Analysis, Design and Construction of Two Extremely Skewed and Slender Post-Tensioned Concrete Bridges

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Abstract

The paper summarises the main issues found in the design, construction and launching of two large and highly skewed post-tensioned concrete slabs built along the A4 motorway connecting Milan and Turin.

The twin slabs (one for each carriageway) are characterised by a very high skew (61°) and a record span-to-thickness ratio. Each slab is 50 m long, 20 m wide and only 0,8 m thick so as to guarantee the underclearance required for the railway lines passing underneath. In order to strengthen the slabs, lateral edge beams shaped in the form of crash barriers have been added along the free sides.

The two structures were cast on-site and post-tensioned beside the abutments before being push-launched over the railway lines without intermediate supports.

Keywords: bridge construction; bridge design; bridges; skew; post-tensioning; slabs; conceptual design.

Introduction

The A4 motorway is the main road infrastructure running in the East–West direction along the so called European Corridor V from the Slovenian border to the French border, via Venice, Milan and Turin. Outside Milan driving westbound, the motorway crosses the railway corridor connecting Milan to north-western Italy, Switzerland and France. The original crossing consisted of two concrete bridges, one for each carriageway, built in 1970 to 1980. Each bridge had two 25 m spans. Widening these bridges to accommodate two additional lanes for each carriageway had to be ruled out because of their extensive deterioration. The replacement structures had to have the same overall length but twice the span as the railway agency required the existing intermediate supports to be removed based on maintenance and safety requirements. While doubling the span, the two structures had to maintain the same deck thickness of the previous ones so as to guarantee the underclearance for the railway lines.

These constrains were fulfilled with two highly skewed post-tensioned slabs, spanning 47 m, with an average depth of only 0,80 m. Because such a thickness was not sufficient to guarantee the required load-carrying capacity, the free sides of the slabs were strengthened with extradosed beams doubling as stiff crash barriers for the motorway (*Fig. 1*).

Slab Geometry and Post-Tensioning Layout

Preliminary analyses pointed out that the maximum allowable deck thickness (0,8 m) was insufficient for the given span and geometry. The span of each slab measured perpendicularly to the abutments is only 22 m compared to the 47 m length (span) of the free sides aligned with the motorway. These free sides were too long to support their self weight and the variable loads with such a small thickness. Since the longitudinal profile of the highway could not be raised, the only viable solution, apart from increasing the slab thickness and thus reducing the underclearance, was introducing extradosed edge beams along these free sides. A systematic investigation of skewed slabs with stiffened free edges is not available, however the behaviour of skewed slabs has been discussed in detail by various authors.[1–3] Strengthening the free sides proved to be effective, although edge beams attract stresses and made design of reinforcement and post-tensioning much more complicated compared with slabs without edge beams. The post-tensioning system consists of three sub-systems of tendons:

- Longitudinal post-tensioning tendons in the edge beams: each beam is post-tensioned with seven tendons of 19 strands each.
- Transverse post-tensioning tendons in the slab: each slab is transversely post-tensioned with 43 tendons of 15 strands each.

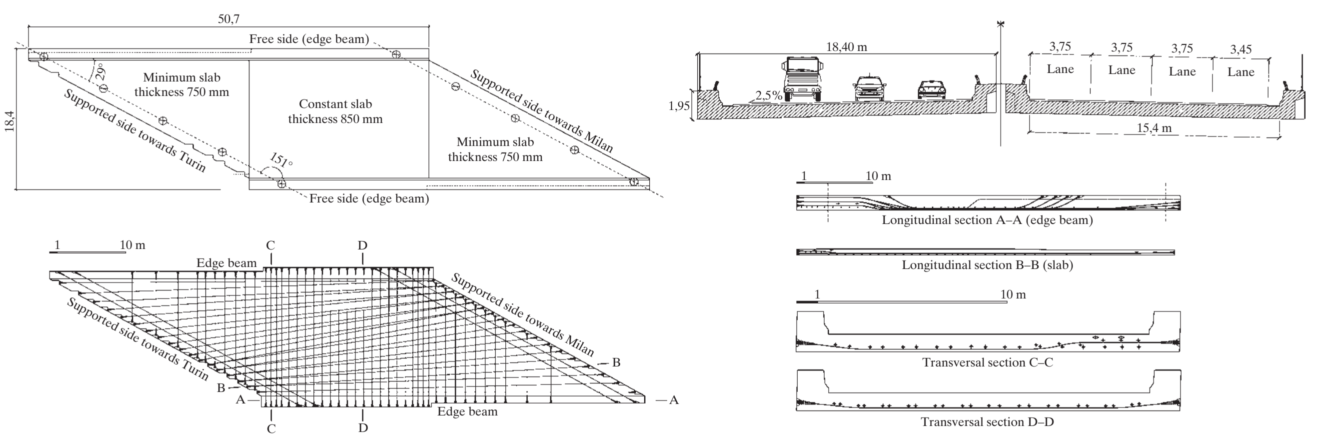

Fig. 1: Slab geometry and prestressing layout

- Longitudinal post-tensioning tendons in the slab: each slab is longitudinally post-tensioned with 24 tendons of 19 strands each.

All strands are of 15,24 mm diameter with 150 mm^2 cross section. The post-tensioning layout had to be achieved by compromising between maximum structural efficiency and economy or reliability of construction. For instance, anchoring is simpler and cheaper when perpendicular to the outer surfaces. With such a skew, achieving square angles is rather difficult and even when possible, not always providing the maximum structural efficiency. A satisfactory compromise could only be found after extensive numerical simulations.

Analysis and Design

Complete three-dimensional (3D) finite element models of brick slabs in the various launching phases were set up and analysed using a software. Given the geometry of the slabs, 3D modelling was the only viable approach, as a plate model could not properly account for the effect of the extradosed edge beams on the slab response nor provide suitable approximations of the forces arising in these elements.

The finite element mesh was aligned with the actual structure. The eight-node isoparametric brick elements were therefore skewed. The side length of the brick elements is roughly 1,0 m, while the thickness is 0,40 m (two superimposed elements are used across the slab thickness). Meshing of the edge beams is similar (3 × 4 elements across the section). Although still slightly coarse for local stress calculation, globally the mesh of a single slab consists of 6456 nodes and 4126 brick elements. Analyses of the launching phases required modelling of the three overhead steel trusses placed above the slab, each made of 167 beam elements connected to the slab top surface with seven spring elements each.

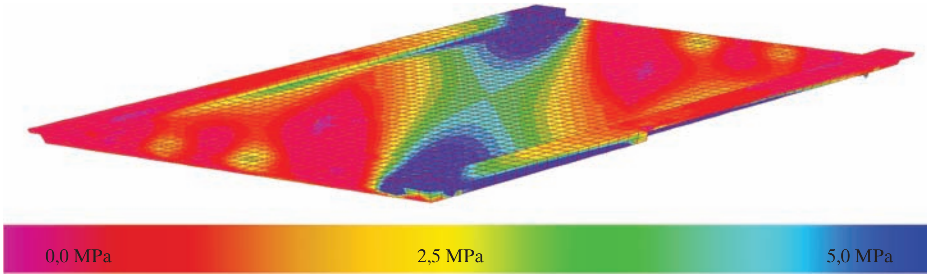

Fig. 2: The slab mesh and the principal tensile stress under service loads

The slabs were analysed and forces/stresses checked for all the different boundary configurations assumed during the push-launching drive (roughly one analysis for every 4 m drive) and for all the service conditions under different live load configurations. The structural response under service loads (30 kN/m^2 circa, self weight and uniformly distributed live load) shows a significant saddle effect where the load is carried by the slab strip aligned with the two obtuse corners, as shown in *Fig. 2*. Without the edge beams, this saddle-like behaviour would carry most of the applied load with the free sides of the slab undergoing large vertical deflection. It should be noticed that in the slabs under consideration, the very high skew is associated with an unfavourable span-to-width ratio, causing this saddle strip to be relatively narrow because the slabs are not wide enough with respect to both the span and the skew. A larger slab with the same skew would develop a larger saddle strip proportionally larger with respect to the overall slab area.

Bearing	**Reactions (% of total dead load)**	
	without edge beams	**with edge beams**
1	57	59
2	8	3
3	19	15
4	13	13
5	3	10

Table 1: Reactions distribution with and without edge beams (bearing 1 under obtuse angle)

Introducing the edge beams does change the reactions at the 5 + 5 equally spaced bearings (*Table 1*). Nonetheless, the increase of the vertical reaction at the obtuse corner is almost insignificant (57–59%) and it was further reduced by introducing suitable distortions. This has a positive effect on the slab stresses because the strength of the slab at the obtuse corner with the edge beam is much more compared to the same slab without edge beams. To further increase shear and torsional strength at the obtuse corner, the edge beams shaped like an inverted L along most of the span, become rectangular closer to the above mentioned supports (*Fig. 3*).

When lowering the slabs into their final position, specified distortions

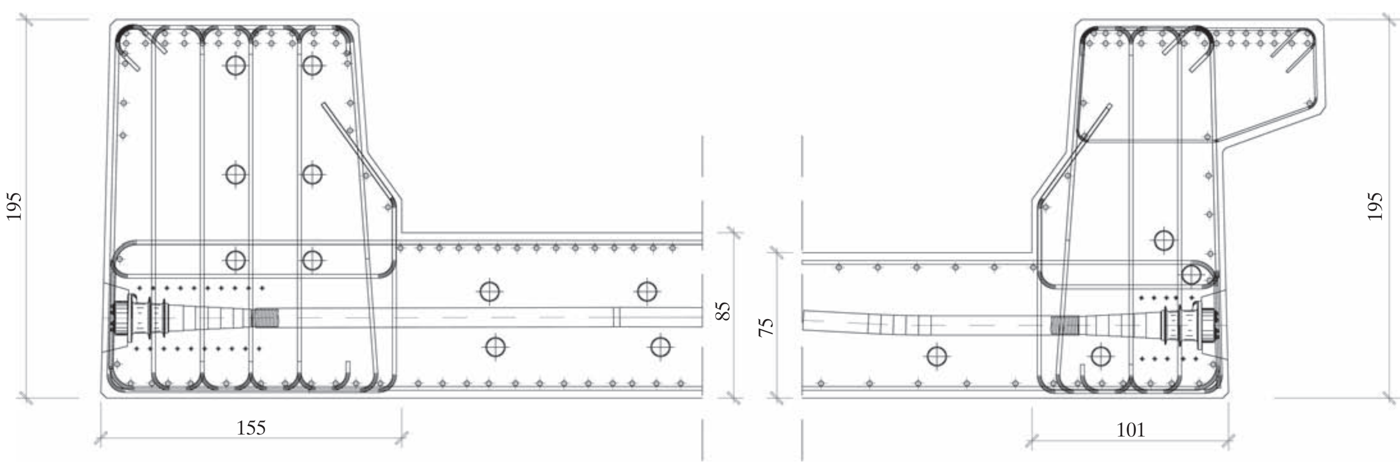

Fig. 3: Detail of cross section (slab and edge beam) near the obtuse (left) and acute angles (right)

Bearing	Reactions		Distortions
	Theoretical (t)	Imposed (t)	Theoretical (mm)
1	685	635	—
2	45	90	4
3	185	180	10
4	160	140	21
5	110	140	39

Table 2: Support reactions and theoretical distortions under permanent load (bearing 1 under obtuse angle)

were applied in order to optimise the force distribution in the structures; basically the slabs were warped by raising the acute corners so as to increase their load carrying quota (see *Table 2*). In any case, given the stiffness of the slabs and the construction tolerances, tuning (adjusting) the reactions when lowering these structures was mandatory because these reactions could otherwise be totally different from those arrived at by finite element (FE) analysis, where perfect planarity of the slabs and of the bearings on the two abutments is assumed (*Table 2*, second column). Consequently, the actual adjustment (shimming) of the supports required to achieve the design reactions (*Table 2*, third column) can be significantly different from the theoretical ones (*Table 2*, fourth column).

The layout of the post-tensioning tendons in the slab has been arranged so as to optimise their effect and reduce their overall quantity. Transverse tendons were required to fan out from the obtuse corners. But it was deemed too impractical because of ever changing angle of attack (anchorage) of each tendon. The numerical analyses showed that the configuration with tendons perpendicular to the longitudinal free sides is only slightly less efficient provided that their longitudinal spacing is properly tapered. These tendons that run along the bottom of the slab to counteract sagging were raised close to the obtuse corners in order to counteract the built-in effect provided by the edge beams. Contrary to transverse tendons the longitudinal ones could not be laid parallel to the edge beams without incurring a significant loss of efficiency. With only a small rotation it was possible to align them to the line connecting the two obtuse corners. Because these tendons are very effective, their count in the surrounding obtuse corners had to be increased to technically feasible numbers. This was achieved by arranging them on two planes across the slab thickness with the transverse tendons running in between. In these areas, the slab thickness was therefore increased to 0,85 m, while close to the acute corners, the thickness was decreased by the same amount (from 0,80 to 0,75 m).

A significant amount of reinforcement was added to the two slabs, especially around the obtuse corners, in order to obtain a ductile response during the launching phase when the insurgence of tensile stresses could not be ruled out. In these areas of roughly 10×10 m^2, the slabs have two orthogonal layers of 26 mm diameter bars at 125 mm spacing (D26@125) on each face. The same reinforcement configuration but with the bar spacing increased to 250 mm is used in the other parts of the slabs.

The main issue to be addressed in the edge beam design was the severe sagging at mid-span. Because these edge beams are attached to the slab, post-tensioning is far less effective compared to stand-alone beams of similar size. In order to obtain a reasonable efficiency of the post-tensioning, tendons were deviated upwards with a small vertical radius and anchored at the upper surface of the edge beams close to mid-span such that deviation forces are applied wherever needed. Shear and negative bending moment in the edge beams close to the obtuse corners can be effectively counteracted by the longitudinal post-tensioning because tendons are inclined where maximum shear occurs and then run along the upper surface of the edge beam in order to resist the negative bending moments (built-in effect) caused by the extreme skew. As far as torsion is concerned, standard reinforcement was used instead, with closely spaced stirrups (two D20@125) and ties (four D20@250), as shown in *Fig. 3*.

Construction and Launching

The two slabs were cast and post-tensioned on the Milan side embankment, before being push-launched into their final position.[4–6] In the preliminary phase of the design, launching was conceived to take advantage of the *ad hoc* strengthened concrete piers of the old viaducts, so as to reduce the forces in the launching equipment and in the slabs, by halving the free span. This solution would have required interruption of the railway traffic to demolish these piers in the second phase. Instead, the client decided to simultaneously demolish the existing structures—deck and piers—and to launch the two slabs without intermediate supports (*Fig. 4*).

Fig. 4: The first slab halfway through launching, rail traffic unaffected

Fig. 5: Front connection between the launching girders and the slab

The mixed behaviour of the two laterally stiffened slabs offered the possibility of adopting different solutions. The final one was a compromise, as the edge beams were used to guide and push the structures, but the launching was carried out using three overhead steel spatial truss girders, each 87 m long and with a bending capacity of 5000 t m each. These girders were anchored to the upper surface of the slab for roughly 40 m; hence, the remaining 50 m cantilevered out in front of the slabs. A transition segment with a stiff-plated girder provided the continuous sliding surface, aligned with the slab intrados, needed for skidding (*Fig. 5*) and the local punching strength to

Fig. 6: The first slab is skidded along the abutment into its final position

resist the reaction of the skates. The connection between the trusses and the slabs was made with 60 mm diameter pre-tensioned steel bars anchored into the slabs. Further anchorage capacity was provided by provisionally anchoring each truss with eight longitudinal post-tensioning tendons. The transverse position of the trusses was chosen so as to evenly distribute the load among the three trusses. Two trusses were therefore placed adjacent to the edge beam passing through the forward obtuse corner and the third truss was placed alongside the other edge beam.

The slabs were cast on two 100 m long concrete launching walls that allowed the launching trusses to be erected on the Milan embankment. The first 50 m run was then used to launch the trusses over the railway lines and the subsequent 50 m to launch the slabs. Once the longitudinal launching of the first slab was completed, the same slab was used to skid the launching girders back onto the Milan side to be re-used for the launching of the second slab. Subsequently, the first slab was laterally skidded along the abutments into its final position in order to allow the casting yard and the provisional launching walls to be re-used for the second slab (*Fig. 6*).

Although it was expected to be straightforward both slabs refused to move on a straight line even though pushing forces were applied from the back by means of two 200 t jacks self-anchored against the concrete launching walls, and the steel trusses were longitudinally aligned and resting on the landing skates on the opposite abutment. During the launching process, the two obtuse corners of the slabs that attract most of the vertical reactions were the one in front and one in the back; hence, different friction coefficients were applied, because at the forward end the sliding occurred between the steel truss girders and the skates, while at the rear end the sliding occurred between the slabs and the launching walls. In this situation, friction forces along the two alignments were significantly different causing a strong torque to be applied to the slabs that had to be counter-reacted by adjusting the thrust of the two rear jacks and by adding other temporary guiding points along the drive.

Half way through the launching phase (*Fig. 4*), the slab and the three steel trusses, acting together as a composite structure, would have experienced very high negative moments when cantilevering out of the provisional launching walls. Use of provisional post-tensioning to counteract these moments, however, was considered too cumbersome and expensive. As an alternative, the slabs were continuously kept under positive bending (sagging) by imposing suitable distortions to the launching truss girders via the hydraulic skates positioned on the landing abutment across the railway lines. This gave the structures a concave configuration similar to the one experienced under service conditions. This solution was simpler, cheaper and much more effective than temporary post-tensioning, given the sizable lever arm (50 m) available between the point of application of the distortion (hydraulic skates on the landing abutment) and the points of maximum negative moments (provisional launching walls on the Milan sides).

Conclusions

Owing to a number of specific site constraints, the A4 motorway crossing over the railway corridor in the outskirts of Milan required the construction of two peculiar concrete structures that were very large, highly skewed and extremely slender. The design and construction of these structures brought in a number of interesting and rather general issues found in other post-tensioned concrete bridges as well.

- Extremely skewed slabs may benefit from the insertion of edge beams that stiffen the free sides, providing a type of regularising effect on the slab response.
- Extradosed concrete structures have a great potential and may be used in a variety of situations, as an alternative to standard beam-and-slab systems. Contrary to extradosed steel structures, extradosed concrete structures are insensitive to buckling and to accidental impacts, which are the most serious causes of concern in steel ones.
- Bridge construction by push-launching can be very fast and straightforward and should always be taken into consideration when erecting large concrete structures.

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