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INNOVATIVE BEAM-TO-WALL JOINTS IN STEEL-CONCRETE STRUCTURAL SOLUTIONS

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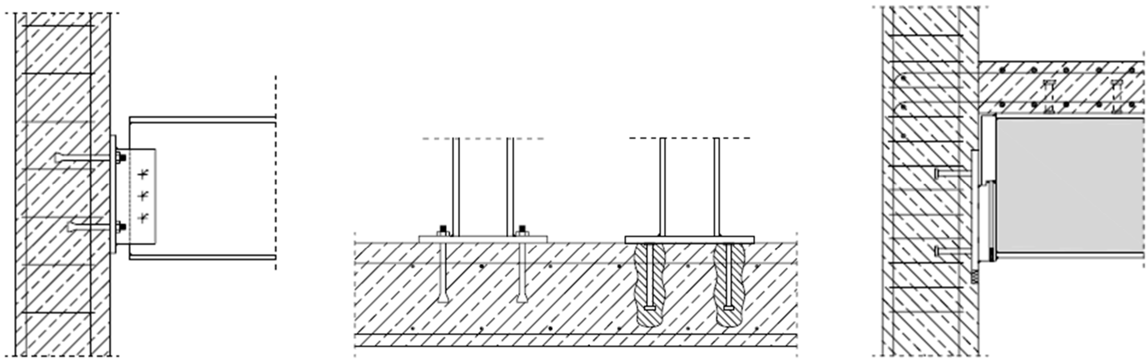
Abstract. This paper presents the innovative solutions to connect steel and composite beams to structural reinforced concrete walls developed within the RFCS research project "InFaSo". Two types of joints were studied: pinned and moment resistant. The evaluation of the joints behavior was performed experimentally and complemented with the development of analytical component based models. The comparison of results showed a good agreement between models and experiments. The analyzed joints demonstrated to be competitive solutions taken into account their structural performance, simplicity of modeling and of execution.

1. Introduction

Aiming at the study of steel-to-concrete joints, the RFCS project entitled "New market chances for steel structures by innovative fastening solutions" ("InFaSo" [1]) was launched. Three types of joint joints were subject of study: i) pinned beam-to-wall (Fig. 1-a); ii) column bases (Fig. 1-b); iii) moment resistant composite beam-to-wall (Fig. 1-c). The connection between steel and reinforced concrete members, for a simple and efficient erection, is the innovation "brought". This is performed by means of steel plates anchored to concrete using headed anchors. In the pinned joints supplementary reinforcement may be used to enhance resistance and ductility. In order to evaluate the joints properties an experimental programme was accomplished and analytical models developed within the project tasks. The proposed models are based on the component method which is usually applied to steel and composite joints. The extension of the method required the characterization of "new" components activated in this type of joints. These involve essentially the participation of the concrete on the possible modes of failure of the joint.

In the present paper the developments of the InFaSo research project on the pinned and moment resistant joint are presented. The experimental results and the validation of the proposed analytical models are discussed.



a) Pinned b) Column base c) Moment resistant

Fig. 1: InFaSo studied joints [1]

2. Component method and additional components in steel-to-concrete joints

2.1 General description

The component method is known as an efficient approach to evaluate the real behaviour of the steel and composite joints. Therefore, the extension to steel-to-concrete joints was sought. In such joints, "new" components are activated which consider the concrete modes of failure associated to the anchorage in concrete using headed anchors. These components, listed in Table 1, are not yet included in the actual versions of the EN 1993-1-8 [2] and EN 1994-1-1 [3]. As required by the component method, the characterization of the components has to be performed in terms of F-d response. Thus, experimental work on the "new" components and the proposal of analytical models was performed within "InFaSo" research project [1] and is presented in the following sections.

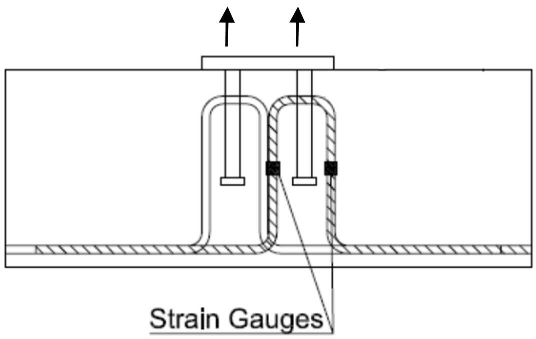
Table 1: List of new components activated in steel-to-concrete joints

Anchors in Tension		Anchors in Shear	
Failure mode	Component ID	Failure mode	Component ID
Concrete cone	T-CC	Concrete edge failure	V-CE
Pull-out/Pull-through failure	T-PO	Pry-out failure	V-PrO
Splitting failure	T-Sp	Pull-out failure	V-PO
Local blow-out failure	T-BO	Hanger reinforcement failure	V-HR
Hanger reinforcement failure	T-HR		

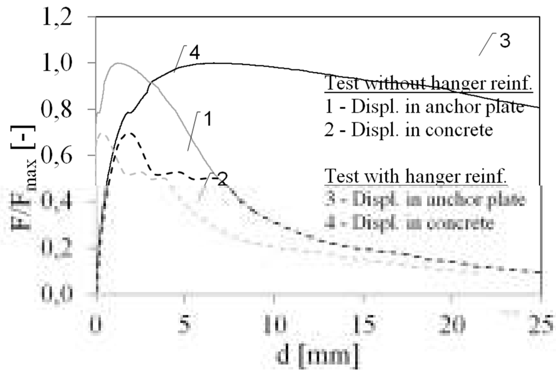
2.2 Experimental research

The experimental programme on components was concentrated on the anchorage in concrete because up to now these components had not been integrated into the philosophy of the component method. Several groups of tests were performed. The following parameters were varied: i) type of fastener, headed anchors or undercut anchors; ii) type of loading, tension and combined shear and tension (mainly pure tension); iii) concrete state, always cracked state (anchor is installed in the crack plane); iv) use of hanger reinforcement, with and without hanger reinforcement; v) position of the hanger reinforcement, close or distant to the anchors. Here, only the tests with headed anchors in tension are discussed.

For the pure tension tested specimens, the general test procedure is illustrated in Fig. 2-a). In Fig. 2-b) are shown the relative load-displacements curves for two specimens, one without and one with hanger reinforcement. The curves show the typical response of this type of anchorage. For each test, two curves are obtained, one representing the displacements measured in anchor plate and the other the displacements in the concrete. The latter allows identifying the contribution of the concrete cone component to the global deformation of the anchorage. In what concerns the use of hanger reinforcement, the results demonstrate that this type of reinforcement increases both the resistance and the deformation capacity of the anchorage. The use of strain gauges (Fig. 2-a), allowed obtaining the force in the hanger reinforcement component and consequently quantify its contribution.



a) Test scheme [4]



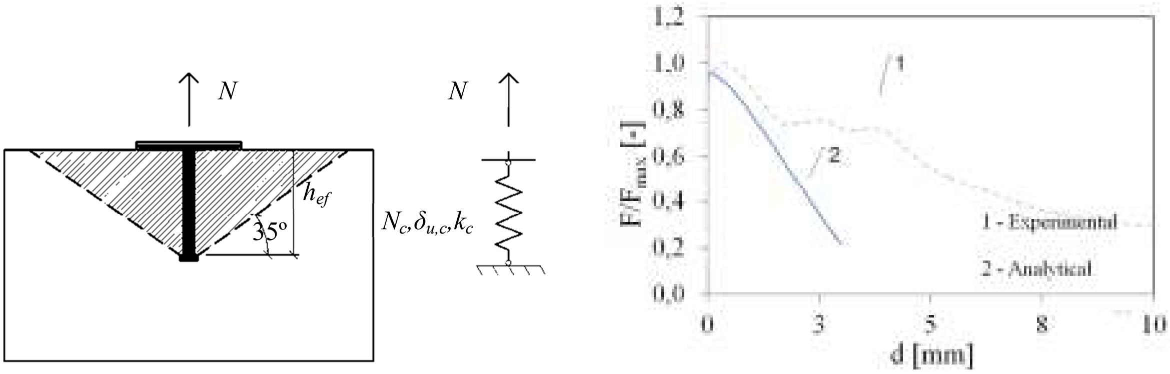
b) Load-displacement response [5]

Fig. 2: Experimental work on components

2.3 Analytical approach

The basis of the analytical models proposed within InFaSo [1] is the experimental data previously presented. As result, the validity of the derived models is limited to the range of the experimental tests. However, taking into account the absence of approaches to characterize these components, as required by the component method, the proposed models are promising. Furthermore, their generalization can be easily performed extending the range of the experimental tests and through the use of numerical models. In the present paper, only the model for the component concrete cone is presented. In [1],[4], the interested reader may find the proposed models for all tension components listed in Table 1.

The concrete cone component (Fig. 3-a) represents a pure concrete failure and its primary development is always independent of the presence of hanger reinforcement [6]. As shown in Fig. 2-b, with curves 2 and 4, the typical behaviour of this component is rigid up to its maximum resistance followed by a softening branch, where the resistance drops with the increase of deformation. As for the resistance, the used CC-Method is a consensual and accurate approach, prescribed by different standards [7],[8] and authors [9], the new proposal [6] relies on the definition of the force-deformation curve. Thus, the component model is expressed as in (1) and (2). Basically, the main objective accomplished was the definition of the stiffness of the descending branch which is given by the stiffness factor k_c as expressed in (3). This factor is formulated based on the influencing parameters of the CC-Method where α_c is pure empirical and limited to the range of the tests. The application and the accuracy of the model are shown in Fig. 3-b) by the presented relative load-displacement curves of test and model. As sought within the InFaSo project, the model is simplified and therefore the descending branch is represented by linear behaviour. In this way, keeping a good approximation, the model is directed to design purposes.



a) Concrete cone component [6] b) Comparison of F-d curves [1]

Fig. 3: Proposed model for a concrete cone component in steel-to-concrete joints

$$\delta_c = 0 \;\rightarrow N = \; N_{u,c} \tag{1}$$

$$\delta_c > 0 \;\rightarrow\; N = N_{u,c} + \delta_c \cdot k_c \tag{2}$$

$$k_c = \alpha_c \cdot \sqrt{h_{ef}} \cdot \sqrt{f_{cc,200}} \cdot \frac{A_{c,N}}{A^0_{c,N}} \tag{3}$$

Where: k_c is the stiffness of the descending branch; N is the load applied to the anchorage on the anchorage without hanger reinforcement; $N_{u,c}$ is the ultimate load of the anchorage without hanger reinforcement;δ_c is the displacement of the component concrete cone; α_c is the factor of the component concrete cone; h_{ef} is the embedment depth of the anchorage; $f_{cc,200}$ is the concrete compressive strength measured on the cubes with 200mm side length; $A_{c,N}$ is the projected area of the concrete cone at surface; and $A^0_{c,N}$ is the projected area of the concrete cone at surface for a single anchor without influence of close edges.

3. Pinned joint of a steel beam to a reinforced concrete wall

3.1 General description

In the pinned joint configuration studied within the InFaSo project [1], the steel-to-concrete connection is accomplished using an anchor plate with welded headed anchors. Then, on the steel side, a steel cam or fin plate may be used to connect the steel beam through welding or bolting, respectively. The performed study was focused on the steel-to-concrete connection. Thus, an experimental programme, focusing this part of the pinned joint, was accomplished. Then, in order to evaluate the joint properties, a mechanical model, extending the field of application of the component method, was proposed. The models developed for the new basic components, described in the previous section, were in this way applied to evaluate the overall behaviour of the joint.

3.2 Experimental research

The test programme was based on the joint solutions described above and illustrated in Fig. 1-a). Thus, a stiff anchor plate with two rows of headed anchors is connected to a reinforced concrete wall. The stiff anchor plate was used so that the concrete components were fully activated. The load was applied to the anchor plate with eccentricity. This eccentricity was varied according to the possible joint solutions. The joints were tested mainly in cracked concrete with and without hanger reinforcement. The cracks were installed perpendicular to the applied load and crossing the anchor row to be activated in tension due to the eccentricity of the Shear load. Furthermore, the disposition and the length of the headed anchors were varied. In Table 2 the complete test programme is presented.

Table 2: Test programme for pinned steel-to-concrete joints [10]

Test specimen	Eccentricity [mm]	Anchorage length h_{ef} [mm]	Hanger reinforcement	Concrete condition	Disposition of anchors
B0-BS	53	160	-	non-cracked	2x3
B1-BS	53	160	-	cracked	2x3
B1-BS-R	53	160	Yes	cracked	2x3
B2-C	139	160	-	cracked	2x3
B2-C-R	139	160	Yes	cracked	2x3
R1-C	139	160	-	cracked	2x2
R1-C-R	139	160	Yes	cracked	2x2
R2-C	139	210	-	cracked	2x3
R2-C-R	139	210	Yes	cracked	2x3

In all tests failure was attained by concrete cone failure and/or pry-out failure. The simultaneously development of these two modes of failure are due to the loading conditions of the anchor plate, shear load and secondary bending moment. According to the level of the eccentricity, one of the failures modes becomes more relevant. In Fig. 4 a comparison of the relative load-rotation behaviour of 4 test specimens is shown. Comparing the results of the specimens with hanger reinforcement (B1-BS-R and B2-C-R) with those without (B1-BS and B2-C) an increase of resistance and ductility of the joints is observed. In what respects to the effect of the eccentricity, in the test specimens with higher eccentricity, the maximum shear load was relatively smaller. In these cases, the tension concrete component governed the behaviour of the joint due to the higher tension introduced to anchor row on the tension side of the joint. For smaller eccentricities the joint behaviour was governed by the shear failure.

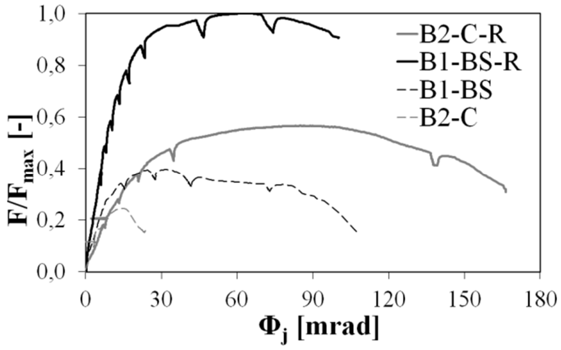
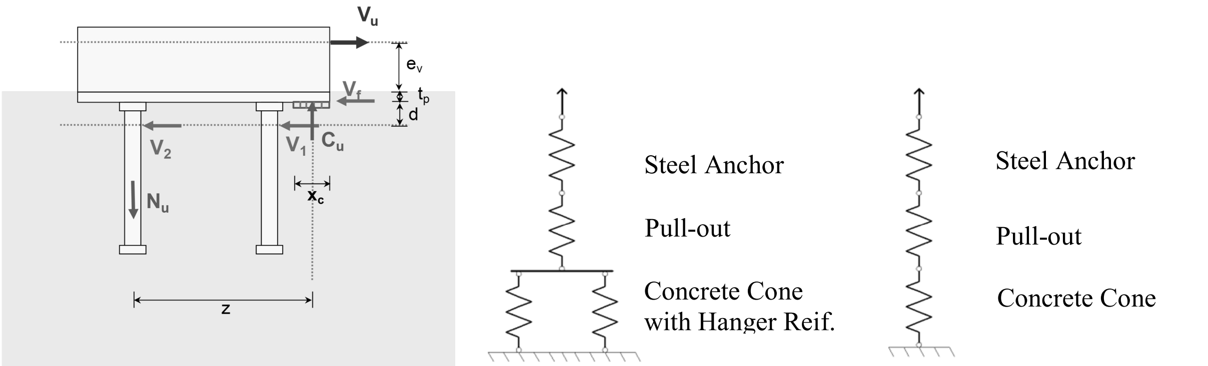


Fig. 4: Comparison between load-rotation curves of test specimens with and without stirrups [10]

3.3 Analytical approach

The focus of the experimental work was on the concrete components therefore, the developed mechanical model mainly consists of the components at the concrete side of the joint. The use of stiff anchor plate and steel cam/fin plate allowed neglecting their behaviour, as they didn't play a role.

In Fig. 5-a) is illustrated the internal loading of the joint to equilibrate the external shear load V_u. Due to the eccentricity of the latter, a secondary bending moment develops and consequently the tension components are activated on the non-loaded side of the plate (left side according to Fig. 5-a). In Fig. 5-b) are represented the tension components to be considered in the model of the joint. As referred in §2, each component represents the possible failure modes associated to the anchorage in concrete. The contribution of hanger reinforcement is considered adding a spring parallel to the concrete cone component. A detailed description of these components may be found in [1].

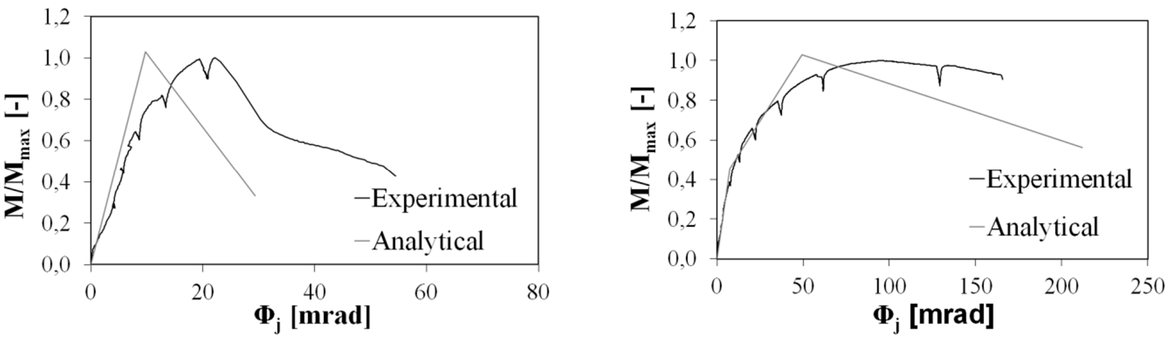


a) Representation of the joint loading

b) Model for the tension components

Fig. 5: Pinned joint model

For the compression zone, a rectangular stress block is assumed under the plate (see Fig. 5-a). Here, the stresses are limited to $3f_{cm}$, as proposed in the prCEN/TS 1992-4 [8]. The stress area A_c is given by the width of the anchor plate x_c (perpendicular to the load) and the length of the compression zone, which results from the equilibrium with the assumed tension force in the headed anchors on the non-loaded side. Thus, the internal lever arm z and the inner bending moment are calculated. The latter defines the resistance to the secondary bending moment introduced by the shear load applied with eccentricity. In what regards to the shear resistance, the contribution of the shear resistance of the anchors and the friction between the concrete surface and the anchor plate is considered. The friction resistance is proportional to the compression force defined above. In the model a friction coefficient μ=0,4 was used as proposed in [11]. The shear resistance of the anchorage is dependent of two possible failure modes: i) steel failure of the anchors shaft; ii) pry-out failure. Finally, the anchor row on the non-loaded side is subjected simultaneously to tension and shear, therefore the interaction as to be taken into account. This may be performed using the appropriate interaction formula given in prCEN/TS 1992-4 [8]. The comparison of the developed component model for the pinned joint with the respective experimental results is shown in Fig. 6. For this purpose, two specimens are used, one without and one with hanger reinforcement. The presented moment-rotation curves demonstrate a good agreement between results. It can be seen that the model can predict the contribution of the hanger reinforcement, for the resistance and ductility, in a satisfying way. The average approximation of the results, either for the case without hanger reinforcement either with, is very good. A maximum deviation of 4% is observed.



a) R2-C (without hanger reinforcement) b) R2-C-R (with hanger reinforcement)

Fig. 6: M-φ curves comparing model and test results

4. Moment resisting joint of a composite beam to a reinforced concrete wall

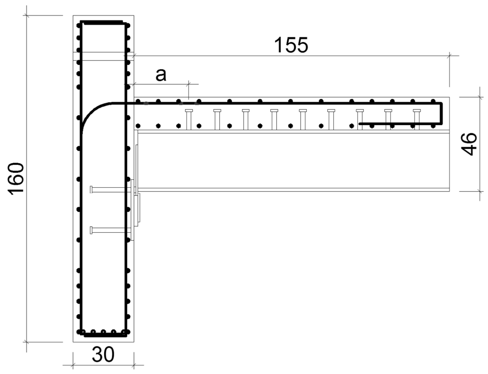
4.1 General description

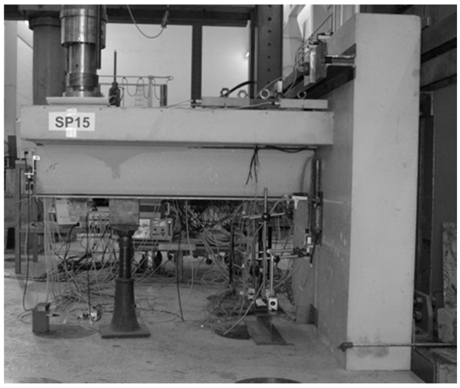
In the studied moment resistant joint (see Fig. 1-c) two regions are distinguished. At the upper of the joint the connection, between the concrete slab and wall, is achieved extending and anchoring the longitudinal reinforcement of the slab in the wall. At the bottom part, the steel beam bottom flange sits in a steel bracket welded to an anchor plate. Using headed anchors, this plate performs the connection to the wall. Then, a contact plate, between steel beam and anchor plate, is used to transfer compression. In order to study the described joint configuration, experimental tests were executed within the experimental programme of the InFaSo research project [1]. In order to evaluate the joint properties to a hogging bending moment, a mechanical model, based on the component method, was developed and validated by the experimental results.

4.2 Experimental research

A total of six tests were performed, three at the University of Stuttgart and three at the Czech Technical University in Prague. In the first, the influence of the percentage of longitudinal reinforcement in the slab and the position of the first shear connector near the joint face were analyzed. In the latter, the thickness of the anchor plate and of the steel bracket was studied. The test layout is illustrated in Fig. 7. This consists in a cantilever composite beam supported by a reinforced concrete wall using the joint configuration described above. The loading is applied by a hydraulic jack at the free edge of the beam inducing the joint to a hogging bending moment. The loading is quasi-static monotonic.

In all tests failure was attained by rupture of one of the longitudinal steel reinforcement bars in the slab. The variations performed on the anchor plate and on the steel bracket, at the Cezch Technical University in Prague, did not produce any significant influence on the behavior of the joint. Thus, the longitudinal steel reinforcement governed completely the response of the joint. In Fig. 8 is shown the relative moment-rotation curves of the experiments executed in the University of Stuttgart. As expected, the joint resistance varied with the percentage of longitudinal reinforcement. The position of the first shear connector near the joint face affected the initial stiffness of the joint and mainly the ultimate rotation capacity.





a) Test specimens' configuration (cm) b) Test layout

Fig. 7: Tests on moment resistant joint [12]

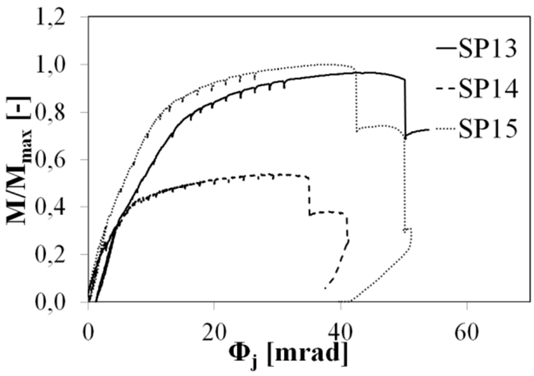


Fig. 8: Relative moment-rotation curves obtained in tests performed at the University Stuttgart

4.3 Analytical approach

Based on the joint configuration, the joint components activated are identified and the simplified model represented in Fig. 9 was developed. This reflects the joint mechanics when subjected to a hogging bending moment. As observed in the experimental tests, the longitudinal reinforcement in tension is the governing component. Consequently, the accuracy of the model will much depend on the level of sophistication introduced in the model of this component. A sophisticated model of the longitudinal reinforcement in tension may be found in [13] where the embedment of the bars in concrete is taken into account. In addition, the ultimate deformation capacity of the component can be performed allowing estimating the ultimate joint rotation capacity. In what concerns to the other components, as observed in the tests, their role on the joint response is minor. Thus, its evaluation was performed as prescribed in the EN 1993-1.8 [2] and EN 1994-1 [3]. For the group of compression components, components 5, 6 and 7, the T-stub in compression (column bases), prescribed by the EN 1993-1.8 [2], was used to evaluate their response. Some similarities were found between the behaviour of the group components and T-stub in compression. Then, in what regards the model assembly, the procedure used is similar as in the case of steel and composite joints. Establishing the joint lever arm (h_r) as the distance between the longitudinal reinforcement and the steel beam bottom flange, the joint bending moment (M_j) and the joint rotation (Φ_j) may be determined as expressed in (4) and (5).

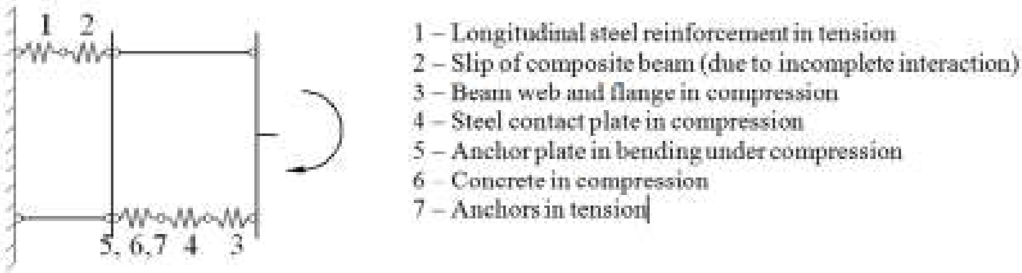


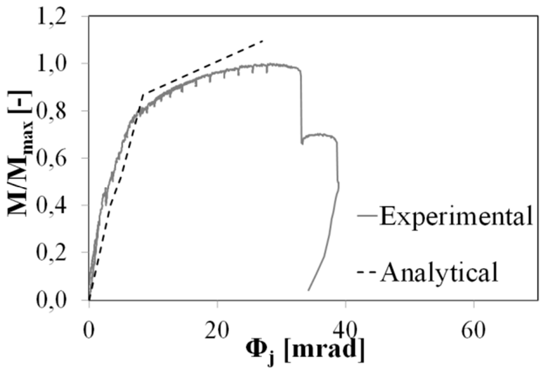
Fig. 9: Simplified mechanical model for the moment resistant joint

$$M_j = Min\{F_{eq,t}; F_{eq,c}\}h_r \tag{4}$$

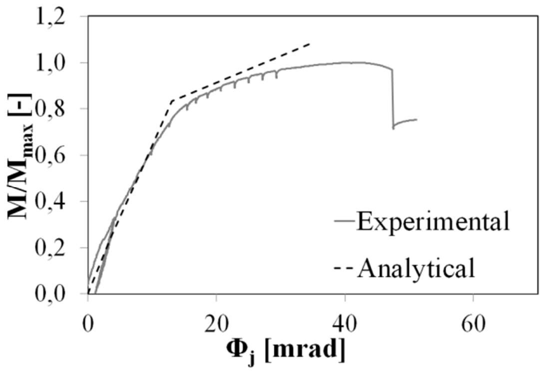
$$\Phi_j = \frac{\Delta_{eq,t}+\Delta_{eq,c}}{h_r} \tag{5}$$

Where: $F_{eq,t}$ and $F_{eq,c}$ are the resistance of the equivalent components, tension and compression, respectively; $\Delta_{eq,t}$ and $\Delta_{eq,c}$ are the deformations of the equivalent components, tension and compression, respectively.

In Fig. 10 the relative moment-rotation curves comparing analytical model and experimental tests are shown. For this purpose, test specimens with different percentage of longitudinal reinforcement were used. As it can be observed a very good agreement was obtained for resistance, initial stiffness and hardening stiffness. The maximum deviation in terms of resistance was approximately 9%. In terms of ultimate joint rotation, taking into account the difficulty to find methods for its evaluation, the obtained approximation is interesting.



a) Test specimen SP14 (lower % percentage of reinforcement)



b) Test specimen SP15 (higher % of reinforcement)

Fig. 10: Relative moment-rotation curves comparing model and test results

5. Conclusions

In this paper a part of the work developed within the RFCS research project "InFaSo" [1] has been presented. The project sought simple and efficient solutions to connect steel/composite members to reinforced concrete members. Within the project tasks, experimental tests were performed and analytical models developed for three types of joints: pinned joint, column base and moment resistant joint. The analytical models proposed, based on the component method, required the identification and characterization of "new" components related to the anchorage in concrete. Thus, based on experimental programme on components, analytical models for these components were proposed. At the joint level, the performed tests demonstrated interesting performance for both studied joint configurations, pinned and moment resistant. In the first case, the enhancement of the resistance and ductility was successfully achieved using hanger reinforcement in the anchor row in tension (on the non-loaded side). In the latter case, the joint configuration showed considerable bending moment resistance and joint rotation capacity. The derived component models showed to be accurate. To conclude, the interested reader may found more detailed information, on the discussed work, in [1].

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