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Safety assessment and retrofitting of a historic double deck riveted steel bridge

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Abstract

The problem of managing existing bridges on the national territory is now extremely topical, considering that most of these works have exceeded their design life and that traffic on its roads and rail networks has increased significantly over the years, both in terms of flow and load intensity. The case study, it is a double deck riveted steel bridge, the lower one dedicated to railroad traffic and higher one to road traffic, dating back to two different eras since the masonry substructures were built towards the end of the 19th century, while the steel truss was replaced after the Second World War. It consists of a single steel truss structure with three spans, two of which have lateral span of 82.40 m and central one of 99.00 m, for a total length of 263.80 m. The study aims to evaluate and ensure the safety of the work in accordance with current code (NTC2018-RFI Manual) by introducing reinforcement systems on the steel truss bridge that increase its strength while preserving its original architectural appearance. The static and dynamic structural analyses were carried out by defining a finite element model (FEM), within which different load conditions and positions between the railway and road deck were implemented, assessing their contemporaneity, in order to maximize the stresses. The reliability of the implemented numerical model was demonstrated by a calibration process, following specific load tests, with displacements reasonably comparable with the experimental results. The safety level of both the existing conditions and the project has been defined through the implementation of VBA scripts and automated spreadsheets, made on an ad hoc basis, which allow the development of all the checks on the structural elements of non-standard sections, i. e. composite riveted sections. In assessing the safety index, it was taken into account the state of conservation of the structure and of the transitional stages in the implementation of the reinforcement systems.

Keywords: modelling, analysis, bridges, structures, steel, reinforcements, degradation.

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1. Introduction

This case study concerns a metal bridge which, since it connects the Lombardy and Piedmont regions, is of both historical and strategic importance, and aims to assess its safety. Its safety was assessed by means of analysis and verification of the structure in its current state, which enabled us to define which structural elements required reinforcement, followed, once the scope of the project had been defined, assessment of the final state of the structure. The safety assessment covered the superstructure, supports and substructures. However, in the following, the case study focuses only on the metal truss structure.

2. General description of the project

It is a double deck riveted steel bridge, the lower one dedicated to railroad traffic and the upper one to road traffic. It consists of a single spatial steel truss structure (Warren type) covering three spans (two 82.40 m lateral spans and a 99.00 m central span) for a total length of 263.80 m and height of 14.3 m.

Fig. 1. Lateral and internal alignment of the bridge in its current condition

The two flat lattice girders are spaced by 9.16 m and connected by cross beams spaced approximately 10 m apart, forming suitably braced sections. All metal elements constructed in plates and angle profile members, fastened with studs, which is the profile connection technology. The external restraint scheme consists of two fixed supports on the Lombardy-side pile and longitudinally movable supports on all other substructures (all transverse displacements are blocked). The masonry piers and abutments are founded on pneumatic caissons.

3. Historical-critical analysis

The metal truss structure was built in the years 1950-51 to replace the previous one, which had been destroyed during World War II, while the piers and abutments dating back to the year 1881 remained intact, and were used to support the existing truss structure.

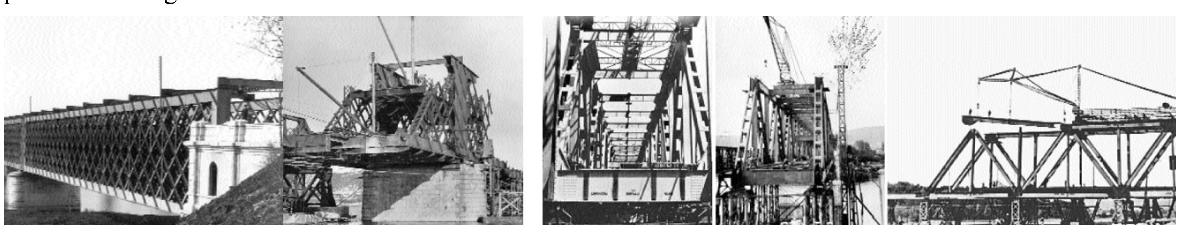


Fig. 2. (a) Photo left: First iron bridge and demolition 1881 – (b) Photo right: Second metal truss structure, construction 1950

4. Surveys and level of knowledge

The mechanical characterisation of the materials constituting the bridge, combined with the geometric survey and detailed analysis of the construction, has led to an KL3 level of knowledge of the metal structures, corresponding to a confidence factor of CF_{KL3}=1,00. Testing of samples enabled us to assign a conservative strength class of S275, while the studs were found to be in class CL6,8. Higher grade steel was used in the project design.

5. Project assumptions

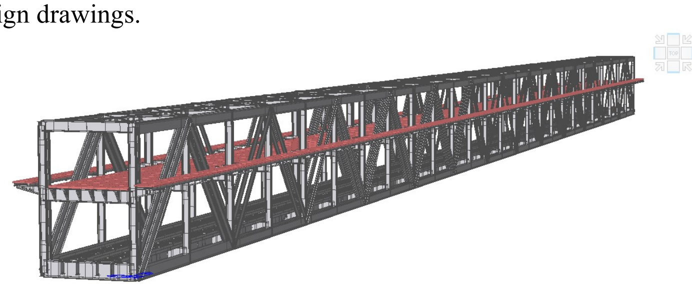
Since the bridge is a heritage structure, the study set itself the objective of assessing and guaranteeing the safety of the structure by reinforcing the metal truss structure to increase its strength while maintaining its original architectural aspect. With regard to the calculations in the analyses, as requested by the Client, the conventional lanes and road loads (tandem and distributed) indicated in Ministerial Decree 2018 for new projects were applied to the road deck, while the railway deck was assumed to be subject to the load conditions characteristic of the actual traffic category, i.e. category D4, in accordance with the RFI Design Manual. The design of the reinforcements accounted not only for the results of the pre-project analysis, but also the condition of the structural elements and the increased loads due to the worksite structures required for installation of the reinforcements themselves. These aspects were accounted for by setting a limit of satisfaction of the pre-project verifications lower than one, so if the index $IR=E_d/R_d$ is higher than the set limit, action was taken to design the reinforcement. As a general rule, a differential safety margin between 5% and 15% was applied to the rail, road, roof and main truss elements. On conclusion of the project, the IR index≤0.95, barring singular cases.

6. Structural analyses

Static and dynamic analyses were run with finite element analysis (FEA) software. The static analysis took into account the effects due to permanent, variable and traffic loads, considering the differing conditions and positions of loading on the rail and road decks, as well as their simultaneity, to maximise the stresses on all structural elements. The linear dynamic analysis was run with an elastic response spectrum (q=1), using the Rayleigh-Ritz method of searching for vibrational modes. The complete quadratic combination (CQC) was used as the modal combination, as required by M.D. 2018 §7.3.3.1. The seismic masses associated with variable traffic loads were reduced by a scaling factor of 0.2. Further analyses were included in the design process, in order to account for additional loads incurred during the execution of the project (scaffolding, equipment, wind on bridges, etc.).

7. Structural modelling

The structure of the bridge was modelled using Midas Civil software (ver.1.2 | 2022) from an ideal version of the real model, using geometric dimensions and sectional characteristics reflecting those given in the original design and survey drawings. The metal elements were modelled with beam-type elements (according to Timoshenko's theory) and truss-type elements where their function is to transfer axial forces alone. The reinforced concrete slabs of the road surface are modelled for the sole function of distributing vertical loads on the stringers and offer no axial or flexural stiffness, as they are modelled as "plate" elements having zero stiffness. The fact that the slab does not act together with the stringers is permissible as there is no connection system between the slab and the stringers in the original design drawings.



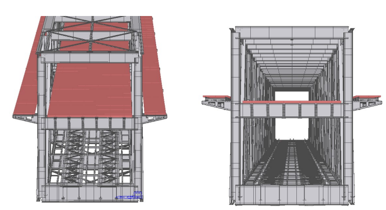


Fig. 3. (a) 3D view of the FEM model; (b) Internal view of the FEM model

The particularity of the metal truss is that it is characterised by sections composed of metal plates and angle members connected by studs. The individual elements consist of a base section and additional reinforcing plates, where necessary, thus defining a variable section along its length. Therefore, to analyse the actual stiffness of the structure, all sections were entered as generic sections using an internal software tool and, in view of the symmetrical structure of the bridge, mirrored with respect to the X, Y and Z axes:

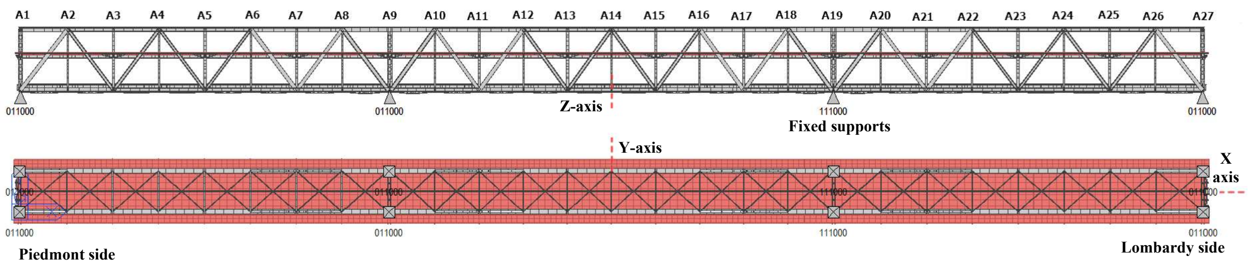


Fig. 4. Elevation and plan view with identification of the axes of symmetry and the constraint system

In order to reproduce the support system of the bridge (as illustrated above), two fixed supports were inserted on the A19 Lombardy side pier (which is therefore the "fixed" pier onto which the braking actions are discharged) and six unidirectional longitudinal supports on all the other substructures, with the transverse displacements therefore remaining blocked. In order to simulate the dimensions/offsets of the structural elements and to model the compound beams, internal constraints of a rigid type have been inserted, appropriately called "Rigid Links". Where the real connection between the elements is hinged, the nodes belonging to such elements have been implemented as being unable to transfer moment (hinge-hinge element). We give some details of the modelling:

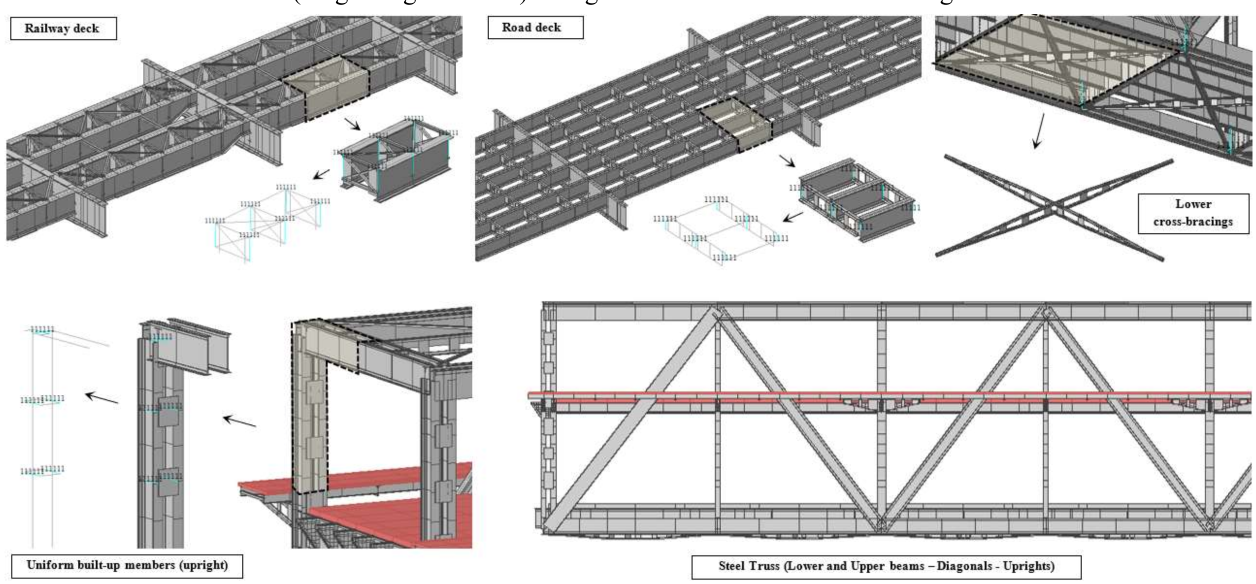
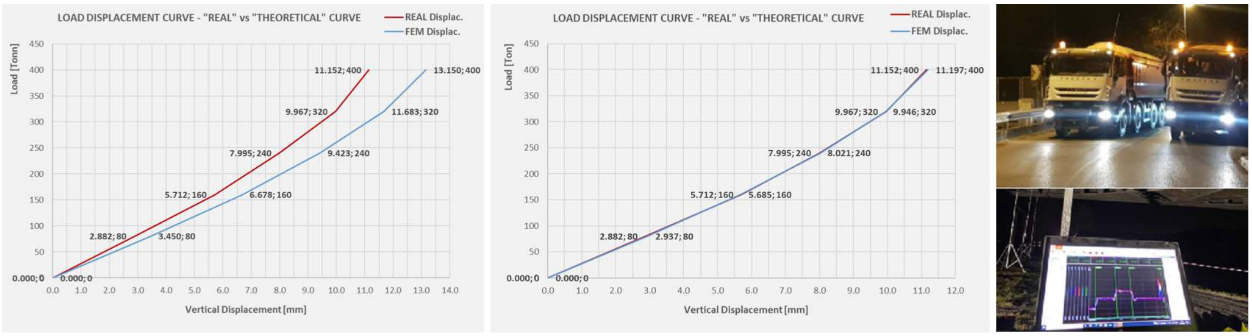


Fig. 5. Modelling details

All structural elements were assigned material properties derived from the structural survey. The truss structure consisting of beams, diagonals and uprights, unlike the other metal elements, were assigned an increased modulus of elasticity following calibration of the model with on-site static load tests.

8. Calibration of the model

The calibration employed the results obtained from the measurement of the downwards deflection of the centreline of the first Lombardy-side span during a static load test. The test was carried out with 10 work vehicles loaded to approximately 40 tonnes each, and comprised 10 steps of loading and unloading, during which a period of time was allowed to stabilise the instruments used to read the downwards deflection. The displacements were measured using 7 displacement transducers: 2 on the abutment, 2 on the piling and 3 on the centre section on the Lombardy side. Given the deviation between the actual and theoretical curves, the model was calibrated by increasing the elastic modulus of the truss structures from the assumed value of 210,000 MPa to 247,800 MPa, thus aligning the two curves:



"Real curve" vs "Theoretical curve (E=210,000 MPa)" "Real curve" vs "Theoretical curve (E=247,800 MPa)"

Fig. 6. Measured results and calibration of the model

9. Conditions and combinations of loading

The actions were evaluated pursuant to Ministerial Decree 17/01/2018, §5.2 and §5.3, as well as the RFI Design Manual. Since the railway line was known, in agreement with the RFI client, a real type D4 train (§2.11.3.1 of the Manual) with a maximum speed of 100km/h was considered. The wind pressure was calculated according to CNR-DT 207/2008 in the "unloaded bridge" and "loaded bridge" conditions.

Table 1. Acting loads

Load cases	*Description*	*Id*
Permanent structural loads	Own weight of steelwork profiles and reinforced concrete slabs	g_{1k}
Permanent non-structural loads	Permanent load on the road deck, pedestrian deck, railway deck and other loads	g_{2k}
Support displacement	Downward deflection of pilings and abutments	ε_{4k}
Variable loads	Contingent on the railway platform, temperature, aerodynamic effects of trains, wind	q_{ki}
Variable road traffic loads	Load diagram 1 and 5 with eccentric arrangement and centred load, braking action	$q_{k,S}$
Variable rail traffic loads	Train with real D4 load, braking, starting and yawing loads	$q_{k,T}$
Seismic action	Seismic loading in Ultimate Limit State (ULS)	E_k

All loads were entered into the numerical model, considering the possible positions and load conditions that maximise their stresses. For this purpose, traffic loads were entered as "Moving Loads" by examining all positions between rail and road. The seismic load was evaluated starting from the definition of elastic spectra derived from the Local Seismic Response (LSR), with reference to the Ultimate Limit State (ULS) pursuant to current standards, noting that the project is catalogued in Use Class III. All loads were combined appropriately pursuant to the above-mentioned standards, obtaining the most severe effects from the results. In particular, given the volume of traffic, it was reasonably assumed that the two types of traffic should be considered simultaneous primary loads. Finally, careful analysis of the permanent loads shows a total weight of approximately 5,790 tonf, of which the metal structure accounts for 2,906 tonf, the reinforced concrete slabs 1,674 tonf and the supported permanent loads 1,210 tonf.

10. Results and validation of the model

The results of the calculations, pursuant to §10.2 of Ministerial Decree 17/01/2018, were checked by comparing them with the results of simple calculations performed using traditional methods. A maximum divergence of less than 4 per cent was obtained from the consideration of all loading cases, which is considered an acceptable value.

From the modal analysis, the first modes of vibration with the largest participating mass result in deflections in the longitudinal and transverse directions with periods of about 1 s, while the first mode in the vertical direction has a period of about 0.3 s. Given the magnitude of seismic actions for long periods, the largest stresses are obtained for combinations of loading under static conditions.

11. Structural verifications

The verification criteria comply with current Italian regulations (M.D. 17/01/2018 and Explanatory Circular 21/01/2019) and the RFI Design Manual. The verifications concerned the structural elements constituting the metal truss structure, as well as the compound profile members and studded connections.

Compound profile members

The profiles were tested for strength and stability as follows:

- in view of the structural particularity of the profiles, assembled from metal plates and angle members assembled with studs, in order to ensure consistency between the verifications and the actual structural behaviour, the sections were classified, at most, as Class 3 (any sections with geometries that could fall into Classes 1 or 2 were accordingly downgraded to Class 3, in order to avoid using the plastic properties of the material);
- in view of the above, it follows that all verifications were run in the elastic range of the material;
- assuming that the peak stresses at the nodes are counteracted by the reinforcing plates within the nodes themselves, the stresses on the elements defined by the free length net of the connections were verified, resulting in a partial reduction of the acting stresses;
- the strength verifications were run pursuant to §4.2.4.1.2 of M.D. 2018 by calculating the elastic mechanical properties referred to the net sections, i.e. those clear of holes as given in the original drawings;
- the stability verifications were run pursuant to §4.2.4.1.3 of M.D. 2018 by calculating the elastic mechanical properties referred to the gross section and the effective mechanical properties of the material, if in Class 4;
  in the stability verifications, the actual free length of deflection was considered by applying appropriate coefficients, depending on the degree of constraint at the ends of the members, in compliance with normal building science methods;
- the press-flexion verification was run pursuant to Method B proposed in Circular 21/01/2019 §C4.2.4.1.3.3.2, while the stability verification of the panels subject to shear was run with reference to §C4.2.4.1.3.4.1 of the same circular.

All elements were verified for each combination and for each section identified along the length of the element, calculating the verification index IR as the maximum of the index given by the IR_{RES} strength verifications and that derived from the IR_{STAB} stability verifications, as follows:

$$IR=E_d/R_d=\max\,(IR_{RES};\ IR_{STAB}).$$

The elements were considered to have been successfully verified if the IR index was below a limit of less than unity, in order to guarantee an adequate margin of safety given the importance of the structural elements and the level of defects inherent in them, in accordance with the project assumptions described in §5.

Given the particularity of the profiles characteristic of the structure and the impossibility of performing verifications on the Midas Civil post-processing platform, since the sections were entered as generic, special spreadsheets in Microsoft Excel were created for the verifications. Given the amount of data to be processed, specific VBA codes were implemented on a case by case basis in the spreadsheets to automatically run all the verifications for each element, section and combination under consideration, and return the results with the maximum IR index for the element in question (cross beam, stringer, etc.). The resulting spreadsheets enable calculation of the mechanical properties of the gross and net sections and, in the case of press-flexion, determine the class transition domain in relation to their stresses and geometry, assigning the correct class to the section and, if in Class 4, determining the effective dimensions of the panels. In the case of more complex sections, such as beams, if the section is in Class 3, the mechanical properties were evaluated using the "IDEA Statica Steel Connections" program, while if in Class 4, the verifications were run with the "PRO_CL4" and "SAITU_CargeoPlus" programs, supplied by "2S.I. Software e Servizi per l'Ingegneria S.r.l." and "S.T.A. Data S.r.l." respectively.

Studded connections

The verification of the studded connections is referred to the connection between the structural elements pursuant to M.D. 2018 §4.2.8.1, by calculating the verification index IR as the maximum value of all verifications, such as stud shear and tension, re-drilling of holes, punching and buckling of plates. These verifications were run by creating appropriately validated spreadsheets. The design stresses were obtained by associating each minimum and maximum shear stress (Vz and Vy), axial stress (N) and bending moment (My and Mz) with the corresponding stresses.

12. Verification Groups

All structural elements were grouped by type, and different variable verification sections were identified for each in both the longitudinal and transverse directions.

Table 2. Verification groups

STRUCTURAL ELEMENTS	ID	GROUP	SECTION
Lower beams	LB	5	5
Upper beams	UB	5	6
Diagonals	D	8	8
Uprights	U	4	6
Railway deck cross beams	RACB	3	5
Road deck cross beams	ROCB	2	8
Covering cross beams	CCB	2	2
Railway deck stringers	RAS	2	3
Road deck stringers	ROS	2	2
Covering stringers	CS	1	1
Lower cross-bracings	LCB	3	3
Upper cross-bracings	UCB	2	2

STUDDED CONNECTIONS	ID
Lower beams + connection plate	LB+CP
Upper beams + connection plate	UB+CP
Diagonal + connection plate	D+CP
Upright + connection plate	U+CP
Railway deck cross beam + upright	RACB+U
Railway deck cross beam + Railway deck stringer	RACB+RAS
Road deck cross beam + Road deck stringer	ROCB+ROS
Lower cross-bracing + plate on lower beam	LCB+PLB
Upper cross-bracing + Plate on upper beam	UCB+PUB
Angle member of Covering stringers + compound beams	LC+CB

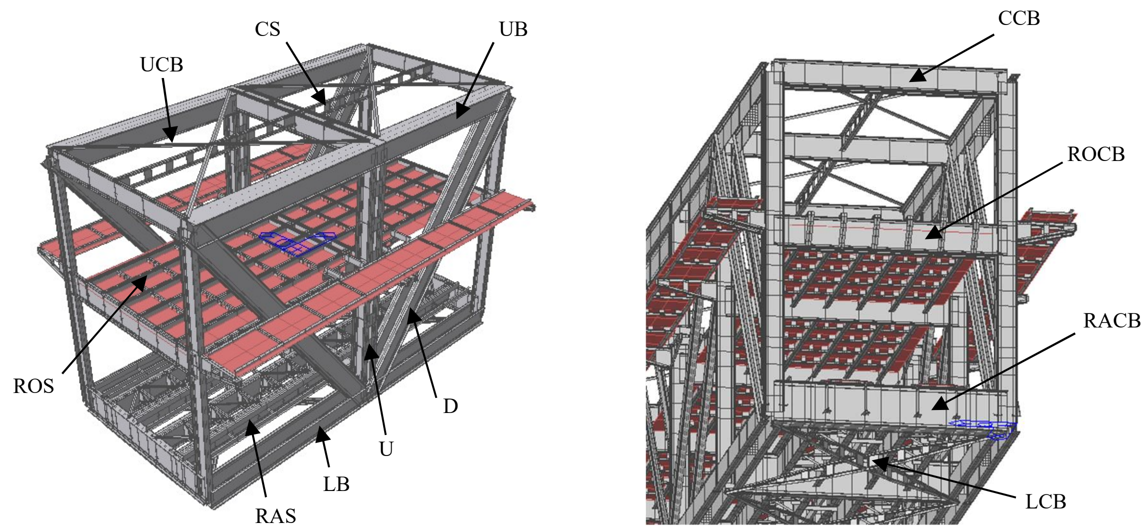


Fig. 7. Overview of model showing verification groups

13. Structural reinforcement

In the project proposals, the aim is to leave the architectural aspect and structural conception of the work unaltered, since it is subject to landscape, environmental and historical heritage protection pursuant to Legislative Decree 42/04 as amended and other sectorial regulations. Therefore, while respecting the structural configuration of the existing structural elements, made by assembling plates and angle members with studs, the overall approach has been to reinforce the sections of the profile members by inserting new plates or angle members, in order to obtain the increase in strength and stability required to satisfy the verifications. In order to maintain the original architectural appearance, the proposed projects included the use of high-strength (class 10.9) dome head bolts on the outer faces of the structural elements. In addition, in order to minimise the visual impact of the aforementioned bolted assembly, where possible, the bolts are distributed at the same pitch as the bridge’s existing bolts. For the sake of brevity, only one section is given below for each verification group, with its proposed structural reinforcement:

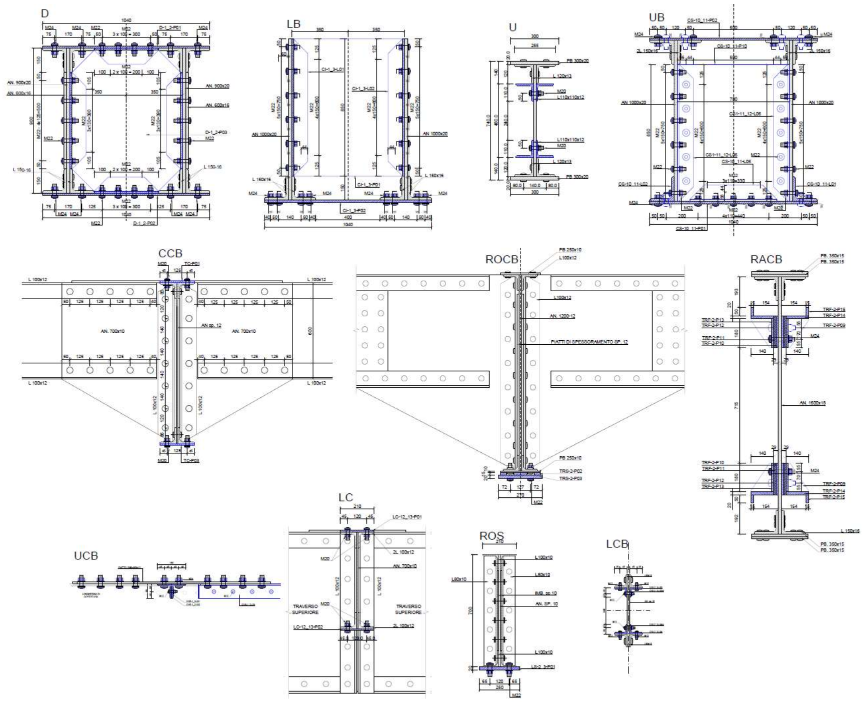


Fig. 8. Structural element profile sections – Example reinforcements (in blue)

14. Results, pre- and post-project

All pre-project verifications were run on all sections of the various structural types and for all studded connections. For the unverified sections, post-project verifications were run following the design of the reinforcements. We give below a table of results for all sections of the various groups with maximum $IR_{max,\ AO}$ and highest percentage of elements present, leaving out singular cases:

STRUCTURAL ELEMENTS	ID	$IR_{max\ AO}$	IR_{PO}
Lower beams	LB	1.196	0.913
Upper beams	UB	2.302	0.828
Diagonals	D	3.906	0.862
Uprights	U	1.076	0.861
Railway deck cross beams	RACB	2.519	0.900
Road deck cross beams	ROCB	1.140	0.936

STRUCTURAL ELEMENTS	ID	$IR_{max\ AO}$	IR_{PO}
Covering cross beams	CCB	0.721	-
Railway deck stringer	RAS	0.742	-
Road deck stringers	ROS	1.110	0.905
Covering stringers	CS	1.061	0.623
Lower cross-bracings	LCB	1.929	0.886
Upper cross-bracings	UCB	1.063	0.831

Note that the higher IR indices are due to buckling of the diagonals and horizontal out-of-plane loading, which were not considered in the design of the bridge at the time.

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