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Numerical Modelling of Column-Base Connection Solutions for Steel Over-roofing Systems

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Abstract

Over-roofing systems have achieved larger and larger popularity in the last decade, becoming one of the top solutions preferred by construction engineers, not only for enlarging the existing living space of the present building stock, but also for its refurbishment by reducing the amount of materials and work on site. Furthermore, steel-intensive solutions are the ideal systems for over-roofing these existing large precast concrete panel buildings due to their lightness, reversibility and clean sites. Also, they can adapt to existing structural systems and several structural typologies can be thought. Thus, the scope of this paper is to present three types of over-roofing structural solutions based on intensive use of steel elements: a) hot rolled steel; b) rectangular hollow sections; c) cold-formed steel sections. Moreover, a numerical modelling analysis was considered for all three types of over-roofing column-base connections, both as rigid or semi-rigid connection, in order to determine the optimum solution.

Keywords: existing residential buildings, large precast reinforced concrete panels, over-roofing, light steel structure, rigid or semi-rigid connections, FEM, numerical modelling.

1. Overview of existing building stock in Romania

According to The National Institute of Statistics on March 18th 2002, 4.234.173 households were built in the urban area, from which 3.021.122 households in blocks of flats. From these, 81.964 units (blocks) are made out of large precast reinforced concrete panels (see Figure 1) [1]. These large prefabricated blocks were built in three main periods, i.e.: (a) 1962-1975; (b) 1975-1982; (c) 1982-1989 [2].

Furthermore, referring to the current status of residential areas, some major urban dysfunctions can be pointed out: (1) lack of green spaces and parking; (2) lack of concern for the maintenance of overall building-facades, cornices, balconies; (3) non-unitary rehabilitation interventions (the attic) of the assemblies; (4) abusive extensions of buildings at ground level; (5) the interior reduced surfaces of the flats and poor space partitioning (see Figure 2).

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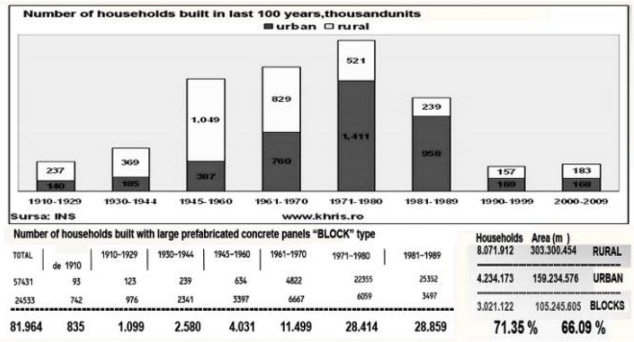


Figure 1. Residential building stock in Romania

Figure 2. Major urban dysfunctions in Romania

2. Why over-roofing solutions?

The structural renovation of existing buildings takes various forms: adapting the internal space, creating additional floors; and remodelling the facades and roof. Renovation includes improvement of weather-tightness and thermal insulation. The techniques known as 'over-cladding' and 'over-roofing' create a new building envelope, and dramatically improve the quality and usefulness of the building. Steel construction is often the only solution to complex renovation problems, as it combines the benefits of lightweight components, flexibility in planning, long span capabilities, robustness and durability, with economy and speed of construction on site [3]. Considering this situation, the refurbishment of buildings by adding a usable living space level has become an usual method of enlarging the existing space worldwide (see Figure 3).

Figure 3. Over-roofing and over-cladding renovations: (a) Italy; (b) Newport City homes [4]

Although in Romania, in the last decade, a lot of standards and guidelines concerning the structural, technological and energetic efficiency for these interventions have been elaborated, some aspects regarding urban planning and exterior aesthetics of new-added storeys have been ignored when applied in reality. Therefore, in present context, there is a large variety of geometries, volumes and aesthetics for the over-roofings and over-claddings even in the boundaries of the same neighbourhood, degrading the overall visual image. Also, the main errors pointed out in the execution of these over-roofings can be divided in two categories: the first regards the bearings and anchorage of the over-roofing system to the buildings structure of resistance, while the second is related to the actual structural errors.

3. Steel solutions for over-roofing

The proposed solution for over-roofing was considered for the block typology E744.R (built in the period 1962-1975), a representative typology for these large prefabricated blocks, with a great appliance not only in Timisoara, but in the entire country. The building was verified according to present standards, proving to be resistant to all loading conditions and different refurbishment strategies were considered for enlarging the interior living space, in earlier papers [2]. These studies showed that, by removing the top thermal insulation layers from the terrace, the adding of a supplementary over-roofing floor would not affect the concrete structure's resistance or stability. Thus, steel was the material chosen for this over-roofing study, due to its lightness, reversibility and clean site.

The over-roofing system was analysed by using the finite element program ETABS 9.7. The following geometry was considered: 3 m height, 9.6 m span and 7 longitudinal frames of varying lengths between 4.2-3.3 m. In order to have a homogenous aspect, the spans and interior compartments of the building were preserved (see Figure 4).

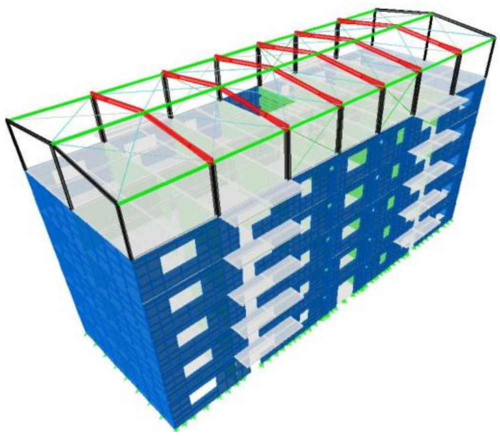
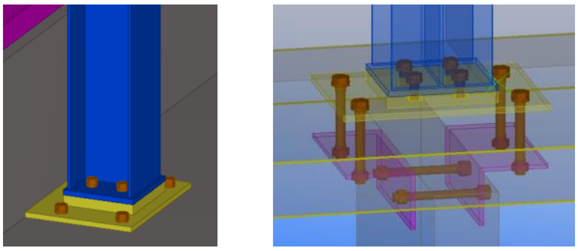


Figure 4. Over-roofing (3D view)

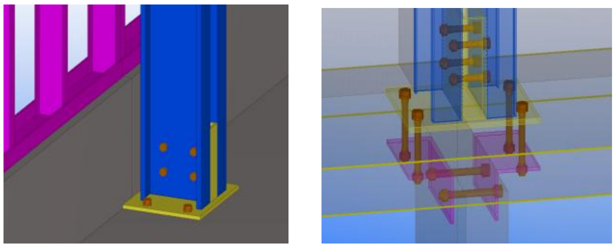
The over-roofing was designed in accordance with the following standards: (1) EN 1990-1-1-2004 "Design code. Basis of structural design"; (2) EN 1991-1-1-2004 "Design code. Evaluation of load actions on structures"; (3) EN 1993-1-1-2004 "Design of steel structures"; (4) EN 1998-1-1-2004 "Design of structures for earthquake resistance". For the seismic analysis of the over-roofing system, the design gravitational acceleration specific to the region of Timisoara was chosen, $a_g = 0.20g$ and a behaviour factor $q = 1$, specific to non-dissipative structures.

Three types of solutions of over-roofing based on intensive use of steel were chosen: (a) Over-roofing solution based on the use of hot-rolled profiles (IPE beams and HEB columns); (b) Over-roofing solution based on the use of rectangular hollow sections; (c) Over-roofing solution based on the use of cold-formed sections.

Regarding the base connection, pinned/semi-rigid and rigid solutions were taken into account for all of the 3 cases. In the first case scenario (a), the hot-rolled HEB profile is connected to the concrete slab through 2 rectangular steel plates, a smaller one with similar dimensions to the profile, welded over a larger one at the base. The first plate is connected to the base plate by two rows of bolts, placed along the web of the profile, while the second plate anchors the column to the last floor concrete slab through 4 bolts placed in its corners as it is shown in Figure 5a. In the third case scenario (see Figure 5b), the columns are also connected to the base concrete slab through a rectangular steel plate, but in this case a lateral connection was needed for proper stability. Thus, a U profile was placed between the two cold-formed C profiles. The connection is done by bolts, both laterally as well as at the base. The wall covering is placed on the attic and supported by a cold-formed C profile pier framework. In all three cases, the column-base connection is strengthened by an angle profile placed under the last floor slab and connected to the steel plates above by a row of two bolts, and respectively to the last floor wall also by two bolts (see Figure 5). The second case scenario, the rectangular hollow tube was not presented as it is a connection more or less as the first case scenario.



(a)



(b)

Figure 5. Column-base connection details case 1 and case 3 respectively

4. Numerical analysis

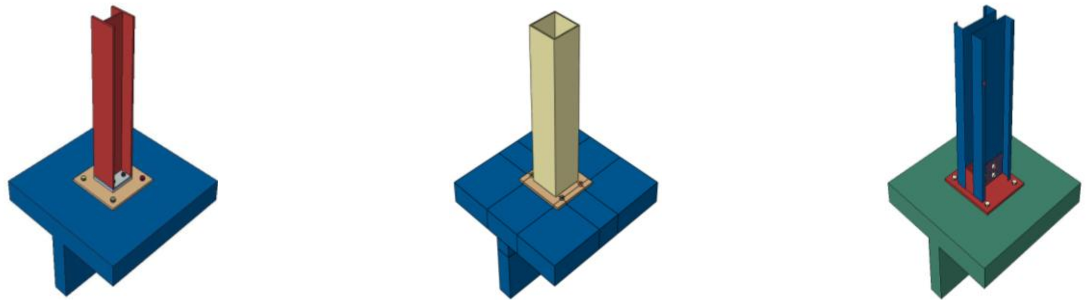
In order to prove the efficiency of the base connections chosen for the study, a numerical simulation was done using FE software ABAQUS 6.7 [5]. Three cases were considered, in semi-rigid and rigid solution as presented in Figure 6.

SHELL Element of S4R type, with 4 nodes, reduced integration, 6 DOF per node was used only for modelling the cold-formed sections in the third case scenario, while for the other sections, in all cases scenarios only BRICK Element of C3D8R type, with 8 nodes, reduced integration, 6 DOF per node, was used

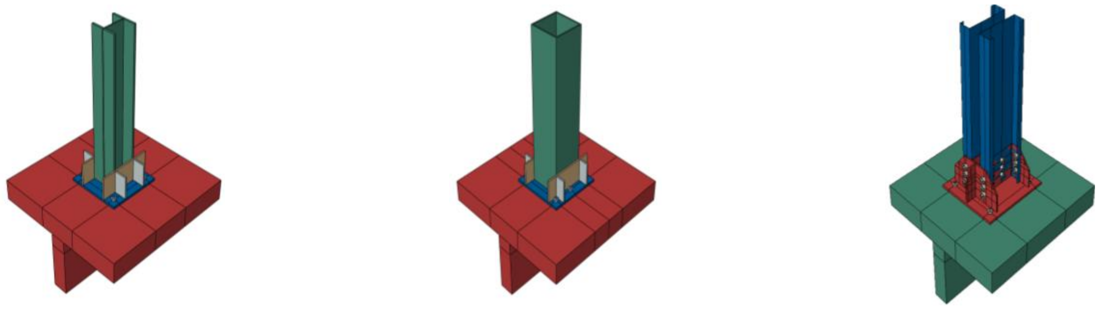
The general contact between elements was chosen as “All with self”, with HARD CONTACT on the normal direction and without friction on the transversal direction. Some constraints were also imposed in order to simulate the welding contact between plates.

A maximum lateral displacement load of 200 mm was imposed on the column in order to obtain the maximum stresses of the connections and the force-displacement curves. The analysis used was nonlinear using dynamic explicit steps as quasi-static.

The first numerical model (see Figure 6a) is a semi-rigid connection with two base plates, connected with two groups of bolts, i.e. M16 and M20. The column is a hot-rolled HEB180 profile. The first plate is welded on the contour to the base plate connection and the bolts that make the connection with the column are countersunk bolts.



a) Semi-rigid hot-rolled profile b) Semi-rigid rectangular hollow section c) Semi-rigid cold-formed section



d) Rigid hot-rolled profile e) Rigid rectangular hollow section f) Rigid cold-formed section

Figure 6: Semi-rigid and rigid base connections

The entire connection is linked to the existing building through a base plate of 390 mm length and a thickness of 25 mm and through angle profiles. For the rigid connection (see Figure 6b), the detail is simplified by eliminating the intermediate base plates and the countersunk bolts, only that the welded plates and the countersunk bolts were eliminated and, supplementary stiffeners of 150 mm height and 10 mm thickness were added.

The second numerical model (see Figure 6b and 6e) is also analysed as semi-rigid and rigid connection, but with only one base plate. The column is a rectangular hollow section 220×220×8 mm. The dimensions of the base plate are 460×420×25 mm. The bolts and angle profiles are of the same dimensions as the previous case; only the bolts' distance being adapted to the new conditions. The rigid connection follows the same pattern as the previous case.

The third numerical model (see Figure 6c and 6f) is also analysed in both cases, as semi-rigid and rigid connection, with one bottom plate. The column is a cold-formed section of two back to back C-profiles of 300 mm height and a 2 mm thickness. Two rows of M16 bolts were used along the web. The geometrical characteristics of the base plate and the angle profiles are the same as for the previous case. Additionally, a metallic box section of 6 mm thickness and 150 mm height was placed between the two sections of the column in order to improve the capacity of connection. For the rigid connection, extra stiffeners and supplementary bolts were added to connect the flanges of the profiles.

The steel material used in the connection was presumed to be elastic perfect plastic. Also, the concrete was input elastic perfect plastic features as seen in Figure 7 [6]. The option used to simulate concrete plasticity was Concrete Damaged Plasticity.

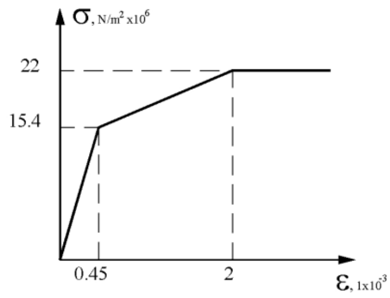
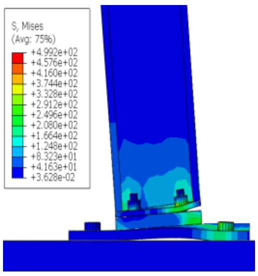
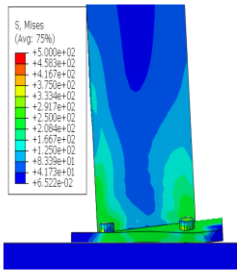


Figure 7. Elastic-plastic model of concrete [6]

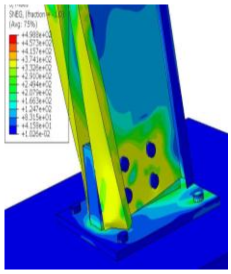
The following results were obtained: (1) the first three connections were modelled as semi-rigid but present mostly a pinned behaviour; (2) the performance of the three case scenarios was obtained under lateral loading; (3) after numerical analysis the load transfer mechanism was identified; (4) the weak components were pointed out, which are presumed to yield first; (5) plastic efforts and deformations were obtained; (6) the maximum stresses for the semi-rigid hot-rolled connection was reported in the countersunk bolts; (7) the welded plate of the semi-rigid hot-rolled profile presents a slight plastic deformation (see Figure 8a); (8) the maximum stresses of the rectangular hollow section connection was noticed in the bolts and on the corresponding holes (see Figure 8b); (9) the maximum stresses for the cold-formed connection was reported in the upper flange of the section on the direction of loading, but also in the holes of the web (see Figure 8c); (10) for the third case scenario (rigid solution), the plastic stresses achieve maximum in the web height; moreover, the phenomenon of crippling occurs in the upper flange (see Figure 8f).



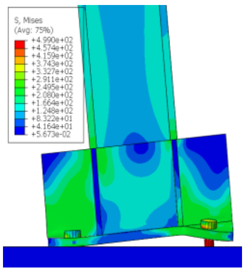
a) Semi-rigid hot-rolled profile



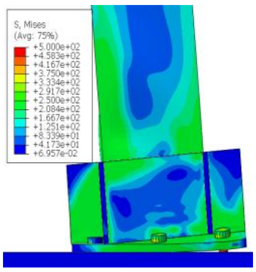
b) Semi-rigid rectangular hollow section



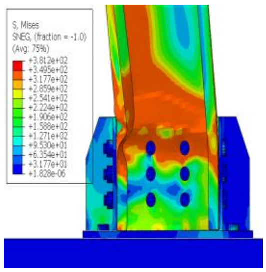
c) Semi-rigid cold-formed section



d) Rigid hot-rolled profile



e) Rigid rectangular hollow section



f) Rigid cold-formed section

Figure 8. Plastic strains in the semi-rigid and rigid based connections

The concrete floors did not reach plasticity, not even in full loading conditions; the stresses are still in the elastic range, presenting only local effects at bolt-hole interaction zone (see Figure 9).

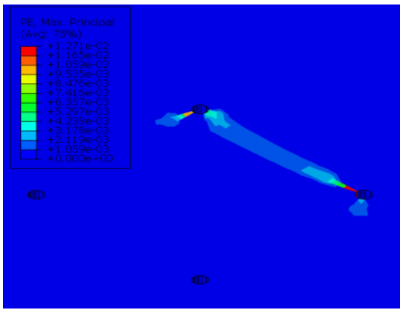


Figure 9. Strains in concrete on bolt hole

The following force-displacement curves were obtained for each case scenario:

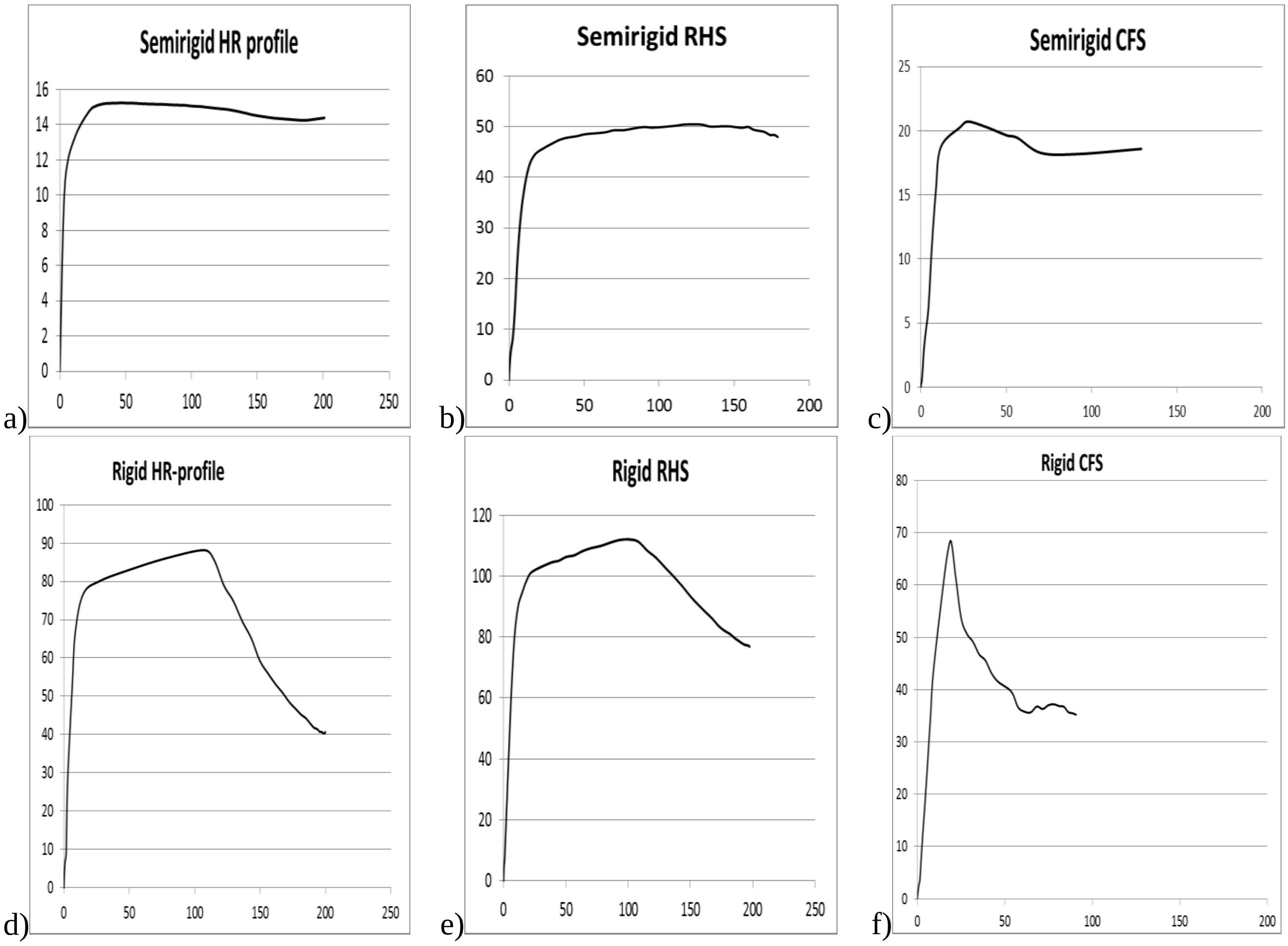


Figure 10. Force-displacement curves in semi-rigid and rigid based connections

The above force-strain curves show the following partial conclusions: (1) the results show no significant results after the threshold of 150 mm, the elements entering in failure; thus, the load imposed to the columns initially should be reduced; (2) as expected the maximum yield capacity was obtained in the second case scenario (the rectangular hollow section connection), for both semi-rigid and rigid solutions(see Figure 10b and 10e); (3) the first and third case scenarios present almost the same capacity, in both semi-rigid and rigid solutions, with the difference that the hot-rolled profile connection presents a higher ductility than the cold-formed section (in rigid solution), which can only behave well in the elastic domain, as expected for a class 4 section (typical behaviour); (4) the semi-rigid solutions present, as expected, smaller loading capacities, but the intention of using theses solution was to minimise the work on site, which was not confirmed; (5)

the stress level on the existing concrete floors and the additional walls are small and do not introduce supplementary stresses over the existing capacity.

The scope of the numerical program is a better understanding of the behaviour of the six types of node connections and to find the optimum solution for a base connection for this kind of over-roofing systems that will not bring additional efforts to the existing concrete panel building. This numerical study is actually a process of finding the optimum solution and it will be followed by a process of parametric study, and last but not least laboratories study of these kinds of connections.

5. Conclusions

The study presented in this paper shows the possibilities of connecting the over-roofing to the existing buildings by using steel-intensive solutions. The numerical analyses show the following: (1) steel-intensive solutions are ideal systems for over-roofing the existing large precast concrete panel buildings due to their lightness, reversibility and clean sites; also, they can adapt to existing structural systems and several structural typologies can be thought; (2) the numerical program shows that all types of connections presented in semi-rigid and rigid solutions can withstand a considerable amount of loading before failing; (3) however the main concern remains to minimise the structural intervention on the existing concrete structure. Finally, supplementary numerical analyses and experimental work will be done in order to validate the proposed solutions.

Acknowledgements

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