SDSS'Rio 2010 STABILITY AND DUCTILITY OF STEEL STRUCTURES
E. Batista, P. Vellasco, L. de Lima (Eds.)
Rio de Janeiro, Brazil, September 8 - 10, 2010

ELEGANCE AND ECONOMY - A NEW VIADUCT OVER THE RIVER LLOBREGAT

Peter Tanner*, Juan L. Bellod* and David Sanz*

* Cesma Ingenieros, Madrid, Spain
e-mail: cesma@cesmaing.com

Keywords: Composite Bridge, Conceptual Design, Detailing, Stress Field Method, Patch Loading, Semi-probabilistic Finite Element Calculation.

Abstract *The viaduct over the River Llobregat at Puig Reig, Spain, illustrates how demanding conditioning factors can serve as inspiration for a satisfactory solution. The functional design adopted, characterized by the simplicity of its lines, was enhanced by the careful shaping of structural members and good detailing. This example shows that modern, technologically advanced bridge design can be compatible with a solution whose elegance meets even the most exacting aesthetic standards, with no need for inefficient members or adornments, at an affordable additional cost over the least expensive solution. The ideas underlying the conceptual design for the River Llobregat bridge, in particular with respect to the specific boundary conditions involved, are explained in the article, along with the actual layout, remarks about the design of structural details and the verification of structural safety during the incremental launching of the steel structure.*

1 INTRODUCTION

The new bridge over the River Llobregat forms part of a scheme to improve motorway C-16 in Catalonia, between Puig Reig and Berga, and shorten the driving time between Berga and Barcelona by more than half an hour. It spans the River Llobregat with a sizeable viaduct not far from the town of Puig Reig [1].

Many key aspects of a bridge project are governed by construction site-related and geometric, functional, constructional and economic constraints. The present case is no exception. But demanding boundary conditions often spur careful and indeed even innovative structural solutions for which specific design rules may be lacking. In these cases, structural safety and serviceability can not be achieved by simply applying codes and standards. Typical examples of such circumstances are structural detail design and finite element analysis-related structural safety. The approach adopted in the design of the River Llobregat bridge to solve both these problems is explained in detail in this paper.

2 BOUNDARY CONDITIONS

The total length of the viaduct on the widened motorway is over 550 m. Its ground plan layout is straight for most of its length, although as it nears the abutment on the Berga side it curves on a radius of 600 m at its sharpest. In elevation, the bridge slopes gently on a grade of 1,4%. Its single deck carries four lanes of traffic, two in each direction, plus four service lanes and the median, and has a total width of 23,8 m. Due to environmental restrictions, no temporary supports could be erected near the River Llobregat during construction, nor were heavy cranes allowed to access the area.

In most structures, economic constraints play a decisive role in the adoption of the final solution, and the Llobregat viaduct was no exception. Nevertheless, in view of the location of the bridge and its visibility from the nearby town, the owners also wanted a landmark structure with a simple and wellbalanced design. All these constraints made it particularly important to strike an optimal balance between cost-effectiveness and aesthetics.

3 CONCEPTUAL DESIGN

3.1 Layout

The shape adopted for the bridge structure was informed by the above functional and site constraints. A continuous nine-span structure with a total length of 568 m was devised for the bridge deck. Topographic considerations led to the two end spans being 60 m long, while each of the seven inner spans is 64 m long. Despite the unfavourable outer to inner span ratio of 0,93 compared to other possible layouts with shorter bays, this solution was preferred to enhance the visual efficiency of the design since two of the most important determinants of elegant bridge design are transparency and slenderness: bridges are often regarded as elegant when characterized by the efficient use of construction materials in solutions with generous span lengths.

The composite deck consists of an open, 6 m wide steel box girder covered with a concrete slab. The steel section is 2,55 m high, for a slenderness ratio on the inner spans of 25. The impression of visual slenderness is heightened by the concrete slab, which cantilevers 8,9 m on each side of the steel box. The cantilevered slabs rest on a space truss consisting of longitudinal chords 3,5 m from the edge attached to horizontal ties and inclined diagonals, in turn connected laterally to the box girder (figure 1). From an elevation view, each set of two diagonals forms a “V”.

Figure 1: Partial view of the finished bridge (Photo: Paco Gómez).

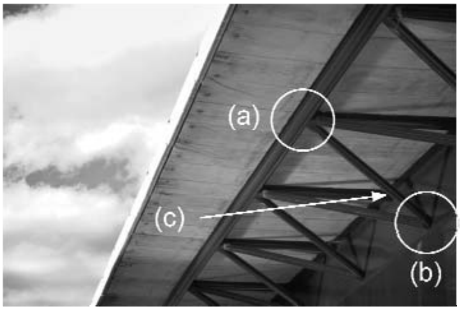

Figure 2: Connections between (a) longitudinal chord, diagonals and tie; (b) diagonals and box girder; (c) tie and box girder.

The diagonals, the horizontal ties running between the diagonals and connecting chord and diagonals both to the top of the box girder, as well as the joints between the diagonals and the bottom of the girder are all spaced at 8 m centres along the chord. Because of the slant on the diagonals and their layout, the trusses not only transmit loads from the cantilevers to the box girder, but form part of the overall resistance mechanism of the deck, reinforcing composite girder stiffness and strength. At the same time, the truss design is visually gratifying. Moreover, the layout adopted impacts the action effects on the truss members and joint behaviour, with the concomitant need for particular care in joint design.

Indeed, good detailing is essential to ensure appropriate member performance, including suitable load transfer mechanisms, fracture toughness, fatigue resistance and durability. Appropriate detailing may also facilitate bridge assembly and significantly affect its visual impact. Since the satisfactory structural behaviour of details and joints depends essentially on their conceptual design, this step is of cardinal importance and should be performed at an early stage in the design process, in keeping with the conceptual design of the system as a whole as well as of the structural members. In light of the importance of this consideration, the approach adopted for structural detail design is treated in section 4.

3.2 Members

The box girder has a composite bottom flange in the area around the internal supports for greater hogging bending strength and system ductility. This is a particularly important issue, for system reliability is much greater in ductile than in brittle systems [2]. Failure mode is of utmost importance, moreover, because brittle structures may be very sensitive to the uncertain effects of actions and influences such as creep, shrinkage, temperature, differential settlements or earthquakes and may collapse suddenly under such circumstances, without prior warning.

The truss diagonals are made of 323 mm diameter hollow section profiles with wall thicknesses ranging from 16 mm to 22 mm, depending on the internal forces. The ties, positioned at right angles to the bridge centreline, are oval composite members whose top flange is connected to the deck slab. The longitudinal chords are also composite, oval-shaped, filled with concrete and connected to the slab, but have an open cross-section. The deck slab itself consists of 80 mm deep precast concrete slabs covered with cast-in-place concrete to the total depth, which varies from 180 mm at the edges to 340 mm over the longitudinal chords. The total slab depth over the 6 m wide box girder is 240 mm.

The concrete piers comprise two 1,3 m wide shafts spaced at 4,7 m centres and inter-connected by a convex concrete wall to form a single monolithic member 6 m wide. Pier depth is variable, tapering slightly to 1,4 m at the top. The piers owe their slender elegance to their height and these cross-sectional dimensions.

4 JOINT DESIGN

4.1 Overview

Further to the bridge layout, solutions had to be provided for three main types of connections (figure 2): longitudinal chord-diagonals-tie; diagonals-box girder; and tie-box girder. The steps involved in designing the structural details with respect to the connection between longitudinal chord, diagonals and tie are described in subsections 4.2 to 4.3. While the connection in question comprises a composite steel and concrete joint, the procedure adopted for its design was equivalent to the procedure used to design steel joints, such as the connection between the diagonals and the box girder.

4.2 Concept

By analogy to Eurocode 3 [3], the connection between longitudinal chord, diagonals and tie can be regarded to be a space KT joint (figure 2) in which the diagonals and tie are positioned on different planes. The longitudinal chord is continuous, while the brace members are arranged so that the diagonals overlap the tie. The ends of the diagonals are prepared for attachment to the curved surfaces formed by the oval chord and tie, without modifying their cross-sections. Forces are transferred from the diagonals to the tie and chord across full penetration butt welds via shear and normal stresses, respectively. Compression stress-induced side wall failure in the chord and tie is prevented by transverse stiffeners installed in both members, and the concrete fill in the chord. Situations such as punching shear failure in the chord or tie wall or brace failure in the event of load inversion in the diagonals upon integration of the slanted truss in the overall resistance mechanism of the deck were prevented by choosing a wall thickness in keeping with the material strength needed in each case. Similarly, the risk of lamellar tearing [4] in the chord and tie walls was mitigated by the choice of steel with suitable through thickness properties.

A specific problem was posed by the concurrence of two circumstances: the integration of the slanted truss in the overall resistance mechanism of the deck and the nature of the connections between longitudinal chord, diagonals and tie as composite steel and concrete joints. In this truss, the composite top chord is the so-called longitudinal chord. The introduction in the joints of this chord of the increment in the axial force, ΔN, due to the horizontal component (parallel to the bridge axis) of the internal forces in the diagonals generates a concentrated longitudinal shear force, V_E. Since the increment in the axial force, ΔN, is received by the steel section whereas the longitudinal chord is composite, the concentrated longitudinal shear force, V_E, is transmitted eccentrically (figure 3). This eccentricity, represented by the vertical distance between the centroid axis of the shear connection and the steel-concrete contact surface,

e_z, induces a local bending moment which may in turn cause the concrete slab to separate upward from the steel structure. To prevent such mechanism, a vertical steel plate welded to the oval shape is embedded in the concrete slab. The transverse reinforcement in the slab is fitted into holes drilled in that vertical plate.

Under this arrangement, the aforementioned local bending moment is resisted by two forces, transmitted by contact pressure. One contact is between the reinforcement bars and the edges of the holes drilled in the vertical plate and the other between the concrete slab and the top flange included in the nodal region of the longitudinal chord (figure 3).

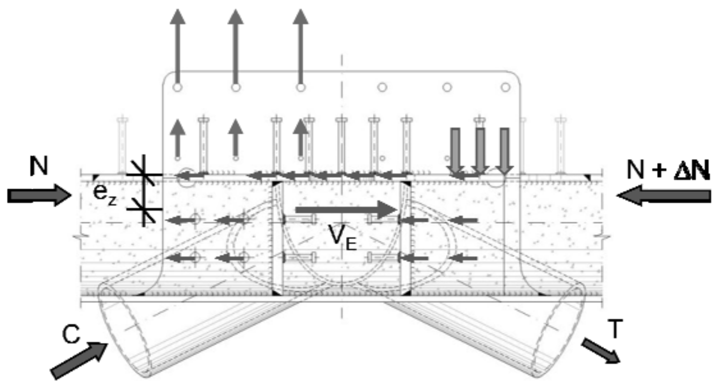

Figure 3: Device for the eccentric transfer of horizontal shear forces.

4.3 Practical design procedure

The methods for structural concrete design currently available do not enable engineers to map forces through a structure. This drawback is particularly troublesome when designing structural discontinuities such as joints and corners. Great strides have been made in reinforced concrete design in recent years in the wake of the introduction of the stress field method [5], with which consistent design models can be developed based on the lower bound theorem of the theory of plasticity. In the present case, the theorem was reformulated for use in composite structure detailing. A practical procedure has been followed in joint design as explained in a previous paper [6].

5 CONSTRUCTION

5.1 Assembly procedure

Once the abutments and the columns had been erected, the steel box girder was lifted into place with cranes in seven of the nine spans, starting at the abutment on the Berga side. Temporary supports were set as close as possible to each mid-span during this stage of construction to reduce bay lengths. The precast concrete slabs were then laid. Finally, concrete was cast in place and the temporary supports were removed (figure 4).

Since environmental legislation banned crane or heavy vehicle access to the banks of the River Llobregat, bridge construction had to proceed without such equipment. The solution chosen was incremental launching from the south abutment across the first two bays, which span the river. A launching nose was used (figure 5), to mitigate the action effects on these two spans and eliminate the risk of patch loading-induced instability (subsection 5.2).

Since the steel box girder cantilevered 16,4 m over the centreline of pier 2 (figure 4) towards the southern abutment, the girder train was 107,6 m long. The 12,4 m length of the nose was established to ensure that during launching the action effects of greatest magnitude would be resisted by the cross section with the highest load-bearing capacity, designed to resist the hogging bending moments at the first pier in the final stage.

Figure 4: Stages of bridge girder construction.

Figure 5: Launching the steel structure across the first two spans.

The 120 m long structure (including the nose) that was to be launched was assembled on the embankment behind the south abutment. It rested on 7 pairs of provisional sliding bearings aligned with the two box girder webs and spaced at 20 m centres. An eighth pair of sliding bearings was positioned on top of the first pier. With this sliding bearing set-up, the structure could be launched with the precast concrete slabs for the first span already in place. This was doubly beneficial, for it lowered the risk of overturning and shortened the time needed to cast the *in situ* concrete.

The front end of the launching nose was fitted with jacks to restore the deflection in the cantilever upon arrival at the first pier. The friction transmitted by the sliding bearings over this highest pier during the launch generated a substantial overturning moment, which was countered by stabilizing the pier with a tension member attached to the abutment. This temporary member consisted of a truss-like structure with an intermediate support to ensure sufficient stiffness and strength (figure 5).

The reactions in the supports during the launch depended on many different factors (subsection 5.2.3). Moreover, the provisional sliding bearings used were not adjustable, nor could the support reactions be monitored during launching. The considerable uncertainties associated with these reactions called for especially thorough process analysis to ensure structural safety during the bridge launching operation.

5.2 Patch loading

5.2.1 Context

At the sliding bearings, sizeable forces were applied perpendicularly to the box girder flange along the plane of the web. The length across which these forces could be distributed was limited to the 1200 mm length of the bearings. During launching, these concentrated forces were transmitted across the flange to the slender web in an area with no transverse stiffeners, for these elements were spaced at 4 m, except in the hogging bending area around the pier, where that distance was shortened to 2 m. Web behaviour may be governed by yielding, buckling or crippling, depending on the steel structure geometry for given loading conditions and material strength. Web plate thickness, particularly important in this context, varied along the launched girder from 15 mm to 26 mm.

Resistance to transverse forces further depends on the internal forces and moments in the girder as a whole. Hogging bending moments generated by the cantilever, for instance, may reduce the ultimate transverse load. Such problems may be particularly relevant where box girders are concerned due to the possible negative interaction between compression flange and web instability. The recommendations set out in codes and standards ([7] and [8]) on resistance to transverse forces, however, refer primarily to rolled and welded I-girders. A practical procedure therefore had to be devised to verify structural safety in connection with the introduction of transverse forces in the box girder webs.

5.2.2 Verification procedure

The procedure adopted in the present design to verify the patch loading and overall bending moment interaction-related structural safety of the steel box girder during launching comprised the following steps:

1. Identification of the variations in the girder cross section, by segments.
2. Step-by-step computation of the action effects on the steel girder during the launching operation.
3. Using the results from step two, establishment, for each of the cross sections identified in step one, of the maximum reaction forces to be introduced in the box girder webs and the concomitant bending moments.
4. Computation of transverse force – bending moment interaction diagrams [9] to represent the strength of each of the segments identified in step one. This called for non-linear finite element analysis taking into consideration box girder segment and sliding bearing geometry, the geometric imperfections associated with girder manufacturing tolerances, and geometric and material non-linearity.
5. Verification of structural safety by representing the design value of the maximum action effects of reaction force – concomitant bending moment for each cross section analyzed on the respective design transverse force – bending moment interaction diagram.

Further information on the implementation of the above procedure, particularly with respect to the last two steps, is provided in item 5.2.3 below. By way of illustration, the results for one of the cross sections are provided.

5.2.3 Results

Five types of segments, defined by their cross sections, were identified on the girder. Most of the variation was related to the flange or web plate thickness or to the stiffener or diaphragm arrangement. Action effects in general and particularly the maximum effects on each of the aforementioned types of cross sections were calculated pursuant to elastic theory, including the launching operation in the model. All relevant parameters were taken into consideration: i.e., the exact girder and launching nose geometry, including precamber, the position of the sliding supports relative to the girder in each launching step, system stiffness including the nose, and the loads applied and their distribution.

Box girder capacity to resist transverse forces in conjunction with bending moments was computed with non-linear finite element models developed [10] for each segment type, using [11]. Cross section symmetry of each 12 m long segment was also taken into consideration (figure 6a). The sliding bearing was aligned with the web in the centre of the segment between two diaphragms, in an area with no transverse stiffeners.

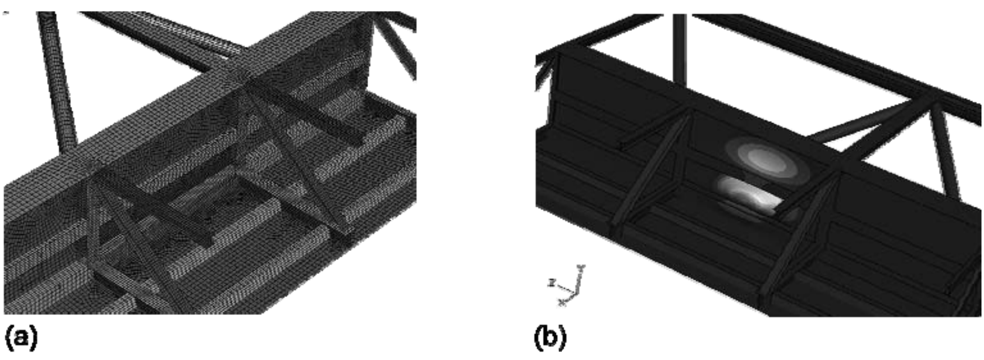

Figure 6: Non-linear finite element analysis to establish the patch load response; (a) mesh for one segment; (b) first failure mode [10].

Different combinations of transverse forces and bending moments were entered into the model to establish the eigenvalues for plate buckling (figure 6b). The first three modes were then scaled to define the imperfections corresponding to manufacturing tolerances.

In the next step, model geometry was redefined by entering these equivalent imperfections for subsequent non-linear analysis. Transverse forces and bending moments were then entered into the modified model. The forces and moments were increased to failure values, taking geometric and material non-linearity into account. Following this procedure and varying the combination of applied forces and moments, the strength, in terms of a transverse force – bending moment interaction diagram, was found for each segment considered (figure 7).

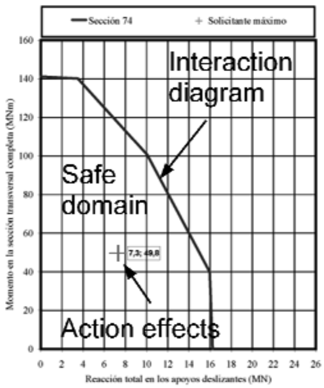

Figure 7: Use of a transverse force – bending moment interaction diagram to verify structural safety in a cross section.

To verify structural safety, the maximum reaction force to be introduced in the web of a given segment, in conjunction with the concomitant bending moment acting on the segment, was represented on the interaction diagram established to determine segment strength (figure 7). Reliability was checked with the design values for action effects and strength. In the absence of specific rules for conducting semi-probabilistic non-linear finite element analysis, these design values were established by applying the partial load and strength factors laid down in [7] to the nominal values computed for the action effects and strength, respectively. The structure was found to be reliable for, as figure 7 shows, the design value of the action effects lay within the safe domain of the design interaction diagram for strength.

6 CONCLUDING REMARKS

Throughout the design stage of the viaduct over the River Llobregat, special attention was paid to conceptual design to successfully translate the many conditioning factors into a reliable, functional, cost-effective, graceful and innovative structure. Such designs may, however, pose problems that lie outside the scope of existing design rules. Two typical examples are discussed in this paper, namely structural detail design and patch loading during incremental launching of the steel structure, in which structural safety could not be ensured by the mere compliance with codes and standards. The stress field method proved to be a very powerful tool for the efficient and reliable design of details and connections. Non-linear finite element analysis, in turn, was very useful for the explicit verification of both local and overall system stability: here the steel box girder during launching. Further studies are required, however, to develop a suitable design format for semi-probabilistic non-linear finite element analysis.

REFERENCES

[1] Tanner, P. and Bellod, J.L., "Modesty prevails: Llobregat river bridge in Spain", *Bridge Design and Engineering*, Issue 51, 30-31, 2008, ISSN 1359-7493.

[2] Tanner, P. and Bellod, J.L., "Increasing reliability of composite bridges by reducing steel. Ductility versus Brittleness", *Composite Bridges. State of the Art in Technology and Analysis*, Martínez Calzón (ed.), Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid, 2002, ISBN 84-380-0221-8.

[3] EN 1993-1-8, *Eurocode 3: Design of steel structures – Part 1-8: Design of joints*, European Committee for Standardisation, Brussels, 2005.

[4] EN 1993-1-10, *Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties*, European Committee for Standardisation, Brussels, 2005.

[5] Muttoni, A., Schwartz, J., Thürlimann, B., *Design of concrete structures with stress fields*, Birkhäuser, Basel-Boston-Berlin, 1997, ISBN 3-7643-5491-7.

[6] Tanner, P. and Bellod, J.L., "Salto del Carnero railway bridge, Saragossa, Spain", *Structural Engineering International*, Vol. 16, nº 3, 200-203, 2006, ISSN 1016-8664.

[7] RPX-95, *Recommendations for the design of composite bridges*, Ministry of Public Works, Madrid, 1996, ISBN 84-498-0223-8 (Spanish version).

[8] EN 1993-1-5, *Eurocode 3: Design of steel structures – Part 1-5: Plated structural elements*, European Committee for Standardisation, Brussels, 2006.

[9] Gillet, G., Bouchon, E., Bouvy, B., Le Faucheur, D., Sablon, J-Y., "Les études du viaduc de Verrières", *Bulletin Ouvrages d'Art*, Nº 38, Sétra, 2001.

[10] Principia, *Puente de Llobregat. Diagramas de interacción momento flector – reacción de apoyo*, Principia S.A., Madrid, 2007.

[11] Abaqus, *Standard Users' Manual*, Abaqus Inc., Providence, Rhode Island, 2006.