JOURNAL OF TRAFFIC AND TRANSPORTATION ENGINEERING (ENGLISH EDITION) 2015; 2 (4): 209–222

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Original Research Paper

An innovative steel-concrete joint for integral abutment bridges

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ARTICLE INFO

Article history:
Available online 3 June 2015

Keywords:
Integral abutment bridges
Steel-concrete bridges
Steel-concrete joints
FEM simulation
Shear connection method

ABSTRACT

Integral abutment bridges are becoming rather common, due to the durability problems of bearings and expansion joints. At the same time, among short- and medium-span bridges, multi-beam steel-concrete composite deck with hot-rolled girder is an economical and interesting alternative to traditional pre-stressed concrete solutions. The two concepts can be linked together to design integral steel-concrete composite bridges with the benefits of two typologies. The most critical aspect for these bridges is usually the joints between deck and piers or abutments. In this paper, an innovative beam-to-pier joint is proposed and a theoretical and experimental study is introduced and discussed. The analyzed connection is aimed at combining general ease of construction with a highly simplified assembly procedure and a good transmission of hogging and sagging moment at the supports in continuous beams. For this purpose, the traditional shear studs, used at the interface between steel beam and upper concrete slab, are also used at the ends of steel profiles welded horizontally to the end plates. To better understand the behaviour of this kind of joints and the roles played by different components, three large-scale specimens were tested and an FE model was implemented. The theoretical and experimental results confirmed the potential of the proposed connection for practical applications and indicated the way to improve its structural behaviour.

1. Introduction

The durability of bridge expansion joints and bearings has become an important concern worldwide. It leads the integral-abutment bridge (IAB) concept becoming rather popular in recent years, not only in the newly built bridges, but also in the retrofit of existing ones (Briseghella and Zordan, 2006; Dong et al., 2014; Yannotti et al., 2005; Zordan and Briseghella, 2007). The formulation of IAB dates back at least to the 1930s, and it has been used to address long-term structural problems which frequently occur with conventional bridge designs (Burke, 1993). The original IAB concept was not very manageable at that time. It had several problems related to the post-construction life of

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Peer review under responsibility of Periodical Offices of Chang'an University.
http://dx.doi.org/10.1016/j.jtte.2015.05.001

the structure. Besides, the specific design and soil-structure interaction problems represent a challenge requiring synergy of both structural and geotechnical engineers. The application of IAB based on bridge's maximum length was investigated by Zordan et al. (2011a, 2011b). The IAB concept has received considerable interest among bridge engineers because of the enormous benefits from the elimination of expansive joints and the reduction of installation and maintenance costs. The superstructure of integral abutment bridges can be made continuously through a composite cast-in-place concrete deck slab over steel or pre-stressed concrete girders and sequential diaphragms. With this design, the system constituted by the sub- and super-structure acts as a single structural unit. The monolithic connection between deck and sub-structure, such as piers and abutments, makes IABs different from other conventional bridges and allows for a remarkably increased redundancy for seismic loading and other extreme events (Xue et al., 2014; Zordan et al., 2011a, 2011b). Furthermore, joints in deck are known to be a crucial part in existing bridges, not only because of their own failure and maintenance problems, but also of the significant amount of corrosion damage in girders and other underlying substructures caused by corrosive de-icing salts in run-off water leaking through the joints to the deck (Arockiasamy and Sivakumar, 2005). Hence, the IAB concept has proved to be successful in eliminating a number of problems related to the management of conventional bridges during their service life, and turns out to be economical with reference to both construction and maintenance costs (Tandon, 2005). The United States can boast the widest experience of IABs, measured in number of application. Nevertheless, a limited but meaningful number of bridges within this family, with composite steel and concrete structures, can be found in central Europe, mainly in Luxembourg and France (Hever, 2001). As noted by some studies (Zanon et al., 2014), among short- and medium-span bridges, multi-beam steel-concrete composite deck consisting of hot-rolled girders is an economical and interesting alternative to traditional pre-stressed concrete solutions.

With this bridge type, continuity and hogging moment can be balanced at piers through slab reinforcement, as well as by the connection of the steel beam to a concrete transverse diaphragm by means of a number of shear studs, which are mainly devoted to the shear transmission. Studies on this topic started in 1970 in Australia (Kell et al., 1982) at the request of the Department of Main Roads of Western Australia. In the same period, several composite bridges were also built in the United States.

Regarding beam-to-beam connections for composite bridges, an interesting study published by Dunai et al. (1996) presented an experimental study on an original type of end-plate steel-to-concrete mixed connection, focussing mainly on its rotational stiffness and cyclic deterioration. The originality of the connection resided in the layout of the joint, inspired by conventional column-base connections. In this special kind of connection, tensile forces were transmitted by bolts and/or studs, shear forces were transmitted by headed studs (or shear connectors in general) and compression forces were transmitted directly to the concrete by the bearing end plates.

Haensel (1998) presented results of a research program sponsored by steel fabricator ARBED, which built bridges with an innovative type of connection for composite steel and concrete girders, where the bending moment of the supports at the top flange was transmitted across the bearings by the tensile reinforcement alone, while the bending moment in the bottom flange was transmitted by means of a couple of compression plates through the concrete. Contemporarily to Haensel (1998) and Svadbik and Strasky (1998) presented the "Modular Span Multi-Cell Box Girder System" for short span (from 6 to 30 m) steel bridge structures. The bridge deck was composed of steel modules and a composite concrete deck slab. Multi-span continuous structures assembled from simple spans, joined by concrete cast-in-place diaphragms, were studied. The reactions were transferred from the bearings into the webs by the concrete diaphragms. The webs with shear studs and the diaphragms had reinforcing bars, which resisted splitting forces.

In 2002, a design guide on "Composite Bridge Design for Small and Medium Spans" was published by the ECSC Steel RTD Programme (EU, 2003). The publication, among other things, focused on using shear connectors transferring forces to a concrete diaphragm inside single and continuous composite decks, with different levels of prefabrication for the concrete slab.

A further meaningful example of hybrid composite bridges was the VFT construction method proposed by Schmitt Stumpf Fruhauf and Partner (Weizenegger, 2003). A type of composite girder that could be almost completely prefabricated was developed, with the connection to the abutments using shear connectors.

More recently, the nonlinear response of similar composite joint typologies to those studied in this paper was investigated by Zordan and Briseghella (2009), while Zanchettin et al. (2011) experimentally studied five different configurations of shear studs.

Somja et al. (2012) studied the influence of key parameters governing joint behaviour using FE simulation based on experimental tests.

The above studies mostly regarded monotonic loads and hogging moment, while in integral abutment bridges, a full connection between girders and piers and/or abutments with sagging and hogging moment resistance could be required.

This paper presents the results of an experimental and theoretical study on an innovative beam-to-pier joint. The proposed connection is introduced with a start of a previous connection studied by the same authors (Zordan and Briseghella, 2007). An experimental campaign on three full-scale specimens is introduced, and the main results are discussed in Section 3. Global and local FE models used in the research are described in Section 4. Based on the obtained results, an improvement to the connection is proposed and analyzed in Section 5. Finally, the conclusions are drawn.

2. Joint

The increasing ratio of labour cost to construction materials cost is urging the development of construction techniques.

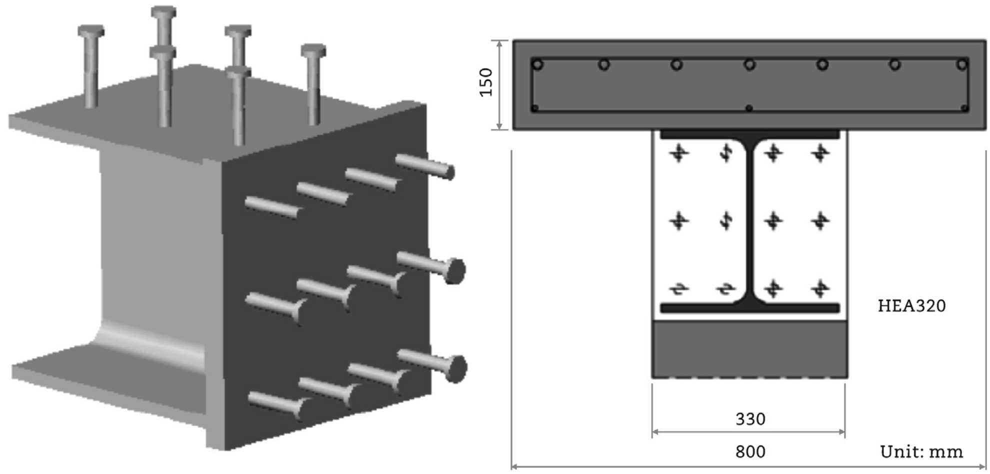

Fig. 1 – Joint studied in previous researches.

The smallest number of skilled workers, and the optimisation of building schedules are required to minimise the overall building yard time.

Composite decks and floors are becoming increasingly widespread for their structural effectiveness, which allows for the optimisation of the mechanical properties of the coupled materials. It also allows for the possibility of achieving a simplified design, with a limited number of structural elements involved in the structural response.

The search for a high global stiffness in the frame response must inevitably consider the achievement of a satisfying joint stiffness in continuous decks. Moment-resisting joint is therefore the key component of the modern conception of composite constructions.

The basis for this work on an innovative typology of composite joints consists of the following requirements:

1. Achievement of the connection between a composite steel and continuous concrete beam (part of a composite deck or floor) or a concrete pier (or column);
2. Minimisation of the joint components;
3. Minimisation of installation time;

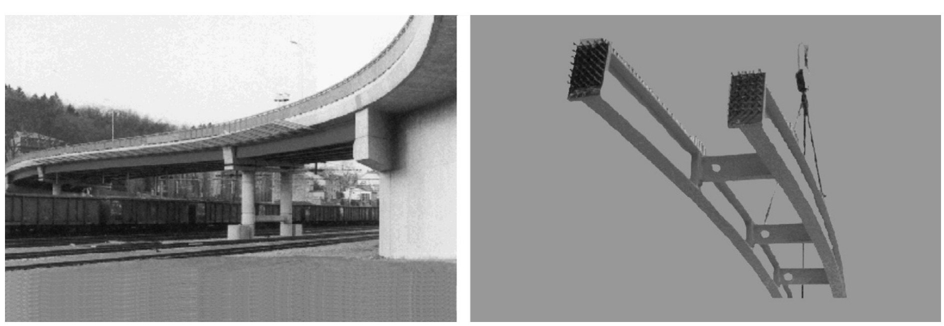

Fig. 2 – Three-span bridge in Differdange (South Luxembourg).

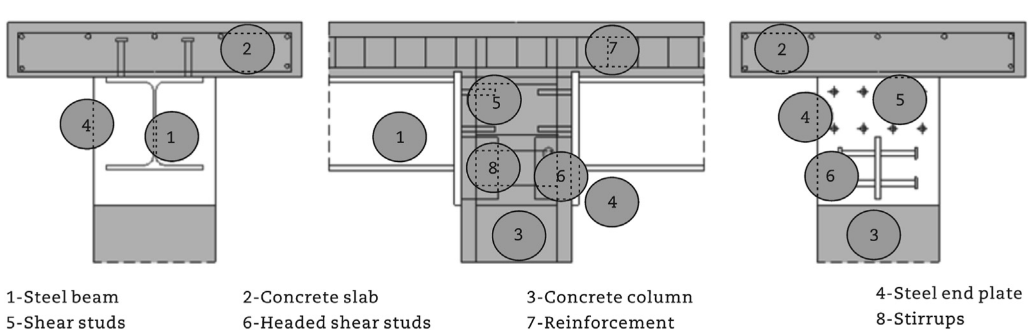

Fig. 3 – Joint studied in this paper.

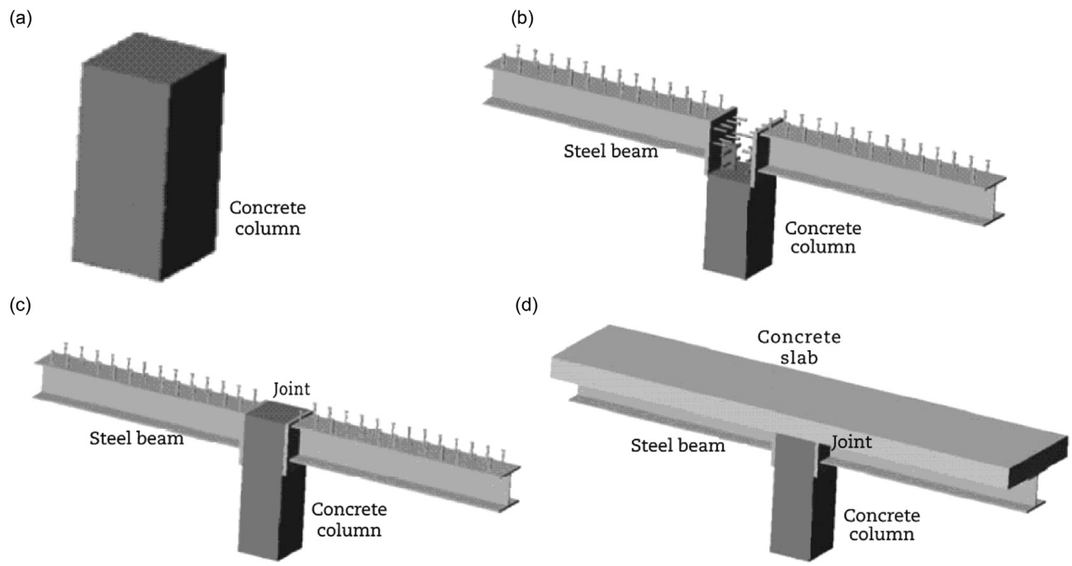

Fig. 4 – Assembly phases. (a) Concrete column. (b) Steel beam. (c) Concrete joint. (d) Concrete slab.

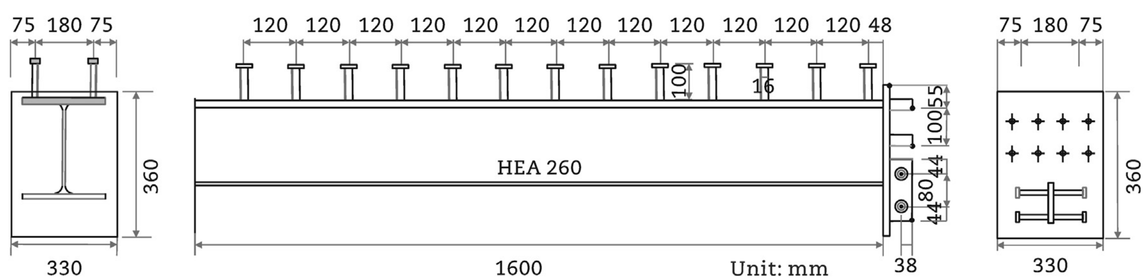

Fig. 5 – Joint steelwork.

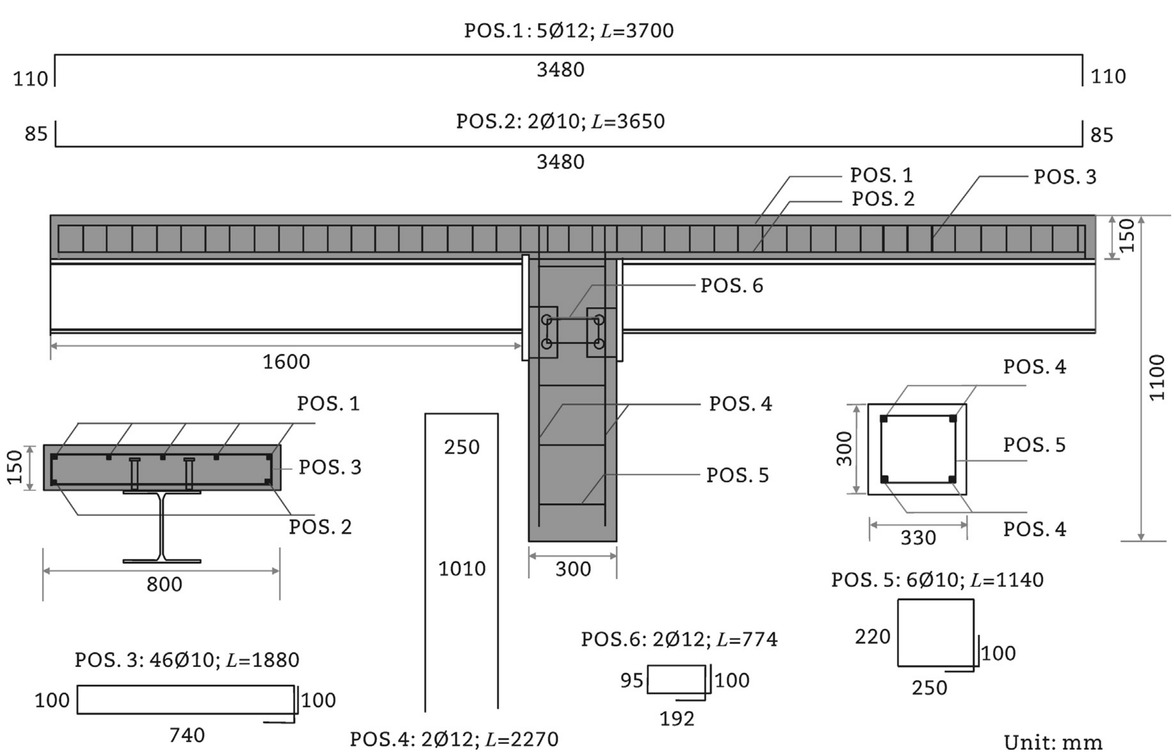

Fig. 6 – Main dimensions of specimens and reinforcements used.

Table 1 – Construction materials used for the specimens.

Component of the joint (Fig. 3)	Concrete	Steel
1–Steel beam		S 355-EN10025 (EC3, Table 3.1)
2–Concrete slab	C 35 (EC2, Table 3.1)	
3–Concrete column	C 35 (EC2, Table 3.1)	
4–Steel end plates		S 355-EN10025 (EC3, Table 3.1)
5–Shear studs		SJ2G3+C450-EN10025 (St 37-3K)
6–Headed shear studs		SJ2G3+C450-EN10025 (St 37-3K)
7–Reinforcement		fyk ≥ 430 MPa (EC2, Table 3.1.5.2)
8–Stirrups		fyk ≥ 430 MPa (EC2, Table 3.1.5.2)

4. Minimisation of tolerance problems due to the connection between steel and concrete;
5. Appropriate stiffness under hogging and sagging moment conditions.

Recently, the increased utilisations of headed studs (for their weldability and limited costs) as connector devices at the interface between steel beams and concrete slabs in composite structures have suggested their possible use at the interface between steel profile and concrete column too. This ensures the transmission of shear forces. The hogging moment is borne in compression by the steel end plate in contact with the concrete pier, and in tension by the steel rebar in the slab. The sagging moment is borne in compression by the concrete slab, and in tension by a number of specifically designed headed studs.

In this way, the same connection system acting between slab and beam is used at the interface between a steel end plate welded to the steel profile and the adjacent concrete column. With this system, the realisation of composite joint is limited by the assembly of an extremely limited number of parts, for the achievement of the composite action in the steel and concrete beam. Furthermore, the elimination of on-site welding could help to minimise the construction time and cost because the connection between horizontal composite members and vertical columns can be achieved simply through concreting.

Finally, the tolerance problems can be limited or totally eliminated because of the final concreting. The proposed system is shown in Fig. 1. The first phase of this research has focused on the hogging moment behaviour of the joint (Zordan and Briseghella, 2009). An experimental test was performed at the laboratory of the University of Trento (Italy) to investigate the joint response under monotonic and symmetric loads until failure. The test results, along with the implementation and calibration of a finite element

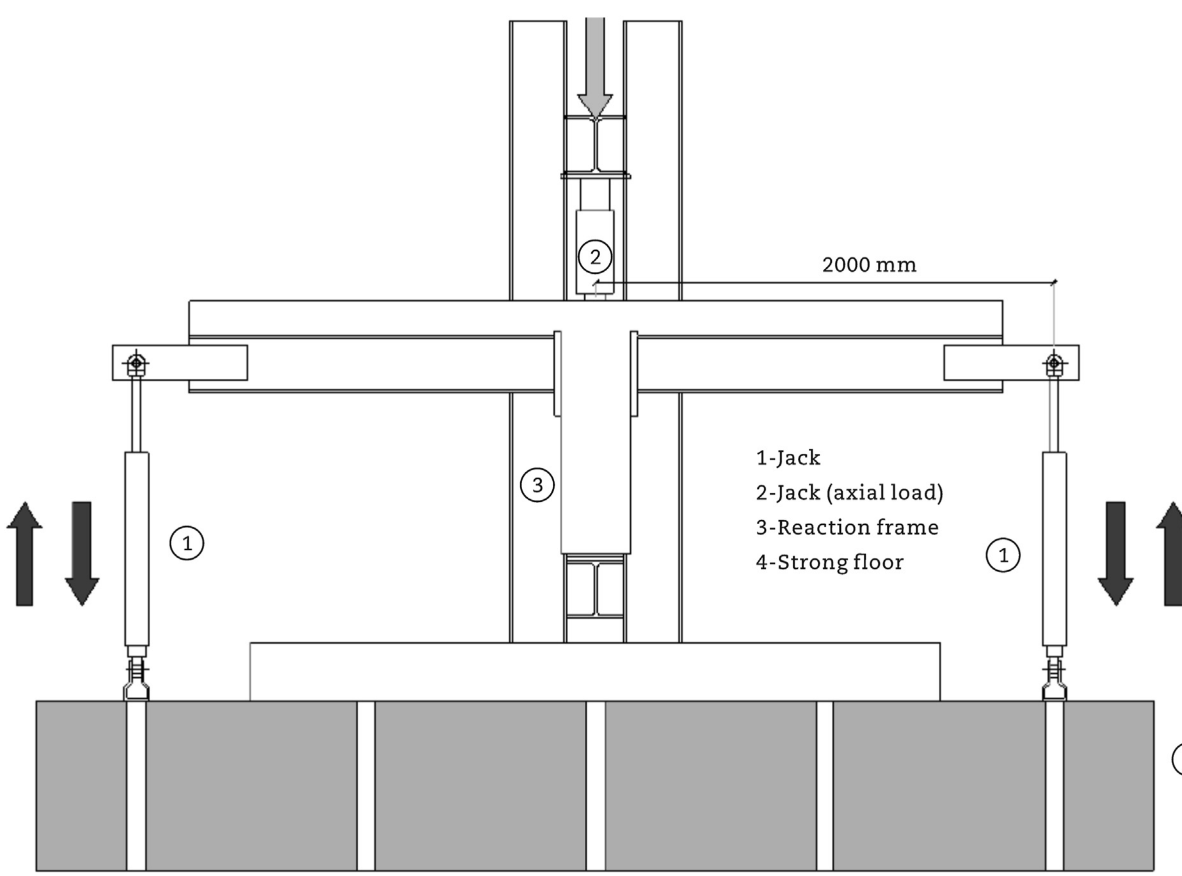

Fig. 7 – Setup scheme.

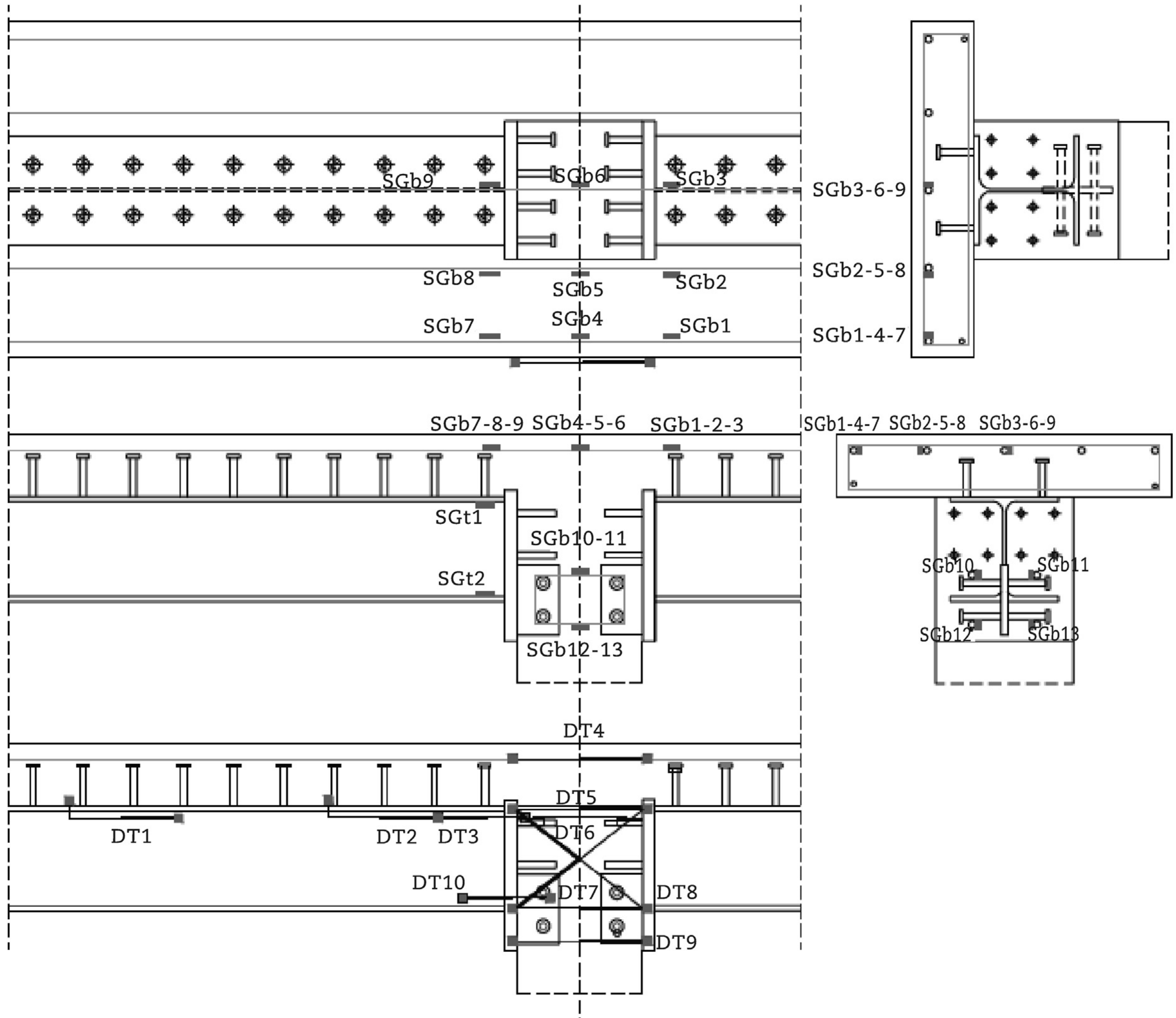

Fig. 8 – Layout of instrumentation.

model, led to an understanding of the behaviour of each element in the final response of joint and the development of a mechanical model according to the component method. Finally, some simplified relations were proposed to use in the global structural analysis.

Similar joints, for their simplicity and relatively easy installation, can potentially be used for bridges, particularly the integral abutment bridges, or for the frames of civil buildings. This kind of connection has recently been applied in real cases, with satisfied results in terms of construction time and cost. As an example, the Differdange town centre bypass road is shown here (EU, 2003). The three span lengths of this heavily skewed and curved bridge are 25, 40 and 25 m, respectively, with a deck width of 12.5 m (Fig. 2).

During the second phase of this research, which is the subject of this paper, the response of connection was investigated under sagging and hogging moment conditions until failure. This is particularly important not only for buildings, but also for integral abutment bridges, where a full connection is frequently realised between the piers and abutments.

Table 2 – Types of instruments.

Abbreviation	Type of instrument	No.
SGb	Strain gauge applied on the rebar	13
SGt	Strain gauge applied on the steel beam	2
DT	Displacement transducer	10

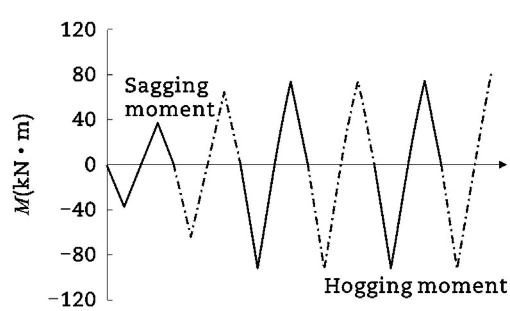

Fig. 9 – Test load history (sagging and hogging moments applied).

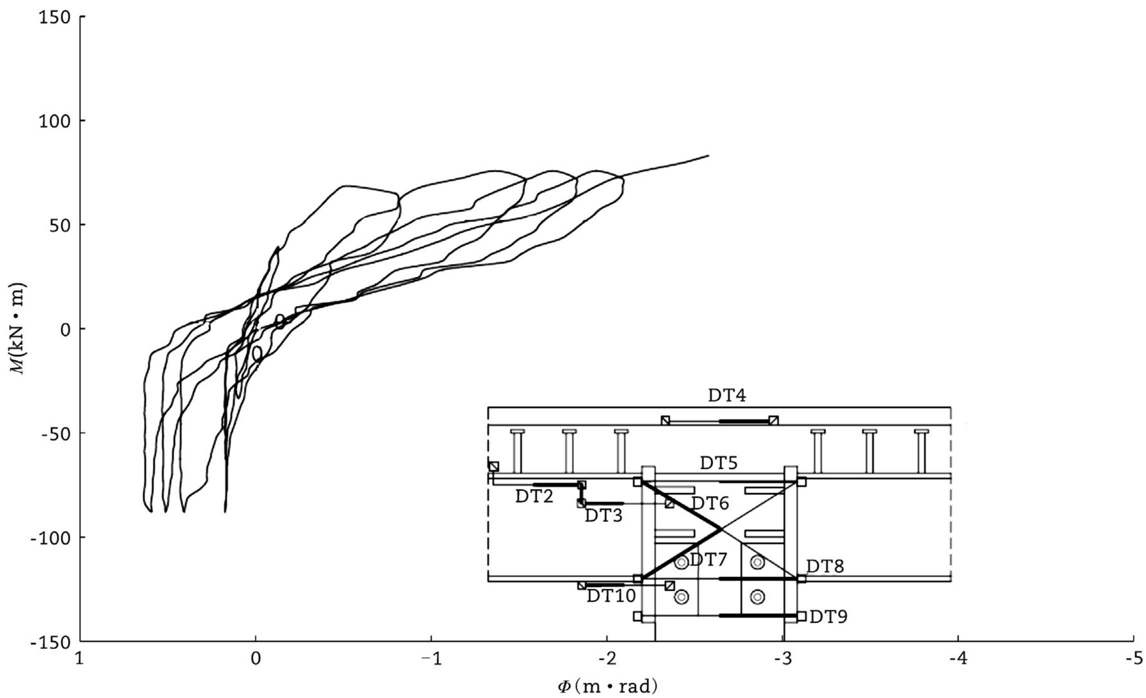

Fig. 10 — M-Φ curve (recorded by DT5-DT9) for specimen S3.

In order to obtain a better response under this type of load, the joint was modified (Fig. 3), adding some horizontal shear studs welded to the end plates. Fig. 4 illustrates the assembly phases of the considered joints, showing the easy and fast procedure.

This research was organised into the following three interconnected steps:

1. Experimental study on full-scale specimens of joints, whose layouts represent the natural evolution of the case study for monotonic hogging moment;
2. Implementation and calibration of a finite element model of the tested joint;
3. Creation and calibration of a simplified mechanical model of the tested joint.

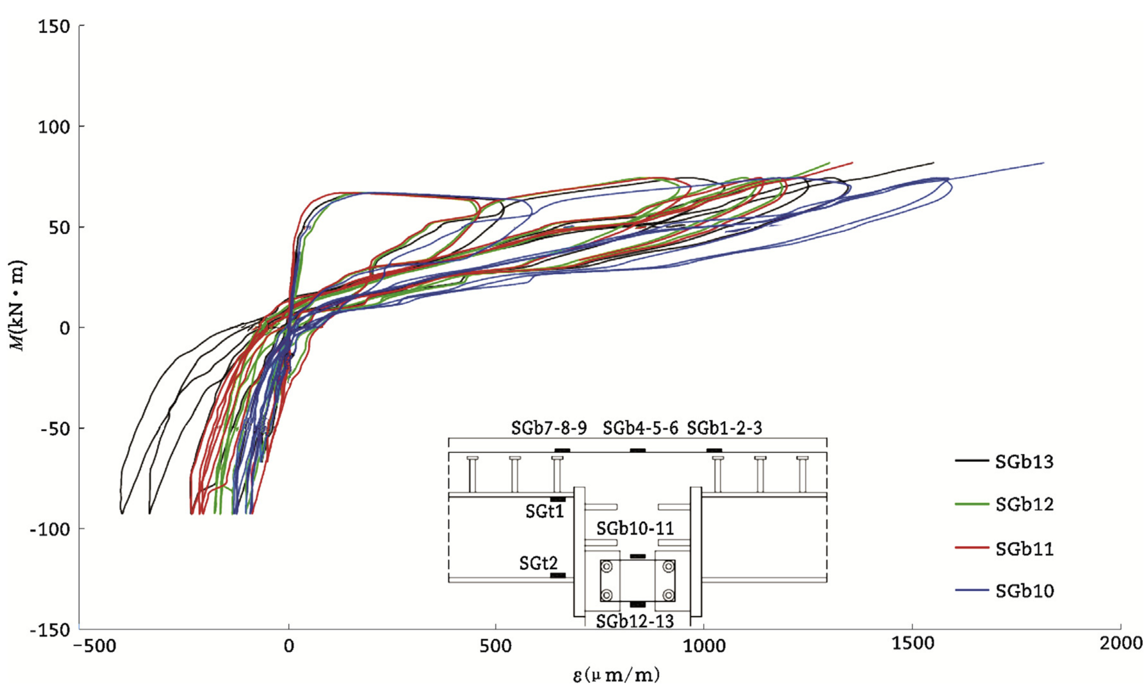

Fig. 11 — Strain level recorded by rebar strain gauges SGb10-11-12-13.

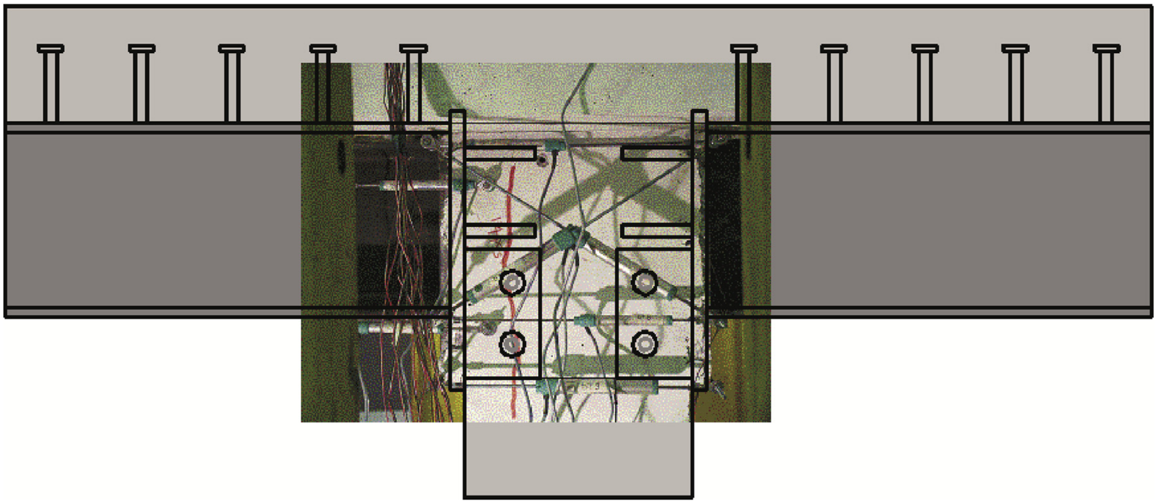

Fig. 12 – Vertical main cracks in the specimens.

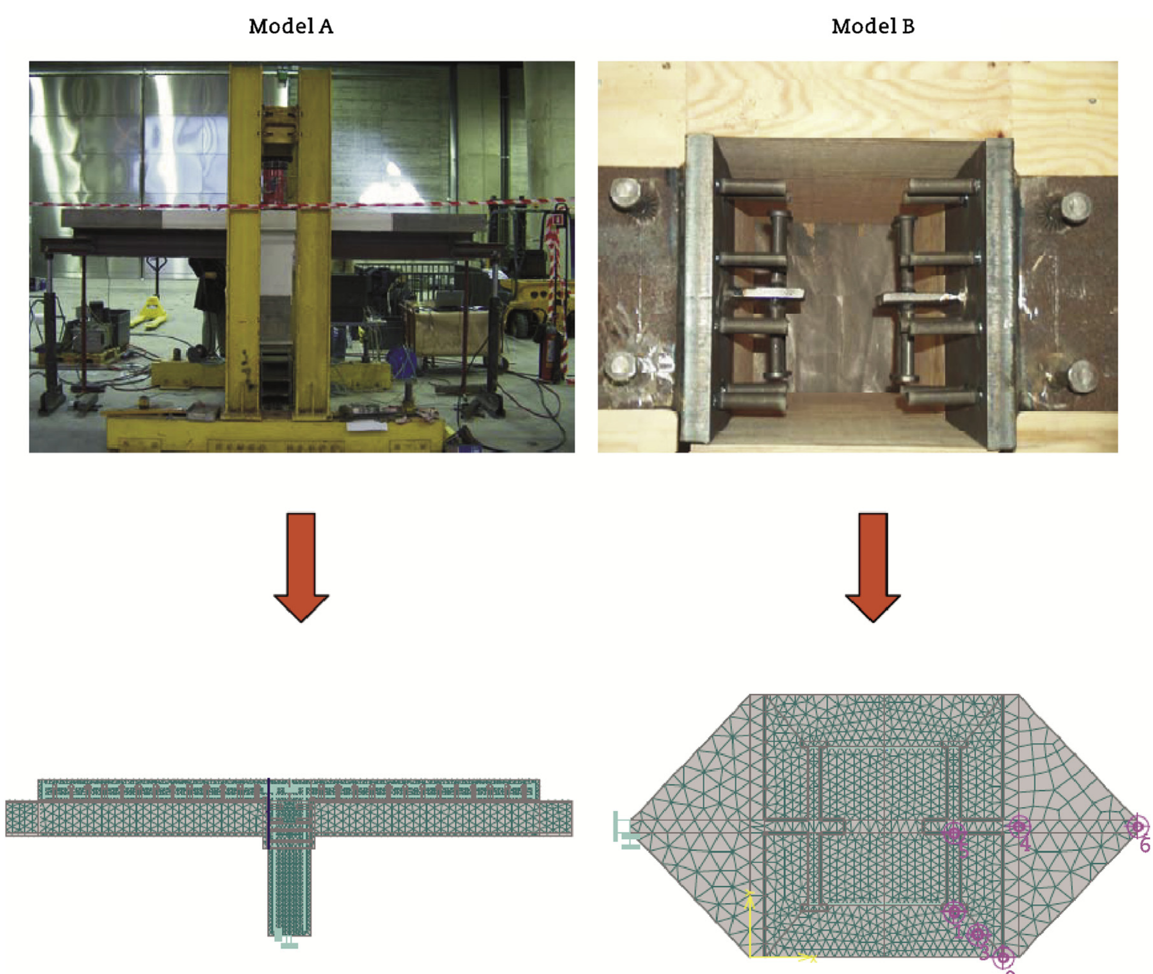

Fig. 13 – Two finite element models of the specimens.

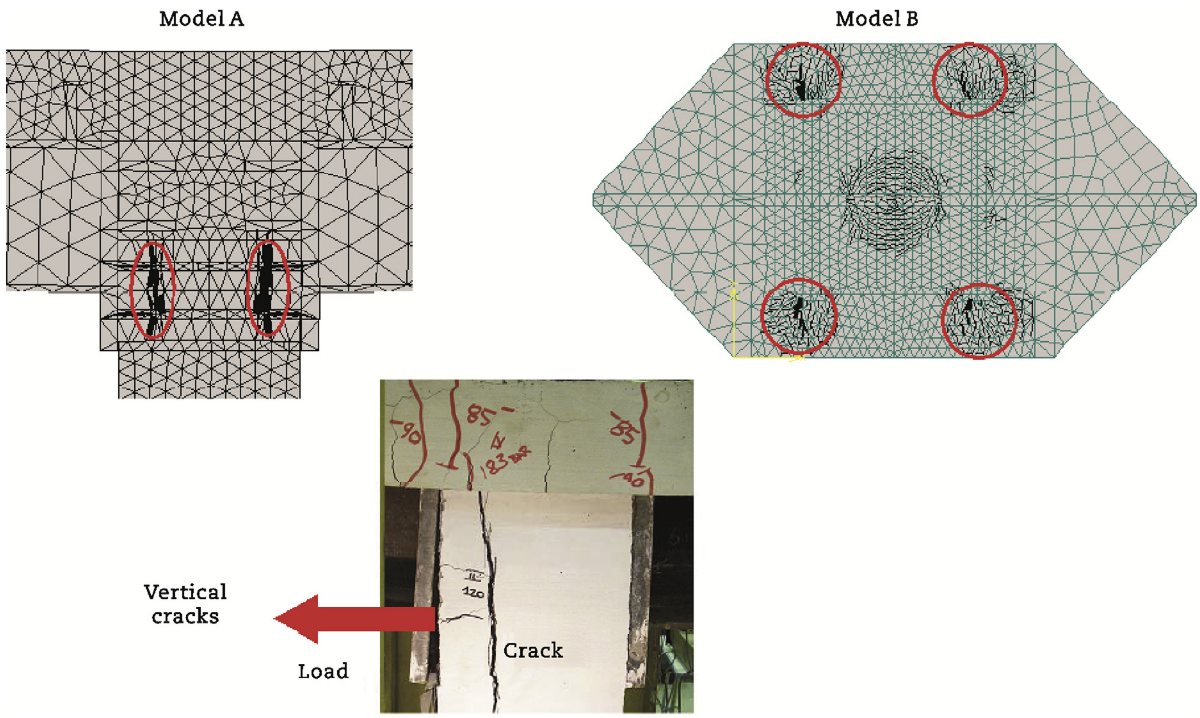

Fig. 14 – Propagation of main cracks in the specimens (M = 74.40 kN·m).

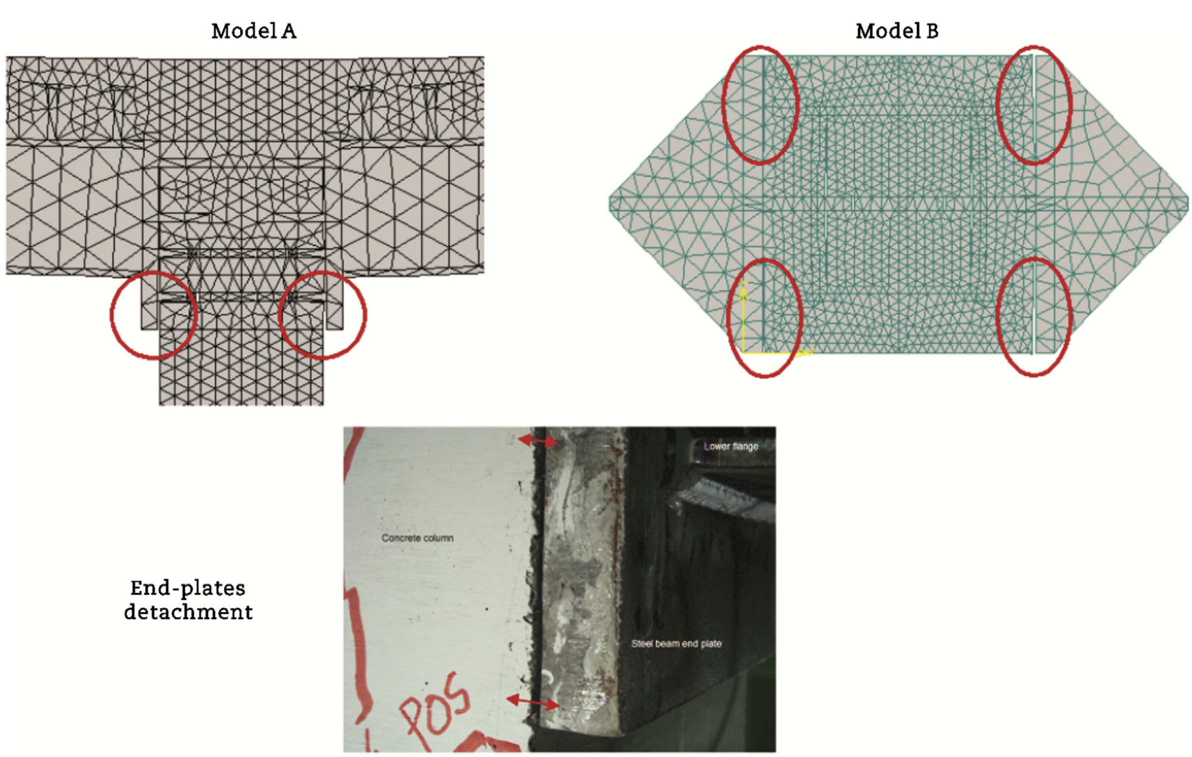

Fig. 15 – Detachment of the end plates.

In the following sections, the first two steps are introduced and explained in detail.

3. Experimental tests

The assessment of the real response to sagging moment and cyclic loads of the proposed joint, in terms of characterising parameters such as moment resistance, rotation capacity and initial stiffness, was carried out through a number of lab tests. The response of joints to hogging bending was expected to match that previously found during the first experimental campaign (Zordan and Briseghella, 2009).

In order to achieve this, a set of three identical, full-scale specimens of the joints and parts of the connected composite beams and concrete column was assembled. The main features of the specimens are shown in Fig. 5.

The steel-to-concrete connection is ensured by the presence of a number of 5/8″ headed studs, both at the connection between the upper flange of HEA260 and the concrete slab, and the connection between the concrete column and the steel end plate of beam. For each upper flange of steel beam, couples of 5/8″ headed studs with an overall height of 100 mm were welded at a wheelbase of 120 mm, with the first row having a distance of 48 mm from the steel end plate. In total, 26 studs were welded to each flange of the steel profile. 12 studs were welded to each of the two steel end plates, with the layout shown in Fig. 5. The reinforcement used in specimens is indicated in Fig. 6.

The materials adopted for different parts (components) of joints are shown in Table 1.

The design shear resistance of the headed studs used in the beam to pier joints was calculated following EC4, Section 6.3.2 as in the following

$$P_{rd} = \min\left(P_{rd}^{(1)},\ P_{rd}^{(2)}\right) \tag{1}$$

$$P_{rd}^{(1)} = 0.8 \times f_{u,St} \times \left(p \times d^2/4\right)/\gamma_n \tag{2}$$

$$P_{rd}^{(2)} = 0.29 \times 1 \times d^2 \times \left(f_{ck} \times E_{cm}\right)^{1/2}\Big/\gamma_n \tag{3}$$

where d is diameter of the shank of studs ($d \leq 25$ mm) and its value is 15.87 mm, f_{ck} is characteristic cylinder concrete strength of concrete at the considered age and its value is

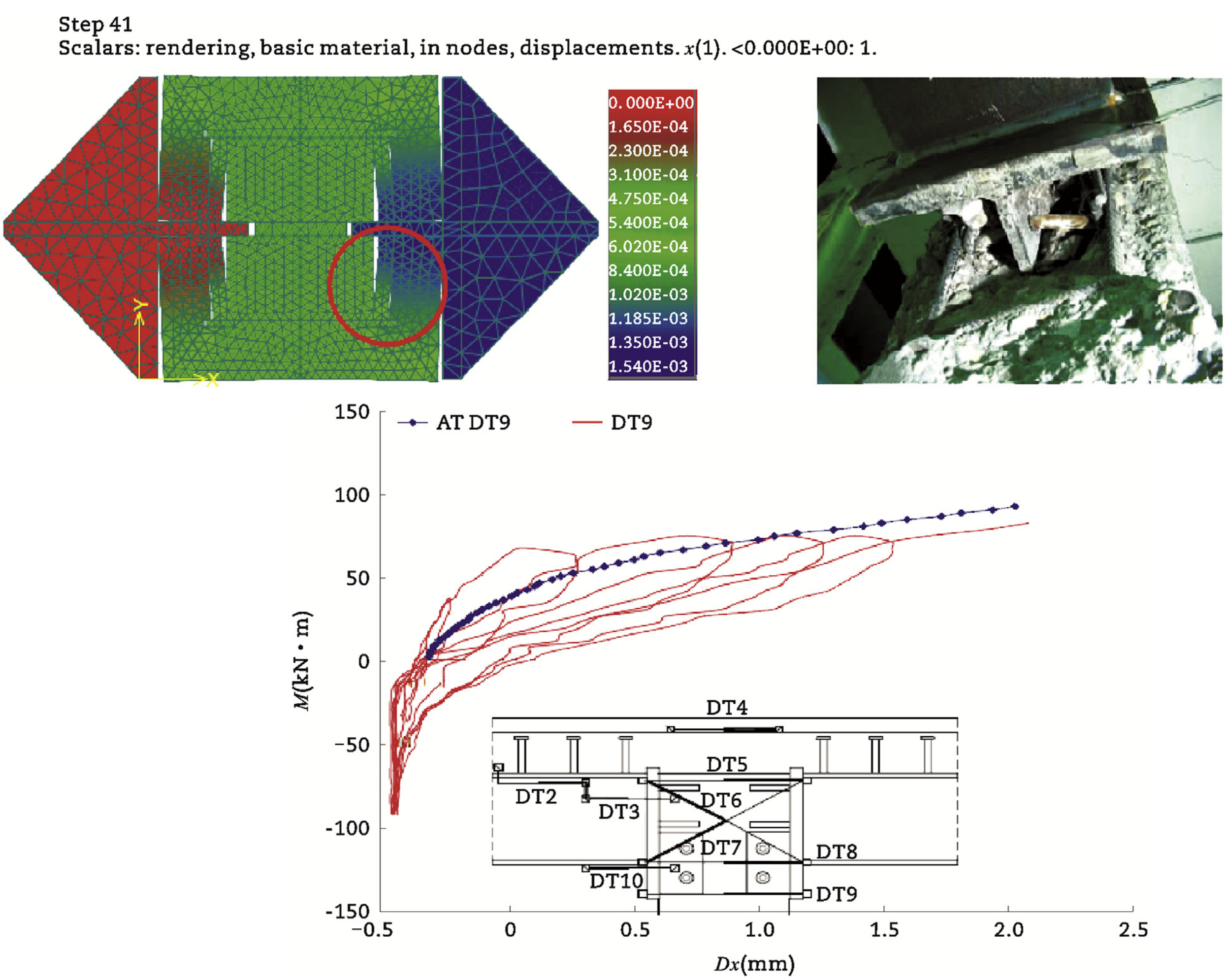

Fig. 16 – Force-horizontal displacement curves.

30 N/mm^2, $f_{u,St}$ is specified ultimate tensile strength of the material of stud and its value is 450 N/mm^2, E_{cm} is short secant modulus value of the concrete and its value is 32,000 N/mm^2, γ_n is a safety factor for connectors and its value is 1.25 (considered as 1 in the following).

Therefore

$$P_{rd}{}^{(1)} = 0.8 \times 450 \times (p \times 15.87^2/4)/1 = 71.21 \text{ kN}$$

$$P_{rd}{}^{(2)} = 0.29 \times 1 \times 15.87^2 \times (30 \times 32000)^{1/2} \Big/ 1 = 71.56 \text{ kN}$$

f_{ck} and E_{cm} values come from EC2, Table 3.2.

$$P_{rd} = \min\left(P_{rd}{}^{(1)},\ P_{rd}{}^{(2)}\right) = 71.21 \text{ kN}$$

3.1. *Test setup*

Three full-scale specimens were tested at the Laboratory of Structures and Materials of the University IUAV of Venice (Italy). The following test setup was used (Fig. 7).

1. A rigid reaction frame restrained to the reaction floor of the laboratory;
2. A hydraulic jack used to apply vertical force to specimen and restrain it to the reaction frame;
3. Two hydraulic jacks, with P_{max} = 220 kN (push), P_{max} = 120 kN (pull) and a stroke of 660 mm, were used to apply cyclic forces.

The hydraulic jacks were anchored to reaction floor. Loads were applied at a distance of 1860 mm from the beams' extremities.

At the joint locations, as well as along the whole specimen, a specific instrumentation (Fig. 8) was installed in order to assess the local and global structural response during testing. For this assessment, a number of displacement transducers were installed, both at the joint location and the beams' extremities. Additionally, a series of strain gauges were welded and glued to the structural steel and to the rebar. The types of instruments are listed in Table 2.

3.2. *Test results*

The test load applied to specimens is presented in Fig. 9 in terms of moments applied in the joint section and cycle number.

The concrete strength of the column near the horizontal shear studs determined the response of the joint for sagging moment. This sequence is shown in Fig. 10 with the rotation of joint calculated starting from the data recorded by transducers DT5 and DT9 for specimen S3.

According to the curve in the previous figure, the following property characterised the specimen: the moment resistance of the joint $M_{j,Rd}$ is 74.40 kN m.

The strain gauges welded to the reinforcement bars confirmed the identified failure mechanism. In Fig. 11, the curves related to readings supplied by the strain gauges welded to vertical stirrups of the column are shown.

The main cracking pattern in the concrete zone between two studs in pier (transparent area in Fig. 12) is shown below.

4. Finite element analysis

An accurate simulation of the response of composite joints should include a three dimensional (3D) analysis, but two-dimensional (2D) models can be used to reduce the computation difficulties associated with large 3D meshes. In the case of a 2D representation, the contribution of uncracked slab generally requires a preliminary 3D elastic analysis, which allows the relevant effective slab width defined.

Due to the complex interaction between main nodal components and their key features, FE joint models should consider both geometrical and material non-linearity. As far as the material laws for steel components (beam, column, steel connection details, studs and rebars) are concerned, a good agreement with the actual behaviour is usually provided by the sole uniaxial multilinear stress–strain relationship, which can be correlated to a more complex and representative loading state via the selection of suitable yielding criteria.

For the slab modelling, concrete constitutive laws provided by general-purpose FE packages are based on a smeared cracked approach, which assumes an equivalent isotropic continuum with smeared cracks for simulation of the slab after attainment of the concrete tensile resistance. Due to the presence of longitudinal bars in concrete slab, two modelling

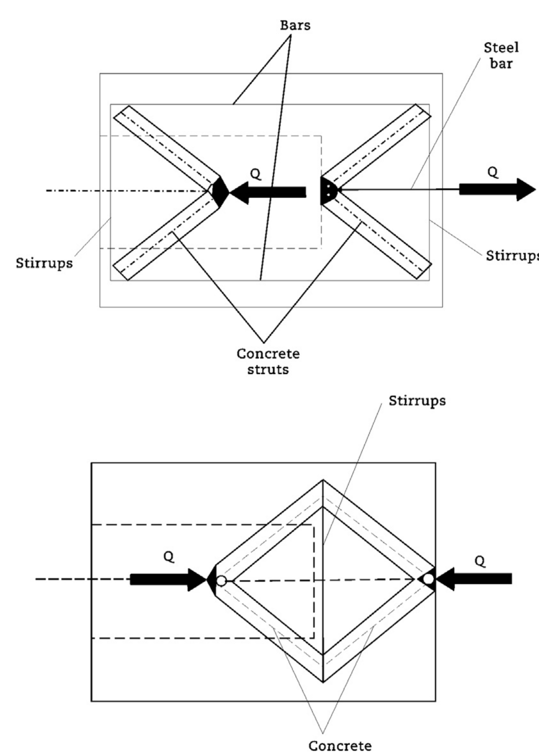

Fig. 17 – Strut and tie model.

techniques can be used for simulation of the slab reinforcements:

1. A discrete rebar approach: rebars are modelled using truss or beam non-linear elements, which satisfy the displacement compatibility with concrete elements;
2. A smeared rebar approach, in which rebar-concrete interaction in tension can be modelled by appropriate modifications of the concrete constitutive relationship.

Moreover, an accurate simulation of interaction (contact/separation) between concrete slab and steel beam, as well as steel connection and member (beam and column), requires the use of interface and/or non-linear spring elements, the constitutive non-linear laws of which can be defined in accordance with proposals available in literature.

As recommended by Ahmed et al. (1996) and many other authors, for a successful numerical modelling of composite connections, great attention and care must be devoted to:

1. Reinforcement in the slab, with a careful representation of bond between concrete and steel;
2. Shear studs, considering the slip between upper beam flange and concrete slab deriving from the percentage of shear interaction provided;
3. Steel beams and columns, including eventual buckling phenomena and plasticity.

For both low and high load levels, the tensile non-linear behaviour of the concrete also plays a fundamental role in the overall structural response.

In this paper, a finite element model of the full-scale specimen was implemented using the software Atena 2D Ver. 2.02 (Cervenka et al., 2013) to create a tool able to interpret non-linear response of the connection and role played by different components of the joint. The model was validated against the test results.

Atena is capable of conducting finite element analysis, which considers the nonlinearities deriving from tensile response of concrete, and of modelling the crack propagation phenomenon. Two models of the specimens were implemented, one global (model A) and one local (model B), considering only the interaction between horizontal shear studs and concrete pier (Fig. 13).

A smeared approach was used to model concrete material properties, such as cracks or stirrup distribution, with an equivalent uniaxial stress–strain law for concrete in tension and compression (ascending branch: MC90; softening branch: van Mier, 1986). In model A, a discrete approach was used to simulate the longitudinal reinforcements, and a smeared approach was used for rebar; while in model B, the rebar in piers was modelled using a discrete approach. An interface material model based on the Mohr-Coulomb criterion with a tension cut-off was used to simulate the contact between two materials. After stress exceeds this condition, dry friction between the two materials in contact is simulated.

In model B a fixed connection is applied to the end of the lower flange of one steel beam (Fig. 13), while the force is applied to the other. The load history is the same used for the experimental tests considering all the tensile force due to the positive moment absorbed by lower flange of the steel beam.

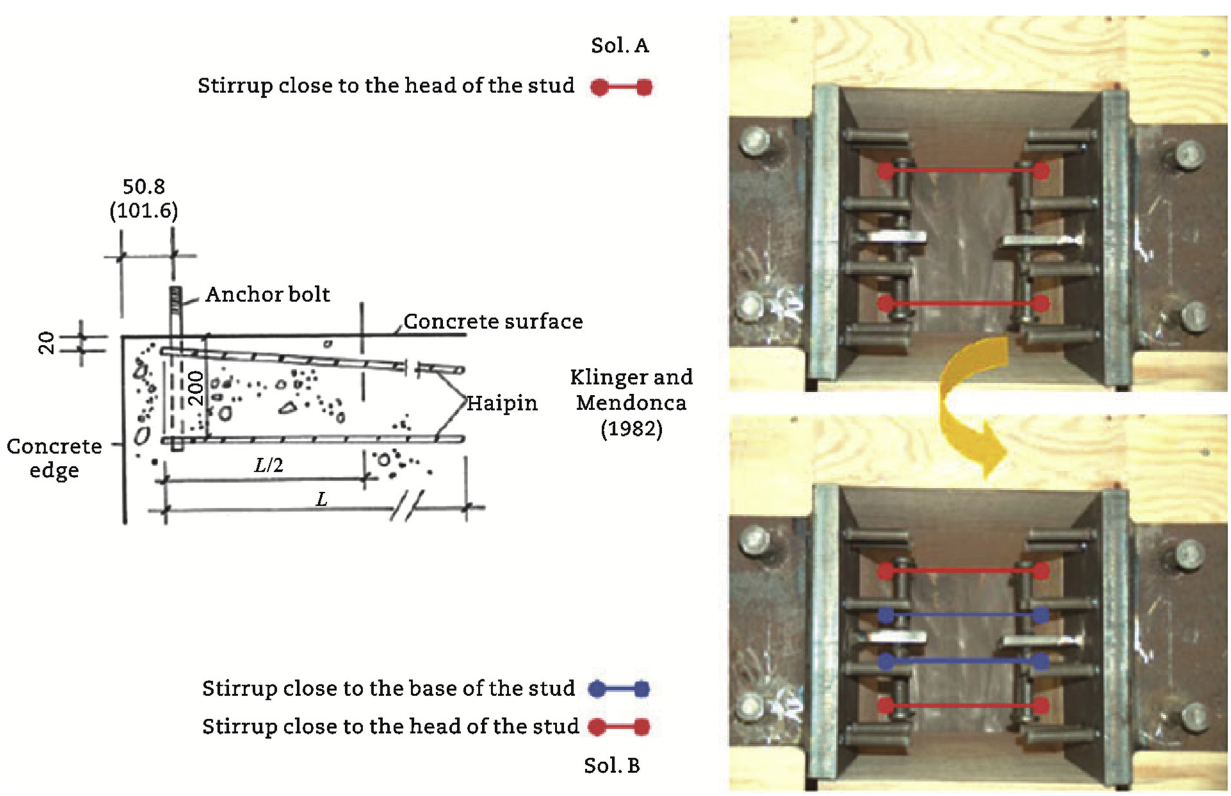

Fig. 18 – Joint update.

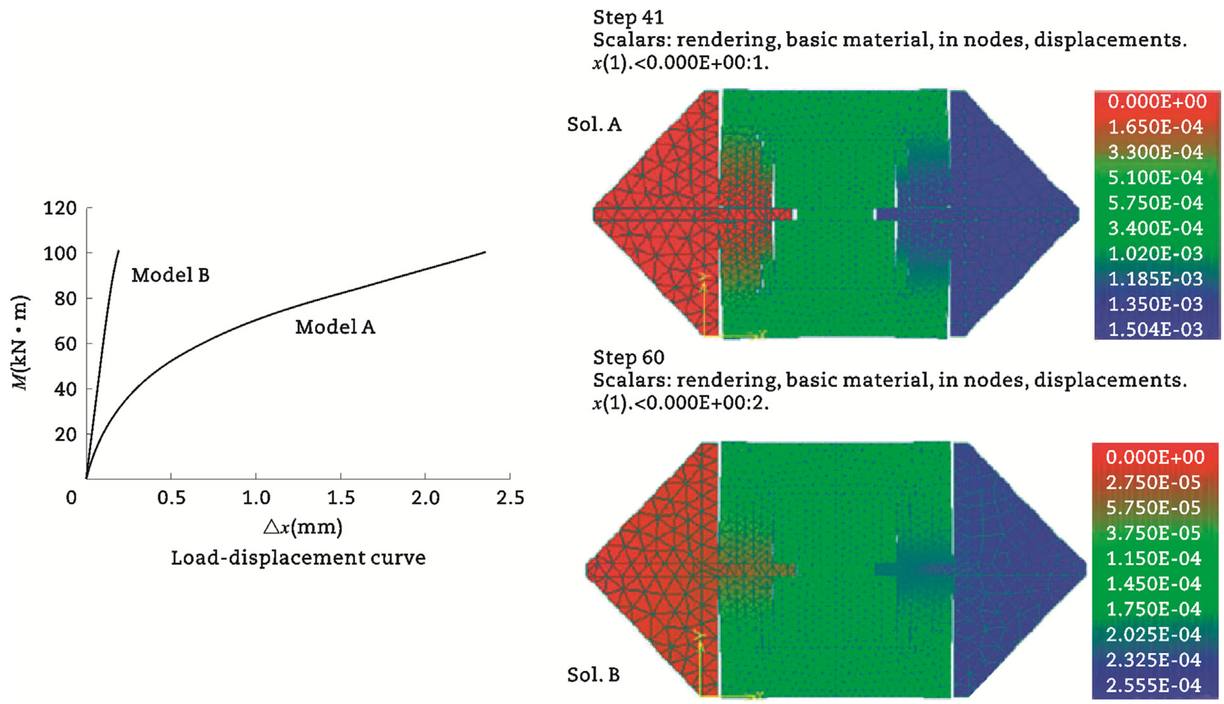

Fig. 19 – Force-horizontal displacement curves for solutions A and B.

Figs. 14 and 15 show the main cracking pattern and the detachment of steel beam end plate, obtained from the FEM analysis and experimental tests. Fig. 16 shows the force-horizontal displacement curves.

5. Discussion

Due to the joints layout used, structural response was mainly influenced by the tensile resistance of pier's concrete; this is apparently ascribable to the location of headed studs welded to vertical plates within the concrete column, and to the position of stirrups. In fact: the limited embedment of studs welded to the vertical steel plates within the concrete column, together with the direct contact between studs and stirrups, had hindered the formation of a "strut and tie" resistance mechanism (Fig. 17).

The beneficial role of the stirrups in connecting two end plates of steel beams through the studs welded to vertical steel plates started for states of strain not compatible with overall integrity of connection. In order to improve the joint behaviour, a joint update, with stirrups close to the base of the studs, as recommended by Klinger and Mendonca (1982), was also proposed (Fig. 18). Using the previously introduced FE model B (Section 4), the study performed a good response from the connection, with failure due to the shear studs (Fig. 19).

6. Conclusions

The following conclusions can be drawn within the limitations of the research presented in this paper:

1. The proposed joint seems to be a good solution for beam-to-pier connections in integral abutment bridges. Its construction method aimed to require the smallest number of skilled workers and to minimise the overall construction yard time, for a general limitation of costs. Furthermore, the layout proposed allows, as much as possible, the reduction of tolerance problems due to the connection between steel and concrete.
2. The experimental tests showed a satisfied joint response under cyclic load conditions, when considering the joint's use in integral abutment bridges. The joint response was mainly influenced by the degradation of the concrete in the pier.
3. The role of stirrups in connecting two end plates of the steel beams through studs welded to the vertical steel plates was also investigated. In order to improve the structural behaviour, a joint update, with stirrups close to the base of the studs, was proposed and analysed using an FE model.

Acknowledgments

The authors would like to thank the technical personnel of the IUAV Laboratory of Structures and Materials for their valuable help in conducting the experimental tests. The help of Metralab and its director Dr. Ezio Giuffre was also appreciated.

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