PRECAST MONO-BOX DECK OF SPLICED U-GIRDERS & PLANKS: TECHNICAL ASPECTS, COST-EFFECTIVENESS & LESSONS LEARNED (V.33)

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ABSTRACT:

Over the last twenty years precast mono-box and continuous decks have been produced using our specially designed methods. The spliced precast U-girders, of various depths, and upper planks have been used in a variety of bridges, including curved-in-plan.

This results in precast bridge designs that are cost-effective over spans of between 115 and 230 ft. However, road bridges with spans of up to 350 ft. and railway bridges with spans of up to 200 ft. have also been constructed.

The technical characteristics are unique because:

- Experience has allowed the optimization and standardization of the designs of both the U-girders and the precast upper slabs.
- The splicing technique has been made simpler, as short threaded bars or tendons are used, as opposed to tendons that run the length of the deck.
- The necessary on-site wet joints are less than 3 in. in width.

All of these factors have the benefit of making the construction of precast bridges considerably simpler and therefore cheaper as a consequence while optimizing construction times. All these factors have been a driving force behind such mono-box decks becoming standard and gaining a significant share of the market in our country.

These deck solutions are particularly well aligned with the ABC and "Every Day Counts" strategies promoted by the FHWA.

Keywords: *Precast splice girder bridges, Mono-box continuous precast deck, U-Girders.*

1. INTRODUCTION

Since 1991, many bridges have been built in Spain employing spliced precast girders. This technology has become standard and now accounts for a significant percentage of all bridges built, particularly medium-sized bridges using precast girders. With the recession, this type of solution has increased its share of the market—even in a depressed market. This demonstrates even more that it is a very important and cost-effective approach.

Although spliced-girder solutions used to extend precast girder deck spans are well known in the United States and are backed by twenty years of testing and a significant body of in-depth research literature[1,3,4,5,9,10], and although there are numerous examples, many of them quite important[2], this technology has not achieved as large a market share as it deserves. However, we note that our splicing method is remarkably different - simpler, faster, and thus less expensive - which may have contributed to Spain's warmer reception toward this bridge engineering solution with a bright future.

In the following pages we will review the different possible methods of splicing precast/prestressed concrete girders and then explain our particular approach. Some of our most recent case studies will be discussed, looking at bridges with very typical spans to demonstrate standardizable solutions. The reader can thus compare and assess the pros and cons of each option.

2. HISTORICAL DEVELOPMENT OF OUR SOLUTION TO EXTEND SPANS IN BRIDGES USING PRECAST BEAMS[8]

The availability of modern hoisting and transportation means, high-performance building materials, refined methods of structural analysis, and the use of prefabricated and cast-in-place elements side-by-side in the same structure, together with pretensioning and post-tensioning techniques, allows the engineers to design innovative and competitive solutions that enjoy the advantages of both prefabrication and on-site construction techniques.

From the beginning we have developed a splicing method using a U-type girder, from which our proposal has developed progressively. This solution, first offered in 1991, consists of continuous concrete box girder bridge decks of constant or variable depth, either straight or curved-in-plan, composed of precast U-girders that are interconnected monolithically by post-tensioning their end diaphragms. The top slab is then cast in place by pouring fresh concrete over our precast planks design: free-standing lost form planking that becomes a structural part of the bridge. The concrete is poured in two phases over a two-day period: the center is poured on the first day, and the flanges the following day.

In 1992-1993, we began varying the girder depth.

Even though this design incorporates a parabolic bottom line (see Figure 1), the only special component that needs to be provided can be precast in the factory. However, this design is not as complicated or expensive as it appears, and the work can be carried out easily. In these early years, the projects were focused on the design and construction of typical highway overpasses using the same number and dimension of spans, structural models, etc. so that these elements could be reused for decks ranging from 35 to 41 ft. (10.5–12.5 m) in width (Figure 1).

Figure 1: Typical three-span crossing, with parabolic variable depth.

Since 1996 girders have been designed that can be used for curved-in-plan bridges. The girder can support a curved cast-in-place slab or even prefabricated curved-in-plan planks; for a common road radius of over 650 ft. (200 m), straight girders may also be used.

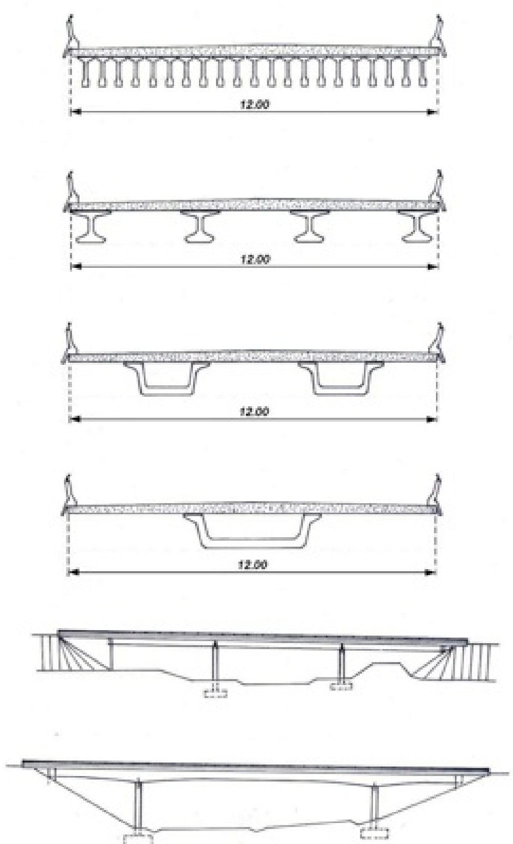

Figure 2: Changes in Cross Section Sections

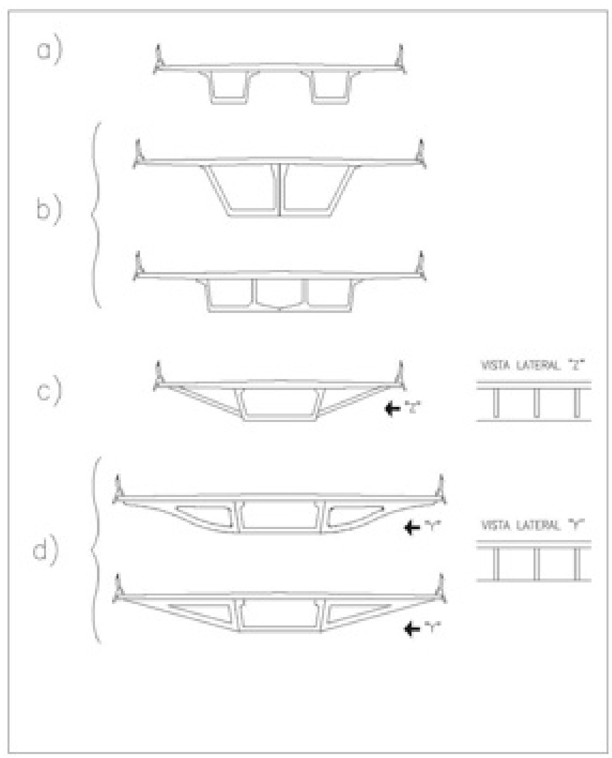

Figure 3: Bridges wider than 42 ft. (13 m).

The development of deck solutions and longitudinal strengthening systems has progressed according to the following models and is shown in the following diagrams (Figure 2).

If decks with widths greater that 42 ft. (13 m) are needed, it can be accomplish using bracing (Figure 3) in order to reach 60 ft. (18 m).

When we look at how this type of deck is built versus how it is done normally, we see significant differences:

- U-girders and not I-girders are used.
- The preference is to build using mono-box cross-section decks.
- The girders are spliced using threaded rods or straight, short post-tensioned tendons, making sure that the joints are always under compression.
- Post-tensioning is usually carried out in a facility, except in recent years in the case of large-span bridges that are more than 215 ft. (65 m) and always in specific zones with tendons which are short and straight.
- In general, all assembly and on-site tasks are simpler in this case, particularly the joint.
- These girders are more complicated to manufacture, but they are manufactured at a facility that is perfectly set up to do so, minimizing errors.
- These girders are more difficult to transport and assemble, to the extent that there are far fewer units involved and they are much larger and heavier.

But the biggest difference is undoubtedly the theoretical concept of the splice, or unification of the girders, which differs significantly: in no case do these projects use one or more post-tensioning tendons that run the entire length of the deck. Below we will offer a theoretical analysis of the origin and justification for the various alternatives that are used.

3. THEORETICAL FOUNDATIONS, CONCEPTUAL EVOLUTION AND TECHNICAL DEVELOPMENT OF A GIRDER SPLICE

THEORETICAL STRUCTURAL BASIS FOR A CONTINUOUS DECK

Figure 4 shows the static model of a bridge with a 3-span continuous deck. The span ratios are very common and functional: L+1.33L+L. The bending moments for a uniformly distributed load are shown; these range from -0.63M to 0.37M, where M is approximately the moment that would correspond to a girder supported at two points.

Replicating these stress calculations in a reinforced concrete girder basically requires additional strengthening of the concrete by reinforcing the areas under tensile stress with steel bars, as is commonly known. Various techniques have been developed to accomplish this:

- The first and oldest is to use passive reinforcement in the form of rebar (Fig. 4, Diagram B). For reasons of cost, these are usually straight and are anchored by straight extension and bonding between the rebar's corrugation and the concrete.

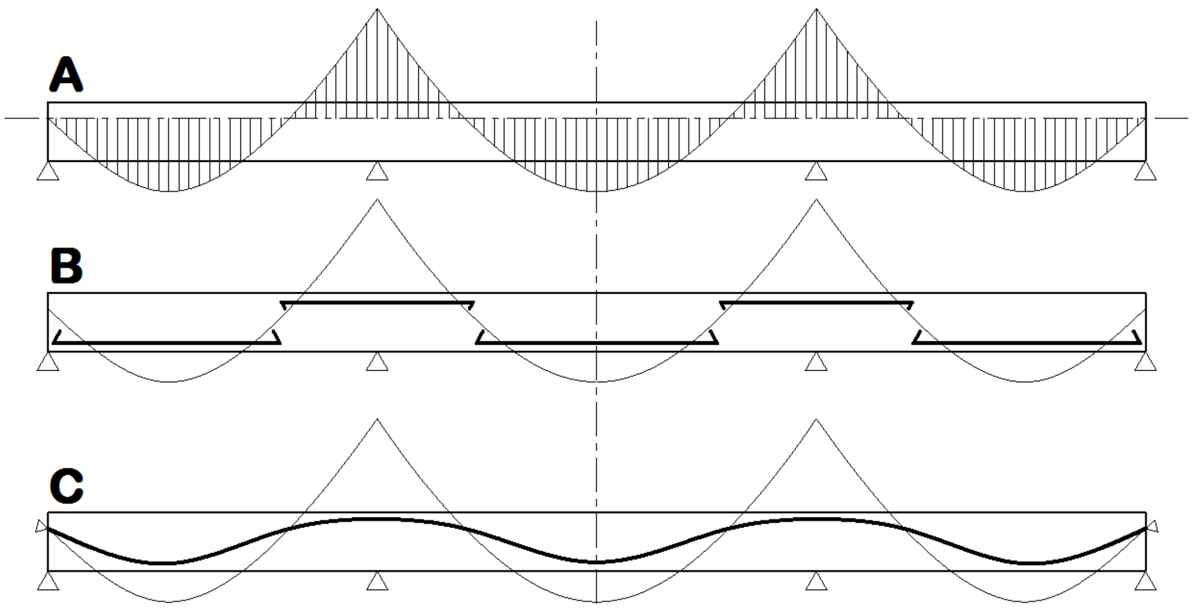

Fig. 4. Model of 3-span continuous deck bridge.

- Later on, the possibility of reinforcing the beam using prestressed/pretensioned strands emerged. This method only works with layouts that are completely straight or designed using straight segments. To minimize costs, straight layouts are preferable, although structurally they are not very effective: there is an excess of material, which is offset because these strands are not jacketed and can be anchored at their ends simply by bonding. But this technique is associated initially with factory-made precast girders, which would not be possible in the case of the 3-span continuous deck we are analyzing.

- Finally there is an option to picking up and counteracting the primary tensile stresses of a concrete girder using prestressed/post-tensioned strands. In this case the tendon is anchored by means of a stop, which must be made using special pieces that are costly. This allows and encourages designs using a single strand for the entire deck, and a parabolic layout (Fig. 4, Diagram C) that runs through the areas of the concrete under tensile stress. The tendon is anchored in its ends, which is a method that is very simple to build and very reliable, even in a bridge cast on site using the duct method.

This is a very brief summary of the basic theories of reinforced concrete and prestressed concrete utilizing pretensioned or post-tensioned reinforcement; its application in the splicing of precast girders to create continuous decks is described below.

ANALYSIS OF COMMON OPTIONS FOR SPLICING PRECAST CONCRETE GIRDERS

Prefabricating the deck for a bridge that uses precast/prestressed concrete girders is now an economically unbeatable solution for certain spans, in the neighborhood of 105 ft. (32 m); and it is also a very durable solution that ages well. But the common and commercially attractive solutions tend to use I-Girders with isostatic spans. Thus, in the case study in Figure 4, a 3-span bridge is normally designed using 3 independent girders supported at two points through piers and abutments, thereby forming a deck with isostatic spans. For this type of bridge, engineering has created various I-section girders over the years, starting with the initial AASHTO girders, which have more recently been optimized with the Bulb-Tee and CAL cross-section girders. It has been found that if pretensioned strands are used for the main family of reinforcement (except for general reinforcement using traditional rebar), more cost-effective girders can be achieved. But when these girders exceed about 130 ft. (40 m) in length, they are no longer very useful due to handling problems. This is why longer spans usually require solutions cast-in-place deck solutions. The option and challenge of splicing precast girders on site was offered as a means to deal with this size of spans, expanding the range of use for precast girder decks.

To find the way to join the classes of precast/prestressed girders already commonly in use is the first goal. This will require strengthening and redesigning the girders for this purpose.

One option would be to modify the layout of the pretensioned strands in the factory, adapting them to the working method that will be most effective when the girder is finally operating together with others in a continuous-deck structural system. (Figure 5, Diagram A). When the difficulties that are associated with such joints are analyzed, it is essential to require the extension and splicing of the different types of reinforcement between two successive girders. This raises complications, because:

(1) The rebar of both girders must be joined, but in a limited space, not more than 5 ft. and typically 2 or 3 ft.

(2) The pretensioned strands must also be joined, which requires special methods.

(3) It is also helpful to transmit loads from the rebar in one girder to the strands in the other, a goal for which there seem to be no reliable solutions, even after many years of studies and trials.

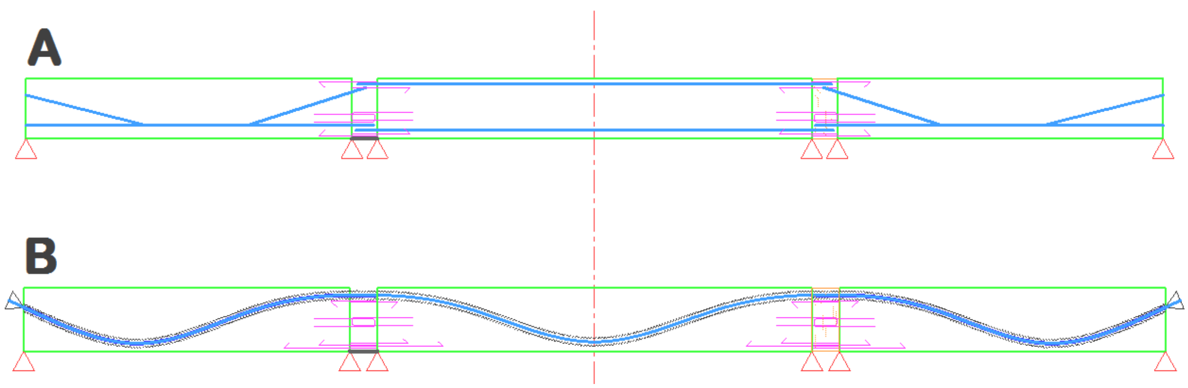

Figure 5: Diagram of 3-span bridge with spliced prestressed and post-tensioned girders

In short, the joint would be complicated and even unreliable: the *"Hoyer Effect"* and the different circumstances that can cause a loss of bond between the steel strand and the concrete are the sources of these problems: the last few feet of both ends of a prestressed/pretensioned girder[a] are dedicated to anchoring the pretensioned strands and

[a] A common benchmark is 1.5 times the girder depth, ranging from 5 ft. up to 7.5 ft. Another

rebar, so the concrete is subjected to strong tensile and compressive forces that are concentrated and combined in directions that are difficult to predict. This subjects those areas of the concrete to strong, localized tensions.

In conclusion, splicing precast/prestressed girders by extending their own reinforcement does not seem to be a reliable, practical or durable solution.

Thus, it was noted that the solution could be found in the reinforcement methods used for cast-in-place bridges, using post-tensioned tendons (Figure 5, Diagram B): this model can be copied, provided that the details of executing the splice on site can be resolved, since the joint between the girders is ensured by a tendon inserted on site into the ducts, exactly as is done with a cast-in-place bridge. This type of concrete reinforcement has been thoroughly tested, even in much more complicated situations such as segmental balanced cantilever bridges.

Indeed, this solution is theoretically correct and well-proven, but it has an initial disadvantage in terms of its cost, since the cost of reinforcement (bars and prestressing strands, pretensioned or post-tensioned) is multiplied because several of them will, in part, no longer act or be necessary when the girders have been spliced.

It is assumed that this additional cost will be offset by achieving comparatively simpler precast solutions for spans where this technique was previously impossible except through the use of segments, a technique that is necessarily expensive even if we only take into account the large amount of special parts needed for the joints and anchors.

Currently I-Girders are the preferred option due to their low cost, as they are the most widespread and easy to manufacture. To put them to this new use, some post-tensioning ducts in their webs must simply be added during the fabrication process (as is done with cast-in-place bridges), which entails a small amount of overhead.

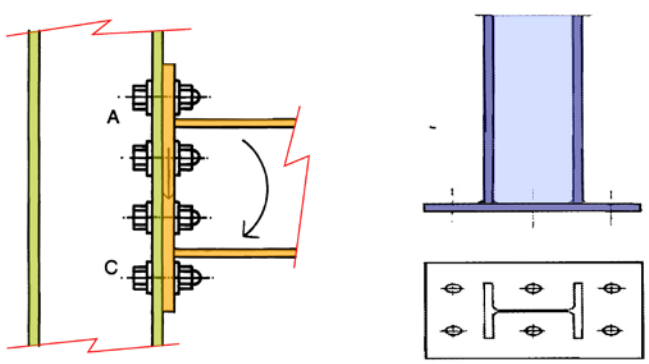

Figure 6: Node joints in steel I-beams.

But in our opinion this does not offset the difficulties and costs that would be introduced by its "I" shape in terms of its subsequent on-site splicing, because its slenderness in one plane

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benchmark indicates that the length at which the strands are anchored by friction is 100φ of the strands, or 3 ft. to 6 ft.

makes this a rather complicated process. As an example, it will suffice to remember that the preferred method when making embedded joints on site using double-T steel girders or columns: is to use a few factory-welded flange plates or auxiliary pieces to bolt the pieces together (Figure 6).

To supplement the post-tensioned strand ducts between the girders to be spliced, and to stabilize them and transmit tensile stresses, it is necessary to make a joint or transition segment between cast-in-place girders that is between 1.5 ft. and 5 ft., using custom-made traditional formwork. This takes time and requires special pieces to hold both girders perfectly immobile; this, together with the consumption of special materials definitely, makes it a costly alternative. These solid bodies can use an inverted T-cap when the joints are made over a pile, but unfortunately in large bridges the sign change points for flexion moments, which are the most appropriate points for splices, are not usually located over piles.

4. ANOTHER OPTION FOR SPLICING PRECAST CONCRETE GIRDERS

As an alternative to all of this, we note that:

• It is possible to develop post-tensioned precast concrete products in the factory.

This statement seems at first contradictory, because precast solutions tend to be associated with pretensioned reinforcement, and cast-in-place solutions with post-tensioned ones. But in fact it is not obligatory and that is what we have been working on.

The geometric design of the post-tensioned tendons is much less rigid than of the pretensioned strands, there are a lot of new opportunities for the precast parts reinforced in the factory with post-tensions tendons, along with an insignificant impact on cost.

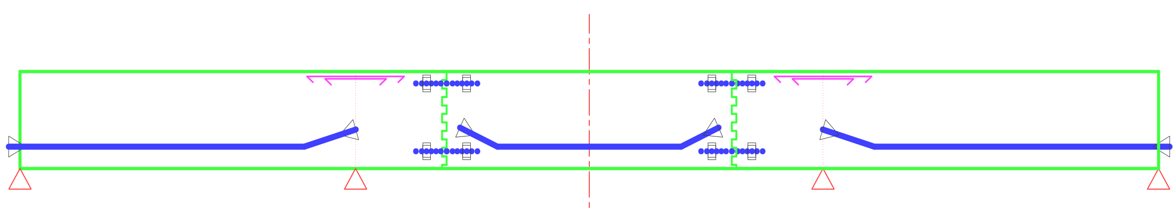


Figure 7: Post-tensioned segments spliced with wet joints and threaded bars.

• Some of the assumptions involved in post-tensioning can be revised or may not be applicable:

The parabolic layout helps to resist shear forces, but perhaps the biggest advantage is to avoid tendon splices anywhere along the deck. Although this is technically feasible, it is expensive and complicated, particularly for small bridges.

Straight layouts are certainly inefficient structurally, but if the tendons are short they are easy to execute, which means that their cost inefficiency will have little impact compared to the benefits that prefabrication can entail.

• U-Girders are easier to join because they have diaphragms; that is, their opposing faces are much larger. In fact, we have never attempted to splice I-Girders.

Because of all this, from the very start, two decades ago, we approached the issue differently:

Using prestressed factory-post-tensioned girders with straight tendon layouts, we would have traditional prestressed girders, but the joint between them is simple, since the anchor for the post-tensioning takes up little space and can be embedded.

The on-site joint is handled using short threaded rods, which join and permanently compress the diaphragms at the ends of the U-girders, as detailed below.

THE SPLICE

The girders are spliced with the help of a provisional cantilever scheme. In some cases temporary supports are needed. The joining of segmental bridges is similar: some elements cross both sections, which creates permanent tensions that compress them. Instead of tendons, certified threaded rods approximately 5 to 15 ft. long and nuts are used. Once the whole assembly is completed, anchoring heads are secured in poured concrete blocks to ensure durability, without any necessary monitoring or maintenance, as the material is fully accredited and has been used for decades in ground anchorage works.

A “wet joint” 1–3 in thick is made every 105 ft. (32 m) instead of every 10–16 ft. (3–5 m) as in segmental bridges: comparing the joints, this joint is less critical, as it can be extended over the whole bridge section or diaphragm (see Figure 15) and not just in the horizontal slabs and webs. Another advantage is the possibility of not joining the girder and the top slab in the same section.

The disadvantage when compared with segmental bridges is that in comparison their segments are shorter, they are often match-cast close to their final position and are transported by a trailer which suffers no load issues. They are also mounted shortly after the production and prestressed at the end of the assembly process. However, the girders are large, are post-tensioned in the factory and evolve throughout the whole process. A designer needs to take the stresses and deformations involved into account, and the precasting factory has to pay special attention to quality control and provide optimal conditions to carry out the whole work successfully.

THEORETICAL BACKGROUND AND REGULATIONS (STANDARDS AND CODES)

The site-specific nature of construction requires design and calculation based on theoretical knowledge, models and tools that were described in an article only recently published in 2000[6]. This means that lots of relevant aspects and details lie outside the current experience of the European standards agencies. There were even certain details where the reliability of the results of the calculations was limited by the available theoretical models: for example, those concerning concentrated heavy loads. The most advanced programs and theories available at that time were then applied in order to clarify these situations.

The analytical model was verified by comparing its results with those obtained in a number of load tests. One of them, in 1996, was considered appropriate with which to compare the theoretical results so as to study the long-term behavior and load-bearing capacity of this type of bridge[7]: A ½ scale two-span model bridge 78 ft. (24 m) long and 13 ft. (4 m) wide was built. It was exposed to a permanent load for 500 days. Its behavior and, above all, the behavior of its splicing was observed and analyzed. This resulted in the unanimous confirmation of the theoretical assumptions: the comparison between the test results and those analytically predicted showed the suitability of the model to reproduce the structural effects of complex interactive time-dependent phenomena.

5. APPLICATION TO A REAL CASE: LAS PILAS BRIDGE (1994)[6]

In 1994 the Las Pilas bridge (Barbastro, Spain) was designed and built; this marked the beginning of the acceptance of this kind of solution as standard. The behavior of stresses and deformations over time was developed in detail in the article for 2011 PCI/NBC[8].

Bridge	Length	Spans	Max. span	Deck width	U-Girder depth
Las Pilas (Barbastro, Spain 1994)	789 ft. (240 m)	5	197 ft. (60 m)	40 ft. (12 m)	Variable

The bridge deck consists of a continuous five-span beam with a parabolic variable depth. The cross-section is a single box girder composed of precast post-tensioned U beams with parabolic variable depth, monolithically connected to a cast-in-place reinforced concrete upper slab. The elevation, plan and cross-section dimensions are shown in Figure 8.

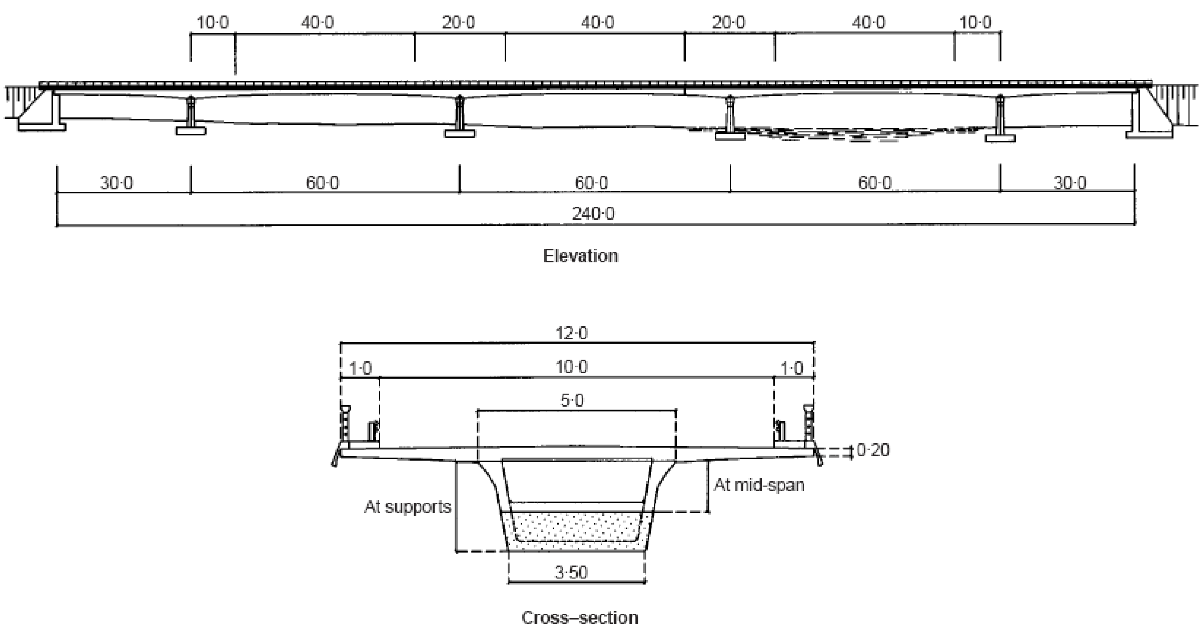


Fig. 8: Drawings of Las Pilas bridge over the Cinca River in Barbastro (Spain) (all dimensions in meters)

All the joints between girders are spaced at a distance of 32´-10” (10 m) from the piers. The central girders have diaphragms at both ends only, while the side beams also have intermediate diaphragms, over the supports. The continuity is provided by means of prestressing bars anchored at the end diaphragms of the connected beams, and by the continuous upper slab.

The characteristic concrete strengths are 7.25 ksi (50 MPa) and 4.35 ksi (30 MPa) for the girders and the slab, respectively. The minimum concrete strength when post-tensioning was 5.8 ksi (40 MPa). The steel strengths are 72 ksi (500 MPa) and 270 ksi (1,860 MPa) for the reinforcing bars and the prestressing tendons, respectively. The tendons were stressed at 80% of the steel strength.

The construction sequence of the deck, once the piers and abutments were built, can be summarized in the following steps:

(a) Step 1: placement of the side girders.

(b) Step 2: placement of the cantilever girders, supported on temporary shores to ensure equilibrium.

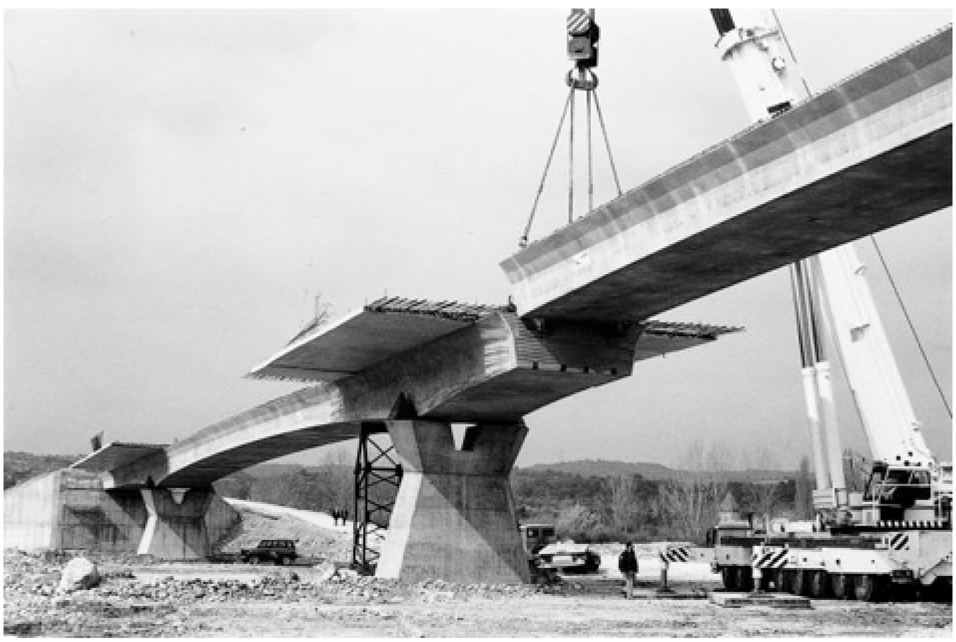


Fig. 9: Las Pilas Bridge during construction.

(c) Step 3: building of the reinforced concrete slab over the side and cantilever girders, in two steps.

(d) Step 4: placement of middle girders.

(e) Step 5: sealing and post-tensioning of joints and building of the slab over the joints, in two steps.

(f) Step 6: removal of temporary shores.

(g) Step 7: building of the upper slab over the middle spans, in two steps.

(h) Step 8: placement of pavement, sidewalks and hand rails.

Figure 9 shows the placement of the central girder. The holes for the continuity bars and the joint shear keys are clearly visible in the picture.

6. THREE RECENT STANDARDIZABLE CASE STUDIES

Three examples of possibilities for standardization, and therefore commercialization, of this type of prefabricated solution are presented below:

- The first is a bridge with 16 equal spans up to 120 ft. (36.6 m).
- The second is the standard solution for bridges with spans of up to 230 ft. (70 m).
- The latter case is structurally and technically similar to the previous, but it stands apart because it was built in difficult conditions, which required an assembly procedure to be designed that would have minimal impact on the land under the project.

Bridge	Length	Spans	Max. span	Deck width	U Girder depth
Lasarte Link. N-I South Branch (2009)	1.640 ft. (500 m)	16	120 ft. (36.6 m)	40 ft. (12 m)	5'-3" (1.6 m)
Pyrenees Highway. Irati Bridge. (Liedena, Spain, 2011)	658 ft. (200.5 m)	4	230 ft. (70 m)	40 ft. (12.15 m)	5’-3” to 8’-6” + 2’-7.5” plank (1.7÷2.6m+0.8)
Egea Road, La Espluga Bridge. (2011)	500 ft. (152 m)	3	200 ft. (60.8 m)	36 ft. (11 m)	63” to 102” (1.7 to 2.6 m)

In all cases one can note the relative simplicity of the structural, technical and assembly solution.

Figure 10: Lasarte Bridge (2009)

This results in a lower cost in all phases of the process and significant savings of time. They are therefore very suitable solutions for the reconditioning of more common decks, which is particularly well aligned with the ABC and "Every Day Counts" strategies the FHWA has promoted for years.

6.1. LINK FOR LASARTE BRIDGE (2009)

This bridge constitutes part of a highway. It is situated on the outskirts of San Sebastián in northern Spain (coordinates 43.27645, -2.0161).

It consists of a 16-span continuous deck, with 10 of them being 111 ft. (34 m) and the longest being 120 ft. (36.6 m), with the remaining ones around 72 ft. (22 m). Other relevant characteristics are that the horizontal radius is significant, with a minimum of 570 ft. (174 m), a slope of 7.2% and banking of 7%.

Up to 5 different types of girders were therefore designed, according to their rebar and prestressed reinforcement, because the 16 girders are not distinguished from each other on the outside, in that they use the same formwork (Figure 12).

The difference is in the reinforcement, which in the case of longest and most reinforced girder combines the 3 types of possible reinforcement (Figure 13):

- Reinforcing bars (rebar).
- Pretensioned strands: 135ø0.6" in the bottom slab and 6ø0.6" in the top headers of the girder.
-Post-tensioning: two tendons (12ø0.6") of post-tensioned reinforcement for additional strengthening of the lower slab (see Figures 13 and 14). These tendons are tensioned and filled with grout in the factory, before transport; It is important to note that in no case is post-tensioning performed on site, which saves time and reduces mistakes, because this is always delicate work.

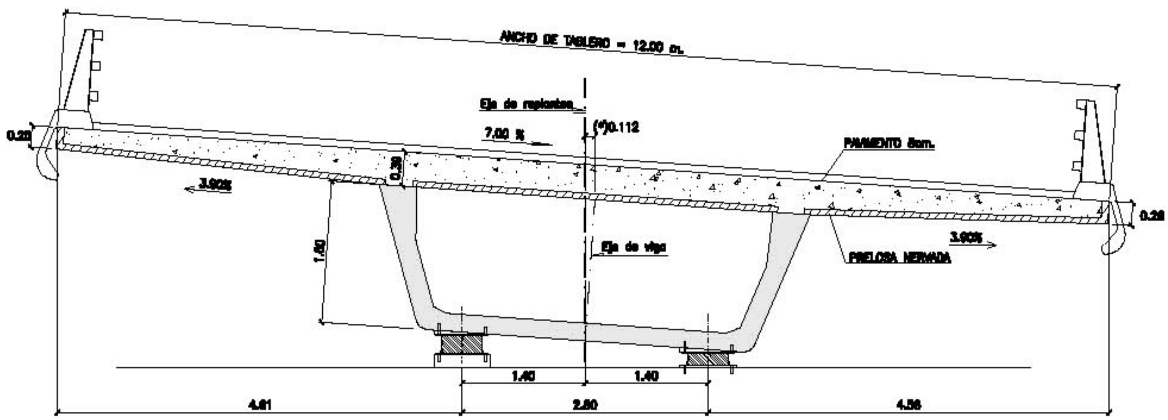
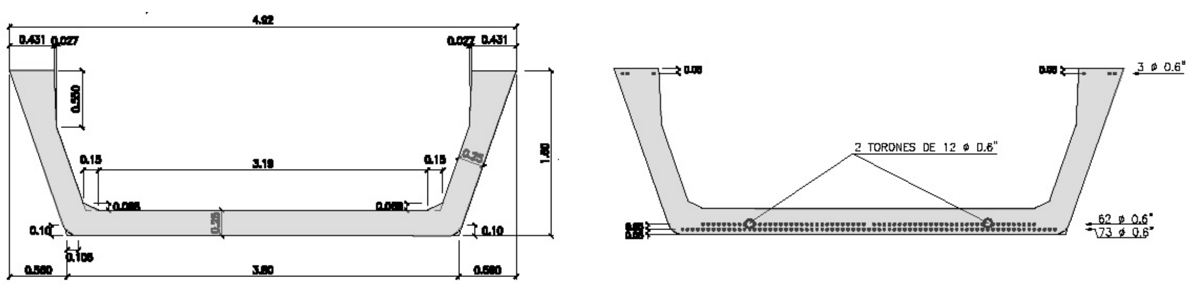


Figure 11: Lasarte Link Bridge. Cross-section (all dimensions in meters).



Figures 12 and 13: Girder details: geometry and reinforcement (dimensions in meters).

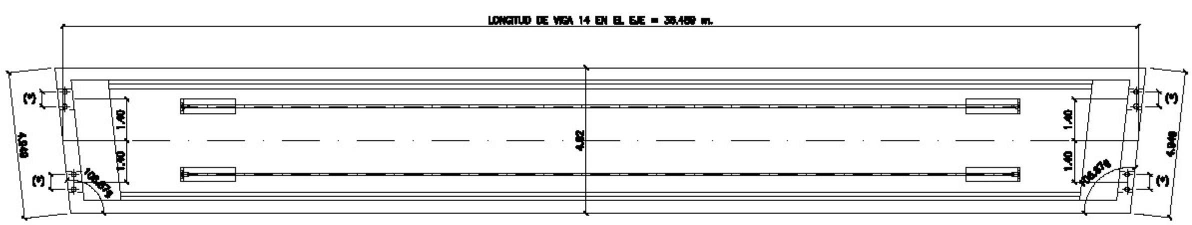


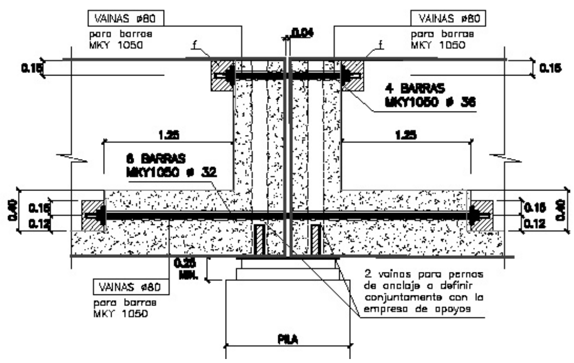
Figure 14: Plan view of the most reinforced girder, with 2 post-tensioned tendons.

This bridge model is particularly simple and fast to erect:

(A) Girders are transported to the site and preferably all assembled without lowering them to the ground from the truck, supporting each girder on a pair of piers. There are no special requirements for order or conditions for progress in this case.

(B) Then all the planks are installed: 200 pc 39 x 8 ft. (12 x 2.5 m) with a unit weight of 7 tons (6.35 MT), which will then comprise the top slab. They are free-standing throughout the process.

At this moment the girders continue to work in an isostatic system, subjected to their own weight and that of the planks. Due to their particular "basin" shape and their weight (169 ton - 153 MT) these beams are very stable. They therefore do not require any type of temporary attachment or fastening, which will give them an extra safety during assembly.



Figures 15 and 16: Diaphragm and Cross-section of a joint between U girders.

(C) The girders are joined using threaded bars, type MK-1050, sheathed. In this case the following units were needed for each joint:

Top: 4 pc. H36 ϕ1⅜" (36 mm) and 4'-3" (1.3 m) long

Bottom: 4 pc. H36 ϕ 1¼" (32 mm) and 13 ft. (4.0 m) long.

Initially, they are bolted down with nuts but not tightened.

(D) Then the 15 vertical joints between each adjacent pair of girders are filled with non-shrink grout, theoretically at a thickness of 1.6 in (4 cm), although in practice it will vary between 0 and 3 in. This is the step that requires the greatest amount of time.

(E) After waiting 24 hours the nuts should be tightened up to 70% of the breaking load of the bar; this is 595 and 750 kN, respectively. Finally the anchor heads are cemented so that the joint will be sealed for lifetime (see Figure 16).

(F) Finally the deck is completed by adding only rebar and pouring concrete on site over the precast planks using self-supporting lost form planking that becomes a structural part of the bridge. This is done in two phases—center area and flanges—spaced only one night apart. The central zone must be poured in two longitudinal sections: the first is the area that takes up 20 ft. (6 m) before and after each pile, and then the rest of the top slab.

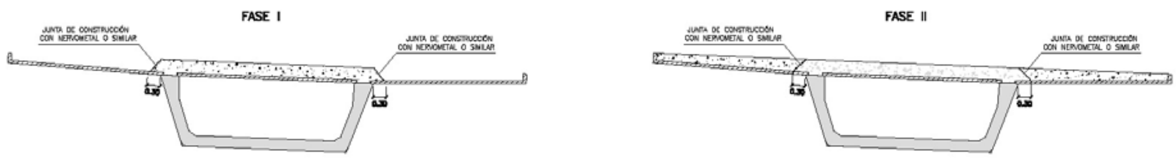


Figure 17: Grouting phases for the top slab, including the self-supporting plank.

COST-EFFICIENCY:

- The section equals: 60 sq ft. (5.57 m^2) of concrete, divided into:
= 18.8 sq ft. (1.75m^2) of girder concrete + 8 sq ft. (0.75m^2) of plank concrete + 33 sq ft. (3.07 m^2) of sited-poured concrete

- The solution with 8 x AASHTO-IV (57”, 0.6” strand and F´c=7.5 ksi) for 12 m wide deck: 70 sq ft. (6.47 m^2) of concrete, divided into:
= 43.8 sq ft. (4.07 m^2) of concrete for 8 girders 26.0 sq ft. (2.4 m^2) concrete for planks and site-poured concrete

- A optimized solution with more modern reinforced girders would be, for example, using beams BULB TEE 37 girders (0.6" strand and F´c = 7.5 ksi). For a 12 m wide deck, it would require 7x BT37-54" : 58.4 sq ft. (5.43 m^2) of concrete divided into:
= 30.0 sq ft. (2.79 m^2) of concrete for 7 girders 28.4 sq ft. (2.64 m^2) of plank concrete and site-poured concrete

As you can see, the differences in terms of material costs are small. The cost of labor to prepare them is nearly the same or comes out in favor of fabricating one large girder in the factory, versus 7 smaller units.

The only difference might be the cost of the formwork, which in the case of the new girders would represent an initial investment; but only in this case it require 16 uses, which means it would be amortized quickly.

Items that increase costs:

- Transport for 16 girders weighing 169 ton, 120 ft. long and 16 ft. wide (36.6 m x 4.92 m) plus 200 39 x 8 ft. (12 x 2.5 m) planks weighing 7 ton (6.35 MT) in groups of 3-4 pc per load, vs. the case using BT-37-54: 112 girders weighing 38.7 ton, 120 ft. long (36.6 m),

-It requires greater tonnage cranes capable of handling 169 ton girders.

The application for this type of bridge is on long elevated highway viaducts, as shown in Figure 18. The advantages and savings are seen in:

- Shorter assembly times, due to the greater ease.
- Safety: Less time spent on the construction site, limited land occupancy.
- No need to build 15 hammerhead piers.

- Maintenance: Dapped-end girders are difficult to preserve in the area around the supports. They require less support apparatus.


Figure 18: Typical elevated highway, on the outskirts of Dallas.

6.2. BRIDGE OVER THE IRATI RIVER IN LIEDENA (2011): 70 m SPAN

The bridge is a type case for major spans of up to 280 ft. (85 m).
It makes use of all the theoretical concepts, structural developments, technical details and manufacturing and assembly as the previous bridge, but expands on them.

Bridge	**Length**	**Spans**	**Max. span**	**Deck width**	**U Girder depth**
Pyrenees Highway. Irati Bridge (Liedena, Spain, 2011)	658 ft. (200.5 m)	4	230 ft. (70 m)	40 ft. (12.15m)	5’-3” to 8’-6” + 2’-7.5” plank (1.7÷2.6 m + 0.8m)

The spans in this case are somewhat irregular, since it is a bridge custom designed to a specific location that required bridging a river. When a bridge with these features is designed, we divide it up into pieces that are as large as possible, making sure they are structurally suitable for both transportation, and for assembly and final usage. It must also comply with all legal and functional constraints on geometry and weight. The following dimensions are typically used:

Length: 115 ft., maximum 130 ft. (35 m, maximum 40 m)
Width: 15 ft., maximum 18 ft. (4.5 m, maximum 5.5 m)
Weight: 175 tons, maximum 220 tons. (160 MT, up to 200 MT).
Height: 8 ft., maximum 10 ft. (2.5 m, maximum 3.0 m)

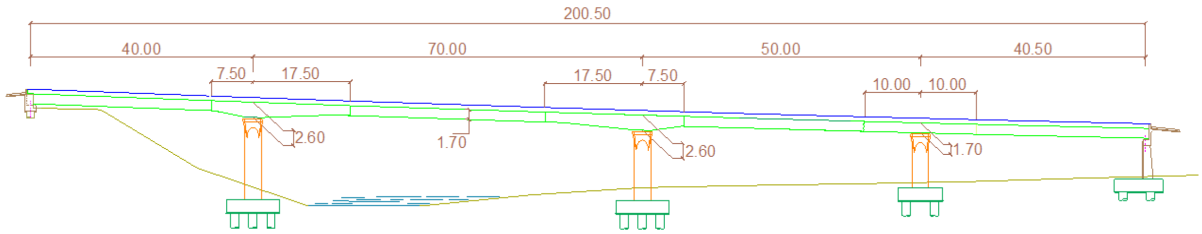


Figure 19: Irati River Bridge. Elevation (all dimensions in meters).

In this case 7 girders for 4 spans, measuring 115, 2x 107, 100, 2x 73 and 65 ft. (35.2, 2x 32.5, 30.5, 2x 22.5 and 20 m) in length were used. The maximum width of each was 15 ft. (4.56 m) and the maximum height 8'-6 "(2.6 m).

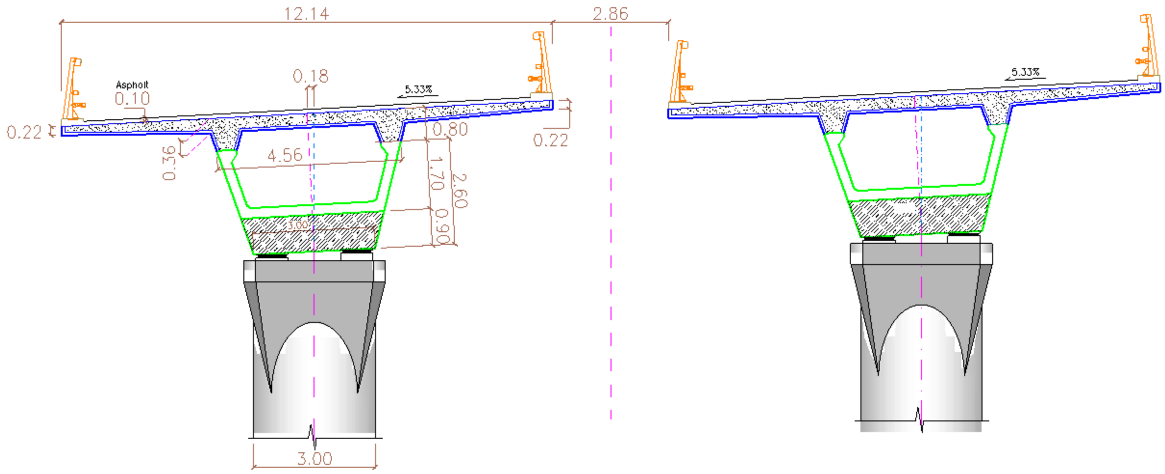


Figure 20: Irati River Bridge. Deck cross-section (all dimensions in meters).

This bridge is clearly a segmental structure, thus requiring a more orderly assembly broken into more phases. But this does not mean that it is much more complicated to build; in fact, taking into account the significant length of the deck spans, it is especially simple, fast and therefore cheaper compared to the more common alternative, a bridge built in successive cantilevers.

This case extends the concepts and techniques set out above to their limits:

• The girders have been reinforced in the factory using only post-tensioned tendons.

For example, the pile-supported girder that is 73 ft. (22.5 m) in length, with 4 strands of 18ϕ0.6” on the top of its flanges and along its entire length, and the center girder with the largest span, 115 ft. (35 m) using 12 strands 18ϕ0.6” but using a U-girder for the bottom slab, with lengths ranging from 56 to 115 ft. (18-35 m).

• The joint must handle very large compressive forces, which can only be achieved through post-tensioned tendons combined with threaded rods.

In this case, the most important joint requires 6 ϕ1¼ (32 mm) bars and 6 18ϕ0.6” strands of between 40 ft. and 85 ft. (12 to 26 m). The layout is essentially straight, so the work on site is quite simple.

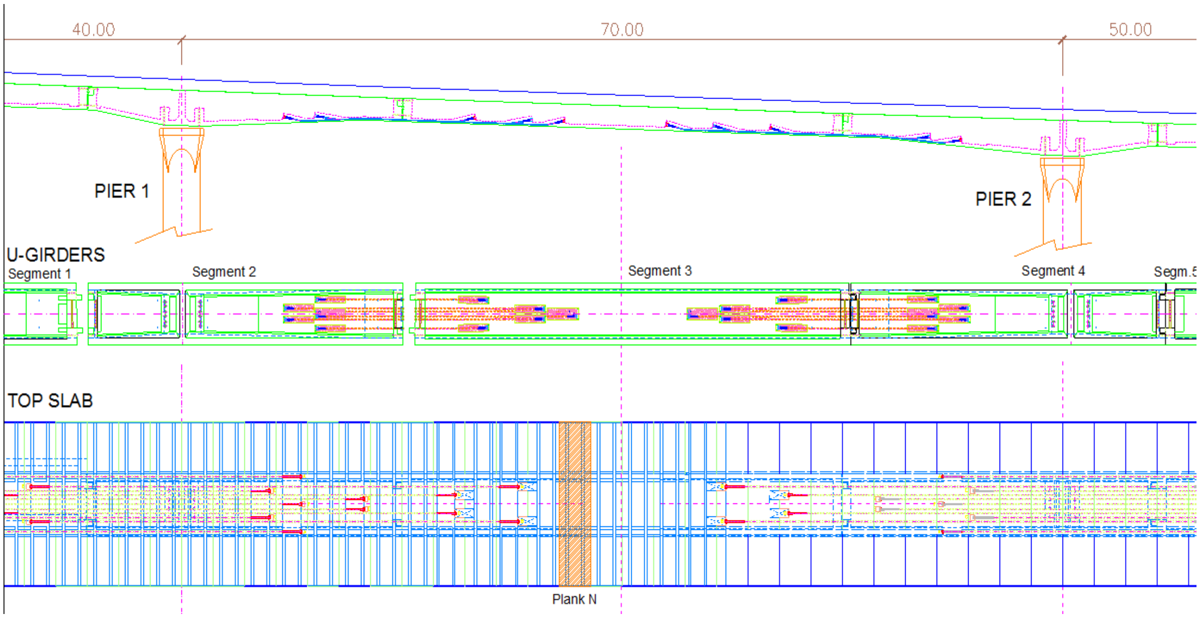


Figure 21: Irati River Bridge. Detail of areas reinforced with post-tensioned tendons.

• It will also be require the joint to be extended to the top slabs themselves, using tendons shown in plan view (Figure 21) and in cross section (Figure 22).

• The pile-supported girder is 8'-6" (2.6 m) deep but requires a reach of 11'-2" (3.4 m).
This is solved by using an upper plank with a new design, which we call "enhanced," as it provides extra depth to the total cross-section of the bridge.

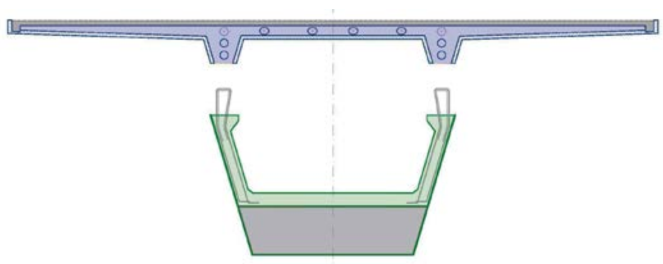


Figure 22: Irati River Bridge. Breakdown of precast elements that make up the deck.


Figures 23 and 24: Irati River Bridge during construction (2011)

Another thing to note is the temporary attachment or securing of the piece on the pile during assembly, until the splice with the adjacent girders is completed and all segments are operating together as a single girder.

There are several options for this, but the most functional ones either use a temporary tower, if the height or other conditions permit, or as in this case, by anchoring the piece to the pile (Figure 23).

Then the adjacent girder will simply be supported on it through simple concrete studs in the girder diaphragm, as can be seen in the example in Figure 25. In comparison, the most widely-used solution is to use temporary special steel pieces with tensioners to secure the two girders to one another while the joint is being made.

Figure 25

6.3. "LA ESPLUGA" BRIDGE (Dec-2011):
QUICK DECK ASSEMBLY WITH NO IMPACT ON THE VALLEY BOTTOM

This last example is structurally and technically similar to the previous one. It is distinguished by being located in difficult construction conditions, a valley up to 230 ft. (70 m) deep, which limited the use of falsework needed to support the formwork for a cast-in-place deck. This led us to redesign the deck as a precast solution, requiring us to also design an assembly technique that would not impact the valley bottom below the bridge.

Bridge	**Length**	**Spans**	**Max. Span**	**Deck width**	**U Girder depth**
Egea Road, La Espluga Bridge. (HU, Spain 2011)	500 ft. (152 m)	3	200 ft. (60.8 m)	36 ft. (11 m)	63” to 102” (1.7 to 2.6 m)

The 5 girders were assembled using incremental launching equipment, including pile-supported pieces, which is a new development. These pieces were anchored temporarily to the pile heads.

Both the speed and the safety achieved during assembly were exceptional, something which is difficult to describe on paper but can be better illustrated via a video which can be seen at the following link: http://www.youtube.com/watch?v=LzGO9l-cWEA.

In 6 weeks it was possible to comfortably assemble this 18,000 sq ft. (1,672 m^2) deck, and 2 more weeks were all that was needed to leave it fully completed, including equipment, ready for commissioning.

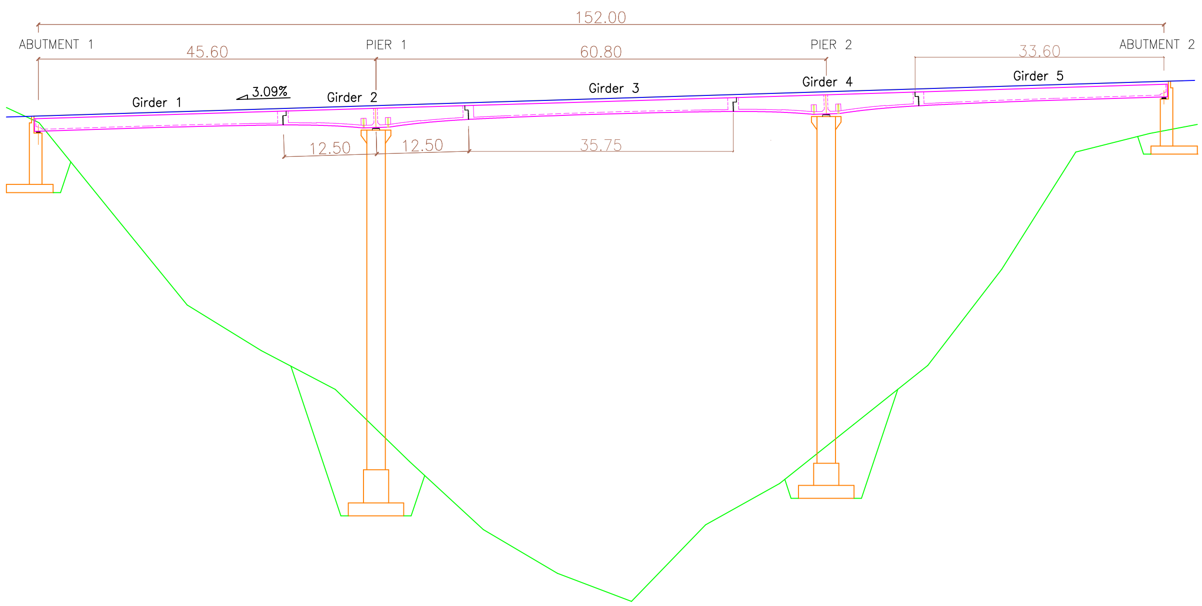


Figure 26: "La Espluga" Bridge: Elevation (all dimensions in meters).

Figures 27 and 28: "La Espluga" bridge during construction (2011).

This assembly solution will be very useful projects need to be assembled very quickly and without affecting any roads that may be situated below the bridge. It also conforms well to the ABC and "Every Day Counts" strategies promoted by the FHWA.

7. LESSONS LEARNED

First we must credit all the lessons and advice provided by colleagues who are working on these solutions, particularly at CALTRANS[9,10]. This is generally due to the fact that these are clearly evolving structures, so that their assembly process require detailed study, ordering and execution plan that must be followed strictly.

The calculations for all tensile states that the structure undergoes is complicated, because they combine parts of very different ages; this leads to a particularly significant redistribution of tensile forces that must be carefully considered in the calculations. Thus, evolving structures require a high degree of planning when considering the assembly scheme, the components, and the final structure. They require a strict assembly schedule, because during the first few weeks it is a "live" product and therefore elastic and plastic movements need to be calculated. In some cases it is necessary to provide temporary tendons for the transport and assembly phases. Most of the above criteria are also shared with segmental bridges.

In our case at first we suffered the consequences of the so-called "Hoyer effect," and in general we had problems with bonding of the pretensioned strands; in fact, common experiences and considerations regarding bonded strand splicing needed to be expanded.

In 1997 at UPC (Universitat Politècnica de Catalunya, Barcelona), lead by Professor A.Mari[7], these joints using threaded rods were studied and tested in order to look at their interaction with other types of reinforcement, as well as the strong, concentrated loads acting on the concrete in these anchor zones, and the evolution of all this over time.

Today, hundreds of bridges we have designed are in operation, with no particular problems. Some have been in use for more than 15 years, demonstrating that these issues are no longer a problem so long as they are adequately taken into account.

8. CONCLUSIONS

The use of precast and cast-in-place elements and pretensioning and post-tensioning systems together in the same structure allows us to build precast continuous-deck bridges with the help of today's powerful means of hoisting and transportation.

The construction of mono-box precast decks using U-girders with constant or variable depths, which are either straight or curved in plan and monolithically connected by means of our practically imperceptible vertical joints, offers an aesthetically pleasing appearance that is very similar to that of cast-in-place bridge decks.

After two decades of experience we know that it is reliable and opens new possibilities, provided that the calculation techniques are mastered well. These advances have been made possible at the design stage, by the use of analytical methods that are capable of taking into account the non-linear and time-dependent behavior of concrete and steel structures, coupled with the effects of segmental construction.

In our opinion, this is a product with a bright future, especially with regard to certain repetitive bridge construction features, such as overpasses or highway crossings and medium-sized spans 115 to 230 ft. (35– 70 m), which are becoming very frequent.

- For spans between 115 and 150 ft. (35–45m) (yellow band in Figure 29) these solutions are the most aesthetically suitable, and their simplicity facilitates and shortens much of the work on site, without increased requirements to provide transport vehicles and cranes capable of handling heavier items.

Particularly with spans of 125 ft. (38 m) and larger, they are already very competitive alternatives compared to regular isostatic girder decks.

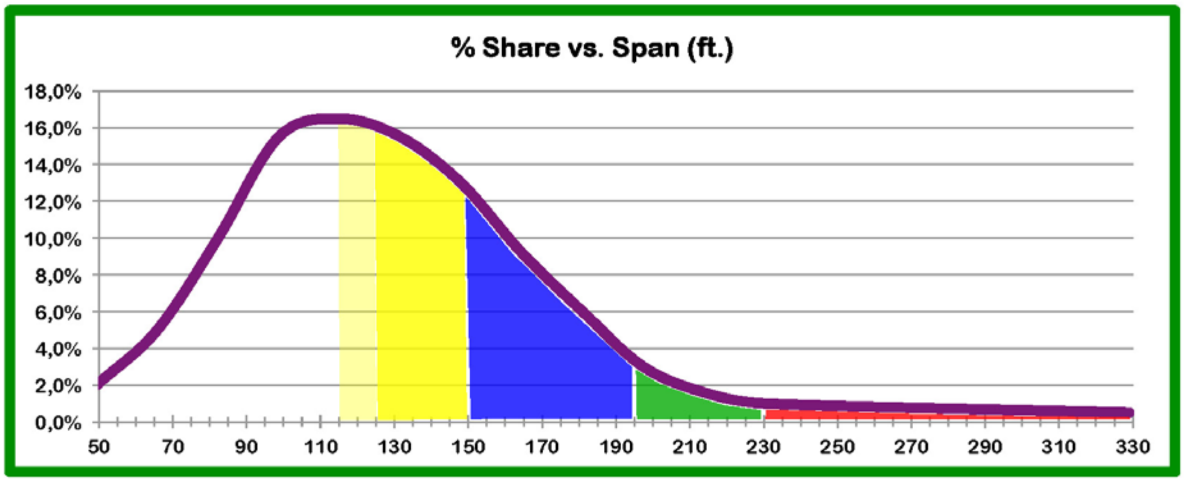


Figure 29: Breakdown of bridges built by span width (tentative percent).

• Between 150 and 230 ft. (45–70 m) (blue and green bands) in addition the advantages above, they are very interesting in comparison with the customary bridge solutions for this span size. Although starting from 195 ft. (60 m) (green band) the design becomes more complicated, it is still much simpler than the competing approach—cantilever bridges.

All over the world, a very high percentage of commonly constructed bridges are found in these two span ranges, and thus we have a choice with a great future for deck renewal, minimizing inconvenience to the public and capable of dealing with all types of conditions in the physical environment: combining both easy erection works and short construction times, which can also be improved with a launcher which minimizes the effects on existing roads and therefore traffic.

Furthermore, the launcher could also be used to deconstruct previous decks after the deck has been cut into segments during redecking works.

It therefore conforms very well to the ABC and "Count Every Days" strategies promoted by the FHWA.

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