Analysis of cold-formed steel truss and prefabricated concrete slab composite floor system

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Abstract. Steel and concrete composite floor systems improve the resistance capacity and flexural stiffness. Studies on composite floor of buildings available in the literature consider, for the most part, open steel cross sections and solid or steel deck reinforced concrete slabs. The present work is based on the results obtained in an experimental campaign carried out by the research group on steel and composite construction at the Laboratory of Structures and Materials from COPPE /UFRJ, and proposes the validation of a numerical model based on the Finite Element Method (FEM), in order to improve the knowledge of the structural behavior of the construction system. This work aims to extend the original investigation of the composite structural system proposed and tested by Luiz Alberto Leal (doctoral thesis in 2019), associating lattice girder in cold-formed steel members (CFS) and composite unidirectional prefabricated concrete slab, for which there are few numerical and experimental studies. In this context, the present work performs the FEM-based numerical modeling, calibrated with the help of experimental results, consisting of CFS trussed girders and unidirectional slabs, combined with innovative thin-walled perfobond (TWP) shear connectors.

Keywords: Composite structures, Numerical analysis, Shear connector, Cold-formed steel members.

1 Introduction

Steel and concrete composite trusses for floor systems have been gradually accepted as technically and economically viable solutions for structural design, especially in spans larger than 10 meters (Machacek and Cudejko) [1]. In Eurocode EM 1994-2:6.6.2.3 [2], there is no particular recommendation for the design of composite trusses, except for the formulas for the push-out test, for the measurement of the effectiveness of the connection between the steel section and the concrete slab. The ABNT NBR 8800:2008 [3] deals with composite beams with steel truss associated with concrete slab, highlighting the models for calculating the bending moment strength, without considering the contribution of the upper chord in the resistant capacity of the composite truss. For composite trussed beams with longer spans, the shear connectors with higher strength are more suitable and the perforbond shear connector can be recommended (Machacek and Cudejko) [4] (Ahn, Kim, and Jeong) [5], because of its better performance if compared with stud bolt shear connector. The design equations available to calculate the resistance of perfobond connectors, as well as the resistance of composite trusses, are mainly based on experimental research. Although the Finite Element Method (FEM) is widely applied, the results obtained are sensitive to many factors, such as the definition of geometry and material, type of element and mesh selected, applied loads, boundary conditions, type of analysis, among others. The usual composite floor systems associate hot rolled and welded I sections or trusses (composed of either tubular or angle members) with the steel deck concrete slab. However, the use of trusses composed by cold-formed members in the floor system allows an efficient and considerably light structural composition, named as lightweight floors. This is due to the increase of the floor stiffness and strength capacity, together with the restraint of the local, distortional and global buckling of the compressed chord by the contact of the CFS member with the reinforced concrete slab. In addition, the original investigation of Leal proposed innovative thin-walled shear connectors, as the one considered on the present study and illustrated in Figure 1, the Thin-Walled Perfobond TWP shear connector.

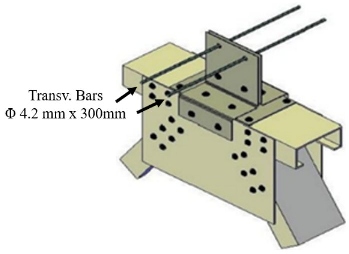

Figure 1. The TWP Thin-Walled Perfobond connected to the lipped channel CFS of the upper chord of the truss (Leal [6]).

In order to complement the studies on the behavior of the proposed composite floor system, this work is based on the referred experimental results and proposes a numerical model based on the FEM with the objective of improving the structural understanding of the construction system.

2 The tested composite floor structure

The experimental full scale prototype tested by Leal [6] is composed of steel trusses with lipped channel members (U 89x40x12x1.25mm), manufactured with ZAR-345 steel (fy=345 MPa), eccentric joints at the diagonal-chord nodes. The connections applied 4.8 mm self-drilling screws. The partially prefabricated unidirectional concrete slab is 130 mm thick, with 60 mm of top layer of reinforced concrete and 70 mm thickness of EPS blocks. The concrete compressive strength resulted in 16.1 MPa (the experimental results showed that higher strength of the concrete was unnecessary). Figure 2 illustrates the experimental set up and the cross section of the composite floor system.

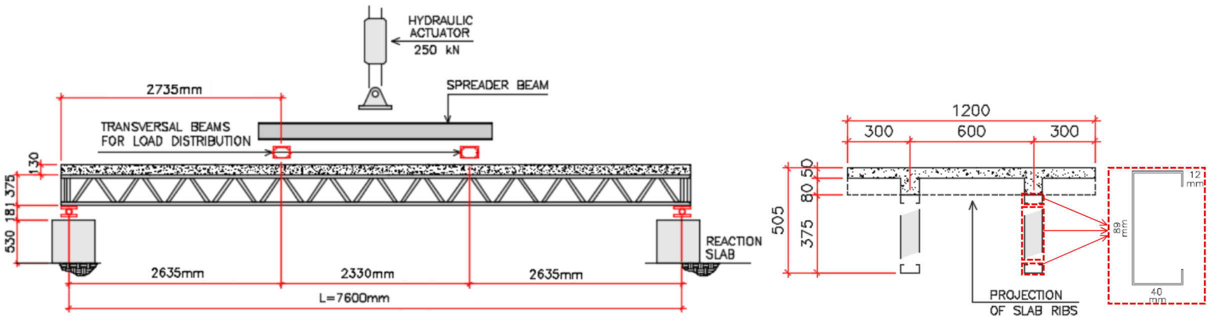

Figure 2. (a) Experimental set up of the four points bending test; (b) cross section of the floor system, dimensions in mm (Leal [6]).

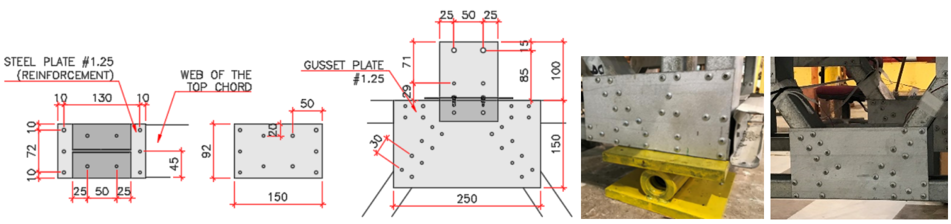

Figure 3. (a) The TWP shear connector; (b) detail of the CFS truss at the support; (c) typical joint of the truss with gousset plate and self-drilling screws (Leal [6]).

The loading was performed in 5 previous loading-unloading steps between 1.5 and 20.0 kN, followed by loading until collapse. The hydraulic actuator operated in displacement control and the instrumentation included (i) displacement transducers for vertical displacements and relative displacements between the truss and the slab and (ii) strain gages in the steel truss and the concrete slab.

Finally, the steel truss was designed with connecting gousset plates as shown in Figure 3, with the objective of reinforcing the connections and preventing premature localized collapse of the structure.

3 Finite element model

The SAP2000 FEM computational package was applied. The FEM model was developed with the combination of frame members for the trussed steel and shell or solid finite elements for the concrete slab. The main challenge is to include the TWP shear connectors, which can be modeled as fully rigid elements with the help of (i) stiff bar members and (ii) ideally rigid springs. The concrete slab was taken with its 60 mm thick top layer, disregarding the lower 70 mm EPS part of the slab, as well as the concrete volume positioned over the top chord of the trusses. This consideration can be considered acceptable since the experimental records showed the neutral line inside the top layer of the slab. For the convergence study of the finite element mesh of the slab (1.2 m x 7.8 m and 60 mm thick), 5 models were created with discretization shown in Figure 4a. The appropriate level of refinement of the finite element mesh started with 30 cm x 30 cm shell elements, with gradual refinement (Slab 1 (30x30), slab 2 (20x17), slab 3 (10x9), slab 4 (5x4.5) and slab 5 (2.44x2.14)) and based on the vertical displacements in control points located in the middle of the span. Figure 4b shows the results of displacements in the center of the slab of the models described above for the different discretization.

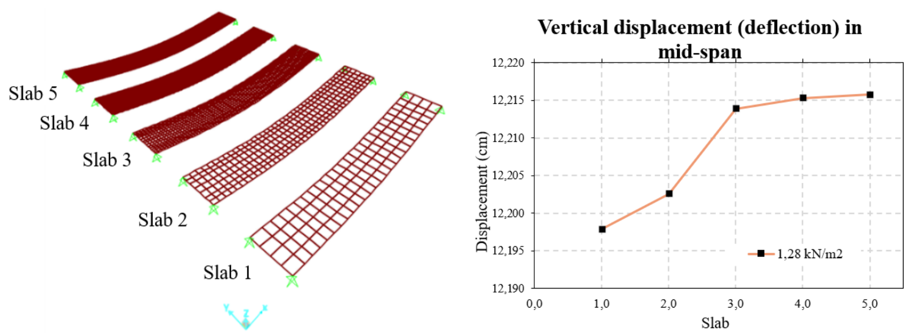

Figure 4. (a) The 5 models of the slab discretization; (b) the convergency results of the vertical displacement at the center of the slab.

According to the results observed for the displacements of the slabs with the different discretization models, slab 5, with mesh of 2.44 cm x 2.14 cm presented a difference of 0.004% compared with the previous mesh of the slab 4.These results validate the choice of the slab 5 in the present study and Shell mesh calibration will also be used for solid elements. The proposed FEM model included both shell and solid finite elements for the concrete slab, besides investigating the sensitivity of the model with the consideration of the contact between the slab and the upper chord. In the following, five model strategies of the numerical model are described.

Strategy 1: Modeling the elements of the CFS truss and the TWP connector as bar frame elements, whereas the TWP connector connects the coincident node of the upper chord to that of the middle plane of the 60 mm thick concrete upper layer of the slab. The slab is taken with 60 mm thick shell element. The truss geometry considers a positive nodal eccentricity.

Strategy 2: The modeling considerations for the truss and the TWP connector are the same as in strategy 1. However, the slab is modeled with solid elements. In strategies 1 and 2 there is no consideration of a contact element between the slab and the upper chord, since the degrees of freedom of translation and rotation of the nodes between the slab and the truss were compatible via nodal coupling.

Strategy 3: The modeling considerations of the truss elements, TWP connector and slab are equivalent to those of strategy 2. However, the frictionless contact between the upper chord and the concrete slab was considered, through the use of the Finite Gap element.

Strategy 4: The arrangement of the trusses considers the gousset plates in the reinforced connection of the diagonal-chord nodes, without positive eccentricity between the diagonals and the chords. Besides this, strategy 3 considerations are taken.

Strategy 5: The TWP connector base nodes, which coincided with both the metal profile nodes and the slab base nodes, were replaced by 3 ideally rigid springs (slack element), connecting the slab to the cold formed profile. Such a method is proposed to David [7].

Figure 5 illustrates the SAP2000 finite elements applied in the described strategies and Figure 6 illustrates the three-dimensional geometry of the above-mentioned models.

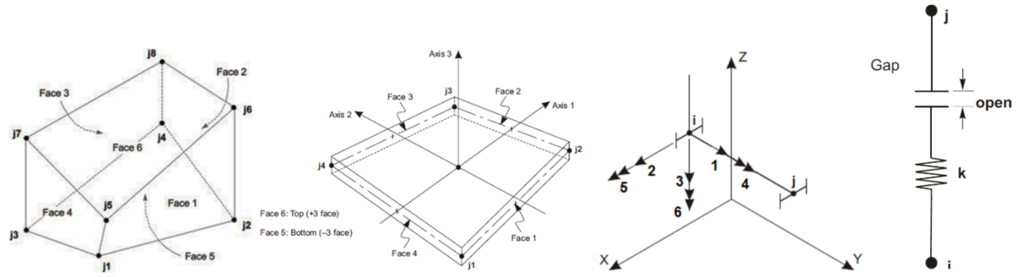

Figure 5. Applied finite elements: (a) solid; (b) shell; (c) frame and (d) gap (SAP2000 [8]).

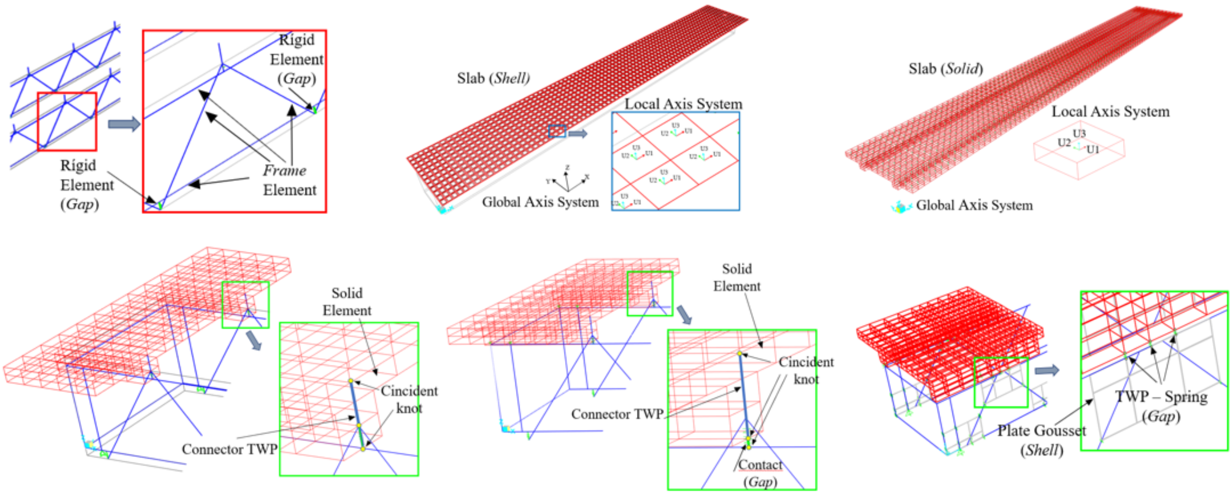

Figure 6. Illustration of the adopted FEM model strategies: (a) eccentricity at the joints of the truss; (b) shell slab; (c) slab with solid elements; (d) strategy 2; (e) strategies 3 and 4; (f) strategy 5.

4 Vertical displacements of the struture

Figure 7 shows the comparison between (i) the results of the adopted computational strategies, (ii) the design recommendations of (ii1) the ABNT NBR 8800, (ii2) Murray's studies (1997) and (ii3) the Canadian standard CAN/CSA with the experimental results. These results are related to the force versus vertical displacement at the central region of the floor structure, highlighting the limit of the elastic behavior presented in the experimental model (dashed line).

Table 1 presents the numerical, analytical and experimental results, including the relative difference between the results obtained in the present study and the test.

It is noticed that the numerical results of strategy 1 were close to the results obtained through the Canadian standard, which considers full interaction disregarding the contribution of the upper chord and reducing by 15% the moment of inertia of the steel truss. In relation to the comparison of the strategy 1 with the the experimental study, it is possible to verify that up to 15 kN the mean percentage difference of the results of strategy 1 is 9%.

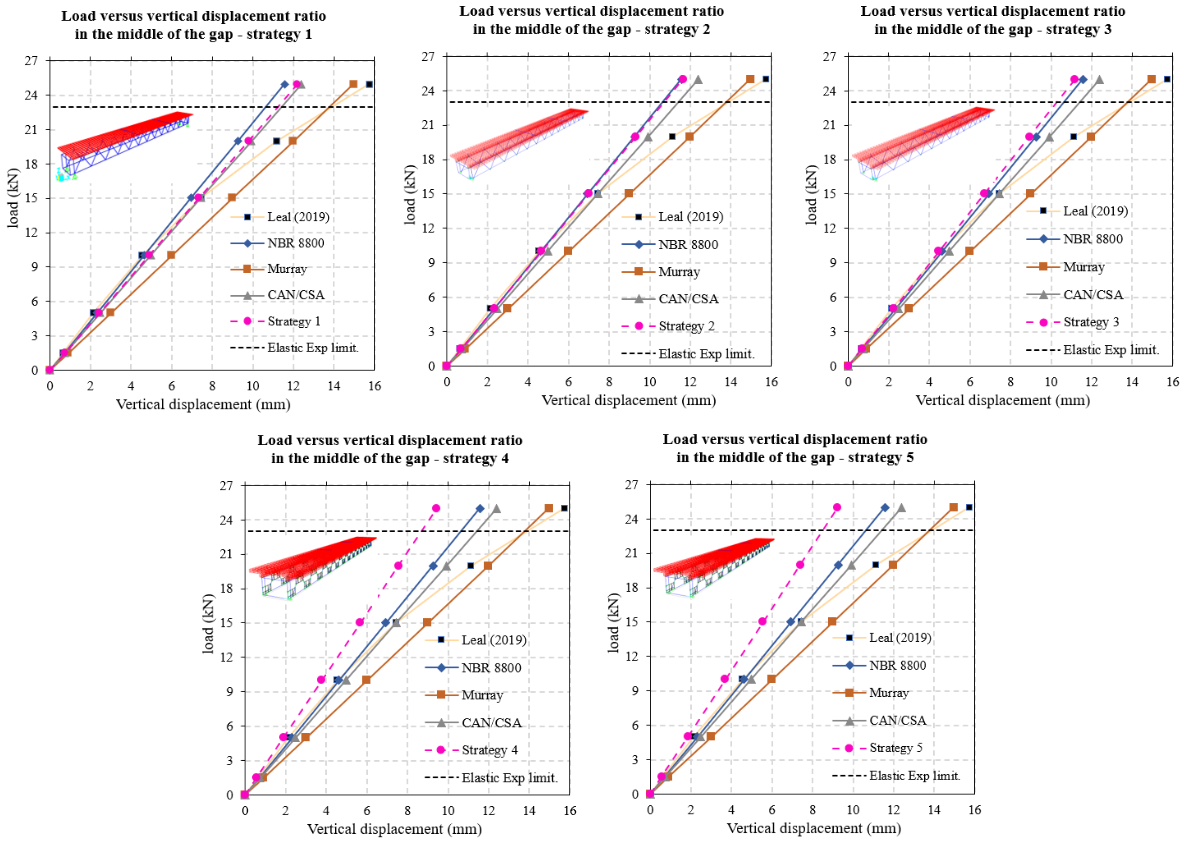

Figure 7. Applied force vs. mid span vertical displacement according with FEM model strategies (a) 1; (b) 2; (c) 3; (d) 4 and (e) 5; and the computed results of the analytical design rules from the Brazilian code ABNT NBR 8800, Murray (1997) and the Canadian code CAN/CSA.

Table 1. Comparison between the vertical displacements computed with the FEM models and the analytical design rules with the experimental results from Leal (2019). Applied force up to 25.0 kN.

Load	Displacement (mm)								
(kN)	Leal (2019)	NBR 8800	Murray	CAN/CSA	Strategy 1	Strategy 2	Strategy 3	Strategy 4	Strategya 5
0,00	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)	0,00 (0%)
1,50	0,64 (0%)	0,69 (-8%)	0,90 (-40%)	0,74 (-16%)	0,73 (-14%)	0,70 (-9%)	0,67 (-4%)	0,57 (12%)	0,55 (14%)
5,00	2,14 (0%)	2,31 (-8%)	2,99 (-40%)	2,48 (-16%)	2,44 (-14%)	2,33 (-9%)	2,24 (-4%)	1,89 (12%)	1,85 (14%)
10,00	4,54 (0%)	4,62 (-2%)	5,99 (.32%)	4,96 (-9%)	4,88 (-8%)	4,66 (-3%)	4,47 (2%)	3,78 (17%)	3,70 (19%)
15,00	7,44 (0%)	6,94 (-7%)	8,98 (-21%)	7,44 (0%)	7,33 (2%)	6,99 (6%)	6,71 (10%)	5,67 (24%)	5,55 (25%)
20,00	11,14 (0%)	9,25 (-17%)	11,98 (-8%)	9,92 (11%)	9,77 (12%)	9,32 (16%)	8,94 (20%)	7,56 (32%)	7,40 (34%)
25,00	15,74 (0%)	11,56 (-27%)	14,97 (5%)	12,40 (21%)	12,21 (22%)	11,66 (26%)	11,18 (29%)	9,44 (40%)	9,25 (41%)

For strategy 2 (which included to match the nodes of the solid type elements of the slab with the nodes of the frame element, referring to the upper chord) it is possible to verify an approximation with the results obtained according to the prescription of NBR 8800 [3], which it does not take into account the participation of the upper chord in the analysis of composite behavior. However, the Brazilian standard considers only shear connectors as stud bolt, hot rolled U and cold-formed element with plate thickness equal to or greater than 3 mm, which is not in accordance with the tested prototype. This strategy has an average percentage difference with the results of the experimental study of 7%, up to the load of 15 kN.

With the consideration of the contact between the slab base and the upper chord, represented by the Gap element included in strategy 3, the difference between the numerical and experimental results became 5% up to the load of 15 kN. This consideration may be taken as satisfactory.

It is noticed that the introduction of gousset plates in the numerical model (strategies 4 and 5) adds stiffness to the structural system, resulting in lower displacements compared with the experimental test results. Moreover, there was no significant difference by changing the shear connector model, so the stiff bar model satisfactorily meets the TWP connector behavior. Therefore, it is possible to conclude that, in terms of vertical displacement, strategy 3 was the numerical model with the best approximation with the experimental results. In addition, it was observed that the design prescriptions from the Brazilian and the Canadian standards are in acceptable accordance with the experimental results.

5 Strength behavior of the composite floor system

The analyses performed in the numerical study were based on the elastic linear behavior of the material, thus, the analytical models proposed by Leal [6] for predicting the position of the neutral line and the strength capacity of the composite trussed beams with TWP connector, are based on the linear distribution of stresses and forces in the cross section, combined with design strength mechanism of the CFS truss and the concrete slab. The analytical models applied in this analysis will be the elastic linear models, with or without the contribution of the upper chord to the resistance of the mixed cross section. Figure 8 illustrates the analytical models mentioned above. Table 2 presents the comparison between the results of the analytical and numerical models with the experimental ones, in relation to the depth of the neutral line and the bending moment strength, referring to the intermediate region (at 3318mm from the support) and 3 for the central region (at 3800mm from the support). the values of $\Delta 1$ and $\Delta 2$ indicate the percentage difference in relation to models 1 and 2, respectively.

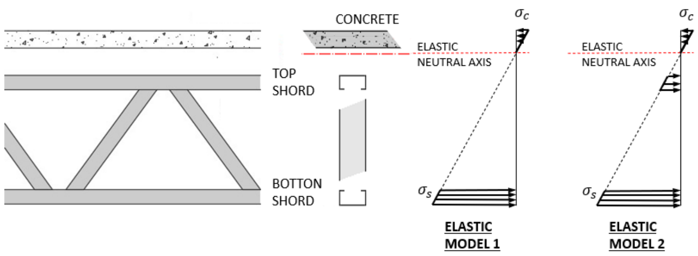

Figure 8. Analytical models for the computation of the strength capacity of the composite trussed beam system (Leal [6]).

Table 2. Comparison between analytical and experimental results of the neutral line position and the bending moment strength, related with the intermediate region.

Intermediate region (at 3318mm from the support)										
Theoretical Neutral Line		Neutral Line					Resistant Moment			
Elastic Model - 1	Elastic Model - 2	model	hNL	Load	Δ1	Δ2	Mr	Load	Δ1	Δ2
(mm)	(mm)	-	(mm)	(kN)	-	-	[kN.cm]	(kN)	-	-
57,64	62,46	TWP	54,01		6%	14%	7403,49		7%	14%
Theoretical Resistant Moment		1	46,72		19%	25%	5738,76		28%	34%
		2	65,06		13%	4%	6549,81		18%	24%
Model - 1	Model - 2	3	60,28		5%	3%	6413,32		20%	26%
[kN.cm]	[kN.cm]	4	59,58		3%	5%	6702,22		16%	23%
7996,16	8651,70	5	52,09	20	10%	17%	6608,64	20	17%	24%

Regarding the position of the elastic neutral line of model 1, strategies 3, 4 and 5 present the lowest values of $\Delta 1$, in the intermediate region. In the central region the lowest $\Delta 1$ values are identified for strategies 2 and 5. For the prediction of the depth of the elastic neutral line, according to the result of model 2, the results with satisfactory difference $\Delta 2$ are found in strategies 2, 3 and 4, for both in the intermediate and central region. The results of the bending strength from the numerical models showed the lowest values of $\Delta 1$ for strategies 5 and 6 in

Table 3. Comparison between analytical and experimental results of the neutral line position and the bending moment strength, related with the Central region.

Central region (at 3800mm from the support)										
Theoretical Neutral Line		Neutral Line					Resistant Moment			
Elastic Model - 1	Elastic Model - 2	model	hNL	Load	Δ1	Δ2	Mr	Load	Δ1	Δ2
(mm)	(mm)	-	(mm)	(kN)	-	-	[kN.cm]	(kN)	-	-
57,64	62,46	TWP	63,19		10%	1%	10007,17		25%	16%
Theoretical Resistant Moment		1	46,72		19%	25%	5738,76		28%	34%
		2	65,06		13%	4%	6549,81		18%	24%
Model - 1	Model - 2	3	66,95		16%	7%	6159,31		23%	29%
[kN.cm]	[kN.cm]	4	66,71		16%	7%	11131,57		39%	29%
7996,16	8651,70	5	65,84	20	14%	5%	11181,78	20	40%	29%

the intermediate region. In the central region, strategy 2 obtained the lowest $\Delta 1$ value. The values of the numerical models presented for $\Delta 2$ in the intermediate region, for resistant moment, did not indicate values lower than 23% and 24% in the central region.

6 Final remarks

The results of the displacements of the numerical models indicate that strategy 3 is satisfactorily calibrated in relation to the experimental study. The design procedures of the Brazilian and the Canadian standards present acceptable results compared with the test records. Regarding the results of the models of strategies 5 and 6, it is clear that, in terms of stiffness, the *gousset* plates turns the numerical model more stiff and, in addition, the different models of the TWP connector does not bring important variation of the numerical results. The analytical formulations presented in Figure 7 indicate that model 2 is well correlated with numerical and experimental models for predicting the depth of the neutral line. Regarding the bending strength, model 1 showed acceptable approximation with the experimental results, and the same conclusion was found for the FEM model, strategies 1, 2 and 3.

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