ANALISI SPERIMENTALE DI SOLETTE DA PONTE SU LASTRE METALLICHE TRALICCIATE COLLABORANTI

EXPERIMENTAL ANALYSIS OF BRIDGE SLABS ON COLLABORATING STEEL PLATES WITH ELECTRO-WELDED LATTICE GIRDERS

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ABSTRACT

Experimental study involved steel plate with electro-welded lattice girders used for concrete casting of bridge slabs to obtain a composite slab, in which the connection between concrete and metal plate is exclusively entrusted to the lattice welded on the sheet.
At "Laboratory of structures and materials tests" of DICEA two kinds of experimental tests have been carried out: slide tests and shear-bending tests. The article describes in detail the shear-bending tests that have allowed to investigate the overall behaviour of the plate-concrete system on smaller samples representing real slabs. All the tests sample have shown a good connection behaviour, a good ductility. The satisfactory experimental results show how an adequate design of welded connections between the lattice and metal plate allows for economically and constructively advantageous solutions.

SOMMARIO

Lo studio sperimentale ha riguardato l'impiego di lastre metalliche tralicciate collaboranti, per la realizzazione di solette da ponte, nelle quali la connessione è affidata unicamente alla saldatura del traliccio sulla lamiera.
Presso il "Laboratorio prove strutture e materiali" del DICEA è stata condotta una campagna sperimentale con due tipologie di prove: prove di scorrimento e prove di taglio-flessione. Nell'articolo sono descritte in dettaglio le prove di taglio-flessione che hanno permesso di indaga-

re il comportamento complessivo del sistema lastra tralicciata-calcestruzzo su campioni in scala ridotta rappresentativi di solette reali. Tutti i campioni hanno evidenziato un buon comportamento della connessione, buona duttilità e resistenze molto più elevate di quelle assunte in progetto. I risultati sperimentali soddisfacenti mostrano come un'adeguata progettazione dei collegamenti saldati tra i tralicci e le lamiere permetta di giungere a soluzioni economicamente e costruttivamente vantaggiose.

1 INTRODUCTION

Steel plates with electro-welded trussed girders are becoming an increasingly popular technique for building reinforced concrete bridge slabs. The tests carried out were aimed at assessing the efficiency of the welding between plate and trussed girder that guarantees the connection between the plate and the concrete. It also supported the technological choice to use composite trussed metal plates without the use of other connectors and to implement this choice - eliminating or profoundly reducing the bottom transversal reinforcements during installation - to build the bridge slabs for the decks of numerous viaducts built along the SS.640 Caltanissetta-Agrigento road by the company Empedocle 2 s.c.p.a..
This article aims to make a viable contribution, in terms of data from experiments that can support the study and use of this technology by technical engineers in the industry.

2 DESCRIPTION OF SAMPLES

The test samples are very similar to the real slab used in the above mentioned viaducts. They were also defined, as far as possible, with reference to the guidelines given in "EN 1994-1-1:2004 – Annex B: Standard tests".

2.1 Materials

The steels used to make the samples all had quality certification listing their actual mechanical characteristics.

- STEEL plate 4 mm thick: S355J0WP (fy=441 MPa, ft=512 MPa)
- STEEL electro-welded lattice: B450C (fy=498 [ϕ12] -505 [ϕ16] MPa, ft=624 MPa)
- CONCRETE REINFORCEMENTS: B450C (fy=545 MPa, ft=643 MPa)

For the concrete casting, a class C32/40 (nominal) was envisaged

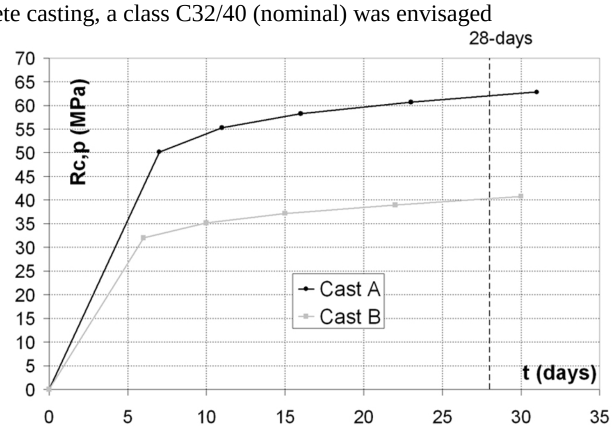

Fig. 1. Concrete curing curves

An accelerator was added to the concrete casts in order to reach the characteristic nominal resistance before the standard 28-day period, and the casts were carried out on two separate days: cast A on 29/02/2016 and cast B on 01/03/2016.
The development of resistance over time was assessed with a series of crush tests, taking care to carry out at least one together with the girder tests. The first cast showed compressive strength values far higher than those of cast B and in general, compared to the reference values for class C32/40.
Over time, the compressive strength curve for the two casts, as shown in the graph, reveals how the differences in strength between the two are due to intrinsic differences in the material and not to different curing levels.

2.2 Shear-bending test samples

The samples used for the shear-bending tests were small beams that, in section, reproduced a strip of slab from the area inside the two main girders of the viaducts mentioned above. The 0.34 mm thickness of the concrete, the reinforcements, lattice and plate reproduced a life-sized example of a plate strip, 0.40 m wide, with a single exception of the effective span.
Four identical 5.30 m samples were made and placed over a 4.00 m span. The metal plate was 4 mm thick and the electro-welded metal lattice was 205 mm high and consisted of an upper bar Ø16, two bottom bars Ø12 and diagonal pieces, Ø10, as shown in the figure.

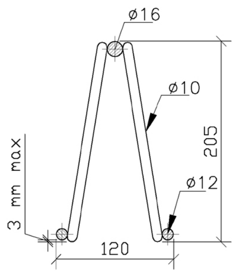

Fig. 2. Lattice

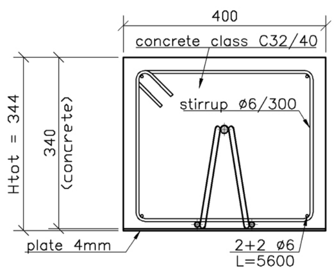

Fig. 3. Sample section

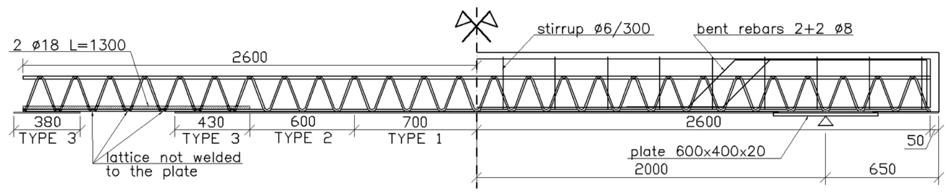

Fig. 4. Shear-bending sample

The lattice was welded to the metal plate in the nodes between bottom bars and diagonal pieces. Three types of weld were defined, set out in different sections along the sample axis, with constant spacing and constant throat height, while the length varied

- TYPE 1 : 2x 45 mm welds, 3 mm throat, 200 mm pitch
- TYPE 2 : 2x 75 mm welds, 3 mm throat, 200 mm pitch
- TYPE 3 : 2x 85 mm welds, 3 mm throat, 200 mm pitch

For a 540 mm section, the lattices were not welded to the plate and the plate itself was not present, in such a way as to reproduce the passage of the lattices on top flange of the viaduct girders.

In the above zone, there were 2 Ø18 reinforcement bars welded to the two bottom bars and diagonal pieces of the lattice, at the nodes.
The concrete element was weakly reinforced to guarantee the cast would be confined. The stirrups were set at a distance sufficient to make their involvement in the shear strength mechanism marginal. Bent reinforcements were placed in the area of the supports, similarly to what has actually been used on the viaduct bridge slab.

3 TEST METHODS

Two types of test were performed, each involving two of the four samples. During the tests, the applied forces were continuously measured as well as vertical displacement and axial deformation of the bottom flange at mid-span and about at a third of the span. This last measurement, taken on each of the 3 sections , was obtained using 3 strain gauges (a total of 9) placed along the longitudinal axis and in two laterally eccentric positions.

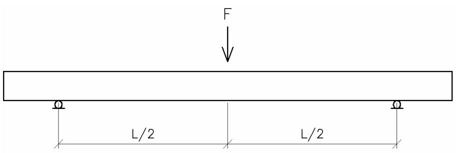

Fig. 5. Single load tests – Girders A and C

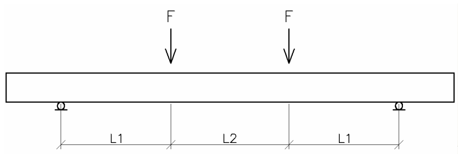

Fig. 6. Tests with two loads -Girders B and D

L = 4.00 m L1 = 1.30 m L2 = 1.40 m

The tests achieved the maximum design stresses and continued until sample failure.
For the envisaged collapse load Frp, the load applied during the test followed the methods below:

1. A settling load cycle with controlled load between 0 and 0.05 x Frp and subsequent unloading
2. Loading ramp with controlled load between 0 and 0.40 x Frp and subsequent unloading
3. Loading ramp with controlled displacement from 0 to 1.00 x Frp
4. Continuation of the loading ramp with controlled displacement, even beyond 1.00 x Frp, until complete failure

The speed of load application during stage 3 was chosen to achieve the forecast failure in no less than 15 minutes and it was also kept constant in stage 4.

4 EXPECTED STRENGTH

In order to assess the resistance of the plate - slab system during the design stages of the bridge, the presence of the steel plate was assumed as very conservatively reduced to 50%, both to take into account the effect of shear lag and of the possible loss of thickness over time, due to corrosion. In the following assessments, reference is however made to the real amount of constituent material of the plate as well as to the bars used to make the lattice and the modest reinforcement in the section.
The actual bending strength was evaluated using the model σ–ε ”stress block” for concrete and the model σ–ε ”elastic-perfectly plastic” for steel. As well as the metal plate, all of the reinforcements were considered and the actual yield strength of the steels was assumed. For the concrete, the fc strengths obtained in the crushing tests were used. No safety coefficient for the materials was taken into account.

The result is that the expected bending strength is constant along the whole length of each sample, i.e.:
- Girder A : 280 kNm - Girder B : 281 kNm
- Girder C : 264 kNm - Girder D : 268 kNm

Shear failure may occur either according to the typical modes for reinforced concrete sections or for steel-concrete sections. Taking into account the aims of the tests, the shear strength value was found taking into account the problem in terms of the loss of adherence between the slab and the concrete due to the failure of the welded connection between the slab and the lattice. Three values for shear resistance were calculated for the three sections with different welds, based on the following expression, derived from the weld strength, as estimated in point 4.2.8.2.4 of NTC 2008:

$$\mathrm{Vrd} = 0{,}9\mathrm{H}\,\frac{\mathrm{n}}{\mathrm{P}}\,\frac{\mathrm{Ls}\cdot \mathrm{a}\cdot \mathrm{ft}}{\sqrt{3}\cdot\beta} \tag{1}$$

where: *H* beam height; *n*, *P*, *Ls* and *a* number, pitch, length and throat of the weld; $\beta = 0.9$; ft = 512 MPa.

The result is: - Vrd,1 = 136 kN - Vrd,2 = 226 kN - Vrd,3=256 kN

Starting with bending and shear strength and also taking into account the weight effect (3.5 kN/m), expected ultimate tensile strength was estimated for the two types of test.

4.1 Single load tests - girders A and C

Expected bending breaking load:

- Girder A : Pr,a,f = 273 kN
- Girder C : Pr,c,f = 257 kN

Expected shear breaking load, limited by Type 1 weld resistance:

- Girder A : Pr,a,v = 265 kN
- Girder C : Pr,c,v = 265 kN

4.2 Tests with two loads - girders B and D

Expected bending breaking load:

- Girder B : Pr,b,f = 209 kN Pr,btot,f = 418 kN
- Girder D : Pr,d,f = 199 kN Pr,dtot,f = 398 kN

Expected shear breaking load, limited by Type 2 weld resistance:

- Girder B : Pr,b,v = 219 kN Pr,btot,v = 438 kN
- Girder D : Pr,d,v = 219 kN Pr,dtot,v = 438 kN

5 TEST RESULTS

5.1 Single load tests - girders A and C

Both girders subjected to tests with load concentrated at mid span were taken to failure exceeding the ultimate load envisaged for bending and shear.

The maximum ultimate loads measured for the two girders are very similar:

- Girder A : Prp,a = 285 kN
- Girder C : Prp,c = 280 kN
- Average value between the two tests: Prp= 282,5 kN

The failure occurred due to bending (concrete crushing) for both tests, without there being any signs of shear failure on the section. The following diagrams show the progress of displacement in the mid-span section and at L/3 as a function of the test load.

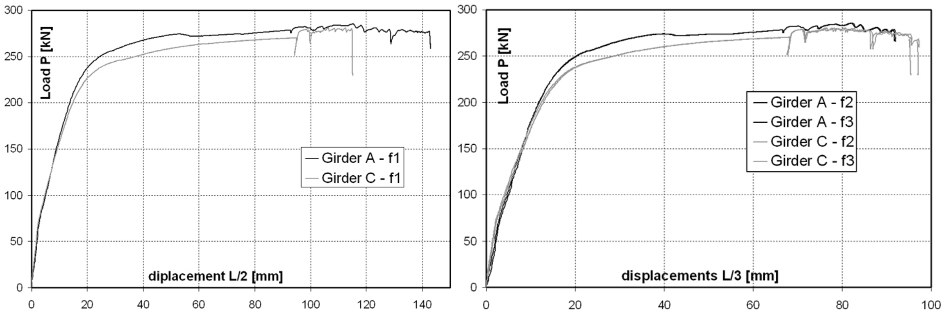

Fig. 7. Girders A and C – Failure test, displacement in the mid-span section and at L/3

The following diagrams show the deformation measured by the strain gauges in the mid-span area. It can be seen that strain gauge “e2b” on the girder axis shows less deformation compared to those on the sides, which are more or less the same. This is considered to be an index of the good connection guaranteed by the lattice-slab welds, since strain gauges “e2a” and “e2c” are placed in their proximity.

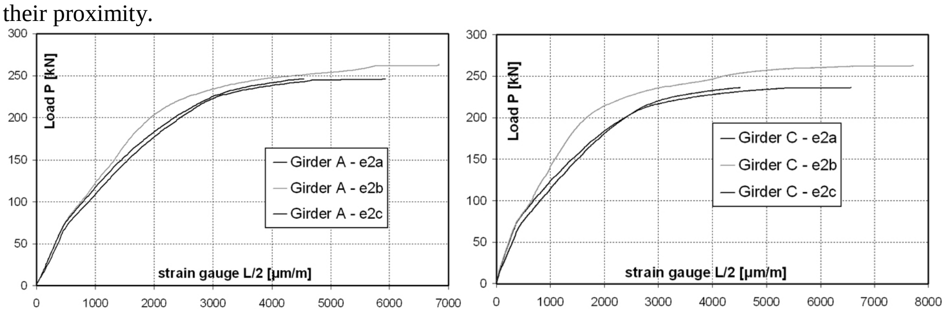

Fig. 8. Girders A and C – Failure test, strain gauges at L/2

The displacement at mid-span exceeds 100 mm (1/40 L) and the samples show a decidedly ductile behaviour on failure.

5.2 Tests with two loads - girders B and D

The girders tested with two loads ends were taken to failure point at loads close to those foreseen for bending strength.
The maximum ultimate loads measured for the two girders are:

- Girder B : Prp,b1 = 187 kN; Prp,b2 = 201 kN; Prp,btot = 388 kN
- Girder D : Prp,d1 = 193 kN; Prp,d2 = 219 kN; Prp,dtot = 412 kN
- Average value between the two tests Prp = 200,00 kN

The different load values on the two actuators for each test are the result of working in displacement control. Since there is a very small difference, the effect can be assumed as the one given by two equal loads.
It was found that failure was due to bending (concrete crushing) in both tests, in spite of the formation, during the test, of some clear shear cracks. The ultimate load value reached on Girder D was higher than foreseen, similar to that observed for samples A and C. For Girder B, it was slightly less than envisaged. This girder, however, had a slight casting defect on the extrados

which was then affected by the failure of the concrete when the test piece failed, and that presumably, led to a reduced resistance which was, any case, marginal.
The following diagrams show the progress of displacement in the mid-span section and at L/3 according to the total load value (P=P1+P2) of the test.

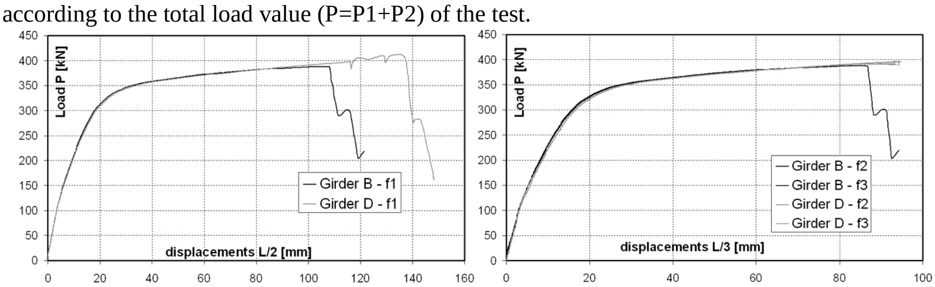

Fig. 9. Girders B and D – Load versus displacement at mid-span and at L/3

The diagrams that follow show the deformation measured on the strain gauges placed in the mid-span sections. In this case, the considerations made in previously regarding the different deformations measured by the strain gauge “e2b” on the axis and the ones on the sides of the girder, also apply.

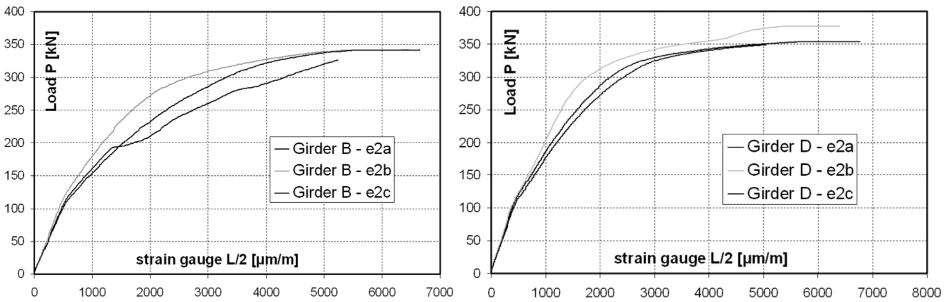

Fig. 10. Girders B and D – Failure test, strain gauges at L/2

In these cases too, as for the tests loaded in the centre (girders A and C), the displacement to failure in the mid-span section exceeded 100 mm (1/40 L), which shows highly ductile behaviour.

8 CONCLUSION

In all the tests performed the ultimate load achieved values very close to or above those envisaged for the failure of the welded connection between the lattice and the slab but the collapse of the samples always occurred due to bending, with concrete crushing. The load reached was very close to that forecast, showing that all the 400 mm wide slab was effectively working together with the concrete, and that the lattice and slab welds, although spot welds, actually allowed the whole slab to be involved in the ultimate bending mechanism.
Even the notable ductility shown can be considered an index of connection efficiency.

Fig. 11. Girder C – Failure test

Fig. 12. Girder D – Failure test

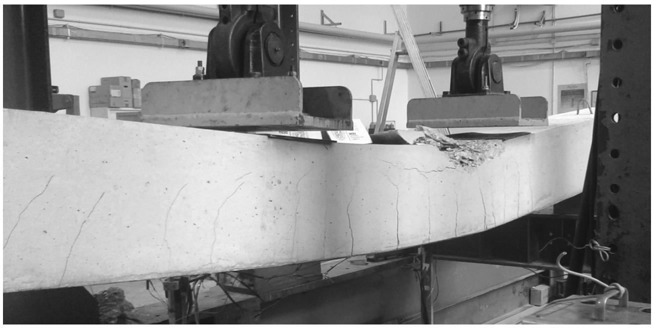

Fig. 13. Girder D – View of the final cracked status

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KEY WORDS

Shear connections, bed plates for bridges, collaborating metal lattice slabs, loading and breaking tests, shear-bending load tests