INTERNATIONAL PERSPECTIVE:

Doubling the width of the platform of the San Pedro bridge in Spain

José A. Torroja,
José M. Simón-Talero,
and Alejandro Hernández

The bridge of San Pedro de la Ribera is located in northern Spain. It was built between 1992 and 1994. The deck of the original bridge is 750 m (2460 ft) long and 12 m wide (39 ft) and was made of post-tensioned concrete. It is a continuous deck divided into six spans: one 75 m (250 ft) span at each end with four 150 m (490 ft) spans in the middle (**Fig. 1**). The deck is curved with a radius of 700 m. The elevation slope is 3.712%. The cross slope is constant, equal to 3.5% over the entire length of the bridge.

The cross section of the deck is a box section. Its depth varies from 3.0 m (9.8 ft) in the middle of the main spans to 7.5 m (25 ft) over the connection area resting on the piers, and the width of the box is 6.5 m (21 ft). Two 2.75-m-wide (9.02 ft) cantilevers are located on both sides of the upper slab of the box, making the total width of the deck 12.0 m (39.4 ft) and providing the platform of the original bridge. The webs are 0.48 m (1.6 ft) thick, and the thickness of the upper slab varies from 0.18 m to 0.35 m (0.59 ft to 1.1 ft). The thickness of the lower slab is 0.25 m (0.82 ft) in the middle of the main spans and increases to 1.5 m (4.9 ft) near the piers.

The deck is a segmental bridge that was constructed using the cantilever method. The heaviest voussoirs located near

Editor's quick points

- This paper describes modifications to an existing bridge in northern Spain to nearly double the width of the deck and meet current Spanish and European code requirements.
- Steel struts were installed to transfer the weight of the cantilevered deck to the base of the post-tensioned concrete box girder.
- A central web was constructed to increase the shear strength of the original concrete box section.
- The existing piers and foundations were determined to be adequate for the additional loads imposed.

Figure 1. The original San Pedro Bridge.

the piers are 3.75 m (12.3 ft) long, and the voussoirs at the center of the spans are 5.00 m (16.4 ft) long. The deck is made of prestressed concrete with a nominal cube strength f_{ck} of 35 MPa (5100 psi). There are two sets of cables on each span. The upper slab has typical post-tensioning cables that are used for segmental construction, while the lower slab has cables that were tensioned after the cantilevers of each span were connected using a central segment.

The piers are made of reinforced concrete with a nominal strength f_{ck} of 30 MPa (4450 psi). Each pier is composed of two parallel walls that are 8.0 m (26 ft) apart. Each wall is a box section with a constant thickness of 0.35 m (1.1 ft). The depth and width vary along the pier. The cross section on the top of the pier is 6.5 m × 1.75 m (21 ft × 5.74 ft), and the highest pier is 81 m (270 ft) high. The piers' vertical reinforcement extends into the deck and overlaps with the deck reinforcement, creating a rigid connection between the deck and the piers.

The foundations of piers 3 and 4 are each composed of 16 concrete bored piles founded on bedrock. The piles are 2.0 m (6.6 ft) in diameter and 20 m (66 ft) long. The pile caps are 23 m × 23 m × 3 m (75 ft × 75 ft × 10 ft). Piers 1, 2, and 5 are placed directly on a rock layer.

Solution for widening the platform

Basis of design for widening the deck

The original platform of the San Pedro bridge deck was 12 m (39 ft) wide (**Fig. 2**). In 2005, the design for the new A-8 freeway was finished, and it was proposed that a second 12 m wide bridge be built, similar to the existing one, that would service each direction of traffic for the freeway. However, before construction of the new bridge began, environmental concerns were raised regarding the feasibility of a second bridge. Therefore, an alternative was considered: widen the existing bridge to a single platform of 23 m (75 ft) (**Fig. 3**).

The following bases of design for widening the deck were established:

- Traffic should not be interrupted during construction.
- The widened deck should be able to withstand the loads defined in the new Spanish code for road bridges,[1] which was approved after the existing bridge had been built.

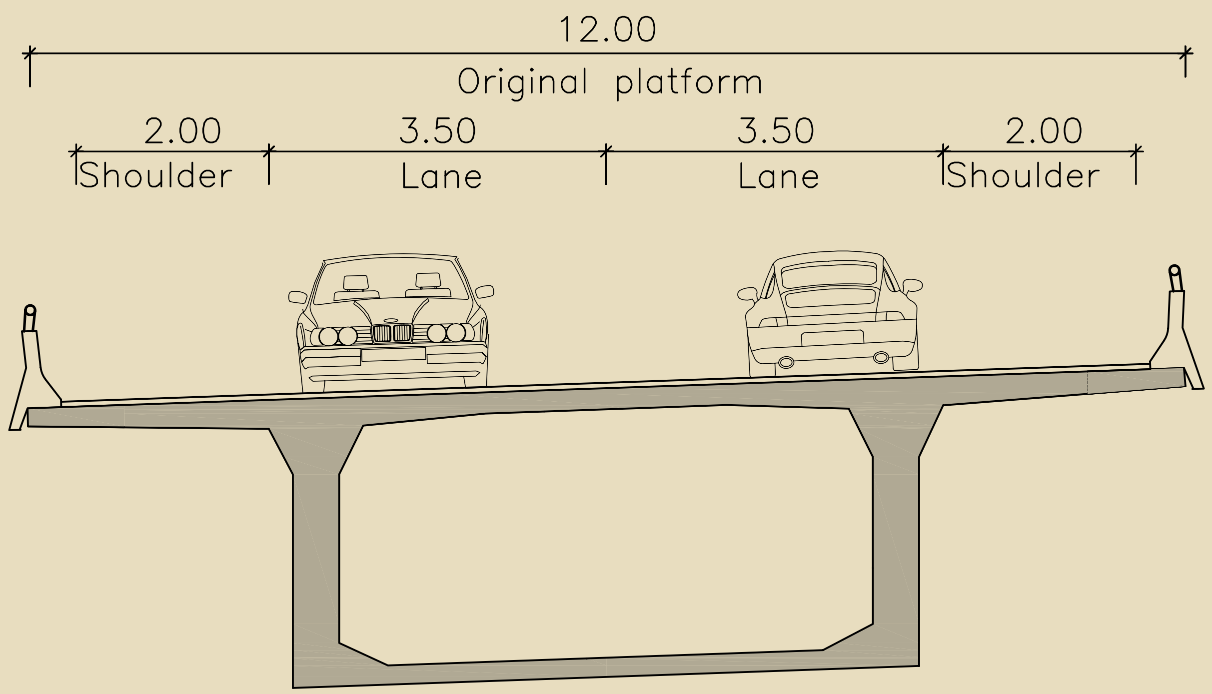

Figure 2. Cross section of the original 12-m-wide deck. Note: All measurements are in meters. 1 m = 3.281 ft.

- Widening the deck from 12 m to 23 m (39 ft to 75 ft) increases the dead load of the structure. The safety factor of the deck and the piers should be equal to the required values defined in the current Spanish codes.

In addition, other requirements were established for the design:

- The existing pier foundations should support the new loads because it is not possible to strengthen the piles and only minor rehabilitation of the pile caps is possible.

- The existing piers should also support the new loads. Minimal changes to the tops of the piers were permitted to properly connect the deck.

- Connecting the widened deck to the existing piers is difficult. It was desirable that no additional external elements be connected to both sides of the pier tops. It

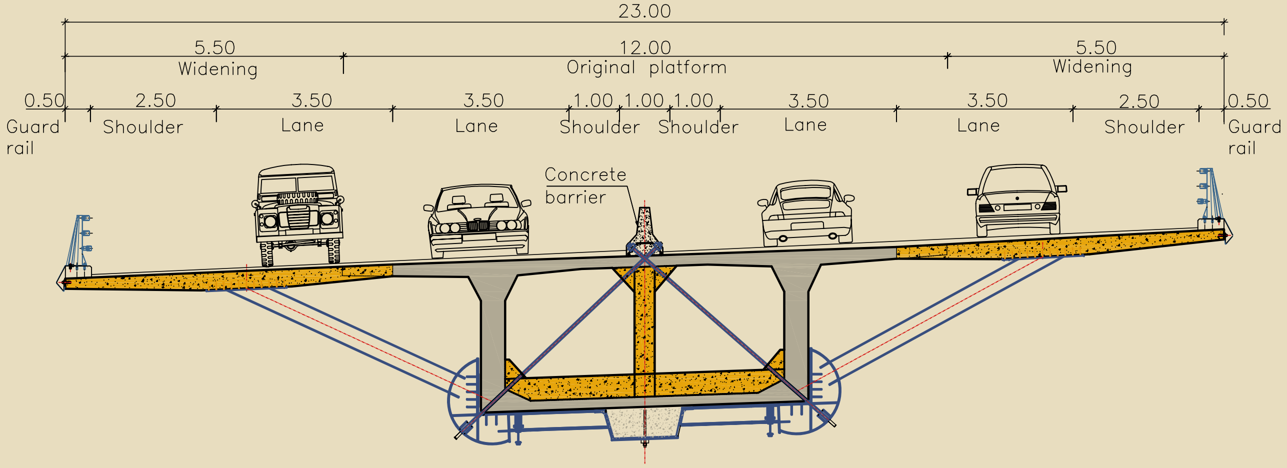

Figure 3. Proposed cross section of the deck widened to 23 m. Note: All measurements are in meters. 1 m = 3.281 ft.

Figure 4. The San Pedro Bridge after widening the platform.

was preferable to connect the deck using the original triangular concrete cell inside the deck.

- If possible, the cross slope of the deck should be increased from the existing 3.5% to 4.5%.

These requirements were all met.

Possible solutions for widening the deck

Two possible solutions for widening the deck were studied, taking into account these criteria. The first solution considered was to add to the existing deck by building a new structure that would withstand the dead load of the deck extensions and some of the additional traffic load. The new structure could be a steel truss, which is lighter than a steel beam structure or a concrete box section.

A detailed study of this solution was completed, and the following difficulties were found:

- Significant differential deflections would be induced because of the different stiffnesses of the existing concrete box girder and the new steel truss.

- The inclination of the steel truss would cause significant horizontal and vertical deflections on the steel truss when the dead load and the live load were considered.

- A preload of the new steel truss would be necessary to minimize the absolute and relative deflections and reduce the extra load supported by the existing concrete deck.

- The connection of the steel truss to the piers would be difficult to build because of the heavy loads to be transmitted to existing elements and the reduced space available for this connection.

The second solution was studied with the intent to mitigate these difficulties that arose with the first solution and involved strengthening the existing deck to support the widened platform and resist the additional loads. In 2007, this became the approved design solution (**Fig. 4**).

The proposed solution included the construction of different elements (**Fig. 5**):

- Two 6-m-span (20 ft) cantilevers support the widened platform. These cantilevers were made of lightweight prestressed concrete with a density γ of 19 kN/m^3 (0.12 kip/ft^3).

- Inclined steel columns support the cantilevers and

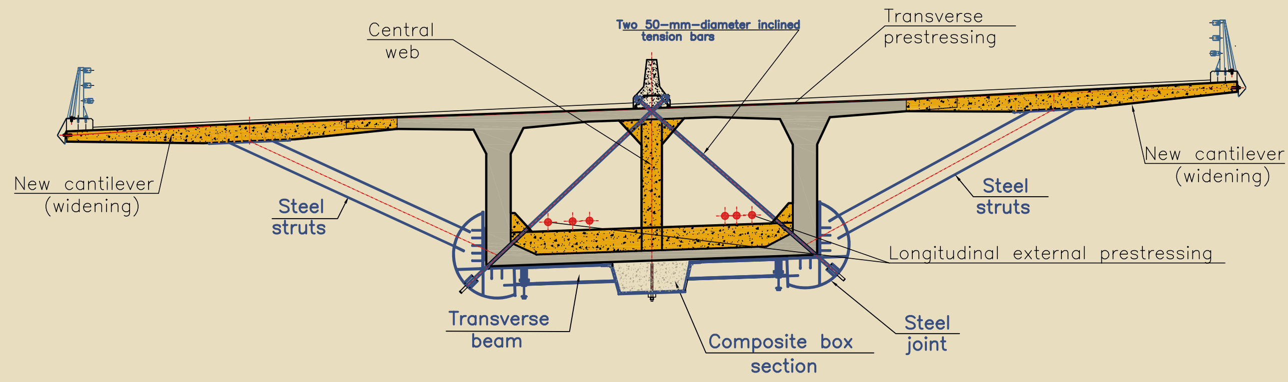

Figure 5. Selected proposal for widening and strengthening the deck.

transfer the vertical loads to the existing concrete box girder.

- Two pairs of steel, diagonal post-tensioning bars connected to the inclined columns transfer the loads to a new central web.
- A new central web made of concrete and vertically prestressed using high-strength steel post-tensioning bars increases the resistance to shear.
- A composite box section connected to the lower slab of the existing concrete box girder increases the resistance of the existing deck for both positive and negative bending moments.
- Two groups of post-tensioning cables placed inside the box section of the deck increase the moment- and torsional capacities of the deck.
- The lower slab and the upper slab of the existing concrete box girder were strengthened using steel plates attached with epoxy.
- The connection of the deck to the piers was strengthened by adding new post-tensioning bars to the existing reinforcement between the deck and the pier.
- The bearings between the deck and the abutments were replaced, and new reinforcement increased the local bearing capacity of the deck.
- Two pairs of impact-transmission devices connect the girder to each abutment.
- The front walls of the abutments were strengthened with post-tensioning bars.

To increase the cross slope from 3.5% to 4.5%, the lateral extensions (Fig. 5) were not exactly aligned with the existing deck but placed with a different slope (4.5%) from the nominal slope. The final transverse line was achieved using lightweight mortar. Further refinement of the surface was achieved using a variable thickness of the base layer of the pavement.

Calculations

Validation of the existing piers

The tallest pier of the bridge is nearly 81 m (260 ft), so wind forces are the most important load when validating the existing reinforcement of the piers' columns. Wind tunnel tests were conducted to permit accurate calculations for the piers and for the widened deck. Pressure coefficients were obtained from the wind tunnel tests, which produced lower values than recommended by the Spanish code of loads for the design of road bridges[1] or by the Eurocode EN1991-1-4[2] (**Table 1**).

For the validation of the piers, a second-order analysis was used. A three-dimensional (3-D) model was created of the piers and deck. The results showed that the existing reinforcement of the pier columns was adequate to resist the

Table 1. Comparison of the Spanish code and results from the wind tunnel test

Wind pressure factors							
Deck				Piers			
Transverse wind		Vertical wind		Transverse wind		Longitudinal wind	
Spanish code	Wind test	Spanish code	Wind test	Spanish code	Wind test	Spanish code	Wind test
1.30–1.64	1.06	Similar		1.13–1.19	0.36	2.09	1.93

Figure 6. Composite box section connected to lower slab.

new loads produced by the widened deck. A detailed finite element model (FEM) was used to validate the pier foundations. A nonlinear distribution of stresses was considered in the FEM to determine the exact load distribution on each pile.

Calculation of the existing deck

The first calculations quantified the effects of the additional loads produced by the widened platform on the existing deck. A detailed 3-D model of the whole structure was made. It considered the effects of creep and shrinkage produced on the original during construction and the 13 years between completion of the bridge and widening the bridge. The results of these calculations showed the following:

- The existing internal post-tensioning cables were inadequate to prevent cracking due to bending moments on the deck. Therefore, the serviceability limit state (SLS) was not achieved.

- The shear and torsional shear stresses were unacceptably high. Again, SLS was not achieved.

- The bending resistance of the center-span section and of the sections of the deck close to the connection of the piers was inadequate to achieve the flexural ultimate limit state (ULS).

- Shear- and torsional ULSs were not achieved.

Structural elements proposed for strengthening the deck

As expected, it was necessary to strengthen the deck. Therefore, different structural elements were proposed.

Longitudinal composite box section connected to the lower slab

The results of the calculations proved that there was insufficient resistance with respect to the flexural ULS in both the center of the spans and on the sections near the connection to the piers. An additional resistant element was connected to the lower slab of the existing box girder (**Fig. 6**). This element had to be a composite section because it was tensioned in the center of the spans, but it was compressed when the section was near the piers.

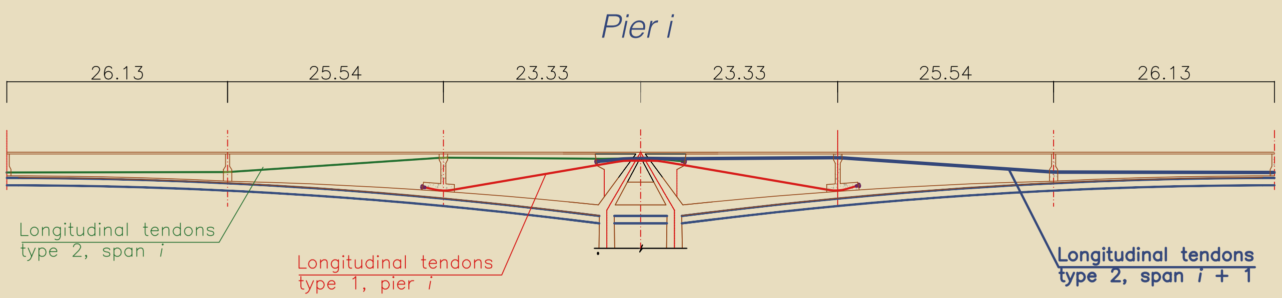

Figure 7. Detail of external post-tensioning. Note: *i* = pier number. All measurements are in meters. 1 m = 3.281 ft.

The composite box section was 1200 m × 800 mm (47 in. × 31 in.). The grade of the steel was S355 (Grade 50), and the concrete was lightweight with an f_{ck} of 30 MPa (4.35 ksi). The connection to the existing deck was made using 40-mm-diameter (1.6 in.), 0.875-m-long (2.87 ft) post-tensioning bars. Because the deck was curved with a radius of 700 m (2.3 ft), special care was taken when building the steel section and cutting vertical holes in the existing deck to correctly place post-tensioning bars for the connections.

External post-tensioning

In addition to the composite box section connected to the lower slab, external post-tensioning cables were added to the existing deck in the existing box girder (**Fig. 7**). Two types of cables were designed on each span.

- Type 1 was made of six tendons 31ϕ0.6 in. (a Spanish designation for tendons that means each tendon was made of thirty-one 0.6-in.-diameter [15 mm] strands). These tendons were tensioned near the lower slab of the deck 23 m (75 ft) from the section of the deck connected to each pier near the upper slab. The six tendons were designed to increase the flexural capacity of the section located over the piers. Because of their inclined geometry, the tendons also contribute to the shear resistance of the deck by transmitting the shear forces in the deck directly to the piers.

- Type 2 was made by another six tendons 28ϕ0.6 in. (15 mm). These tendons were tensioned near the upper slab of the deck near the section connected to each pier. These tendons were designed to increase the flexural capacity of positive bending moments of the midspans.

Deviation walls were necessary to transfer the forces due to the external post-tensioning of the existing deck. Difficulties were created by introducing the post-tensioning forces from type 1 cables to the deck. Therefore, vertical and horizontal post-tensioning bars connect the deviation wall to the webs and to the slabs of the existing deck. Also, at the bottom of the deck near the abutments it was necessary to use post-tensioning bars to transfer the forces of the final anchorages of type 2 cables to the webs of the deck.

The geometry of the tendons is not parallel to the axis of the deck. The tendons placed near the upper slab make a zigzag opposite those located near the lower slab. Using this shape increases the resistance to torsion.

To consider the local effects of the tendons on the deviation walls, a 150-m-long-span 490 ft) FEM was implemented. This model was also used for the detailed study of the distortion of the box section due to eccentric loads.

New central concrete web

A new central web was constructed along the deck (**Fig. 8**). This web is made of lightweight concrete with an f_{ck} equal to 30 MPa (4.4 ksi). It is connected to the upper and lower slabs with post-tensioning bars that are 40 mm (1.6 in.) in diameter and 0.875 m (2.87 ft) long. This new web increases the shear capacity of the existing deck. A large proportion of the shear force produced by the extra load due to the widening of the platform is transferred to this new web. Therefore, the existing lateral webs resist the torsional forces produced by the new loads, but they are not subject to additional shear forces.

Transverse bending moment resistance elements

The existing concrete box section was 6.5 m (21 ft) wide, and the original platform was 12 m (39 ft) wide. Widening the platform to 23 m (75 ft) meant that the upper-slab cantilever span had to be increased from 2.75 m to 8.25 m (9.02 ft to 27.1 ft). Obviously, the existing reinforcement of the cantilevers was not sufficient to resist the new loads. Thus, a new type of structural mechanism had to be implemented. Steel struts were placed at intervals between 4 m and 5 m (13 ft and 16 ft) along the bottom of the existing box section. The top end of the strut was placed 4.65 m (15.3 ft) from the lateral web of the box section, and the bottom end was connected to the lower corner of the bottom box section (**Fig. 9**).

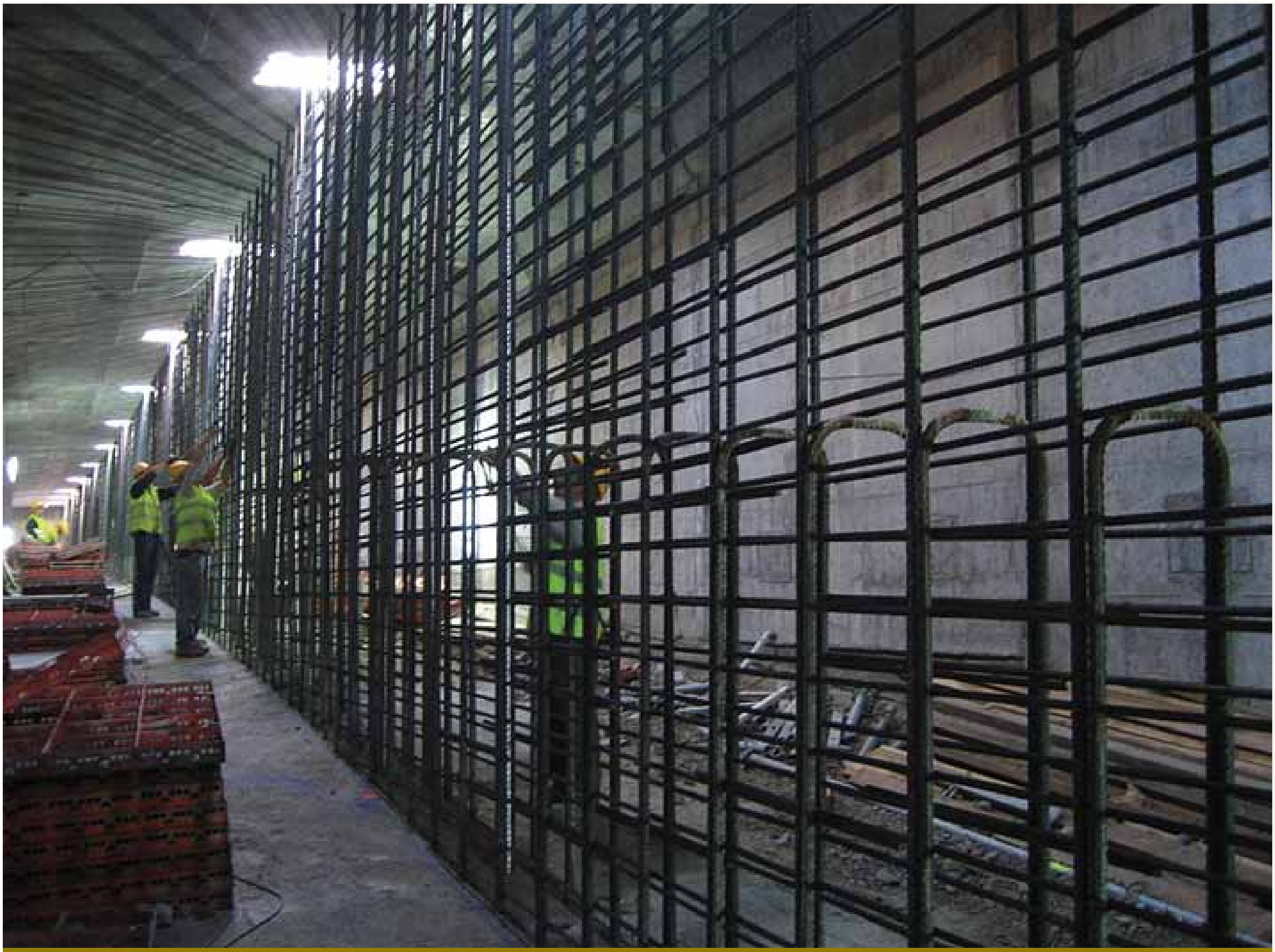

Figure 8. New central web.

These external struts are the compression elements of the strut-and-tie system used to resist the transverse bending moment produced by the new cantilevers (**Fig. 10**). The tension caused in the upper slab was mitigated by transverse post-tensioning 10ϕ0.6 in. tendons. Tendons were placed into a 30-mm-thick (1.2 in.) flat duct.

The struts extend from the cantilever of the upper slab to the lower corner of the original box section, where a steel joint was used to split the inclined force at the bottom of the column into its two components. The horizontal component is transferred to the lower slab using elastomeric bearings. The vertical component of the joint could not be transferred directly to the adjacent lateral web of the original concrete box section because it would increase the load to be resisted by those webs, which have a limited shear resistance. Therefore, two pairs of diagonal post-tensioning bars connect the steel joint to the upper part of the new central web. These diagonal tensioning bars were made of high-strength steel and were post-tensioned to eliminate relative deflections when the total load was applied and to reduce the shear force acting on the lateral webs.

As a result of the described resistance model, high transverse compression forces are applied to the lower slab of the original concrete box section. In the sections close to the center of the spans, the lower slab is not thick enough for stability. Therefore, a new beam had to be designed.

Connection of the deck to the piers

The connection of the deck to the piers was originally made using a triangular cell located in the deck and connected to the walls of each pier. Reinforcement of this concrete triangular cell of the deck went into the piers, so the vertical load and the bending moment of the deck were transferred to the top of each pier. The loads to be transmitted from the deck to the piers are greatly increased when the platform is widened from 12 m to 23 m (39 ft to 75 ft). As a consequence, it was necessary to strengthen the connection. Furthermore, some of the existing reinforcement was cut when making holes in the existing triangular cell for the external post-tensioning of the concrete elements.

Detailed calculations of the connection were performed using linear FEM. The goal was to determine the portion of the loads that were transmitted to the pier by the original strut-and-tie system and the load that was transferred using

Figure 9. Inclined steel columns to support the cantilevers.

the mechanism of shear distortion of the webs (**Fig. 11**). The results of the calculations showed that nearly 55% of the total load was transferred to the piers using the strut-and-tie system and the other 45% was transferred from the webs.

Construction of the bridge expansion

The final detailed design was completed in September 2006. In August 2007, improvements to the soil near the foundation of pier 5 began. In April 2009, the widened bridge was opened to traffic. Different tasks were undertaken simultaneously. Once initial activities were finished on spans, other construction activities were initiated on these spans. The initial activities were simultaneously started on the adjacent spans and so on. A complex space- and time-dependent construction process was proposed to minimize the construction schedule. Sometimes construction would proceed inside the original box section of the

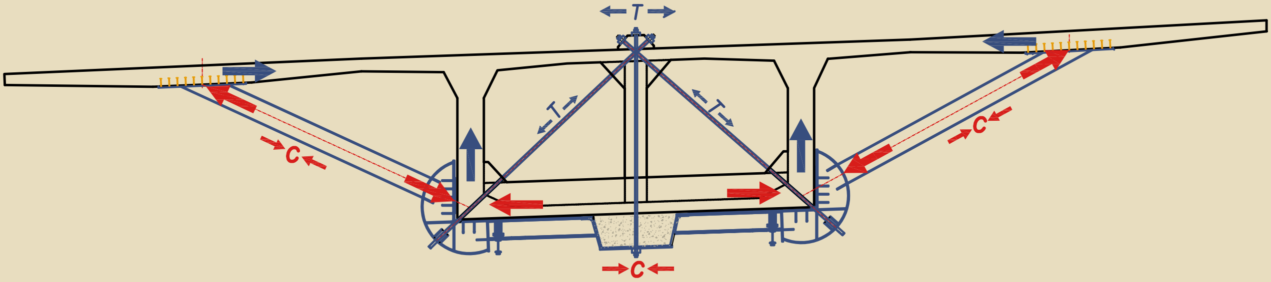

Figure 10. Strut-and-tie system for resisting transverse bending moment.

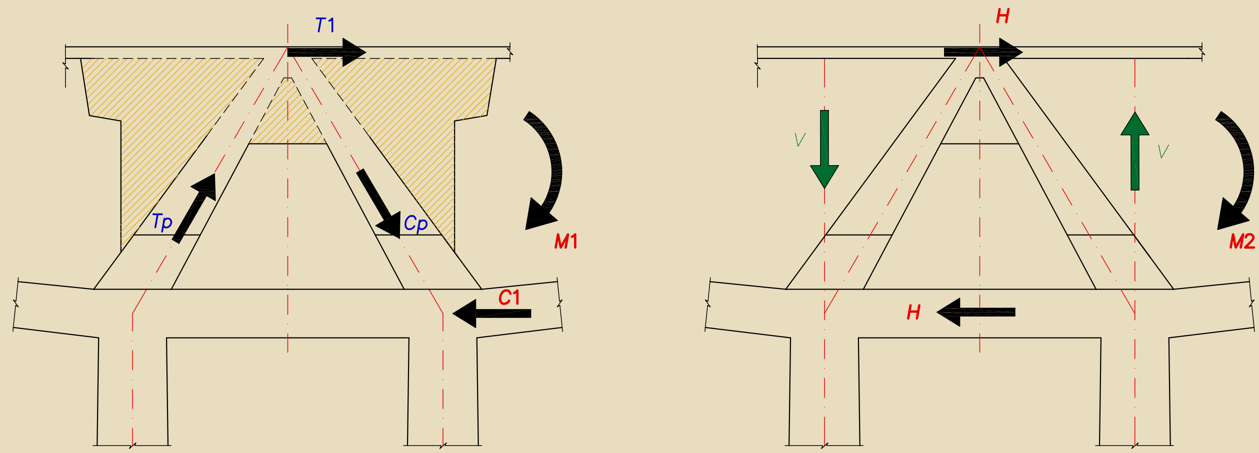

Figure 11. Resistance models of the connection of the deck to each pier. Note: $C1$ = compression on the lower slab: C_p = compressed wall; H = horizontal force; $M1$ = unbalanced bending moment (strut and tie resistant system); $M2$ = unbalanced bending moment (web-shear-resistant system); $T1$ = tension on the upper slab; T_p = tensioned wall; V = vertical force.

deck and on the platform at the same time. The original requirement of not interrupting traffic (**Fig. 12**) while construction was in progress was fundamental in the planning phase of the construction, with a few exceptions.

There were three different types of movable platforms (one of platform 1, two of platform 2, and two of platform 3) that were specially built for the bridge expansion: platform 1, which weighs 68 tonnes (152 kip) and is 23 m (75.44 ft) high; platform 2, which weighs 6 tonnes (13.5 kip) and is 2 m (6.6 ft) high; and platform 3, which weighs 60 tonnes (135 kip) and is 9 m (29.5 ft) high.

Figure 12. Traffic on the bridge using platform no. 1 when strengthening operations were in progress.

Figure 13. Special device used to eliminate the interference of the external longitudinal tendons with the diagonal post-tensioning bars.

It was difficult to install the external post-tensioning where it crosses the diagonal bars located between the lower corner of the original deck and the upper part of the new central web. Also, the curved deck (700 m [2296 ft] radius) made installation challenging. When it was impossible to avoid interference with the existing diagonal post-tensioning bars, a special device was designed (**Fig. 13**) to allow the longitudinal tendon to pass through.

In chronological order, the main activities for the construction of the new platform and strengthening the deck are as follows:

1. Strengthen the upper slab.
    - Demolish the original pavement and replace the original steel guard rails.
    - Cut 50-mm-deep (2 in.) paths for the transverse post-tensioning tendons in the upper slab.
    - Strengthen the upper slab using normal reinforcement and steel plates attached to the concrete with epoxy.
    - Thicken the upper slab using 30 mm (1.18 in.) of fiber-reinforced concrete.
2. Strengthen the original concrete box section (**Fig. 14**).
    - Bore holes in the upper and lower slab for crossing the steel post-tensioning bars that will connect the original concrete with the new web and the lower composite box.
    - Construct the new central web using lightweight self-consolidating concrete.
    - Manufacture and place the composite box section connected to the lower slab. Each piece of the longitudinal composite section was erected jointly with the transverse beams and a pair of lateral joints making a cross-shaped section.
    - Place concrete of the lower composite longitudinal box section using lightweight self-consolidating concrete.
    - Demolish the original pavement and replace the original steel guardrail section.
3. Perform external post-tensioning.
    - Construct the deviation walls inside the original deck. (The deviation walls had to be constructed after the diagonals that connect the steel lateral joints to the central web were tensioned to avoid disturbing the distortion of the original concrete box section when the diagonals were tensioned.)
    - Strengthen the connection of the deck to the piers.
    - Provide first-phase tensioning of the external post-tensioning cables prior to widening the upper slab.
    - Provide second-phase tensioning of the external post-tensioning cables after widening the upper slab.
4. Construct the transverse structure.
    - Tension the vertical post-tensioning bars connecting the longitudinal composite box section with the original deck.
    - Tension the diagonal post-tensioning bars that connect the original concrete with the new central web.
    - Manufacture and place the steel inclined columns.
    - Construct the new 6.5-m-span (21 ft) cantilevers to widen the upper slab (**Fig. 15**).

Figure 14. Composite box placed on-site using platform 1.

- Construct new transverse beams in the lower slab of the original deck to resist potential instability due to the transverse compression transferred by the inclined columns to the existing lower slab.

- Tension the transverse post-tensioning tendons of the upper slab.

5. Finish.

- Place the waterproofing device.

- Build the guardrails.

- Pave the deck.

- Load test (dynamic and static load test) the bridge.

Monitoring

The construction of the new cantilevers, central web, and lower composite box section increased the original loads but also provided additional strength to the bridge. The distribution of new loads on all of the supporting elements was evaluated with detailed calculations that were validated through monitoring some of the elements. Stress gauges, inclinometers, and temperature sensors were installed on the bridge to validate the following:

- distribution of vertical loads among the three webs

- loads transferred to the lower longitudinal composite box section

- structural behavior of the supporting system (distribution of loads of the joint located in the lower corner of the original concrete box section)

- loads transferred to the piers

All of the devices were connected to a computer, and real-time measurements were sent to the consultant. **Figure 16** compares the measured rotation of the top of pier 3 with the calculated rotation. The accuracy of calculations was validated by the measurements.

Cost and material quantities

The total expected costs for the construction amounted to nearly 13 million euros ($19 million).

The quantities of the most important materials used for the construction are as follows:

Figure 15. New cantilevers built with platform 3.

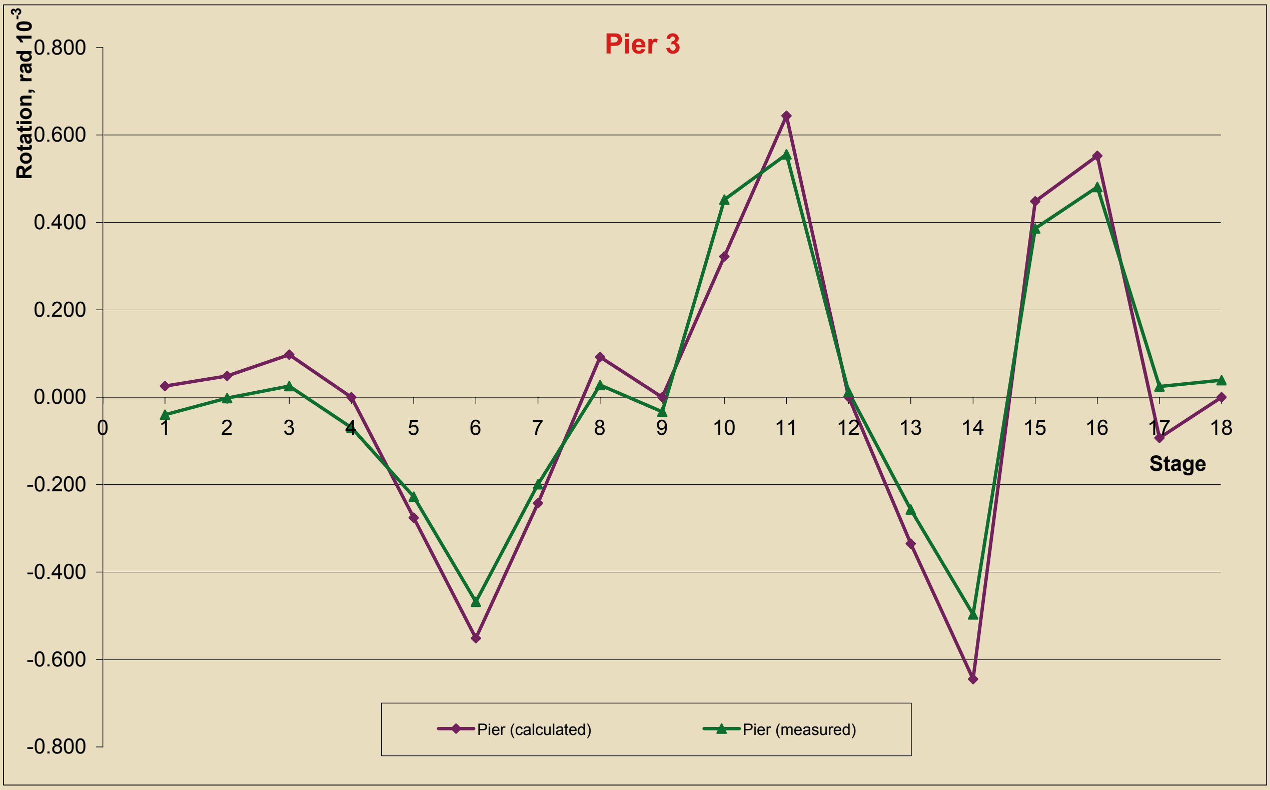

Figure 16. Comparison of measured and calculated rotation of the top of pier 3.

- lightweight self-consolidating concrete: 6700 m^3 (8800 ft^3)
- reinforcing steel: 940,000 kg (2100 kip)
- post-tensioning cables: 320,000 kg (710 kip)
- post-tensioning bars: 190,000 kg (420 kip)
- structural steel: 1,390,000 kg (3060 kip)

Acknowledgments

The bridge owner is the Ministry of Public Works in Spain. Construction design was performed by José A. Torroja, José M. Simón-Talero, and Alejandro Hernández of Torroja Ingeniería SL in Madrid, Spain. The contractor was Cudillero Joint Venture, Spain (Antonio Navarro, Arturo Bahamonde). Technical assistance during construction was provided by Ignacio García–Arango and Jesús Villameriel of the Ministry of Public Works in Spain; José A. Torroja, José M. Simón-Talero, and Alejandro Hernández of Torroja Ingeniería SL in Madrid, Spain; and Manuel Serralta and Manuel Juliá of AEPO in Madrid.

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Notation

$C1$ = compression on the lower slab

C_p = compressed wall

f_{ck} = characteristic compressive strength (cube strength) of the concrete

H = horizontal force

i = pier number

$M1$ = unbalanced bending moment (strut-and-tie resistant system)

$M2$ = unbalanced bending moment (web-shear resistant system)

T = tensile force

$T1$ = tension on the upper slab

T_p = tensioned wall

V = vertical force

γ = unit weight

About the authors

José A. Torroja, PhD, a civil engineer, is president of Torroja Ingeniería SL in Madrid, Spain, and professor for the Structural Engineering Department at the University of Madrid.

José M. Simón-Talero, a civil engineer, is technical director for Torroja Ingeniería SL and associate professor for the Structural Engineering Department at the University of Madrid.

Alejandro Hernández, a civil engineer, is a structural engineer for Torroja Ingeniería SL.

Synopsis

The San Pedro Bridge has six spans and is 750 m (2460 ft) long, 88 m (290 ft) high, 12 m (39 ft) wide, and curved with a radius of 700 m (2300 ft). It was built in 1993 using the cantilever method. Its superstructure is a prestressed concrete box girder with main spans of 150 m (490 ft). In 2008 and 2009, the width of the platform was enlarged to 23 m (75 ft) using five movable sets of scaffolding. The bridge remained open to traffic during construction. The original platform was widened 6 m (20 ft) on each side by connecting a new lightweight concrete cantilever to the original upper slab. These cantilevers were supported by steel struts. The tie into the upper slab was made with new transverse post-tensioned tendons.

The original superstructure was strengthened to resist the additional dead load of the expansion and live loads of the extra traffic. An additional new central web and a composite concrete-steel section were constructed and connected to the concrete box and central web using vertical high-strength post-tensioning bars. Also, external post-tensioning cables were implemented.

It was also necessary to strengthen the connection of the original concrete box section to the piers. Detailed calculations were made to evaluate the load distribution transmitted to the piers by the webs and by the original inclined concrete walls of the box girder. Finally, a detailed second-order-analysis of the complete structure was made to guarantee the resistance of the piers compared with actual loads.

Keywords

Bridges, cantilever, post-tensioning, rehabilitation, widening.

Policy Note

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