A simple computational tool for the verification of concrete walls reinforced by embedded steel profiles.

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SUMMARY: (10 pt)
Composite steel-concrete structural shear walls with steel encased profiles are commonly used as horizontal resisting system for buildings requiring an important horizontal resistance, like for instance buildings to be designed to resist important seismic horizontal accelerations. This type of structural sytem is currently not explicitly addressed in usual design codes, like Eurocodes or UBC. The paper aims at presenting a simple verification tool implemented in a spreadsheet format and aiming at an easy calculation of the interaction diagrams between the axial force and the bending moment that are commonly used to represent the resistance capacity of a cross-section. The method is based on simplifications similar to those proposed in Eurocode 4 (European standard for steel-concrete composite structures), the scope of this latter standard being normally limited to the consideration of one single steel profile. In the perspective of evaluating the resistance of a composite section with more than one embedded steel profile, the pivot method proposed for RC sections is adopted. In order to check the adopted method and its accuracy, results obtained with the spreadsheet have been derived for several representative examples, starting from a basic reinforced concrete section and extending it to a composite one with increasing number of embedded profiles. In particular, the simple tool has been validated by comparison with experimental results taken from technical literature and with numerical simulation results using advanced finite element programs, showing a good accuracy of the simple verification tool.

Keywords: pivot method, composite steel-concrete sections, shear walls,

1. INTRODUCTION

In high-rise buildings, core walls are usually concentrated at the centre of the building to provide substantial lateral strength and stiffness, as well as inelastic deformation capacity needed to meet the earthquake motions. In such systems, although traditionally reinforced concrete (RC) walls are applied with their advantages of easy construction and lower cost, they have many disadvantages. The main disadvantage of an RC shear wall is the development of tension cracks in tension zones and compressive crushing in the localized compression areas during large cyclic excursions. Such cracks and crushing in the compression area can result in splitting and failure of the wall with serious deterioration of stiffness and reduction in strength. To avoid this unwanted situation, the codes recommend that in the design of ductile RC walls to provide in the boundary element regions, concentrated vertical reinforcement confined by closely spaced hoops. This boundary element reinforcement is difficult to place in a wall with thickness of 30 cm or less. One possible solution to this problem is to replace the large amount of reinforcement with steel profiles, structural elements that are called composite steel reinforced walls (CSRCW).
No available design standards are providing information on how to design properly RC sections with more than one embedded profile. The present paper presents therefore an extension of an existing method classically applied to reinforced concrete sections and its use to determine the bending moment – axial force interaction curve of composite sections with more than one steel section embedded. The reason of chosing this method for composite sections is the similarities between the Simplified Method presented in Eurocode 4 and the Pivot Method. The easiest ways to compute this type of sections is to create a C.T. (computational tool) that includes the principle of the two methods.

2. DEVELOPMENT OF THE COMPUTATIONAL TOOL

The pivot method is classically used to determine the failure of a reinforced concrete section subjected to bending moment and axial force. From a graphical point of view the pivots, named A, B and C as shown in Figure 1.1., represent the points defined by ultimate limit strains of the concrete and steel. There are three distinct domains depending on the way in which the failure occurs. Pivot A is the point when the reinforcement bars reach the strain limit in tension, pivot B represent the concrete compression strain limit and pivot C deals with the strain limit for concrete in pure compression.

To determine the ultimate limit moment resistance of a reinforced concrete cross-section, the following assumptions are made:

- Plane sections remains plane.
- The strain in the bonded reinforcement or profile (tension or compression) is the same as in the surrounding concrete.
- The stresses in the concrete in compression are derived from the design parabola rectangle stress-strain diagram as proposed by Eurocode 2 and the tensile strength of concrete is ignored.
- The stresses in the steel part are derived from a bi-linear law for steel.

For a composite cross-section symmetrical about the axis of bending, Roik and Bergmann (1992) proposed a simple method to evaluate its M-N interaction diagram, adopted by Eurocode 4. In fact, this diagram provides the failure conditions and forms the basis of design for the composite column. The procedure of evaluating the M–N interaction diagram for a composite cross-section is:

- Setting the concrete strain at the furthest compression fibre to its crushing strain.
- Assuming an arbitrary position for the neutral axis and the strain distribution in the composite cross – section as linear.
- Evaluating the stress distribution of the composite cross–section according to its strain distribution and stress–strain relationship of the constituent materials. It is assumed that concrete has no tension resistance.
- Obtaining the axial load by integrating the stress over the whole composite cross–section and the bending moment by taking the moments about the plastic centroïd of the cross–section. This determines one point in the M-N interaction diagram.
- Changing the value of the axial force, and based on the linearity of the strain distribution along the cross-section, the value of the curvature that gives the position of the neutral axis in the section is calculated.

Due to the similar assumption of both methods (i.e. Pivot method for RC sections and Roik method for composite section with one profile), it can reasonably be assumed that the failure in a composite cross-section occurs in the same way as in the reinforced concrete cross-section, even for composite steel concrete cross-section that have embedded more than one steel profile, like Figure 1.1.

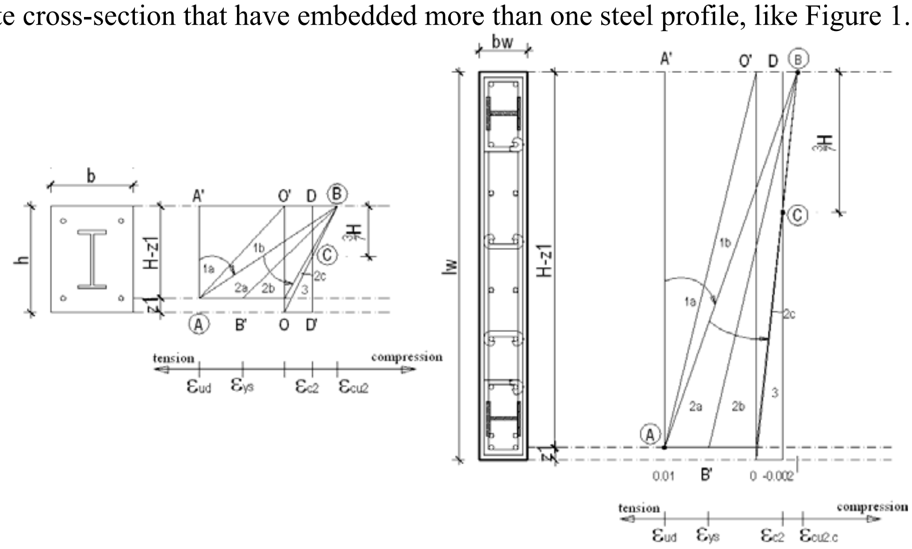


Figure 1.1. Possible strain distribution in the ultimate limit state.
a) composite column; b) ductile wall;

The C.T. is divided in two main parts. The first part takes care of generating the geometry of the section. A subroutine is created in order to divide the section into layers. The thickness of the layer is adjustable, but usually has the unit thickness.
The stress strain relationship used for concrete is a parabola-rectangle diagram. The relation between σ_c and ε_c that is valid for the design of the cross–sections is described by the following expressions, where the values of a, b and c result from the end condition:

$$\sigma_c = \begin{cases} a\cdot\varepsilon_c^2 + b\cdot\varepsilon_c + c & \text{if } 0<\varepsilon_c<\varepsilon_{c2} \\ f_{ck} & \text{if } \varepsilon_c>\varepsilon_{c2} \end{cases} \qquad a = -f_{ck}/\varepsilon_{c2}^2 \qquad b = -2\cdot a\cdot\varepsilon_{c2} \qquad c = 0.$$

where:

f_{ck} - concrete compressive strength;

$\varepsilon_{c2} = \dfrac{f_{ck}}{E_c}$ – strain at reaching maximum strength;

The flexural capacity of a ductile wall depends on the arrangement of the steel part (reinforcement bars and steel profiles) in it. For an ductile wall having a large amount of vertical reinforcement placed at the extremity plus the boundary confining hoops, Eurocode 8 recommends to use the value of an confined strain of the concrete. The value of $\varepsilon_{cu2,c}$ is calculated according to specifications from Eurocode 8 for reinforced concrete ductile walls.
For the steel part, the code recommends the use of a bi-linear stress-strain curve. In this case the bi-linear curve has a horizontal top branch in order to keep the design equation and the design aids reasonably simple.
In the case of multiple steel profiles embedded in the cross-section, for the purpose of simplicity, the area of the profiles that are embedded in the wall section are taken into account as reinforcement bars with the same area as the profile has, and an equivalent diameter.
The second part of the C.T. is used to solve the equilibrium in the cross-section at a fixed value of axial force. Due to the pivot method assumptions, it is assumed that the failure of the section can occur in the most compressed fibre of the concrete part, or in the tensioned reinforcement. The section analysis consists in finding the equilibrium in the cross-section at a given value of the axial-force, based on the position of neutral axis and to determine the strain values at each layer of the material (concrete, steel profile and reinforcement).
The output values that the C.T. gives are:
- The position of the neutral axis at a given axial force value.
- The curvature of the cross section.
- The moment resistance of the section, M_{Rd} at a corresponding axial force.
- The length of confined boundary element, l_c calculated according to [Figure 5.8 – Eurocode 8].

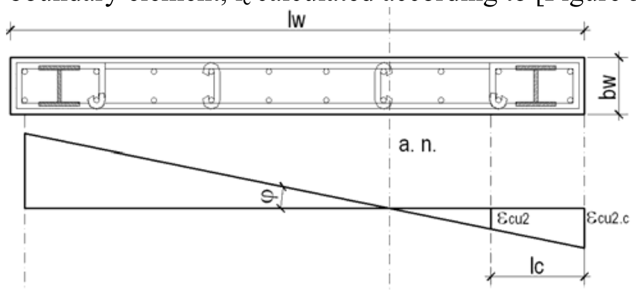


Figure 1.2. Strain distribution in the section at the ultimate curvature.

Based on the assumption that the stresses are linear along the cross-section of composite element, the value of l_c is determined with the formula:

$$l_c = \frac{\varepsilon_{cu2.c} - \varepsilon_{cu2}}{\varphi}$$

where:

ε_{cu2} – ultimate compressive strain of the unconfined concrete;

φ - cross-section's curvature;

3. VALIDATION OF THE PROGRAM.

The C.T. was built starting from a simple composite column cross-section and extended to a double symmetrical section with more than one steel profile embedded. In order to validate the C.T. three types of experimental tests were taken and analysed from the open literature. First, two sets of experimental samples are regarding composite columns that use regular concrete class in one hand and high strength class concrete in the other hand. The third set of experimental tests deals with composite steel reinforced walls (CSRCW).
For all three experimental series, the loading system is composed by one pair of concentrated forces in vertical and horizontal direction. The vertical force (acts like an axial force) has a constant value, while the horizontal one is increasing its value up to the collapse of the structural element.
The finite element model created using the software FinelG, developed at the University of Liège, acts as a link between the experimental tests and the mechanical and analytical modelling, permitting a better understanding of the experimental behaviour and the simplified methods.
The numerical model is modelled by means of plane beam elements with three nodes, as shown in Figure 3.1. Node 1 and 3 presents three degree of freedom (u, v and θ); node 2 only presents one degree of freedom (u) which allows taking into account of an eventual relative displacement between the concrete and the steel profile. This type of elements does not permit to involve cross section local buckling phenomenon. A 2-D numerical analysis is performed; the out-of-plane buckling phenomena as lateral-torsional buckling are not taken into account in the computation.

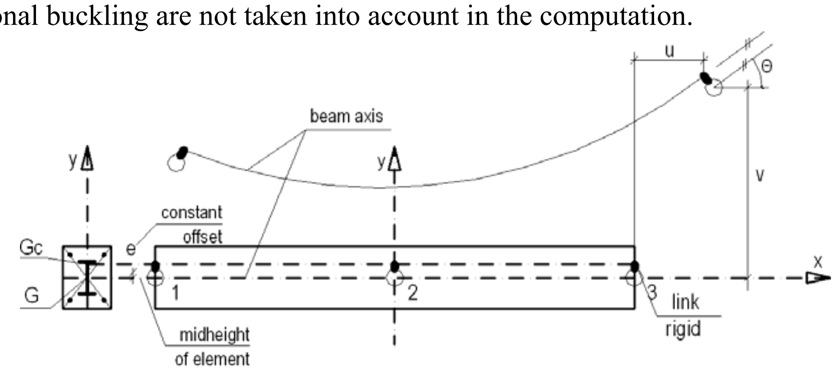


Figure 3.1. Plane beam finite element with three nodes.

For all the materials, the characteristic values for the resistance are used (security coefficient for materials are equal to 1). For the steel elements (steel profile and rebars), a bi-linear law is used for the non-linear analyses (Figure 3.2, a). For the concrete material, a parabolic law with tension stiffening is introduced in the modelling (Figure 3.2, b).

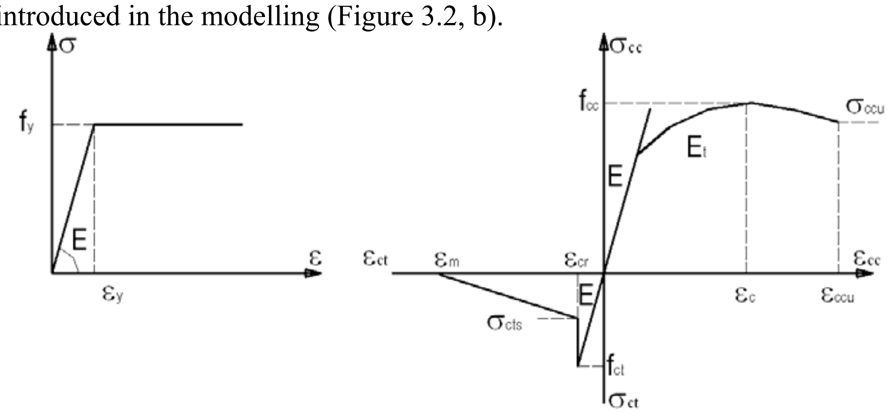


Figure 3.2. Material laws for FinelG
a) Bilinear behaviour law for the steel. b) Parabolic law with tension stiffening for the concrete.

The results obtained in the numerical analysis, in terms of load–displacement responses are compared with the results obtained in the experimental tests. The comparative study is presented below for each set of tests experiments. The small differences between the experiments and numerical model is due

to other nonlinear phenomenon which occurred into the elements behaviour and have not been taken into account in the numerical models, as for example the behaviour of the connection between steel and concrete and the shear transfer realized due to the friction between the cracks faces.
After the calibration was done, using the numerical model and changing the value of the constant axial force, other points on the interaction curve were obtained. Supplementary, there were added the points on the interaction curve obtained with the Simplified Method used for section with only one profile embedded, as shown in Figure 3.3 where the key points are:
- A –squash load point.
- B - pure flexural bending point
- D – the maximum bending moment point
- C – point with bending moment equal to the pure bending moment capacity

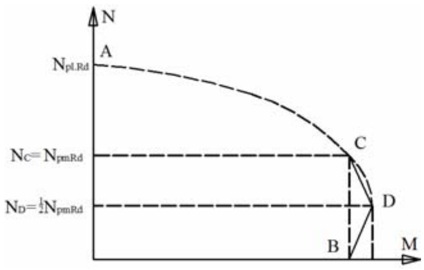


Figure 3.3. Bending moment (M) – axial force (N) diagram for a composite cross – section.

3.1. Composite steel concrete column with regular concrete class

Composite steel concrete column with regular concrete class was tested at Technical University of Cluj Napoca, the mechanical model and composite cross-section of the element are shown in Figure 3.1.1. The length of the column is 3m and the constant axial force applied has the value of N = 100kN.
The material used and section properties:
- Steel profile: S235 with IPE 120 section, f_{ay} = 302 N/mm^2, E_a=207000 N/mm^2.
- Reinforcement: 4 Φ10, S550, f_{sy} = 560 N/mm^2, E_s=207000 N/mm^2.
- Concrete section: 170 x 220 cm with C 20/25 concrete class, f_c = 24.5 N/mm^2, E_c=29000 N/mm^2.

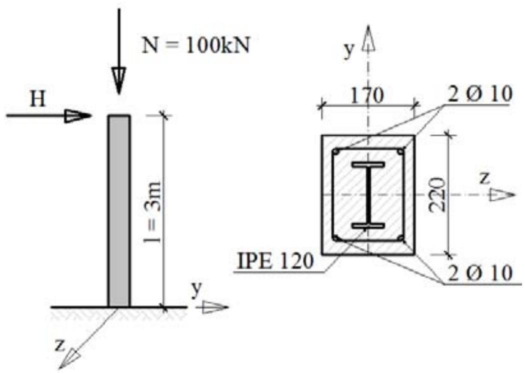


Figure 3.1.1. Composite column with regular reinforced concrete.

Figure 3.1.2. presents a comparison between the force (P) - displacement (Δ) curves obtained using the numerical model and the experimental tests.

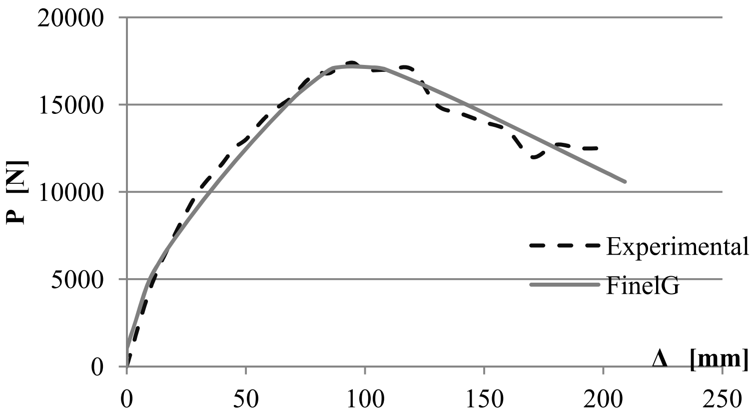
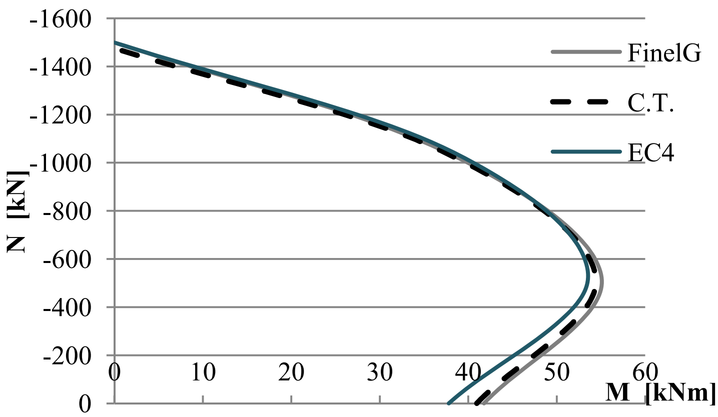


Figure 3.1.2. Comparative P- Δ curves for composite column with regular concrete class.

The similarities between the elastic branch of the curves show that the numerical method follows the experimental test setup. Differences between values are under 10%. The comparison between the interaction curves is presented in Table 3.1.1. and Figure 3.1.3. The value of bending moment is determined in the points suggested by Simplified Method from Eurocode 4. Since in this case the codes are applied there is a third M-N curve drawn using Simplified Method from Eurocode 4.

Tabel 3.1.1. Interaction curve points.

	Eurocode 4		**C.T.**		**FinelG**		
	N [kN]	**M_{Rd} [kNm]**	**N [kNm]**	**M_{Rd} [kNm]**	**N [kN]**	**M_{Rd} [kNm]**	**M_{Rd} - Ratio**
Point B	0	37.79	36.80	36.80	0	37.00	99%
Point D	526	53.54	54.41	54.41	526	57.06	95%
Point C	1052	37.79	36.79	36.79	1052	37.07	98%
Point A	1499	0	1480	0	1527	0	------



3.2. Composite steel concrete column with high strength of the concrete class

The Pivot Method determines the value of the bending moment by limiting strains in the concrete and steel part. In this case, the method is adopted for composite sections using high-strength concrete.
The experimental tests for composite steel concrete column with high strength of the concrete were done at Technical University of Cluj-Napoca, following the same loading mechanism and cross-section shape as the before tests and just changing the concrete class.
The material used and section properties, as shown in Figure 3.2.1:
- Steel profile: S355 with IPE 120 section, $f_{ay} = 355$ N/mm^2, $E_a = 210000$ N/mm^2.
- Reinforcement: 4 Φ10, S355, $f_{sy} = 355$ N/mm^2, $E_s = 210000$ N/mm^2.
- Concrete section: 170 x 220 cm with C 70/85 concrete class, $f_c = 90.74$ N/mm^2, $E_c = 43790$ N/mm^2.

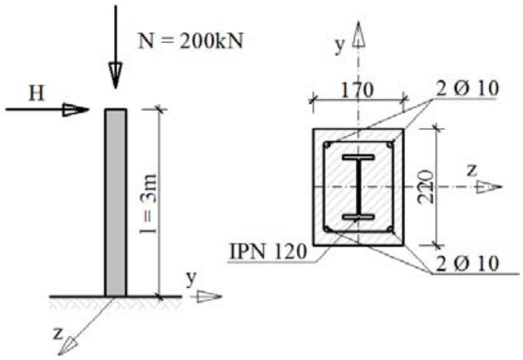


Figure 3.2.1. Composite column with high strength concrete – cross-section.

In Figure 3.2.2 is presented a comparison of the force (P) - displacement (Δ) curves drawn by the experimental tests and numerical model.

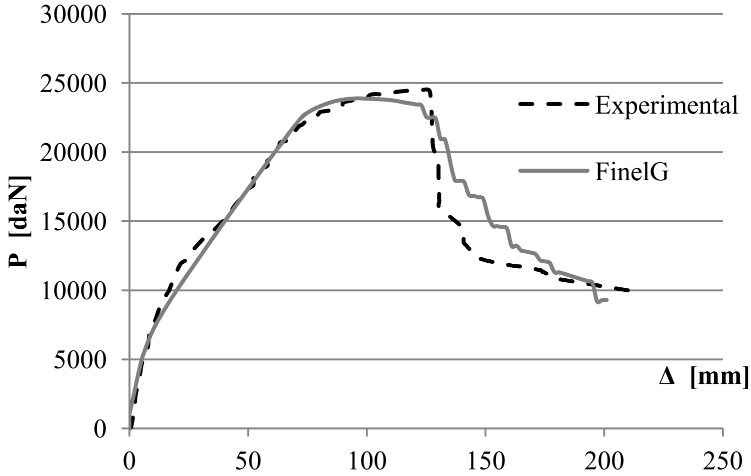


Figure 3.2.2. Comparative P- Δ curves for composite column with high strength concrete class.

Using the numerical model and the axial force points from the M-N curve as in the Simplified Method, a comparison between C.T. and FEM-method was made as shown in Table 3.2.1. and Figure 3.2.3. The results show that this method can be applied to composite cross-section with high-strength concrete.

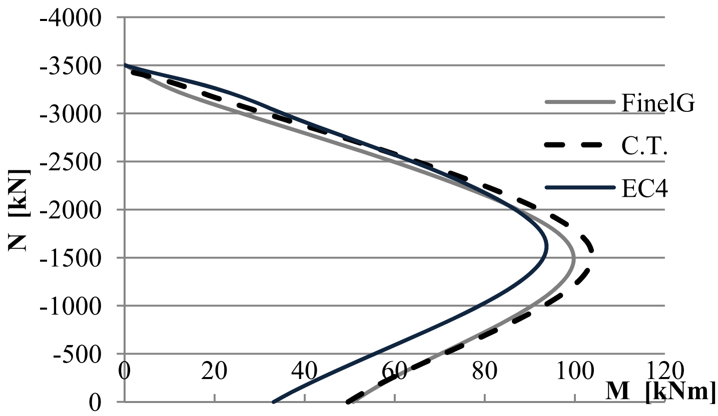


Figure 3.2.3. M–N Interaction curves for composite column with high strength concrete.

The difference between the current design method presented in Eurocode 4 (normally not intended for high strength concrete) shows the need of developing a new tool to compute this type of material.

Tabel 3.2.1. Interaction curve points.

	Eurocode 4		C.T.		FinelG		
	N [kN]	M_{Rd} [kNm]	N [kNm]	M_{Rd} [kNm]	N [kN]	M_{Rd} [kNm]	M_{Rd} - Ratio
Point B	0	33.11	0	49.67	0	50.8	98%
Point D	1618	93.74	1618	103.31	1618	99.0	96%
Point C	3236	33.11	3236	15.81	3236	11.28	71%
Point A	3500	0	3447	0	3506	0	------

3.3. Composite steel reinforced walls (CSRCW)

The behaviour of the ductile walls was experimentally studied by the Politehinca University of Timişoara. The experimental program consists of six 1:3 scale elements (CRSCW 1 to 6), designed using principles from the existing codes. The structural steel profiles were connected with the concrete web by headed shear stud connectors. For all the specimens the reinforcements of the RC web panels consists of Φ10/100 mm vertical bars and Φ 8/150 horizontal bars. Vertical and horizontal reinforcements were placed on the both faces and were connected together with Φ10/100 mm steel ties. The height of the wall is 3 m and the testing procedure consist in one constant vertical load and a cyclically increasing horizontal lateral load. In this case the subject of discussion are only the first 4 walls, as shown in Figure 3.3.1, being the most representative.

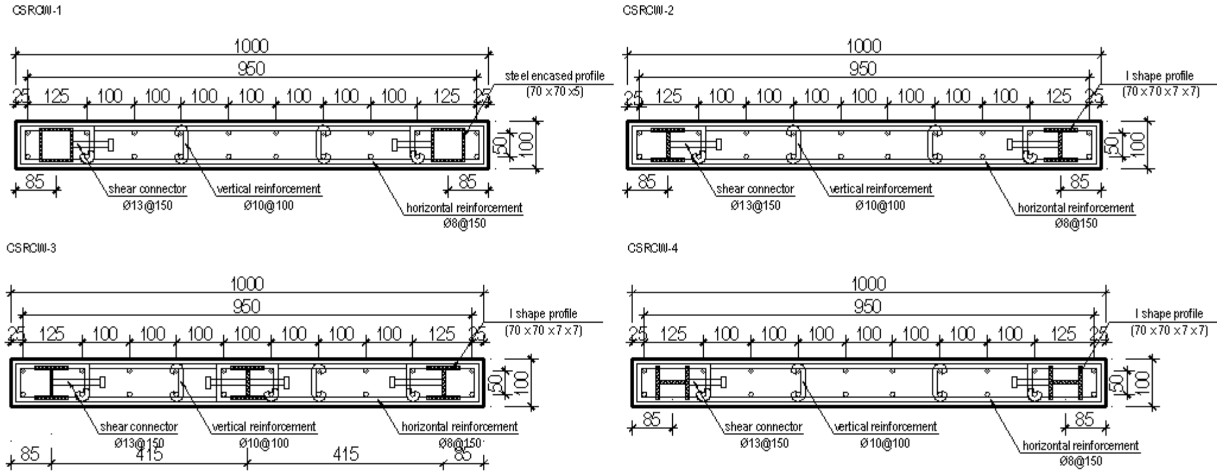


Figure 3.3.1. Details of the composite steel –concrete walls.

The designed concrete was C20/25 class, the reinforcement S355 steel and the structural steel Fe510. Material proprieties are listed in the table below:

Tabel 3.3.1. Material properties.

Specimen label:	f_{cm} [N/mm²]	E_{cm} [N/mm²]
CSRCW 1	54.7	36628
CSRCW 2	46.0	34773
CSRCW 3	65.1	38591
CSRCW 4	62.0	38031

Type:	f_y [N/mm²]	f_u [N/mm²]	E_s [N/mm²]
Steel rebar 10	548	622	211000
I-shaped steel	328	516	203000
Steel tube	342	532	207000

In Figure 3.3.2 is presented a comparison of the force (P) - displacement (Δ) curves drawn by the experimental tests and numerical model.

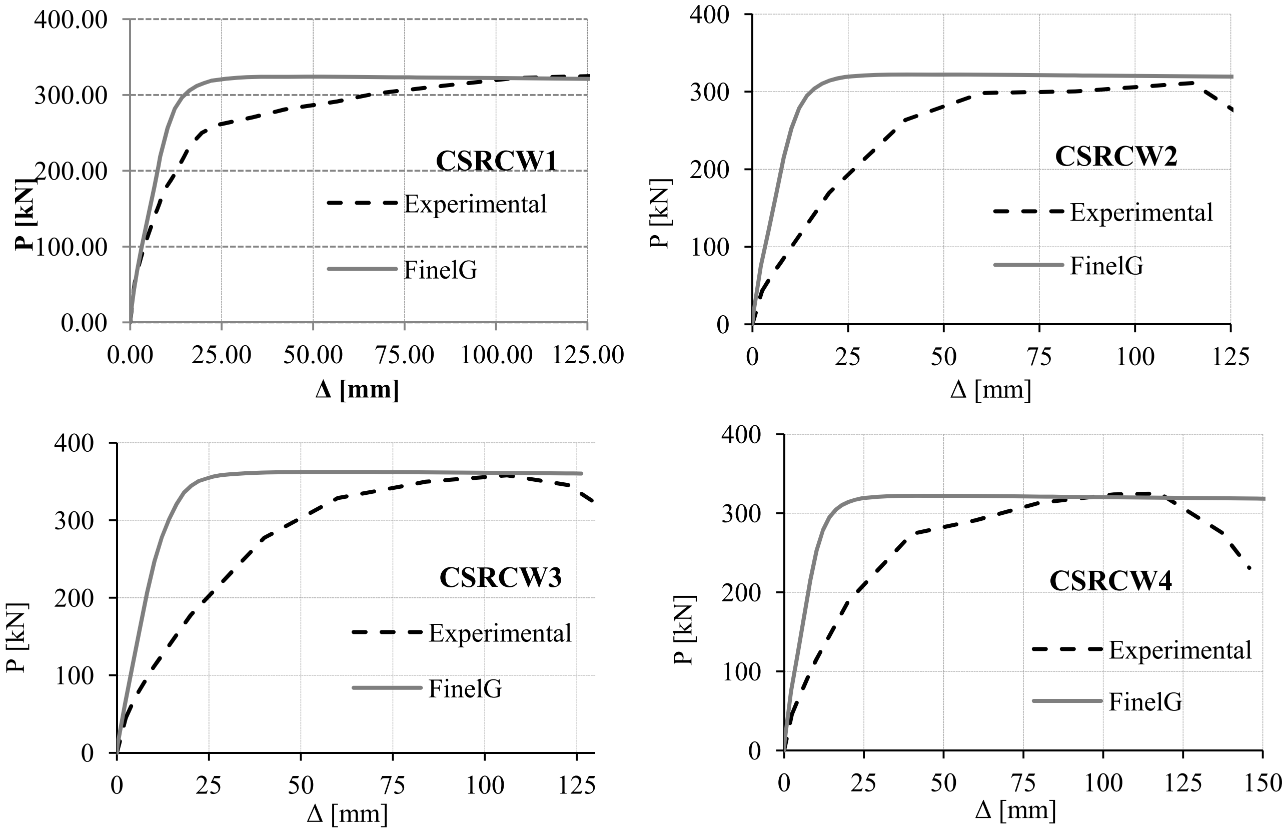


Figure 3.3.2. Comparative P- Δ curves for CSRCW .

The FEM model was calibrated using the pushover curves and used to drawn the M-N interaction curves, as shown in Table 3.3.2. and Figure 3.3.3.

Tabel 3.3.2. CRSCW - Interaction curve points.

CSRCW 1	**N - FinelG [kN]**	**M_{Rd} -FinelG[kNm]**	**N -C.T. [kN]**	**M_{Rd} –C.T.[kNm]**	**M_{Rd} - Ratio**
Point B	0	610	0	641.67	95%
Point D	-2623.12	1198	-2623.12	1179.57	98%
Point C	-5246.23	619	-5246.23	562.28	90%
Point A	-7298.22	0	-6968.87	0	
CSRCW 2	**N - FinelG [kN]**	**M_{Rd} -FinelG[kNm]**	**N -C.T. [kN]**	**M_{Rd} –C.T.[kNm]**	**M_{Rd} - Ratio**
Point B	0	695	0	650.97	94%
Point D	-2200.76	1089	-2200.76	1053.80	96%
Point C	-4401.52	619	-4401.52	585.96	94%
Point A	-6259.55	0	-6085.05	0	
CSRCW 3	**N - FinelG [kN]**	**M_{Rd} -FinelG[kNm]**	**N -C.T. [kN]**	**M_{Rd} –C.T.[kNm]**	**M_{Rd} - Ratio**
Point B	0	797	0	721.64	90%
Point D	-3069.89	1294	-3069.89	1263.24	97%
Point C	-6139.79	741	-6139.79	709.24	99%
Point A	-8409.98	0	-8362.74		
CSRCW 4	**N - FinelG [kN]**	**M_{Rd} -FinelG[kNm]**	**N -C.T. [kN]**	**M_{Rd} –C.T.[kNm]**	**M_{Rd} - Ratio**
Point B	0	706	0	650.97	92%
Point D	-2200.76	1090	-2200.76	1053.80	96%
Point C	-4401.52	630	-4401.52	585.96	92%
Point A	-6259.55	0	-6085.05	0	

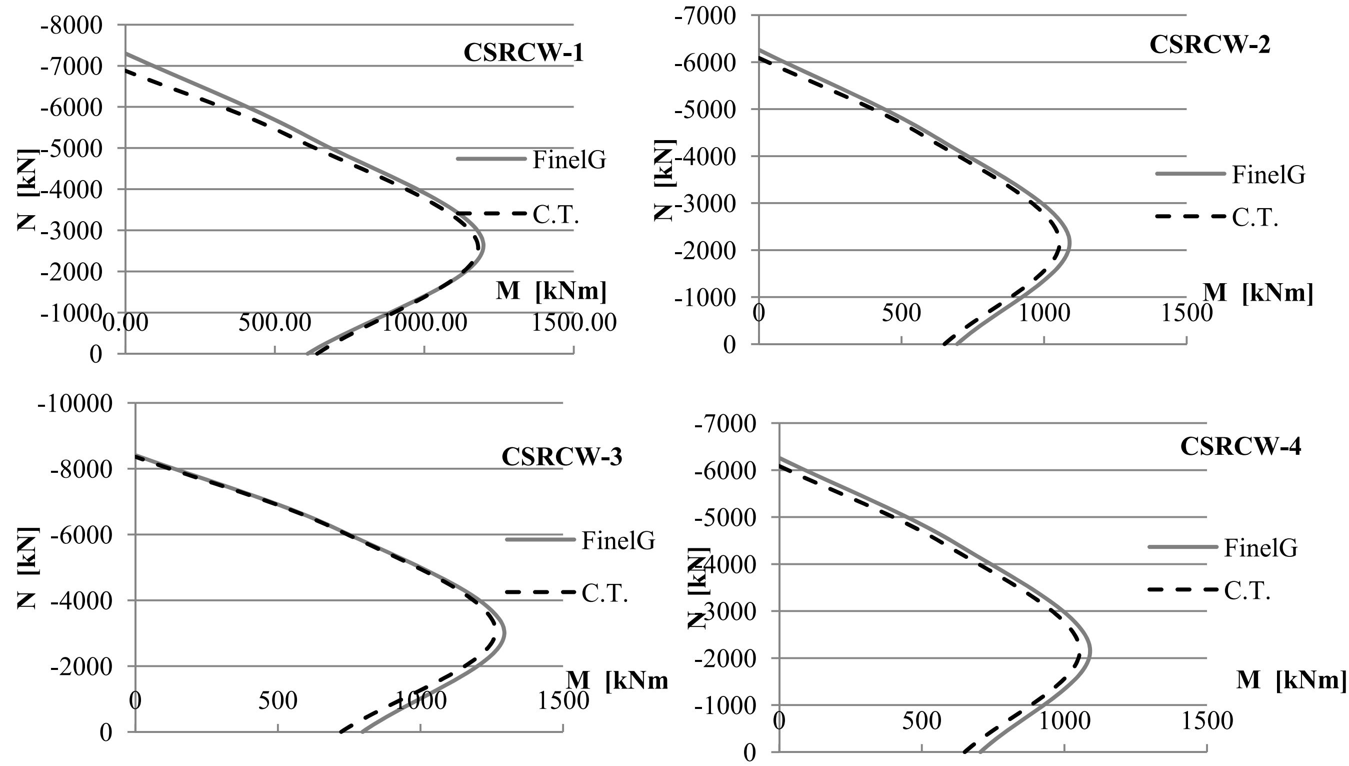


Figure 3.3.3. M–N Interaction curves for composite column with high strength concrete

4. CONCLUSIONS.

The structural systems that use steel concrete composite shear walls have an improved performance at earthquakes. The technical literature about ductile walls shows poor level of knowledge in designing composite steel-concrete steel sections with more than one steel profile embedded. Based on the methods available in the design codes, an extension was developed in order to be able to draw the M-N interaction curves for this type of structural elements. The computational tool was implemented as Microsoft Excel spreadsheet in order to facilitate the practical use of the design method and validated against experimental tests and advanced finite element models. For ductile walls, the C.T. takes into account the ultimate confined value of the concrete stress of the boundary concrete as recommended by Eurocode8. Assuming a linear strain distribution across the composite section, it is also possible to determine easily the confinement length l_c, as required by Eurocode 8 Figure 5.8 in case of reinforced concrete ductile walls.

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