XL CILAMCE
IBERO-LATIN AMERICAN CONGRESS ON COMPUTATIONAL METHODS IN ENGINEERING
NOVEMBER 11-14, 2019
Praiamar Natal Hotel & Convention
Natal, RN-BRAZIL

STRUCTURAL BEHAVIOR OF NEW THIN-WALLED COMPOSITE TRUSSED BEAMS AND PARTIALLY PREFABRICATED CONCRETE SLABS

Luiz Alberto Araújo de Seixas Leal
Eduardo de Miranda Batista
albertoleal26@hotmail.com
batista@coc.ufrj.br
Programa de Engenharia Civil – COPPE – UFRJ
Centro de Tecnologia - Av. Horácio Macedo, 2030 - 101 - Cidade Universitária, 21941-450, Rio de Janeiro, Brasil

Abstract. The composite trussed beams consist of a very traditional solution for building floors and are associated to several benefits, such as reduced consumption of materials, excellent structural response in terms of bending capacity, as well as flexural stiffness, ease of fabrication and erection. Furthermore, the adoption of open-web composite beam enables larger free-spanning, compared to traditional T-sections (composed by the association of hot-rolled I-shaped steel member and concrete slabs), and allows the detailing of electric, hydraulic and HVAC (Heat, Ventilation and Air-Conditioning) lines through the web of the trusses. One of the first researches regarding the structural behavior of this type of composite floor system was conducted in 1960's, from the experimental full-scale tests in the United States. In the last 40 years, a lot of work has been published to provide reliable information in terms of shear connection solutions, degree of interaction between the steel trusses and concrete slabs, mechanisms of collapse and analytical models for the prediction of the bending capacity. In almost all cases, the steel members are composed with hot-rolled sections (double angle, for example) and the shear connectors are welded to the top chord of the truss. In this context, the objective of the present work consists of presenting (a) alternative types of shear connectors, (b) innovative structural floor system, composed by thin-walled trusses and prefabricated concrete slabs. Three types of steel shear connectors were developed, manufactured with 0.95 mm thick plates: Thin-Walled Perfobond, Thin-Walled Channel and Thin-Walled Vertical Post. The connection between the connectors and the top chord of the trusses were provided with the help of 4.8 mm self-drilling screws. Cold-formed lipped channel members (0.95 or 1.25 mm thick), partial prefabricated one-way concrete slabs and styrofoam blocks compose the floor systems. Based on the results of an extensive experimental program, important preliminary conclusions can be assessed. The obtained data revealed higher bending capacity compared to traditional analytical predictions and excellent structural response in terms of flexural stiffness and ductility.

Keywords: Steel and concrete composite floor system; Steel and concrete composite trussed beam; Composite open-web joist; Perfobond shear connector

1 Introduction

The adoption of composite steel and concrete structures is justified by several benefits associated to the referred structural solution, such as improved bending capacity, higher flexural stiffness, reduction of construction costs and optimized material consumption.

In recent years, many researches have been published in order to investigate analytical and experimental behavior of steel-to-concrete members, including composite trussed beams and the connection between the concrete slab and the top chord of the truss. In countries such as United States and Canada, extensive research efforts are being made since 1960’s, as a result of investments provided by steel fabricators.

The first important work related to open-web composite structures was published by Lemberck [1], The research involved the experimental full-scale bending tests of two prototypes without shear connectors and three prototypes with different types of connectors. The mechanisms of collapse, associated to the referred composite steel-to-concrete trusses, were defined by the rupture of the diagonals next to the end supports. In this context, it was possible to observe the benefits related to the association between the steel trusses and the concrete slab (metal deck), such as improved structural response, in terms of stiffness and bending capacity. However, as a consequence of premature collapse of the diagonals, composed by round sections (diameter of Ø 1/2"), the actual resistance capacity of the mid-section (composed by top chord, bottom chord and effective compressive concrete) was not reached.

The connections between diagonals (round sections) and the chords (double angle) of the trusses was assessed by fillet welds. The prototypes were defined by two trusses (spacing of 610 mm) supporting concrete slabs with metal deck (total height of the slab was 64 mm). In addition, the shear connections were provided by means of (a) diagonal members (positioned over the top chord during the fabrication stage and inside the slab after the concrete pouring) and (b) top chord member (one side of each angle was involved by the slab after the concrete pouring).

Wang & Kaley [2] conducted four experimental full-scale bending tests, in which one of the prototypes was not designed as a composite open-web joist. The chords’ cross-section was defined by a hat member and the concrete slab was composed by metal deck with total height of 64 mm. In terms of the shear connection, one of the three composite prototype adopted plug-weld to attach the top chord of the trusses to the metal deck.

The experimental results demonstrated improved bending capacity, compared to the non-composite truss system. The obtained ultimate loads were, approximately, 87% and 126% higher than bare steel trussed beams. Furthermore, the composite trusses showed reduced vertical displacements, almost 33% up to 61% lesser, denoting improvements in terms of flexural stiffness.

Tide & Galambos [3] developed five full-scale bending tests, in order to investigate the structural behavior of the composite open-web joists, as well as stud connectors (defined by diameter of 3/8”). The adopted concrete slab was composed by conventional rectangular cross-section with a height of 76 mm. In all cases, the collapse mechanisms were associated to the rupture of the bottom (tension) chord.

In recent years, Merryfeld et al [4] tested two full-scale prototypes, defined by a clear span of 6.7 m and a width of the concrete slab (corrugated steel deck of 0,90 mm thick) of 2.4 m. The top and bottom chord are composed by different double angle sections (L44x44x3.2 mm and L51x51x4.8 mm), supporting the slab with total height of 115 mm (65 mm over the metal deck). All web members were fabricated with round sections with a diameter of 17.5 mm

The connection between the steel top chord and the concrete is provided by two different solutions: (a) puddle weld and (b) hilti self-drilling screws with a diameter of 6.35 mm. The experimental results indicated that both alternatives were able to provide adequate degree of interaction between the steel trusses and the concrete slabs.

2 Full-scale experimental tests

2.1 Presentation of the shear connectors

In this present work, three different composite floor systems were investigated in order to provide reliable information about the behavior of the structures. The referred solutions are composed by two thin-walled steel trusses, partially prefabricated concrete (one-way) slabs and styrofoam blocks. In addition, three different types of innovative shear connectors were conceived in order to guarantee full interaction between the steel trusses and concrete slabs: (a) Thin-Walled Perfobond, (b) Thin-Walled Channel and (c) Thin-Walled Vertical Post.

In all cases, the shear connectors are composed by cold-formed steel members and are attached to the top chord of the trusses by means of self-drilling screws (diameter of 4.8 mm). In Figure 1a, some details of the Thin-Walled Perfobond (TWP) during the fabrication stage of the full-scale prototype can be seen. In Figure 1b, note that the referred connector is constituted by three components: (a) twin “Z” sections, connecting each other by two screws, (b) two transversal steel bars (diameter of 4.2 mm, length of 300 mm and nominal yield stress of 600 MPa) and (c) steel plate with dimensions of 92x150x1.25 mm, placed between the top chord of the trusses and the twin “Z” section. The connection of the TWP and the referred chord is detailed with eight self-drilling screws (four units connecting the twin “Z”, the steel plate and the top chord).

In Figure 2, the dimensions of the TWP are presented, including the length of shear connector (100 mm), as well as the positions of the screws. The holes made at twin “Z” sections were fabricated with a diameter of 8.0 mm, enabling the positioning of the transversal bars. It is important to emphasize that all the referred cold-formed members are composed by 1.25 mm thick plates and galvanized steel (nominal yield stress of 345 MPa).

Figure 1. Details of TWP shear connector: (a) during fabrication stage of the full-scale prototype and (b) 3D computational modelling

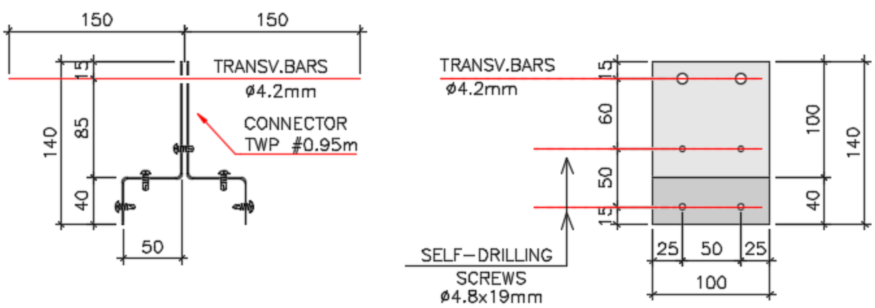

Figure 2. Representation of TWP dimensions: (a) cross-section and (b) side view

The TWP was designed to ensure adequate resistant mechanisms against forces acting on the referred connection, such as longitudinal and transversal forces, which tends to produce relative displacements between the top chord of the trusses and the slabs. The transversal steel bars, as well as the fictitious concrete dowels (between the referred bars and the holes at the surface of the TWP connector) are able to sustain the forces in both directions.

On the other hand, the vertical component of the twin “Z” sections (height of 100 mm, as indicated in Figure 2) provides a resistant mechanism against longitudinal forces by means of the contact area between the referred member and the concrete slabs

The Thin-Walled Channel (TWC), as can be seen in Figure 3, is also composed by three components (galvanized steel and nominal yield stress of 345 MPa), such as: (a) cold-formed lipped channel U89x40x12x0,95 mm (width of 60 mm), (b) single angle L40x40x0.95 mm, providing local reinforcement to the web of the referred lipped channel, and (c) steel plate with dimensions of 92x150x0.95 mm. The connection between all members of the TWC and the top chord of the trusses is composed by fourteen self-drilling screws, as indicated in Figure 4a.

The design of the TWC considered two main shear transfer mechanisms between the concrete slab and the connector. In this context, the top flange and the reinforced web of the lipped channel are able to provide support against longitudinal and transversal (uplifting effects) forces.

Figure 3. Details of TWC shear connector: (a) front view, (b) back view and (b) 3D computational modelling

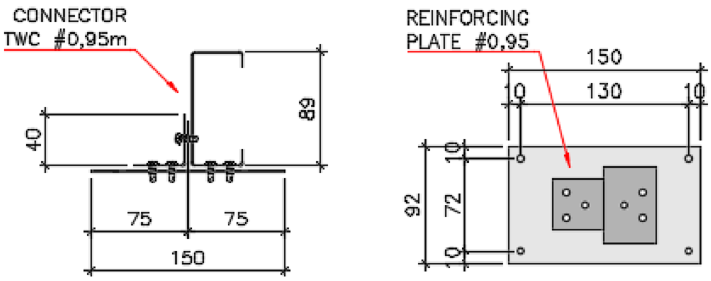

Figure 4. Representation of TWC dimensions: (a) side view and (b) top view

The TWVP presents a different concept compared to the other solutions, especially because of the adoption of one member (vertical post) of the trusses to work as shear connector. In figures 5, observe that the TWVP is constituted by three structural components: (a) vertical post of the trusses, composed by a cold-formed section (U89x40x12x0.95 mm), (b) single angle L40x40x0.95 mm (width of 85 mm) and (c) transversal bars (diameter of 4.2 mm).

The referred shear connector was conceived in order to reduce time, as well as cost, associated to the fabrication stage. The adoption of the vertical post to act as a member of the trussed beams and, at the same time, contributes to the connection between the top chord and the concrete slabs certainly consists of a good alternative (from the fabrication point of view).

However, it is important to emphasize that the shear connector can be embedded by the concrete slab only because of the opening at the top chord (fabricated in an automated process), producing negative effects in terms of strength and flexural stiffness of the composite trussed beams, as can be seen in the following. Note, in Figure 5c, that the top chord of the truss is composed by two additional members (lipped channel U140x40x12x0.95 mm), as an alternative to minimize (not eliminate) the weakening of the chord. In addition, the referred members were conceived to connect the diagonals and verticals of the trusses.

In Figure 5c, it is possible to observe the presence of the single angle, attached to the vertical post by means of two self-drilling screws, and the transversal bars, placed in the holes (diameter of 8.0

mm) at both flanges of the sections U140x40x12x0.95 mm.

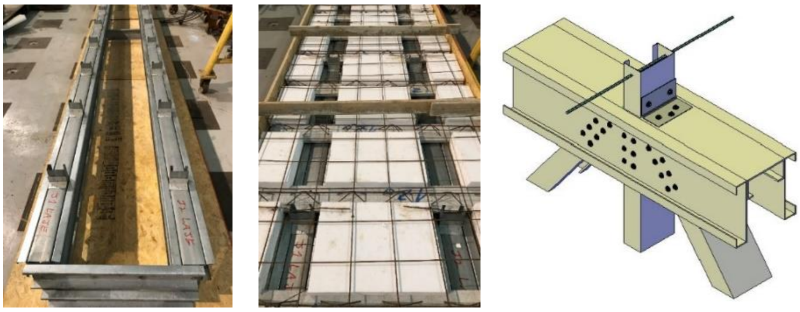

Figure 5. Details of TWVP connector: (a) top view, (b) prior to the concrete pouring and (c) 3D computational modelling

Once all innovative solutions for shear connectors were presented, it is important to emphasize that the TWP and TWC were subjected to push-out tests (according to recommendations established by Eurocode 4 [5] for traditional stud bolts) and the results indicated that the referred members are suitable for the connection between steel and concrete. Based on experimental data of eight specimens (four prototypes for each type of connector), the performance of the referred members was significant better, in terms of load capacity, when compared with analytical predictions. In addition, the deformation capacity (ductility) of the connectors, based on longitudinal relative displacements between the steel and concrete, is in accordance to the minimum values recommended by Eurocode 4 [5].

On the other hand, the TWVP shear connectors could not be investigated by push-out tests because problems related to premature collapse of the truss (mechanism of load transfer from the hydraulic actuator to the connectors), prior to the rupture of the TWVP. Based on preliminary push-out tests, performed by Leal & Batista [6], the collapse of the prototype would be associated to local buckling of the truss, avoiding reliable information about the structural behavior of the TWVP.

Note, in Figure 6a, the presence of two solid concrete slabs and one steel truss, connecting each other by means of four vertical posts (TWVP). In Figure 6b and 6c, it is possible to observe the characteristics of the TWP prototypes (TWC prototypes are similar, except for the type of the connector). Instead of steel truss, the load transfer mechanism was composed by a tubular compact member (double lipped channel U100x50x17x3,00 mm), shown in Figure 7.

In Table 1, the experimental results of eight push-out tests are presented. Observe that the average ultimate load associated to the TWC prototypes, applied for the hydraulic actuator (250 kN load capacity), is 100.9 kN (mean value for all indicated ultimate load). In this context, the shear capacity of each one of the four connectors, embedded in the concrete slabs, is 25.2 kN.

The ultimate force, related to the TWP prototypes, is 203.7 kN (mean value), as can be seen in Table 1. Consequently, the shear capacity associated to each one of the four connectors inside the concrete is 50.9 kN, which is considerably higher compared to TWC members.

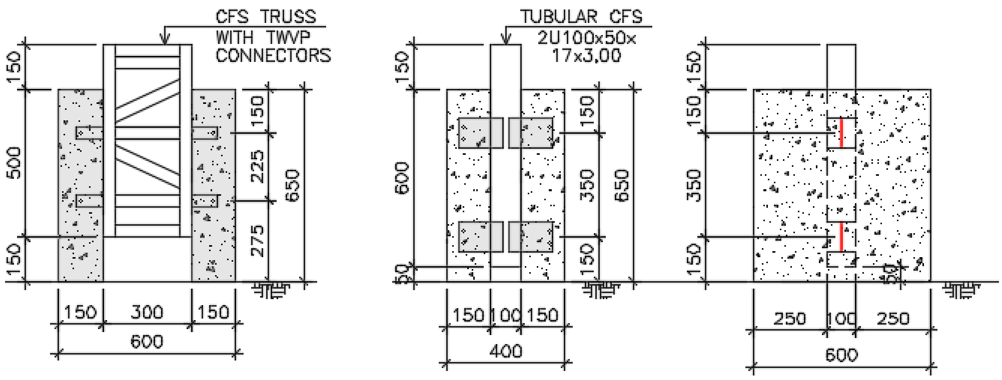

Figure 6. Configuration of the push-out tests: (a) TWVP prototype, (b) front view of TWP prototypes and (c) side view of the TWP and TWC prototypes

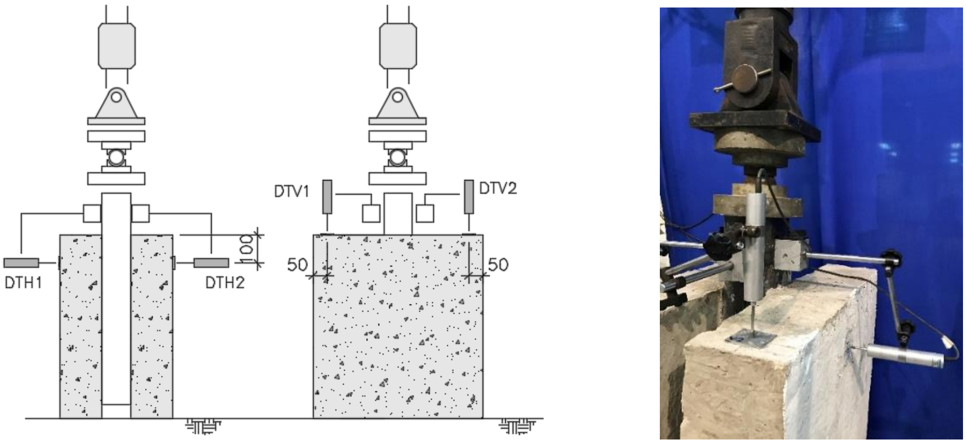

Figure 7. Instrumentation of the push-out tests: (a) front view, (b) side view and (c) vertical and horizontal displacement transducers

Table 1. Experimental results of push-out tests of the TWC and TWP connectors

Specimen	Concrete f_c (MPa)	Thickness (mm)	Ultimate load $P_{u,exp}$ (kN)
TWC1	18.9	0.95	87.7
TWC2			86.1
TWC3			119.4
TWC4			110.2
TWP1	18.9	0.95	193.9
TWP2			178.1
TWP3			209.0
TWP4			234.4

* The thickness is associated to the adopted steel for the shear connectors;

The mechanisms of collapse, associated to the TWP and TWC shear connectors, are shown in Figure 8. All prototypes associated with the TWP presented a failure produced by net rupture of the connector next to the four self-drilling screws, as indicated in Figure 8a.

The ultimate force, related to the TWP prototypes, is 203.7 kN (mean value), as can be seen in Table 1. Consequently, the shear capacity associated to each one of the four connectors inside the concrete is 50.9 kN, which is considerably higher compared to TWC members

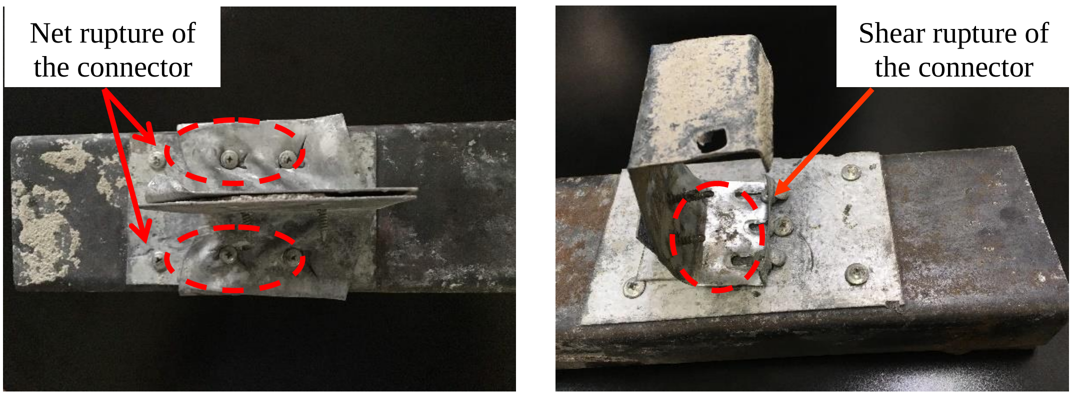

Figure 8. Mechanisms of collapse, associated to: (a) TWP and (b) TWC shear connectors

2.2 Full-scale prototypes: fabrication and test set-up

The present research involves, as mentioned before, the experimental investigation of three full-scale prototypes, subjected to bending tests, in order to evaluate the structural behavior of innovative composite trussed beams, in terms of ultimate capacity, flexural stiffness, mechanisms of collapse, as well as the performance of the shear connection between steel and concrete.

The steel trusses were fabricated in an automated process, typically adopted for Steel Framing construction, and are composed by thin-walled lipped channel (U89x40x12x0,95 mm), connected by means of self-drilling screws (Ø 4.8 x 19 mm). It is important to emphasize that the TWP prototype components are constituted by lipped channel U89x40x12x1,25 mm, due to availability of the materials, provided by GypSteel (Brazilian company located in Rio de Janeiro state).

The cross-section of the specimens TWP and TWC, including the concrete slab, chords and diagonals, can be seen in Figure 10a. Note that the referred prototypes are constituted by twin steel trusses and a partially pre-fabricated concrete slabs (width of 1,200 mm), supported by the top chords of the trussed beams.

Another important aspect refers to the adoption of additional reinforcing members, defined by thin-walled lipped channel U140x40x12x0,95 mm, at the top chord of the TWVP prototype (Figure 9b). This reinforcement was needed to minimize loss of effectiveness (bending capacity and flexural stiffness, for example) produced by the opening at the top chord member of the referred specimen.

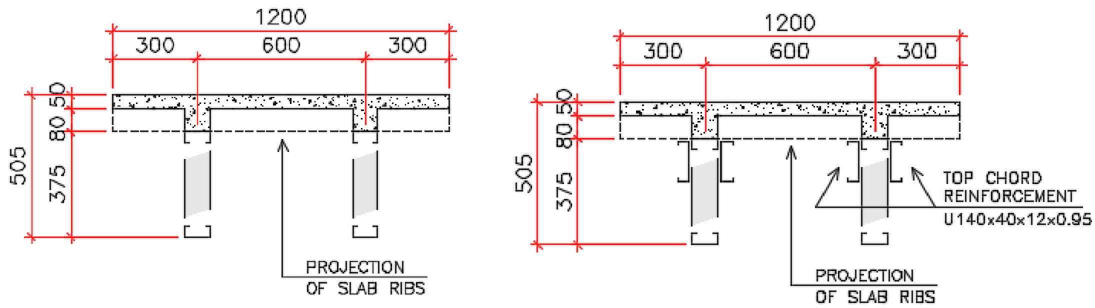

Figure 9. Cross-section of the full-scale composite trussed beams: (a) TWP and TWC specimens and (b) TWVP specimen

In Figure 10, a typical detail of the concrete slab's cross-section is represented. The referred member is composed by pre-cast concrete ribs (associated to a spatial steel truss), styrofoam blocks, steel welded mesh and a second stage concrete (60 mm of concrete over the blocks plus the concrete fulfilling the ribs). The welded mesh is constituted by steel bars with 4.2 mm diameter and spaced by 150 mm in both directions.

In figures 11, 12 and 13, typical details of the connection between the diagonals and the top chords are represented, including the distribution of the self-drilling screws, dimensions of the gusset plates and local reinforcements (steel plates for the TWP and TWC prototypes, as well as U 140x40x12x0.95 sections for TWVP specimen). As mentioned previously, the adoption of the reinforcement for the top chord of the TWVP trussed beams was required to minimize (not eliminate) loss of effectiveness, produced by the web opening (Figure 13).

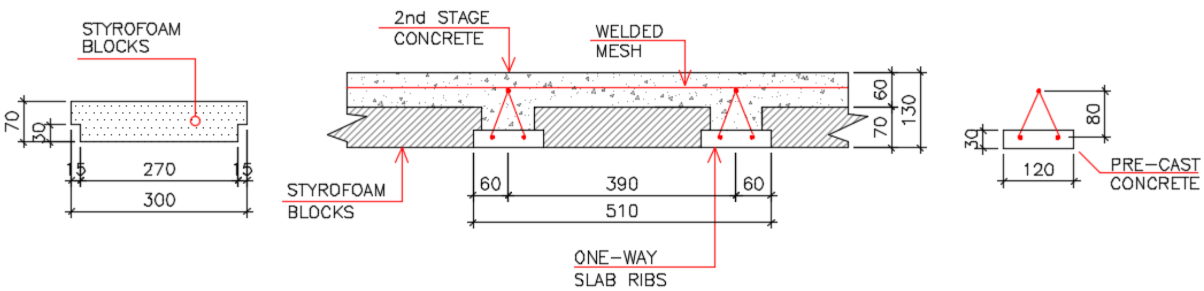

Figure 10. Cross-section of the one-way partially pre-cast concrete slab

Once all the composite trussed beams were presented, the bending test set-up and the adopted instrumentation deserve special attention. The full-scale four-point bending tests were performed in the Structures Laboratory of Federal University of Rio de Janeiro (COPPE/UFRJ) and the applied forces were produced by a 250 kN hydraulic actuator. During the experimental tests, the response of the specimens, in terms of axial deformation (both chords and the top face of the concrete slab), vertical displacements (in the direction of the applied loads) and relative displacements between steel trusses and concrete slabs. All data were obtained by means of the Kyowa Data Logger (frequency of acquisition of 2 Hz) and the applied load was introduced at a rate of 0,005 mm/s (displacement control).

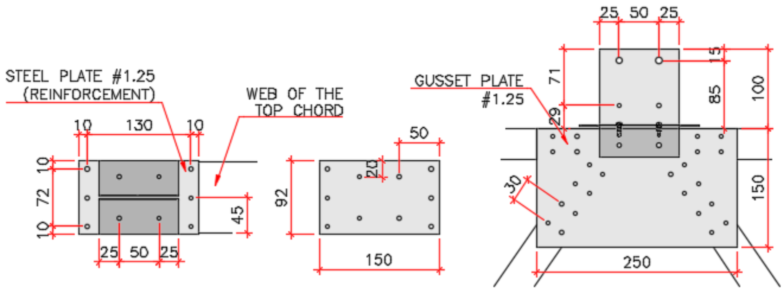

Figure 11. Typical details of the connection between chords and diagonals of the TWP prototype

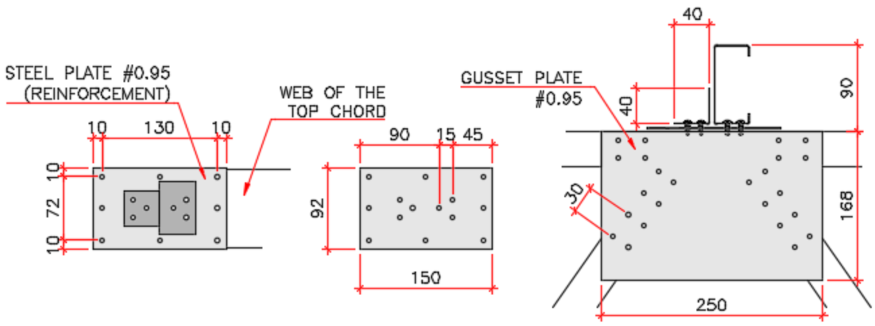

Figure 12. Typical details of the connection between chords and diagonals of the TWC prototype

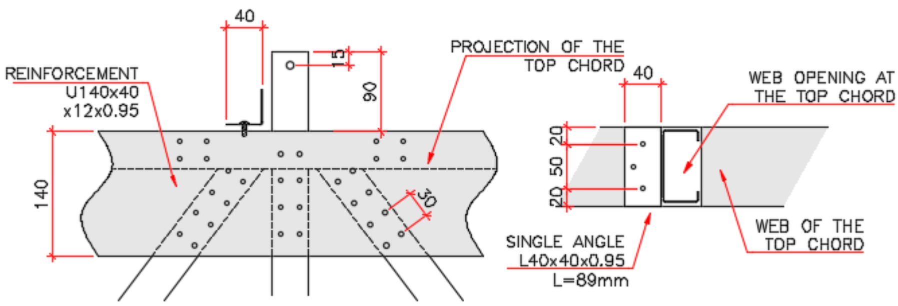

Figure 13. Typical details of the connection between chords and diagonals of the TWVP prototype

In figures 14 and 15, the representation of the test set-up (TWP/TWC and TWVP, respectively) are indicated. The hydraulic actuator transmits the forces with the help of three rigid spreader beams (one heavy longitudinal I-shaped beams and two transversal double channel U6”x12,2 kg/m), creating a pure bending zone (between the referred double channels).

The extension of the pure bending zone for the TWVP prototype, given by the location of the transversal spreader beams, was intentionally defined in order to guarantee a minimum of five vertical posts shear connectors (each steel truss) along the length of 3235 mm (between the end of the specimen and the axis of one of the transversal beams).

The instrumentation, adopted for the prototypes, involved extensometers and displacement transducers, in order to investigate the behavior of the composite trussed beams in terms of force distribution at the cross-section, relative displacements (slippage) between the top chord and the concrete slab, flexural stiffness and depth of the neutral axis. In addition, it is possible to evaluate the degree of interaction between steel and concrete, based on the obtained data.

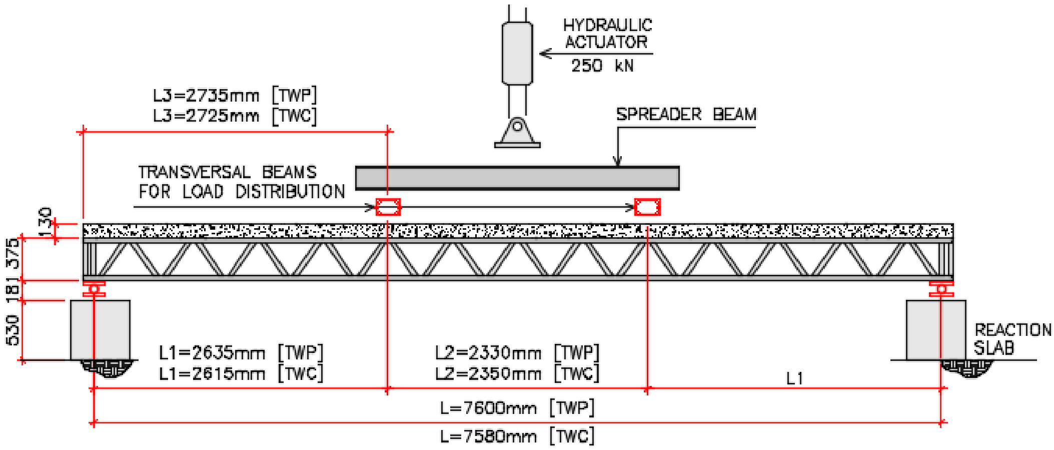

Figure 14. Full-scale bending test set-up (TWP/TWC prototypes)

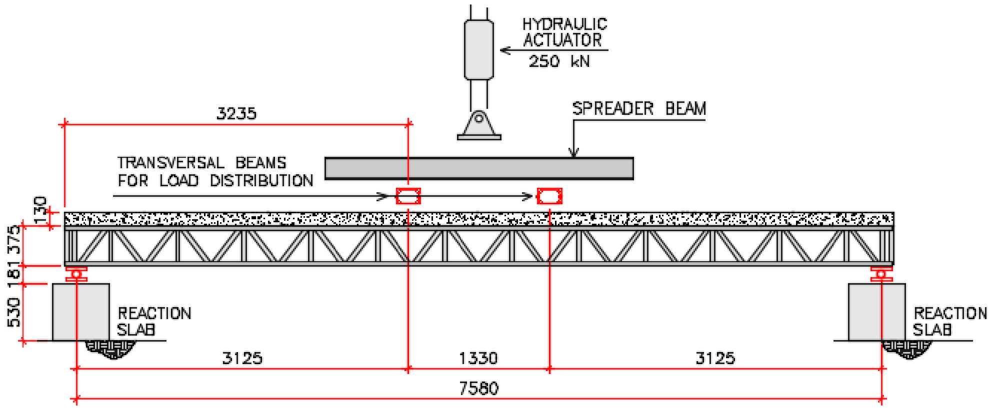

Figure 15. Full-scale bending test set-up (TWVP prototype)

In figures 16, 17, 18 and 19, the location of the sensors is represented, including ten extensometers placed at the top face of the concrete slab, eight extensometers for the steel members (four each truss) and eight displacement transducers. The extensometers, as indicated in figures 16, 17 and 18, were placed in the pure bending zone, in order to provide reliable information about the behavior of the composite structures in a situation of collapse, especially in terms of the actual contribution of the top chords of the trusses.

In Figure 20, a typical arrangement of the extensometer along the cross-section of the trussed beams is shown. Observe that the distribution of the sensors placed at the bottom chord, as well as at the concrete, is the same for all investigated prototypes. On the other hand, the extensometers associated to the top chord of the TWVP specimen were placed at the web of the reinforcement composed by U140x40x12x0.95 mm.

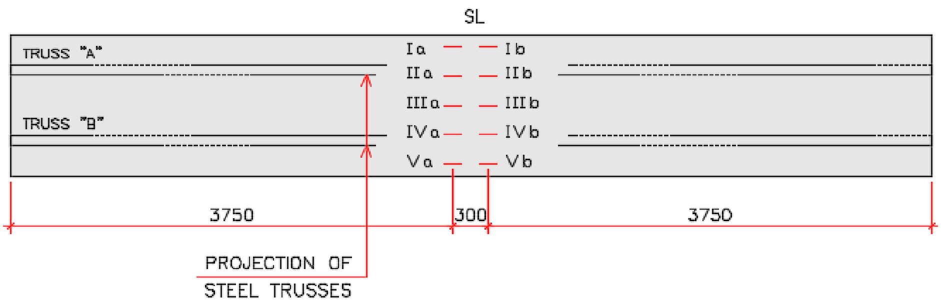

Figure 16. Arrangement of the extensometer at top of the concrete slab

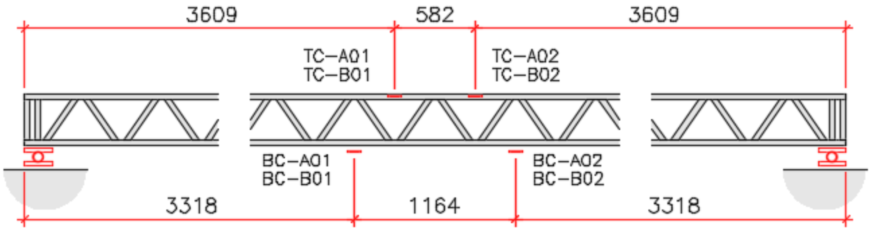

Figure 17. Arrangement of the extensometer at steel chords of the trusses (TWP/TWC prototypes)

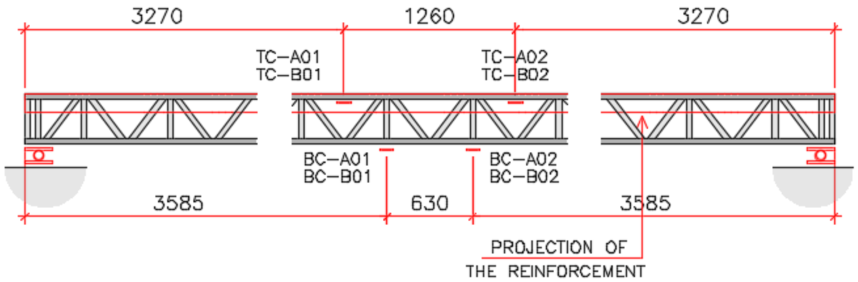

Figure 18. Arrangement of the extensometer at steel chords of the trusses (TWVP prototype)

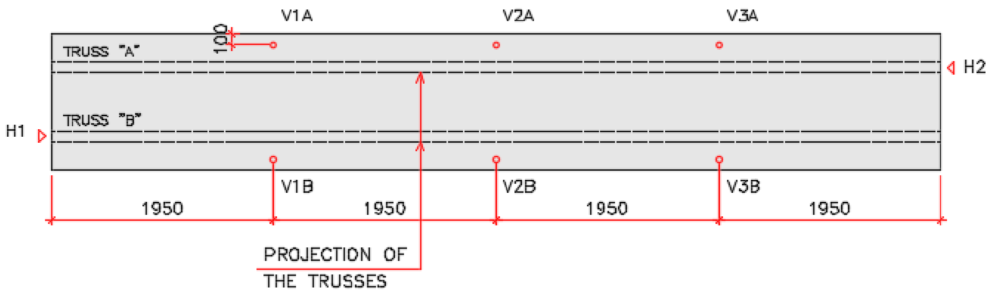

Figure 19. Arrangement of the displacement transducers located at the concrete slab

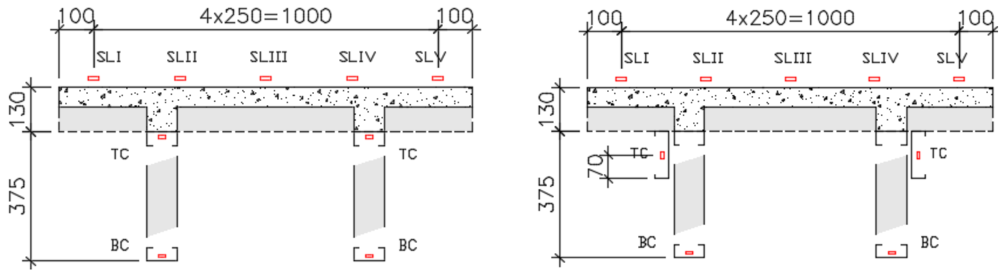

Figure 20. Arrangement of the displacement transducers at top of the concrete slab

2.3 Analytical models for prediction of ultimate capacity

In this present work, the obtained results are compared with some traditional analytical models in order to provide a comparative analysis between the experimental data and the theoretical predictions for ultimate bending capacity of the prototypes. In this context, Figure 21 shows four models, usually adopted for the evaluation of composite trussed beams with full interaction between the top chord and the concrete slabs.

The Model-1 consider an elastic stress distribution at the cross-section, composed by both chords and the compressive concrete (above the elastic neutral axis). This model, generally adopted by international standards (such as, ABNT NBR 8800 [7] and ANSI/AISC 360 [8]) for the verification of traditional composite W-shaped beams for service loading conditions (displacements and vibration, for example), is based on the transformed cross-sectional elastic properties.

The Model-2 and Model-3, on the other hand, are based on the plastic stress distribution, as can be seen in Figure 21. The constitutive models associated to the steel and concrete members are elastic-perfectly plastic and rigid-plastic, respectively. It is important to emphasize that the only difference between the models refer to the contribution (or not) of the top chord of the trusses.

The Model-2, which doesn't take into account the top chord, is recommended by ANSI/SJI 200 [9] and ABNT NBR 8800 [7]. The Model-3 was adopted by recent researchers, such as Mujagic, Easterling & Murray [10], for verification the ultimate strength of composite trussed floor systems. In addition, the referred model is based on the assumption of full-plastic stress distribution of the cross-section, usually adopted for the verification of bending capacity of composite W-shaped compact beams.

The Model-4 assumes that the resistant mechanisms of the cross-section are composed by the compressive concrete (rigid-plastic constitutive model) and the tension bottom chord, subjected to tensile stress distribution. The effective area of the cross-section, considered for the determination of the force acting on bottom chord, is the net area.

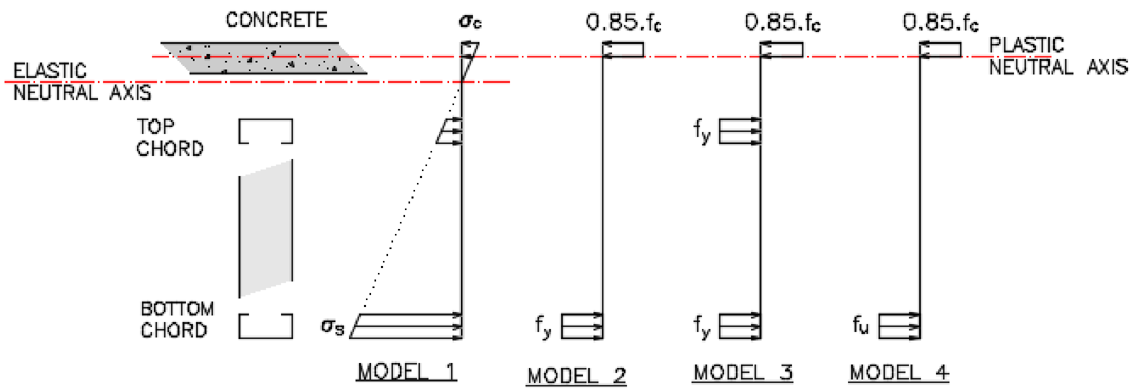

Figure 21. Analytical models for ultimate bending capacity predictions

In tables 2,3 and 4, some information about the prediction of bending moment and estimated actuator force, associated to the prototypes, are indicated. The depth of neutral axis, as well as the estimated bending capacity, were obtained by the equilibrium of forces and moments at the cross-section, respectively.

The self-weight moment, represented in referred tables, takes into account the effects produced by the self-weight of the spreader beams, as well as each prototype. The self-weight associated to the spreader beams 9.0 kN.

The applied moment, on the other hand, represents the effects produced by the hydraulic actuator until the composite trussed beam reach the theorical bending capacity. In addition, considering the referred applied moment, it is possible to calculate the estimated actuator forces, based on the tests set-up represented in figures 14 and 15.

In tables 5, 6 and 7, important information regarding the mechanical and geometrical properties of the cross-section of the prototypes are indicated. Additionally, the properties of the steel and concrete members, based on standard experimental tests, are shown. The tests were conducted according to ABNT NBR ISO 6892-1 [11] and ABNT NBR 5739 [12], respectively for the steel and the concrete specimens.

Table 2. Prediction of bending capacity of the TWP floor system and estimated actuator force

Analytical Model	Contribution of Top Chord	Depth of Neutral Axis [cm]	Bending Capacity [kN.cm]	Self-W Moment [kN.cm]	Applied Moment [kN.cm]	Estimated Actuator Force [kN]	Estimated Total Force [kN]
1	Y	6,25	8658		5436	41,3	65,7
2	N	1,06	8478	3222	5256	39,9	64,4
3	Y	2,12	10695		7472	56,7	81,2
4	N	1,13	9010		5788	43,9	68,4

*Self-weight of the TWC prototype: 22.0 kN

Table 3. Prediction of bending capacity of the TWC floor system and estimated actuator force

Analytical Model	Contribution of Top Chord	Depth of Neutral Axis [cm]	Bending Capacity [kN.cm]	Self-W Moment [kN.cm]	Applied Moment [kN.cm]	Estimated Actuator Force [kN]	Estimated Total Force [kN]
1	x	5,03	6755	3202	3553	27,2	51,7
2		0,50	6468		3265	25,0	49,5
3	x	1,00	8261		5058	38,7	63,2
4		0,50	6503		3301	25,2	49,7

*Self-weight of the TWC prototype: 22.0 kN

Table 4. Prediction of bending capacity of the TWVP floor system and estimated actuator force

Analytical Model	Contribution of Top Chord	Depth of Neutral Axis [cm]	Bending Capacity [kN.cm]	Self-W Moment [kN.cm]	Applied Moment [kN.cm]	Estimated Actuator Force [kN]	Estimated Total Force [kN]
1	x	5,97	8176	3439	4736	30,3	52,3
2		0,49	6367		2928	18,7	40,8
3	x	1,75	12684		9244	59,2	81,2
4		0,50	6503		3064	19,6	41,6

* Self-weight of the TWVP prototype: 21.4 kN;

Table 5. Mechanical and geometrical properties of the cross-section of the TWP prototype

Steel				Concrete		Composite cross-section properties			
Young Mod (Es) [MPa]	Yield Stress [MPa]	Ult Stress [MPa]	Yield Deform (ε) [μstr]	Concrete Str (f_c) [MPa]	Secant Mod (E_c) [MPa]	Modular Ratio (η) -	Moment of Inertia [cm4]	Elastic Mod W_t [cm3]	Elastic Mod Wb [cm3]
188605	375.2	445.2	1989	16.1	19099.4	9.9	10211	1635	230

Table 6. Mechanical and geometrical properties of the cross-section of the TWC prototype

Steel				Concrete		Composite cross-section properties			
Young Mod (Es) [MPa]	Yield Stress [MPa]	Ult Stress [MPa]	Yield Deform (ε) [μstr]	Concrete Str (f_c) [MPa]	Secant Mod (E_c) [MPa]	Modular Ratio (η) -	Moment of Inertia [cm4]	Elastic Mod W_t [cm3]	Elastic Mod Wb [cm3]
185768	359.6	424.1	1936	26.0	24271.3	7.7	8293	1649	182

Table 7. Mechanical and geometrical properties of the cross-section of the TWVP prototype

Steel				Concrete		Composite cross-section properties			
Young Mod (Es) [MPa]	Yield Stress [MPa]	Ult Stress [MPa]	Yield Deform (ε) [μstr]	Concrete Str (f_c) [MPa]	Secant Mod (E_c) [MPa]	Modular Ratio (η) -	Moment of Inertia [cm4]	Elastic Mod W_t [cm3]	Elastic Mod Wb [cm3]
185768	359.6	424.1	1936	26.0	24271.3	7.7	9827	1645	221

2.4 Axial deformation of the bottom and top chords

The results of the full-scale bending tests of the three prototypes, in terms of deformation of the chords, are shown. The following figures represent two stages of applied loads: (i) self-weight of the prototype and the spreader beams (negative values for the applied force and corresponding deformation, as indicated in the red dashed rectangle) and (ii) forces produced by the hydraulic actuator.

Some important information, regarding the behavior of the composite trussed beams, can be assessed by the experimental analysis. In all cases, both chords, at mid-section, were subjected to high levels of tension stresses and some analytical models demonstrated be able to predict the bending capacity of the structures.

Another aspect that could be attained by the experimental results refers to the behavior of the innovative shear connectors. Once both chords of the steel trusses were subjected to tension forces, it is possible to affirm that the TWP, TWC and TWVP connectors are able to provide full interaction between the steel and concrete members. In addition, the relative displacements between the top chord and the concrete slabs, which results were obtained by the transducers "HA" and "HB" (Figure 19), are negligible, indicating an insignificant tendency of slippage between both materials (maximum value approximated equal to 0.5 mm).

In Figure 22a, the obtained results, associated to the bottom chords of the TWP prototype, are indicated. The ultimate applied load (produced by the actuator) is 44.7 kN and the corresponding mechanism of collapse was related to the net rupture of the bottom chord (extensometer BC-B01). Considering the self-weight equivalent load of 24.5 kN, the ultimate total load is 69.2 kN, which is in good agreement with the prediction made by Model-4 (Table 2).

In Figure 22b, the "force *vs* deformation" relation, associated to the top chord, are represented. The maximum deformation, produced by the actuator loads and related to the extensometer TC-A02, is 1299 με. Considering a previous estimated deformation of 159 με, the total deformation is 1458 με, indicating that the top chord of the steel trusses did not achieved the yield stress (the yield deformation, indicated in Table 5, is 1989 με).

It is important to mention some problems that occurred during the positioning of the spreader beams over the TWP prototype and caused important consequences for the experimental results. Because a misalignment of the spreader beams, the TWP specimen was subjected to a significant unbalanced load distribution and, therefore, affecting the performance of the composite trussed beams.

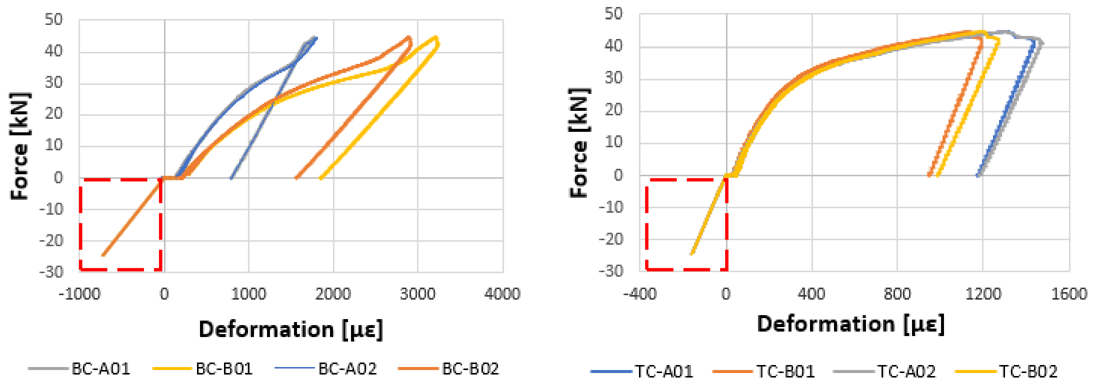

Figure 22. Force vs Deformation relation, obtained by the experimental investigation of TWP prototype: (a) bottom chord and (b) top chord

In Figure 23, the relation between applied force and deformation of both chords of the TWC prototype are shown. The obtained ultimate actuator force is 37.2 kN and the ultimate load, considering the contribution of an equivalent force of 24.5 kN, is 61.7 kN (which is well related to Model-3, as can be seen in Table 3). The mechanism of collapse of the TWC specimen was associated to the net rupture of the bottom chord of the truss "A" (extensometer BC-A01), similar to the one observed for the TWP prototype.

In Figure 23a, the deformation associated to all sensors placed at the bottom chord of the trusses presented similar results, indicating a uniform applied load distribution. In addition, considering a previous (average) deformation of 953 με, generated by the self-weight forces, the final (estimated) deformation of the bottom chord BC-A01 is 6539 με.

In Figure 23b, the obtained results indicate that the top chord of the trusses did not achieve the yield deformation of 1936 με (Table 6). Considering the previous average deformation of 177 με, the final (estimated) deformation of the top chord TB-B02 is 1486 με. The estimated axial stress is 276 MPa, assuming that the Young's Modulus is 185768 MPa (Table 6).

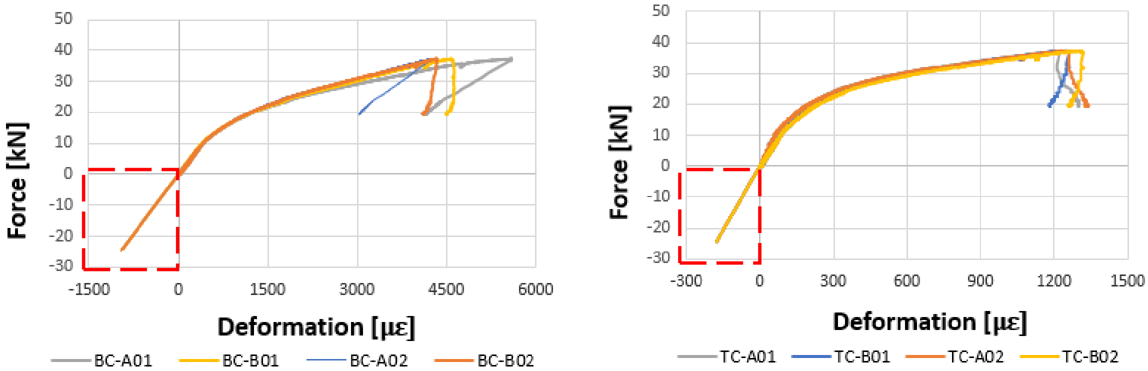

Figure 23. Force vs Deformation relation, obtained by the experimental investigation of TWC prototype: (a) bottom chord and (b) top chord

In Figure 24, the relation "force *vs* deformation" of both chords of the composite trussed beams (TWVP) are indicated. The ultimate applied force, obtained during the full-scale test, is 36.3 kN and the mechanism of collapse is related to the net rupture of the bottom chord BC-A02. Considering an equivalent self-weight load of 22.0 kN, the ultimate total load is 58.3 kN, which is good related with Model-1 (Table 4).

Observing the results related to the TWVP specimen, note that all sensors placed at bottom chords, as well as those positioned at the top chords, showed similar values for axial deformation, indicating a uniform applied load distribution.

The maximum deformation of the top chords of the composite trussed beams, generated by the actuator forces, is 1055 με. The total (estimated) deformation is 1315 με, lower than the yield deformation of 1936 με (Table 7). It was assumed a previous average deformation of 256 με.

A comparison between the experimental results of the TWVP specimen and the other prototypes reveals a similar mechanism of collapse, associated to the net rupture of the bottom chord. Additionally, even the top chords of the steel trusses did not achieve the yield stress in the instant of rupture of the prototypes, it was observed that the referred members are able to develop high levels of stresses.

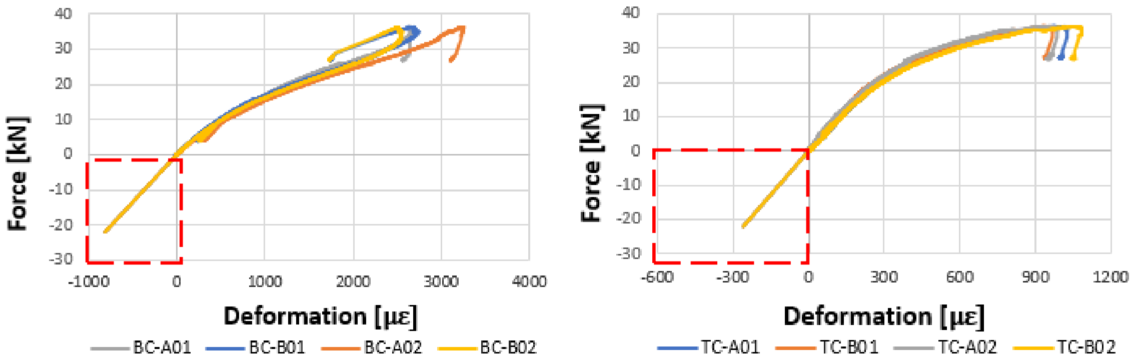

Figure 24. Force vs Deformation relation, obtained by the experimental investigation of TWVP prototype: (a) bottom chord and (b) top chord

Based on the presented experimental results, observe that the prediction of the ultimate bending capacity by means of Model-2, recommended by traditional standards (ABNT NBR 8800 [7] and ANSI/SJI 200 [9]), provides conservative values. In opinion of the authors, for trusses which the areas of cross-section are equal (or similar), the Model-4, related to the rupture of the bottom chord without taking into account the contribution of the top chord, demonstrates to be more adequate to estimate the moment capacity.

It is important to emphasize that, although the TWP prototype ultimate load was well related to the Model-3, the experimental test was significantly affected the eccentricity of the applied load, conducting the referred composite structure to premature collapse.

Another important issue refers to the prediction of bending capacity of composite trussed beams which presents high concentration of steel at the top chord. In this case, the adoption of traditional analytical procedures, which neglect the top chord, certainly is not recommended. An interesting alternative is related to the Model-1.

2.5 Axial deformation of the concrete slabs

The axial deformation of top face of the concrete slabs, measured in pure bending zone, reveals important information about the distribution of deformation (as well as stress) of the cross-section. Based on experimental results, the maximum deformations associated to the full-scale prototypes indicate that the slabs were relatively far from the collapse deformation (ε_{cu} = 3,5‰, according to ABNT NBR 6118 [13]),

In Figure 25, the "force vs deformation" relation, associated to the extensometers placed at the top face of the concrete slab, is shown. The maximum deformation, produced by the hydraulic actuator, is 908 με. Therefore, considering a previous deformation of 119.4 με, generated by the self-weight loads, the maximum total deformation is equal to 1027.4 με.

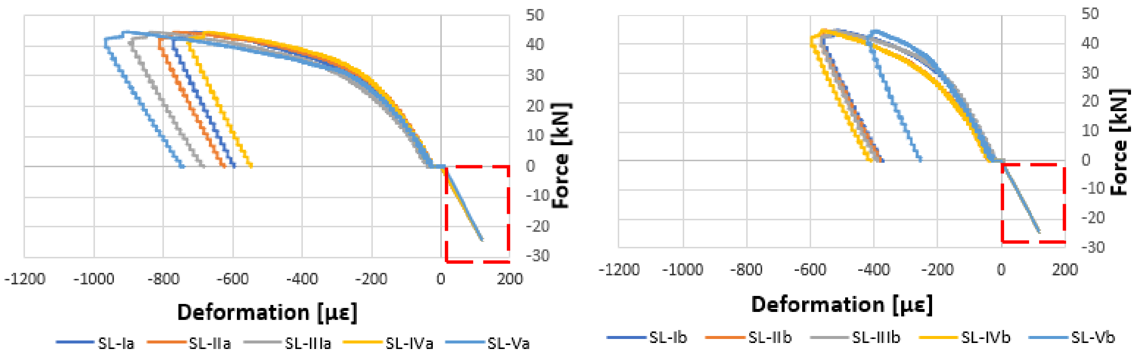

Figure 25. Force vs Deformation relation, obtained by the experimental investigation of TWP prototype: (a) SL-a and (b) SL-b

Observe, in Figure 25, that the obtained results for the TWP prototype indicate an unbalanced load distribution, probably produced by some problems during the positioning of the spreader beams, as mentioned before (for the results of axial deformation of the chords). The values of deformations related to the positions "a" and "b" (Figure 17), theoretically equal, presented significant discrepancies. The maximum deformation associated to the position "b" is 566 με, approximately 38% less than 908 με (extensometer (SL-Va).

Figure 26 shows the results, in terms of "force *vs* deformation", related to the TWC prototype. The maximum total deformation is 761.7 με (extensometer SL-Va), obtained by summing of 664 με (produced by the actuator) and 97.7 με (estimation of the effects produced by the self-weight loads). Note that the TWC prototype was subjected to a uniform applied force, once the measured deformations, related to the positions "a" and "b", are approximated equal (during the whole experimental test).

In Figure 27, the results of the TWVP prototype, regarding the "force *vs* deformation" related to the concrete slabs, is presented. The maximum total deformation is 624.4 με (SL-Ib), considering 498 με (produced by actuator force) and 126.4 με (estimated by self-weight loads). In addition, it is possible to affirm that the results, associated to the positions "a" and "b", are well related. The measured deformation of the extensometer SL-Ia (469 με), for example, is approximated 6% less than the deformation associated to SL-Ib).

Another important aspect regarding the "force vs deformation" curves, associated to the concrete slabs, refers to the estimation of compressive stresses, based on the following analytical equation, adapted from ABNT NBR 6118 [13]:

$$\sigma_c = f_c[1 - (1 - \varepsilon_c/\varepsilon_{c2})^n] \tag{1}$$

where σ_c refers to the compressive stress of the concrete; f_c is the compressive nominal strength of the concrete; ε_c is the concrete strain ; ε_{c2} is the strain associated to the compressive nominal strength (assumed to be equal to 2000 με); n is a constant, which can be taken as 2,0 for $f_{ck} \leq 50$MPa.

In this context, the estimated stresses associated to TWP, TWC and TWVP, in the instant of collapse, are 13.3 MPa, 16.1 MPa and 13.7 MPa, respectively. In all cases, as mentioned before, the concrete slab did not achieve the average compressive strength, indicated in tables 5, 6 and 7.

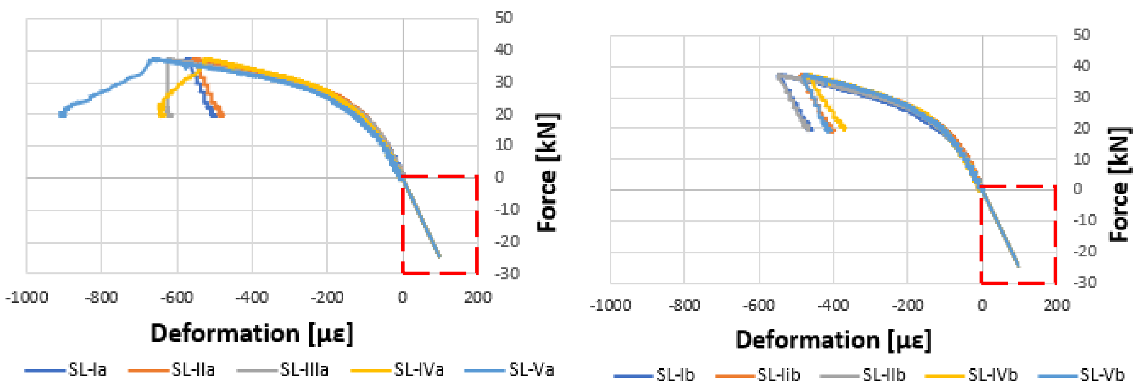

Figure 26. Force vs Deformation relation, obtained by the experimental investigation of TWC prototype: (a) SL-a and (b) SL-b

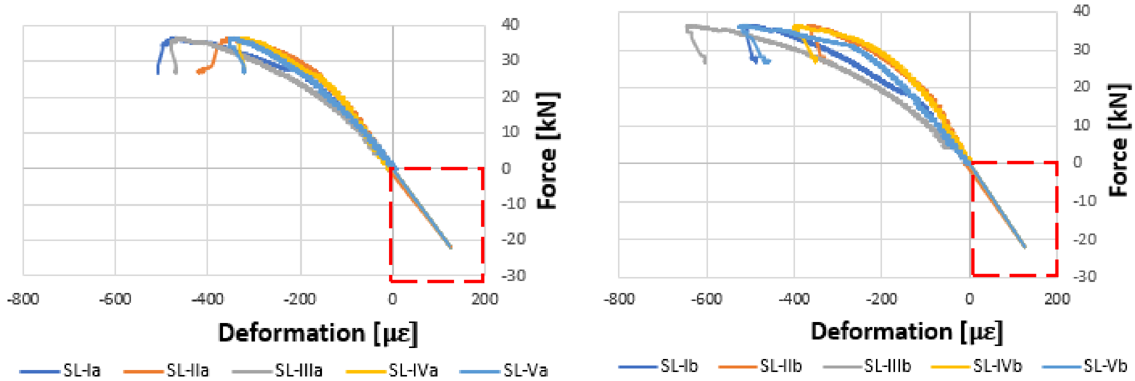

Figure 27. Force vs Deformation relation, obtained by the experimental investigation of TWVP prototype: (a) SL-a and (b) SL-b

2.6 Vertical displacements of the composite trussed beams

The vertical (transversal) displacements of the prototypes revealed important aspects regarding the behavior of the structures. In the instant of collapse, all prototypes presented high levels of displacements and, therefore, indicating adequate deformation capacity of the composite floor systems. This requirement is generally observed during design and project activities, in order to avoid the possibility of sudden rupture of the structures.

In addition, although the steel trusses without shear connectors were not subjected to the experimental investigation, a comparison between the theoretical flexural stiffness of bare steel trusses (top and bottom chord only, without the contribution of the concrete slab) and the experimental results, it is possible to affirm that the adoption of the three alternatives of shear connectors provides improved stiffness (approximated four times higher than the moment of inertia of a bare steel cross-section).

In Figure 25a, the relation between applied forces and displacements (δ) at mid-section are represented and it is possible to observe a maximum (average) deflection (considering a previous displacement at mid-section of 10.3 mm) of 87 mm, which means a ratio δ/L approximately equals to $1/87$. In Figure 25b, the deflected shape of the prototype, along the clear span (L), takes into account a self-weight load of 24.5 kN (equivalent force).

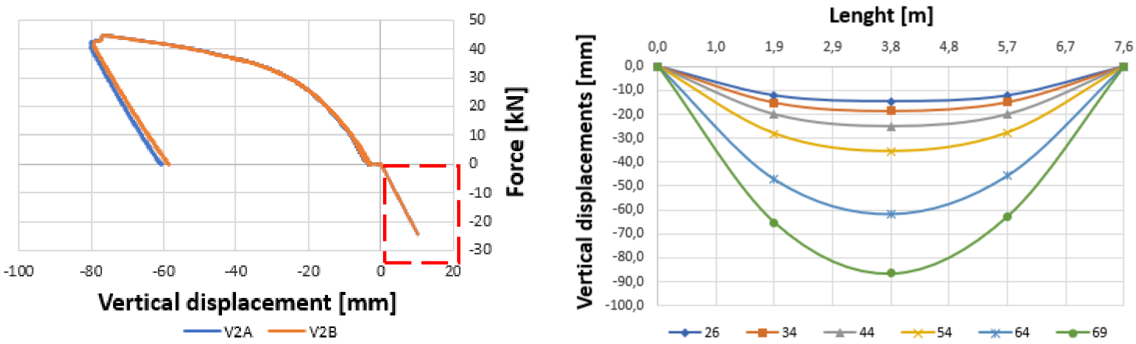

Figure 25. Vertical displacements of the TWP prototype: (a) Force vs Displacement relation and (b) Deflected shape (average) of the composite trussed beams

In Figure 26, the displacements associated to the TWC prototype is shown, indicating that, in the instant of failure, the maximum deflection is approximately 130 mm and the ratio $\boldsymbol{\delta}/\boldsymbol{L}$ equals $\mathbf{1/58}$. These referred obtained data consider the contribution of a previous displacement of 14.3 mm, produced by the self-weight of the specimen, as well as the spreader beams.

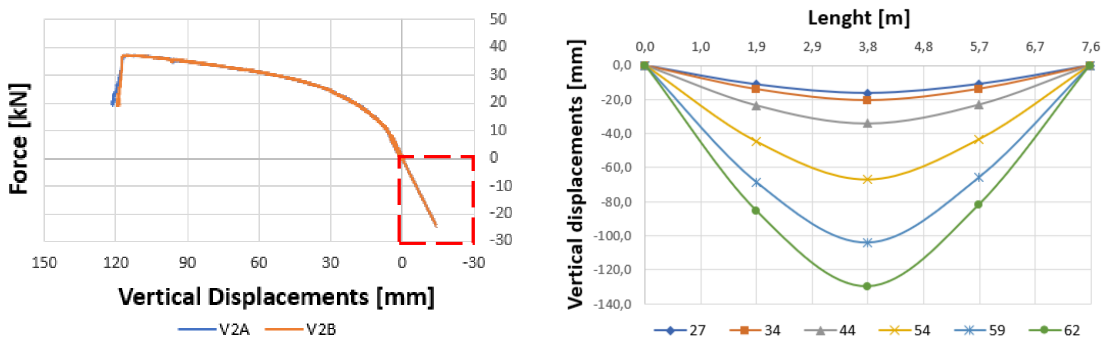

Figure 26. Vertical displacements of the TWC prototype: (a) Force vs Displacement relation and (b) Deflected shape (average) of the composite trussed beams

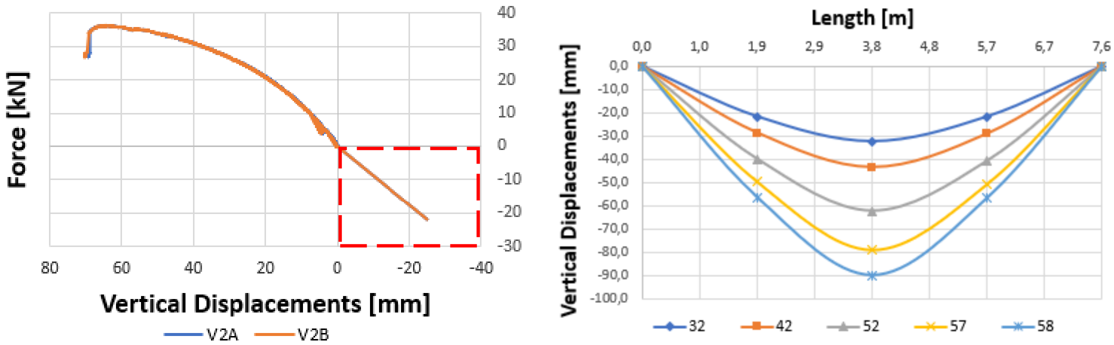

Figure 27. Vertical displacements of the TWVP prototype: (a) Force vs Displacement relation and (b) Deflected shape (average) of the composite trussed beams

Finally, Figure 27 represents the displacements associated to TWVP prototype and indicates the deformation capacity of the referred composite structure. The maximum average deflection, including the contribution of the self-weight loads, is 90,1 mm, which is related to a ratio δ/L of 1/84.

3 Conclusions

The objective of this present work involved the presentation of three new innovative solutions for shear connectors (including preliminary experimental results) and three different composite trussed beams, composed by thin-walled steel members and partially prefabricated one-way concrete slabs, subjected to full-scale bending tests. Based on the experimental results, two main aspects can be highlighted: (a) performance of the TWP, TWC and TWVP connectors and (b) behavior of the composite structure in terms of ultimate bending capacity, as well as in terms of deformation capacity (ductility).

The TWP, TWC and TWVP shear connectors, composed by thin-walled members, demonstrated be able to provide full interaction between steel and concrete, once, during the entire bending test, both chords of the trusses were subjected to tension forces. In addition, the obtained data for relative displacements between the top chord and the concrete slabs showed very small values, which can be considered negligible.

The full-scale bending tests were performed in Structures Laboratory of Federal University of Rio de Janeiro (COPPE/UFRJ) in order to evaluate the behavior of three new composite floor systems. The results indicated that the ultimate capacity of the composite trussed beams is higher than predicted by traditional analytical, recommended by ABNT NBR 8800 [7] and ANSI/SJI 200 [9]. The mechanisms of collapse, in all cases, is associated to the net rupture of the steel bottom chord at mid-section (pure bending zone).

As mentioned before, the preliminary recommendation for the prediction of bending capacity, stated by this work, is the adoption of Model-4 for steel trusses which both chords are composed by the same (or similar) cross-sectional area. In composite floor systems similar to the TWVP prototypes, which there is much more cross-sectional area at the top chord, it is recommended to use (conservatively) the Model-1 to estimate the ultimate capacity.

The connection between steel and concrete, based on the results of relative displacements between the top chord and the slab, did not achieve its ultimate capacity. Although the force/displacement curves were not represented in this work, the maximum slip between both materials was approximated equal to 0.5 mm.

Finally, the investigated prototypes demonstrated adequate deformation capacity (ductility), once large vertical displacements were observed, in all cases, in the instant of collapse. The ratios δ/L (displacements over clear span), obtained for TWP, TWC and TWVP prototypes, are 1/87, 1/58 and 1/84, respectively.

Acknowledgements

This study was financed in part by the Coordenação de Aperfeiçoamento de Pessoal de Nível Superior- Brasil (CAPES) - Finance Code 001. Moreover, the authors would like to acknowledge GypSteel Group for the delivery of the steel CFS members.

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