

DESIGN AND CONSTRUCTION OF DOUBLE COMPOSITE STEEL TUB BRIDGES IN SPAIN



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BIOGRAPHY

Juan Sobrino, Ph.D., P.E., is the founder and President of PEDELTA Inc. based in Coral Gables, FL, with offices in PA, Spain and Latin-America.

He has been involved in the concept or design of more than 500 bridges worldwide and has promoted the use of advance materials in bridge construction and aesthetics as a key value as important as efficiency, economy and constructability in bridge design.

Juan is collaborating as a part-time docent on structural analysis and conceptual bridge design with the Technical University of Catalonia (Barcelona, Spain) and with Carnegie-Mellon University at Pittsburgh respectively.

Javier Jordan received his civil engineering degree from the Technical University of Catalonia (UPC) in 1996. He is the director of PEDELTA in Spain.

Javier has been involved in the design of the most relevant bridges developed in PEDELTA during the last 12 years. He is one of the Spanish experts at the evolution group “Bridges” of Eurocode-4, parts 1-1 and 1-2 (Steel Bridges).

SUMMARY

Steel-concrete composite action offers great advantages in bridge design due to the synergy of both materials with an increase of load-carrying capacity and stiffness compared to non-composite beams. Typically, composite action is used in positive bending moment regions transferring shear forces at the steel-concrete interference using stud shear connectors normally welded to the top flanges of the girder.

This paper summarizes the design approach and general criteria for composite beams in the Eurocode-4 and in the Spanish Code for composite bridges. In addition the paper presents the design and construction procedures using double composite action of various vehicular bridges designed by PEDELTA and recently built in Spain.

DESIGN AND CONSTRUCTION OF DOUBLE COMPOSITE STEEL TUB BRIDGES IN SPAIN

1. Double Composite Action

Steel-concrete composite action offers great advantages in bridge design due to the synergy of both materials with an increase of load-carrying capacity and stiffness compared to non-composite beams. Typically, composite action is used in positive bending moment regions transferring shear forces at the steel-concrete interface using shear connectors normally welded to the top flanges of the girder.

Continuous girder bridges experience the largest bending moments over intermediate supports. These negative moments induce compression forces at the bottom flange of the girder and, as a consequence, steel plates are heavily stiffened and steel plate thicknesses are larger than in positive moment regions. Composite action over the supports, the addition of a bottom concrete slab cast over the bottom steel flange and connected using steel stud or other shear connectors, allows to reducing the thickness of steel plates in negative moment regions and to increase both its buckling strength and ductility. The bottom concrete slab reduces the need of structural steel (the bottom flange working in compression) providing savings on the construction cost making steel bridges a competitive solution.

Double composite action has been widely used in bridges in Spain and other European countries since the last three decades with great success as an economical and efficient solution. Double composite action has been used successfully and provides clear advantages and competitiveness for tub bridges with spans ranging from 30 to 120 m. Due, probably, to a lack of specific design criteria at the AASTHO Code, the application of double composite action in the US is not extended. Some research has been done supported by FDOT (Florida Department of Transportation) [1].

The structural analysis of composite action in negative moment regions uses the same basic principles of composite in positive moment areas. Construction of the bottom slab is very easy, as the box girder's bottom steel flange and webs are used as a formwork.

1.1. Design criteria

The composite sections are designed in compliance with the Serviceability Limit States and Strength Limit States showed in the respective regulation. For Strength Limit States, the composite sections should be checked for resistance of cross-section, resistance to lateral-torsional buckling and resistance to shear buckling. Additionally, it is necessary to provide capacity to longitudinal shear and to fatigue. It is important in the design to account the superposition of stresses due to each stage of construction and in service.

European codes regulation give a classification of cross-sections. Four classes of cross-sections are defined (compact or non-compact sections with different bending rotation capacities). Classification of a cross-section depends on the width to thickness ratio of the elements that make up the cross-section subject to compression. The classification is similar to that defined at the Canadian Highway Bridge Design Code CAN/CSA-S6-06. Different methods, either elastic or non-linear, are defined at the Code to obtain the capacity of a cross-section in bending, shear and torsion.

A composite section shall be designed and constructed in compliance with the limitations in vibrations, deformations and stresses. Calculation of stresses at the Serviceability Limit States should consider the following effects: shear lag, creep and shrinkage of concrete, cracking of concrete, construction sequence,

inelastic behavior of steel and reinforcements, torsional and distorsional warping, if any.

The general design criteria can be summarized as follows:

1. Double composite action is used in negative bending regions (around 20-25% of the span length on either side of intermediate supports).
2. Internal efforts may be computed by a global elastic analysis, considering concrete creep for short-term and long-term situations. Creep can be handled adjusting n values ($n=E_s/E_c$, effective modular ratio) or by means of specialized software that calculates time dependent concrete shrinkage and creep effects.

Where: E_s , Modulus of elasticity of steel
 E_c , Modulus of elasticity of concrete

3. Cracking (reduction of stiffness over intermediate supports) is also considered when calculating the section properties of the box girder. Typically, the deck is considered to be cracked in the structural analysis. Concrete deck in negative regions is considered to be ineffective but, as studs are provided, the deck longitudinal reinforcement is considered as a part of the composite cross-section. Shear-lag may be ignored in general cases. Design codes provide general criteria for the structural analysis (section 5 of Eurocode 4-part 2).
4. The Bottom slab is subjected to significant compression stresses and buckling has to be checked considering that the slab is restrained by stiffeners and webs. Connection is ensured through studs/shear connectors over the entire contact steel-concrete interface.
5. Steel plate deformation during casting shall be limited to avoid excessive deformation that can affect both appearance and instability.
6. Eurocode 3 (Design of steel structures) [2] and Eurocode 4 [3] (Design of composite –steel/concrete-structures) provide general rules for the design of steel bridges. The design of composite plates is included in EC-4 part 3 (bridges) section 9.
7. Section 6 of the Spanish Code for steel-concrete bridges, RPX, [4] provides guidelines for the design of box girders with double composite action.
8. Eurocode 4 and RPX provide a simplified procedure to control crack width. The maximum stress in the reinforcing bars immediately after cracking is limited depending on the bar size (or limiting bar spacing) and the design crack width.
9. The maximum longitudinal compressive stress at the bottom slab at the Serviceability Limit States is limited to: $0.4 f_c'$ (under permanent loads) to prevent large creep effects and should not exceed $0.6 f_c'$ for the characteristic service load combinations (under maximum permanent and live loads) to prevent crushing of concrete (micro-cracking).
10. The maximum stresses resulting from the characteristic load combination in Service are typically limited to the yield stress. The formulation of RPX-Composite Bridge Code [4] and Eurocodes ([2] and [3]) is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_y$$

Where: σ , Normal stress
 τ , Shear stress
 f_y , yield strength

The Spanish code is more restrictive than the other regulations and it limits the maximum stress to 90% f_y for characteristic service load combinations and to 75% for frequent service load combinations (SLS).

11. The minimum reinforcement required on the top slab varies between 0.10 and 0.15% of the effective concrete area. Final required reinforcement is obtained verifying both Serviceability and Strength Limit States. Typically, the amount of reinforcement at the top slab in negative bending moment regions is larger than the required minimum reinforcement.
12. The analysis of the plastic resistance moment (Strength Limit State) of a composite cross-section is calculated assuming full interaction between structural steel, reinforcement and concrete. Compression at the concrete slab is limited to 0.85 f_{cd} (design compressive concrete strength).
13. Eurocode 4 specifies that the capacity of a composite concrete-steel slab may be designed as a reinforced concrete slab considering the effective width of the steel plate as a part of the reinforcement. Compression forces at the composite slab should be computed considering instability.
14. Design of shear connection at the composite plate for negative bending moments may be computed in the same way as in positive moment regions, considering shear forces in both longitudinal and transverse directions. The regulation gives a minimum distance for both transversal and longitudinal spacing between studs to prevent local buckling of the composite slab.
15. RPX provides a method to compute the strength of composite plates under compressive forces. The method is conservative as it takes into account the instability of the composite plate neglecting constrain of the webs. The method provides the axial strength capacity of the composite plate.

1.2. Example

To illustrate the behavior of a double composite cross-section, three different cross-sections have been analyzed under negative bending moment according to method provided by the Spanish Code RPX.

- | | |
|-----------------------|--|
| Cross-Section Type 1: | Double composite cross-section (Figure 1-1). |
| Cross-Section Type 2: | Composite cross-section designed to provide a similar bending capacity as Type 1 (Figure 1-3). |
| Cross-Section Type 3: | Composite cross-section similar to Type 1 without the bottom concrete slab (Figure 1-5). |

Behavior of Cross-Section Type 1

Figure 1-1 illustrates a composite section working at negative bending region. Double composite action is used. The characteristics of the composite section are as follows:

- Transverse stiffeners spaced at 2.00 m in longitudinal direction.

- Concrete bottom slab, Grade 45 MPa
- Steel grade is S355-J2+N (equivalent to a Grade 50W).
- Upper reinforcement $A_s' = 420\text{cm}^2/\text{m}$, Grade 500MPa
- This section has a structural steel weight of 2586 kg/m

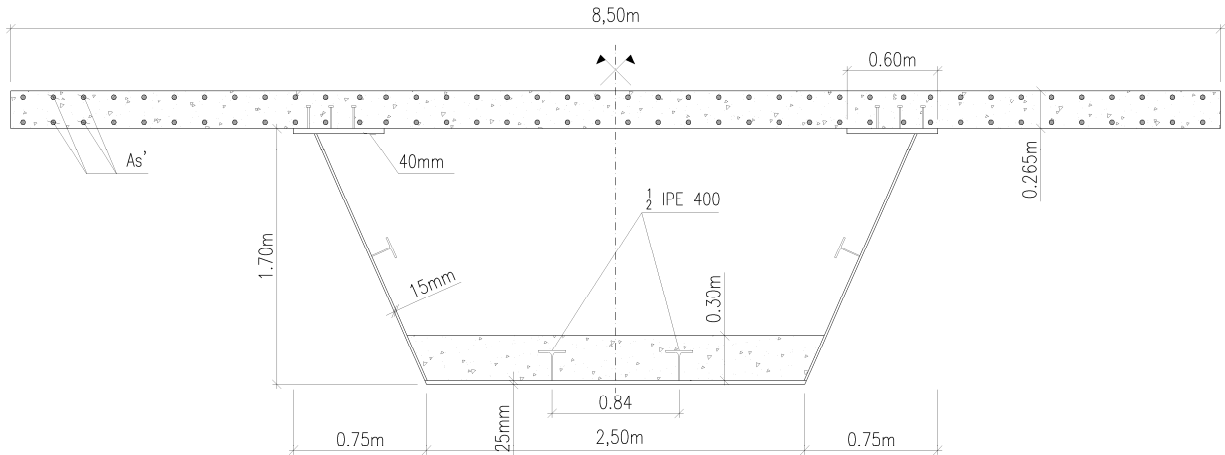


Figure 1-1. Composite section Type 1.

According to RPX, the axial compressive capacity of the bottom composite plate is $N_{rd} = 34,200$ kN. Figure 1-2 shows the axial compressive strength of the composite bottom slab to longitudinal deformation curve. Where, $N_{rd,ult}$, Compressive strength of the composite slab and ϵ is the strain of the steel plate.

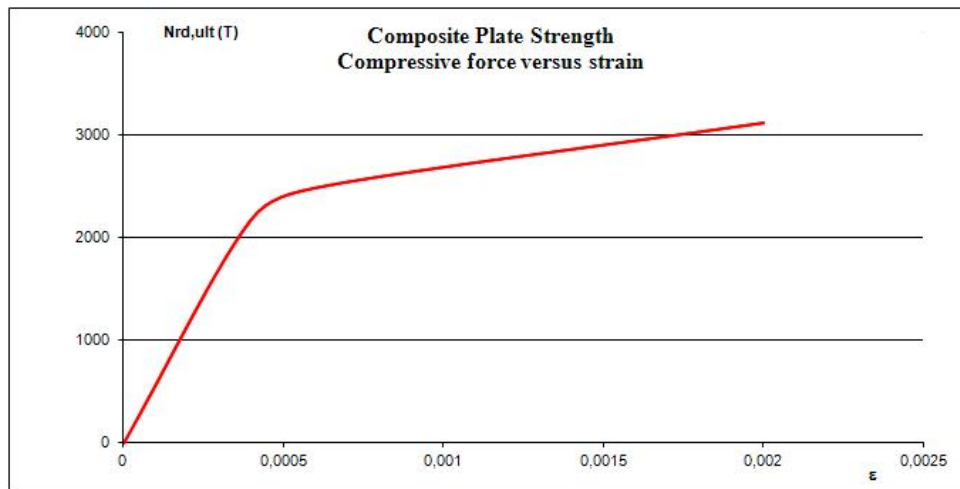


Figure 1-2. Compressive strength of the bottom composite slab.

The negative bending moment capacity ($M_{rd,ult}$), considering full interaction between steel plate, reinforcement and bottom slab (without web contribution) is $M_{rd,ult} = 60,298$ kNm.

Behavior of Cross-Section Type 2

Figure 1-3 illustrates a section working at negative bending region. In this case, double composite action is not considered. The bottom steel thickness has been designed to achieve a compressive strength similar to section Type 1. The characteristics of the cross-section type 2 are:

- Transverse stiffeners spaced at 2.00 m in longitudinal direction
- Steel grade is S355-J2+N (equivalent to a Grade 50W)
- Upper reinforcement $A_s' = 420\text{cm}^2/\text{m}$, Grade 500MPa
- This section has a structural steel weight of 3560 kg/m (38% more than the previous section)

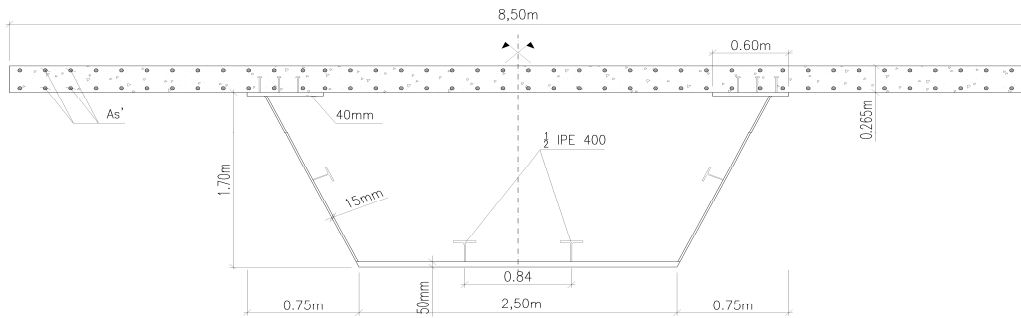


Figure 1-3. Cross Section. Type 2

According to RPX, the axial compressive capacity of the bottom steel stiffened plate is $N_{rd} = 37,870$ kN. Figure 1-4 shows the axial compressive strength of the bottom stiffened plate to longitudinal deformation curve.

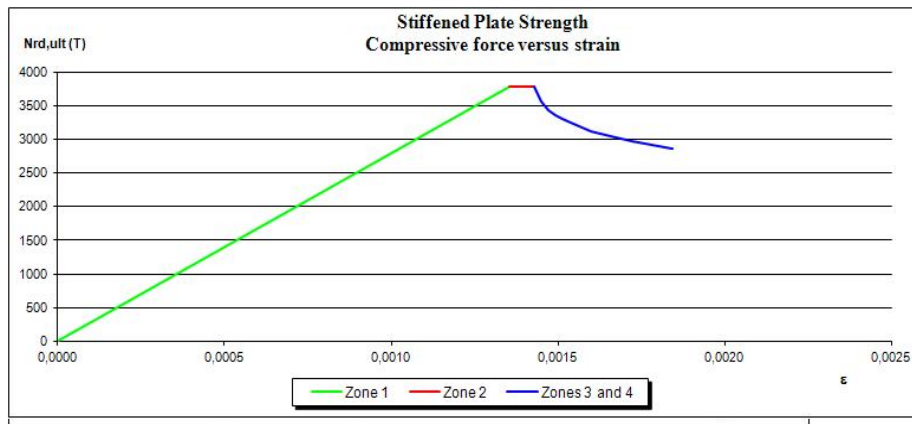


Figure 1-4. Compressive strength of a stiffened plate.

The negative bending moment capacity (without web contribution) is $M_{rd,ult} = 59,870$ kNm

Note that to obtain the same negative bending moment capacity cross-section Type 2 requires about 38% more steel than Type 1. Double composite action increases both strength and ductility of the cross-section.

Behavior of Cross-Section Type 3

Cross-section Type 3 has the same steel plates as Type 1 but without longitudinal bottom stiffeners neither the bottom slab. Figure 1-5 illustrates the geometry. . The characteristics of the cross-section Type 3 are:

- Transverse stiffeners spaced at 2.00 m in longitudinal direction.
- Steel grade is S355-J2+N (equivalent to a Grade 50W).
- Upper reinforcement $A_s' = 420\text{cm}^2/\text{m}$, Grade 500MPa
- This section has a structural steel weight of 2350 kg/m.

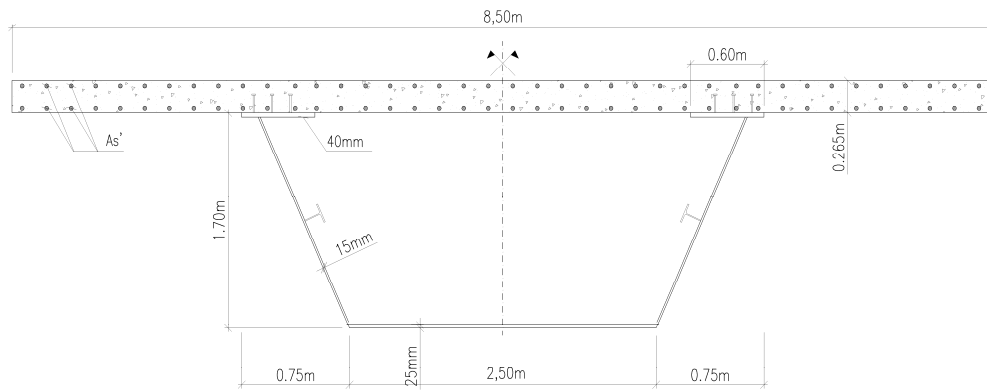


Figure 1-5. Cross Section. Type 3

The axial compressive capacity of the bottom steel plate is $N_{rd} = 10,000$ kN. Figure 1-6 shows the curve axial compressive strength of the bottom steel to longitudinal deformation.

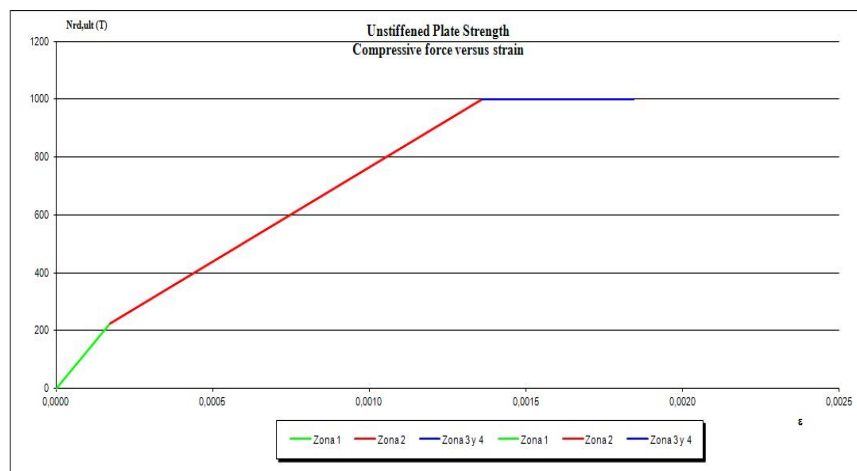


Figure 1-6. Compressive strength of a unstiffened plate.

The negative bending moment capacity (without web contribution) is $M_{rd,ult} = 17,750$ kNm. Note that the capacity to bending of cross-section Type 3 is dramatically reduced compared to sections Type 1 and Type 2.

1.3. Conclusions

Double composite action provides a significant reduction of the bottom steel plate thickness. In this example, savings on steel weight are 38%. Typically, double composite action is extended around 15% to 20% of the span length over intermediate supports. Thus, double composite action in a 3-span continuous girder allows savings on structural steel around 15% compared to a non-double composite bridge. Double composite action increases flexural stiffness of negative bending regions and additional savings (approx.. 5%) may be obtained due to reduction of positive bending moments.

Double composite action increases both strength and ductility of cross-sections subjected to negative bending moments.

2. Vall D'en Bas Bridge

2.1. Context and conceptual design

For several years, the connection between the towns of Olot and Vic was one of the main priorities for the counties of Garrotxa and Osona, in Catalonia, Spain. Due to its unique importance and singular surroundings – Olot Volcanic Zone Natural Park as well as several other protected landscape areas and villages – the environmental impact of this project had an extreme importance. As a result, the Vall d'en Bas Viaduct was required to be as transparent as possible, with moderate big spans, a sober structure and a sustainable and environmentally-friendly type of construction (Figure 2-1). With these premises in mind, the decision towards a composite steel-concrete solution was essential.



Figure 2-1. Bridge at Vall d'en Bas

2.2. Description of the structure

Several alternatives were developed during the preliminary phase of the project, with environmental impact as well as aesthetics impacts included. The intention was to create a sober structure not dominating over the environment and to minimize the impact during the construction using precast construction with minimum auxiliary equipment.

The bridge is of 565 m long, 14.5 m wide, and carries three lanes of traffic with two narrow sidewalks. The bridge comprises nine spans: $(65+3*80+70+3*50+40)$ m). The deck is a tub steel girder which has variable depth along the first six spans and with constant depth over the last three shorter spans. The box girder depth varies from 1.75 m at the center of the spans to 3.85 m at the intermediate supports. The variable depth and the constant web slope results in a variable bottom flange width of 6 m at the pier

and 6.82 m at mid span. The steel section includes two 3.5 m long cantilever outriggers, which complete the overall width, spaced at 4 m. These elements are connected at the outer-edge by a longitudinal fascia beam (Figure 2-3).

Webs have constant slope and their plate thickness varies from 12 to 20 mm. Bottom plate thickness varies from 15 to 30 mm and top flange thickness from 15 to 40 mm. Webs and bottom flange are stiffened to prevent buckling. All structural steel is Euronorm-EN10025 Grade S355 J2G3, with yield strength of 355-MPa equivalent to a Grade 50W.

K-frame diaphragms are located at 4 m spacing throughout the bridge length. End diaphragms at the supports and over piers are stiffened plates with access holes for future inspections. Top lateral bracing, using an X-frame system is designed to provide torsional stiffness of the tub girders during construction.

The double composite section is extended over the negative moment region at the piers 12 m on either side of the intermediate support for the longer spans and 7 m on the smaller spans. These lengths are about 0.15 L (L= span length). The bottom slab has variable thickness from 0.25 m to 0.65 m and it is reinforced and connected to the bottom steel flange with welded shear connectors (Figure 2-4).

The top concrete slab thickness is constant of 0.25 m and it was designed to act compositely with the main box girder and the transverse diaphragm and cantilever outriggers. Precast reinforced concrete panels (0.07 m thickness) have been used as a form and as a part of the slab.

The concrete piers, which range in height from 7.5 to 24 m, have an individual shaft of curved and variable dimensions on the transverse direction and it is 1.80 m wide in the longitudinal direction. The shape and texture provide a transparent and sober appearance to the bridge. As the geotechnical investigation indicated that the sound rock was at different depths. The two first piers are founded on rectangular footings and the others supported on 71-in diameter drilled shafts. Abutments consist of concrete walls supported on three 71-in drilled shafts. Laminated elastomeric bearings are used at the supports.

The main quantities are as follow:

- The amount of steel used in the bridge is 2,130 ton (234 Kg/m^2).
- Concrete top slab (0.25 m/m^2), bottom slab (0.05 m/m^2).
- The amount of shear connectors is about 7 units per m^2 .

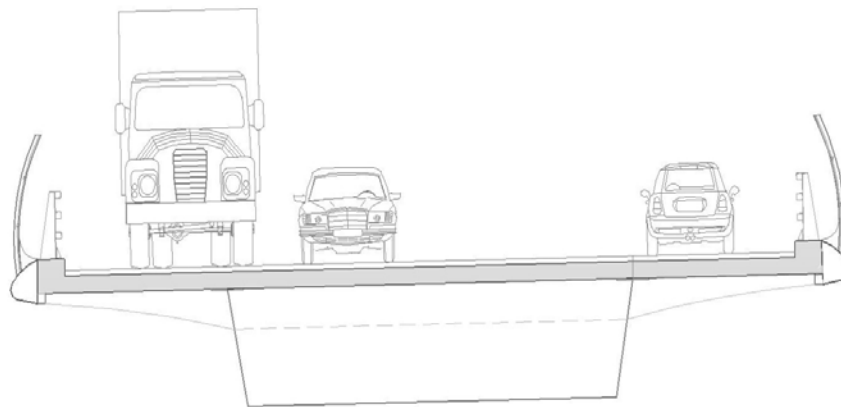


Figure 2-2. Typical cross-section at mid-span

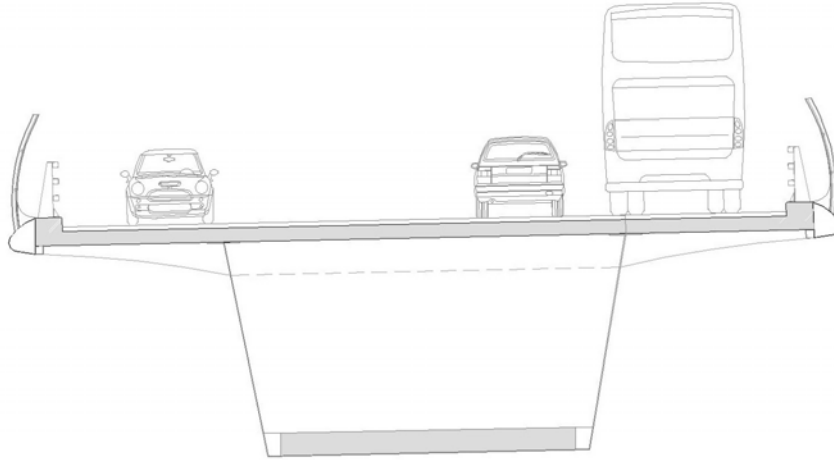


Figure 2-3. Typical cross-section at supports



Figure 2-4. Steel girder sections to be lifted

2.3. Construction

Construction of the bridge began in March 2008 and was completed in June 2009. Once the drilled piles and the abutments had been built, the steel girder including the cantilever outriggers was assembled on site (Figure 2-4). Splices are located to produce the length of $\frac{3}{4}$ of one span and $\frac{1}{4}$ of the following span (Figure 2-5). All connections at the site were welded. An exhaustive quality control process was undertaken. All the welds, both off and on site, were checked using techniques such as ultrasonic testing, X-rays and magnetic particles. The construction is unshored.

After the installation of the steel girder, the reinforced bottom concrete slab over the supports was cast.



Figure 2-5. Typical section lifted to the final position

Once the steel girder was installed and the bottom slab was cast, the precast panels of the top slab were installed to configure the final cast in place reinforced top slab (Figure 2-6).



Figure 2-6. Installation of the steel girder and precast panels

The bridge was completed with the fitting of railings, waterproofing of the deck, pavements and illumination. All the finishes and, in particular, the selection of materials were designed in harmony with the design of the structure.

3. Bridge Over La Muga River In Girona Province

3.1. Context and conceptual design

The bridge crosses over La Muga River as a part of the improvement of road GI-V-5041 at Girona province (Spain) to improve the connection between Pont de Molina and Les Escaules with a very limited budget. The bridge replaces an existing bridge with a short hydraulic capacity. The bridge is located in a rural area in the south east Pyrenees at the Alt Empordà, a County with a large history and well preserved landscape.

To meet technical, aesthetical, hydraulic and environmental requirements, a sober continuous girder is designed. The bridge features three spans, the central one crossing the streambed, and it is supported on circular slender columns (Figure 3-1).

The bridge has been designed using double composite action to optimize the cost. The bridge has recently completed and was opened to traffic in December 2011.



Figure 3-1. Bridge over La Muga River at Les Escaules (Girona, Spain)

3.2. Description of the structure

The 112.4 m long bridge features three spans (Figure 3-2), with a configuration of 33.7+45+33.7 m. The cross-section consists of 8.5 m wide box girder 1.7 m deep.

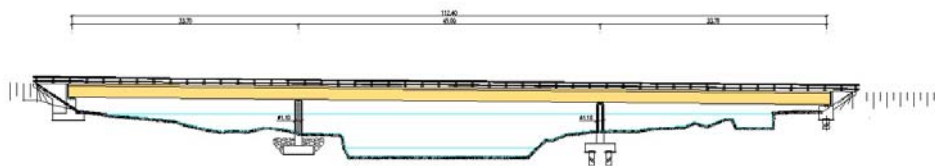


Figure 3-2. Elevation.

The steel box girder geometry is constant along the bridge but plate thickness varies at different regions. Figure 3-3 illustrates a typical transverse steel box section consisting of 1.83 m web stiffened plates, two 0.50 or 0.60 m wide top flanges, and a 2.50 m bottom stiffened flange. Web thickness varies from 10 to 15 mm; bottom plate thickness varies from 12 to 35 mm and top flange thickness from 20 to 40 mm (Figure 3-4). All structural steel is Euronorm-EN10025 Grade S355 J2G3, with yield strength of 355-MPa equivalent to a Grade 50W. The steel girder is not a non-compact section and has been design to meet the Spanish Bridge Code specifications.

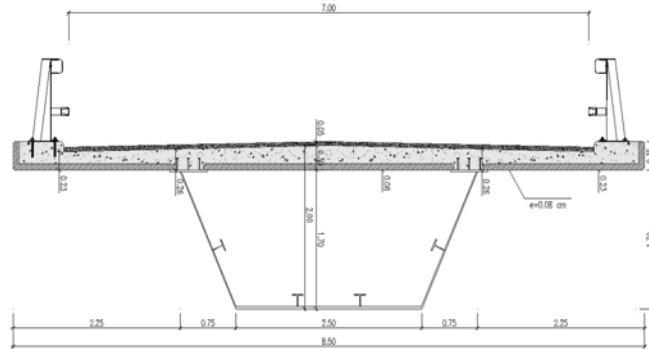


Figure 3-3. Typical cross-section.

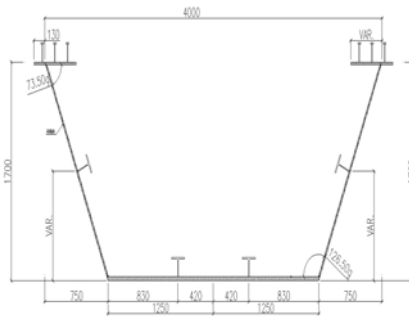


Figure 3-4. Steel girder. Typical cross-section.

K-frame diaphragms are located at 4 m spacing throughout the bridge length. Between each diaphragm, a transverse web and bottom flange frame stiffener is placed. End diaphragms at the supports and over piers are stiffened plates with access holes for future inspections. Top lateral bracing, using an X-frame system is designed to provide torsional stiffness of the tub girders during construction (Figure 3-5).

The double composite section is extended over the negative moment region at the piers 17.7 m. This length is divided into two sections: 7.92 m / 0.23 L at the side span and 9.78 m / 0.22 L at the central span. The bottom slab is 0.30 m deep and it is reinforced and connected to the bottom steel flange with welded shear connectors (Figure 3-6).

The top concrete slab thickness varies from 0.23 to 0.3 m and the 8 cm precast reinforced concrete panels have been used as a formwork and as a part of the slab.



Figure 3-5. Detail of top lateral bracing



Figure 3-6. Typical and interior diaphragms

The concrete piers, which range in height from 8.5 to 5.5 m, have an individual column of circular shape with a 1.20 m in diameter. Piers were cast in place and provide the bridge a simple and elegant appearance. As the geotechnical investigation indicated that the sound rock was at different deeps. One of pier is founded on a square 4.5 m footing and the other on a footing supported on four 0.85 m diameter drilled piles. Abutments consist of concrete walls. Laminated elastomeric bearings are used at the supports.

The main quantities are as follow:

- The amount of steel used in the bridge is 182 ton (171 Kg/m^2).
- Concrete top slab (0.26 m/m^2), bottom slab (0.025 m/m^2).
- The amount of reinforced bars on both top and bottom slabs is 60 kg/m^2 .
- The amount of shear connectors is about 6 units per m^2 .

3.3. Construction

Construction of the bridge began by October 2010, and was completed in November 2011. Once the concrete foundations, abutments and piers had been cast, the steel box girder was erected. The steel girder was fabricated in sections of about 14 m to facilitate transportation to the site. These sections were preassembled in three longer field sections of 44.80 m, 22.8 m and 44.8 m that generated two splices located at 25% of the main span from the intermediate supports. The segments were lifted and assembled at their final position and connected by welding (Figure 3-7).



Figure 3-7. Steel girder erection.

After the erection of the steel box girder, the bottom slab at negative moment regions was cast. Once the bottom slab reaches its specified strength (10 days), the top slab was cast in two phases (using precast panels, Figure 3-8), first the sections on positive moment regions and finally over the supports to optimize material consumption. Construction was unshored.



Figure 3-8. Precast concrete panels of the top slab.

4. Sant Celoni Bridge Over Tordera River

4.1. Context and conceptual design

Sant Celoni is a municipality located 53 km north of Barcelona (Spain) with more than 15,000 inhabitants. The city, the entrance to the natural parks of Montnegre and Montseny, was created in the 12th Century and has a well preserved historical heritage. The municipality is crossed by Tordera River which divides Sant Celoni and La Batlloria, a small town part of the municipality located 5 km away of the city center. To improve the connection between the two areas, the city council decided to build a bridge over the Tordera River with one lane for traffic and one lane for both pedestrians and bikes. A 100

m long bridge is required to meet the hydraulic requirements and the minimum span to meet hydraulic regulations is 50 m. With these geometric constraints, a two-span continuous girder of simple and pure shapes has been designed. A steel tub girder with double composite action was selected as it provides a reasonable cost, elegant appearance, and the ease of construction allows minimizing the environmental impact during construction (Figure 4-1).



Figure 4-1. Bridge over Tordera River in Sant Celoni.

4.2. Description of the structure

The structure is a two span continuous steel concrete box girder of 50+50 m long.

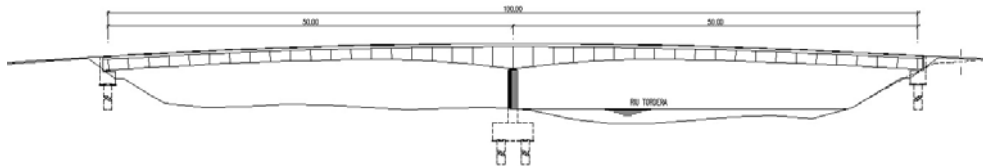


Figure 4-2. Elevation.

The girder depth (steel + top slab) is constant of 1.5 m over 30 m, from the abutments, and varies from this section to 2.75 m at the supports (Figure 4-2). The platform width is 7.6 m and carries two lanes, one for vehicular traffic and the other for pedestrians and bikes. The bridge has been designed using double composite action. Figure 4-3 illustrates a typical transverse steel box section. The variable depth and the constant web slope results in a variable bottom flange width of 2.74 m at the pier and 3.30 m at mid span. The steel section includes two cantilever outriggers that complete the overall width spaced at 2.5 m.

The two webs are stiffened plates with plate thickness varying from 8 to 12 mm. The bottom flange is also longitudinally and transversally stiffened and plate thickness varies from 15 to 30 mm. The two top flanges consist of plates with dimensions varying from 750x40 mm at the pier to 500x20 mm at the abutments. Steel is Euronorm-EN10025 Grade S355 K2G3, with yield strength of 355-MPa equivalent to a weathering steel Grade 50.

The use of weathering steel reduces future maintenance and provides a visual aesthetic value; the contrast of the dark steel and the clear lines of the overhangs increase visual slenderness. The interior of the girder is shop-coated to ensure good protection.

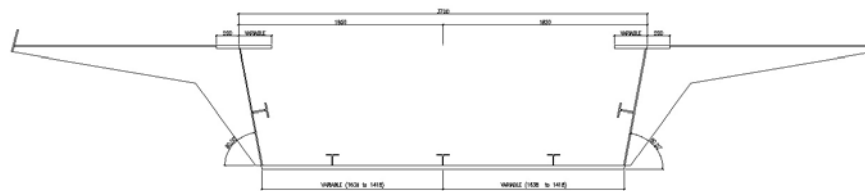


Figure 4-3. Steel girder. Typical cross-section.

The steel girder is a non-compact section and has been designed to meet the Spanish Bridge Code specifications.

K-frame diaphragms are located at 2.50 m spacing throughout the bridge length. End diaphragms at the supports and over piers are stiffened plates with access holes for future inspections. Top lateral bracing, using an X-frame system is designed to provide torsional stiffness of the tub girders during construction.

The girder is designed composite for permanent (partially shored construction: part of the wet concrete is taken by the steel girder as two temporary supports were used to optimize cost) and live load. The cross-section has a 0.30 m. concrete top slab, made using precast concrete deck panels as wells cast-in-place concrete to complete the final thickness. The top concrete slab thickness varies from 0.23 to 0.3 m and the 8 cm precast reinforced concrete panels have been used for both as a formwork and as a part of the slab.

The double composite section is extended over the negative moment region over 12.5 m on either side of the pier (0.25 L). The bottom slab has variable thickness from 0.25 m at 12.50 m from the support to 0.30 m over the support. The slab is reinforced and connected to the bottom steel flange with welded shear connectors (Figure 4-4).

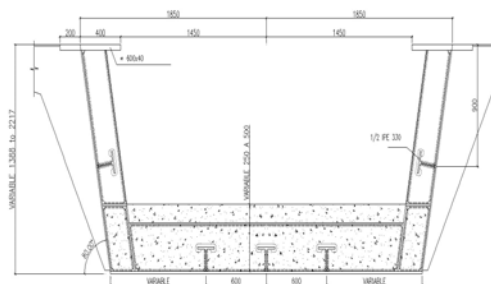


Figure 4-4. Cross-section of the steel girder and bottom slab at the pier

The concrete pier is 5.80 m in height and its typical cross-section can be inscribed in a rectangular of 1.20x2.75 m. The pier was cast in place and provides a simple and elegant appearance. The pier is founded on a rectangular 4.50x8.00 m footing which is supported on six 1.00 m diameter drilled circular shafts. Abutments consist of concrete walls. Laminated elastomeric bearings are used at the supports.

The main quantities are as follow:

- The amount of steel used in the bridge is 170 ton (222 Kg/m²).

- Concrete top slab (0.30 m/m²), bottom slab (0.033 m/m²).
- The amount of reinforced bars on both top and bottom slabs is 51 kg/m².
- The amount of shear connectors is about 7 units per m².

4.3. Construction

Foundations and concrete supports were built by means of ordinary concrete construction techniques. The steel girder was prefabricated in four parts, weighing around 42-ton each and measuring 25 m in length (Figure 4-5). The four pieces were lifted to their final position, using two auxiliary supports (Figure 4-6), and finally welded together. Maximum pre-camber is 156 mm (L/320) at a distance of 30 m from the intermediate support.

Thereafter, the bottom slab was cast in place (Figure 4-7) and the top precast panels and top slab were built (Figure 4-8). Two weeks later, the temporary supports were removed and static load test was carried out. The measured movements of the structure under construction were according to the theoretical predicted values. Construction of the bridge began in January 2011 and was completed in December 2011.



Figure 4-5. Erection of the steel girder.



Figure 4-6. Steel girder on temporary supports.



Figure 4-7. Double composite region near intermediate support.



Figure 4-8. Installation of the top slab precast panels.

5 Conclusion

Typically, composite action is used in positive bending moment regions. The negative bending moments induce compression forces at the bottom flange of the girder. As a consequence, steel plates are heavily stiffened and steel plate thicknesses are larger than in positive moment regions.

Double composite action over the supports, the addition of a bottom concrete slab cast over the bottom steel flange and connected using steel studs, allows to reducing the thickness of the bottom steel plate in negative moment regions and to increase both its buckling strength and ductility.

The savings due to double composite action on structural steel in negative bending regions can reach up to 40 %. The reduction of steel plate thickness over the intermediate supports results in savings on steel around 15% (considering the whole structure). Double composite action increases flexural stiffness of negative bending regions and additional savings (approx.. 5%) may be obtained due to reduction of positive bending moments.

The bridges presented in this paper illustrate the advantages of double composite action to provide efficient and cost-effective solutions.

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La Muga Bridge: Agnès Currás, Sergio Carratalá and Xavier Barrau

Sant Celoni Bridge: Ricardo Ferraz

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