IOSUD - Universitatea Politehnica Timişoara
Şcoala Doctorală de Studii Inginereşti

STEEL OVER-ROOFING SOLUTIONS FOR EXISTING PREFABRICATED CONCRETE BUILDINGS

PhD Thesis – Summary

To obtain the scientific title of PhD at
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ABSTRACT

An important segment of the Romanian urban population, similar to other Eastern European countries, lives in collective residential apartment buildings made using large prefabricated concrete panels. Most of these structures were built between 1960 and 1989 and present major issues concerning aesthetical aspects, lack of internal space, problems related to thermal comfort and, last but not least, weak energy efficiency. The presence of flat roofing system, generally with hydrothermal faults, represents an additional problem which leads to deficient living conditions of the last-storey inhabitants in many cases. Besides other ways of improving the conditions of such apartment houses, the over-roofing is welcome to both increase the habitable area and provide adequate roofing for the building. This thesis presents the problematics of these kind of buildings, together with the various solutions that can be employed to refurbish and optimize these kinds of blocks. A state of the art of various hot-rolled and cold-formed solutions existent in literature are further detailed.

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 to which the construction sector turns to more and more every day, instead of the counterpart solution of demolishing the old structures and building new ones. Henceforth, the lightness, reversibility and time-consuming features of the materials used in building these over-roofing systems have made engineers turn to steel-based solutions. In this context, the paper summarizes three types of steel-based over-roofing structural solutions, i.e. a) hot-rolled profiles; b) rectangular hollow sections and; c) cold-formed profiles. The investigation is mainly focused on details concerning the connection of over-roofing to the existing structure. Further on, details of a numerical study on these column-base connections are presented, in both semi-rigid and rigid solutions. Finally, for the comfort of building owners, but also for ease in erecting of the over-roofing, an option of column-base connection based on chemical anchorage was analysed numerically.

Moreover, the use of the lightweight steel construction with cold-formed steel structural components leads to important advantages such as strength, lightness, durability, easy adaptation, recyclability, and prefabrication. However, the connection between the old and the over-cladded structure often represents a problem, which can limit, in many cases the application of desired over-roofing systems. Present study is focused on the analysis and behaviour of column-base connections in case of over-roofing solutions with cold-formed thin-walled structural elements.

The thesis presents, an integrated refurbishing solution, which combines the idea of implementing highly industrialized over-roofing systems using cold-formed steel profiles with an integrated over-cladding system for complete renovation.

CHAPTER 1

This chapter gives a general background for the motivation and necessity of building and improving these over-roofing structural solutions.

The necessity of an integrated renovation solution is further detailed at a local scale in the following subchapters, following a case study for the area of Timisoara city, Romania (see fig. 1).

Fig. 1. Problems regarding the building blocks in Timisoara.

As stated in the general background of the issue, building adaptation is a well-known established solution for rejuvenating an existent structure and offering new living spaces to the population. On this note, over-roofing is one of the practices most popular among structural engineers, as together with over-cladding it offers an integral solution for rejuvenating an existing building.

Moreover, using steel as base material for these over-roofings is common, due to its flexibility, long spans and easy erections. Also, steel is known for its recyclability quality, making the structure more sustainable, impact on the environment being a raising concern nowadays. On this idea, cold-formed steel has become preferable to hot-rolled one. There are several benefits to using cold-formed steel sections, including reduced material, fabrication, transport, and construction costs, and a higher strength to weight ratio compared to hot-rolled steel. Not to mention, a better behaviour in seismic areas.

However, cold-formed connections are less easy to design, detailing becoming a challenge for most engineers.

Thus, the aim of this thesis is analysing the behaviour of these cold-formed connections and to offer the best solutions for these over-roofing structures. Moreover, the scope of this study is not only to improve the structural techniques used for building these building adaptions but to improve the quality of life of the occupants by offering them a holistic solution from structural and energetic point of view.

CHAPTER 2

This chapter contains an even more detailed overview of the general issues concerning existing large precast panel blocks of flats, starting from world-wide and general context and focusing on a particular and local case study of Timisoara city in Romania. Additionally, it gives a general outline of the proposed solutions applied in Europe at present for over-roofing and building adaptation, existing codes for design and material options.

The chapter presents the legislative framework created by the EU: the Energy Performance of Buildings Directive 2010/31/EU (EPBD) [1] and the Energy Efficiency Directive 2012 /27/EU [2]. These directives are considered essential in achieving the energy performance enhancement of buildings. The main goal promoted by these policies is to achieve by 2050 a highly energy efficient and decarbonised building stock, creating a stable environment for investment decisions and enabling consumers and businesses to make more informed choices to save energy and money.

In this scope, Landolfo R. within the European Project TABULA [3] tries to summarise and analyse the quantitative and qualitative data gathered from different European countries, in the effort of portraying an accurate building technologies map showing both the seismic-resistant structures and the envelope systems, the latter to also evaluate the environmental behaviour of buildings in terms of Life Cycle Assessment (LCA) and energetic consumptions. This map is, afterwards, compared to the seismic and climatic characteristics of all the European countries considered, thus giving a complete view of zones with different combined hazard conditions. The combinations of these seismic and climatic zones were comprised in a hazard matrix, where particular case studies can be placed, allowing them to offer scenarios of different retrofit solutions from seismic and energy viewpoints (see fig. 2). The rate of success of applying these different solutions at a particular level, can afterwards allow the constructing of a general analysis method which can be used in every country in Europe.

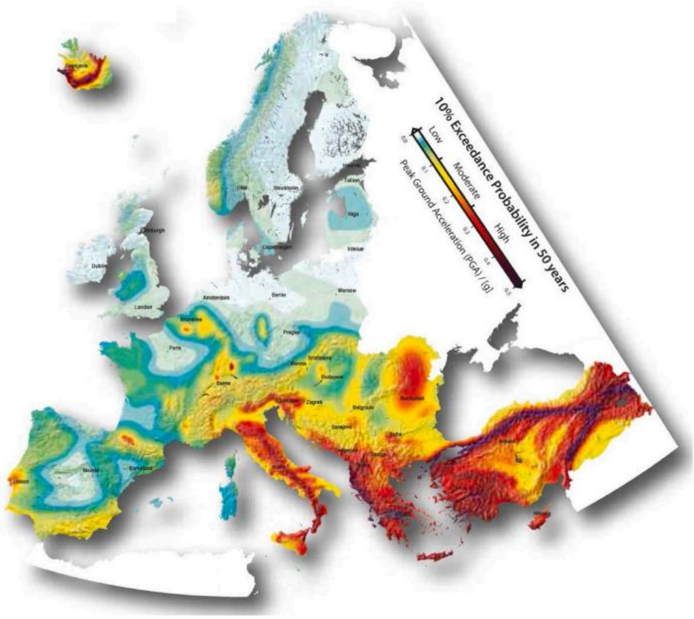

Fig. 2. European Seismic Hazard Map (Source: European project SHARE, 2013) [3].

The problem is then analysed locally to Romania, presenting the building stock and periods of erection, their numerous structural, energetical and aesthetical deficiencies and the non-compliant legislative framework existent, but also the new programs that enforce the energy enhancement of new and existing buildings like the program „ Casa Verde" [4].

CHAPTER 3

This chapter details the existing legal framework, norms and technical guidelines concerning over-roofing structures, from global setting to local one.

On a European scale, the main financing model proposed in Horizon2020 [5] project relies on the valuation of the unused buildability factor in benefit of increased energy efficiency of the whole building. This approach requires to take into account one legal variable making such financing model possible. Thus, the "Air right" legal variable was introduced.

Air rights are a type of development right in real estate, referring to the empty space above a property. Generally speaking, owning or renting land or a building gives one the right to use and develop the air rights (see fig. 3).

This legal concept gives the idea to build market opportunities, specifically in the retrofitting of existing buildings. In the urban plans of all cities exist one volumetric parameter linked to each building or property that is not being fully utilized in many cases. This availability of urban "land" not used, usually in the downtown areas of major cities, is going to be used as a catalyst for the retrofitting of these buildings with large energy gaps at the same time as rundown neighbourhoods (typically peripheral areas and old town) are revitalized. This improves the accessibility of these buildings, helps maintain historic buildings, eliminates uneven skylines and dividing walls in the city and increases urban density without the construction of new infrastructure.

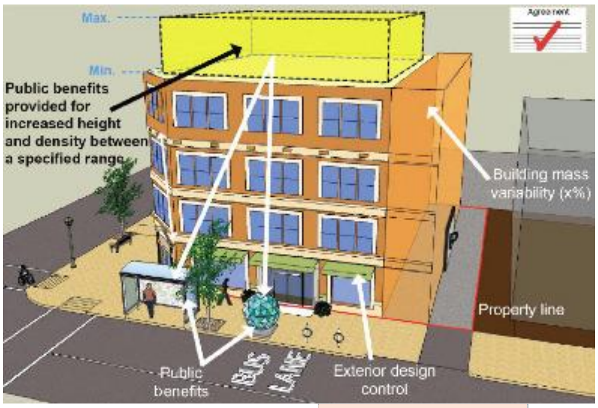

Fig. 3. Development Permit System (s.70.2 and O. Reg. 608/06) [5-7].

As housing typology, Timisoara has its urban space divided into ten inhabitable neighbourhoods (areas), with a total of 21.837 housings units of various kinds. The total number of individual buildings is of 15.039 units of single residence buildings and 3.159 units of two or more residences, with variable height (from 1 to 3 storeys). The collective dwellings (with 5 to 11 storeys) counted for a total of 3.639 units. From these 88% were made of precast panels. The inhabitable fund includes 122.195 apartments, with a total area of 4.372.696 square meters and 277.944 inhabitable rooms.

The period between 1960's and 1990's generated a major densification of the housing areas through collective dwellings constructions. This evolution and densification of the city was accomplished through the construction of neighbourhoods both in undeveloped areas at that time, as well as through insertions in the built fund.

The prefabricated concrete residential buildings, ('the concrete blocks') present major difference between typologies according to few basic criteria:

- urban criteria: density (number of units/ha); related exterior facilities (schools, shopping centres, green spaces); number of storeys; accessibility (type of roads, parking areas, distance from these to the housing).
- architectural criteria: surface (built square metres, inhabitable area, and usable area); built space, the facades, space configuration, access definition;
- energy consumption and CO_2 emission: the differences between the finishing of prefabricated panels and the difference in envelope stratification (panels) and the slab;
- finishing criteria: (thermal, waterproof and noise insulation);
- engineering: (differences in seismic conformation according to the time when the housing units were executed) [8].

These densifications appeared in these neighbourhoods without being based on concise urban planning documentation and based on L114/96; of HCL 141/2007, attics of residential buildings were made, adopting different geometries even within the same neighbourhood/section (fig. 4)

Referring to the current status of residential areas on a local scale, some major urban dysfunctions can be pointed out [8]: (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. Some examples of these dysfunctions are presented in fig. 4.

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Fig. 4. Major urban dysfunctions in the neighbourhoods of Timisoara city.

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-roofing 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-roofing can be divided in two categories: (1) the first regards the bearings and anchorage of the over-roofing system to the existing structure, while (2) the second one is related to the actual and common structural errors.

In Romania, the main laws for over-roofing and over-cladding construction are: NP-064-02, Law 50/1991 (re-issued in 2018), and Law 193/ 2019 [9]. However, these laws are not enough to resolve to the vast situations present in the Romanian building sector. The necessity of detailed laws and guidelines for these kinds of constructions is furthermore emphasized.

Secondly, best structural practices for over-roofing are presented and structural solutions based on steel material are becoming more and more used by engineers due to their numerous advantages: lightness, modularity, fast erection and recyclability. The pros and cons of different construction material like wood and concrete are also presented in order to paint a wholesome picture with the information needed to make an educated decision when choosing the material for future over-roofings and over-claddings.

Last but not least, current practice has proven that cold-formed steel structures are the optimal solution for these kinds of modular extensions which will be furthermore studied in Chapter 4.

CHAPTER 4

This chapter focuses on presenting the advantages of using steel material for over-roofing structures.

In this effort, a study has been conducted by the author on 3 different steel solutions: hot-rolled profiles, rectangular hollow section and cold formed (CF) sections. The comparison was done by implementing 3 the steel over-roofing options on a representative case study, namely block E744.R, built between 1962-1975.

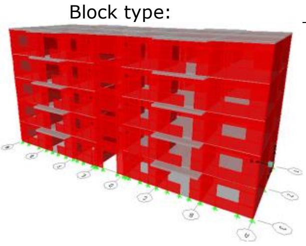
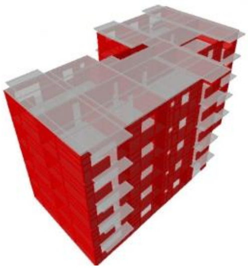
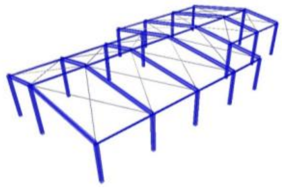
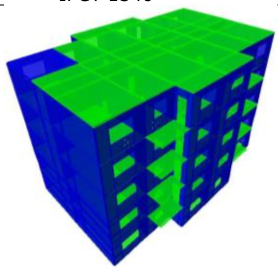
Block type:	Over-roofing system	Main frame cross-sections
 IPCT T744R	 (a) Hot-rolled profiles (HR)	Columns: HEB180 Beams: IPE220
 IPCT 1340	 (b) Rectangular hollow section (RHS)	Columns: RHS220×220×8 Beams: IPE220
 IPCT 770	 (c) Cold-formed section (CF)	Columns: 2C300×2 (built-up members) Beams: 2C300×2

Fig. 5. Proposed steel over-roofing solutions for the main block types: a) T744R, b) 1340, c) 770 [10-13].

In order to apply these over-roofing solutions safely on this existing block of flats, the structural resistance of this block typology needed to be assessed. In a second step, the entire assembly, with the added over-roofing was analysed.

Moreover, a FEM analysis was necessary on the column-base connections of the 3 different steel solutions in order to assess the viability of this solution chosen (see fig. 6). The FE study was done by using ABAQUS 6.7 software and consisted in 6 column assemblies (pinned and splayed). All connection systems proved a semi-rigid behaviour and failure was noted at the level of the steel material and not the concrete slab (see fig. 7).

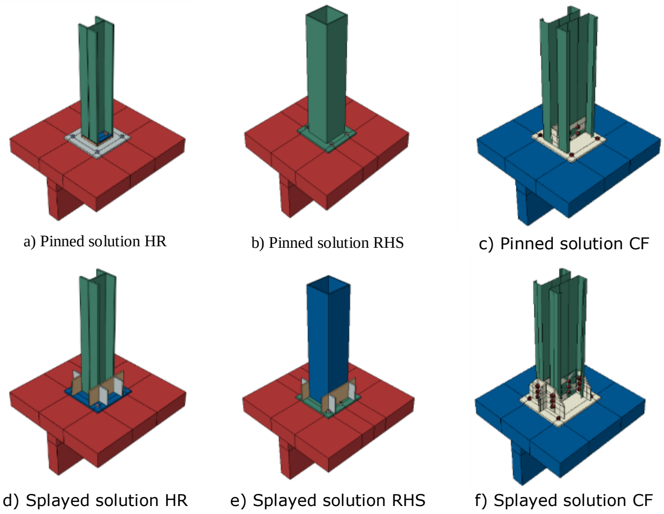


Fig. 6. Solutions for column-base connections.

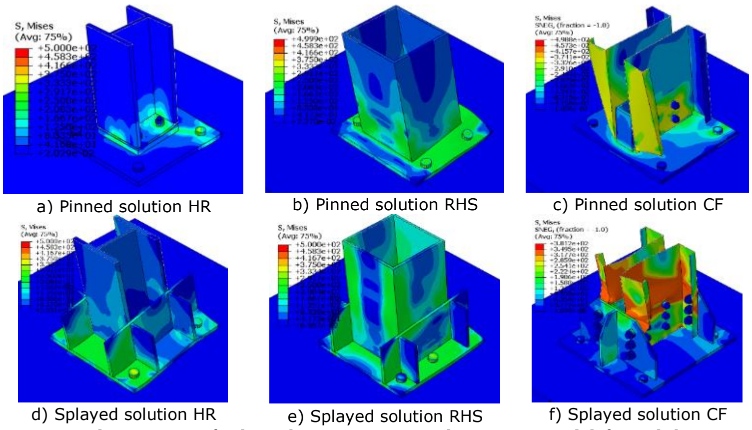
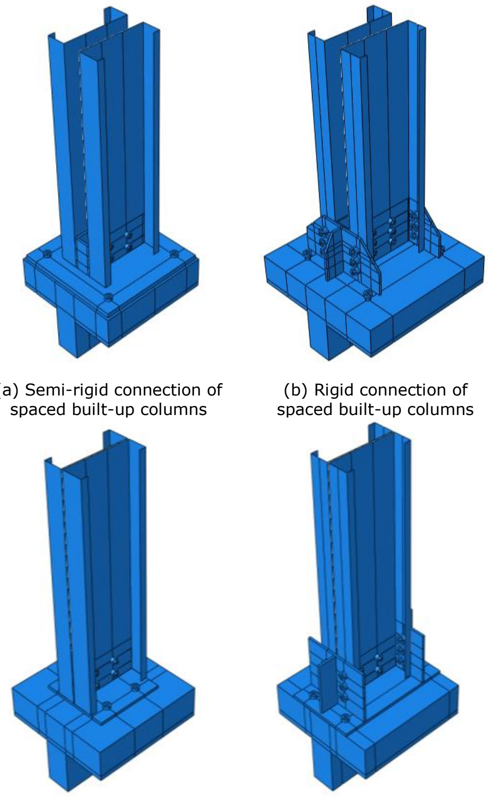


Fig. 7. Ultimate state of column-base connections: plastic strains and deformed shapes.

The study led to the following main conclusions:

- the over-roofing of precast concrete apartment buildings is possible by unloading the existing structure of the original slope layers. In this way, similar loading conditions are attained for the existing structure;
- the original structures possess important reserves of resistance in order to respond to actual norm conditions for loading and checking;
- steel intensive-solutions represent suitable options for over-roofing, with additional care for column-base connection detailing. All the solutions analysed - HR, RHS and CF - are feasible and lead to different advantages;
- simple or splayed column-base connections can be applied. Both systems develop stiffness values that do not affect the distribution of forces within the structure.

The FE study was then developed on the CF section solutions, trying to optimize the anchoring system to the concrete block and also a spaced and non-spaced column solution.



(a) Semi-rigid connection of spaced built-up columns

(b) Rigid connection of spaced built-up columns

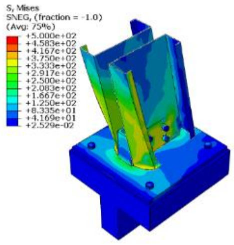
(c) Semi-rigid connection of closely spaced built-up columns

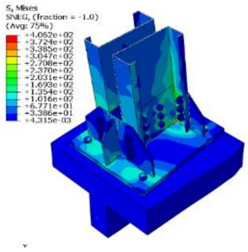
(d) Rigid connection of closely spaced built-up columns

Fig. 8. Cold-formed steel column-base connections.

The following results were obtained:

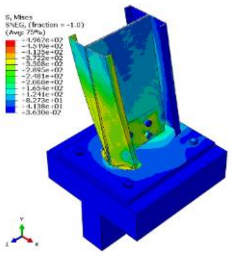
(1) the semi-rigid models present maximum stresses around web bolt holes and in flanges in compression; at maximum loading conditions the flanges in compression developed a phenomenon similar with web crippling;
(2) the base plate of the column presents deformations in the tension zone (see fig. 9a, c);
(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 fig. 9 b, d);
(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 fig. 9).

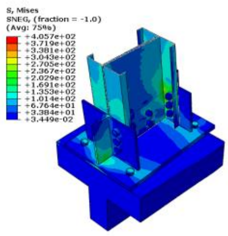




(a) Semi-rigid connection of spaced built-up columns

(b) Rigid connection of spaced built-up columns





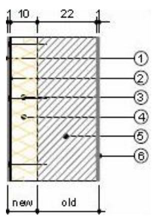
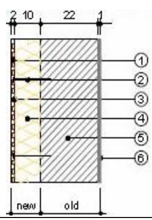
(c) Semi-rigid connection of closely spaced built-up columns

(d) Rigid connection of closely spaced built-up columns

Fig. 9. Stress distribution of cold-formed steel column-base connections using chemical anchors.

Finally, the chapter tries the analyse 3 over-cladding options for this over-roofing systems in order to improve the thermal and energetic balance of the entire module and to give a holistic solution on the whole. A LCA is therefore performed on these 3 thermal insulation panels.

Table 1. Wall over-cladding solutions.

	Element type	t [mm]	Weight/area [kg/m^2]
Solution 1 	1-Adhesive mortar	10	7.5
	2-Glass fiber mesh		0.16
	3-Connecting elements (steel rivets/ plastic discs)	-	0.35 0.056
	4-Extruded polystyrene (14 cm)	140	4.5
	5-Reinforced concrete	220	-
	6-Interior plastering	10	-
Solution 2 	1-Decorative brick	15	33
	2-Connecting elements (steel bolts/ plastic discs)	-	1 0.168
	3-Ventilation space	0	-
	4-Rockwool (16 cm)	160	11.5
	5-Reinforced concrete	220	-
	6-Interior plastering	10	-
Solution 3	1-Sandwich panel	0.5	0.135

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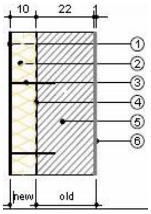


steel case sheet (exterior)		
2-Mineral wool	160	1
3-Sandwich panel steel case sheet (interior)	0.5	0.135
4-Connecting elements	-	1.4
5-Reinforced concrete	220	-
6-Interior plastering	10	-

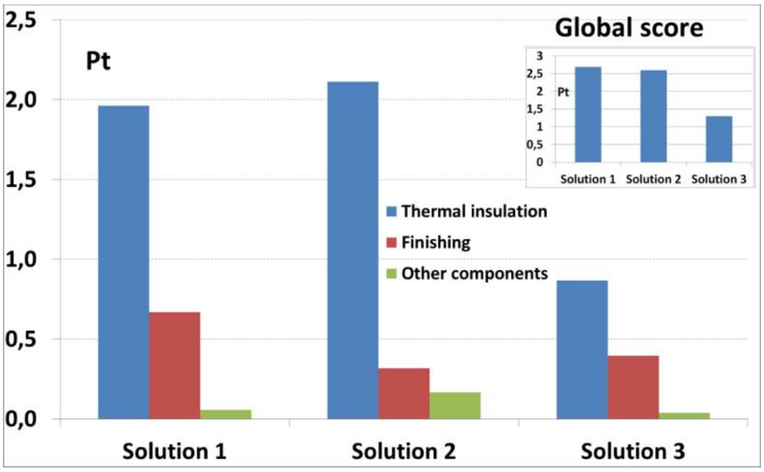


Fig. 10. Environmental impact analyses for wall over-cladding systems.

CHAPTER 5

This chapter contains the details on the full-scale column-base experiments (6 trials). This includes the layout, load application, instrumentation. Also contains details on the component tests, including coupon tests to determine material properties.

The 6 experimental tests are as follows:

1) Semi-rigid column concrete base in monotonic loading (2C300x1) (SRCB-M-1);
2) Semi-rigid column concrete base in monotonic loading (2C300x2) (SRCB-M-2);
3) Semi-rigid concrete column base in cyclic loading (2C300x1) (SRCB-C-1);
4) Semi-rigid concrete column base in cyclic loading (2C300x1) (SRCB-C-1-2);
5) Rigid column steel base in monotonic loading (2C300x2) (RCB-M-2);
6) Rigid column steel base in cyclic loading (2C300x2) (RCB-C-2).

a) b)

Fig. 11. Column-base assembleys: a) with concrete base, b) with steel base.

Initial tests on 2C300x2 columns were performed in order to assess the concrete base capacity. Previous FEM modelling of this assembly has proven that the concrete base height (14 cm), found on these types of residential blocks is less than the depth required for chemical anchoring embedment. Thus, a 6 cm concrete layer was poured on the existent concrete plate specimens with HILTI connectors. This also ensures a delay in the cracking of concrete base and increases the capacity of the concrete material. The new concrete plates were initially tested in experimental trials in CEMSIG laboratory by Prof. Florea Dinu as commissioned by ASTERA CONSULTING S.R.L.

Fig. 12. Concrete-base assembleys used in experimental tests.

Coupon tests were performed to determine the material properties of the column members. Coupons were cut from the web and flange of the columns, both sections used (C300x1 and C300x2) and from the web-flange radius (see Fig. 13). The coupons were cut and tested (tensile and compression tests) according to standards (ISO 6892-1, 2019), with a displacement loading rate of 1.5 mm per minute (see fig. 14).

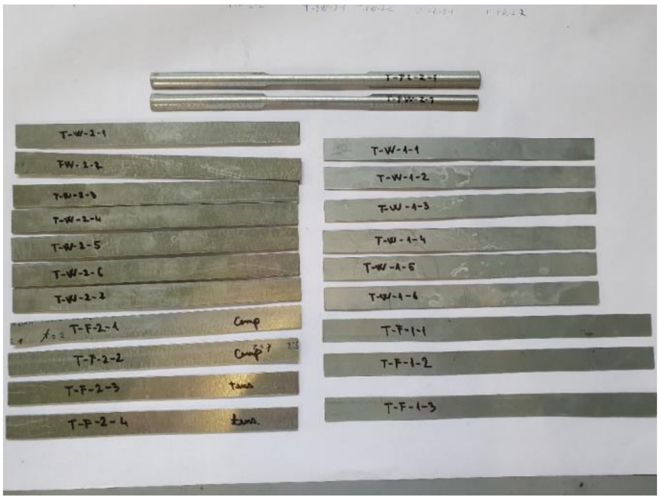


Fig. 13 Coupon test samples for material tests.

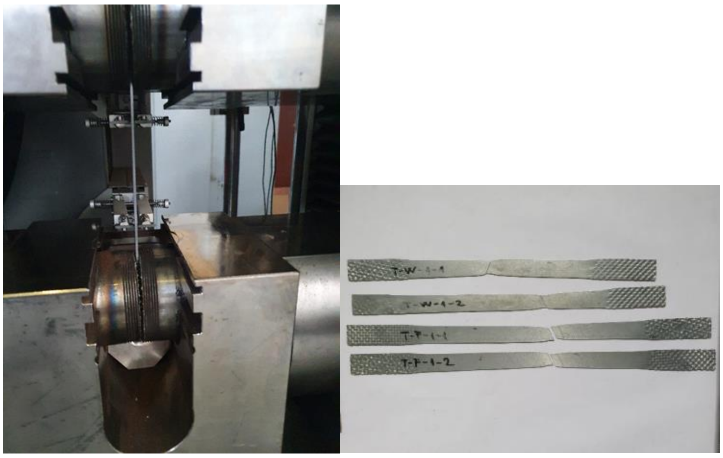


Fig. 14. Material testing of coupon samples.

For all the tensile tested specimens, the characteristic curve of the material is similar as can be seen in fig. 15 -16.

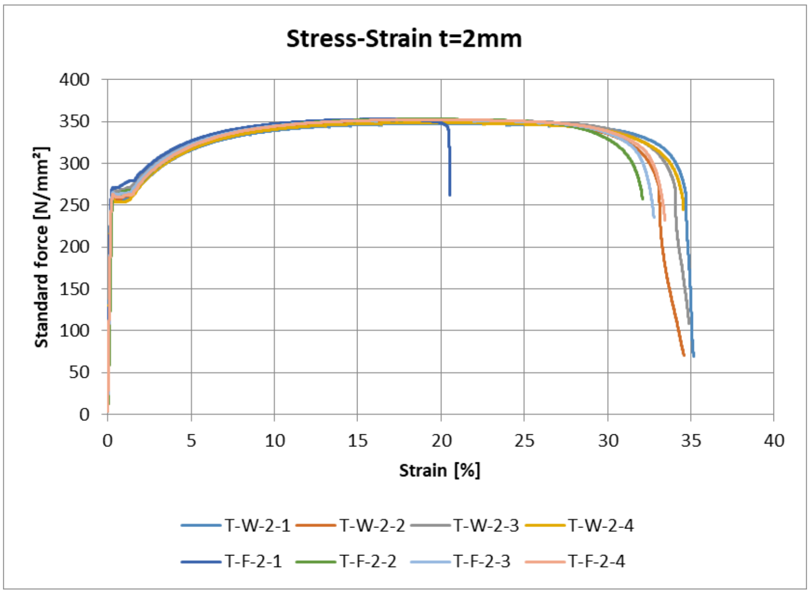


Fig. 15 Characteristic curves σ-ε of the base material for C300x2 sections.

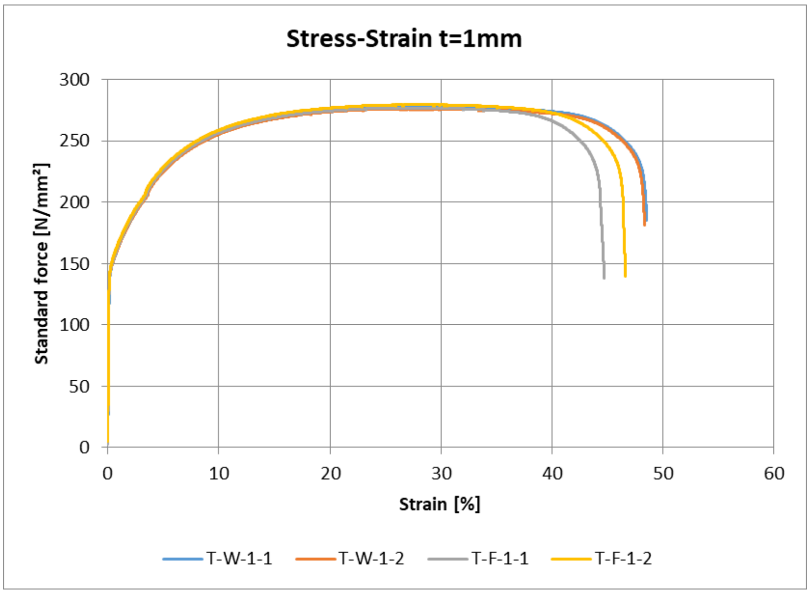


Fig. 16. Characteristic curves σ-ε of the base material for C300x1 sections.

Furthermore, the material layers thickness of the C-sections was analyzed, a radiographic procedure being performed.

The radiographic procedure on the flange and webs of the SRCB and RCB thin-walled specimens emphasized the following conclusions:

1) The actual thickness of the steel sheets in web and flanges of the SRCB specimens is less than 1 mm varying from 0.9831 to 0.9993 mm;
2) The coating layers for the SRCB specimens in number of three prove to be consistent of Zinc element mainly;
3) The actual thickness of the steel sheets in web and flanges of the RCB specimens is more than 2 mm varying from 2.021 to 2.023 mm;
4) The coating layers for the RCB specimens in number of four prove to be consistent of Zinc element mainly, but also of a variety of alloys for e.g. Ti, C, Mg, Si etc.;
5) The layers of coating are very discontinuous in thickness and homogeneity, which can be attributed to poor manufacturing and errors in procedure of fabrication.
6) The above imperfections need to be accounted for in the calibration of the experimental procedure.

After concrete base-steel base assembleys were ready, the experimental test layout was set. The experimental trials were conducted at CEMSIG laboratory in Timisoara. The base assembleys were positioned on the steel beam presented below and lateral Z welded plates were used to prevent the lateral displacement of the concrete block (see fig. 17). Moreover, plastic beams were used to prevent lateral out of plane displacement at the top of the column. The loading jack was positioned at the top of the column and the column was pulled monotonically and cyclically.

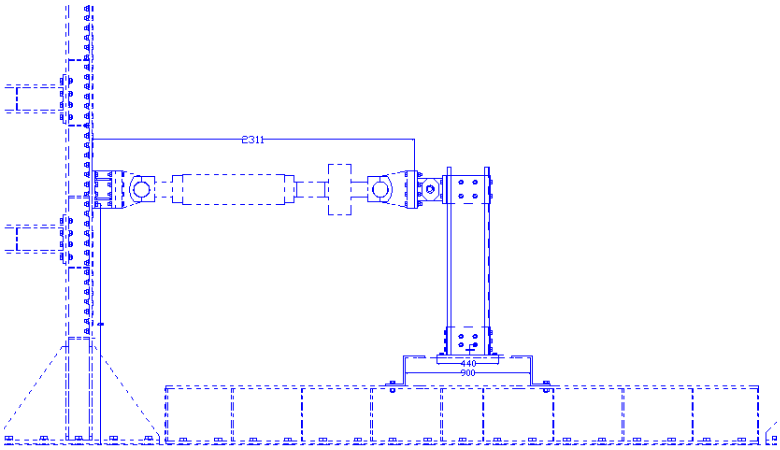


Fig. 17. Experimental column-base setup.

Before applying load, the initial imperfections of the C profiles were registered by using VIC 3D, for the frontal web and for the compression flanges.

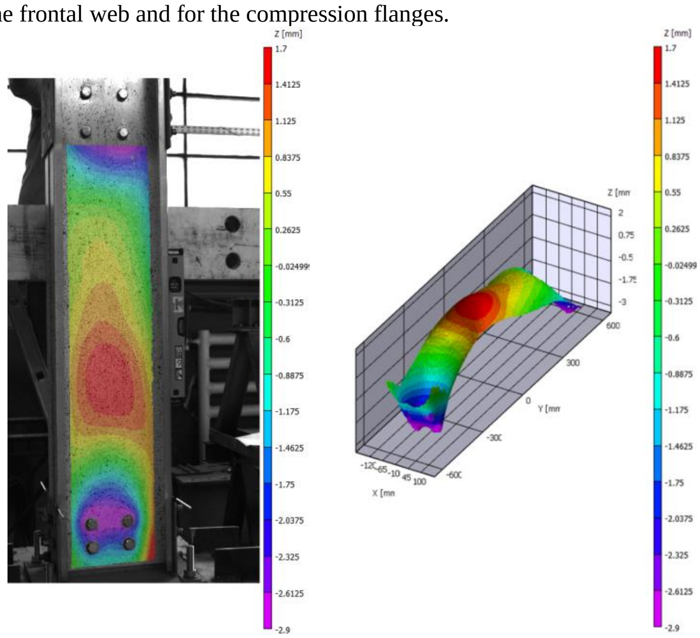


Fig. 18. Monitored area for web imperfection and 3D interpretation.

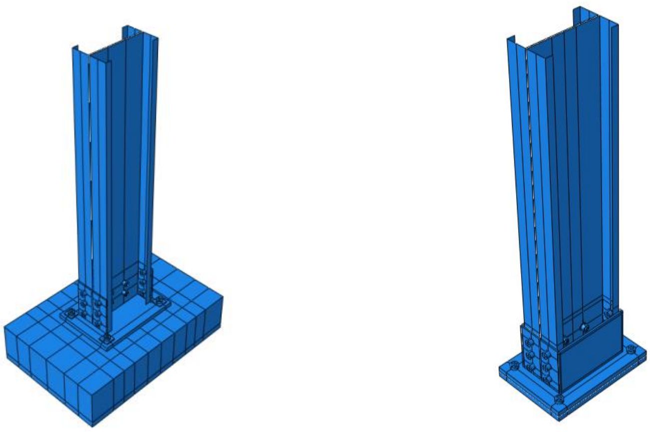
Experimental results are presented including column strengths and failure modes for the various column configurations and loading conditions. Contributions and effects of the various base connections and stiffeners are discussed.

The experimental program consisting of a series of 6 column-base systems conducted for determining the behaviour of column-base connections for over-roofing solutions gave the following conclusions:

1) All the column specimens presented imperfections due to fabrication process. These imperfections affect the behaviour of the system.
2) The micrographic procedure showed uneven and thin layers of coating which affect the properties of the material.
3) The 2 mm thickness C profiles present sufficient rigidity to external forces, their deformation being minimum; however, the stress is thus higher in the concrete block, which causes failure by cracking in cones in the anchor area.
4) All specimens presented buckling of web and flanges on the direction of loading, a behaviour specific to these thin-walled profiles.

CHAPTER 6

It discusses the finite element model, including calibration and validation of the model to experimental results and also to existing works. Last but not least, it includes parametric studies for the column-base connections using parameters such as plate thickness, stiffener thickness, bolt grade etc.



a) b)

Fig. 19. FEM 3D view of main calibration models.

SHELL elements of S4R type, with 4 nodes, reduced integration, 6 DOF per node were used for modelling the cold-formed steel sections. This four-node shell element has six degrees of freedom per node. It can account for nonlinear material properties and finite membrane strains and features hourglass control and reduced integration. Following a mesh sensitivity analysis, a mesh size of 10 × 10 mm was selected to guarantee adequate numerical accuracy while keeping the computational time within acceptable limits.

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.

Bilinear and multilinear material models were used for all elements, besides the analysed cold formed sections. The steel material behaviour used in the experimental column profiles was input in the FEM analysis, after transforming it to true stress and true strain. A particular attention was given to the resin in which the anchors are embedded, as documented elastic properties were given to the material to simulate the exact effect with an elastic modulus of 3240 N/mm2 (Mapefix EP 385).

The concrete was considered in the analysis with elastic-plastic behaviour [14]. The option used to simulate concrete plasticity was Concrete Damaged Plasticity. This is the same concrete class as the one used in experimental program (C20/25), which was proven to have same or stronger characteristics as design ones by sample material tests performed. Concrete reinforcement is also simulated in FE analysis as wire type and as embedded interaction property.

In a next step the engineering strains and stresses were converted to logarithmic plastic strains and stresses (true stress-strain), while the linear kinematic hardening rule available in ABAQUS was adopted to simulate the hardening behaviour of the material.

Dynamic explicit analysis (displacement control) was used with a maximum lateral displacement which represented more than 0.025h (h – level height) lateral drift at the ultimate limit state.

Initial imperfections were introduced in the FE analysis by perturbations in the geometry.

In the experiments, as well as in the FE model, the global buckling mode of the CFS beam was prevented due to the presence of the lateral bracing system. Hence, only a local or a distortional imperfection (whichever mode had the lower critical buckling load) was included in the FE models. Imperfection amplitudes of $0.94t$ and $0.34t$ (where t is the plate thickness) were used for the distortional and local imperfections, respectively [15].

Boundary conditions were imposed onto the FE model of the. All three translational degrees of freedom of the nodes at the bottom of the column concrete base were restrained (UX = UY = UZ = 0), while the horizontal displacements of the top nodes were also restrained (UX = UY = 0). The ‘‘Hard’’ contact feature was employed between the connecting faces of the column webs and the I profile plates to avoid penetration of the surfaces into each other. All degrees of freedom of the column end section, where the external load was applied, were coupled to a Reference Point (RP) located at mid-height of the webs.

For the cyclic loading cases, the loading protocol applied was computed using FEMA-461.

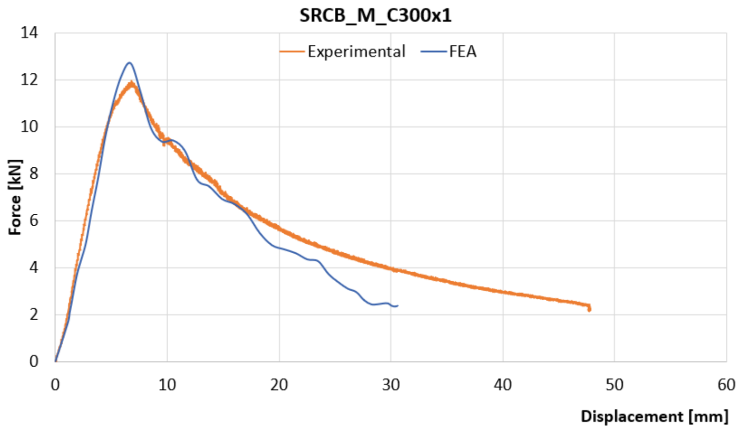


Fig. 20. Force-displacement curve in FE analysis for SRCB_M.

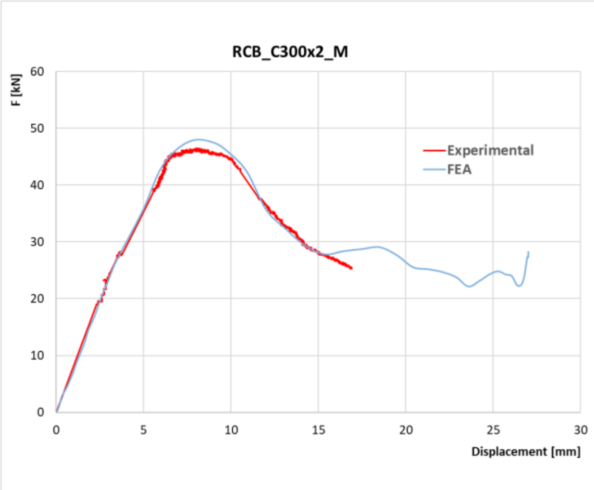


Fig. 21. Force-displacement curve comparison (experimental vs. FE model).

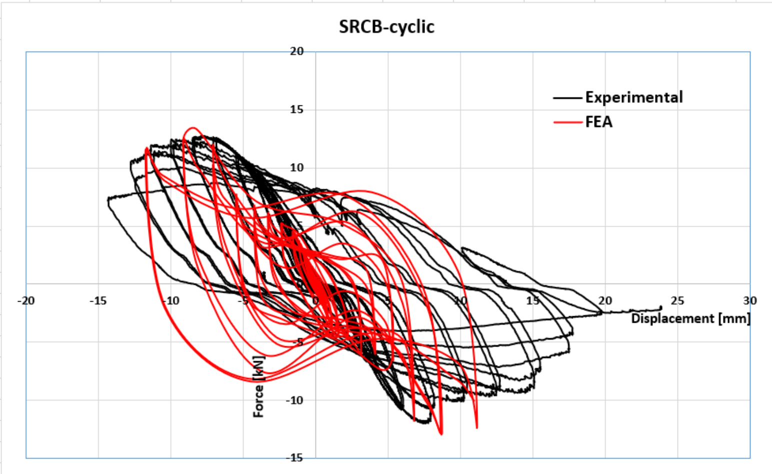


Fig. 22. Force-displacement curve comparison (experimental vs. FEM model).

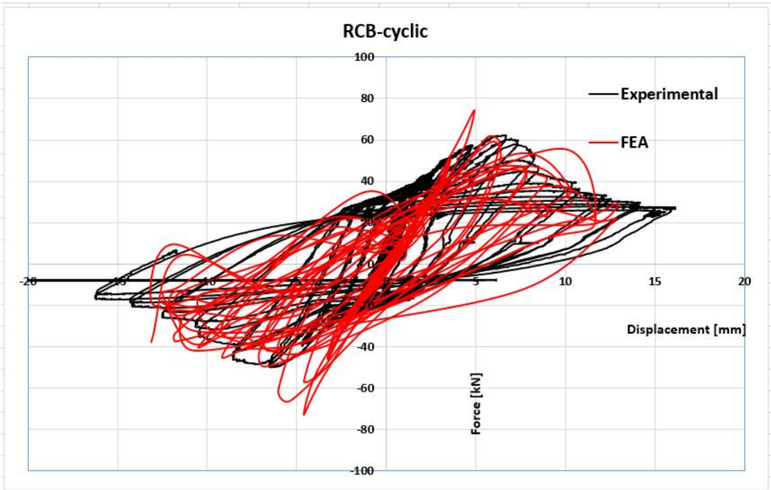


Fig. 22. Force-displacement curve comparison (experimental vs. FEA).

The force-displacement diagrams show a good correlation between the experimental program and FE model, the displacement discrepancy at yield can be blamed on human error and imperfections of materials that could not be totally accounted for in FE models. However, the numerical FE model shows a greater ductility and a lower strength capacity at yield, which can be correlated to differences between material curves.

Further parametric numerical analysis was performed in order to prove the efficiency of the cold-formed column-base connections chosen. Numerical simulations were performed using FE software ABAQUS 6.7. for the monotonic case scenarios.

Table 1. Parameters analysed for each case study.

Case study	Parameters				
	Thickness of steel base plate [mm]	Thickness of C profile [mm]	Diameter of mechanical anchors [mm]	Thickness of I profile [mm]	Bolt grade
SRCB	10	2	16	8	8.8
	15	3	24	10	10.9
	25	-	-	-	-
RCB	10	2.5	-	-	-
	15	3	-	-	-
	25	-	-	-	-

The study showed that for both SRCB and RCB scenarios, the base plate thickness and C-section profile thickness prove to be very important in the variation of the strength and ductility of the entire assembly.

Thus, a decrease of the base plate thickness lower than 15 mm is not recommended. Also, a decrease in C-profile thickness is only recommended if higher ductility and earlier collapse of the column is the objective. The latter is desired in order to avoid concrete base cracking and anchor pull-out, outcome which is important for these kind of over-roofing solutions.

CHAPTER 7

This part details the analytical procedure, which entrails the design of cold-formed structures with semi-rigid connections.

The column-base assembly was numerically and experimentally detailed in chapter 5 and 6. However, for a global view of the entire frame system, the knee and ridge connections of the over-roofing portal frame system needed to be investigated further.

Thus, the analytical software IDEA STATICA [16] was used to determine the capacity and behaviour of the ridge and knee joints of the over-roofing frame system. The analytical models were compared to the experimental results from the reference models presented and detailed by Zsolt Nagy in his Ph.D. Thesis [17].

In order to prove the efficiency of the software, in a first step, the apex and corner connections used in the experiment (2C350x3 sections for beams and columns) were modelled and their capacity and behaviour were compared. After confirmation, the knee and ridge joint connections used in present thesis (2C300x1 and 2C300x2) were modelled and analysed.

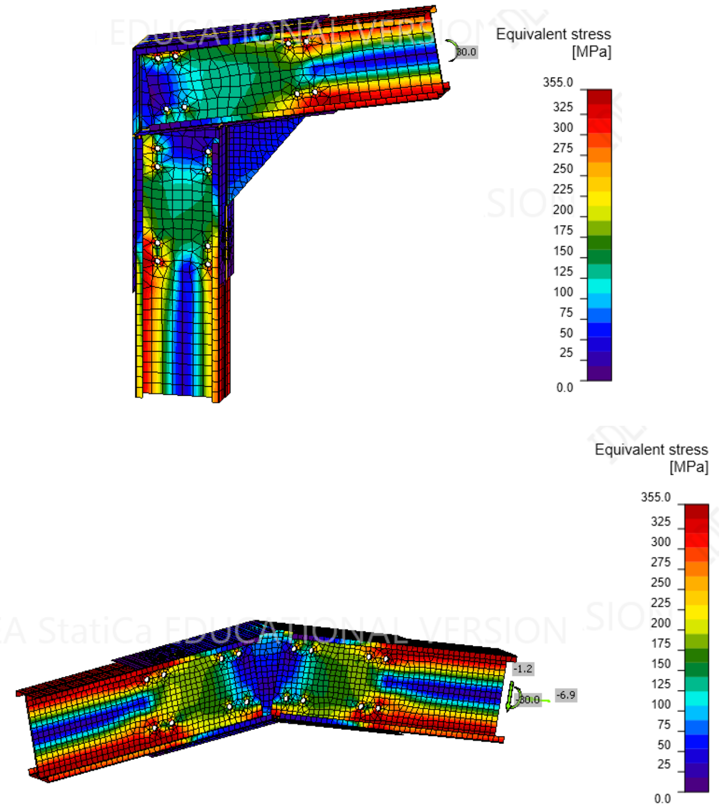


Fig. 23. Knee and ridge joint stresses in IDEA StatiCa at design moment.

A parametric program was then proposed for this type of ridge and knee connections. The proposed design situations can be seen in Table below. The modified parameter of each case scenario is highlighted.

The analysed parameters were:

1. Thickness of C profile (3, 2.5, 2 for C350; 2;1 for C300);
2. Steel grade of C profile (S355, S450);
3. Bolt grade of connection (M16, M20, M24);
4. Bolt class of connection (6.8, 8.8);
5. Bolt distribution (1-2 bolts on flanges; 2-3 bolts on flanges; 3-no bolts on lower flanges);
6. Steel grade of connecting plates (S235, S355).

In a second step, different column-base design proposals are analysed in IDEA STATICA to give a better outline of the possibilities of connecting this type of over-roofing solution to the existing concrete block. Moreover, the column-base connections used in the experimental program and described in chapter 5 (SRCB-2C300x1 and SRCB 2C300x2) are modelled in order to determine their rigidity and resistance and correlate them to the experiment.

The proposed design situations are:

1) **Pinned/semi-rigid solution** made up 2 back-to-back C-profiles (2C300x2) with an intermediary connecting plate of 6 mm thickness; bolts M16 8.8. only on the web of C-profiles and 3 rows of anchoring bolts M24 4.6.
2) **Semi-rigid solution** (similar to the one used in the experimental program) made up 2 back-to-back C-profiles (2C300x2) with an intermediary connecting I welded section of 6 mm thickness; bolts M16 8.8. on the web and flanges of C-profiles and 3 rows of anchoring bolts M24 4.6.
3) **Semi-rigid/Rigid solution** made up 2 back-to-back C-profiles (2C300x2) with an intermediary connecting I welded section of 6 mm thickness; bolts M16 8.8. only on the web of C-profiles and 2 rows of anchoring bolts M30 8.8; stiffeners are welded perpendicular to the I section in the direction of the applied load.
4) **Semi-rigid/Rigid solution** made up of a built-up section of 4 C-profiles (2 C400 + 2 C250) with an intermediary connecting I welded section of 6 mm thickness; bolts M16 8.8. only on the web of C-profiles and 4 rows of anchoring bolts M24 6.8; stiffeners are welded perpendicular to the I section in the direction of the applied load.
5) **Semi-rigid solution (the one used in the experimental program-SRCB model)** made up 2 back-to-back C-profiles (2C300x1) with an intermediary connecting I welded section of 6 mm thickness; bolts M16 8.8. on the web and flanges of C-profiles and 2 rows of anchoring bolts M20 6.6. Concrete block base.
6) **Semi-rigid solution (the one used in the experimental program - SRCB model)** made up 2 back-to-back C-profiles (2C300x2) with an intermediary connecting I welded section of 6 mm thickness; bolts M16 8.8. on the web and flanges of C-profiles and 2 rows of anchoring bolts M20 6.6. Concrete block base.
7) **Semi-rigid/rigid solution (the one used in the experimental program - RCB model)** made up 2 back-to-back C-profiles (2C300x2) with an intermediary connecting I welded section of 6 mm thickness and lateral stiffeners of 8mm thickness; bolts M16 8.8. on the web and flanges of C-profiles and 2 rows of anchoring bolts M20 6.6. Steel beam base.

Last but not least, the rigidities computed for the ridge, knee and base joints are implemented on the 3D portal frame system presented in Chapter 4 for block type E744.R. The semi-rigid base frame system is compared to a rigid and pinned base similar solution.

An analytical calculus was performed in ETABS software on 3D over-roofing system to determine the forces/displacements after applying rigidities computed in IDEA StatiCa software. Rigidities were applied at column base, apex and knee joints. The two types of frame sections used in the experimental study were used: SRCB(2C300x1) and SRCB (2C300x2).

In a second step, the results are compared to the rigid and pinned base systems case scenarios.

The setup used was detailed in previous chapters (see Chapter 4) for block type E744.R. The portal frame span is 9.6m and column height is 3m. Same geometry was used and similar loads specific for Timisoara region were applied. The moment, shear and axial loads were taken from permanent load combination (ULS) (1.35P+1.5S), which has proved to be the critical load scenario.

The displacement verifications for beams were taken from SLS combination (P+S).

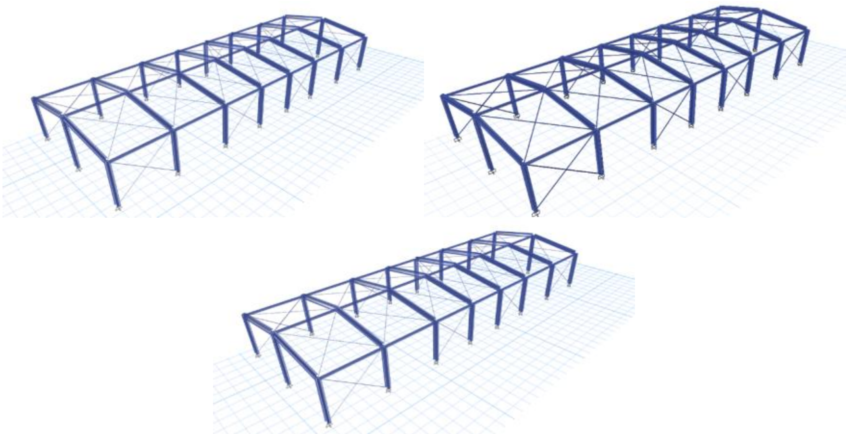


Fig. 24. 3D models of 3D of semi-rigid, rigid and pinned over-roofing case scenarios.

CHAPTER 8

The final part summarizes the conclusions of the study, personal contributions and further directions.

According to a European Commission's study, buildings are responsible for approximately 40% of energy consumption and 36% of CO2 emissions, classifying them as the largest energy consumers in Europe. The building sector is thus an important problem that needs a solution in order of achieving the European Union's energy and environmental goals. In this effort, The EU created a legislative framework, that includes the Energy Performance of Buildings Directive 2010/31/EU (EPBD) and the Energy Efficiency Directive 2012 /27/EU, which are essential to the energy performance enhancement of buildings. These directives promoted policies, that are aimed to achieve in 2050 a highly energy efficient and decarbonised building stock, creating a stable environment for investment decisions and enabling consumers and businesses to make more informed choices to save energy and money.

Regulations and legal framework in Europe and worldwide for over-roofing and modular extensions has already been established and is very common. However, in Romania, the legal policies and guidelines are still lacking and an improvement in this direction is still necessary.

Restructuring by vertical addition consists of adding one or more stories above the existing structure, resulting in an increase of overall volume of the building. Depending on the size and height of the new addition masses, it is necessary to recheck the load-bearing capacity of the original structure in order to decide whether or not to take consolidation measures. In seismic zones this problem can be very serious, and traditional building techniques cannot be always used for this type of restructuring. The necessity to minimise the weight of the new storey structure added above, makes cold-formed steel sections the most suitable solution.

The studies conducted on the existing building stock for Timisoara city, confirmed the fact that in the period 1962-1990, three different types of projects were mainly used. In the first period of this urban development, between 1962 and 1975, the most used standard project was T744R-IPCT. In the second period, 1975-1982, frequently used was the project type 770-IPCT, while in the period 1982-1989 the project type 1340-IPCT had the largest application [9-12].

The proposed solutions for over-roofing considered for the three main block typologies built in Timisoara in the period 1962-1989 were: (a) T744R – hot-rolled profiles (HR); (b) 1340 – rectangular hollow sections for columns and hot-rolled profiles for beams (RHS); (c) 770 – cold-formed steel sections (CF).

The Project 744 R (Db1) was chosen for further analysis (see Stage 1962-1975 – project type I.P.C.T.: „RESIDENTIAL P+4 BUILDINGS MADE OF PREFABRICATED PRECAST REINFORCED CONCRETE PROJECT T 744.R).

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The design conditions included the checking of both old concrete structure and steel over-roofing structure. For concrete structure the same verifications were made as in case of original structure. Considering the loading conditions expressed above, it resulted that the concrete part can largely overtake the loads induced by steel additional structure.

The most important conclusion retrieved from the structural design of old concrete and new steel structural components revealed that the over-roofing system proved to be light enough for not overloading the existing structure. In this process, the removal of old hydro and thermal insulation layers lightens the concrete structure by important dead-loads. Also, the analyses made for the steel structures lead to the conclusion that there is no significant difference in designing the steel structure on the top of concrete structure or on the ground, due to the very stiff behaviour of concrete structure. In consequence a ground design of steel structure is safe and can be used by designers.

Regarding the base connection, a FEA analysis was performed in ABAQUS 6.7 a pinned and splayed solution was chosen for all three cases as considered in structural design.

The study led to the following main conclusions:

- the over-roofing of precast concrete apartment buildings is possible by unloading the existing structure of the original slope layers. In this way, similar loading conditions are attained for the existing structure;
- the original structures possess important reserves of resistance in order to respond to actual norm conditions for loading and checking;
- steel intensive-solutions represent suitable options for over-roofing, with additional care for column-base connection detailing. All the solutions analysed - HR, RHS and CF - are feasible and lead to different advantages;
- simple or splayed column-base connections can be applied. Both systems develop stiffness values that do not affect the distribution of forces within the structure.

In order to further prove the efficiency of the column-base connections chosen as most suitable for this study, numerical simulations were performed using FE software ABAQUS 6.7. Semi-rigid and rigid base connections using cold-formed steel profiles were analysed using chemical and mechanical anchors.

The following results were obtained:

(5) the semi-rigid models present maximum stresses around web bolt holes and in flanges in compression; at maximum loading conditions the flanges in compression developed a phenomenon similar with web crippling;

(6) the base plate of the column presents deformations in the tension zone (see fig. 28 a, c);

(7) 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 fig. 28 b, d);
(8) 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 fig. 29-32).

The last checked cold-formed solutions were then implemented on a large scale in the over-roofing frame system to observe the global behaviour.

The structural analysis led to the following main conclusions:

- the over-roofing of precast concrete apartment buildings is possible by unloading the existing structure of the original slope layers. In this way, similar loading conditions are attained for the existing structure;
- the original structure possesses important reserves of resistance in order to respond to actual code conditions for loading and checking;
- lightweight structural systems are suitable options for over-roofing, with additional care for column-base connection detailing.

Moreover, the next step was to offer a holistic solution for the entire assembly by checking an over-cladding solution system from sustainability, cost, safety point of view.

In order to make a choice of the thermal-insulation system, three solutions were chosen for over-cladding the existing building and the added over-roofing: polystyrene, rockwool and mineral wool.

The over-cladding options presented above comply with the following criteria, which classify them as integrated refurbishing solutions:

- structural aspects: capacity to attain the structural performance objective set in design, structural compatibility with the initial structural system; adaptability to actual design loads, including seismic loads;
- technical and comfort aspects such as reversibility, durability, thermal, acoustic and spatial comfort, aesthetical and functional solutions, technical support provided, available materials, quality control;
- economical aspects such as integrated costs;
- environmental aspects (minimizing the operational energy, use of low-impact or recyclable materials.

All the three solutions were aimed to provide an internal thermal comfort translated in a thermal resistance of 2.5 m2K/W (U-value = 0.4W/m2K).

From environmental point of view, it clearly results that the third solution (sandwich panel on mineral wool) leads to the smallest impact, about half of the other two solutions.

From cost point of view, it results that the most economical solutions are usually the one with lower thermal insulation material cost (mineral wool – solution 3) and having a highly-industrialized prefabricated process.

The experimental program consisting of a series of 6 column-base systems conducted for determining the behaviour of column-base connections for over-roofing solutions gave the following conclusions:

5) All the column specimens presented imperfections due to fabrication process. These imperfections affect the behaviour of the system.
6) The micrographic procedure showed uneven and thin layers of coating which affect the properties of the material.
7) The 2 mm thickness C profiles present sufficient rigidity to external forces, their deformation being minimum; however, the stress is thus higher in the concrete block, which causes failure by cracking in cones in the anchor area.
8) All specimens presented buckling of web and flanges on the direction of loading, a behaviour specific to these thin-walled profiles.

The numerical calibration program showed similar behaviour in stress and strain capacity to the experimental program. The small discrepancies noticed can be blamed on human error and imperfections of materials that could not be totally accounted for in FE models.

A parametric study was afterwards conducted on the SRCB and RCB specimens in order to evaluate the elements for which the case studies are most vulnerable.

The parametric study was conducted only for the monotonic loading conditions in the FEA software ABAQUS 6.7.

The study showed that for both SRCB and RCB scenarios, the base plate thickness and C-section profile thickness prove to be very important in the variation of the strength and ductility of the entire assembly.

Thus, a decrease of the base plate thickness lower than 15 mm is not recommended. Also, a decrease in C-profile thickness is only recommended if higher ductility and earlier collapse of the column is desired. The latter is desired in order to avoid concrete base cracking and anchor pull-out, outcome which is important for these kind of over-roofing solutions.

The rigidity and capacity of the knee and ridge joint were further analysed by using the FEM software IDEA StatiCa. The software was first validated by comparing its results to the ones in an experimental program, which showed great accuracy. In a second step, the connections used in our present frame system were modelled. The following conclusions could be made:

1) At the design moment from calculus, both and ridge and knee connections are close to maximum capacity.
2) Both connections are semi-rigid.
3) The two joint sections used in design present similar behaviour to the ones of the reference model and are semi-rigid. However, as expected their capacity is lower than the one with 2C350x3 sections, decreasing gradually with 30% for each connection (2C300x2 and 2C300x1).
4) Profile thickness, bolt number on flanges/ web, steel grade are all parameters which affect the rigidity and capacity of the analysed joint connections, as given by the parametric program.

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