

## Composite hollow truss with multiple vierendeel panels

### *Treliça tubular mista com múltiplos painéis vierendeel*

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#### **Resumo**

O objetivo desse trabalho foi avaliar, através de cálculo analítico e modelagens elástica bidimensional e plástica tridimensional, a capacidade resistente e o modo de ruptura de treliças tubulares mistas biapoiadas com 15 metros de vão, variando-se o número de painéis Vierendeel centrais. O estudo apontou a proporção vão/3 - vão/3 - vão/3 como a ideal para a relação entre trechos treliçado - Vierendeel - treliçado, pois, ao se aumentar a proporção do trecho central, ocupado pelos painéis Vierendeel, os novos sistemas perdem muita rigidez, passando a não suportar mais a carga estipulada em projeto. Além disso, podem passar a apresentar deslocamentos verticais excessivos e resistência às forças cortantes externas atuantes sobre os painéis insuficiente.

**Palavras-chave:** Treliça mista, painel Vierendeel, estrutura tubular.

#### **Abstract**

*The aim of this study was to evaluate through analytical calculation, two-dimensional elastic modeling, and three-dimensional plastic modeling, the bearing capacity and failure modes of composite hollow trusses bi-supported with a 15 meter span, varying the number of central Vierendeel panels. The study found the proportion span/3 - span/3 - span/3, as the ideal relationship for the truss - Vierendeel - truss lengths, because by increasing the proportion of the length occupied by the central Vierendeel panels, the new system loses stiffness and no longer supports the load stipulated in the project. Furthermore, they can start presenting excessive vertical displacements and insufficient resistance to external shear forces acting on the panels.*

**Keywords:** Composite truss, Vierendeel panel, hollow structure.

## 1. Introduction

The composite beams of steel and concrete began to be used with stud bolt type connectors in the 1950s. They were composed of a section I filled web which worked in conjunction with a concrete slab, sometimes or not with a steel deck. However, with the need to gain larger spans, with the height limitations often imposed by zoning regulations and, as a high-headroom is normally required to allow for the passage of pipes and ducts of large diameter on full web beams, new systems of composite beams have emerged: among them, the composite beams with variable inertia, the composite beams with openings in the web (Clawson &

Darwin, 1982), the composite cellular beams, the stub girders, the composite steel joists (Tide & Galambos, 1970) and, finally, the composite trusses (Chien & Ritchie, 1984).

The composite trusses, a more efficient alternative to overcome large spans, are generally used in commercial and industrial buildings, and rail and road bridges. In many cases, in order to enable the passage of ducts, with complications in the frames with the presence of diagonals, a central Vierendeel panel is built, but in some situations, if this single panel is insufficient, then new panels need to be created to meet the intended use for the structure. In this

case, the objective of the study was to determine, through analytical calculation, two-dimensional elasticity modeling and three-dimensional plastic modeling, the bearing capacity and failure modes of a truss with a 15 meter span, and the entire central third consisting of Vierendeel panels. Then, keeping the span of 15 meters and the sections determined in the design, a parameterization of the results was made for structures having 3, 7, 9 and 13 panels. This structure, here called composite Vierendeel-truss, was designed with hollow members to present perspectives of an efficient structural solution, combining constructive strength and speed (Samarra et al., 2012).

## 2. Structural characteristics

The structure consisted of a reinforced concrete slab over a steel deck, supported by a plane vertical metal structure of parallel chords made up of welded hollow sections and multiple Vierendeel panels in the central part of the span (Figure 1). The metallic structure was designed with circular hollow section (CHS) bottom chord with a diameter of  $d=168.3$  mm and thickness of  $t = 7.1$  mm; square hollow section (SHS) top chord with a width  $b = 95$  mm and thickness

$t=6.4$  mm; and CHS brace members with a diameter of  $d=73$ mm and thickness of  $t = 6.4$  mm. The connection between the parts was made by stud bolts. For a better understanding of the work, the names of the members and frames are shown in Figure 2.

The steel deck MF-50 with a nominal thickness of 1.25 mm, as found in catalog Metform (Metform, 2010), was used together with connectors of 19 mm diameter. The total thickness used for the

slab was 110 mm and the nominal resistance to concrete and hollow steel bars were, respectively,  $f_{ck} = 25$  MPa and  $f_{yk} = 300$  MPa. The modulus of elasticity and Poisson's ratio for the steel and concrete were, respectively,  $E_s = 205000$  MPa and  $n_s = 0.3$ , and,  $E_c = 23800$  MPa and  $n_c = 0.2$ . The supports for the Vierendeel-truss were designed to simulate a bi-articulated structure, on a support device or roller to allow turning without suffering the influence of supporting links.

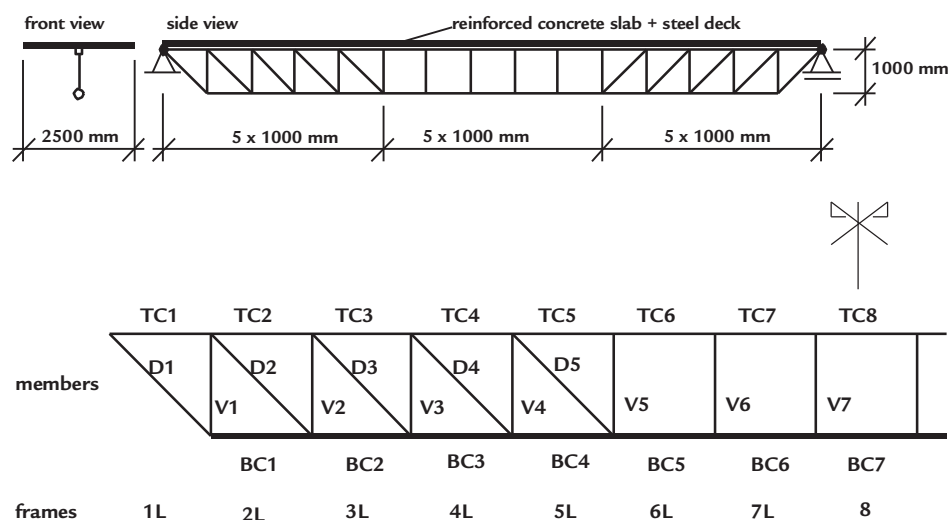


Figure 1  
Dimensions of the composite Vierendeel-truss.

Figure 2  
Names of the members and frames.

## 3. Resistance of the members, joints and stud bolts, and ultimate limit state

The Canadian Standards Association (CSA) through standard CAN/CSA-S16-01 (CSA, 2001) and the Brazilian Association of Technical Standards (ABNT) by NBR 8800 (ABNT, 2008) indicate that the design of the top chord shall be done by loading the non-composite truss, and that

the design of the bottom chord, verticals, diagonals and shear connectors must be done by loading the composite truss. The isolated steel truss was subjected to a construction load of 0.5 kN/m<sup>2</sup>, leading to a distributed load of 9.33 kN/m, and a maximum bending moment in the middle

span of 262.41 kN.m. The composite truss had been subjected to an occupancy load of 5 kN/m<sup>2</sup>, leading to a distributed load of 30.22kN/m, and a maximum bending moment at a mid-span of 849.94 kN.m. The loads were factored in accordance with the standard NBR 8800 (ABNT, 2008).

The design process firstly checked the resistance of the steel members. The safety condition to be met by welded hollow steel members subjected to the combined efforts of axial force and bend-

ing moments, loaded so that there is no torsion, is provided by Equations 1 and 2, according to NBR 8800 (ABNT, 2008), where  $N_{Sd}$  and  $N_{Rd}$  are, respectively, the design normal force and design resistance

to normal force,  $M_{x,Sd}$  and  $M_{y,Sd}$  are, respectively, the design bending moments about x and y axis, and,  $M_{x,Rd}$  and  $M_{y,Rd}$  are, respectively, the design resistances for bending moments about x and y axis.

$$\text{If } \frac{N_{Sd}}{N_{Rd}} \geq 0.2 : \frac{N_{Sd}}{N_{Rd}} + \frac{8}{9} \left( \frac{M_{x,Sd}}{M_{x,Rd}} + \frac{M_{y,Sd}}{M_{y,Rd}} \right) \leq 1.0 \quad (1)$$

$$\text{If } \frac{N_{Sd}}{N_{Rd}} < 0.2 : \frac{N_{Sd}}{2 \cdot N_{Rd}} + \left( \frac{M_{x,Sd}}{M_{x,Rd}} + \frac{M_{y,Sd}}{M_{y,Rd}} \right) \leq 1.0 \quad (2)$$

Then, the project kept the longitudinal shear in the connectors, and, the resistances of the slab and welded joints within safe limits, thus avoiding the appearance of undesirable ultimate limit states, which would lead to the composite structure breaking sharply. It was guaranteed the complete interaction between slab and upper chord leading the ultimate state achieve with the yielding of the lower chord, as desired for the case of composite trusses, but at this

time, due to the presence of Vierendeel panels, by the combination of tension and bending moments.

The calculation of the required number of connectors was carried out according to NBR 8800 (ABNT, 2008), and the installation of 48 stud bolts with a 19 mm diameter and 305 mm spacing was determined. To avoid stress concentration at the ends of the steel members, overlapping welded connections were chosen, and the performance was veri-

fied as provided by the European Committee for Standardization in Eurocode 3 - Part 1-8 (ECS, 2005), respecting all the conditions prescribed for the validity of geometric relations between hollow members. As the strength of the welds are greater than the resistance of the sections, and, assuming that the welds would be well executed at the time of construction of the steel structure, the hypothesis that they would not create a limit state was considered.

#### 4. Group resistance

The introduction of multiple Vierendeel panels gives rise to considerable bending moments located in the bottom chord of the composite truss as the shear force is transferred through the panels. Although, the bottom chord doesn't resist alone. It resists in conjunction with the performance of the top chord and with a

composite action between the slab and the top chord. This is called the group resistance, described in the works of Lawson (Lawson, 1987) and Lawson and Hicks (Lawson & Hicks, 2011).

The bending resistance due to the composite action between the slab and the top chord,  $M_{tc,s}$ , is given by Equation 3,

$$M_{tc,s} \cong n \cdot Q_{Rd} \left( \frac{t_c + h_f}{2} + x_{tc} \right) \quad (3)$$

$$M_{Rd,red} = M_{Rd,pl} \left[ 1 - \left( \frac{N_{Sd}}{N_{Rd}} \right)^2 \right] \quad (4)$$

The flexural strength in the chords may be reduced due to the effect of shear force and shall be reduced due to the presence of the axial force. The interaction between bending and axial force is complex. However, Lawson and Hicks (Lawson & Hicks, 2011) evaluated in the composite phase that when the section is compact, the reduced moments,  $M_{Rd,red}$ , can be determined from the plastic resistant bending moment,  $M_{Rd,pl}$ ,

according to Equation 4, where  $N_{Sd}$  and  $N_{Rd}$  are respectively, the design normal force and design resistance to normal force on the chords.

For the case of full slab-truss connection, the total resistance of Vierendeel local bending moments,  $M_v$ , is then calculated by summing up the moment of resistance due to the composite action,  $M_{tc,s}$ , the two portions of reduced moments on the top chord,  $M_{Rd,tc,red}$ , and

two portions of reduced moments on the bottom chord,  $M_{Rd,bc,red}$ .  $M_v$  is then compared to the applied moment  $V_{Sd} \cdot l_v$  (Inequality 5) where  $V_{Sd}$  is the design external shear force acting upon the panel, and  $l_v$  the length of the Vierendeel panel. If the bending resistance,  $M_v$ , exceeds the applied moment the project is satisfactory. If not, a heavier section must be chosen or bend stiffeners shall be introduced.

$$M_v = M_{tc,s} + 2 M_{Rd,tc,red} + 2 M_{Rd,bc,red} \geq V_{Sd} \cdot l_v \quad (5)$$

$$\delta_{max} = \frac{5 \cdot p \cdot L^4}{384 \cdot E_s \cdot I_{e,ct}} \quad (6)$$

## 5. Maximum vertical displacements

Immediate maximum vertical displacement,  $\delta_{\max}$ , of a isostatic bi-supported composite truss subjected to a uniformly distributed load  $p$  can be calculated through Equation 6, determined by the basic theory of strength of materials, where  $L$  is the span,  $E_s$  the modulus of elasticity of steel, and  $I_{e,ct}$  the effective

moment of inertia of the composite truss, which takes into account the effect of shear deformations.

The prescribed values for  $I_{e,ct}$  by the Canadian rule CAN/CSA-S16-01 (CSA, 2001) and the Brazilian standard NBR 8800 (ABNT, 2008) are shown in Equations 7 and 8, respectively.

The moment of inertia of a com-

$$I_{e,ct} = 0,85.I_{ct} + 0,1275.I_{st} \quad (7)$$

$$I_{e,ct} = I_{ct} - 0,15.I_{st} \quad (8)$$

## 6. The analytical calculation and the elastic and plastic modeling

Based on the requirements described in items 3, 4 and 5, an analytical calculation was performed leading to the determination of the structural characteristics already presented in item 2. The step-by-step process can be clearly checked in the thesis of Silva (Silva, 2013).

The sections described in item 2 were then used as input data for elastic and plastic modeling.

The two-dimensional elastic modeling was performed through the software Ftool (Martha, 2008) building up the geometry of the structure and defining to each member the material, the cross-sectional area (Figure 3) and moment of inertia. The connecting element between the upper chord and the slab was considered to be concrete, with moment of inertia calculated by a rectangular element with a width equal to the influence width,  $b_e$ , of the slab and height equal to the average width of the rib.

Modeling within the finite element method was achieved with the use of the

Ansys software version 10.0 (ANSYS INC., 2005) and the element type shell181 for all parts of the composite structure. Shell181 is suitable for analyzing thin to moderately-thick shell structures and well-suited for large strain nonlinear applications. It is a 4-node element with six degrees of freedom at each node: translations and rotations about x, y and z axes. The accuracy in modeling is governed by the first order shear deformation theory, usually referred to as Mindlin-Reissner theory of plates, which involves a constant through-the-thickness transverse shear distribution. For the element domain, reduced integration scheme was chosen (Keyopt(3) = 0), in other words, the number of points in Gauss-Legendre integration was reduced, and the same number of integration points of the stiffness matrix was adopted for the portions of shear and bending.

The non-linearity of the materials was considered by building the stress x strain diagrams for steel and concrete. The diagram for steel used in the hollow

posite truss,  $I_{ct}$ , is calculated by reducing the area of the concrete slab to an equivalent steel area.  $I_{st}$  is the moment of inertia of the non-composite steel truss. To consider the effects of creep the Canadian rule suggests a 15% increase in the value found for the initial vertical displacement of the composite truss.

members was incorporated by modeling Ansys bilinear, with material type bilinear isotropic hardening (BISO). The design diagram of the concrete (Figure 4A) was applied through the multilinear type material with isotropic hardening (MISO), and the points for the chart to plot stress x strain were determined by taking the base parabola-rectangle diagram prescribed in NBR 6118 (ABNT, 2003). For the geometry construction of the composite truss, 533 areas were utilized and 51,427 items were created during the process of generating the mesh. The types of elements used were preferably the smart sized quadrangle used in the bottom chord, diagonals and verticals, as shown in Figure 4B. However, due to analysis of geometric nonlinearity, these elements can provide maximum corner angles not allowed. In these cases, it was chosen the free mesh, obtaining triangular elements to some areas of the model, like the top chord and the supports. For the concrete slab and ribs it was decided for tri-mapped, 3 or 4 sided mesh.

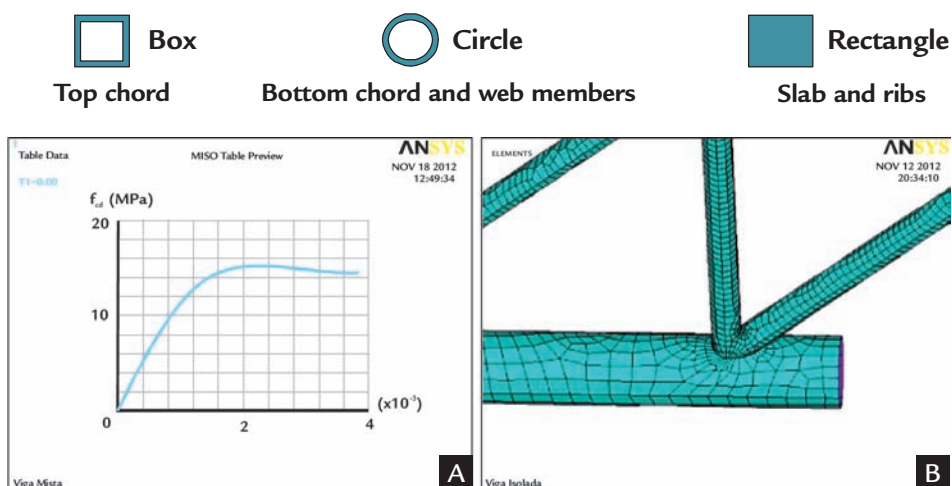


Figure 3  
Element types for two-dimensional elastic modeling.

Figure 4  
Modeling with shell181.  
(A) Concrete stress x strain curve.  
(B) Mesh in a joint between diagonal, vertical and bottom chord.

### 7. Behavior of a composite truss with five central vierendeel panels

The comparative internal forces obtained via analytical calculation and elastic modeling are shown in Table 1, and the ratio between them demonstrates that the values obtained

via analytical approach were close to the more precise process developed via elastic modeling. According to Table 2, with a loaded truss, it was found that the limit state of the composite structure

occurs simultaneously on the upper face of BC5 member (frames 6L and 6R) and on the underside of BC7 (frame 8) defined by the safety condition presented in Equation 1.

Table 1  
Bending moments and axial forces obtained via analytical and elastic modeling calculation, according to ultimate limit state of the composite beam.

Member (frame)	Analytical calculation	Elastic modeling	Ratio <sup>(1)</sup>
BC7 (frame 8)	$M_{Sd} = 12 \text{ kN.m}$	$M_{Sd} = 11.26 \text{ kN.m}$	1.066
	$N_{Sd} = +760.57 \text{ kN}$	$N_{Sd} = +722.20 \text{ kN}$	1.053
BC5 (frames 6L and 6R)	$M_{Sd} = 15.11 \text{ kN.m}$	$M_{Sd} = 11.49 \text{ kN.m}$	1.315
	$N_{Sd} = +697.13 \text{ kN}$	$N_{Sd} = +713.36 \text{ kN}$	0.977

<sup>(1)</sup> ratio obtained by dividing analytical by elastic modeling calculation.

Table 2  
Safety condition in the bottom chord of the composite beam.

Member (frame)	Design Resistance	Internal forces <sup>(1)</sup>	Safety Condition
BC7 (frame 8)	$N_{t,Rd} = 981.82 \text{ kN}$ $M_{Rd} = 50.45 \text{ kN.m}$ TC d = 168.3 mm t = 6.4 mm	$M_{Sd} = 11.26 \text{ kN.m}$	0.93 < 1.0
		$N_{Sd} = +722.20 \text{ kN}$	
BC5 (frames 6L and 6R)		$M_{Sd} = 11.49 \text{ kN.m}$	0.93 < 1.0
		$N_{Sd} = +713.36 \text{ kN}$	

<sup>(1)</sup> obtained via elastic analysis.

For the Ansys nonlinear analysis, 3 sub-steps of loading were used (80%, 90% and 100% of the ultimate load). Figure 5A shows the von Mises stresses in the deformed composite truss. When applying full imposed load throughout the span, 80-90 mm of the top face of the bar BC5 comes into flow, presenting stresses between 270 and 310 MPa (Figure 6A). At the same time, as illustrated in Figure 6B, a long stretch of the underside of the bottom chord (bars BC7 and 50% of

BC6, sides left and right) also presents stresses of this magnitude demonstrating, as expected from the design process, that the two sections would come into simultaneous yield under stresses in the order of 272.7 MPa.

The reduced moments of the top and bottom chords value, respectively, 17.64 kN.m and 25.50 kN.m. The moment due to composite action between the slab and the upper chord is 18.36 kN.m. The shear force acting on the panel is 60.44 kN.

Thus, the group resistance was checked by replacing the values in Inequality 5:  $M_v = 18.36 + 2 \times 17.64 + 2 \times 25.50 = 104.64 > 60.44 \text{ kN.m}$

The values of the immediate maximum vertical displacements calculated according to the theories proposed by CSA and ABNT were, respectively, 12.4 mm and 11.8 mm, and the values determined via elastic and plastic modeling (Figure 5B) were, respectively, 17.3 mm and 19.9 mm.

Figure 5  
Overviews of the composite beam subjected to: (A) Ultimate load. (B) Service load.

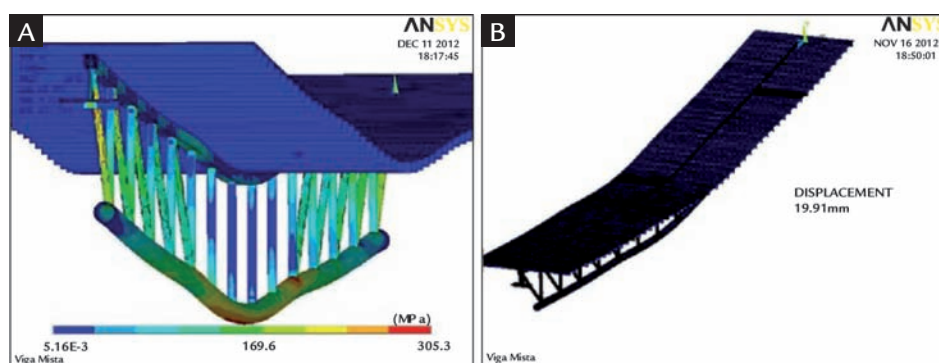
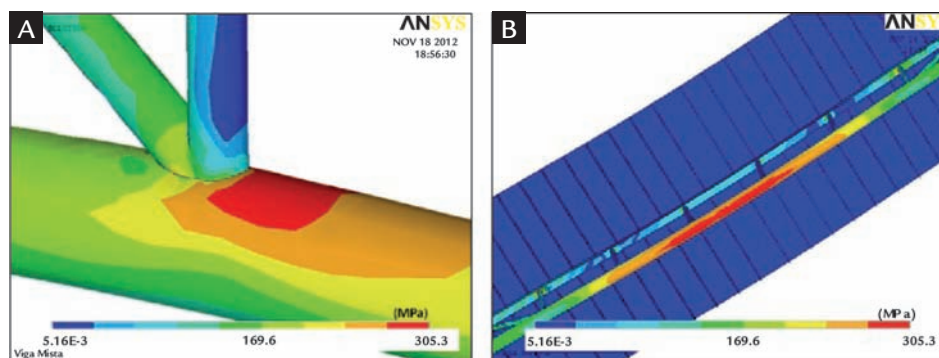


Figure 6  
von Mises stresses in the composite beam subjected to ultimate load: (A) In the upper face of the bar BC5. (B) In the underside of the bar BC7.



## 8. Behavior of the composite truss when varying the number of vierendeel panels

Assuming that the proposed analytical calculation was satisfactory, or rather, leading to the correct interpretation of the structural behavior of a composite Vierendeel-truss composed of 5 central Vierendeel panels, the method was repeated for similar trusses composed of 3, 7, 9 and 13 panels. The study applied to a truss with 3 Vierendeel panels made to bear a greater load (Table 3), but with the disadvantage of having less available panels for duct passage. The limit state occurs with the yield of the upper face

of BC6 and the composite truss is safe with respect to the Vierendeel bending moments (Table 4). When constructing 7 panels, the structure no longer supports the load stipulated in design ( $1.24 > 1.0$ ), and when the truss is fully loaded, the bar BC7 flows before it enters BC4. The composite beam is safe with respect to the Vierendeel bending moments (Table 4). When building 9 panels, the structure supports even less than the structure with 7 panels, and the bar BC7 enters flow before BC3. The group resistance is very close to the

maximum Vierendeel bending moment. With 13 panels, the structure supports a smaller load than a structure with 9 panels and once again, the member of the bottom chord located at the mid-span, BC7, enters into flow before BC1. In addition, the composite truss is not able to withstand the local bending moments acting on the Vierendeel panel.

The behavior of the composite truss with 5 panels showed a state of transition or design equilibrium, which led to simultaneous yielding of the upper and lower faces of the bottom chord.

Number of Vierendeel panels	Member and face (frame)	Bending moments and axial forces via analytical calculation	Safety condition
3	upper face BC6 (7L/7R)	$M_{sd} = 6.04 \text{ kN.m}$ ; $N_{sd} = 730.70 \text{ kN}$	$0.85 < 1.0$
	lower face BC7 (8)	$M_{sd} = 3.01 \text{ kN.m}$ ; $N_{sd} = 743.93 \text{ kN}$	$0.81 < 1.0$
5	upper face BC5 (6L/6R)	$M_{sd} = 15.11 \text{ kN.m}$ ; $N_{sd} = 697.13 \text{ kN}$	$0.98 < 1.0$
	lower face BC7 (8)	$M_{sd} = 12.00 \text{ kN.m}$ ; $N_{sd} = 760.57 \text{ kN}$	$0.99 < 1.0$
7	upper face BC4 (5L/5R)	$M_{sd} = 18.13 \text{ kN.m}$ ; $N_{sd} = 624.90 \text{ kN}$	$0.96 < 1.0$
	lower face BC7 (8)	$M_{sd} = 27.11 \text{ kN.m}$ ; $N_{sd} = 743.93 \text{ kN}$	$1.24 > 1.0$
9	upper face BC3 (4L/4R)	$M_{sd} = 24.18 \text{ kN.m}$ ; $N_{sd} = 532.32 \text{ kN}$	$0.97 < 1.0$
	lower face BC7 (8)	$M_{sd} = 48.19 \text{ kN.m}$ ; $N_{sd} = 743.93 \text{ kN}$	$1.61 > 1.0$
13	upper face BC1 (2L/2R)	$M_{sd} = 36.26 \text{ kN.m}$ ; $N_{sd} = 267.81 \text{ kN}$	$0.91 < 1.0$
	lower face BC7 (8)	$M_{sd} = 108.43 \text{ kN.m}$ ; $N_{sd} = 743.93 \text{ kN}$	$2.67 > 1.0$

Table 3  
Safety condition of the composite Vierendeel-truss, varying the number of Vierendeel central panels.

Number of Vierendeel panels	$M_v = \text{Vierendeel resistant bending moment}$	Safety condition
	$V_{sd} \times I_v = \text{bending moment}$	
3	$M_v = 18.36 + 2 \times 17.64 + 2 \times 22.51 = 98.65 \text{ kN.m}$ $V_{sd} \times I_v = 30.22 \times 1 = 30.22 \text{ kN.m}$	Secure
5	$M_v = 18.36 + 2 \times 17.64 + 2 \times 25.50 = 104.64 \text{ kN.m}$ $V_{sd} \times I_v = 60.44 \times 1 = 60.44 \text{ kN.m}$	Secure
7	$M_v = 18.36 + 2 \times 17.64 + 2 \times 30.02 = 113.67 \text{ kN.m}$ $V_{sd} \times I_v = 90.66 \times 1 = 90.66 \text{ kN.m}$	Secure
9	$M_v = 18.36 + 2 \times 17.64 + 2 \times 35.62 = 124.88 \text{ kN.m}$ $V_{sd} \times I_v = 120.88 \times 1 = 120.88 \text{ kN.m}$	Limit
13	$M_v = 18.36 + 2 \times 17.64 + 2 \times 46.70 = 147.04 \text{ kN.m}$ $V_{sd} \times I_v = 181.32 \times 1 = 181.32 \text{ kN.m}$	Not secure

Table 4  
Group resistance of the composite Vierendeel-truss, varying the number of Vierendeel central panels.

## 9. Vertical displacements in function of quantity of vierendeel panels in the composite truss

The vertical displacement limit allowed for a floor beam, according to NBR 8800 (ABNT, 2008) is span/350 therefore, 42.9 mm. The truss, with none or with one Vierendeel panel, taking into account the effects of creep estimated at 15% of

initial displacement, can be constructed with or without cambering (Table 5). The ratio of the immediate values found via elastic modeling and through procedure proposed by NBR 8800 (ABNT, 2008) is  $r_{d,st} = 1.016$  (19.4mm / 19.1 mm) for

the non-composite truss and  $r_{d,ct} = 1.186$  (14.0mm / 11.8 mm) for the composite truss (Table 6), or also, the presence of a panel or none, makes for a correct prediction of the elastic maximum vertical displacement of the non-composite and

composite trusses should be made by multiplying the value found by means

ABNT for 1.016 and 1.186, respectively (Figure 7). Similar values were presented

for the case of trusses with 3, 5, 7, 9 and 13 panels.

Number of Vierendeel panels	$\delta_{\max,i,st}^{(1)}$	$\delta_{\max,i,ct}^{(2)}$	$\delta_{\text{tot},ct}^{(3)}$	$\delta_{\text{tot}}^{(4)}$	$\delta_{\text{tot},c}^{(5)}$
None or 1	19.4 <sup>(6)</sup>	14.0	16.1	35.5<42.9	16.1<42.9
3	20.0	14.7	16.9	36.9<42.9	16.9<42.9
5	23.0	17.4	20.0	43.0>42.9	20.0<42.9
7	29.4	23.6	27.1	56.5>42.9	27.1<42.9
9	39.9	34.7	39.9	79.8>42.9	39.9<42.9
13	81.0	77.0	88.6	169.6>4.29	88.6>42.9

Table 5  
Maximum vertical displacements, varying the number of Vierendeel central panels.

<sup>(1)</sup> initial or elastic vertical deflection in the non-composite steel truss. <sup>(2)</sup> initial vertical deflection in the composite truss. <sup>(3)</sup> total vertical deflection in the composite truss, obtained by multiplying  $\delta_{\max,i,ct}$  by 1.15. <sup>(4)</sup> total deflection of the composite truss not providing cambering. <sup>(5)</sup> total deflection of the composite truss providing cambering equal to the elastic deflection of the non-composite steel truss. <sup>(6)</sup> values in millimeters obtained via elastic modeling.

Number of Vierendeel panels	Non-composite Vierendeel-truss		Composite Vierendeel-truss	
	$\delta_{\max,i,st}^{(1)}$	$r_{d,st}^{(2)}$	$\delta_{\max,i,ct}^{(1)}$	$r_{d,ct}^{(3)}$
None or 1	19.4	1.016	14.0	1.186
3	20.0	1.047	14.7	1.246
5	23.0	1.204	17.4	1.475
7	29.4	1.539	23.6	2.000
9	39.9	2.089	34.7	2.941
13	81.0	4.241	77.0	6.525

Table 6  
Ratios between the values of maximum initial vertical displacements found via elastic modeling and calculated by NBR 8800 (ABNT, 2008), in the steel ( $r_{d,st}$ ) and composite ( $r_{d,ct}$ ) Vierendeel-truss.

<sup>(1)</sup> values in millimeters obtained via elastic modeling.  
<sup>(2)</sup>  $r_{d,st} = \frac{\delta_{\max,i,st}(ftoll)}{\delta_{\max,i,st}(ABNT)}$       <sup>(3)</sup>  $r_{d,ct} = \frac{\delta_{\max,i,ct}(ftoll)}{\delta_{\max,i,ct}(ABNT)}$

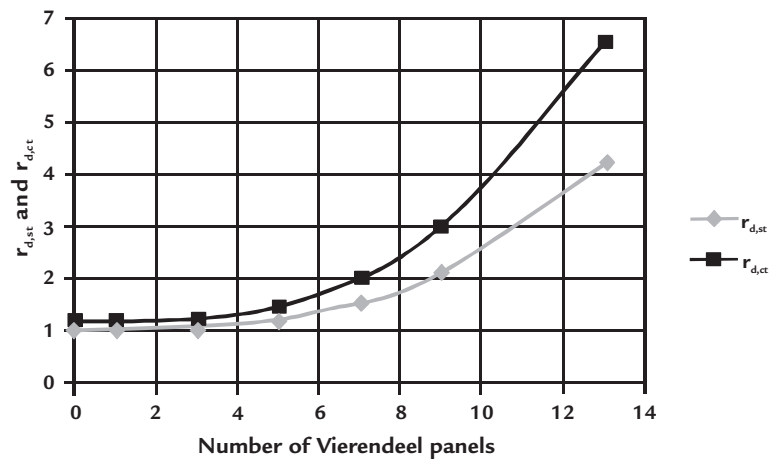


Figure 7  
 $r_{d,st}$  and  $r_{d,ct}$  x number of Vierendeel panels diagram.

## 10. Conclusions

The proposed method for the analysis of a composite truss bi-supported with a 15 m span, consisting of Vierendeel panels in the middle third position, showed that the ultimate limit state occurs by combination of the tension and bending moment, simultaneously at the underside of the BC7 member and upper face of BC5, provided the horizontal shear in the connectors and the resistances of the slab, steel members and joints are maintained within safe limits. When increasing the space where the

panels are arranged, the structure is no longer capable of supporting the maximum bending moment required in the project, showing in some cases vertical displacements above the limit specified in the NBR 8800 standard and insufficient group resistance. Thus, the study found the proportion span/3 - span/3 - span/3 is the ideal relationship between the stretches trusses - Vierendeel - trusses, because the composite structure with 5 panels showed a transition state, or project balance, where there occurs a si-

multaneous yield of the upper and lower faces of the bottom chord. The values of elastic maximum vertical displacements provided by equating for composite trusses were not suitable for use in a composite truss with multiple Vierendeel panels. The modeling via Ftool and Ansys softwares demonstrated that as the number of Vierendeel panels increases, the structure becomes less rigid, and the analytical values move away from that obtained by modeling, supposed to be more realistic.

## 11. References

- ANSYS INC. *Ansys version 10.0 - basic analysis procedure*. Houston, PA, United States: 2005.
- ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS (ABNT). *NBR 6118 - Projeto de estruturas de concreto - procedimento*. Rio de Janeiro: ABNT - Associação Brasileira de Normas Técnicas, 2003.
- \_\_\_\_\_. *NBR 8800 - Projeto de estruturas de aço e de estruturas mistas de aço e concreto de edifícios*. Rio de Janeiro: ABNT - Associação Brasileira de Normas Técnicas, 2008. 237p.
- CANADIAN STANDARDS ASSOCIATION (CSA). *CAN/CSA-S16-01 Limit states design of steel structures*. Mississauga, Ontario: 2001. 167p.
- CLAWSON, W. C., DARWIN, D. Strength of composite beams at web openings. *Journal of the Structural Division - ASCE*, v. 108, n. ST3, p. 623-641, mar. 1982.
- CHIEN, E. Y. L., RITCHIE, J. K. *Design and construction of composite floor systems*. Canadian Institute of Steel Construction, 1984. 323p.
- EUROPEAN COMMITTEE FOR STANDARDIZATION (ECS). *EN 1993-1-8:2005. Eurocode 3: Design of steel structures – Part 1-8: design of joints*. Brussels: 2005. 135p.
- LAWSON, R. M. *Design for openings in the webs of composite beams – SCI-P-068 – CIRIA Special Publication 51*. Ascot, UK: Steel Construction Institute, 1987. 44p.
- LAWSON, R. M., HICKS, S. J. *Design of composite beams with large web openings – SCI-P-355*. Ascot, UK: The Steel Construction Institute, 2011. 116p.
- MARTHA, L. F. *Ftool - two-dimensional frame analysis tool - versão educacional 2.12*. Rio de Janeiro: Pontifícia Universidade Católica do Rio de Janeiro. 2008. Disponível em: <<http://www.tecgraf.puc-rio.br/ftoll>>. Acesso em 19 abr. 2008.
- METFORM. *A solução definitiva em lajes*. Betim: 2010. 8p.
- SAMARRA, F. A., REQUENA, J. A. V., JÚNIOR, N. O. P. Análise numérica-experimental de treliças tubulares planas para coberturas padronizadas para grandes vãos. *REM - Revista Escola de Minas*, v. 65, n. 4, p. 459-466, 2012.
- SILVA, A. O. B. *Treliça tubular mista com múltiplos painéis Vierendeel*. Campinas: Faculdade de Engenharia Civil, Arquitetura e Urbanismo, Universidade de Campinas, 2013. 339p. (Tese de Doutorado).
- TIDE, R. H. R., GALAMBOS, T. V. Composite open-web steel joists. *Engineering Journal (American Institute of Steel Construction)*, v. 7, n. 1, p. 27-36, 1970.

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