviour of a structure beyond the ultimate design condition. It is thus a reflection of the behaviour and safety of the structure during failure. Providing robust structures is a key element for new designs and where there is low ductility great care should be taken to demonstrate robustness.

In building structures that have multiple load carrying paths, robustness focuses on the ability of the structure to mobilise these other, secondary, load paths when the primary load path is no longer viable to mitigate progressive collapse or disproportionate collapse.

If no alternative load paths are available, the primary load path should be capable of sustaining an appropriately high level of required loads. For a bridge structure there is often a mixture of elements, the failure of which can cause local failure or more global effects. In these cases the robustness of the elements feeds into an overall assessment.

Like many bridge structures, the primary load path of a precast segmental bridge is the deck. There is little transverse spanning. Failure of the deck causes failure of the structure.

For a precast segmental bridge, the notion of robustness is a concern in that there should not be disproportionate consequences of a failure. In a precast segmental bridge the robustness is provided by large joint rotation that enables large forces to develop in the tendons.

Robustness should be addressed to ensure there is no progressive collapse of the structure, to produce alternative load paths to share loads in the post-failure condition and to allow for loss of prestressing cables. This can be done using continuity cables between spans. The use of built-in piers and the design of prestressing cables allow for a proportion of loss.

Many of these considerations are not specific to precast segmental bridges; for example, stay loss is often a design consideration for cable-stayed bridges. However, for a segmental bridge, the lack of continuity at the joints reduces the ability to span by itself and hence relies on the prestressing for both bending and shear capacity.

The primary difference between a segmental bridge and other bridge structures is the lack of ability to allow plastic redistribution, due to the lack of continuity of the reinforcement at the joints. The only mechanism for developing redistribution is via the tendons.

To design a segmental bridge for robustness, the designer needs to consider the use of continuity cables to produce continuous bridges rather than simply supported ones, and the use of built-in pier-to-deck connections.

It should be noted that although the notion of robustness is novel in some codes, in others, particularly those that consider seismic actions, such design details are not unusual.

It should also be noted that there is a difference in the behaviour of internal and externally prestressed bridges, which is heightened with segmental construction, as there is overall less ductility in the failure of an external tendon.

The usual approach to assess robustness is to ensure that there is no progressive failure of the structure when an element suffers a local failure. This is the approach of the Eurocodes (EN 1990, 6.4.3.3).

Redundancy can be considered as a potential mitigation for robustness, which is often exemplified by the provision of spare ducts in a post-tensioned bridge.

Practically, a bridge should be designed with the potential to add additional prestress via spare ducts or blisters. If rotation is the mechanism to allow large deformations, then these need to be accommodated by the design, such as restraint of bearings and flexibility of the piers.

7.2 Maintenance

Whilst the majority of maintenance requirements and processes are the same for segmental and non-segmental bridges, the reliance on the tendons for structural integrity requires greater care in the inspection and maintenance of the tendons, be they exposed or not (i.e. external/internal PT). In this regard, external tendons are easier to inspect and could be replaceable. It is also very common to make provision in the design for additional tendon(s) that could be installed at a later stage if strengthening or repairs are required.

Having joints introduces a potential route for water and chloride ingress. If the tendon protection is breached this can lead to tendon corrosion. There are examples, for example in the UK, where corrosion has occurred, particularly in top slab cables. If water and chloride ingress is a risk, it is suggested that the designer use deck waterproofing, and that this is maintained.

It is also noted that the lower ductility of a segmental bridge compared with an in-situ bridge means that if there is poor maintenance, for example bearings becoming locked, then the resulting stresses cannot be relieved by widespread small scale concrete cracking, and deformations and cracking are concentrated at the joints.

Similarly, maintenance of drainage is necessary for a prestressed bridge to minimise the risk of corrosion of tendons from water within the deck.

Inspection access to the interior of the box girders is generally useful, but is particularly necessary for bridges with external prestressing tendons. Access hatches should be sufficiently robust to avoid water ingress. The location of the hatches should be chosen with care to balance often competing requirements, for example of ease of access and aesthetics. Particular care is needed for bridges crossing water to avoid hatches in the splash zone. The size of the access should generally accommodate a man. However, if passive provision for additional prestress cables or replacement of prestressing cables is envisaged, then the access should take into account the future requirement for access of cables and jacks in the deck. Drainage should be provided to avoid pooling of water within the decks, particularly if the bottom slab of the slab is stepped in thickness.

A particular maintenance issue for segmental bridges is to maintain the systems employed to protect the tendons at joints.

In addition to deck waterproofing, which, as noted previously, is used commonly by some designers in some areas but is not universal, there are both proprietary and bespoke site applied joints and duct couplers. These should not need maintenance, but care is needed to ensure that they are not damaged during installation or during service.

7.3 Demolition

Design for safe demolition of a structure relies to an extent on having ductility and multiple load paths to minimise the amount of temporary works needed.

With little continuity at the joints and low shear capacity, care is needed for demolition. With internal tendons, as with all prestressed structures, the reliability of bond on grouted tendons is of concern if they are cut during demolition.

8 Case studies

This section describes several bridges where this technology has been used, with each case presented in a two-page layout.

While the list does not claim to be in any way comprehensive, it reflects the flexibility and wide range of applications of segmental construction technology by covering a variety of structural types, spans, deck widths and crossing types.

The bridges included in these case studies are:

- Si Rat Second Stage Expressway, Thailand. 1996.
- Bang Na–Bang Pli–Bang Pakong, Expressway, Thailand. 2000.
- HartelfietBrug – Spijkenisse Cycle Bridge, The Netherlands. 2011.
- Marin Viaduct. Arlaban Estoriatza, Spain. 2008.
- Batalla Viaduct. Alicante, Spain. 2011.
- San Juan Viadcut. Tenerife, Spain. 2012.
- Cieza Arch Bridge, Santander, Spain. 2004.
- Navia Viaduct, Asturias, Spain 2006.
- Danube Bridge, Romania. 2013.
- TGV Méditerranée, Avignon Viaduct, France. 1999.
- SEA Tours-Bordeaux, France. 2015.
- East Tsing Yi Viaduct, Hong Kong. 2009.
- Four Bears Bridge, USA. 2005.

SI RAT SECOND STAGE EXPRESSWAY, THAILAND

Completed in: 1996
Location: Bangkok, Thailand
Function/usage: Motorway bridge

Structural type

Hollow box-girder bridge, single spans
 longitudinal external prestressed
 transverse internal prestressed

Technical information

Match-casting method: short-line
 Construction method(s): span-by-span
 overhead gantry
 Joint type: dry

Dimensions

Total length: 38.5 km
 Typical span length: 45.3 m
 Typical width: 7 - 15.60 m

Building materials

Concrete f_{ck} : 35 MPa
 Steel grade:
 Post-tensioning: St 1670/1860

Quantities

Concrete: 0.45 m³/m²
 Steel: 71 kg/m³
 Post-tensioning: 24 kg/m³

Firms, offices, etc. involved with this structure

Role	Name
Owner:	Bangkok Expressway Company Ltd. (BECL)
Design:	Kumagi Gumi
Contractor:	Kumagi Gumi, Ch. Karnchang Public Company Ltd./Bilfinger+Berger AG

Relevant literature

Takahashi S, Takagi R: Over-Viaduct Crossing Expressway by Precast Segmental Method. Proc. 13th FIP Congress 1998, edited by Stoelhorst D., Boer G.P.L. Rotterdam 1998, pp. 51-54
 Takebayashi T, Deeprasertwong K, Leung Y: A Full-Scale Destructive Test of a Precast Segmental Box Girder Bridge with Dry Joints and External Tendons. Proceeding of the Institution of Civil Engineers, August 1994, pp. 297-315
 Girmscheid G: Spannbeton-Hochstraße in Bangkok, Planung und Ausführung. Beton- und Stahlbetonbau 88 (1993), Volume 6, pp. 161-166
 Girmscheid G: Entwurfs-und Konstruktionsgrundsätze von extern vorgespannten Straßenbrücken in Segmentbauart, dargestellt am Bangkok Expressway, Thailand. Beton- und Stahlbetonbau 89 (1994), Volume 3, pp. 70-75



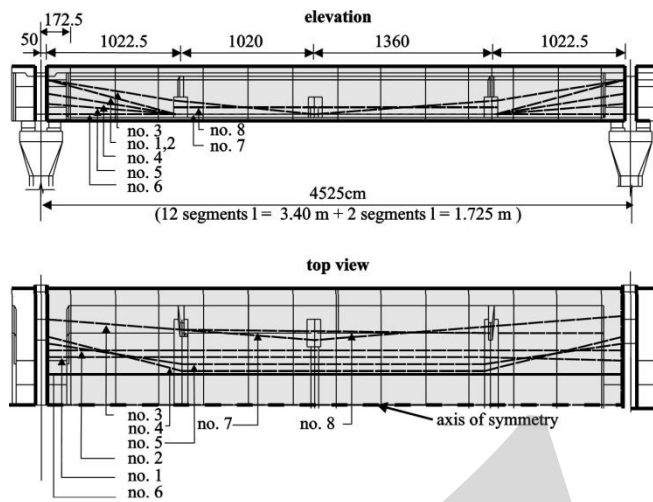


Figure 1: Typical span - tendon layout

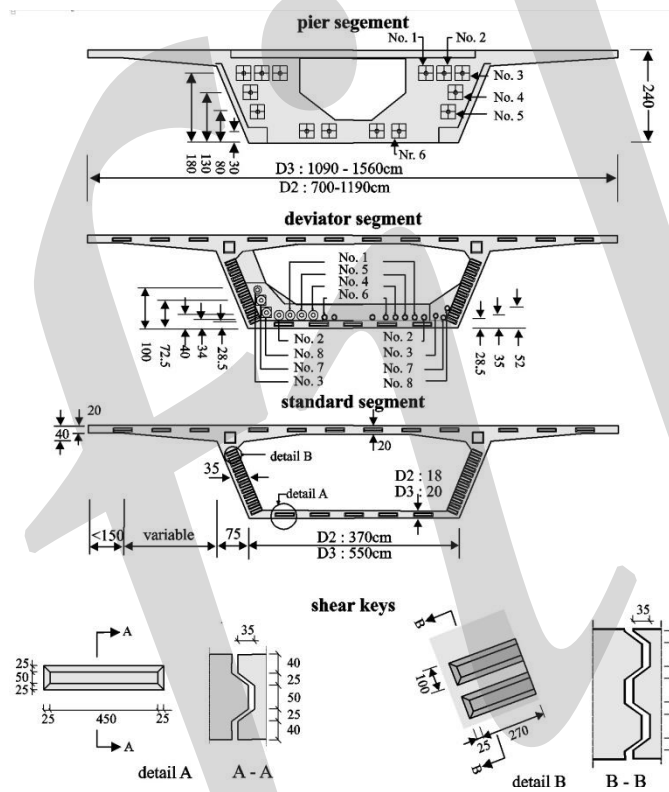


Figure 2: Typical segments



Figure 3: Tendons inside



Figure 4: Assembling of segments

BANG NA-BANG PLI-BANG PAKONG EXPRESSWAY THAILAND BURAPHA WITHI EXPRESSWAY

Completed in: 2000
Location: Bangkok, Thailand
Function/usage: Motorway bridge

Structural type

Hollow box-girder bridge, single spans
longitudinal external prestressed
transverse internal prestressed

Technical information

Match-casting method: short-line
Construction method(s): span-by-span
underslung truss
Joint type: dry

Dimensions

Total length: 54 km
Typical span length: 44.4 m
Typical width: 27.2 m

Building materials

Concrete f_{ck} : 55 MPa
Steel grade:
Post-tensioning: St 1670/1860

Quantities

Concrete: 0.46 m³/m²
Steel: 60 kg/m³
Post-tensioning: 24 kg/m³



Firms, offices, etc. involved with this structure

Role	Name
Owner:	Expressway Authority of Thailand (EXAT)
Design:	Jean Muller Int.
Contractor:	Ch. Karnchang Public Company Ltd./ Bilfinger+Berger AG

Relevant literature

Brockmann Ch, Rogenhofer H: Bang Na Expressway, Bangkok, Thailand - World's longest bridge and precasting operation. PCI Journal, Jan.-Feb. 2000, pp. 26-38
Brockmann Ch: Der Bang Na Expressway, Bangkok: Teil 1: Aufgabe, Bauausführung, Fertigteilwerk. Beton- und Stahlbetonbau 92 (1997), Heft 12, S. 332-337
Fischer O: Der Bang Na Expressway, Bangkok: Teil 2: Tragwirkung, Großversuch, Berechnung. Beton- und Stahlbetonbau 93 (1998), Heft 2, S. 36-41
Shafer G, Brockmann C: Design and Construction of the Bang Na Bang Pli Bang Pakong Expressway. Proc 13th FIP Congress 1998, edited by Stoelhorst D., Boer G.P.L.: Rotterdam 1998, pp. 275-280
Rogenhofer H, Herold W: The Bang Na Expressway - Fabrication of precast concrete boxes in the world's largest segmental yard. FIP Congress, Amsterdam 1998, S. 197-200

Fischer O, Krill A: The Bang Na Expressway, Bangkok: A full-scale loading test of a precast segmental box girder bridge for a 6 lanes of traffic. FIP Congress, Amsterdam 1998, S. 503-506

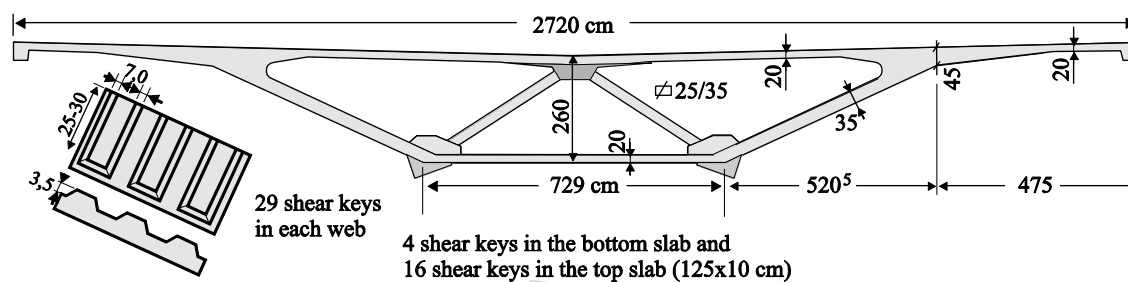


Figure 1: Typical segment

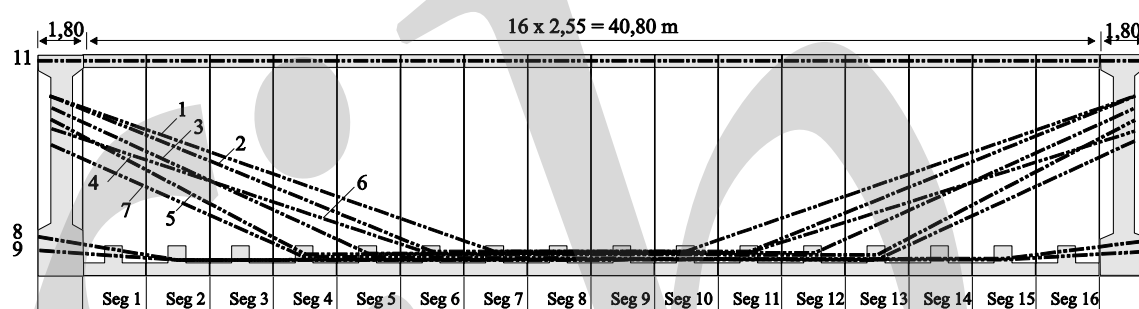


Figure 2: Tendon layout



Figure 3: Shuttering

HARTELFIETSBRUG – SPIJKENISSE CYCLE BRIDGE

Completed in: 2011
Location: Spijkenisse, the Netherlands
Function/usage: Cycle bridge

Structural type

Hollow box-girder bridge, one main span, two side spans
longitudinal internal prestressed
transverse reinforced

Technical information

Match-casting method: long-line
Construction method(s): free cantilever
Joint type: wet (glue)

Dimensions

Total length of bridge: 612 m
Length segmental box girder: 248.7 m
Main span length: 114.0 m
Side span lengths: 67.85 and 66.85 m
Typical width: 4.80 m

Building materials

Concrete f_{ck} : 45 MPa
Steel grade: B 500 S
Post-tensioning: Y 1860 S7
Quantities
Steel: 120 kg/m³
Post-tensioning: 57 kg/m³



Firms, offices, etc. involved with this structure

Role	Name
Owner:	Provincie Zuid - Holland
Design:	BAM Infraconsult bv in cooperation with Ingenieursbureau Oranjewoud bv
Contractor:	Bouwcombinatie BAM-HSM
Post-tensioning:	Freyssinet Nederland bv

Relevant literature

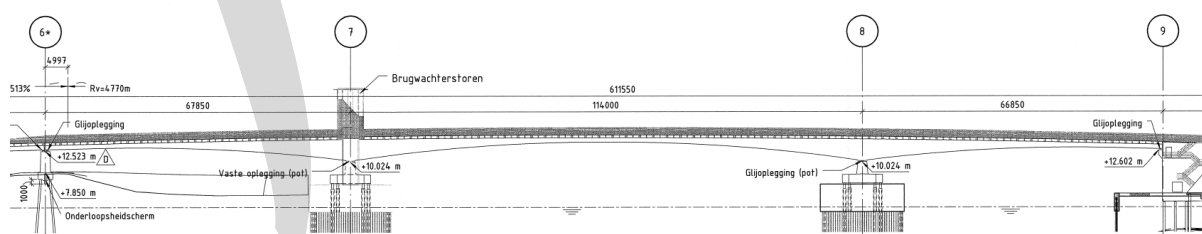


Figure 1: View of precast segmental box girder



Figure 2: Tendon anchor at bottom plate

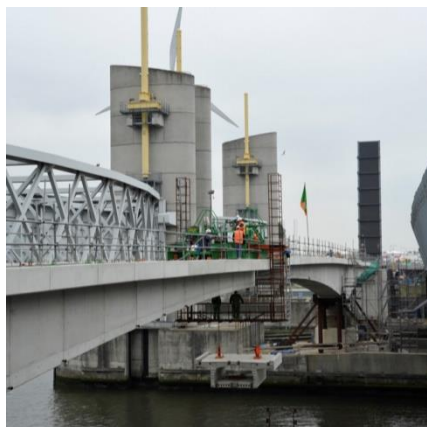


Figure 3: Lifting bridge segment

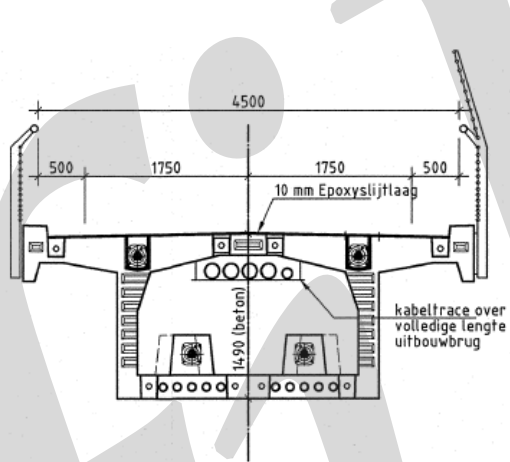


Figure 4 Typical segment at centre span and at pier

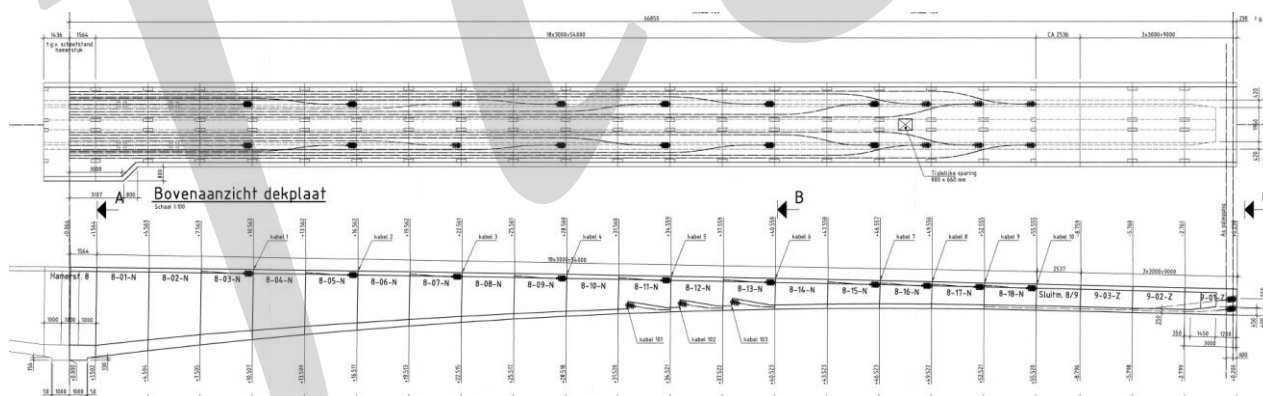
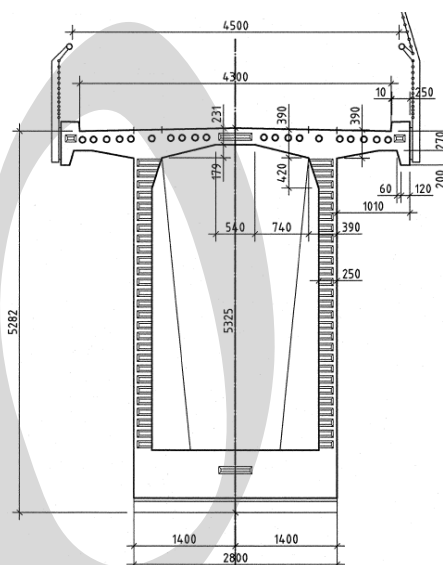


Figure 5: Tendon layout (side span shown)

MARIN VIADUCT: VITORIA-EIBAR HIGHWAY ARLABAN-ESKORIATZA SEGMENT

Completed in: 2008
Location: Guipuzkoa, Spain
Function/usage: Motorway bridge

Structural type

Hollow box-girder bridge, continuous spans, longitudinal internal prestressed

Technical information

Match-casting method: short-line
Construction method(s): balanced cantilever
Joint type: dry

Dimensions

Total length: 2 × 597 m
Typical span length: 96 m
Typical width: 11.3 m

Building materials

Concrete f_{ck} : 40 MPa
Steel grade: B 500 S
Post-tensioning: Y 1860 S7

Quantities

Concrete: 0.79 m³/m²
Reinforcement Steel: 172 kg/m³
Post-tensioning: 40 kg/m²



Firms, offices, etc. involved with this structure

Role	Name
Owner:	Guipuzkoa Provincial Council
Design:	Ferrovial-Agroman S.A. Technical Office
Contractor:	Ferrovial-Agroman S.A.
	Post-tensioning: Tecpresa

Relevant literature

- (1) Pérez-Fadón S., Herrero J.E., Sánchez J.J. "Viaductos de dovelas prefabricadas en Arlabán-Eskoriatza", Proceedings of the 4th ACHE (IABSE) Congress, Valencia (Spain), 459-470, www.e-ache.net, Nov 2008.
- (2) Pérez-Fadón S., Herrero J.E., Sánchez J.J. "Viaductos del tramo Arlabán-Eskoriatza contruidos con dovelas prefabricadas", 2nd Bridge Conference, San Sebastian (Spain) April 27th – 29th, 2010.

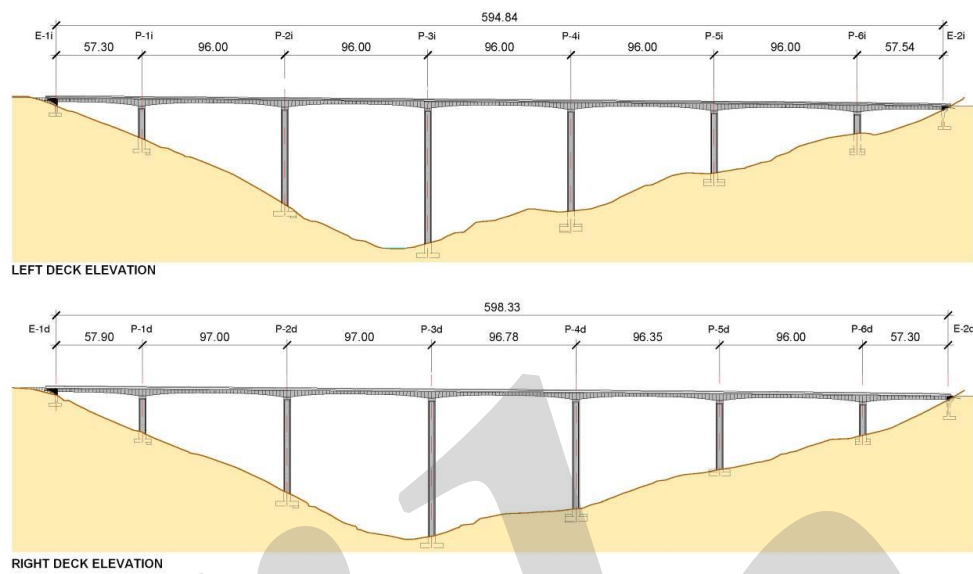


Figure 1: General arrangement

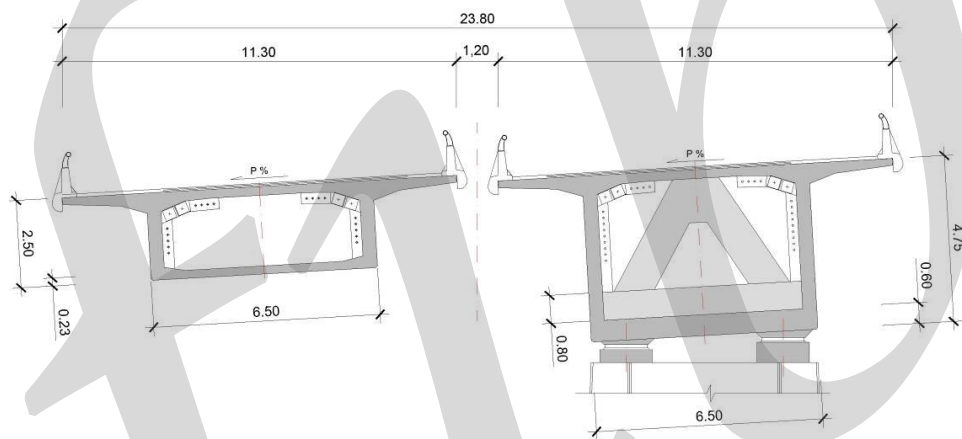


Figure 2: Typical segment

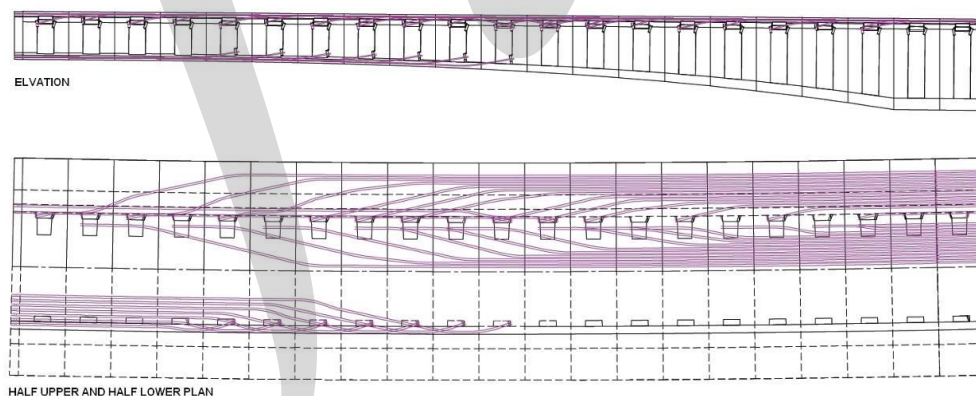


Figure 3: Tendon layout

BATALLA VIADUCT A7 MOTORWAY

Completed in: 2011
Location: Alcoy, Alicante, Spain
Function/usage: Motorway (2 bridges)

Structural type

Hybrid: Precast segmental bridge + composite beams.
Continuous spans, longitudinally internal prestress

Technical information

Match-casting method: short-line
Construction method(s): balanced cantilever
Joint type: dry

Dimensions

Total length: 205 m
Span length: 2×102.8 m (40+T120+40)
Width of the deck: 11.3 m & 13.3 m

Building materials

Concrete f_{ck} : 50 MPa
Steel grade: B 500 S
Post-tensioning: Y 1860 S7

Quantities

Concrete: $0.58 \text{ m}^3/\text{m}^2$
Reinforcement Steel: $168 \text{ kg}/\text{m}^3$
Post-tensioning: $43 \text{ kg}/\text{m}^2$



Firms, offices, etc., involved in the bridge project

Role	Name
Owner:	Spanish Administration
Design:	Ferrovial-Agroman S.A. Technical Office
Contractor:	Ferrovial-Agroman S.A.
	Post-tensioning: Tecpresa

Relevant literature

(1) Pérez-Fadón S., Herrero J.E., Martín-Tereso L. "Viaductos de La batalla y Alcoy Sur", Proceedings of the 5th ACHE (IABSE) Congress, Barcelona (Spain), pon266_2, www.e-ache.net, Oct 2011.

(2) Pérez-Fadón S., Herrero J.E., Martín-Tereso L. “Batalla Viaduct (adapting bridges to challenging places)”, 2012 PCI Convention and National Bridge Conference, September 29 – October 3, Nashville, Tennessee (USA).

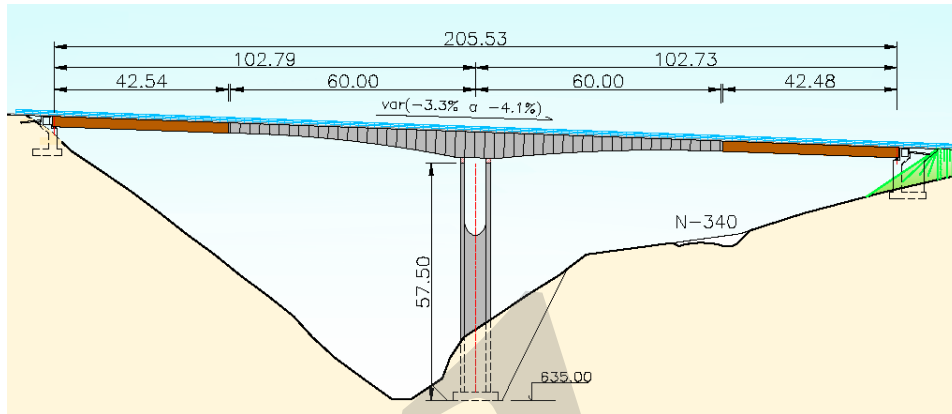


Figure 1: General arrangement

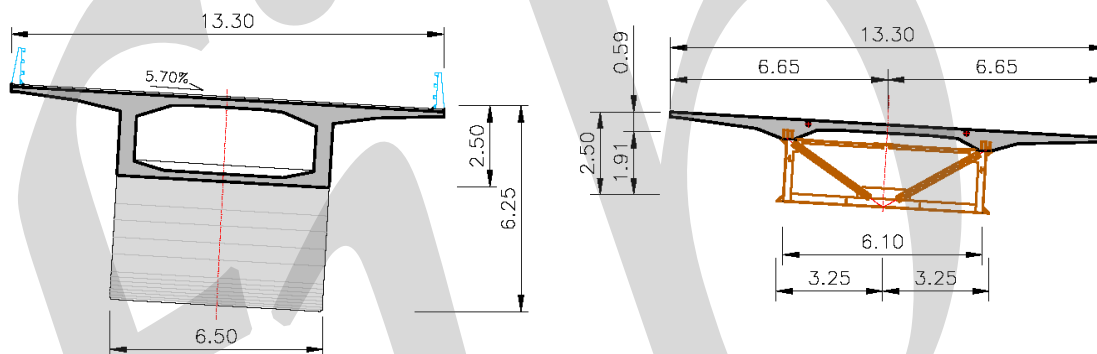


Figure 2: Cross-section of the deck



Figure 3: Bridge finished (before demolishing the provisional piers)

SAN JUAN VIADUCT NEW ROAD ADEJE–SANTIAGO DEL TEIDE

Completed in: 2012
Location: Tenerife, Canary Islands, Spain
Function/usage: Motorway bridge

Structural type

Hollow box-girder bridge, continuous spans,
longitudinal internal prestressed

Technical information

Match-casting method: short-line
 Construction method(s): balanced cantilever
 Joint type: dry

Dimensions

Total length: 205 m
 Typical span length: 98 m
 Typical width: 16.5 m

Building materials

Concrete f_{ck} : 50 MPa
 Steel grade: B 500 S
 Post-tensioning: Y 1860 S7

Quantities

Concrete: 0.62 m³/m²
 Reinforcement Steel: 212 kg/m³
 Post-tensioning: 31 kg/m²



Firms, offices, etc. involved with this structure

Role	Name
Owner:	Canary Government
Design:	Ferrovial-Agroman S.A. Technical Office
Contractor:	Ferrovial-Agroman S.A. Post-tensioning: Tecpresa

Relevant literature

- (1) Pérez-Fadón S., Herrero J.E., Sánchez J.J. “Tableros de dovelas prefabricadas ampliados con puntales de hormigón”, Proceedings of the 5th ACHE (IABSE) Congress, Barcelona (Spain)., 767-768, www.e-ache.net, Oct 2011.
- (2) Pérez-Fadón S., Herrero J.E., Sánchez J.J. “Precast segmental bridges in Tenerife, Canary Islands, Spain: simplicity in constructing a wide deck”, 2012 PCI Convention and National Bridge Conference, September 29 – October 3, Nashville, Tennessee (USA).

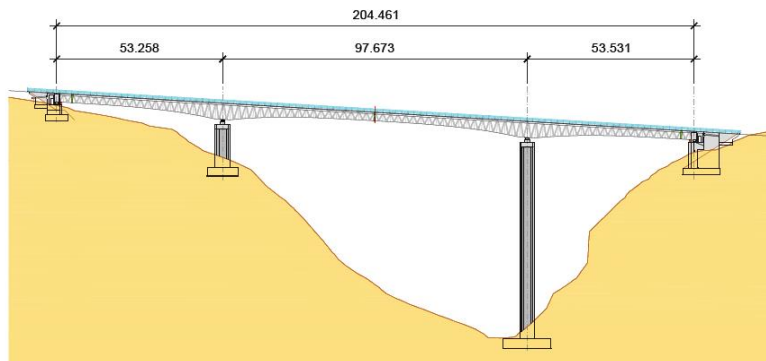


Figure 1: General arrangement

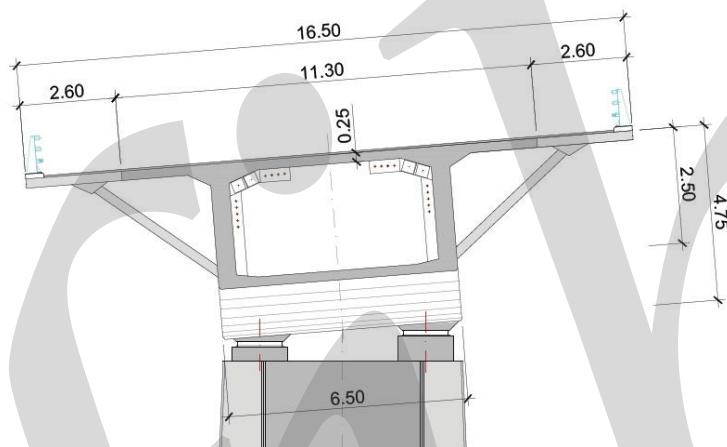


Figure 2: General cross-section of the deck



Figure 3: Preparation of the segment to complete the second phase of the precast

CIEZA VIADUCT

Completed in: 2003 - 2004
Location: Santander, Spain
Function/usage: Arch bridge

Structural type

A 141 m span arch executed with precast segments stressed of the deck. A precast box beam deck placed with launchers. The interface between arch segments is a dry joint.

Technical information

Type: 1.20 m depth precast box girder
Depth: 1.55 m.

Dimensions

Total deck length:

Left road: 233.00 m

Right road: 239.80 m

Deck width: 11.70 m

Deck surface:

Left road surface: 2,735.46 m²

Right road surface: 2,815.02 m²

Span distribution:

Left road: 13.00 + 2×17.00 + 148.80 (8× 18.60) + 2× 18.60

Right road: 13.00 + 2× 17.00 + 148.80 (8× 18.60) + 2× 17.00 + 10.00

Building materials

Concrete f_{ck} : 45 MPa

Steel grade:

Post-tensioning: Y 1860 S7

Quantities

Steel: 133 kg/m³

Post-tensioning: 23 kg/m³

Firms, offices, etc. involved with this structure

Role	Name
Owner:	Ministerio de Fomento
Design:	PONDIO Ingenieros S.L
Contractor:	ESTRUCTURAS Y MONTAJE DE PREFABRICADOS, S.L



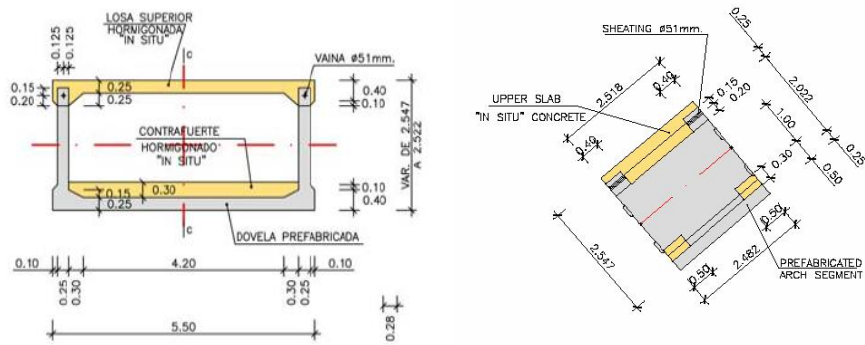


Figure 1: Typical segment

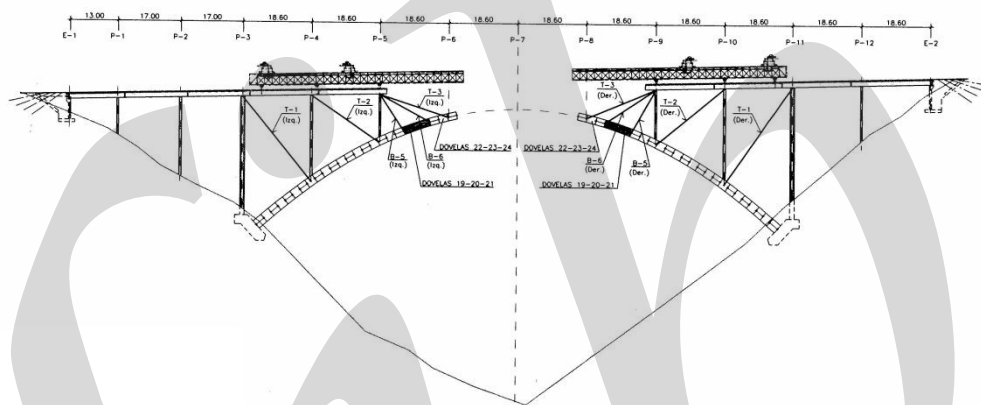


Figure 2: Construction stage

NAVIA VIADUCT

Completed in: 2006
Location: Navia, Spain
Function/usage: Motorway bridge

Structural type

Precast hollow box-girder segments 10.7 m width.
 Cast-in-place cantilevers up to 27 m width.
 Longitudinal internal prestressed + external prestressed in main (arches) spans

Technical information

Match-casting method: Short-line
 Construction method(s): Free cantilever
 Underslung truss epoxy
 Joint type:

Dimensions

Total length: 905 m
 Typical span length: 75 m approach spans
 160 m main spans
 Typical width: 23.1 m approach spans
 27.0 m main spans

Building materials

Concrete f_{ck} : 60 MPa deck
 Post-tensioning: Y 1860 S7

Quantities

Steel in deck: 135 kg/m²
 Post-tensioning in deck: 41.5 kg/m²



Firms, offices, etc. involved in this structure

Role	Name
Owner:	Dirección General de Carreteras. Ministerio de Fomento. Spain
Design:	Jose Ignacio González Esteban (FCC Construcción S.A) David Arribas Mazarrazín (FCC Construcción S.A) María Luisa Castro Sayas (FCC Construcción S.A) Rafael Jiménez Aguilar (FCC Construcción S.A) Cristina Martínez Núñez (FCC Construcción S.A)
Contractor:	FCC Construcción S.A.

Relevant literature

GONZALEZ ESTEBAN JI: Hormigón y Acero – Viaducto de Navia. Oct.-Dic. 2008. Vol. 59 n°250 pag. 07-37 ISSN:0439-5689.

Special mention in 2010 fib Awards for Outstanding Concrete Structures competition 2010

GONZALEZ ESTEBAN JI; JIMENEZ AGUILAR JR; “Construcción y proyecto del Viaducto de Navia”. Congreso ACHE 2008

ÁLVAREZ POYATOS JL.; GONZÁLEZ ESTEBAN JI. “Construcción de Viaductos de Dovelas Prefabricadas por Avance en Voladizo”. I Congreso nacional de prefabricación de la ACHE, Madrid Mayo 2002. pp 279-287.

ÁLVAREZ POYATOS JL.; GONZÁLEZ ESTEBAN JL. “Construcción de viaductos con dovelas pretensadas prefabricadas en la Autovía del Noroeste”. Revista Cemento y hormigón num. 832, Enero 2002 pp 72-92.

GONZÁLEZ ESTEBAN JL. “Construcción de viaductos de dovelas prefabricadas”. Ponencia presentada en las jornadas: La vida de los puentes, San Sebastián 27 de Mayo 2005. pp 393-415.

[GONZÁLEZ ESTEBAN JL. Y JIMÉNEZ AGUILAR R. “Viaducto sobre el río España”. III Congreso de puentes y estructuras de edificación de la ACHE, Zaragoza Noviembre 2005. Vol III pp 2753-2762.

NAVARRO VAQUERIZO J.M. Y PUERTA GUTIÉRREZ F. El viaducto de Navia. Una experiencia en Hormigón Autocompactante. Comunicaciones del Ier Congreso Español sobre Hormigón Autocompactante. Valencia, febrero 2008, pp 703-715.

FERNANDEZ TROYANO L.; IGRLESIAS PEREZ C.; GONZALEZ ESTEBAN JL. Study of long term behaviour of Espiñeiro, Ferreiras, Acebo and Sella viaducts. *fib* Congress, Osaka, Japan, 2002

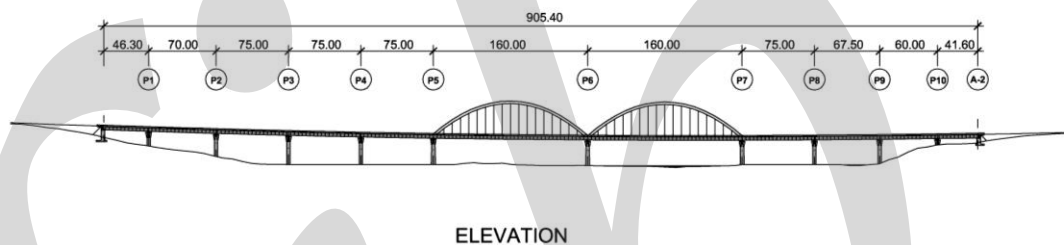


Figure 1: Elevation

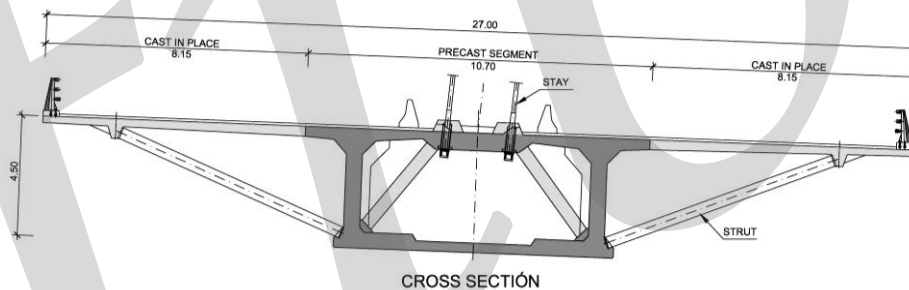


Figure 2: Cross-section main spans

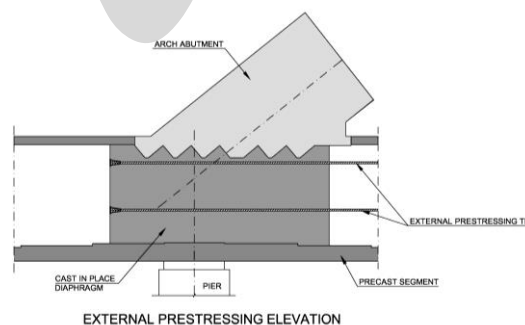


Figure 3: Deck-arch connection

DANUBE BRIDGE II

Completed in: 2013
Location: Vidin, Romania
Function/usage: Motorway – railway bridge

Structural type

Precast hollow box-girder segments 10.9 m width and cast in place cantilevers up to total width. Longitudinal internal prestressed

Technical information

Match-casting method: Short-line
 Construction method(s): Span-by-span
 Underslung truss
 Joint type: epoxy

Dimensions

Total length: 1391 m
 Typical span length: 180 m (extradosed spans)
 80 m (approach spans)
 Typical width: 31.25 m

Building materials

Concrete f_{ck} : 50 to 65 MPa
 Post-tensioning: Y 1860 S7

Quantities

Steel: 248 kg/m² Extradosed spans
 10 kg/m² Approach spans
 Post-tensioning: 44.2 kg/m² Extradosed spans
 32.8 kg/m² Approach spans



Firms, offices, etc. involved in this structure

Role	Name
Owner:	Ministry of Transport, Information, Technology and Communications. Republic of Bulgaria
Design:	Javier Manterola Armisen (Carlos Fernandez Casado S.A.) Antonio Martinez Cutillas (Carlos Fernandez Casado S.A.) Juan Antonio Navarro González-Valerio (Carlos Fernandez Casado S.A.)
Contractor:	FCC Construcción S.A.

Relevant literature

ÁLVAREZ POYATOS JL.; GONZÁLEZ ESTEBAN JI. "Construcción de Viaductos de Dovelas Prefabricadas por Avance en Voladizo". I Congreso nacional de prefabricación de la ACHE, Madrid Mayo 2002. pp 279-287.

ÁLVAREZ POYATOS JL.; GONZÁLEZ ESTEBAN JI. "Construcción de viaductos con dovelas pretensadas prefabricadas en la Autovía del Noroeste". Revista Cemento y hormigón num. 832, Enero 2002 pp 72-92.

GONZÁLEZ ESTEBAN JI. "Construcción de viaductos de dovelas prefabricadas". Ponencia presentada en las

jornadas: La vida de los puentes, San Sebastián 27 de Mayo 2005. pp 393-415.

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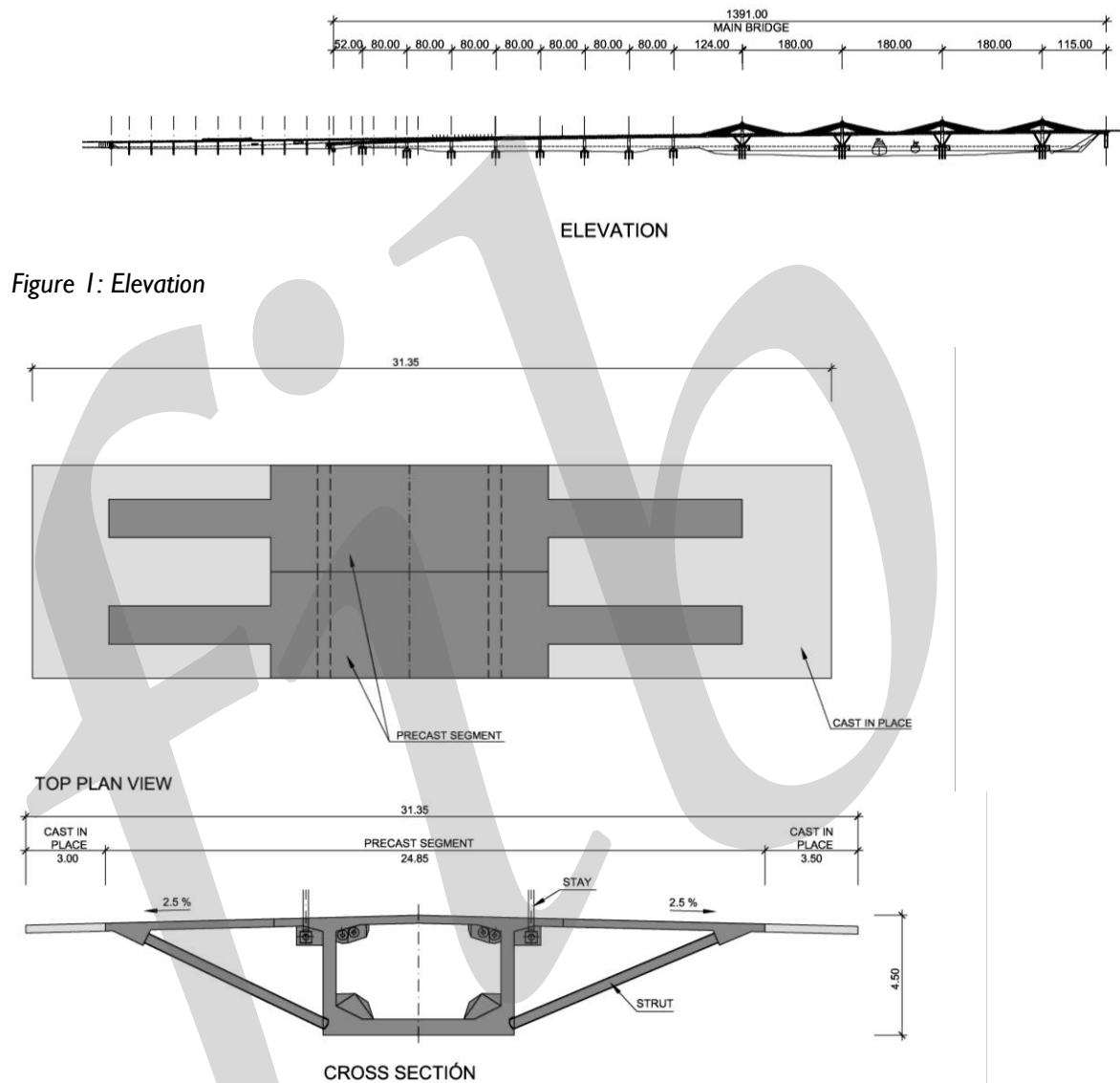


Figure 2: Typical segment

TGV MÉDITERRANÉE, LOT 2 H, VIADUCS D'AVIGNON

Completed in: 1999
Location: Avignon, France
Function/usage: High-speed railway bridge

Structural type

Hollow box-girder bridge
 longitudinal internal prestressed
 longitudinal external prestressed

Technical information

Match-casting method: short-line
 Joint type: epoxy glued

Construction methods

- 100 m span: balanced cantilever with overhead gantry
- 50 m span: span-by-span with crane

Dimensions

Total length: 2 987 m
 Typical span length:
 - viaducts over river: 100 m
 - estacade: 50 m
 Typical width: 12.345 m

Building materials

Concrete f_{ck} : 52 MPa

Quantities

Concrete: 37 860 m³
 Steel: 169 kg/m³
 Post-tensioning: 66 kg/m³

Segments

Unit length: 2.0 m to 4.3 m
 Unit weight: 80 to 150 tons
 Total quantity: 838 units



Firms, offices, etc. involved with this structure

Role	Name
Owner:	RFF (Réseau Ferré de France)
Designer:	SNCF/GTM Construction (Vinci)/Bouygues Construction
Contractor:	JV of GTM Construction (Vinci)/Bouygues Construction

Relevant literature

Bousquet C, Cussac JM, Fauvelle A, Radiguet B: TGV Méditerranée – Viaducts over the River Rhône, The French Technology of Concrete, 13th FIP congress Amsterdam, edited by AFPC-AFREM, 1998, pp.

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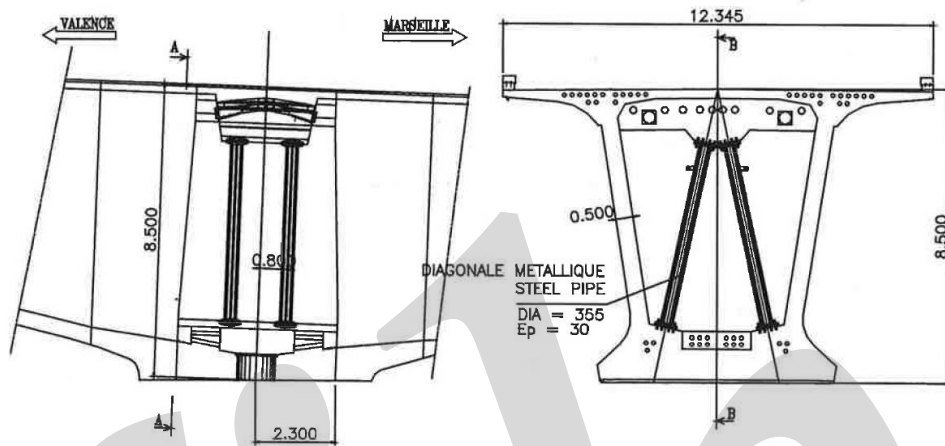


Figure 1: Pier segment

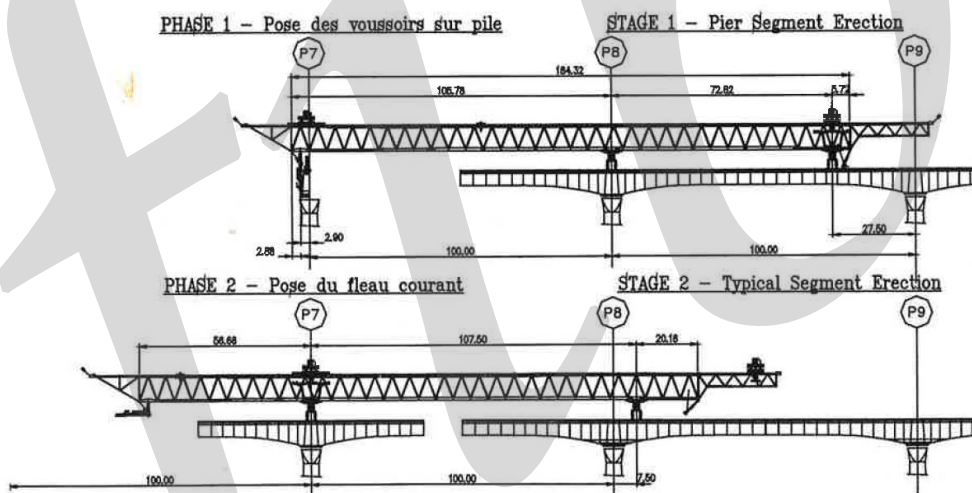


Figure 2: Pier segment erection

SEA TOURS-BORDEAUX – HIGH-SPEED RAILWAY VIADUCTS

Completed in: 2015
Location: 7 viaducts located between
Tours and Bordeaux, France
Function/usage: High-speed railway bridge

Structural type

Hollow box-girder bridge
Longitudinal external prestressed
Longitudinal internal prestressed

Technical information

Match-casting method: short-line
Construction method: span-by-span with
temporary staying mast
Joint type: epoxy glued

Dimensions

Unit length: 446 m to 524 m
Total length: 3 270 m (7 viaducts)
Typical span length: 47 m
Typical width: 12.90 m

Building materials

Concrete grade: C50/60
Steel grade: FeE500
Post-tensioning f_{pk} : 1860 kN/mm²

Quantities

Concrete: 33 140 m³
Steel: 210 kg/m³
Internal post-tensioning: 15 kg/m³ (19T15 tendons)
External post-tensioning: 35 kg/m³ (27T15 tendons)

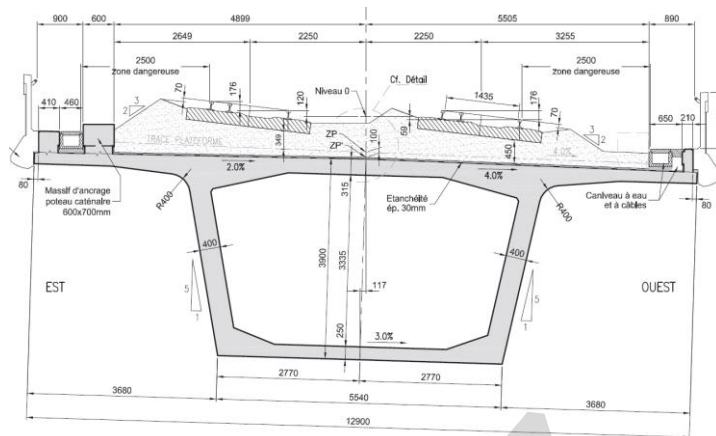
Segments

Unit length: 2.80 m
Unit weight: 65 tons
Total quantity: 1 340 u



Firms, offices, etc. involved with this structure

Role	Name
Owner:	RFF (Réseau Ferré de France)
Design:	ARCADIS/Vinci Construction
Contractor:	COSEA-SGI (JV of Vinci Construction companies)



Typical cross-section

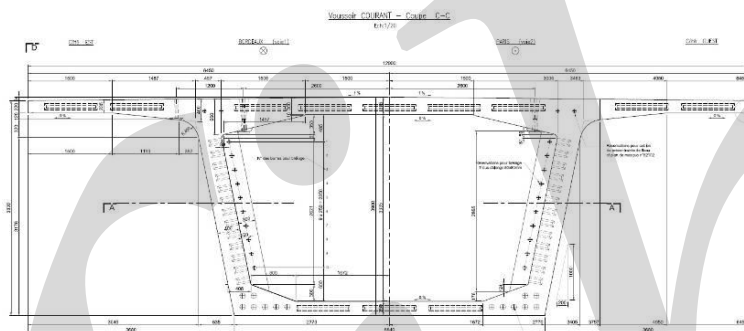


Figure 1: Segment dimensions

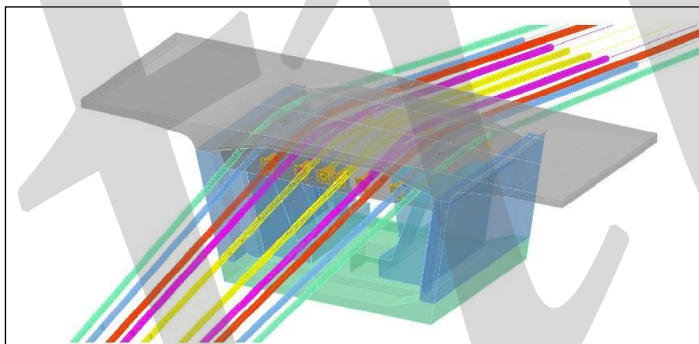


Figure 2: External prestressing layout - SOP detail

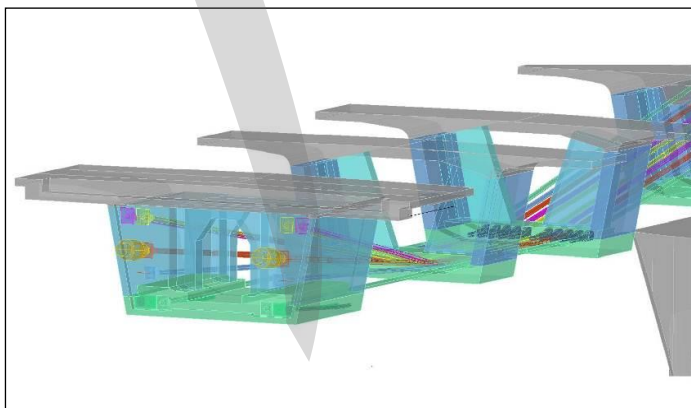


Figure 3: External prestressing layout

EAST TSING YI VIADUCT

Completed in: 2009
Location: Hong Kong
Function / usage: Motorway bridge

Structural type

Hollow box girder bridge
 Longitudinal internal and external prestressed

Technical information

Match-casting method: short line
 Construction method(s): balanced cantilever
 overhead gantry
 epoxy glued
 Joint type: epoxy glued

Dimensions

Total length: 7.0 km
 Typical span length: 55 m – 84 m
 Typical width: ~11 m

Building materials

Concrete f_{ck} : C60
 Steel grade: Fe 500
 Post-tensioning: St 1670/1860

Quantities

Segments: 1983 u
 Spans: 121 u
 Concrete: 0.79 m³/m²
 Steel: 280 kg/m³
 Post-tensioning: ~35 kg/m³



Firms, offices, etc. involved with this structure

Role	Name
Owner:	Highways department
Design:	ARUP HK Limited
Contractor:	Bouygues Travaux Publics, Dragages Hong Kong Ltd, China Harbour Ltd

Relevant literature

Droniou Dominique: Dragon's Tale in "Bridge Design & Engineering", 1st Quarter 2007, n. 46 v.13

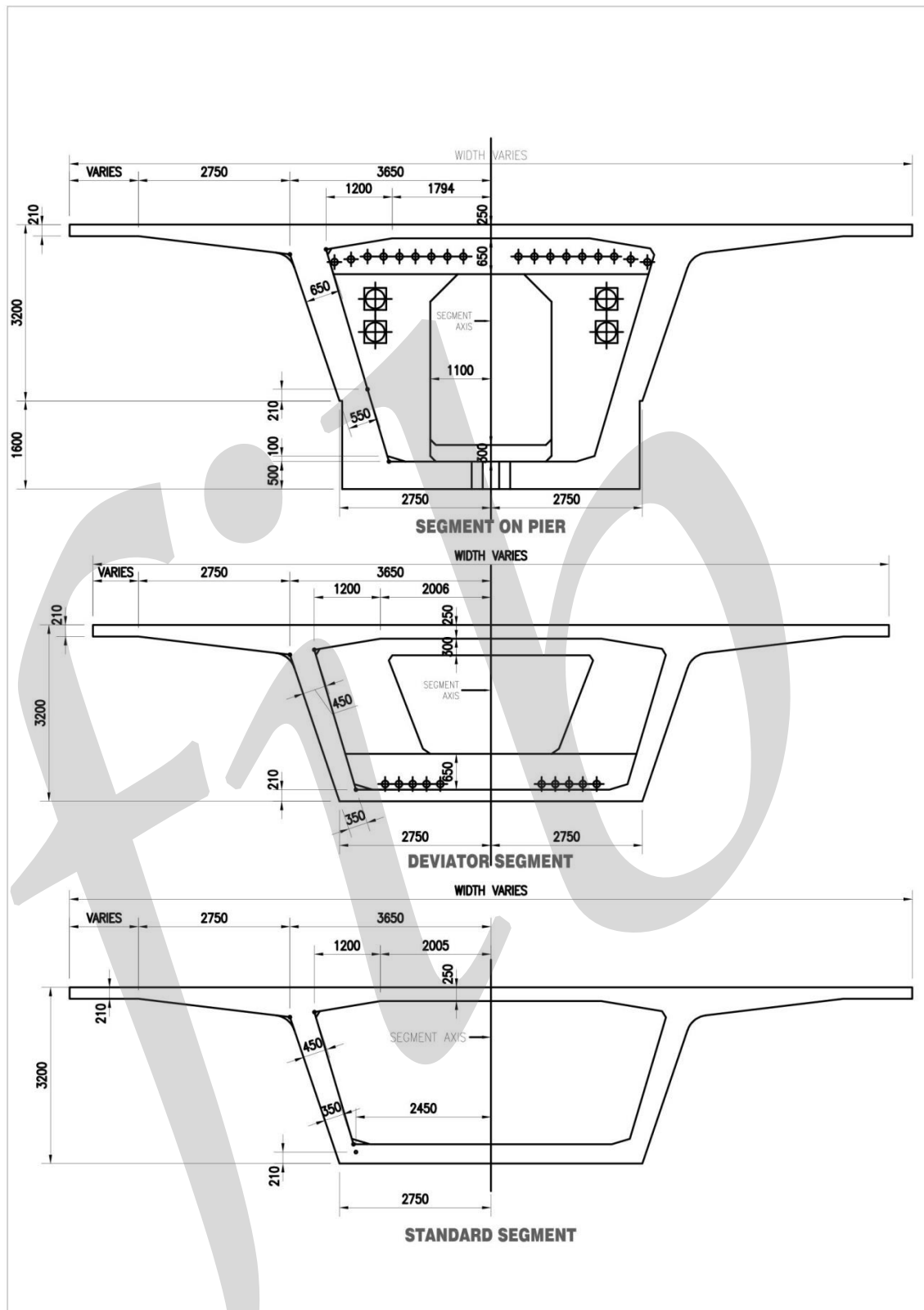


Figure I: Typical segments

FOUR BEARS BRIDGE MOTORWAY BRIDGE ACROSS MISSOURI RIVER

Completed in: 2005
Location: New Town, Mountrail County
 North Dakota, USA
Function / usage: Motorway bridge

Structural type

Precast segmental bridge, box girder, continuous span, longitudinal and transverse (internal) post-tensioning

Technical information

Match-casting method: short line
 Construction method(s): balanced cantilever
 Joint type: epoxy

Dimensions

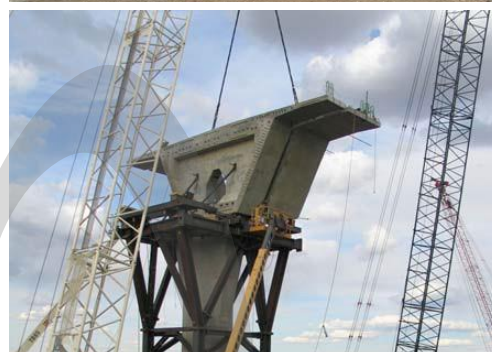
Total length: 1,371.4 m (15 spans)
 Span length: 59.7 – 13 x 96.3 – 59.7 m
 Width of the deck: 12.2 m

Building materials

Concrete f_{ck} : 50 MPa
 Steel grade: B 500 S
 Post-tensioning: Y 1860 S7

Quantities

Concrete: 0.68 m³/m²
 Reinforcement steel: 157 kg/m³
 Post-tensioning: 43 kg/m² (longitudinal)



Firms, offices, etc. involved with this structure

Role	Name
Owner:	North Dakota Department of Transportation (NDDOT)
Design:	FIGG Engineering Group; Bilfinger Berger Design Dept. (temporary works, geometry control)
Contractor:	Fru-Con Construction (Bilfinger Berger Civil, Inc.)
	Post-tensioning: VSL

The Four Bears Bridge, North Dakota (USA), serves as a critical link for the communities of the Fort Berthold Indian Reservation. The previously existing steel-truss bridge was narrow and could not accommodate the existing traffic (functionally obsolete). The new PSB bridge eliminates restrictions, accommodates pedestrians and also possessed aesthetic treatments that met needs of local Native American tribes in the area. Construction on the new Four Bears Bridge started in April 2003. The new bridge design includes two abutments, 14 piers (typical span width 96.3 m), 454 typical segments (length 2.90 m) in the spans between the piers, 28 pier segments (1.68 m) on top of the columns, and 13 closure joints (60 cm) at the

mid-point of each span. In addition to the longitudinal post-tensioning the roadway slab of all segments is prestressed transversally. Pier segments weigh about 93.5 tonne each and include some 36 m³ of concrete and some 5.6 tonne of epoxy-coated rebar.

The piers and foundations are interesting on the new bridge. Up to 20 m deep waters and tremendous ice loads called for a driven pile foundation with piles placed in a conical precast concrete element at water level, which serves as permanent formwork for the pile cap and as a template for the driving work. The side walls of the cones are set at 65 degrees, which will deflect ice upwards, reducing the stress on the piers. Each pier is supported by 13 or 14 spiral-welded pipe piles (typical diameter 0.91 m, length up to 50 m), which are closed at the bottom end and have a wall thickness of 25 mm. With the exception of the centre pile, all piles were installed at an inclination of 1:5. Having piers ranging from 17.20 m to 22.25 m and a height of the haunched deck between 5.05 m (pier segments) and 2.31 m (midspan) the roadway is placed as high as 27.30 m above the water.



Figure 2: Segment production, epoxy coated reinforcement (left); Balanced-cantilever erection of segments



Figure 3: Foundation works for new bridge (existing steel truss bridge to be demolished)

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