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NUMERICAL AND EXPERIMENTAL STUDY OF BEAM-TO-COLUMN STEEL JOINTS UNDER DYNAMIC LOADING IN LOW-RISE PREFABRICATED MODULAR BUILDINGS

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Abstract.*In the present paper the behavior and the structural performance of steel beam to column connections is investigated for an innovative type of low-rise prefabricated modular building system. The load bearing system is composed of lightweight steel frame members connected with double-shell composite walls that play the role of smart façade elements. Aiming to obtain a clear perspective of the dynamic performance of the system, a series of experimental and numerical tests of critical structural details such as the steel joints of the systems has been carried out. Two alternative types of steel beam to column specimens consisted of welded and bolted parts with differences in rigidity were tested in quasi-static cyclic loading in order to investigate their bearing capacity, as well as their hysteretic behavior. Using ANSYS software a three-dimensional nonlinear FEM model was developed and validated comparing the respective numerical results with the experimental results from the tested specimens. The same quasi-static cyclic loading history scheme was applied both for the numerical models and the test specimens to register the critical issues of the joint components under investigation. The obtained experimental and numerical results are presented and discussed in terms of moment-rotation response, failure mode, load-bearing capacity, ductility, and energy dissipation capacity. Moreover, the developed numerical models were used for the evaluation of the structural response of the system and for the optimization of the cost, the shape and the bearing capacity of the joint. With regards to energy dissipation capacity of the system under investigation, it is obvious that both the tested specimens as well as the respective numerical models absorb a significant amount of energy from the overall steel frame system of the composite modular structure.*

1 INTRODUCTION

In recent years there has been a significant increase in interest in the production of prefabricated building elements with the ultimate goal of composing from them, in whole or in part, a complete and safe construction. The reasons that justify this fact are among others the reduced required time of completion of the project, the reduced cost for the construction to complete and the increased reliability of the prefabricated elements which are manufactured at the factory and just assembled at the site, giving greater reliability to the construction [2]. Meanwhile, there is a requirement that the prefabrication does not adversely affect the stability of the structure and if it is possible even to contribute as well as to be a filling element. The behavior and the structural performance of steel joints (see Figure 1) become crucial for that safety and the reliability of such systems [3]. The distribution of forces and moments in the structure due to the loadings is a result of their strength and stiffness. So, the structural characteristics of the joints such as stiffness, strength and rotation capacity, together with those of the structural components steel frame members and composite wall modules respectively, produce these forces in the joints [7]. This means that the choices about the design of joint, are of direct influence on the level of actions not only in the joint but in the steel frame system in general [8].

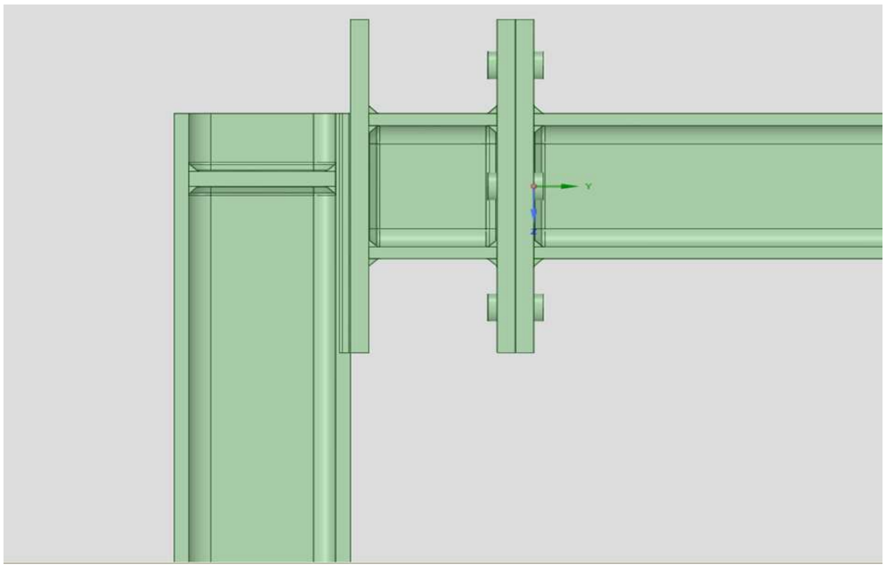


Figure 1. Schematic view of the initial configured JL-6 beam-to-column steel joint

In this context there has been designed a standard prefabricated unit of innovative building element which is composed by lightweight steel frame members connected with double-shell composite wall with advanced properties [11]. The double shell wall member in this system is composed of two prefabricated of small thickness outside concrete walls and a number of normally distributed steel RHS sections inside. The two outside concrete panels attached with full contact to the inside sited vertical steel hollow sections or to the insulating material and moreover with steel connecting devices welded to the vertical steel hollow sections at distributed vertically distances (See Figure 2).

Moreover, a standard steel connection system has been investigated to connect the composite walls to the adjacent main steel frame elements of the building. That design is completely satisfying the demands of a filling element as well as enhancing the whole strength of the structure in static or dynamic loads, by the time the double shell composite wall is a part of the bearing structure.

The purpose of this study is the comparison of the results of the experimental specimens by the numerical analysis and of various formulations of beam-to-column steel joint of the frame. By this comparison is investigated the influence of various changes of the joint based on the arrangement of bolts and plates and the limits of welds. It is noteworthy that this study is conducted in the context of an experimental investigation (see Figure 2).

In a first step, the formulation of the steel joint is assumed and simulated as it was during the experimental investigation and in a second step various changes introduced to the computer numerical models and the results from these two different procedures are being compared [14].

2 DESCRIPTION OF THE STRUCTURAL PROBLEM

2.1 Design aspects of the proposed double-shell composite wall module

The proposed study presents a comprehensive experimental research project on the seismic response of a novel Precast Reinforced Concrete Panel Infilled Steel Frame (PRCP-ISF) as a structural module, performed by a research team at the department of Civil Engineering, Aristotle University of Thessaloniki. The Precast Reinforced Concrete Panel (PRCP) is a composite structural system apart of double concrete thin wall with a gap inside which is filled by insulated material and distributed steel vertical frame RHS sections (see Figure 2).

Figure 2. Out of plane bending test on the elementary module of double-shell composite wall

The inside sited steel sections in this system works as a mechanism to enhance the buckling strength [10] of the double thin walls. The composite behavior is covered by the interconnected between steel members and the respective walls distributed steel shear bars. The wall module attached to the columns of the steel frame through bolted joints providing access for easy assembly and service for a prefabricated optimal building construction system for low rise buildings.

2.2 Prefabrication procedure of the structural system and the role of joints

The basic concept of this novel system is to increase the robustness of the structure under seismic hazard and to reduce the weight of the used structural steel members by optimizing the structural system. During loading, the PRCP-ISF system develops a resisting mechanism which is complex and directly dependent on its constituent elements, i.e. the PRCP system and the steel frame bolted joints. The bolted connections were chosen as they are more suitable for easy construction, service and fast removal of the damaged PRCP after an earthquake hazard. Therefore, in order to obtain a clear perspective of its structural performance, an experimental campaign included tests at specimens of system components providing under consideration their monotonic and cyclic behaviors together with failure modes for each critical case.

2.3 Description of the beam-to-column steel joint of the system

In order to optimize the structural design of the system, the possible configurations and their bearing capacity of the beam-to-column steel joint of the system are studied and evaluated separately of the overall module system. More specifically, the connecting steel members of the presented system i.e. beam and column, in a corner configuration representing the upper joint configuration of the system and are formed both by HEA100 (steel grade S235) sections.

They were connected using two endplates of which the one was welded to the beam and the other was welded on a supplementary member (for serviceability reasons) of a HEA100section and a small of 70mm length equal to the designed gap. This member was welded on a third endplate which was, also, welded on the column steel member. These three used end plates had the same dimensions of 150×220×10mm (see Figure 1). In addition, two stiffening plates were welded on the column web and flanges underneath the end plate so that any local failure due to concentrated loading to be prevented. The welding configuration composed of fillet welds with 235MPa nominal yield strength and a_w=5.0mm throat thickness in line with EN1993-1-8 [4] requirements. It is noteworthy that after welding, an ultrasonic technique was used to ensure whether the quality requirements of welding are fulfilled.

In order to obtain a moment resisting beam-to-column joint the bolted part of the joint designed to work as T-stud mechanism in tension and the welded part of the end plate to the flange of the column designed to resist to bending by normal and shear stresses respectively.

3 STRUCTURAL PERFORMANCE ASSESSMENT OF JOINT BY CYCLIC TESTS

The performance based structural design is an optimal design procedure for a steel frame structure. Strength and deformation capacities of a structural component depend on cumulative damage of past damaging events. Thus, performance depends on the history of previously applied damaging cycles, and an easy way to assess the consequences of an event is to replicate the load and deformation histories to a component will undergo in a seismic event. This procedure includes quantification of performance of the enhancement systems of the structure like bracings, joints etc. by an evaluation of cyclic loading histories as an equivalent seismic acceptance testing [9]. Cyclic tests are useful to provide basic information on steel elements or assembly behavior, including data on strength and stiffness characteristics, deformation capacities and deterioration behavior of specific details under consideration in large deformations [1]. A load deformation history unit consisting of positive and negative sequential excursions represents in general a cyclic loading on the component. A force-deformation history could be very useful because shows all important aspects of the response, including the point at which first yielding of material was detected. A strong earthquake will, in most case, lead to inelastic behavior in most of steel frame structures. In the design of moment resisting steel frames, the inelastic deformations may occur in the beam-to-column connection [8].

In order to define a loading history scheme able for capturing the critical issues of the structural performance of the beam-to-column joint under consideration, particular emphasis was given in the excursions in the inelastic range since they cause cumulative damage leading to ultimate limit state. An equivalent loading history that is conservative but statistically representative for capturing the most of the known ground earthquake motions for several structural configurations is the applied in the experimental an equivalent cyclic loading scheme. This loading scheme includes three cycles at each displacement amplitude (see Figure 3), where the yield displacement δ_y was used as reference for increasing the amplitude of the loading cycles [5]. In this loading scheme the value of yield displacement δ_y as well as the maximum displacement $2.5 \cdot \delta_y$ which corresponds to failure at a strain of (ε=0.28mm/mm) was previously measured by laboratory tensile coupon tests.

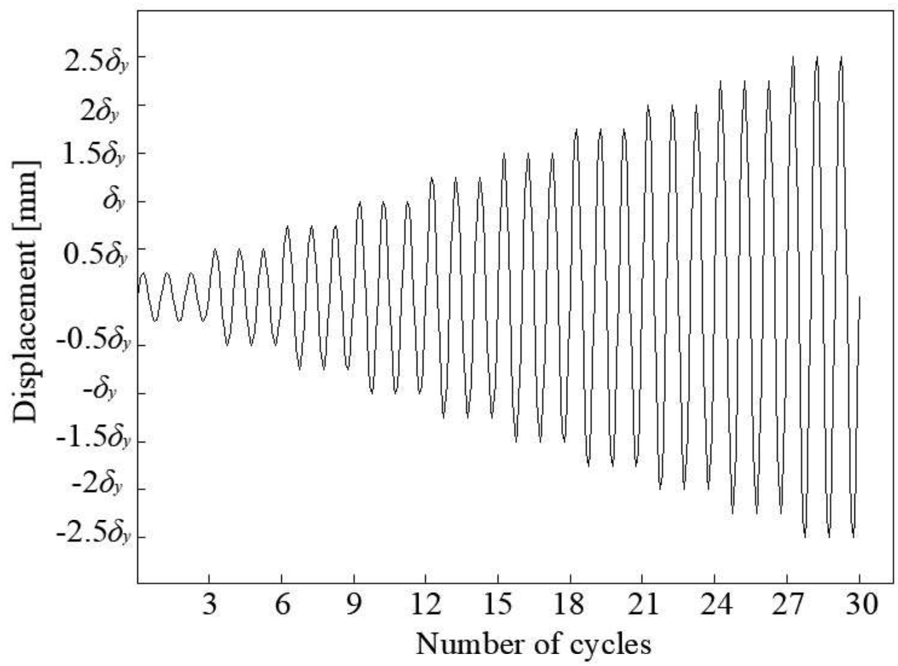


Figure 3. Applied cyclic loading scheme on the beam-to-column experimental and numerical test models

The same loading schemes were used in the numerical models (see §5) which follows the same variation and displacement amplitudes with the experimental ones. By applying the previously described cyclic loading scheme to the joint models under consideration the hysteretic energy dissipation capabilities of the joint were investigated as well as the mechanisms of failure was identified [12].

4 OVERVIEWS OF THE EXPERIMENTAL TEST PROGRAM

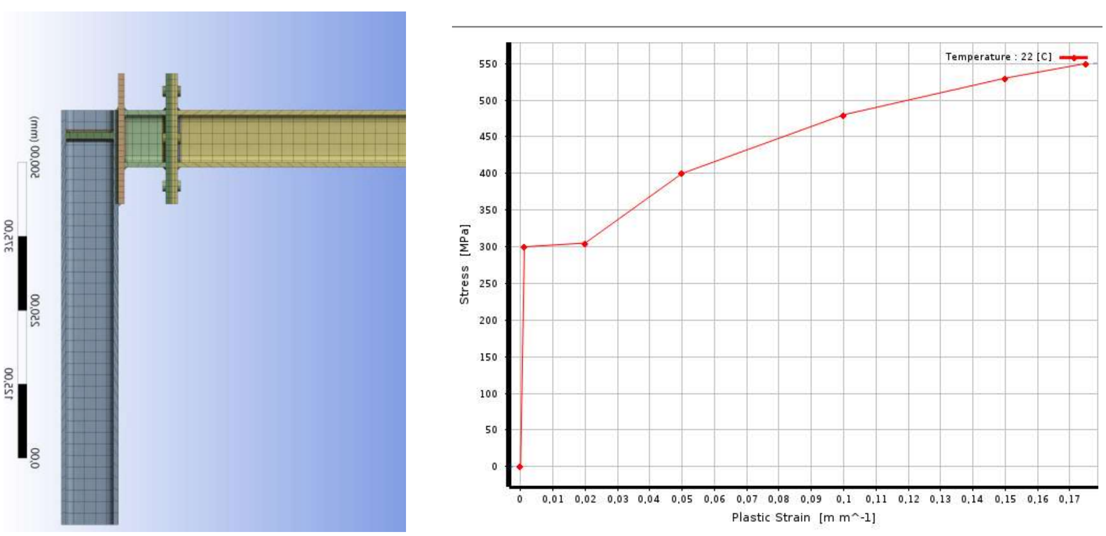
Two joint specimens named JL-4 and JL-6 respectively of an isolated part of the structure near the beam-to-column joint were tested in the Laboratory for Strength of Materials and Structures, Dept. of Civil Engineering, Aristotle University of Thessaloniki [15]. On one hand, the specimen JL-4 configured with end plates, where placed two lines of2 bolts each one, over and under the respective flanges of the beam. On the other hand, the specimen JL-6 configured with an increased stiffness where between the end plates placed three lines with 2 bolts each one. The two lines placed at the same positions with the JL-4's configuration and a third line placed at the position of axis of the beam. This joint model seems to resist more efficiently to positive or negative active bending moment by a T-stub mechanism as well as to shear forces acting on the beam.

A reaction frame anchored on the laboratory strong floor and a hydraulic MTS actuator with 2500kN maximum capacity, were used for the execution of the cyclic tests on the JL-4 and JL-6 (see Figure 7(a)) specimens respectively. The testing procedure which was followed in the respective cases of specimens was identical. The selected boundary conditions for each specimen in the Laboratory, for the column edge was fixed whereas the beam edge was free to move in order to represent identically isolated parts of the overall structure. The employed instrumentation comprised of a number of LVDTs to measure various displacements of the joints steel members and based on the previously presented (see §3) cyclic loading scheme. The hydraulic actuator was driven by displacement control method and applied quasi-static cyclic loading. The test was terminated upon joint's failure occurrence.

The obtained force-displacement and moment-rotation curves discussed in §5 in comparison with the respective numerical one. For both specimens the corresponding hysteresis loops show a stable response against in-plane bending and satisfactory energy dissipation capacity but the results of JL-6 specimen where its failure mode predicts a slow and gradual deterioration.

5 FINITE ELEMENT ANALYSIS

The main object of the beam-to-column steel joint design is to design a robust component of the structural frame system with an easy assembly procedure and a low construction cost. Numerical tests were carried out by ANSYS software with 3-D finite element non-linear analysis.



(a) (b)

Figure 4. Numerical model of the JL-6 beam-to-column joint (a) and material low of the steel joint components (b)

Eight nodes SOLID185 elements were used to simulate all the structural components of the joint including bolts and welds. Contact surfaces using CONTA174 elements between the end plates, the bolt shanks and end plates with

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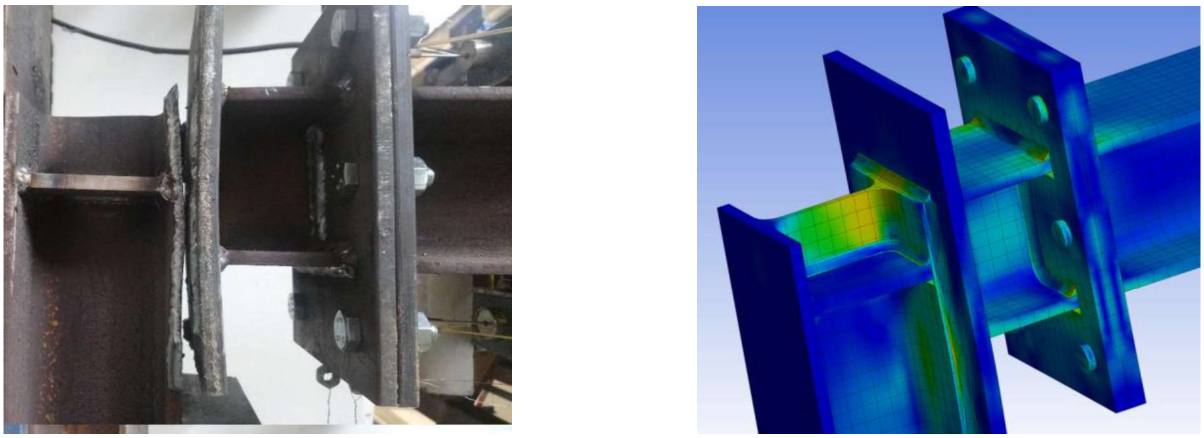
a usual frictional low in respect with the used materials was introduced to simulate the joint behavior and to confirm the experimental results. Moreover, this procedure helps to extend the analysis to parameters that can be more easily numerically evaluated as well as to reduce the number of required tests due to the high cost of laboratory tests. Such a systematic investigation provides a detailed insight on the basic features of the elastic and inelastic behavior of the joint and of the load transmitting mechanism that leads to a better understanding of the system's response, accompanied by a variety of original results.



(a) (b)

Figure 7. Typical deformation and failure mechanism of experimental JL-6 joint specimen (a) and the respective numerical Model 1 (b)

The initial joint model for numerical investigations has the same geometrical properties (see Figure 4(a)), the same material properties (see Figure 4(b)) and the same loading conditions and constrains as the experimental specimens (see §4).

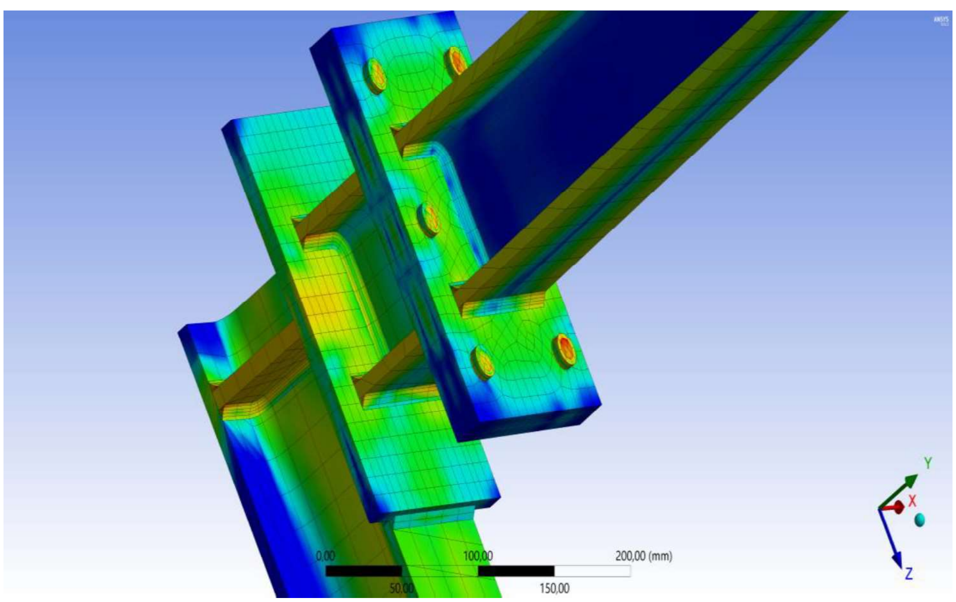


Figure 5. Von-Mises stress distribution for maximum deformation on Model-1 (JL-6) due to vertical cyclic loading

In the mesh process of the numerical models, the size of finite elements of the joint's components was taken into account, where the welds and bolts discredited with a denser mesh than the beam and column members. Moreover, in

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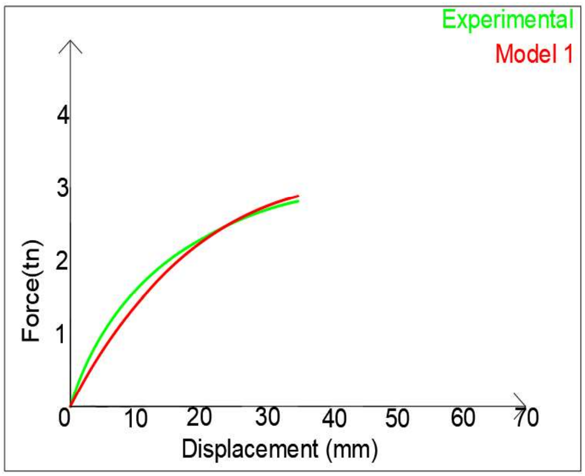
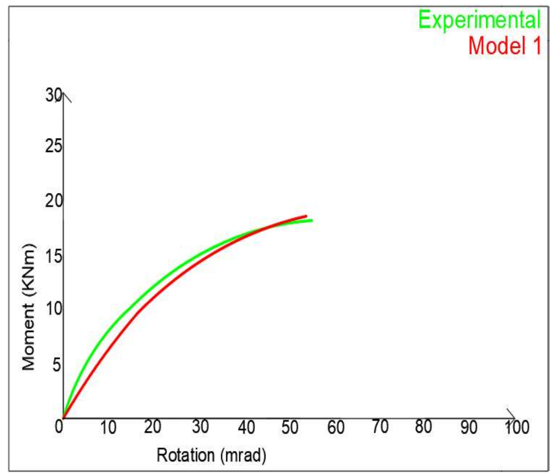
order to investigate the influence of the double shell composite wall of the steel frame on the bearing capacity of the beam-to-column joint the numerical models were additionally tested on cyclic loading schemes in vertical direction. Taking into account the results of Model 1, which represents the JL-6 experimental specimen, it is clear that the stress arrangement at the end plates and the bolts of the system remain under the yielding point of the used steel grade (see Figure 5).

On one hand, for a maximum horizontal deformation of δ_y=70mm, the stress in the welds of the end plate with the steel column are slightly beyond their breaking limit (see Figure5), so it can be assumed from the numerical investigation that the bearing capacity of the joint reduces obviously and the structural system is failed. Moreover, it is observed that the finite element analysis results showed fairly agreement with the experimental ones in terms of strength, deformation and unloading stiffness. The two confirmed curves are close enough to assume that the Model 1 and the respective experimental JL-6 is reliable (see Figure 6(a), (b)), where the moment capacity becomes equal to $M_{1H,max}$=18KNm.



(a)



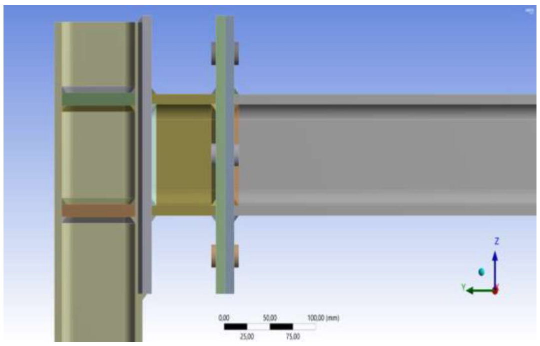
(b)

Figure 6. Evaluation force-displacement (a), moment-rotation response (b) of JL-6 model under a horizontal cyclic loading scheme

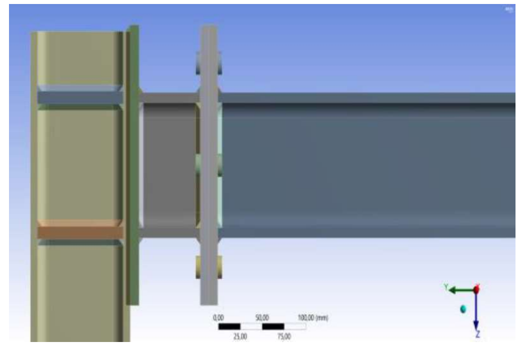
On the other hand, the overall response of the joint model due to a composed diagonal acting force as a result of the horizontal and vertical cyclic loading schemes actions respectively does not differentiate the results of the experimental tests at the critical bearing capacity assessment points.

The bearing capacity of the specimen JL-6 seems to be much higher that the respective of JL-4 one and in the following alternative configurations only the quantities of JL-6 model and the respective numerical model named Model 1 will be analyzed and presented. In addition, however, because of the convenience of using numerical analysis modeling to reduce the cost for experiments in the laboratory, alternative forms of the beam-to-column steel joint under investigation were simulated. In these changing forms are taking into account critical design components that appeared to degrade the bearing capacity of the joints.

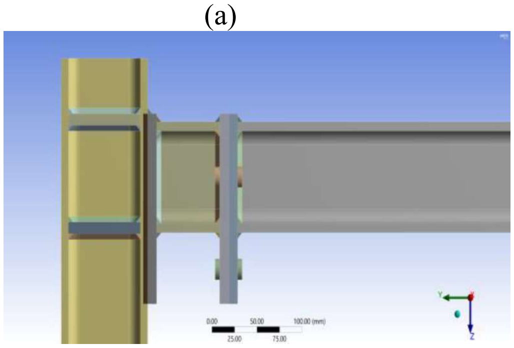
Thus, with respect to Model 1 in respect to the experimental JL-6 one, Model 2, Model 3, Model 4 and Model 5 were formed. In Model 2 configuration, the length of the column was increased so that full contact with the corresponding welded end plate was achieved and 2 symmetrically distributed stiffeners are added in respect to centroid one (see Figure7(a)). In Model 3 the configuration of Model 2 was kept as described previously because the response of the joint was significantly improved, but the cross section of the members was increased from HEA100 to HEA120 (see Figure 7(b)). At the same time, an alternative model was examined, Model 4 where the upper line of bolts is absent from the configuration of Model 2, so the connection seems to be a shear connection or to be a semi-rigid connection (see Figure 7(c)). Finally, Model 5 was examined, which contains all the features of Model 4, but the overlying concrete slab of the building's composite configuration also participates in its loading bearing system (see Figure7(d)).



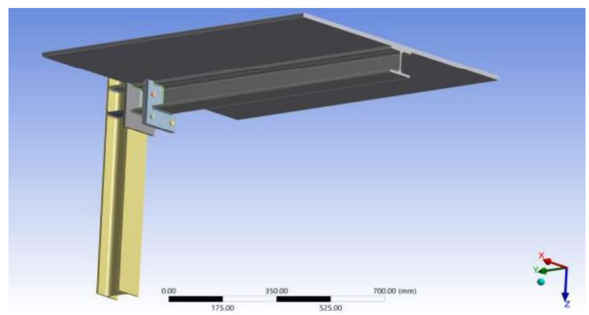
(a)



(b)



(c)



(d)

Figure 7. Alternative joint configurations of Model 2 (a), Model 3 (b), Model 4 (c) and Model 4 (d)

All these models were compared in order to obtain features that lead to an optimal design and to assess the presence of other elements (such as the concrete slab) in the beam-to-column steel joint response.

6 PARAMETRIC EVALUATIONS OF NUMERICAL ANALYSES

The change of Model 2 in respect to Model 1 refer to raising of the height of column to cover the top of the welded end plate as well as an increase on the number and the arrangement of the column stiffeners. A much better distribution of stress among the elements of the steel joint can be noticed in the second model as more welds reach at the same time their breaking limit. For maximum the displacement of cyclic loading scheme during this analysis, maximum stress appeared on weld elements is equal to Model 1 but now their extension they capture is much less and are restricted only at small areas (see Figure 11). All other joint components show less stress distribution, so the changes, that were made, affect the joint in whole with a better and more normal distribution of stress. The bearing capacity of this confirmation of the joint is expected to be higher than the first model and it is estimated to a higher failure displacement (around 80mm) at the end of the beam as well as the results of load-displacement response and moment-rotation response are shown below (see Figure 8). The joint is much stiffer now and the maximum moment that the joint can undertake has increased from 18KNm to 22.5KNm.

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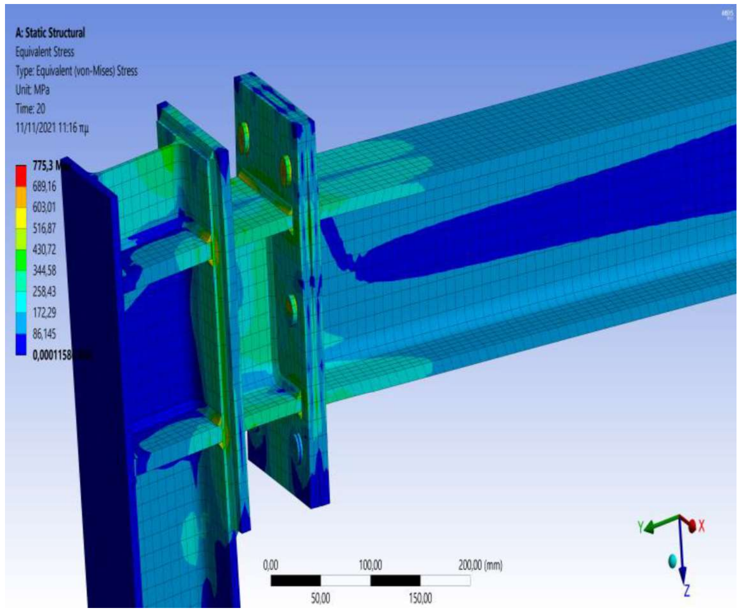

Figure 8. Von-Mises stress distribution on Model-2 for maximum deformation under cyclic loading scheme

The geometry of Model 3 is exact the same with that of Model 2, but the steel section of the column and the beam has converted from HEA100 to HEA120 which could be used in order to upgrade the structural response of the system. The numerical results show that the distribution of stress is similar with that of Model 2, as expected to be, because the geometry of the steel joint has not changed at all. Weld elements reach again all together their breaking limit and they show maximum stress similar to Model 2 for the same displacement critical displacement. Bolts on the other hand show a stress increasement on this model, beyond their yield point but not over their ultimate tensile strength. Moreover, the results of the load-displacement response and the moment-rotation response of the system show that the steel joint is indeed much stiffer than previous models and it can undertake even bigger moment than the second model equal to 29KNm (see Figure 9(a), 9(b)).

The last Model 5 has the same configuration like Model 4, but the concrete slab of the composite configuration of the floor system, above the main beam is added. It is estimated that the concrete slab is going to affect a lot of the previous semi-rigid behavior of the joint, making it much stiffer (see Figure 8), where the stress arrangement around the joint components viewed without the concrete slab to better presentation of the distribution on the steel elements). The joint's behavior has changed compared with the other models because it has become again rigid as it was in Model 1, Model 2 and Model 3.

The changes that were made in Model 4 in respect to Model 1 refer to cutting a part of the two end plates that are connected with bolts, as well as the respective line of two bolts that were also connected there. So now the bolts are in total 4 and not 6 that were in the previous Model 1. The steel sections of column and beam are HEA100 as they were in Model 1. That formulation of the steel joint is being studied because in some cases it is offering construction facility. The joint's behavior has changed compared with the other models. More specifically, the joint has become semi-rigid in contrast with the previous models that the joint was much rigid. The joint is unable to undertake big moments resulting in big displacements for small amount of force. One weld component seems to appear maximum stress and it is the one connecting the end plate with the mail beam and is observed at the bottom flange of it. For maximum displacement of 70mm of cyclic loading scheme, the stress on the critical weld exceeds its breaking limit. The results of the load-displacement response and moment-rotation response of the system show that the joint has become much less stiff and the maximum moment it can undertake is about 8,50KNm.

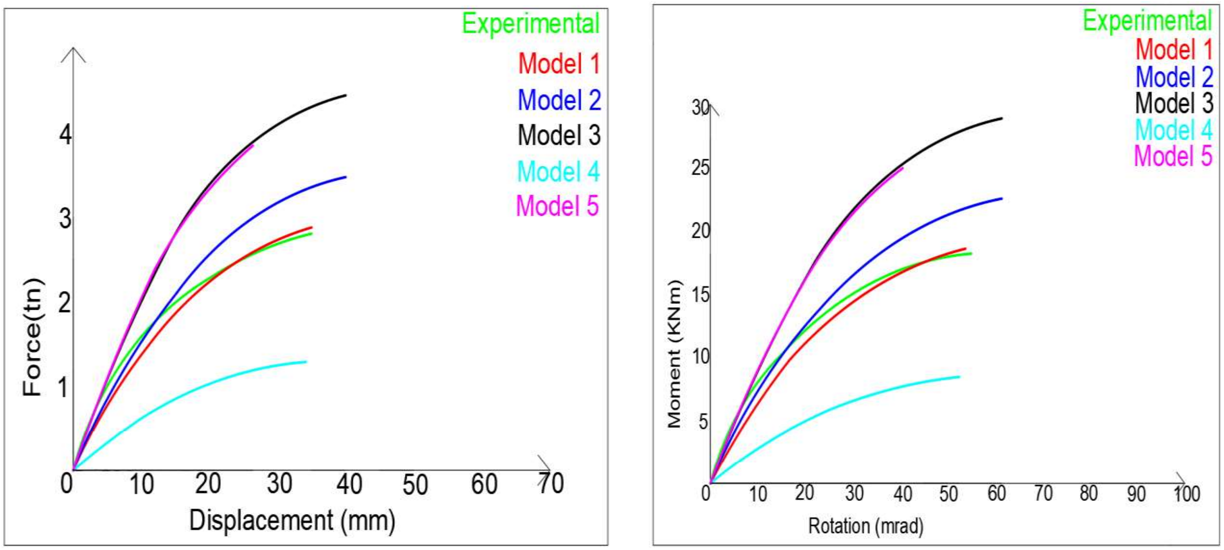

Figure 9: Load-Displacement (a) and Moment-Rotation (b) response for several numerical joint models in respect to the initial experimental JL-6 specimen.

Maximum stress is observed on weld elements and for maximum displacement of 70mm under the cyclic loading scheme the stress overtakes the breaking limit of welds. Thus, it is estimated that the joint fails earlier for a smaller displacement. It is assumed that joint fails for displacement of 49mm after looking carefully at the results, because that is the point when weld elements reach their breaking limit. All other elements appear the same time with much less stress under the yield limit. The results referring to load-displacement response and moment-rotation response show that the joint is much stiffer than of Model 4 and it can undertake bigger moment. These are very interesting results, because in the semi-rigid Model 4 the joint was observed being capable to undertake a moment of only 8,50KNm, but with the addition of the concrete slab that moment increased three times more to 25KNm, (see Figure 9(a), 9(b)).

7 CONCLUSIONS

The experimental and numerical studies are performed to investigate the behavior of beam-to-column steel joint of an innovative including double shell composite of steel and concrete PRCP-ISF frame system. The performance of the joint with respect to bearing capacity, geometrical alternatives and rigidity are examined. The validity of the experimental results is confirmed through numerical analysis one. 3-D nonlinear finite elements are established to analyze the mechanical properties of alternative joint configurations.

The load displacement curves of the numerical analyses are in good agreement with those of the experimental tests in terms of ductility, strength and hysteretic behavior as well as the respective failure modes after the stress fractures of welds between the end plate and the respective flange of the column. The whole investigation shows that in any case the experimental test specimens must be checked with corresponding numerical one and vice versa in order to obtain a reliable joint configuration.

In order to investigate the influence of the double shell composite wall of the steel frame on the bearing capacity of the beam-to-column joint the numerical models were additionally tested on cyclic loading schemes in vertical direction. The overall response of the joint models due to a composed diagonal acting force as a result of the horizontal and vertical cyclic loading schemes actions respectively does not differentiate the results of the experimental tests at the critical bearing capacity assessment points. So, the configuration of joint taking into account the direct connection of the double composite wall panel on the upper corner using the lower line of bolts of the joint is not expected to cause out of limit deformations and stresses on the joint's component.

Furthermore, according to the last numerical analysis under investigation model, the influence of the concrete slab in a composite floor configuration was observed. The results were very interesting mainly due to the fact that after the addition of the concrete slab, the joint was able of carrying a much bigger moment than the previously formed as shear or semi-rigid configuration. This important influence needs to be studied furthermore both experimentally and computationally.

Lastly, this paper is also proving the very good overlapping between experimental and numerical results. Confirming at first place some experimental results therefore also the numerical results continuing with more numerical results of alternative configurations gives to them more reliability. Furthermore, the cost of conducting many experiments is much more than numerical analyses, so a lot of cost is saved that way.

8 ACKNOWLEDGMENTS

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