TORSIONAL BEHAVIOUR IN BEAM AND SLAB DECKS

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Abstract

The paper deals with the torsional behaviour of bridge composite girders, built with plated I beam and top concrete slab. This typology has different variants in the way transverse bracing (diaphragms) are designed, either strut-and-tie type or flexural, and in the optional presence of a bottom horizontal torsional bracing.

The torsion stiffness for these girders is made of different contribution: a) the "equivalent Bredt" torsion stiffness of the girder cross-section; b) the concrete slab own torsion stiffness; c) the differential bending in both beams and slab (warping). Depending on the cross section bracing arrangement and overall bridge geometry, the importance and the necessity for the three different contributions varies significantly. Curved bridges, also discussed in the paper, poses further questions strictly connected to torsion.

The paper clarifies and compares the different solutions with the support of F.E. analyses and with the direct experience of designing and building few recent examples along Italian motorways and railway lines. The F.E. study shows how to account for the different torsional contributions when simplified grillage types of meshes are used.

1. Introduction

Composite deck girders became very popular in Italy in the late '90 and early '00 for a number of different reasons, some of them peculiar to the Italian situation, other supported by objective advantages intrinsic to the technology. Among them:

- very low prices of structural steel plates (down to 300$ per ton),
- significant improvements in coating efficiency and development of auto protective steel grades,
- speed of fabrication (fully automatic welding) and erection compared to concrete cast in situ solutions and handiness of transportation,

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- reduced demand for on site skilled labour.

In Italy further advantages where boosted by the comparison with much overused precast prestressed simply supported beam and slab decks. These types of decks, built in the ’70 and ’80 were, by then, significantly deteriorated because of aging of joint and bearings (each span requires up to 10 bearings and a joint), concrete carbonation and spalling because of water leaking trough the joints. The latter being the most important single cause of poor driving comfort given the high incidence of bridges and viaducts along the hilly Italian countryside.

Last but not least significant savings in material and labour cost were obtained with more advanced design, mostly torsionless, with simplified transverse connections and overall savings in secondary elements detailing as discussed in the following.

2. Recent experience along Italian motorways and railway lines

Reference will be made to the overpasses of the Asti-Cuneo motorway and those over the high speed railway line Turin-Milan and adjacent motorway.

Figure 1: Location Map of Asti-Cuneo and Turin-Milan motorways

The first one is a new motorway, currently under construction, requiring 20 new overbridges for crossing roads and junctions, most of them rectilinear although with varying degrees of skew, and only few of them curved, with radius as small as 200m. The client, a privately owned motorway concessionaire, gave the designer complete freedom in the design of the new structures with the only constraint being the minimum clear span over the new motorway (36m circa required for crossing 2 carriageways of 3 lanes each) without intermediate supports. This constraint, together with the above mentioned historic low prices of structural steel, made the choice for composite deck an obvious one. It should be considered that in Italy, contrary to other north European countries, cast in situ concrete bridges are simply not an option, even when scaffolding from the ground does not obstacle the traffic flow as for the case under consideration (new motorway construction). The composite deck where therefore made of steel

I beams connected by normal profile transverse beams (typically IPE 500-600) as shown in Figure 2.

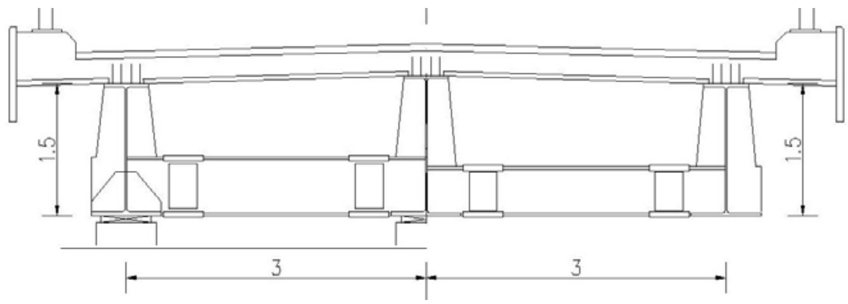

Figure 2: Typical cross section of the Asti-Cuneo motorway overpasses

The cross section arrangement, although already used in Italy, was not so popular as for example in France. Especially for skewed and curved bridges, the so called "open box girder" arrangement (with bottom horizontal torsional bracing) was still the main choice among structural engineers.

Another peculiarity of the Italian constructions technology is the use of prefabricated concrete slabs (*predalle*) used as permanent formworks for casting the deck slab, thus eliminating the need for the travelling formworks generally used elsewhere. This peculiarity in turn does have an influence on the number and spacing of the main beams. Although the 2 beams configuration can generally be used up to 12-14m width platform, this is not optimal for the *predalle* because the 2 beams configuration would require a single, self supporting, *predalla* resting on the 2 beams, making it a bit heavy and awkward to handle. Another problem with using the *predalle* is obtaining a variable slab thickness, generally required to reduce weight when main beams are wide apart and transverse bending significant. Therefore, 3 or more beams are often used reducing the size of the *predalle* and allowing a typical constant slab thickness of 25-28cm (5-7cm *predalle* with 20-22cm cast in situ).

Although the Turin-Milan new high speed railway line runs close and parallel to the homonymous motorway, the existing overpasses could not be used. They were therefore demolished and replaced by 50 new structures, crossing the new railway line and the adjacent motorway. The resulting structures came out to be quite imposing as they run high on ground because the railway itself runs well above ground for hydraulic reasons and requires a further 7m clearance. Maximum spans are also the largest realized so far for this type of structures since the motorway is being widened to 4+4 lanes plus 3m stopping strip each carriageway taking the maximum clear span to 50m circa with no intermediate supports between the carriageways.

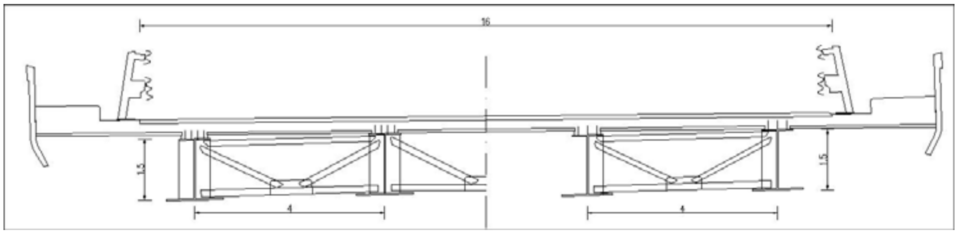

Figure 3: Cross section of the Turin-Milan overpasses

The overpasses can be divided into two groups: the first one includes all the interchange viaducts that do not cross over the railway line, the seconds, crossing both the motorway and the railway, consists of all the other bridges serving national and provincial roads. All overpasses have the central part, crossing the two infrastructures, made of a continuous composite deck. Additional simply supported precast concrete girders are used for the approaches.

The first group, although made of curved bridges with radius as small as 70m, sport a torsionless design with plated (flexural) cross-beams at 5m centre, similarly to the Asti-Cuneo scheme. The second group, since crossing the rail, was subjected to stringent verifications by the railway technical office. Torsionless design was not accepted and therefore the beams are braced in the horizontal plane and have truss type transverse beams. The beams where lift and put in place in couples during night time with temporary interruption to traffic (incidentally the most trafficked motorway in Italy). Bracing between these couplets was carried out before lifting them into position. More awkward was fixing the bracing between the couples in their final position over the motorway within the time slot allocated by the motorway owner. Shear type bolted connection of the bracings, (friction alone would not provide sufficient strength), did not allow for much of a tolerance. Another factor complicating the on site assembly of the whole bridge was the temperature as the steel beam were mostly lifted in place on very cold winter nights, having been manufactured in different part of Italy at a different time of the year.

Photo 1 – Lifting the SS11 overpass into place

3. The torsional behaviour

Torsional behaviour of beam and deck slab is made of three different contributions, namely, the Saint Venant torsional stiffness of the deck (if any), the torsional stiffness of the concrete slab and the section warping. A review of these three components shall be carried out with respect to the cross-section depicted in Fig.4. This section is assumed to be constant over the whole span. Vertical stiffeners are at 6m centre and have 15mm thickness. Diagonals of transverse bracings are made of two 2L150*75*10, the bottom chord with ½ IPE550. Bottom horizontal bracing, sporting a "diamond" configuration, are made of ½ IPE400.

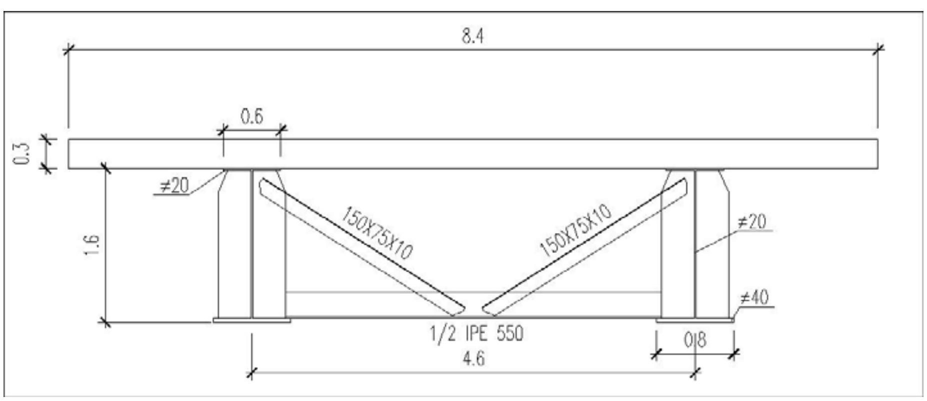

Figure 4: Section geometry used for the test cases

3.1 The Bredt contribution

Only with horizontal bracing between the bottom flanges of the longitudinal beams a significant torsion stiffness arises. This stiffness can be calculated with the formula for thin walled hollow sections once the *equivalent thickness* for the bottom horizontal bracing has been found, using, for example, the expression proposed by Kollbrunner [1]:

$$t^* = \frac{E}{G}\frac{ab}{\dfrac{d^3}{2A_d} + \dfrac{a^3}{6A_s^*}} \tag{1}$$

Where "a" is the bracing spacing, "b" that of the main beams, "d" the length of the diagonals, "A_d" their area and "A_s^*" the area of the beam bottom flange plus 1/3 of the web. The section under consideration with diagonals made of ½ IPE400 has an equivalent thickness of 1.5mm rising to 1.9mm with ½ IPE500. The Bredt torsional stiffness, assuming an intermediate equivalent thickness of 15mm, and a homogenization factor between concrete and steel of 6, is therefore equal to:

$$I_t = \frac{4\cdot\Omega^2}{\sum \dfrac{l}{s}} = \frac{4\,(4.6\times1.75)^2}{\dfrac{4.6}{0.05} + \dfrac{4.6}{0.0015} + \dfrac{2\times1.6}{0.02}} = 0.078\ m^4 \tag{2}$$

3.2 The deck slab contribution

The Saint Venant torsion stiffness for a flat rectangle is equal to $I_t = b\ s^3 /3$. For the section under consideration, it gives:

$$I_t = 8.4 * 0.3^3 / (3 * 6) = 0.013\text{m}^4 \tag{3}$$

In case of grillage analysis (see [2]) this stiffness must be partitioned between the longitudinal beams and the transverse ones. A 50-50 partition is generally used.

The example under consideration shows the slab contribution can easily account for more then 15% of the total torsional stiffness. This percentage rise sharply if the horizontal bracing is not fully efficient because of slack/buckling of members, yield of friction bolts, etc... In torsionless design, the deck slab contribution remain the only one as the steel beams posses none of their own.

3.3 Torsional resistance by section warping

Although hardly a common practice, calculation of the section warping constant can be very useful. For the section under consideration we start calculating the shear centre Y_c. With bottom horizontal bracing it falls 22cm below the slab axis, without it is placed 9cm below the same axis. The warping constant for the slab is then found as follows:

$$\omega(x) = Y_c\, x \qquad I_{\omega,s} = 2 * 0.3/n * \int \omega^2(x)\, dx = 2 * 0.3/n * Y_c^2 * x^3 / 3 \tag{4}$$

Where *x* is the abscissa measured along the slab axis and *n* the homogenization factor.

Similar expressions are used to calculate the contributions of the steel beam main web and flanges. The results are summarised in the table below.

	Concrete slab	Beam webs	Bottom flanges	Total
Torsionless	0.019	0.325	0.948	1.29
Horizontally braced	0.117	0.537	1.319	1.97

How the torsion is distributed among the different mechanisms is not straightforward to calculating as the Saint Venant stiffness varies linearly with length while warping varies with $L^{3.}$ Boundary conditions do also influence how torsion is resisted by the deck. It should be noticed that a torsional restraint for the Saint Venant behaviour is not necessarily effective for warping and viceversa. The following scalar parameter [4] may be a rough indicator of which is the prevailing mechanisms:

$$K = L\sqrt{\frac{GI_t}{EI_\omega}} \tag{5}$$

For small value, typically below 3, the warping resistance is very important or predominant while it becomes negligible for higher values. For the section under consideration used in a 48m simply supported deck (L=24m) we get K=1.1 for the torsionless section and 2.9 for horizontally braced one.

4. The deck behaviour: FE simulations

Analyses have been carried out for a single, two and four spans girders using the FE program Algor of Algor Inc. Span length is 48m with 8 equal fields of 6m between transverse diaphragms. The analyses focus on torsion alone and therefore dead and superimposed loads have been neglected. The only applied loading is a concentrated torque of 9.2kNm obtained by two 2kN forces applied at midspan on the beam webs. Linear elastic behaviour is assumed for all the deck components including the concrete slab.

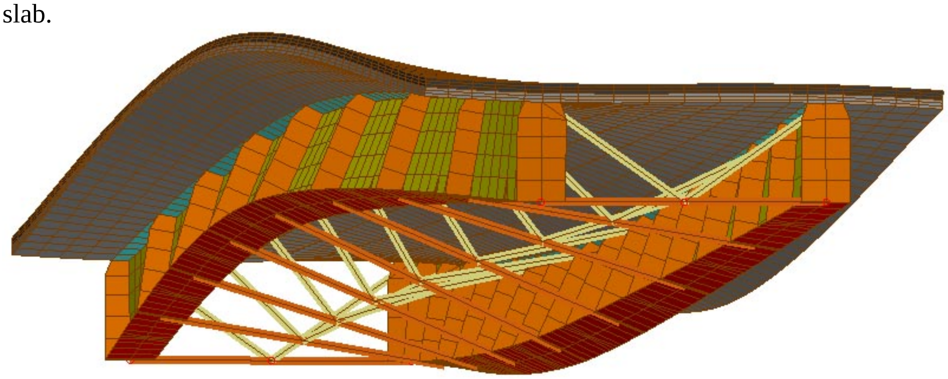

Figure 5: Deformed shapes for the single span case. Torsionless and horiz. braced

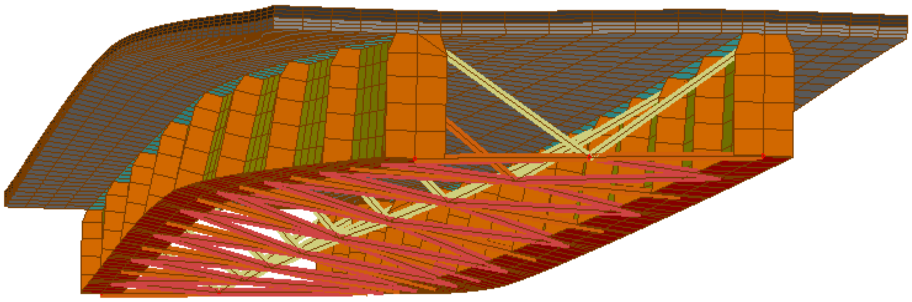

Figure 6: Deformed shapes for the single span case. Torsionless and horiz. braced

The models have the longitudinal axis (deck axis) parallel to the global X axis and the Z axis vertical. Boundary conditions have been fixed so as to preserve symmetry as much as possible. Only vertical support is provided at the bearings with lateral (Y direction) restraint being applied in the middle of the bottom cord of the transverse bracing. Longitudinal restraint is applied at midspan in the middle of the concrete slab.

The concrete slab has been modelled with 8 nodes linear brick elements with 3 DOF at each node. A fine mesh has been used with 6 elements along the slab thickness and maximum length of 0.75m. Steel beams have been modelled using 4 nodes linear plate/shell elements with 5 DOF at each node. Truss elements have been used for all the bracings in the vertical and horizontal plane.

All models have been subjected to the concentrated torque specified above applied at midspan. For the single span case a single torque has been applied, for the 2 and 4 span models, two torques on the adjacent (central) spans have been applied both in the same and opposite direction. Since the warping resistance is magnified with torque applied in the same direction, the results shown in the following shall refer to this case only. The rotation at midspan for the 6 cases (3 meshes, torsionless and horizontally braced) are summarised in the following table.

The increase of warping stiffness with varying static scheme is significant even for the horizontally braced model. This increase of warping stiffness is due to the warping restraint provided by the adjacent spans. A measure of this restrain can be obtained calculating the equivalent warping length $L_{\omega,}$ as follows. For a concentrated torque applied at midspan of a single simply supported span, the warping torsional stiffness is proportional to $3EI_{\omega}/L_{\omega}^{3}$ and the Saint Venant one to GI_t/L_t, with L_t equal to half span (24m). The result is confirmed by the FE analysis as shown in the table below. For multiple spans, L_{ω} was then calculated as the value that provides the matching stiffness ratio to the finite element results. This value is reported in the last column and varies between half span, for the simply supported scheme, to almost ¼ span for the 4 spans case. The increase of equivalent torsional stiffness, when varying the static scheme, is more then 150% for the torsionless design and 40% for the horizontally braced deck. The stiffness ratio between the torsionless and horizontally braced model therefore decrease from 4.3 to 1.9 only confirming how insignificant the De Saint Venant torsional stiffness is for small span continuous girders.

	Torsionless rotation (deg)	**Horiz. braced rotation (deg)**	**Ratio**	**Eq. warping length (m)**
Single span	0.0274	0.0082	4.31	23.8
Two spans	0.0170	0.0071	2.39	16.1
Four spans	0.0112	0.0059	1.90	12.2

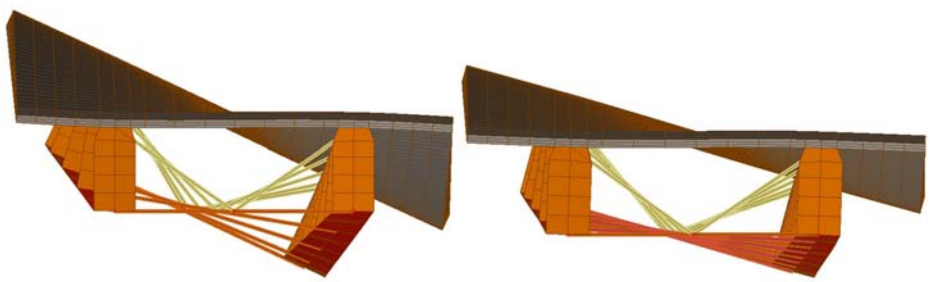

Figure 7: Single span case front view (model sectioned at mid span)

4.1 Simplified grillage analysis

The same case studies have been analysed using simple grillages with two main beams and transverse beams at 6m centre, simulating the transverse bracing and the concrete slab. The analyses have been performed using grillages made of linear elastic beam elements and the same FE code Algor.

The deck torsional inertia of the torsionally braced deck has been assigned 50% each to the two main beams. The concrete slab torsional inertia, found with the formula for thin rectangles ($I_t = ls^3/3$), has been assigned, half each, to the longitudinal and transverse beams. Flexural inertia for the main beams has been calculated with a participating concrete slab width of 4.2m (half slab).

	Torsionless rotation (deg)	**Ratio grillage/3D**	**Horiz. braced rotation (DEG)**	**Ratio grillage/3D**
Single span	0.0302	1.102	0.0089	1.085
Two spans	0.0178	1.047	0.0073	1.028
Four spans	0.0107	0.955	0.0061	1.034

Although these models can account for differential bending (warping) in the vertical plane only but not in the horizontal one as the grillage lays on a single plane, the results summarised in the table above show a good agreement with the 3D model thus confirming the validity of grillage analysis. It should be noticed that the grillage analysis is sensitive to the flexural inertia of the beams which in turns depends on the participating width assumed for the concrete slab.

The results also confirm that neglecting the warping contribution in the horizontal plane cause the grillage model to overestimate rotations. This effect reduces when moving from the single to the 4 span configuration. This is due to the decreasing effect of warping in the horizontal plane because of lack of sufficient shear stiffness of the transverse bracing contrary to warping in the vertical plane where shear is carried by the beam webs.

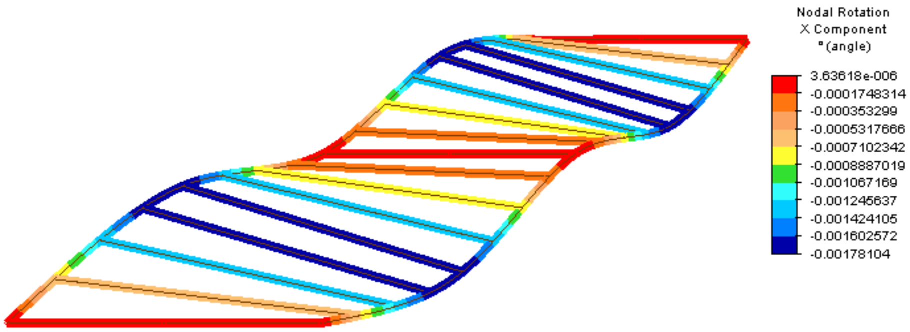

Figure 8: Grillage analysis of the 2 span case.

5. Curved bridges

The behaviour of curved bridges does not differ significantly from that of rectilinear ones (ref. [3]). This also applies to torsionless design since torque is then resisted by section warping similarly to what happen for eccentric live loading in rectilinear ones. In curved bridges, the centre of gravity eccentricity with respect to the axes connecting the supports is:

$$E_{cc} = (L/2)^2 / \pi R \tag{6}$$

Where R is the plan radius and L is the span length. For a typical 35m span and 70m radius (the minimum found along the Turin-Milan interchange viaducts), the eccentricity come out at 1.4m only. This eccentricity is easily carried by differential bending in the beams, as in the above mentioned viaducts where the two beams are 4.6m apart (see Fig. 4). In order to reduce the difference in bending between the two beams, the concrete slab may be shifted inwards thus increasing the amount of load carried by the inner beam.

Photo 2 – One of the Carisio interchange viaducts

With differential bending playing a major role, the girder configuration is obviously very important. Continuous beams will behave much better than simply supported ones where in fact, torsional stiffness may be required to adequately transfer the load between the beams thus increasing the structural efficiency.

In torsionless design though, transverse beams do play a fundamental role in deflecting bending along the curved main beams, a task carried out otherwise by horizontal bracing. In curved bridges therefore, forces in transverse beams are much higher and persistent thus requiring a tougher configuration for the connections between these elements and the main beams and adequate detailing against fatigue. Fatigue can, in fact, affect the welding between the vertical stiffeners and the top flange of the main beams [5].

6. Conclusion

With the full mechanization of plated girder assembly, these types of beams became very popular and are today widely used for small to medium span bridges. Still different arrangements are used for the transverse and torsional bracings in otherwise very similar structures with respect to spans, carriage width, etc..

To the authors experience, truss type bracing is hardly optimal both in term of stress distribution and construction procedure. Truss type transverse beams become convenient for main beam height above 2.5m circa reducing to 2m for highly curved bridges. Since transverse beams (either plated or truss type), are connected to the vertical stiffeners, the design should focus on these elements and their connections, especially with the top flange where fatigue problem may arise [5].

Photo 3 – Detail of the Carisio interchange viaducts

Torsional bracing is hardly convenient at all except for extreme curvatures or simply supported schemes. Plated girders are made for bending in the vertical plane and their efficiency is better exploited without superimposing a truss type behaviour triggered by torsional bracing. When significant torsional resistance is required, a caisson type section is generally more efficient or conversely a space truss may do the job.

Grillage type of analysis although providing consistent and reliable results in terms of overall bending and displacements, disregards the local effect caused by transverse and torsional bracing. This may be the cause for the persistence, in engineer practice, of transverse and horizontal bracing that are hardly optimal and often over dimensioned and redundant while still requiring significant maintenance because of their fragmented geometry and the recesses they create where dust and moist may trigger corrosion.

7. References

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