Bulletin of the *Transilvania* University of Braşov
CIBv 2015 • Vol. 8 (57) Special Issue No. 1 - 2015

# NUMERICAL ANALYSIS OF THIN-WALLED COLD-FORMED STEEL COLUMN-BASE OVER-ROOFING SOLUTIONS

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***Abstract:*** *In an increasingly developing world, in which sustainability has become not only an option but more of a demand, over-roofing of existing buildings, together with over-cladding form an optimum complete retrofitting solution. In this context, the paper is mainly focused on numerical analysis of cold-formed steel solution, in both semi-rigid and rigid solutions used for addition of new storey. Also, for ease in erecting, an option of column-base connection with chemical anchorage was analysed.*

***Key words:*** *light steel structure, small building system, over-roofing, over-cladding, retrofitting solution.*

## 1. Introduction

More and more buildings nowadays are being subjected to the process of refurbishment and conversions. Even more, taking into account the sustainability criterion which has become a very important factor for construction works, there is an increasing tendency of choosing refurbishment of existing buildings rather than building new ones [4].

According to the European Commission, buildings are responsible for 40% of the EU energy consumption and 36% of CO2 emissions. This is the reason why improving the overall energy efficiency of buildings is crucial for the overall resource efficiency of the European economy [3].

Over-roofing is defined simply as the installation of a new roof on an existing building, creating new habitable spaces. However, over-roofing is frequently combined with over-cladding, thus, forming a complete retrofitting solution.

The main advantages for over-roofing, in view of RFCS Project RFSR-CT-2007-0043 [8], include: heat reduction by improving thermal insulation, enhancement and modification of the overall building appearance, creation of new space for habitable use or building services, arrestment and overcoming of deterioration of the existing roof and of leakage problems (by a new pitched roof), avoiding disruption that is inevitable in the replacement of an existing roof.

From structural point of view, over-roofings add an extra weight to the existing building, and thus, rechecking the load-bearing capacity of the original structure is

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a demanded measure for safety reasons, as to decide whether or not consolidation measures are needed. Nevertheless, in seismic countries, as Romania's case, this precaution measure becomes more of a demand, and traditional building techniques cannot be always used for this type of restructuring. Thus, in order to minimize the weight of the new storey structure added above, engineers choose to use steel based solutions because of their lightness in comparison with other materials (concrete, brick) (see Figure 1).

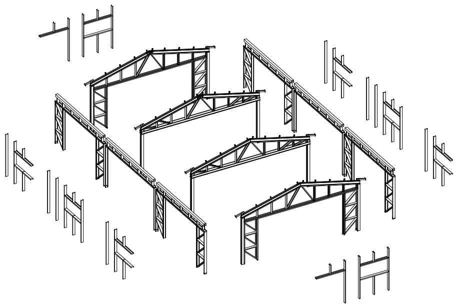
(a)

(b)

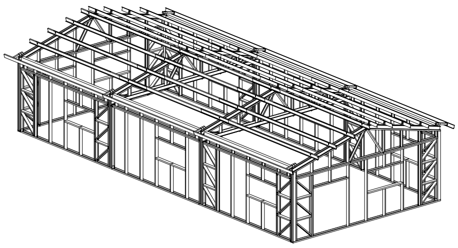
Fig. 1. *(a) Over-roofing of a single storey industrial building and (b) view of the new building façade [8]*

Even more frequent, nowadays, construction companies turn to light weight steel sections due to their numerous advantages: very good strength to weight ratio, fast and easy erection and installation, fully recyclable, adaptability and flexibility to architectural design, and, last but not least, proven optimal solution for seismic areas, due to their lightness. However, some aspects regarding structural issues must be controlled, namely stability problems due to their slenderness and complex detailing connections [5].

Moreover, over-roofing and over-cladding of existent buildings using these kind of light weight solutions becomes an optimal retrofitting technique as these small building systems are a prefabricated product that can be easily erected on sight, piece by piece (see Figure 2).



(a)



(b)

Fig. 2. *(a) Modular assembly of prefabricated components and (b) view of the small building system module [5]*

Therefore, what was designed initially as a modular structure for industrial purposes can also be transformed into an over-roofing, or a storey extension of an already existing building.

Referring to current building stock in Europe, there are about 196 millions of dwellings corresponding to about 160 millions of buildings, while new buildings represent yearly about 1.5% of the building stock [9].

According to data from the 2002 Census and, there are about 493,000,000 sqm. of building floor area in Romania, of which 86% are accounted for by residential buildings [3]. Moreover, over 71% of existing building stock in urban areas is composed of collective large prefabricated concrete panel houses. These large prefabricated blocks were built in three main periods, i.e.: (a) 1962-1975; (b) 1975-1982; (c) 1982-1989 [2].

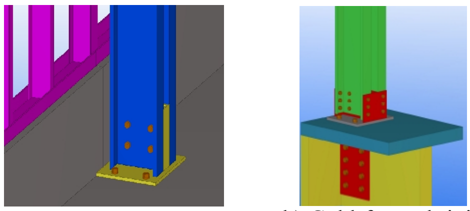
The current major problems of these buildings can be categorized to: problems regarding thermal comfort, ventilation, energy consumption; problems caused by the small inhabitable area and interiors rigid partitions; problems concerning the public space, accessibility, lack of parking areas, elevators, space for social interaction etc.

Therefore, in present context, there is a large variety of geometries, volumes and aesthetics for the over-roofing and over-claddings even in the boundaries of the same neighbourhood, degrading the overall visual image. Due to this lack of ratified methodology, errors in the execution of these over-roofings are unavoidable.

## 2. Over-Roofing Solutions Based on Intensive Use of Steel

The proposed solution for over-roofing, presented in Figure 3, was considered for the main block typology built in Timisoara, Romania in the period 1962-1989, i.e.: IPCT 744 specific to the period 1962-1975 – cold-formed steel sections (CF);

In pinned solution (see Figure 3a), the columns (two back-to-back lipped channel sections) are connected to the base concrete slab through a rectangular steel plate, but in this case a steel box was needed to connect webs of the profiles too. The second case scenario, i.e. the one using rectangular hollow section, was not presented here because is similar to the first case scenario. The connection is done by bolts, both for webs as well as at the base. In both 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 3).The rigid column-based connections are presented in Figure 3b.



a) Cold-formed semi-rigid section (CF)    b) Cold-formed rigid section (CF)

Fig. 3. *Proposed connections for steel over-roofing: (a) semi-rigid; (b) rigid*

The design of the structures subjected to over-roofing is composed of two stages: (i) verification of the current state of the original structure and (ii) verification of the structure after intervention (including the new-added steel structure).

The initial concrete structural system was made of precast panels assembled on site. The project has 3 longitudinal and 8 transversal, concrete diaphragms composed of standardized precast concrete panels. The interior panels are single layered elements of 14 cm thickness, in B250 (equivalent to C16/20) concrete class. The exterior panels are composed of three specific layers, with different functions: the load bearing layer (12 cm), the thermal insulating layer (6 cm) and protection layer (6 cm), respectively.

The state of the original structure and the refurbished one was done through 3D analyses using ETABS computer code, by using shell finite elements (see Figure 4).

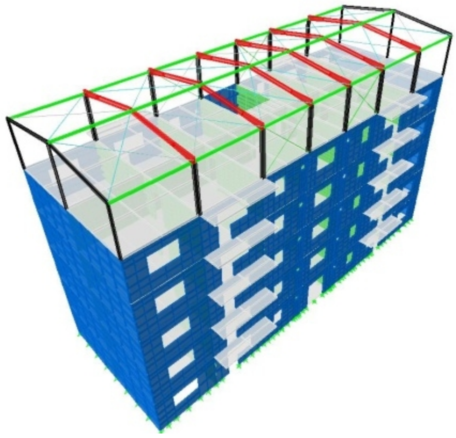


Fig. 4. *3D view of representative common block typology T744R*

The building was checked according to the actual standards, proving to be resistant to all loading combinations. These verifications also showed that, by removing the top thermal insulation layers from the terrace, the adding of a supplementary over-roofing floor would not affect the capacity of existing concrete structure.

The buildings were verified according to the actual standards, proving to be resistant to all loading combinations. The seismic load was accounted through a spectral analysis using the ground acceleration value $a_g$=0.20g and a control period Tc=0.7s.

These verifications also 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 systems were analysed by including in the already built models in ETABS, the new steel structures as shown in Figure 4. 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, a behaviour factor $q$ = 1, specific to non-dissipative structures was chosen.

One type of solution for over-roofing based on intensive use of steel was chosen for numerical studies: cold-formed steel profiles. Therefore, pinned/semi-rigid and rigid column-based connections were further investigated, by using numerical analyses, in order to optimise this solution (see Figure 5).

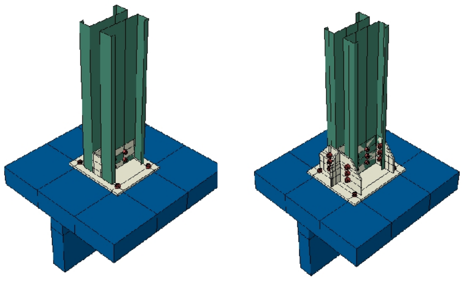
Cold-formed steel members are efficient in terms of both their stiffness and strength. Additionally, because the base steel is thin, even less than 1 mm thick when high strength steel is used, the members are lightweight. The use of thinner sections and high strength steel leads to design problems for structural engineers which may not normally be encountered in routine structural steel design.

Cold-formed steel design is dominated by two main problems, i.e. (1) stability behaviour, which is dominant for design criteria of thin sections, and (2) connecting technology, which is specific and strongly influences the structural behaviour and design detailing. Last years, the seismic performance (3) of cold-formed steel structures started to be examined [5].

## 4. Numerical Analysis

In order to prove the efficiency of the column-base connections chosen for this study, numerical simulations were performed using FE software ABAQUS

6.7 [1]. Semi-rigid and rigid base connections using cold-formed steel profiles (CF) will be detailed in the following, as presented in Figure 5.



(a) Semi-rigid CF (b) Rigid CF

Fig. 5. *Cold-formed steel column-base connections*

The numerical models (see Figure 5a and 5b) are also analysed in both cases, as semi-rigid and rigid connection. The column is composed of two back-to-back cold-formed lipped channels of 300 mm height and a 2 mm thickness. Two rows of M16 bolts were used to connect the webs of the profiles via a steel box section of 6 mm thickness of the walls and 150 mm height, placed between the two sections of the column, composing the semi-rigid solution. The geometrical characteristics of the base plate and the angle profiles are the same as for the previous case. For the rigid connection, supplementary stiffeners and bolts were added to connect the flanges of the profiles.

Elasto-plastic material models were used for all elements. SHELL elements of S4R type, with 4 nodes, reduced integration, 6 DOF per node were used for modelling the cold-formed steel sections, while for all other sections, BRICK elements of C3D8R type, with 8 nodes, reduced integration, 6 DOF per node, were used. In order to obtain accurate results the dynamic explicit solver was used. Initial convergence problems were issued by a preliminary step of preload and the actual load of 200 mm displacement was imposed.

Also, the concrete was considered in the analysis as elastic-perfect plastic, as shown in Figure 6 [10]. The option used to simulate concrete plasticity was Concrete Damaged Plasticity.

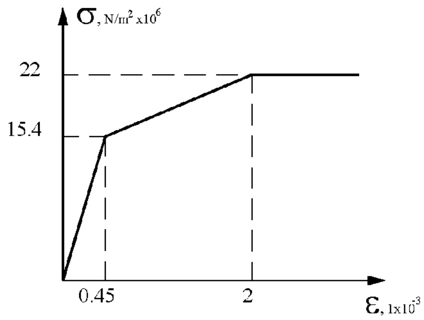
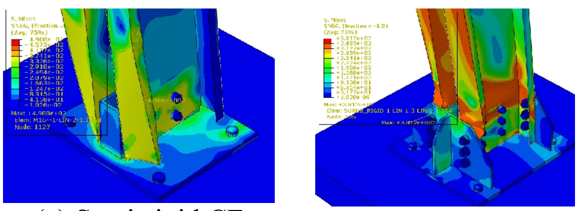



Fig. 6. *Elastic-plastic model of concrete*

The following results were obtained: (1) the first connection was modelled as semi-rigid but presents mostly a pinned behaviour; the maximum stresses were reported in the upper flanges of the section on the direction of loading, but also around the holes of the webs (see Figure 7a); (2) for the rigid solution, the plastic stresses achieve maximum values both in the flanges and the webs; a web crippling phenomenon occurs (see Figure 7b); (3) 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, as shown in Figure 8.



(a) Semi-rigid CF (b) Rigid CF

Fig. 7. *Distribution of stresses in the based connections*

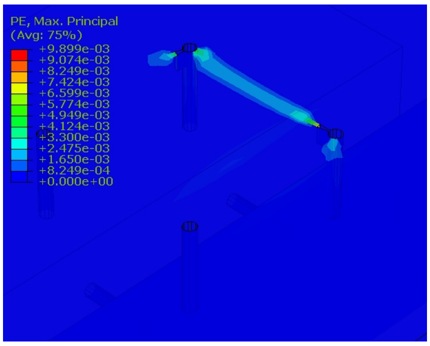



Fig. 8. *Strains in concrete on bolt hole*

The force-displacement curves obtained for each case scenario are presented in Figure 9.

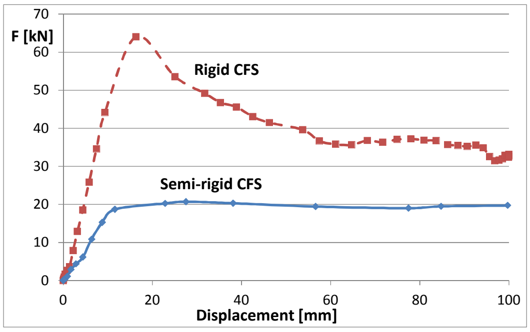


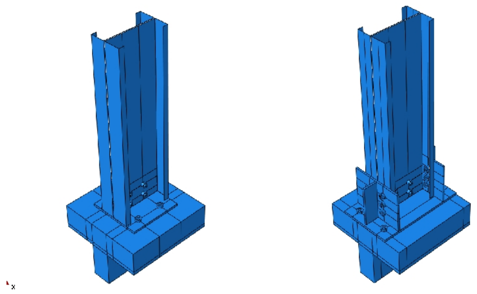

Fig. 9. *Force-displacement curves for the semi-rigid and rigid cold-formed steel base connections*

The above force-displacement curves show, as expected, the semi-rigid solution has smaller loading capacities than the rigid one, but the intention was to reduce the local effects on the existing structure. The stress level on the existing concrete floors and the additional walls are small and do not introduce supplementary stresses on the existing structure.

Taking the study further, a chemical anchorage system based on epoxy resin was numerically investigated for these two cold-formed solutions in order to offer a better solution for the comfort of the occupants. This solution was chosen for testing as it gives minimum disruption to the last level apartment owners and also for minimum concrete damage.

Generally speaking, there is a large variety of chemical anchor types available on the market. As a basic description they are basically a steel screw positioned in a predrilled hole in the concrete material. The connection between the steel screw and concrete is assured by pouring glue-based material (resin) (see Figure 12). According to most used design methods, at ultimate tensile force of these chemical anchors the concrete or bond should fail separately. This assumption is also used in certification regulations for steel post-installed anchors for concrete e.g. ETAG [6].

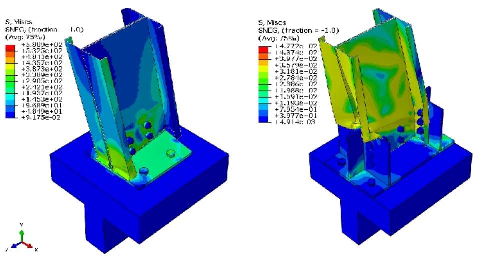
The two cold-formed models were re-designed in the same finite element software, with chemical anchorage. Most geometrical characteristics were preserved, with a few minor adjustments of bolt distance. However, the steel box section connecting the two back to back cold-formed sections were replaced by a more adequate and practical steel sheet of 2 mm thickness. Nevertheless, the angle profiles were not necessary in this case as the anchors were embedded in concrete (see Figure 13). Also, the same elasto-plastic materials were used in designing the assembly. A particular attention was given to the resin material in which the anchors are embedded, as documented elastic properties were given to the material in order to simulate the exact effect. Dynamic explicit analysis was used with a lateral displacement force of 200 mm. The initial axial preloading was still necessary for the semi-rigid section in order to simulate real-life loading conditions.



(a) Semi-rigid CF (b) Rigid CF

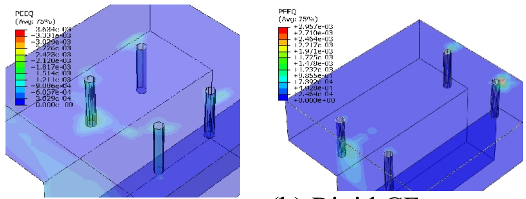
Fig. 10. *Cold-formed steel column-base connections with chemical anchors*

The following results were obtained: (1) the semi-rigid model presents maximum stresses around web bolt-holes and in upper section flanges; moreover at maximum loading conditions the flanges on the direction of loading are crushed; (2) the base-plate of the column presents large deformations in the tension zone (see Figure 11a); (3) for the rigid model, the stiffeners do not allow section rotation, so the flanges are crushed and a phenomenon similar to web crippling appears (see Figure 11b); (4) the concrete floor in both models shows some amount of plasticity at full loading conditions and mostly locally on bolt holes; however, the column sections failed much earlier before concrete reaches its plastic limit (see Figure 12).



(a) Semi-rigid CF (b) Rigid CF

Fig. 11. *Distribution of stresses in the column-based connections with chemical anchors*



(a) Semi-rigid CF (b) Rigid CF

Fig. 12. *Distribution of stresses in concrete with chemical anchors*

The force-displacement curves obtained for each case scenario are presented in Figure 13.

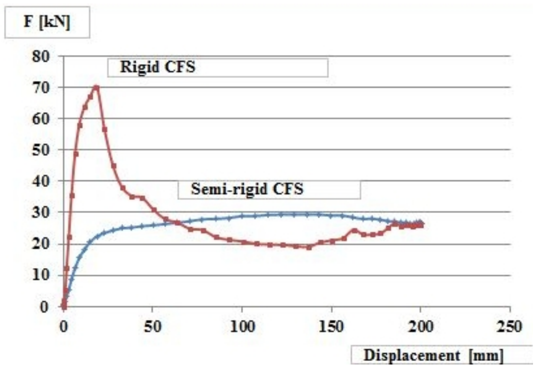



Fig. 13. *Force-displacement curves for the semi-rigid and rigid cold-formed steel base connections with chemical anchors*

The above force-displacement curves show, as expected, the semi-rigid solution has smaller loading capacities than the rigid one, but the intention was to reduce the local effects on the existing structure. The stress level on the existing concrete floors and the additional walls are small and do not introduce supplementary stresses on the existing structure.

The scope of the numerical program is a better understanding of the behaviour of these types of column-base connections and to find the optimum solution for this kind of over-roofing systems, by limiting the stresses to the existing concrete panel building. Based on this numerical study several types of these connections will be experimentally investigated.

## 5. Conclusions

The study presented in this paper shows two possibilities of connecting steel-intensive over-roofing to the existing concrete buildings. The numerical analyses show the light 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. The numerical analyses show that the cold-formed connections can withstand a considerable amount of loading, for this kind of four class section. However, the main concern remains to minimize the structural intervention on the existing concrete structure. A better solution for the comfort of the occupants seems to be the connection by chemical anchors. The results show that the column behaviour is mostly the same as for the first solution. The concrete displays some plasticity at full loading conditions, mostly on bolt holes; however, cracking does not occur as the column section has already failed before concrete reaches its ultimate limit state. Experimental tests will be done in order to validate the proposed solutions.

## Acknowledgements

This work was partially supported by the strategic grant POSDRU/159/1.5/S/134378 (2014) of the Ministry of National Education, Romania, co-financed by the European Social Fund – Investing in People, within the Sectoral Operational Programme Human Resources Development 2007-2013.

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