HABILITATION THESIS

Sustainable Design of Seismic Resistant Steel and Composite Building Structures

Associate Professor Liviu Adrian CIUTINA, Ph.D.

Politehnica University of Timişoara

TABLE OF CONTENTS

A. ABSTRACT

The research activity of the candidate started in November 1997, when he was recruited as Ph.D. student at the Politehnica University of Timisoara under the coordination of Professor Dan Dubina. In 2001 the candidate was also enrolled at the Institut National des Sciences Appliquées de Rennes, France in a co-tutoring Ph.D. thesis. The candidate passed the Ph.D. defence at the Institut National des Sciences Appliquées de Rennes, France in November 2003.

The present thesis summarizes the most important part of the research activity of the candidate after defending the PhD Thesis. The selected activity was considered to be relevant in terms of originality and importance, in order to anticipate an independent development of the further research and teaching career.

The post-doctoral activity is addressed in two main thematic directions developed by the candidate: (i) ***Seismic Behaviour of Steel and Concrete Composite Structures*** presented in Chapter 2 below and respectively (ii) ***Sustainable Development of Buildings***, presented in Chapter 3.

Continuing the main theme of the Ph.D. thesis – ***seismic behaviour of composite structures***, the candidate obtained by competition a research grant (name of the grant: *"Numeric and Experimental Study on the Connecting Devices between Steel and Concrete for Composite Buildings" Located in Seismic Zones*) soon after his Ph.D. defence, offered by the Romanian Ministry of Education and covering the seismic ***behaviour of connecting devices*** between steel and concrete in composite elements. The grant investigated experimentally and then numerically the cyclic behaviour of connecting devices, by variation of different parameters such as type of connectors, loading type, steel profile flange class and concrete class. Since 2010 the research subject was enriched by the study of the connection between steel Rectangular Tubes and concrete through the use of shot nails, by the integration of the candidate in the team of the European project type RFCS *"High Strength Steel in Seismic Resistant Building Frames - HSS-SERF"*. Although the main purpose of the research was focused on the global use of high strength steel in building frames subjected to seismic loads, a particular attention was devoted to the connection between steel column tubes and in-filled concrete. The research was based on an initial experimental program, aiming to evaluate the load introduction within composite columns realized as CFT of high strength steel

Following another subject touched in his Ph.D. thesis, the candidate explored the ***dissipative zones in steel and steel and concrete composite frames***. Based on the previous experimental and analytical work, not entirely developed in the Ph.D. thesis, the candidate published several papers on the *ductility of Column Web Panel Zone*, among which two on ISI indexed journals. Other dissipative zones of steel and composite frames included the *joint zones for Moment Resisting Frames (MRF) and link elements in case of Eccentrically Braced Frames (EBF)*. The research was conducted mainly by integration of the candidate into two Romanian research grants: grant PNCDI II "Partnerships", contract no. 31.042/2007 "*Structural systems and innovative technological solutions for the protection of buildings subjected to extreme actions in the context of sustainable development PROACTEX*" and respectively the grant type CEEX-ET, module II, contract 1434/27.04.2006: *"Dual steel structures with dissipative removable links for structures located in seismic zones"*. The study concerned experimental and numerical behaviour of dissipative zones in dual MRF+EBF structures. A special attention was paid to the composite aspect.

Following the investigation of the dissipative zones of frames, the ***implications on the structural response*** were investigated in a series of studies mainly within the same research grants as mentioned before. The structural response in the case of important seismic motions depends directly on the elasto-plastic behaviour of elements and hinges. The numerical investigation considered elasto-

plastic analyses of low and medium height steel frames, considering the interaction of the steel beam with the concrete slab.

Lately, the candidate was integrated in the team of the national grant 55/2012 (PN-II-PT-PCCA-2011-3.2-1303): „*Structural conception and design based on control of the failure mechanism of multi-storey frames subjected to accidental loads (CODEC)*” for the behaviour of composite elements. The project is devoted to the robustness behaviour of steel and composite frames in the case of a column loss. Progressive collapse resistance is a measure of the structural robustness and relies primarily on resistance of key elements, continuity between elements and ductility of elements and their connections. The different nature and intensity of extreme loading events make difficult the development of explicit design requirements for such design situations. A better strategy is to limit the extent of damage in case of such events, so that the progressive collapse is not initiated.

The second subject of research covered by the candidates is related to the ***Sustainable Development of Buildings***. The topic is detailed in Chapter 3 of the present thesis. The subject was developed by the candidate after his integration in the team of the international grant COST C25 (2006-2010) type TUD COST C25 "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*". There are two main themes addressed by the candidate in the topic of sustainable development of buildings: (i) ***approach for new steel-intensive structures*** and respectively (ii) ***sustainable retrofitting solutions for existing building stock***.

The topic of ***sustainable development of steel-intensive buildings*** was achieved through a series of research grants related to the topic: European grant type RFSR (CT-2010-00027) „*Sustainable Building Project in Steel – SB_STEEL*” , industry grant (founded by ARCELOR MITTAL) “*Affordable House*” and recently RFS2 (CT-2013-00016): "*Large Valorisation on Sustainability of Steel Structures – LVS3*". The main purpose of the research was the quantification of environmental impacts, economic and social aspects for steel based structures. The most important direction in the integrated design of new structures is to find a good relation between cost and comfort. A supplementary parameter could be in some cases the erection time. The environmental impact analyses were made with two main purposes: internal analysis for the identification of the main sources of environmental impact and respectively external analysis for comparison with other structural solutions and/or other locations in Europe (SB Steel Project). The main outcomes of the research lead to the publication of five papers in journals (one ISI indexed) and seventeen papers at conferences among which one ISI indexed.

The ***sustainable retrofitting solutions for existing building stock*** represents one of the issues of large interest in Romania: in this moment more than one third of the Romanian population lives in about 84000 block of flats (apartment house type) built between 1960 and 1990 with important issues to be reviewed. The interior repartitioning of concrete residential buildings can improve the comfort of inhabitants. The apartment coupling on horizontal or on vertical can conduct to new internal configurations and can offer new interior space perspectives with implications at a larger scale on the local community, such as the decrease of densification in the urban areas. Structurally, both types of interventions are possible but care should be given at local interventions: (i) when cuts in the vertical diaphragms are performed, these should be reinforced by additional steel frames or concrete jacketing; (ii) if cuts are made on horizontal diaphragms, additional reinforcement near the cut zone is needed.

Another method of improving the overall performance of large precast concrete panel buildings is by overcladding. This solution offers additional space to inhabitants and also an adequate roofing system. Several possibilities of overcladding were investigated by using steel-intensive solutions. The studies were performed within the ERA-NET research grant (3002/2011): "*INSPIRE - Integrated strategies and policy instruments for retrofitting buildings to reduce primary energy use and GHG emissions*” and led to the publication of four journal papers and eight papers presented in conferences.

Directly connected with the research activity, the candidate was taking part in the implementation of Eurocode system in Romania through translation of documents and realisation of National Annexes, process coordinated at the national level by ASRO (Romanian Association of Standardisation) such as Realisation of the national annexes for EN 1994 1-1 , EN 1993-1-4, EN 1993 1-6, EN 1999 Part 1-3, EN 1999-1- 4 and EN 1999-1-5 and translation into Romanian of EN 1993-1-4, EN 1993-1-5 and EN 1999-1-5.

In the same direction it could be noticed that the candidate was directly involved in the realisation of the design guide "*Design of steel structural connections according to SR EN 1993-1-8. Recommendations, comments and design examples*" in a contract with the Ministry of Regional Development (2010) for use of civil engineers and students as well.

Most of the research mentioned above was done in cooperation with the Ph.D. students within the Department of Steel Structures and Structural Mechanics from The Politehnica University of Timisoara.

The candidate is member of the European Convention of Structural Steelwork (ECCS), with activities in two of the Technical Committees (TC):

- ECCS TC11 – Composite Structures;
- ECCS TC14 - Sustainability & Eco-Efficiency of Steel Construction.

It could be noticed that they have identical directions to the main research topics addressed by candidate.

The research activity of the candidate was embodied in a series of books in the fields of composite steel and concrete structures and sustainability of buildings as author, author on chapters or editor. Also, the didactic activity of the candidate is also related to the main research themes described in chapters 2 (Seismic Behaviour of Steel and Concrete Composite Structures) and respectively 3 (Sustainable Development of Buildings): The candidate sustained several invited presentations in workshops for Ph.D. and master level students as well as summer-courses for bachelor students.

The involvement of the candidate in national and international grants as director or team member provided the relevant skills and competences on management of such projects. One important aspect in the further development of the career of the candidate is to build a research team focused in the directions of composite structures and sustainable development of buildings at home university. It is the intention of the candidate to recruit further potential PhD students among the students involved in Master Thesis on the topics similar to those mentioned above. It has to be mentioned that the candidate already guided three doctoral students for obtaining their Ph.D. degree and in this moment is guiding other four Ph.D. students at the Politehnica University of Timisoara.

A. REZUMAT

Activitatea de cercetare a candidatului a început în noiembrie 1997, odată cu înscrierea ca bursier pentru studiile doctorale sub coordonarea d-lui Profesor Dan Dubina. În 2001 candidatul a fost admis ca doctorand şi la Institutul Naţional de Ştiinţe Aplicate din Rennes, Franţa, pentru o teză în cotutelă. Candidatul a obţinut titlul de doctor în noiembrie 2003, la Institutul Naţional de Ştiinţe Aplicate din Rennes, Franţa.

Teza de abilitare prezintă cele mai importante aspecte privitoare la activitatea de cercetare a candidatului după susţinerea tezei de doctorat. Activitatea selectată a fost considerată relevantă în termeni de originalitate şi importanţă, pentru anticiparea dezvoltării independente a cercetărilor ulterioare şi a carierei didactice.

Activitatea post-doctorală este direcţionată pe două subiecte tematice dezvoltate de către candidat: (i) ***Comportarea Seismică a Structurilor Compuse Oţel-Beton*** prezentată în capitolul 2 al tezei, respectiv (ii) ***Dezvoltarea Durabilă a Construcţiilor***, prezentată în capitolul 3.

Continuând principala temă a tezei de doctorat - ***comportarea seismică a structurilor compuse oţel-beton***, candidatul a obţinut prin competiţie un grant de cercetare (nume grant: *Studiul numeric şi experimental al sistemelor de conexiune între oţel şi beton la construcţii cu alcătuire mixtă în zone seismice*) imediat după susţinerea tezei de doctorat, grant oferit de Ministerul Educaţiei, care s-a concentrat pe ***comportarea seismică a elementelor de conexiune*** dintre oţel şi beton în elemente compuse. Grantul a investigat experimental şi apoi numeric comportarea ciclică a conectorilor prin varierea diferiţilor parametri, inclusiv a tipului de conectori, modul de încărcare, clasa secţiunii metalice şi clasa betonului. Din 2010 subiectul de cercetare a fost dezvoltat cu studiul conexiunii dintre ţevile rectangulare din oţel şi beton, prin intermediul conectorilor tip bolţ împuşcat, prin integrarea candidatului în proiectul de cercetare de tip RFCS *"Utilizarea Oţelurilor de Înaltă Rezistenţă în Cadrele Metalice Rezistente la Seism - High Strength Steel in Seismic Resistant Building Frames - HSS-SERF"*. Cu toate că scopul principal al proiectului s-a concentrat pe utilizarea oţelurilor de înaltă rezistenţă în cadrele metalice, o atenţie particulară a fost consacrată conexiunii dintre ţevile metalice rectangulare şi betonul care umple aceste elemente. Cercetarea s-a bazat pe un program experimental, cu scopul evaluării introducerii încărcărilor în stâlpii cu secţiune compusă realizaţi din ţevi rectangulare din oţel de înaltă rezistenţă umplute cu beton.

Urmărind un subiect abordat în cadrul tezei de doctorat, candidatul a explorat ***zonele disipative ale cadrelor metalice şi compuse oţel-beton***. Pornind de la cercetările anterioare experimentale şi analitice nedezvoltate complet în teza de doctorat, candidatul a publicat mai multe lucrări referitoare la *ductilitatea panoului de inimă a stâlpului metalic*, dintre care două indexate ISI. Alte investigaţii privitoare la zonele disipative ale cadrelor metalice şi compuse oţel-beton au inclus *zonele de îmbinare ale cadrelor necontravântuite (MRF) respectiv elementele de tip link în cazul cadrelor contravântuite excentric (EBF)*. Studiile de cercetare au fost conduse în principal prin integrarea candidatului în două granturi de cercetare naţionale: grantul PNCDI II „Parteneriate", contract nr. 31.042/2007 *"Sisteme structurale si soluţii tehnologice inovative pentru protecţia clădirilor la acţiuni extreme in contextul cerinţelor pentru dezvoltare durabila PROACTEX*" şi respectiv grantul de tip CEEX-ET, modului II, contract 1434/27.04.2006: *"Structuri metalice duale cu elemente disipative demontabile pentru construcţii amplasate în zone seismice"*. Studiile s-au axat pe comportarea experimentală şi numerică a zonelor disipative din cadrele duale de tip MRF+EBF. O atenţie sporită a fost dedicată caracterului compus al secţiunilor.

Ca urmare logică a studiilor efectuate la nivelul zonelor disipative, au fost investigate ***implicaţiile la nivel structural*** printr-o serie de cercetări în principal în cadrul aceloraşi proiecte de cercetare menţionate mai sus. În cazul mişcărilor seismice importante răspunsul structural depinde direct de comportarea elasto-plastică a elementelor şi a articulaţiilor dezvoltate de acestea. Investigaţiile numerice au fost bazate pe analize ale cadrelor metalice cu înălţime medie şi mică, considerând interacţiunea grinzii metalice cu placa din beton armat.

Recent, candidatul a fost integrat în echipa de cercetare a proiectului naţional 55/2012 (PN-II-PT-PCCA-2011-3.2-1303): „*Concepţia structurala si proiectarea pe baza controlului mecanismului de cedare a structurilor multietajate supuse la acţiuni accidentale (CODEC)”* pentru comportarea elementelor compuse oţel-beton. Proiectul este dedicat comportării la robusteţe a cadrelor din oţel şi compuse oţel-beton în scenariul pierderii unui stâlp. Rezistenţa la cedări progresive este o măsură a robusteţii structurale şi se bazează primordial pe rezistenţa unor elemente cheie, a continuităţii dintre elemente, a ductilităţii acestora şi a îmbinărilor dintre elemente. Natura diferită şi intensitatea evenimentelor de încărcare extremă face dificilă dezvoltarea unor cerinţe explicite de proiectare pentru astfel de situaţii de proiectare. O strategie mai bună este limitarea extinderii avarierilor în cazul unor astfel de evenimente, astfel încât cedările progresive să nu fie iniţiate.

A doua direcţie de cercetare dezvoltată de către candidat este referitoare la ***Dezvoltarea Durabilă a Clădirilor***. Topica este detaliată în Capitolul 3 al tezei. Subiectul a fost dezvoltat după integrarea candidatului în echipa grantului internaţional COST C25 (2006-2010) tip TUD COST C25 "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*". Subiectul a fost dezvoltat de către candidat în două direcţii de cercetare: (i) *abordarea pentru construcţiile noi cu utilizare intensivă a oţelului*, respectiv (ii) *soluţii de reabilitare sustenabilă a clădirilor existente*.

Abordarea primului subiect (***dezvoltarea durabilă a construcţiilor noi cu utilizare intensivă a oţelului***) a fost realizată prin intermediul integrării candidatului în echipele de cercetare ale unei serii de granturi: grantul european tip RFSR (CT-2010-00027) „*Sustainable Building Project in Steel – SB_STEEL*” , grant cu partener industrial (finanţat de ARCELOR MITTAL) “*Affordable House*” şi recent grantul RFS2 (CT-2013-00016): "*Large Valorisation on Sustainability of Steel Structures – LVS3*". Principalul scop al cercetărilor a fost cuantificarea impactului asupra mediului, aspecte economice şi sociale pentru structurile metalice. Cea mai importantă direcţie în proiectarea integrată a structurilor noi este găsirea unor bune corelări cost-confort. Un parametru suplimentar poate fi în anumite cazuri timpul de construire. Analizele de impact asupra mediului au fost efectuate cu două scopuri principale: analize interne efectuate cu scopul de identificare a surselor majore de impact şi respectiv analize externe pentru comparaţii cu alte soluţii şi/sau alte locaţii din Europa (proiectul SB Steel). Principalele rezultate ale studiilor au condus la publicarea a cinci lucrări în jurnale (din care una indexată ISI) şi şaptesprezece lucrări la conferinţe dintre care una indexată ISI.

Reabilitarea în parametri de dezvoltare durabilă a stocului de clădiri existente reprezintă una din provocările domeniului din România: în momentul actual mai mult de o treime din populaţia României trăieşte în aproximativ 84000 de blocuri de apartamente construite între 1960 şi 1990 cu probleme importante care trebuie controlate. Repartiţionarea interioară a apartamentelor pe direcţie orizontală sau verticală poate îmbunătăţi substanţial confortul locatarilor. Cuplarea apartamentelor pe orizontală sau verticală poate oferi noi configuraţii interioare şi pot conduce la perspective noi ale spaţiului cu implicaţii la o scară mai largă asupra comunităţii locale, cum ar fi scăderea densificării ariei urbane. Structural, ambele tipuri de intervenţii sunt posibile dar efectuate cu atenţie sporită în zonele de intervenţie: (i) atunci când sunt practicate goluri în diafragmele verticale acestea trebuie ranforsate cu cadre metalice sau cămăşuiri cu beton; (ii) dacă golurile sunt practicate în diafragmele orizontale de planşeu, este necesară armarea suplimentară a golului nou creat.

O altă metodă de îmbunătăţire a performanţei globale a clădirilor realizate din panouri mari prefabricate din beton este prin supraetajare. Această soluţie oferă un spaţiu adiţional locatarilor şi un sistem adecvat de acoperiş. În cadrul studiului au fost investigate mai multe soluţii de mansardare utilizând cadre metalice. Cercetările au fost conduse în cadrul proiectului de cercetare ERA-NET (3002/2011): "*INSPIRE - Integrated strategies and policy instruments for retrofitting buildings to* *reduce primary energy use and GHG emissions*" şi a condus la publicarea a patru lucrări în jurnale şi opt prezentări la conferinţe.

În strânsă legătură cu activitatea de cercetare, candidatul a contribuit la implementarea sistemului normativ Eurocode în România prin traduceri ale documentelor şi realizarea anexelor naţionale, proces coordonat la nivel naţional de către ASRO. Candidatul a fost direct implicat în realizarea anexelor pentru EN 1994 1-1 , EN 1993-1-4, EN 1993 1-6, EN 1999 Part 1-3, EN 1999-1- 4 şi EN 1999-1-5 şi traducerea în română a EN 1993-1-4, EN 1993-1-5 şi EN 1999-1-5.

În aceeaşi direcţie trebuie notată implicarea candidatului în realizarea ghidului de proiectare "*Calculul şi proiectarea îmbinărilor structurale din oţel în conformitate cu SR EN 1993-1-8. Recomandări, comentarii şi exemple de aplicare.*" printr-un contract cu Ministerul Dezvoltării Regionale (2010) pentru uzul inginerilor constructori şi al studenţilor.

Cea mai mare parte a cercetărilor menţionate mai sus au fost efectuate împreună cu doctoranzii Departamentului de Construcţii Metalice şi Mecanica Construcţiilor (CMMC) al Universităţii Politehnica din Timişoara.

Candidatul este membru al Convenţiei Europene de Construcţii Metalice (ECCS) cu activităţi în două din comitele tehnice ale acesteia:

- ECCS TC11 – Structuri compuse oţel-beton;
- ECCS TC14 – Dezvoltarea Durabilă şi Eco-Eficienţa Construcţiilor Metalice.

Se poate observa faptul că acestea urmăresc topice identice cu direcţiile de cercetare dezvoltate de către candidat.

Activitatea de cercetare a candidatului a fost concretizată inclusiv prin publicarea unor cărţi în domeniul construcţiilor metalice şi compuse oţel-beton respectiv dezvoltării durabile a construcţiilor ca autor, autor de capitole sau editor. De asemenea, activitatea didactică a candidatului este legată de principalele teme de cercetare descrise în capitolul 2 al tezei (comportarea seismică a structurilor compuse oţel-beton) respectiv capitolul 3 (dezvoltarea durabilă a construcţiilor). Candidatul a susţinut mai multe prezentări invitate la seminarii pentru studenţii doctoranzi şi de master, precum şi în cadrul cursurilor de vară organizate de studenţii de la ciclul de licenţă.

Implicarea candidatului în granturi de cercetare naţionale şi internaţionale ca director de grant sau membru în echipa de proiectare a asigurat nivelele relevante de competenţe asupra managementului proiectelor. Un aspect important în dezvoltarea ulterioară a carierei candidatului este realizarea unei echipe de cercetare la Universitatea Politehnica axată pe cele două direcţii de cercetare: construcţii compuse oţel-beton respectiv dezvoltare durabilă. Candidatul intenţionează să recruteze potenţiali doctoranzi din cadrul masteranzilor care au abordat subiectele similare de cercetare în cadrul dizertaţiilor. Trebuie menţionat faptul că până în momentul de faţă candidatul a ghidat trei doctoranzi în realizarea tezelor de doctorat, iar în momentul actual candidatul este implicat în ghidarea altor patru studenţi doctoranzi în cadrul Universităţii Politehnica din Timişoara.

B. SCIENTIFIC, PROFESSIONAL AND ACADEMIC ACHIEVEMENTS

1. INTRODUCTION

The present thesis summarises the most relevant research activity of the candidate post Ph.D. defence. The candidate passed the Ph.D. defence at the Institut National des Sciences Appliquées de Rennes, France in a co-tutoring thesis with Politehnica University of Timişoara, Romania in November 2003. The activity presented herein is considered as significant in terms of originality and importance for anticipating an independent development of the further research activity and teaching career.

The post-doctoral activity is addressed in two main thematic directions developed by the candidate: (i) ***Seismic Behaviour of Steel and Concrete Composite Structures*** presented in Chapter 2 below and respectively (ii) ***Sustainable Development of Buildings***, presented in Chapter 3.

Continuing the main theme of the Ph.D. thesis – ***seismic behaviour of composite structures***, the candidate obtained by competition a research grant (name of the grant: *"Numeric and Experimental Study on the Connecting Devices between Steel and Concrete for Composite Buildings" Located in Seismic Zones*) soon after his Ph.D. defence, offered by the Romanian Ministry of Education and covering the seismic ***behaviour of connecting devices*** between steel and concrete in composite elements. The grant investigated experimentally and then numerically the cyclic behaviour of connecting devices, by variation of different parameters such as type of connectors, loading type, steel profile flange class and concrete class. The results show that in monotonic response, the global behaviour of connectors (judged in terms of Force-Slip curve) depends mainly on the type of connectors chosen. The ductility under monotonic loading could be characterized as satisfactory (according to EN 1994-1 4). However, cyclic loading in inelastic range drastically diminished the slip capacities in all the cases considered. The cyclic loading introduces an important reduction in the characteristic shear resistance P_{Rk}, ranging from 10 to 40% as compared to the corresponding monotonic response. Consequently, the 25% reduction in the connector's resistance (as requested by EN 1991-1 chapter 7) is justified for headed stud connectors, but in other cases this reduction is insufficient. The subject is largely developed in chapter 2.1. The results of the research have been disseminated in two journal papers and six papers in conferences among which two ISI indexed and presented in the presented in the list of publications.

Since 2010 the research subject was enriched by the study of the connection between steel Rectangular Tubes and concrete through the use of shot nails, by the integration of the candidate in the team of the European project type RFCS *"High Strength Steel in Seismic Resistant Building Frames - HSS-SERF"*. Although the main purpose of the research was focused on the global use of high strength steel in building frames subjected to seismic loads, a particular attention was devoted to the connection between steel column tubes and in-filled concrete. The research was based on an initial experimental program, aiming to evaluate the load introduction within composite columns realized as CFT of high strength steel. For this purpose load introduction tests were performed on six specimens varying parameters such as: loading procedure (monotonic, cyclic), connection type (steel-concrete bonding, steel-concrete bonding combined with connectors), and steel grade (S460, S700). The second step of the research investigated the failure conditions and proposed, based on a numerical analysis analytic formulae for ultimate resistance of fire-shot connectors. The main conclusion of the study shows that the presence of shear connectors lead to a significant contribution to the load transfer from steel tube to the concrete core, in both monotonic & cyclic loading. Concerning failure analysis, under monotonic loading the concrete was crushed at the contact with the nails which bent. In Under alternating cycles the nails broke at the interface between concrete core and steel tube. The results of the research were disseminated in two publications, one journal (international Database) and one international conference while other publications are in progress.

Following another subject touched in his Ph.D. thesis, the candidate explored the ***dissipative zones in steel and steel and concrete composite frames***. Based on the previous experimental and analytical work, not entirely developed in the Ph.D. thesis, the candidate published several papers on the *ductility of Column Web Panel Zone*, among which two on ISI indexed journals.

Other dissipative zones of steel and composite frames included the *joint zones for Moment Resisting Frames (MRF) and link elements in case of Eccentrically Braced Frames (EBF)*. The research was conducted mainly by integration of the candidate into two Romanian research grants: grant PNCDI II "Partnerships", contract no. 31.042/2007 "*Structural systems and innovative technological solutions for the protection of buildings subjected to extreme actions in the context of sustainable development PROACTEX*" and respectively the grant type CEEX-ET, module II, contract 1434/27.04.2006: *"Dual steel structures with dissipative removable links for structures located in seismic zones"*. The study concerned experimental and numerical behaviour of dissipative zones in dual MRF+EBF structures. A special attention was paid to the composite aspect. In case of composite dual moment resisting and eccentrically braced frames, the current design practice is to avoid the disposition of shear connectors in the expected plastic zones, and consequently to consider a symmetric moment or shear plastic hinges, which occur only in the steel beam or link. Even without connectors, the real behaviour of the hinge may be different from the symmetric assumption, since the reinforced concrete slab is connected to the steel element close to the hinge locations, and also due to contact friction between the concrete slab and the steel element. When using composite beams with Reduced Beam Section (RBS) or composite link elements, the European norms in force are poor in detailing and requirements, practically limiting the composite interaction up to the physical boundaries of the dissipative element according to paragraphs 7.6.2, 7.7.1, 7.7.5 and 7.9.3 of EN 1998-1. Thus, the plasticization is thought of as for a steel element, ignoring the fact that the beam is composite up to the boundaries of the dissipative element. For ordinary design, the influence of the adjacent composite beam on the dissipative capacity of the dissipative element is very difficult to consider. The experimental-based study was done considering two ductile sub-structures: short links (through one-storey EBFs tests) and respectively beam-to-column joints of MRF. The research has shown that the simple disconnection of the steel beam from the concrete slab over the dissipative zone is not sufficient to ensure a pure steel-like behaviour of the element. Also, the resulted behaviour is practically very close to that of a full-composite specimen. Another conclusion shown that for both dissipative zones (joints and links) the use of composite sections improves the global resistance and stiffness characteristics of the dissipative zones, while maintaining the ductile nature of the solution. The results of this research have been disseminated into two journals (among one ISI indexed) and eight papers sustained at national and international conferences.

Following the investigation of the dissipative zones of frames, the ***implications on the structural response*** were investigated in a series of studies mainly within the same research grants as mentioned before. The structural response in the case of important seismic motions depends directly on the elasto-plastic behaviour of elements and hinges. The numerical investigation considered elasto-plastic analyses of low and medium height steel frames, considering the interaction of the steel beam with the concrete slab. Several parameters, such as the inter-story drift, plastic rotation requirements and behaviour factors q were monitored. In order to obtain accurate results, adequate models of plastic hinges were proposed for both the composite link and composite reduced beam sections. The results of the research, presented in more detail in chapter 2.3 below, were published as papers into two journals and eleven conferences (among which four ISI indexed). The analysis made at structural level revealed that the composite aspect of the dissipative zones plays an important role in the overall behaviour of frames: structures where the interaction between steel and concrete was modelled have had a different behaviour from the bare steel ones. Low-rise steel structures (4 stories, 5 stories) show higher drift and rotation requirements than the similar frames modelled with composite beams. For the high-rise structures, with a higher vibration period, the increase in strength and rigidity induced by the composite effect also leads to smaller rotations in links and RBS. Consequently, in an optimum design, smaller sections could be considered in the case of composite elements, leading to an overall economy in terms of steel.

Lately, the candidate was integrated in the team of the national grant 55/2012 (PN-II-PT-PCCA-2011-3.2-1303): *„Structural conception and design based on control of the failure mechanism of multi-storey frames subjected to accidental loads (CODEC)*" for the behaviour of composite elements. The project is devoted to the robustness behaviour of steel and composite frames in the case of a column loss. Progressive collapse resistance is a measure of the structural robustness and relies primarily on resistance of key elements, continuity between elements and ductility of elements and their connections. The different nature and intensity of extreme loading events make difficult the development of explicit design requirements for such design situations. A better strategy is to limit the extent of damage in case of such events, so that the progressive collapse is not initiated. The partial conclusions of the on-going project show that the contribution of the concrete slab to the ultimate resistance of the structure increases when shear studs are welded on main and secondary beams. The catenary forces develop under large displacements and lead to a significant increase of the ultimate resistance.

The second subject of research covered by the candidates is related to the ***Sustainable Development of Buildings***. The topic is detailed in Chapter 3 of the present thesis. The subject was developed by the candidate after his integration in the team of the international grant COST C25 (2006-2010) type TUD COST C25 "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*". Following the main objective of the programme – *promotion of a science-based developments in sustainable constructions in Europe through the collection and collaborative analysis of scientific results concerning life-time structural engineering and especially the integration of environmental assessment methods and tools for structural engineering* – , during the participation in COST C25 action, the candidate performed first Life Cycle Assessments for the evaluation of environmental impact in buildings.

There could be mentioned two main themes addressed by the candidate in the topic of sustainable development of buildings: (i) ***approach for new steel-intensive structures*** and respectively (ii) ***sustainable retrofitting solutions for existing building stock***.

The topic of ***sustainable development of steel-intensive buildings*** was achieved through a series of research grants related to the topic: European grant type RFSR (CT-2010-00027) „*Sustainable Building Project in Steel – SB_STEEL*" , industry grant (founded by ARCELOR MITTAL) "*Affordable House*" and recently RFS2 (CT-2013-00016): "*Large Valorisation on Sustainability of Steel Structures – LVS3*". The main purpose of the research was the quantification of environmental impacts, economic and social aspects for steel based structures. The research was conducted on real and conceived case-studies among which there could be mentioned:

- Constantin House (real project in Ploiesti)
- Arghirescu Building (real building in Timisoara)
- Multi-storey office building (real building in Constanta)
- Affordable steel house (Affordable house project)

Concluding on the mentioned case-studies, it can be said that the most important direction in the integrated design of new structures is to find a good relation between cost and comfort. A supplementary parameter could be in some cases the erection time. In this context, an innovative structure-envelope should:

- use applications of industrial building technologies in dwelling building systems (residential application) providing a fast erection and fabrication time. The basic assumption is that a sustainable building should rely on standard details and common technologies, available to most of the builders, instead of experimenting with new materials without track record;

- development of modular systems in such a way that at any time the client can add new modules, both by vertical and/or horizontal addition, with high solution diversity for flooring and envelope;
- the design of for envelope systems (walls, slabs, roofs etc) should be based on performance based design, including not only structural performance but also should consider aspects related to internal comfort such as thermal, humidity, air flow and so on.
- using structural systems made of lightweight steel frames, hot-rolled profiles or wood framing assures the lightness of the building and proper response to climatic and seismic loading.

The environmental impact analyses were made with two main purposes: internal analysis for the identification of the main sources of environmental impact and respectively for comparison with other structural solutions and/or other locations in Europe (SB Steel Project). The main outcomes of the research lead to the publication of five papers in journals (one ISI indexed) and seventeen papers at conferences among which one ISI indexed.

The ***sustainable retrofitting solutions for existing building stock*** represents one of the issues of large interest in Romania: in this moment more than one third of the Romanian population lives in about 84000 block of flats (apartment house type) built between 1960 and 1990. The analyses performed at structural level revealed the fact that from the resistance point of view, the block type buildings satisfy in the large majority the actual normative requirements (both Eurocode and national norms). However, minor structural problems are regarding the joints of concrete precast panels, due to bad execution and/or ageing. Nevertheless, the main problem of these types of buildings is the low thermal efficiency of their envelopes. Taking into account that Romania has predominantly a continental climate, this issues lead to human discomfort during cold and warm seasons, as well as to large amount of energy dissipation. Different techniques can be applied for retrofitting such buildings but they have to be compliant with existent structure and envelope. For an integrated design that include sustainability, the solutions that may be considered have to respect the criteria imposed in evaluation, as well as a conceptual global methodology. However, the final choice in retrofitting of old buildings represents a decisional task, based on a multi-criterial analysis. Economic, social and environmental aspects should be considered. Several methods could be employed in decisional process for choosing a final solution.

Also, the study evaluated the interior repartitioning of concrete residential buildings which can improve the comfort of inhabitants. The apartment coupling on horizontal or on vertical can conduct to new internal configurations and can offer new interior space perspectives with implications at a larger scale on the local community, such as the decrease of densification in the urban areas. Structurally, both types of interventions are possible but care should be given at local interventions: (i) when cuts in the vertical diaphragms are performed, these should be reinforced by additional steel frames or concrete jacketing; (ii) if cuts are made on horizontal diaphragms, additional reinforcement near the cut zone is needed.

Another method of improving the overall performance of large precast concrete panel buildings id by overcladding. This solution offers additional space to inhabitants and also an adequate roofing system. Several possibilities of overcladding were investigated by using steel-intensive solutions. These represent ideal systems for overcladding of the existing large precast concrete panel buildings due to their lightness, reversibility and clean sites. Also, they can adapt to existing structural systems and various structural connection typologies can be realised between old and new structures. The results show that the base structures are robust and safe even to new generation of norms and loading conditions. Concerning the environmental impact, the analyses have shown that the recyclability of the steel elements lowers the overall impact of the overcladding systems. In this manner, the impact is due mainly to thermal and hidro insulation materials, which for now are with low recyclability and or reusability.

The studies were performed within the ERA-NET research grant (3002/2011): "*INSPIRE - Integrated strategies and policy instruments for retrofitting buildings to reduce primary energy use and GHG emissions*" and led to the publication of four journal papers and eight papers presented in conferences.

Directly connected with the research activity, the candidate was taking part in the implementation of Eurocode system in Romania through translation of documents and realisation of National Annexes, process coordinated at the national level by ASRO (Romanian Association of Standardisation):

- Realisation of the **national annexes** for EN 1994 1-1 Eurocode 4: Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings; Design of steel structures - Part 1-4: General rules - Supplementary rules for stainless steels EN 1993 1-6 Eurocode 3: Design of steel structures - Part 1-6: Strength and Stability of Shell Structures; EN 1999 Eurocode 9: Design of aluminium structures - Part 1-3: Structures susceptible to fatigue, and Eurocode 9: Design of aluminium structures - Part 1- 4: Cold-formed structural sheeting and EN 1999 Design of aluminium structures - Part 1-5: Shell structures.
- **Translation** of EN 1993 Design of steel structures - Part 1-4: General rules - Supplementary rules for stainless steels, EN 1993 Design of steel structures - Part 1-5: Plated structural elements and EN 1999 Design of aluminium structures - Part 1-5: Shell structures

All the documents were realised at the Department of Steel Structures and Structural Mechanics, Politehnica University of Timisoara under the supervision of Prof. Dan Dubina.

In the same direction it could be noticed that the candidate was directly involved in the realisation of the design guide "*Design of steel structural connections according to SR EN 1993-1-8. Recommendations, comments and design examples*" in a contract with the Ministry of Regional Development (2010) for use of civil engineers and students as well.

Most of the research mentioned above was done in cooperation with the Ph.D. students within the Department of Steel Structures and Structural Mechanics from The Politehnica University of Timisoara. For this reason the candidate was and is involved in guiding a number of Ph.D. students in subjects related to the main of research activities:

- Danku Gelu, Ph.D. – research related to composite behaviour of dissipative zones in MRF and EBF;
- Vulcu Cristian, Ph.D. – composite RHS columns with shot nail connectors; beam-to RHS column design.
- Nicolae Muntean, Ph.D – seismic behaviour of steel connections;
- Alexandru Botici, Ph.D. – sustainable retrofitting of existing building stock;
- Ioan Marginean and Andreea Handabut, Ph.D students – robustness performance of steel and composite MRF;
- Floricel Andra, Ph.D student – Sustainability of existing building stock;
- Vataman Oana, Ph.D student – Static and Cyclic ductility of steel and composite dissipative zones in MRFs and EBFs.

The candidate is member of the European Convention of Structural Steelwork (ECCS), with activities in two of the Technical Committees (TC):

- ECCS TC11 – Composite Structures;
- ECCS TC14 - Sustainability & Eco-Efficiency of Steel Construction.

It could be noticed that they have identical directions to the main research topics addressed by candidate.

The research activity of the candidate was embodied also in a series of books in the fields of composite steel and concrete structures and sustainability of buildings as author / author on chapters or editor among which:

- Ciutina A. Comportarea structurilor în cadre compuse din oţel-beton şi a îmbinărilor acestora, Editura Orizonturi Universitare (Behaviour of framed composite steel and concrete structures and their connections), Timişoara, 2007 ISBN 978-973-638-337-3;
- Ciutina A. Design of Steel and Concrete Composite Elements, Timişoara, Ed. Orizonturi Universitare, 2014 ISBN 978-973-638-578-0.
- Ungureanu V., Ciutina A., Koukkari H. Sustainable building projects in steel - case studies for a new design approach. Editura Orizonturi Universitare, 2013, 978-973-638-526-1

The didactic activity of the candidate is also related to the main research themes described in chapters 2 (Seismic Behaviour of Steel and Concrete Composite Structures) and respectively 3 (Sustainable Development of Buildings):

- Course on Steel and Concrete Composite Structures for bachelor (Romanian and English language) and master level (English language);
- Course on Environmental Impact of Buildings (bachelor level - Romanian) and applications on Sustainable Development (master level – English).

The candidate sustained several invited presentations in workshops for Ph.D. and master level students as well as summer-courses for bachelor students.

Relevant Publications:

The list of publications presented below is considered as relevant for the professional achievements obtained by the candidate, sustaining his activity presented in the Habilitation Thesis for the post-doctoral period. The publications are listed in the order of appearance:

1. **A. Ciutina**, G. Danku, D. Dubina, „Influence of steel-concrete interaction in dissipative zones of frames: I – Experimental study" Steel and Composite Structures, Vol. 15, No. 3 (2013), ISSN 1229-9367, pp: 299-322.

2. G. Danku, **A. Ciutina**, D. Dubina, „Influence of steel-concrete interaction in dissipative zones of frames: II – Numerical study" Steel and Composite Structures, Vol. 15, No. 3 (2013), ISSN 1229-9367, pp: 323-348.

3. Tuca, I., Ungureanu, V., **Ciutina, A**., Dubina, D., 2012, Life-cycle assessment of a steel framed family house 2012 Pollack Periodica 7 (1).

4. Vulcu, C., Stratan, A., **Ciutina, A.**, Dubină , D. Beam-to-column joints for seismic resistant dual-steel structures, 2011, Pollack Periodica 6 (2) , pp. 49-60

5. **A. Ciutina**, Tuca, Iulia, Ugureanu, Viorel, Dubina Dan "Design of Buildings Including Environmental Impact", Environmental Engineering and Management Journal, Vol. 9-2010 ISSN 1582-9596, pp. 1121-1133.

6. Florea Dinu, Dan Dubina, **A. Ciutina** "Robustness performance of seismic resistant building frames under abnormal loads" proceedings of: „Structures and Architecture", Guimaraes, Portugal, 21-23 July 2010, ISBN 978-0-415-49249-2.

7. D. Dubina, V. Ungureanu, **A. Ciutina**, I. Tuca, M. Mutiu, 2010 Sustainable detached family house — case study, Steel Construction, Volume 3, Issue 3, pages 154–162, September 2010

8. **A.L. Ciutina**, and D. Dubina "Column Web Stiffening of Steel Beam-to-Column Joints Subjected to Seismic Actions" ASCE, Journal of Structural Engineering, Vol.134, No.3, Mar. 2008, ISSN 0733 – 9445, pp.505-511.

9. **Ciutina, A.L**., Dogariu, A., Behaviour of different connectors under monotonic and cyclic loading, Source: Proceedings of the 3rd International Conference on Steel and Composite Structures, ICSCS07 - Steel and Composite Structures, p 779-785, 2007.

10. A. Lachal, J.M. Aribert, **A. Ciutina.** "Seismic Performance of End-Plate Moment Resisting Composite Joints", Proceedings of Composite Construction In Steel And Concrete V: July 18-23, 2004 The Kruger National Park Conference Centre Berg-en-Dal, Mpumalanga, South Africa Ed. by the American Society of Civil Engineers – ASCE, ISBN 0-7844-0826-2, pp. 631-640.

2. SEISMIC BEHAVIOUR OF STEEL AND CONCRETE COMPOSITE STRUCTURES

2.1. Introduction

The term "composite construction" describes the association of two or more materials, which are used in order to form elements that perform better than the materials used individually. In this context, the association of steel and concrete with buildings and civil engineering structures is judged by engineers as a very effective way of enhancing structural performance. However, the design should consider the inherent differences between the properties of the two materials used: steel and concrete. Moreover, careful attention should be paid to the forms of interconnection between the two materials:

- steel structural elements are generally fabricated from thin elements (welded or rolled profiles) and have a very good behaviour in tension, but are prone to local and global buckling in compression;
- concrete elements are usually massive and present a good behaviour in compression. In exchange, the tensile behaviour of concrete is poor and is long-term actions such as creep and shrinkage is likely to occur.

The composite elements can enhance the global behaviour of structures. However, the design of composite elements should also consider other particular aspects such as the change of resistance of beams in bending unlike steel beams or concrete beams with similar disposition of reinforcing bars which offer the same resistance in both sagging and hogging bending. Also, in case of composite columns, a smaller or larger concrete area can be compressed in direct function with the compression/bending forces which act on the element.

The most used form of composite construction is considering ***composite beams***, in which a steel joist is connected to the concrete slabs usually by mechanical connectors. Considering its application and the imagination of the design engineer, different shapes could be used for the cross-section. The concrete slab can be plain or supported on corrugated steel sheeting, as shown in Figure 1. The ribs of the steel sheeting can be parallel or perpendicular to the steel beam, disposed in function of the shorter distance between main and secondary beams. In rare cases the ribs could be oriented oblique to the steel elements. The deck is executed by cold-forming plain galvanised steel sheeting and is designed to resist casting of concrete and could be designed to act compositely with the hardened concrete.

The current design of composite beams considers a section formed by the steel with a part of concrete on a so-called "effective width". Considering that the characteristics of the two materials is quite different, the design is usually made on an equivalent steel section, by converting concrete into equivalent steel. Unlike steel elements, the behaviour of composite beams is different in hogging and sagging bending, as concrete is in tension respectively compression.

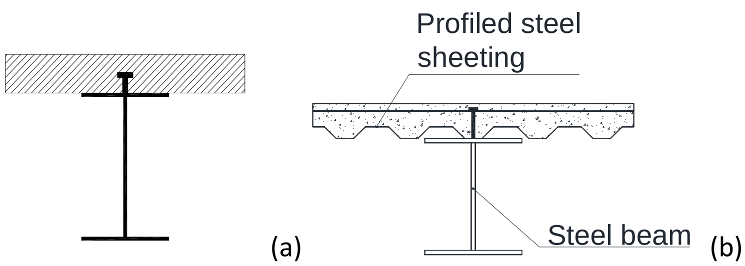


Figure 1. Main typology of composite beams: plain slab (a) and composite slab (b).

Composite columns represent other form of application of composite construction. The current usage on composite columns considers two main categories: as steel elements encased totally or partially in concrete and respectively circular or rectangular hollow sections filled with concrete (see Figure 2). In first case the columns are made by encasement of one or more steel profiles, usually I shapes. The final form of the columns can be rectangular or circular and is completed by supplementary longitudinal reinforcement. The steel tubular sections named also Concrete Filled Tubes – CFT are made by filling the steel circular or rectangular elements with plain or reinforced concrete. However, for both typologies (rectangular or circular CFT), the loads are shared between the two materials. The resulting structural elements are prone to local buckling while the interior concrete is in tri-axial compression.

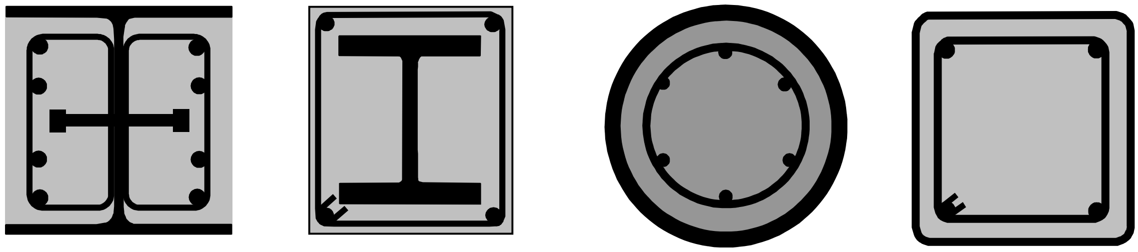


Figure 2. Typical cross-sections of composite columns.

Because the section of composite columns contains both steel and concrete materials, the behaviour of such elements can be assimilated to the behaviour of steel or concrete columns. In practical design both approaches are correct as far as the composite aspect is considered. The design of composite columns is based on the bending moment – axial force interaction diagram, as shown in Figure 3. For static ultimate and serviceability limit design, it is considered that there is a total interaction between steel and concrete materials. This implies that the forces are linearly distributed over the cross-section and no relative slip exists between two materials. This hypothesis can be considered rational as the surface of contact is large and consequently good adherences exist between steel and concrete. In cases in which the transfer between steel and concrete is not sufficient, additional connectors could be disposed.

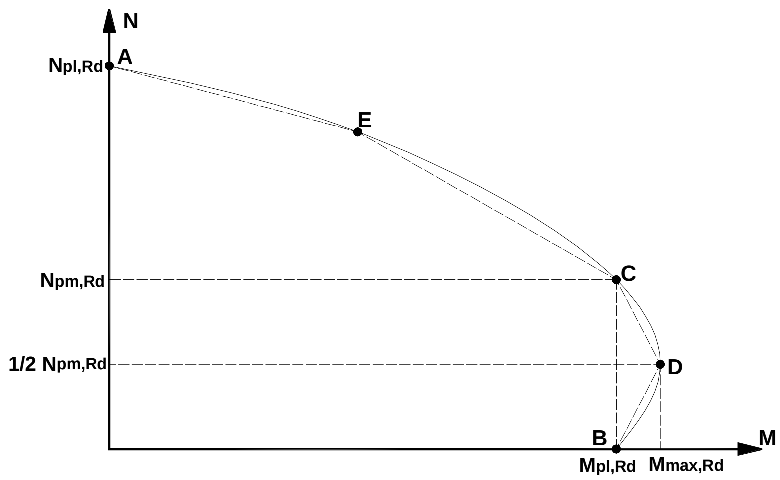


Figure 3. N-M interaction curve for composite column cross-sections.

In many cases, the connection between beams and columns is realised by using the abilities of a single material to transfer loads, as steel or concrete. Considering that both materials are employed in transmitting loads, the connection becomes composite. The advantage of ***composite connections*** is that additional capacity may be obtained if reinforced concrete is used in the overall design.

The principal purpose of a composite connection is to transmit vertical reactions of beams to columns. For moment-resisting frames, it should also transmit bending moments. However, this function is realised in a smaller or larger extent in function of the abilities of the connection to transmit rotation. From the technological point of view, there are various systems that may ensure the connection between beams and columns. Figure 4 presents some classic configurations of composite beam-to-column joints. In design, account should be taken on the asymmetry of behaviour hogging/sagging. Nevertheless, in order to achieve composite behaviour, the connection between the steel beam and the concrete slab should be ensured, as well as the continuity of reinforcement over the joint. The usual design of composite connections is by considering the so-called “component method” presented in EN 1993-1-8 and adapted for composite aspects in EN 1994-1 Section 8.

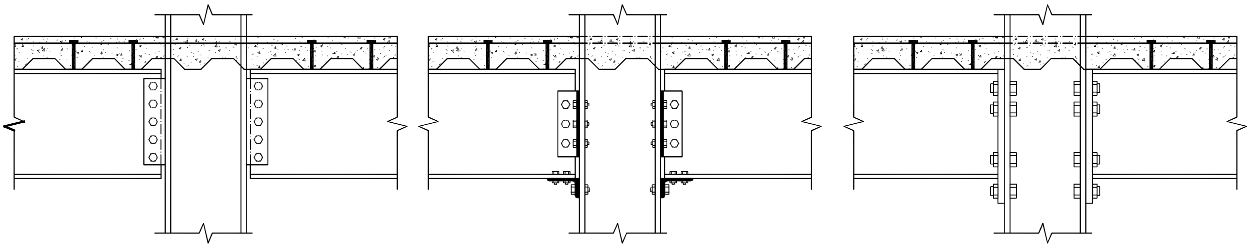


Figure 4. Usual beam-to-column composite connection configurations.

In modern applications of floor systems the steel profiled sheeting can take part in resisting loads, under the form of ***composite floor systems*** (see Figure 5). However, in order to realise a composite floor account should be taken on the ability of corrugated sheeting to prevent the slip of concrete on steel sheeting. The main advantage of composite floor is that the lower reinforcement could be reduced in some cases even not disposed. Taking into account that on the market there are various producers of corrugated sheeting, the design process should be based on experimental tests. By manufacturing, the profiled steel sheet shall be capable of transmitting horizontal shear at the interface between the sheet and the concrete; pure bond (physical-chemical bond) between the steel sheeting and the concrete is not considered as effective for a composite action.

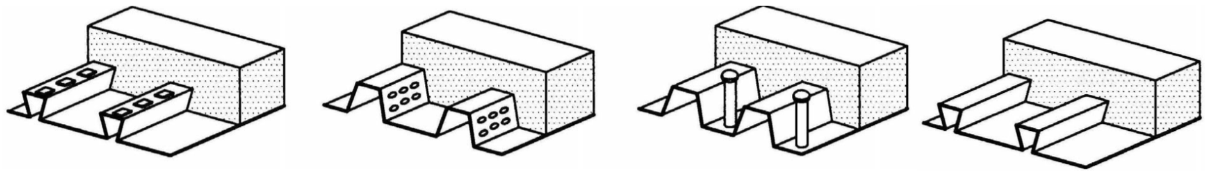


Figure 5. Typical forms of composite slabs (source EN 1994-1).

2.2. Behaviour of connecting devices

In the case of steel and concrete composite elements, the connection is essential for transmitting loads between the two materials. When the contact between steel and concrete is made on a very small surface, as in the case of steel and concrete beams, the connection is made by mechanical anchoring systems. Figure 6 schematically presents some examples of connectors working in shear. This type of connectors is mainly used for steel beams acting compositely with the concrete slab.

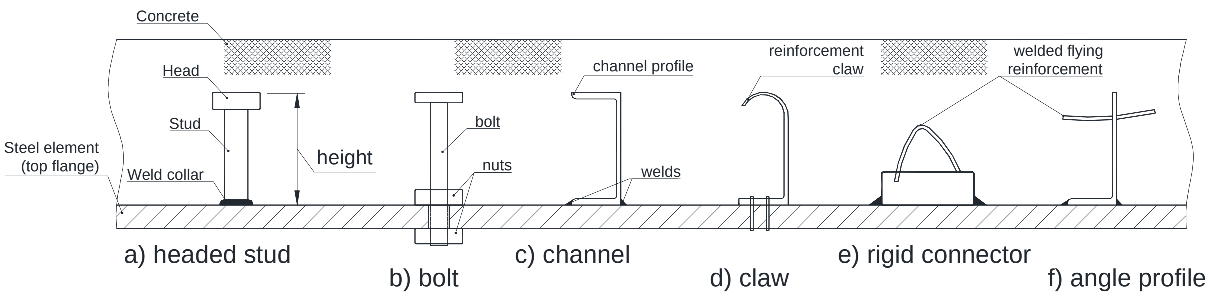
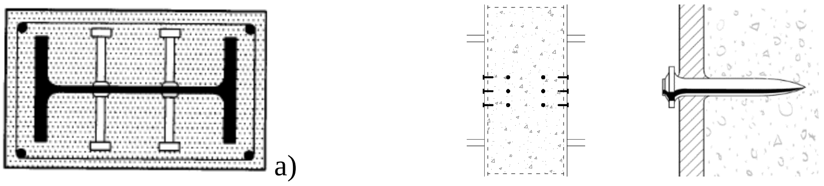


Figure 6. Examples of beam-to-column composite connection.

A relatively new system of connectors is comb-shaped strip connectors and perfobond connectors. For both cases, the connectors are a steel strip cut in such a way as to allow the access of the lower reinforcement of the concrete slab. This type of connectors is continuously welded on the steel flange and allow for the development of longitudinal forces on the entire length of the composite beam.

For concrete encasing a steel element, such as columns, the two elements are connected by shear forces induced by the geometry of profiles as well as by chemical bond which forms naturally between steel and concrete. For this type of elements, the longitudinal shear is naturally achieved due to the relatively large contact area between steel and concrete elements. However, when the natural bond is not enough for the load introduction, generally within the connection zone, additional mechanical shear devices should be installed. For open sections, such as H or I profiles, usual headed stud connectors may be disposed, as presented in Figure 7 a). For box profiles (Figure 7 b), shot fire nails have proven to be adequate to provide additional shear.



a) b)

Figure 7. Additional shear connectors in the case of columns: a) – shear connectors for open profiles; b) – shear connectors for box sections.

In the practical application of composite design, there is a large variety of mechanical connectors which vary in terms of form, size and the method of anchoring, but all systems present some important similarities:

- they refer to steel elements encased in concrete;
- the connectors have components able to transmit longitudinal shear;
- the connectors have components able to resist forces perpendicular to the contact surface in order to prevent concrete from being separated from steel;
- the connectors transmit concentrated loads to the steel element.

The headed stud connectors, are perhaps the most usual types of connectors and are formed by a steel headed stud electrically welded on the steel flange. The stud and the welded collar are designed as to resist longitudinal shear, while the head of the connector takes over the up-lifting forces. The most usual diameter for civil engineering composite elements is 19 mm. The headed stud connectors are anchored to the steel flange before the casting of concrete on the steel shop or on site.

2.2.1. Behaviour of connecting devices for composite beams

The behaviour of composite beams is governed by the shear connection between the concrete slab and the steel section. For this reason, many types of devices have been conceived and tested by researchers in order to realize an optimum shear connection. The economic considerations continue to motivate the development of new products, while in other cases the researchers try to use new techniques for an economic use of traditional connectors. The new generation of connectors, such as the perfobond connectors seems to be a good alternative to the standard headed-stud connectors.

Standard push-out procedure given in the Annex B of EN 1994-1 provides a good tool for investigation of the shear connectors. The procedure, however, gives information only about the monotonic loading. In case of reversal loading in principal beams of composite moment resisting frames under seismic loading, the shearing of the connectors could change from one sense to the other as the bending moments on the beam changes from hogging to sagging. In literature, there are relatively few reports on the behaviour of connectors under cyclic loading, and most of them deal with the dowel behaviour of standard headed-studs connectors.

Feldmann & Gesella (2004) make a very detailed analysis on the fatigue analysis on the headed studs under non-static loading of headed studs. In low-cycle fatigue, Bursi & Ballerini (1997) analysed the low-cycle fatigue behaviour of welded stud connectors under variable and constant non-sequential phase displacement histories. A comparison between the cyclic and monotonic results led the authors to the conclusion that the strength predicted by the EN 1994-1 appears unsafe when directly applied to seismic design.

Aribert & Lachal (2000), on a research conducted on two different types of connectors, have also proved that the cyclic response lead to a major decrease in the global resistance and ductility, stressing out that in the case of cyclic loading, as is the seismic action, the use of partial-shear connection should be avoided.

The present synthesis reports the results of eleven push-out shear specimens under monotonic loading and five push-pull specimens under cyclic loading. The dimensions of specimens for the push-out standard tests (including the metallic section and the reinforcing plan) were initially derived from the paragraph B 2.2 from the Annex B of EN 1994-1. The concrete and steel profile global dimensions were kept constant for all the tests, as well as the configuration and diameter of the reinforcing bars (Φ 10mm) according to Figure B.1 of EN 1994-1.

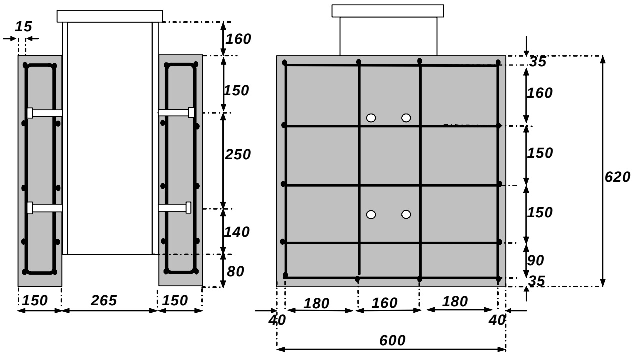


Figure 8. Test specimen for standard push test according to EN 1994-1, Annex B.

The description of push-out specimens it is shown in the Table 1. Four parameters have been considered for the study:

- type of connectors (Φ16 KO&CO headed studs on two rows, Φ22 KO&CO headed studs on one row, LL120x80x8 angle profile, UNP 120 channel profile, perforated steel plate and reinforcement anchor hooks of Φ 10mm,). Figure 1 shows a 3D view of the steel specimens (connectors included);
- concrete strength class (C25/30; C30/37);
- steel profile class (class 1 corresponding to standard HEB 260 profile, class 2 by considering a 10mm steel flange and class 3, by a flange thickness of 8mm respectively);
- monotonic (11 specimens) and cyclic loading (applied to 5 specimens).

Table 1. Description of push-out and push-pull specimens

Specimen	Type of connectors	No. of connectors	Concrete class	Steel profile
PT-16/I-M	8Φ16 (2 rows)	8	C25/30	HEB 260
PT-16/II-M	8Φ16 (2 rows)	8	C25/30	Class 2*
PT-16/III-M	8Φ16 (2 rows)	8	C25/30	Class 3**
PT-16/S-M	8Φ16 (2 rows)	8	C30/37	HEB 260
PT-16/I-C	8Φ16 (2 rows)	8	C30/37	HEB 260
PT-22-M	4Φ22 (1 row)	4	C25/30	HEB 260
PT-22-C	4Φ22 (1 row)	4	C25/30	HEB 260
PT-A-M	Reinf. hooks (Φ 10mm)	4	C25/30	HEB 260
PT-A-C	Reinf. hooks (Φ 10mm)	4	C25/30	HEB 260
PT-A/S-M	Reinf. hooks (Φ 10mm)	4	C30/37	HEB 260
PT-II-M	Perforated steel plate ***	2	C25/30	HEB 260
PT-II-C	Perforated steel plate ***	2	C25/30	HEB 260
PT-LS/II-M	L120x80x8	4	C25/30	Class 2*
PT-LS/III-M	L120x80x8	4	C25/30	Class 3**
PT-LS/I-C	L120x80x8	4	C25/30	HEB 260
PT-US-M	UNP 120	4	C25/30	HEB 260

* 260X260 profile t_f=10mm; ** 260X260 profile t_f=8mm; *** longitudinal steel plate (t=8mm) on each side of steel profile, perforated for the passage of reinforcement

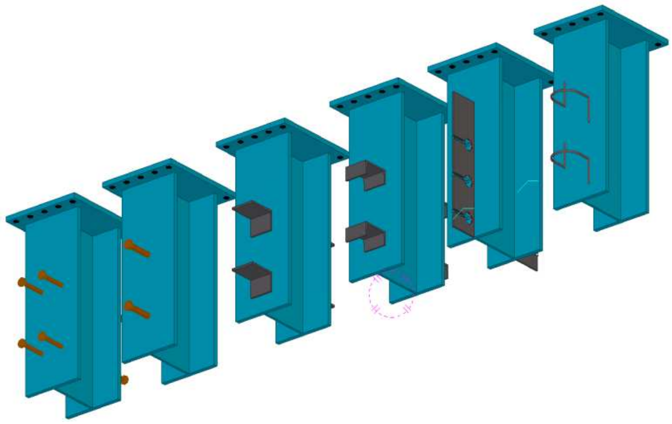


Figure 9. Disposition of connectors on steel profiles.

The testing set-up is presented in Figure 10. The load was applied in displacement control through a traction and compression actuator. In case of monotonic loading, the load was first applied in increments up to 40% of the expected failure load and then cycled 25 times between 5% and 40% of the expected failure load, in accordance to the En 1994-1 4 stipulations. Subsequent load increments have been imposed up to failure. In case of cyclic loading was applied the ECCS (1986) procedure, with the yielding displacement computed on the results obtained from the monotonic test.

According to the normative requirements, the connector shear capacity $P_{R,k}$ represents the maximum load capacity reduced by 10% and divided to the number of shear connectors. Also, the connector's slip capacity δ_u is taken from the load-slip deformation curve, as corresponding to the shear capacity $P_{R,k}$ (see Figure 10). The δ_u value was resulting from the relative slip measurements made by displacement transducers disposed on specimens.

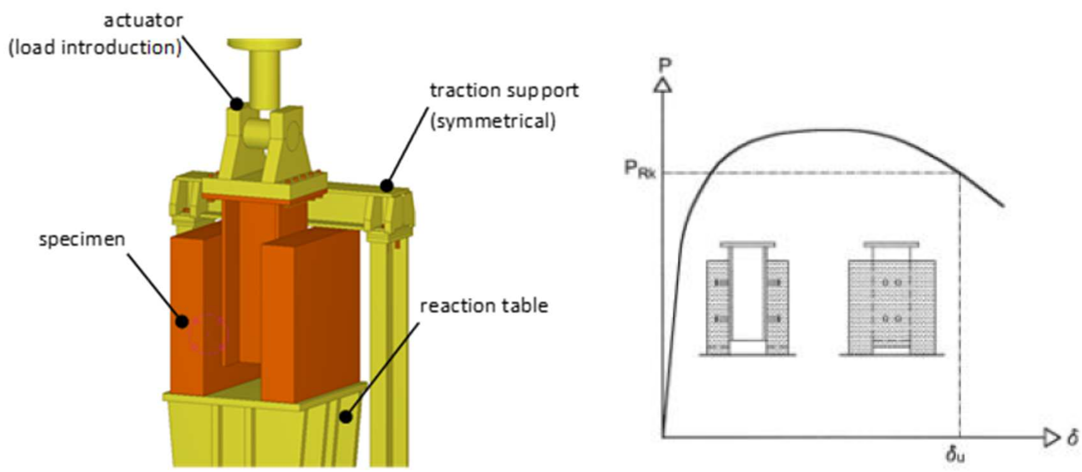


Figure 10. Testing set-up and determination of the slip capacity δ_u.

Table 2 summarizes the interpretation of experimental results derived from the monotonic curves (given in Figure 11) and the envelopes of the cyclic tests, in which:

- F_y represents the yielding force, in the sense of ECCS loading procedure (1986);
- δ_y the corresponding yielding displacement;
- $S_{j,ini}$ the initial stiffness of the F-δ curve;
- F_{max} is the maximum recorded force during testing;
- δ_u is the slip capacity of the connectors;
- $P_{R,k}$ is the connector's shear capacity.

Table 2. Main test results derived from the interpretation of experimental data

Specimen	F_y [kN]	δ_y [mm]	$S_{j,ini}$ [kN/mm]	F_{max} [kN]	δ_u [mm]	$P_{R,k}$/conn. [kN]
PT-16/I-M	396.0	0.20	1983.4	801.1	4.91	99.7
PT-16/II-M	514.6	0.37	1349.8	862.6	7.52	100.1
PT-16/III-M	501.2	0.34	1399.6	831.8	6.42	103.1
PT-16/S-M	436.2	0.17	2359.8	844.0	7.18	102.9
PT-16/I-C	430.0	0.29	1454.0	579.0	2.33	60.66
PT-22-M	405.8	0.35	1093.5	737.6	14.43	177.2
PT-22-C	407.0	0.44	871.0	474.0	2.11	95.6
PT-A-M	396.0	0.20	1813.0	590.6	7.84	132.8
PT-A-C	339.0	0.08	4361	544.3	1.98	117.9
PT-A/S-M	356.9	0.08	4331.1	649.6	6.51	146.6
PT-II-M	557.5	0.13	3701.4	1033.0	21.46	465.6
PT-II-C	403.0	0.09	3939	586	0.60	272.5
PT-LS/II-M	586.8	0.21	2677.1	794.4	2.36	179.1
PT-LS/III-M	494.6	0.10	4726.9	790.4	3.09	177.8
PT-LS/I-C	650.0	0.27	2274.0	774.3	0.98	165.9
PT-US-M	804.9	0.41	2052.9	1256.0	9.65	314.0

The results of the monotonic curves are quite dispersed. The maximum applied load ranges from 590kN (PT-A-M specimen) to 1256kN (PT-US-M specimen) while the slip capacity ranges from 2.36mm (PT-LS/II-M) to almost 22 mm (PT-II-M specimen). For a better understanding of results, an insight analysis is made below in function of analysed parameters.

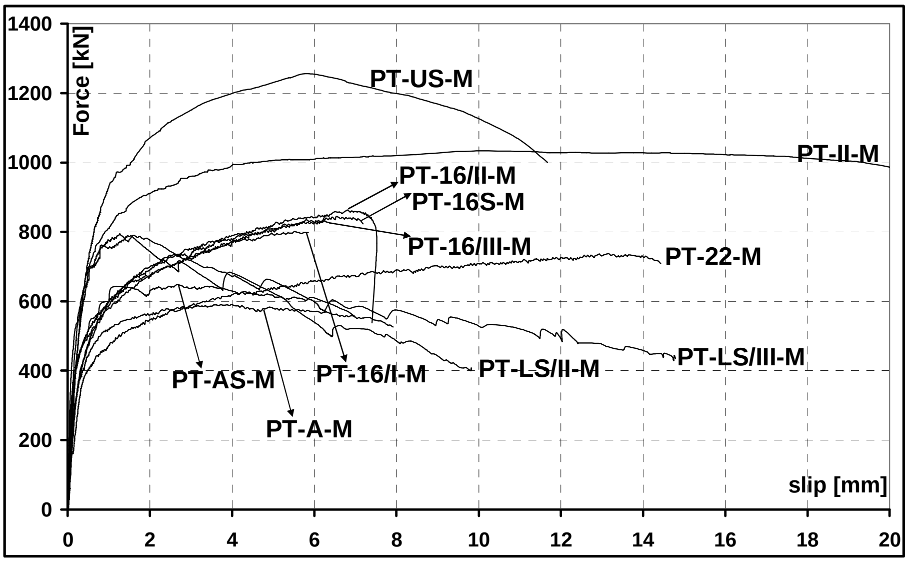


Figure 11. Testing set-up and determination of the slip capacity δ_u.

- *Influence of type of connectors*

Figure 12 presents the behaviour curves for the six types of connectors considered in experimental study. In terms of resistance, the specimen having channel connectors (PT-US-M) have shown the greater resisting force, while the specimen with anchoring bars (PT-A-M) had the resistance less than half from the first specimen. In terms of ductility, the PT-II-M specimen have the larger ductility, but it has to be noted the fact that it represents the ductility of reinforcing bars - concrete system (the failure in this case was by splitting and crushing of concrete nearby the perforated steel plate, and bending of reinforcement).

The specimen having LL120 (PT-LS/II-M) connectors behaved rather badly, the failure being in this case very early by puling-out of the angle connectors from the concrete. This is in fact demonstrated by the fast descending of the strength in monotonic curve characteristic of the specimen.

The headed stud connectors proved an expected behaviour, by a rather good ductility and a resistance according to their classic design. However, there are quite important differences between the two specimens (PT-16/I and PT-22) although their shear area are almost the same, both in terms of resistance and ductility.

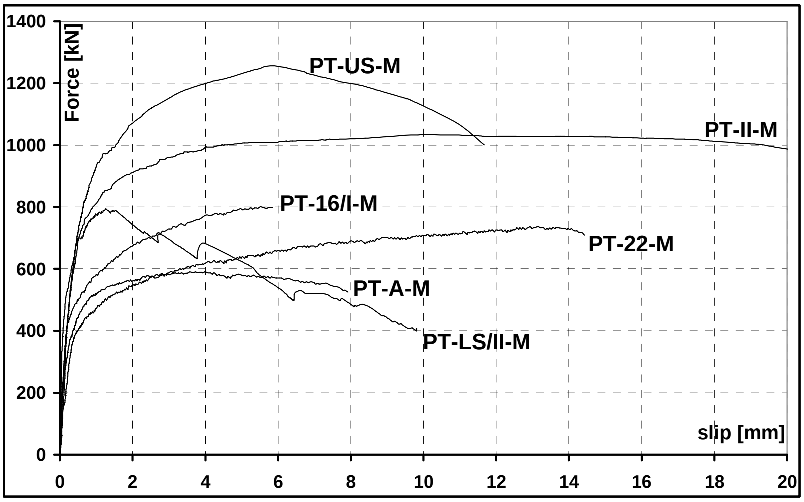


Figure 12. Force-slip curves on different types of connectors.

- *Influence of concrete class*

The differences between the characteristics of specimens with different concrete classes can be seen numerically in Table 2 and are shown graphically in Figure 13 (for specimens PT-16/I and PT-A for which have been designed similar specimens with a different concrete class). Because the observed failure mode of specimens was by shear of the steel connectors and not by concrete crushing, there is no clear evidence in the increase of the specimen's shear resistance, neither of the ductility with the increase of the concrete class. However, the initial stiffness of the curves specific to the specimens with C30/37 concrete class is significantly greater than the usual concrete specimens and seems to be the only influence of the concrete class in the case of monotonic loading.

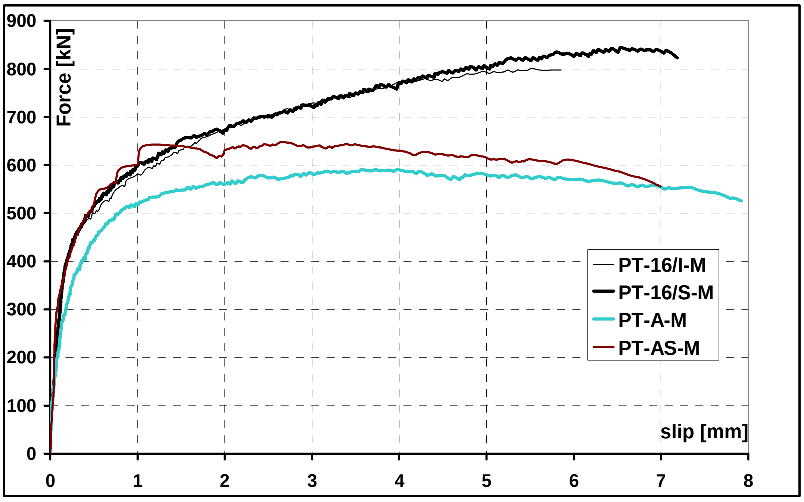


Figure 13. Force-slip curves showing concrete class influence.

- *Influence of steel flange class*

The 16mm headed stud (PT-16) and the L (PT-L) specimen series have been chose to have different thicknesses of steel flange (17.5mm for HEB 260 profile representing the class I, 10 mm for class II and 8mm for class III respectively). The results given in Table 2 and also graphically shown in

Figure 14, shows the fact that there is no major difference among the series specimen's behaviour. This is valid for both resistance and ductility. However, the initial stiffness is rather different within the specimens of each series, but this does not follow a certain rule with the change of the thickness of the steel flange. Concluding, the reduction in the steel profile flange thickness should be more important (to about 6 or even 4mm) in order get a logical increase of the ductility due to local bending of the flange in the vicinity of the connector.

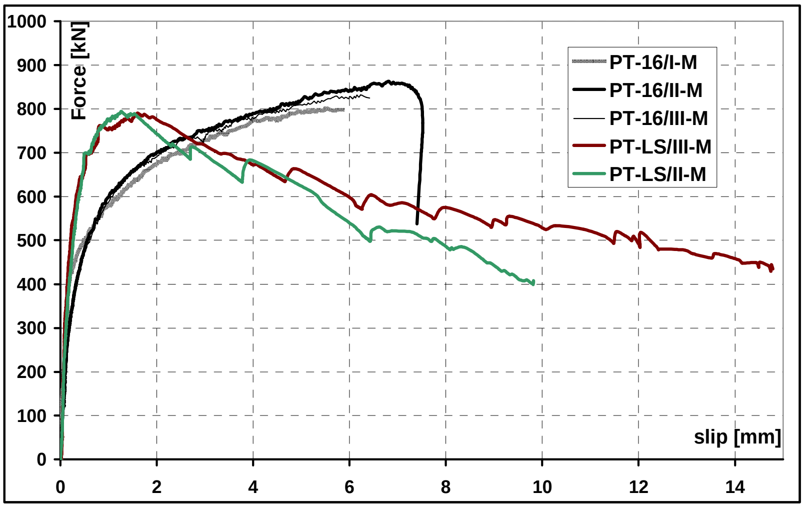


Figure 14. Force-slip curves showing concrete class influence.

- *Influence of loading type*

Figure 15 left displays the cyclic behaviour of the five cyclic tests, while the right side shows the comparison of the positive cyclic envelopes to the monotonic corresponding curves. Generally, all the monotonic tests have proved a reduction both in the resistance and ductility capacity. For example, for the specimen PT-16 with 16 mm headed studs, the P_{Rk} resistance reduction is about 40%, while the slip capacity is reduced by more than 50%, as compared to the monotonic tests. The same conclusion could be stated also for the PT-22 specimens, but with a higher degree of reduction in ductility.

The PT-II specimens with perfobond connectors represent, as proved by the tests the most dangerous situation. Although the monotonic test have shown a very good behaviour, with a very good resistance and the highest ductility among the monotonic tests, the cyclic test have proved a very bad cyclic behaviour, with about 40% reduction in resistance and an ultimate slip displacement δ_u of 0.6mm. In fact, the problem in this case appears due to the fact that the shear connection between the steel connector and the concrete is lost in the very first cycles due to the "knife" effect of the perfobond connector. In this way, practically there is no energy dissipation during the cycles and a rapid decrease of its resistance. In the case of LL (PT-L) and anchor hook (PT-A) specimens, despite the important reduction in ductility (δ_u < 2mm in both cases), the shear capacity reduction remains under 10% for the cyclic tests.

It has to be stressed out that in all the cyclic cases, the slip capacities do not pass the ductility criterion of 6 mm stipulated in EN 1994-1, 6.6.1.1. Although a limited study, the cyclic tests show very clear that the monotonic results do not necessarily conduct to the same cyclic results and in consequence the use of shear connectors in seismic prone areas should be reconsidered.

- *Analysis of failure modes*

Figure 16 shows the typical failure modes for different types of connectors. In case of shear studs (Figure 16 a), although the global behaviour could be characterized as ductile, the failure was brittle in nature, by the shearing of the headed studs situated on one side of the specimen. There is no evidence of concrete crushing, with the exception of the base of the shear stud.

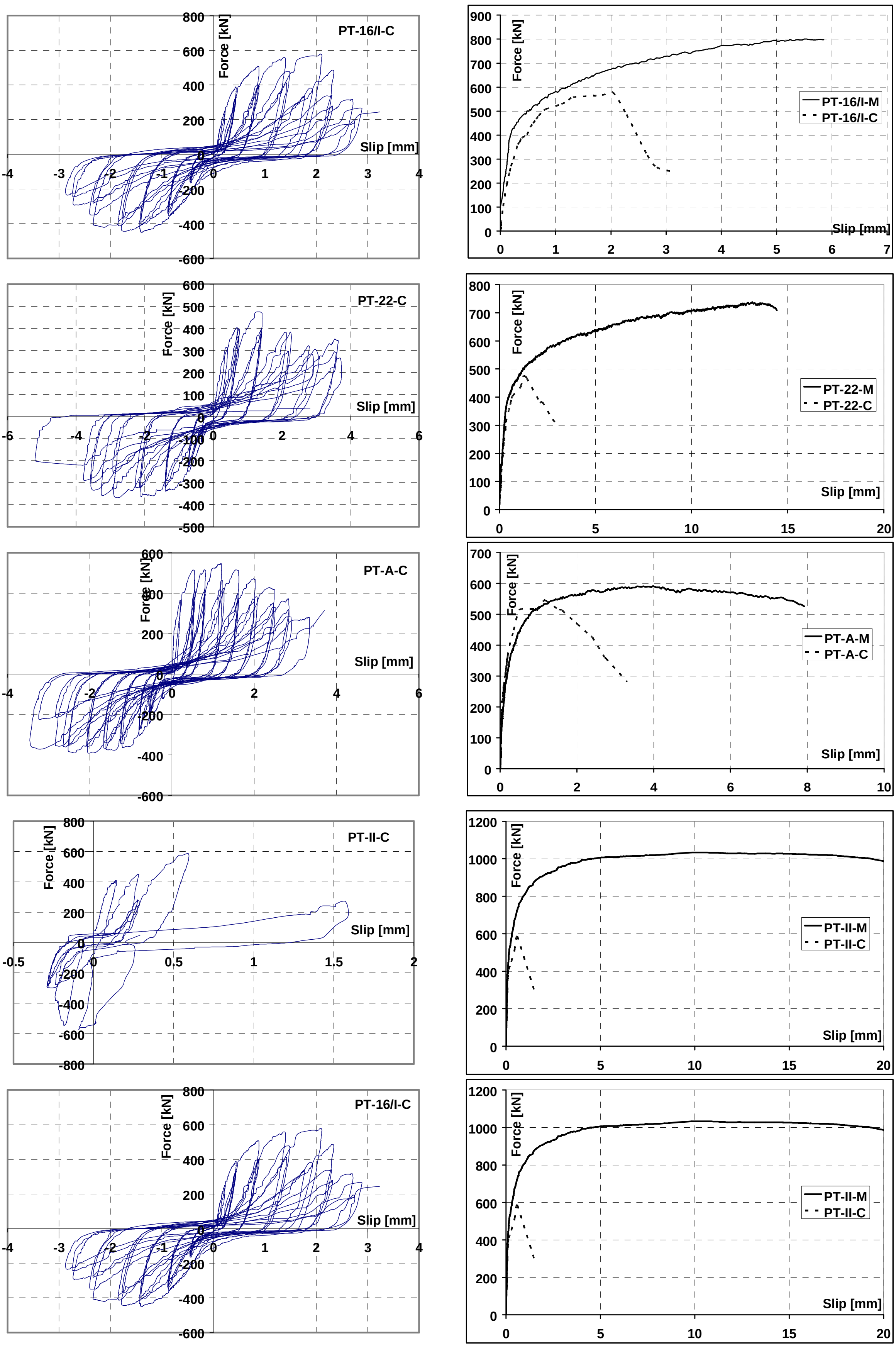
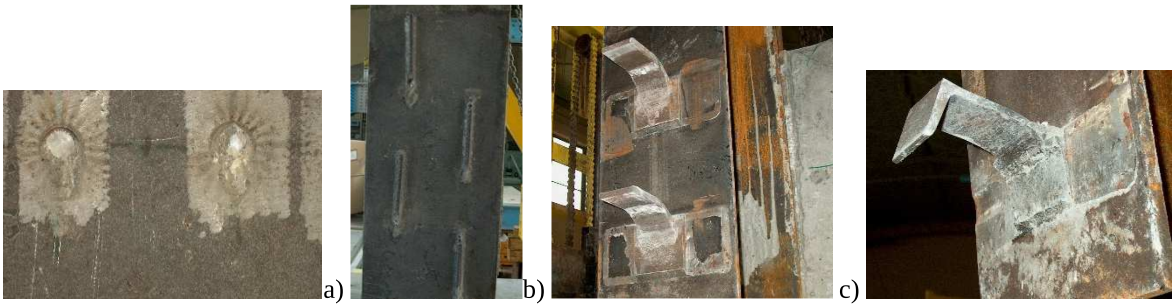


Figure 15. Force-slip curves for cyclic specimens and differences in monotonic curves and cyclic envelopes.



a) b) c) d)

Figure 16. Failure modes by shear connectors: headed studs (a), reinforcement hooks (b), LL connectors (c) and channel profiles (d).

Failure mode of specimens having hook anchors (Figure 16 b) was by shear of the reinforcement hooks. No crushing of concrete was observed. In nature, this type of failure was considered to be ductile, due to the fact that not all the shear fractures were in the same time. For LL and UPN specimens, the failure occurred by bending of the shear profiles (Figure 11), but without a real fracture of the steel material. In the case of LL specimens, the flange embedded in concrete was pulled out very rapidly from the concrete after reaching the maximum load. Probably the use of reinforcing bars through the embedded flange could retard the failure. Besides, the channel connector was very well embedded in concrete, and in this case, the failure mode was combined, by local crushing of the concrete, and bending of the channel web near the welding to the steel flange.

- *Behaviour of shear connectors for composite beams - conclusions*

The change in the connector typology introduced the dispersion of results in terms of Force-Slip curves for the monotonic tests. A very good monotonic performance in terms of resistance was obtained for the channel connectors and for the perfobond specimen. The worse performance in terms of ductility was obtained for LL specimens, which should be avoided without using additional reinforcing bars (flying reinforcement) passing through the LL flange. The headed stud specimens provided a good response both in terms of resistance and ductility, in accordance to their well-known design characteristics and expected failure. The anchor hook specimens provided a good monotonic ductility but a rather small resistance;

The influence of the concrete class is not evident in the studied cases, with the exception of the initial stiffness of the Force-Slip curve. The other parameters seem not to be affected by the change of concrete class, unless the failure mode of the specimens is changed (by crushing of concrete for example). Also, for the studied specimens, the change in the steel flange class do not affect significantly the monotonic response in terms of Force-Slip deformation curve.

The cyclic loading introduces for all the specimens a significant reduction in the slip capacity of the connectors, reduction that leads to non-ductile connectors in accordance to EN 1994-1. Also, there is an important reduction in the characteristic resistance P_{Rk}, ranging from 10 to 40 %. In consequence the reduction in the resistance of the design resistance of shear connectors in seismic loaded beams of 25% as required by may not be appropriate for certain types of typologies, such as perfobond connectors.

2.2.2. Connecting devices for RHS composite columns

The square columns realized as Concrete Filled Tubes (CFT) combine the good resistance and ductility of steel elements with the improved rigidity and resistance of concrete. In addition, the good properties are preserved on principal loading directions. The combination the two materials (steel and concrete) may lead also to a reduced steel section. However, a particular attention should be given to distribution of stresses between the steel tube and the concrete core. According to EN 1994-1, section 6.7, the load introduction in critical zones should be shared between steel and concrete elements. In case in which the natural bond between materials is not sufficient, supplementary connectors should be provided. Unlike open-type sections (e.g. I or H profiles), the RHS do not allow the use of regular connectors as is the case of welded head-studs. The research was focused on the use of shot fired nails, also termed as powder-actuated fasteners in investigated for an improved composite action of CFT.

The nailed shear connection was developed at the Technical University of Innsbruck, (Fink, 1997; Larcher, 1997), and the use of powder–actuated fasteners represents a relatively new method for assuring shear connection in areas where loads are induced to composite tubular columns (Beck, 2005). The powder-actuated fasteners are driven through the tube walls from the outside and then protrude inside the tube. The connection between the surrounding tubular section and the concrete inside is then provided by direct pressure of the concrete against the shanks of the nails. The main advantage of this solution is that it is quick and easy to apply, especially for columns that are continuous over several storeys. Initial experimental investigations were performed by Beck (1999) covering a number of 30 push-out tests with pipe specimens. Influences of pipe geometry, concrete strength and type of fastener on the load-deflection characteristics were investigated. The load deflection behaviour exhibited excellent ductility combined with high load-bearing capacity per fastener. Based on these findings the nailed shear connection was introduced into practice with the Millennium Tower in Vienna, a fifty storey high rise building completed in 1999, where it has proven to be a reliable and cost effective connection method as no welding work was required (Huber, 2001). Additional experimental investigations were performed, by Hanswille et al. (2001) with the aim to investigate the behaviour of the shear connection subjected to a serviceability limit state loading sequence, and the long term behaviour and influence of concrete creep on the nailed shear connections.

The evaluation of the load introduction within composite columns was realised by considering short column studs of concrete filled tubes of high strength steel (see Figure 17). The objective of research was to assess the efficiency of the shot fired nails in providing the shear connection between the steel tube and the concrete core under monotonic and cyclic loading. For this purpose load introduction tests were performed on 6 specimens varying parameters such as: loading procedure (monotonic and cyclic), connection type (steel-concrete bonding, and steel-concrete bonding combined with connectors), and steel grade (S460 and S700). As connectors, 24 Hilti X-DSH32 P10 shot fired nails were used per column stub. Table 1 summarizes the experimental program.

Figure 17. Failure modes by shear connectors: headed studs (a), reinforcement hooks (b), LL connectors (c) and channel profiles (d).

The column stub specimens were manufactured using cold formed steel hollow section tubes of 800 mm length. The concrete core was considered with a depth of 600 mm inside the steel tubes. For the load application, plates were welded in the shape of a double T beam on two opposite tube walls. For the positioning of the connectors, installation instructions were prepared at Hilti AG (2010), based on the particularities of the current experimental program (steel grade and thickness of the tubes). Consequently, it was recommended to use the X-DSH32 P10 nails, and to apply them before concreting in order to drive them consistently straight through the tube walls. Furthermore, due to the use of high-strength steel, pre-drilling of the steel tube was recommended.

Table 3. Experimental program - load introduction tests on column stubs

Nr.	Specimen	Tube	Bond	Load
1	S700-F-M	RHS 250x10 S700	Friction	Monotonic
2	S700-F-C	RHS 250x10 S700	Friction	Cyclic
3	S700-F-H-M	RHS 250x10 S700	Friction & Nails*	Monotonic
4	S700-F-H-C	RHS 250x10 S700	Friction & Nails*	Cyclic
5	S460-F-H-M	RHS 300x12.5 S460	Friction & Nails*	Monotonic
6	S460-F-H-C	RHS 300x12.5 S460	Friction & Nails*	Cyclic

The experimental test set-up considered both monotonic and cyclic loading conditions, top and bottom supports (see Figure 18) in contact with concrete core only. The load was applied on the steel tube. The instrumentation of the specimens (see Figure 18 - right) consisted in the measurement of the force applied by the testing machine and the relative displacement between steel tube and concrete core, using two displacement transducers at the top side and two at the bottom side. The parameters used to control the tests on column stub were the slip between the steel tube and the concrete core d and the force F (see Figure 4). The load was applied in force control within the elastic range and in displacement control for the plastic range.

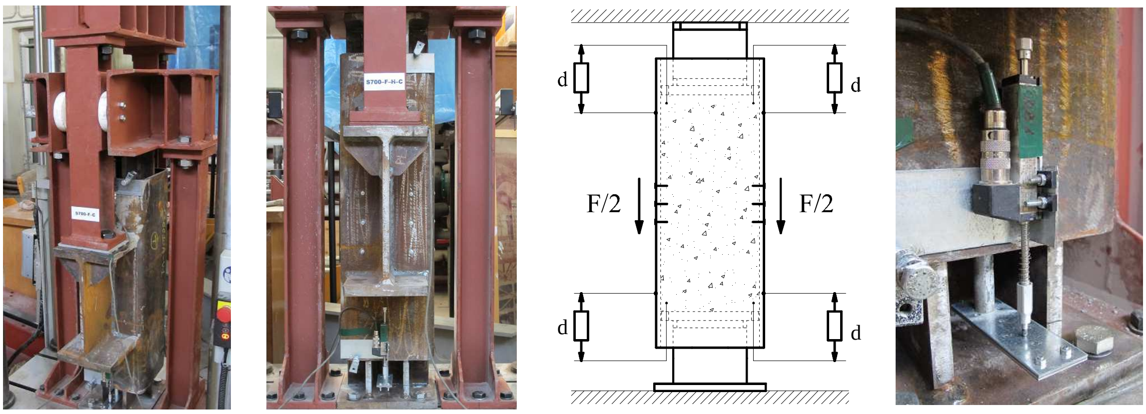


Figure 18. Experimental testing set-up and instrumentation of specimens.

The loading of specimens contained a pre-loading (three alternating cycles with a peak load of up to 25% of the expected yield load for the stabilisation of the system), followed by the monotonic or cyclic loading protocol:

- monotonic loading: by progressively increasing the displacement;
- cyclic loading: by ECCS (1986) procedure with an yield displacement d_y obtained as the deformation corresponding to intersection of the initial stiffness (K_{ini}) line and a tangent to the moment-rotation curve with a stiffness equal to $K_{ini}/10$. Four elastic cycles and three plastic cycles for each even multiplier of d_y were applied in displacement control.

The monotonic and cyclic response of the friction, developed between the steel tube and the concrete core, is shown in Figure 19 a) and b). Beyond a relative displacement of 2 mm, the monotonic force was about 200 kN. which was almost constant even for high levels of relative displacements. However, under cyclic loading the force developed through friction was lower (approximately 160 kN corresponding to the maximum amplitude of each cycle, and 20 kN corresponding to the initial position, i.e. 0 mm relative displacement).

The monotonic and cyclic response of the identical specimen combining friction and 24 shot fired nails is shown in Figure 19 c) and d) for the column stub with S700 steel tube and respectively in Figure 19 e) and f) for the column stub with S460 steel tube. The shear capacity heavily increased as compared to monotonic response. For these cases, under monotonic loading the transferred forces were approximately 1100 kN corresponding to a relative displacement of 5 mm. Under cyclic loading the maximum forces were slightly lower. In addition, the force-displacement curve decreased significantly corresponding to a relative displacements of 5 mm.

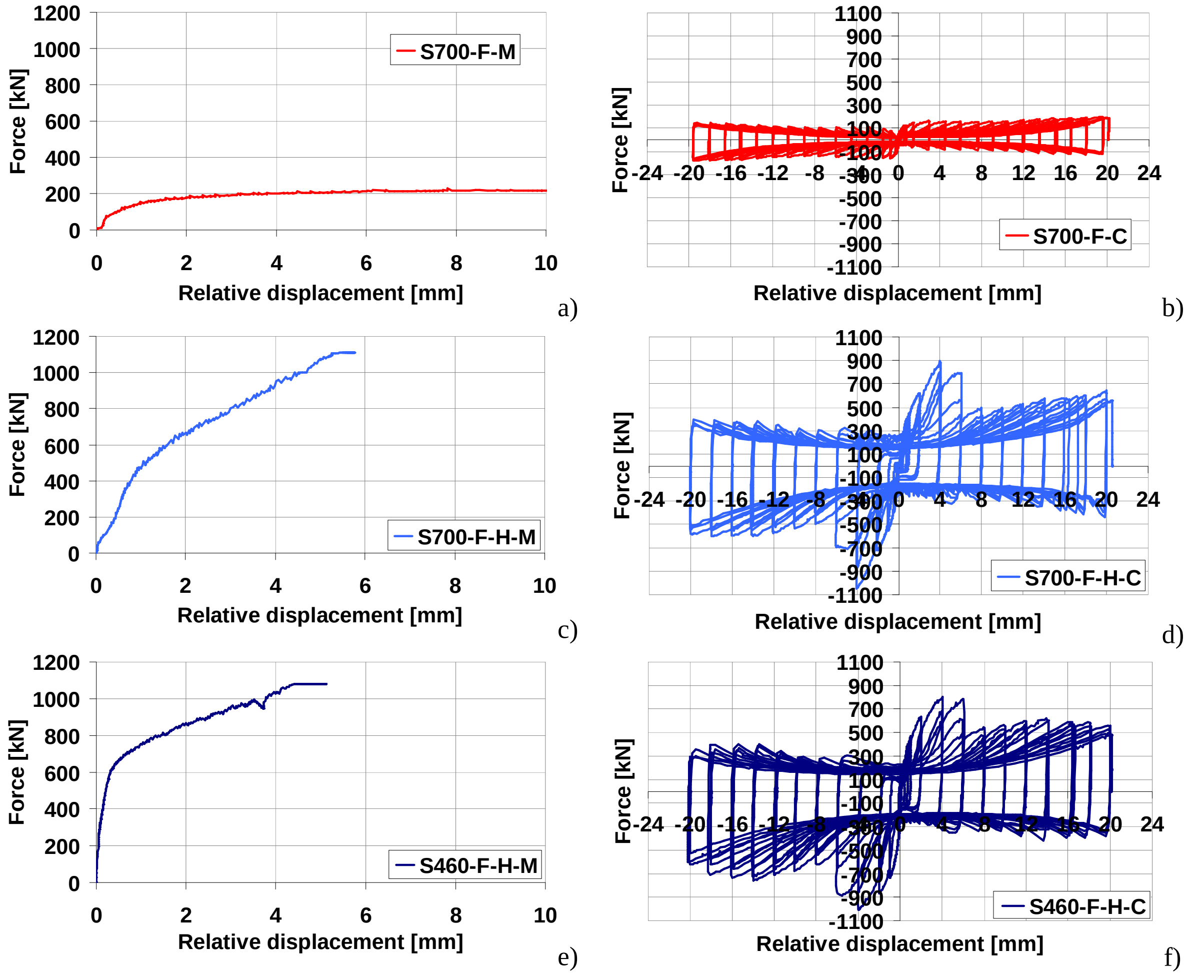


Figure 19. Monotonic and cyclic response of: a), b) steel-concrete adhesion (S700-F), c), d) friction and connectors (S700-F-H), e), f) friction and connectors (S460-F-H)

- *Influence of connection*

Figure 20 shows a graphical comparison in terms of monotonic response corresponding to the column stubs without nails (S700-F) and with nails (S700-F-H). The contribution of the 24 connectors is significant as it conducts to a maximum resistance at least five times higher in both cases. The connectors show a significant contribution also under cyclic loading conditions. However, in case of cyclic specimen, for relative displacements exceeding 5 mm, the capacity of the steel-concrete connection decreased due to failure of the connectors through fracture at the interface between steel tube and concrete core.

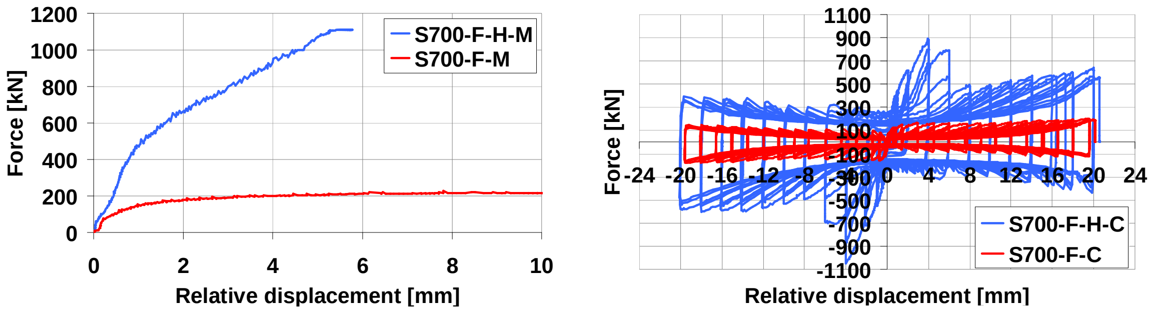


Figure 20. Contribution of connectors under monotonic and cyclic loading respectively.

Also in terms of comparison, Figure 21 presents the response under cyclic loading of the two specimens with both connectors and steel-concrete bonding. The difference between the two configurations is given by the steel tube geometry and material: RHS 250x10 S700 and RHS 300x12.5 S460 respectively. It is to be noted that the concrete depth in both cases was 600 mm. The lateral surface of the concrete core in the two cases was $A_{int,RHS,S460}$=660000 mm2 and respectively $A_{int,RHS,S700}$=552000 mm^2. In consequence, a slightly higher friction force was developed within the column stub with larger steel tube (S460).

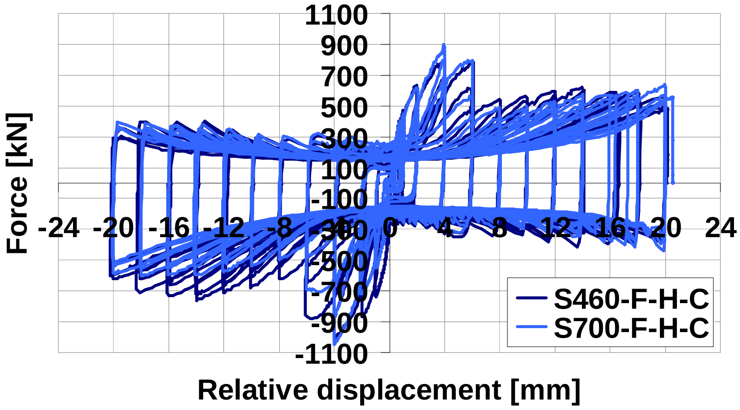


Figure 21. Comparison of cyclic responses of column stubs: steel-concrete bonding and connectors.

- *Analysis of failure modes*

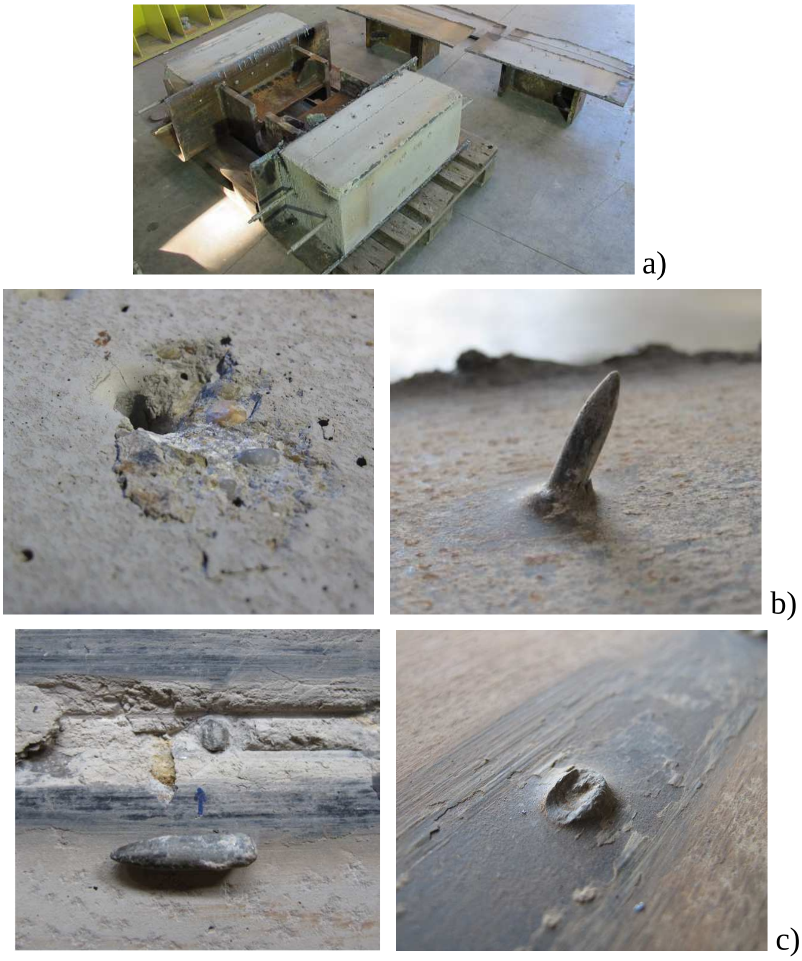


Figure 22. Analysis of failure modes of specimens: a) cutting of RHS walls; b) Detail of concrete and connector after monotonic loading; c) Detail of concrete and connector after cyclic loading.

In order to have a full explanation of the global behaviour of specimens under both cyclic and monotonic loading, a detailed analysis of the failure modes. This was by flame-cutting removal of two side walls of the column stubs S460-F-H-M and S460-F-H-C and detailed inspection (Figure 22 a).

A detail of concrete and connector after monotonic loading is shown in Figure 22 b). It can be observed that the concrete was crushed in a small amount at the contact with nails which were bent. A detail of concrete and connector after cyclic loading is shown in Figure 22 c). It can be observed that under alternating cycles the connectors were broken at the interface between concrete core and steel tube. The protruding part of the steel tube, which can be observed in Figure 20, rubbed on the concrete surface and lead inward marks on the concrete surface.

- *Behaviour of shear connectors for composite RHS columns - conclusions*

The main conclusions of the study revealed the following:

- the shear strength that developed through friction was obtained in amount of 0.4 N/mm2, which is equal to the value recommended by EN 1994-1 for rectangular hollow sections;
- the push-out tests have proven that connectors can take the major shear contribution to the load transfer from steel tube to the concrete core, in both monotonic and cyclic loading;
- the investigation of the behaviour and failure mode, lead to the observation that under monotonic loading the concrete was crushed in a small amount at the contact with the nails which bent. Under alternating cycles the nails eventually fractured at the interface between concrete and steel tube. In addition it was observed that from the cyclic loading the capacity of the connectors slightly decreased compared to the monotonic loading;
- as previously confirmed by Beck, the X-DSH 32 P10 shot fired nails proved a significant contribution to the steel-concrete connection considering the monotonic loading. In addition, the current study proved a significant contribution of the connectors also for the case of cyclic loading conditions and for the use of high strength steel rectangular hollow sections (S460 and S700);
- the effect of creep and shrinkage was not taken into account within the current study. Future research activities shall be devoted to the study of the steel-concrete connection through the use of shot fired nails under the long term effect of creep and shrinkage.

2.3. Dissipative zones in steel and steel and concrete composite frames

In case of steel structures the Moment Resisting Frames (MRF) and Eccentrically Braced Frames (EBF) are recognized as very effective structures in dissipating seismic input energy. Both typologies could take advantage of large values of seismic load reduction factors, in the range of 6 to 8. The dual MRF+EBF systems (see Figure 22) combine the architectural freedom allowed by MRF with the reduction in the lateral displacements due to presence of braces, while keeping the seismic dissipative performances. The use composite elements such as steel and concrete composite beams or columns can improve the structural resistance and rigidity (Elghazouli et al., 2008; da Silva et al., 2001).

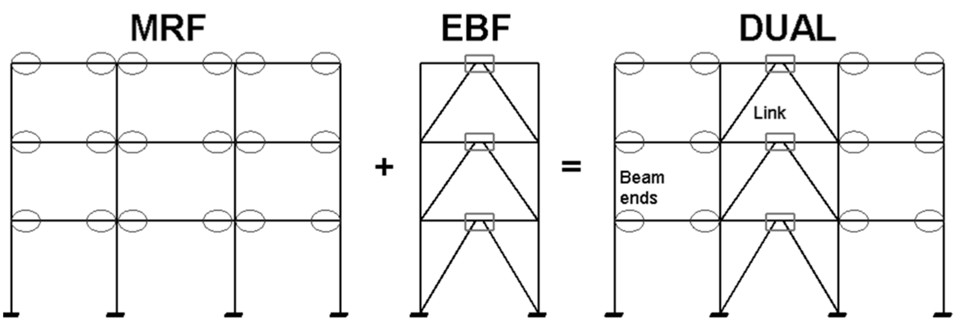


Figure 23. Possible dissipative zones in case of Moment Resisting Frames, Eccentrically Braced Frames and respectively dual MRF+EBF configurations.

For composite steel-concrete systems, the modern seismic norms advise to disconnect the steel and concrete elements in the zones where the plastic hinge is expected to develop and to consider a symmetric plastic behaviour for the beam, as usual for the steel section alone. In reality, due to the presence of the reinforced concrete slab (although no connectors are installed) and due to the friction between the steel profile and slab, the plastic hinge on beams will not have a symmetric behaviour under hogging and sagging moments. This could have implications in the specific seismic design criteria, including the values that should be used for the behaviour and overstrength factors.

Composite action between steel beams and concrete slab generally improves the rigidity and resistance of the member, needing careful design and detailing according to specific sections of EN 1994-1 and EN 1998-1 (2003). A special attention is paid to the detailing of the shear connection between the two materials: steel and concrete (which have a very different elastic and post-elastic behaviour). However, when lateral loads such as earthquakes are acting on a structure having composite beams, the bending moment may suffer sign reversals leading to cracks in the concrete slab due to tension stresses. Special detailing and requirements are given in the actual seismic design codes for composite beams subjected to seismic loads, such as: (i) special requirements for connecting devices (EN 1998-1, §7.6.2), (ii) special requirements for the detailing and positioning of the reinforcement of the beams adjacent to beam-to-column joints (given in EN1998-1, §7.6.2 and Annex C.3.1.2) and (iii) special requirements for design as dissipative or non-dissipative elements.

When using composite beams with Reduced Beam Section (RBS) or composite link elements, the in-use European norms are very poor in detailing and requirements, practically limiting the composite interaction up to the physical boundaries of the dissipative element according to the paragraphs 7.6.2, 7.7.1, 7.7.5 and 7.9.3 of EN 1998-1. In consequence, the plasticization and dissipation of energy is thought of as for a steel element, ignoring the fact that the beam is composite up to the boundaries of the dissipative element. For usual design, the influence of the adjacent composite beam on the dissipative capacity of the dissipative element is very hard to consider.

Generally, the behaviour of composite beams under monotonic loads is very well covered by the world-wide researches (Johnson et al., 1972; Plumier, 2001, 2001, 1998). However, the oligocyclic behaviour of composite beams of EBF and composite beam-to-column joints in MRF subjected to cyclic loads may need special attention in design. As proven by Liang et al. (2005), in case of composite beams the contribution of concrete slab in shear is not negligible and influences the behaviour of the element. Also, a bad detailing or execution in these cases may lead to modifications of the dissipative element components and consequently in a possible global alteration of the structural seismic capacity (Clifton et al., 2011).

On the other hand, the columns realised from steel hollow sections can be filled with concrete with the aim to combine the properties of the two materials, the final element being characterised by higher stiffness, capacity and ductility, as well as enhanced fire resistance in comparison with bare steel configurations. A particular advantage of using a composite column is the reduction in column cross-sectional area, and by using steel tubes as permanent formwork, construction speed is increased. The framing system with concrete filled columns and welded joints has been investigated and applied on a large scale in Asia, Australia and America. In contrast, this solution was used at a lower scale in Europe.

As shown by Morino & Tsuda (2003), the typical connections between a concrete filled tube (CFT) and I-beams often used in Japan are based on the use of stiffeners which can be either a through diaphragm, internal diaphragm or external diaphragm. Considering recent research activities, Chen & Lin (2004) investigated the cyclic behaviour of flange plate connections between steel beam and rectangular CFT column, in which the flange plates were penetrated through the CFT column. The study of Shin et al. [6] was focused on the experimental and analytical behaviour of CFT column to H-beam welded moment connections with external T-stiffeners. Fukumoto & Morita (2005) conducted tests on the panel zone within beam-to-CFT column moment connections made from high-strength material to investigate their elasto-plastic behaviour. Park et al. (2005) investigated the force transfer mechanism and the cyclic performance of wide flange beams to square CFT column joints reinforced with stiffening plates around the column. Cheng et al. (2007) investigated the seismic performance of steel beams-to-CFT column connections with floor slabs, and Yuan et al. (2014) investigated the behaviour and design modification of RBS moment connections with composite beams. Wang et al. (2010) investigated the seismic behaviour of H-beam to circular tubular column connections stiffened by an outer ring diaphragm, employing a three-dimensional connection subassembly testing system.

The ductile behaviour of steel link elements has been proved by several researchers, investigating different solutions for best performance. Okazaki el al. (2005) have performed a large experimental study investigating the disposition and the geometrical form of the web stiffeners and critically analysed the results in regard to AISC provisions. Yurisman et al. (1978) show through experimental and numerical simulations the good capacities of link elements working in shear by using diagonal web stiffeners. Chao et al. (2006) investigated the fissure initiation and propagation in stiffened links through experimental and numerical simulations, proposing stiffener configurations that eliminate the presence of welds near k-areas. Shayanfar et al. (2011) have shown the influence of concrete encasement of link web through experimental investigation.

2.3.1. Behaviour of composite beam-to-steel column joints

The beam ends of MRF represent an important source of ductility. As shown by Chen and Chao (2001), the presence of concrete slab influences the beam-to-column joint plastic deformation and can lead to premature failure. In consequence, several possibilities can be applied in order to enhance the beam-to-column joint ductility, such as use of RBS (Pachoumis et al., 2010) or use of dissipative devices such as fuses (Castiglioni et al., 2010). The experimental study on composite beam-to-steel column joints is focused on the use of Reduced Beam Section (RBS) for concentrated plasticity and different interaction with the concrete slab.

Figure 24 shows the testing set-up: the column, disposed horizontally was pinned at both ends (at half-story distances) while the beam was vertical and loaded in bending by the actuator at beam top. A lateral-restraining frame was considered for keeping the in-plane post-elastic behaviour of the beam. The following cross-section and material characteristics were considered for beam-to-column specimens:

- column: HEB260 (S355) and HEB300 (S460);
- beam: HEA260 (S235);
- typology of the connection: direct welding of the beam on the column flange through full penetration weld (see Figure 25 a for welding detail), with reduced beam section (RBS) near the connection. The design of the RBS was performed according to EN1998-3:2005, Annex B, section 5.3.4 resulting the geometric properties detailed in Figure 25 b).

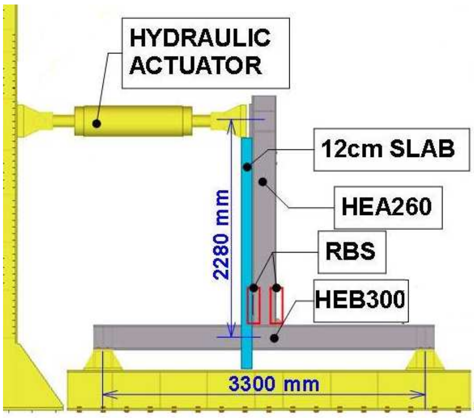


Figure 24. Testing set-up for composite beam-to-steel column joints.

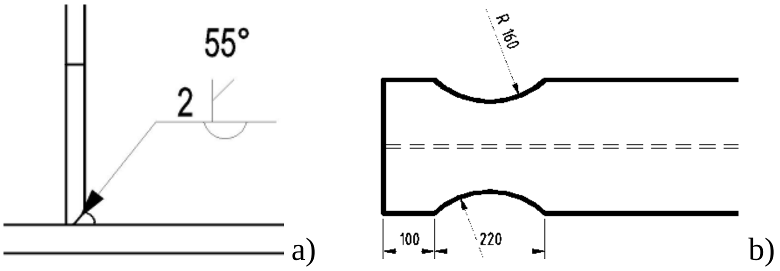


Figure 25. Detail of beam-to-column weld and geometry of the reduced beam section.

The connection was loaded in bending through the actuator located at top of the beam. The load cell attached to actuator was measuring the force during testing. The general instrumentation of the joint specimens is shown in Figure 26. Additionally, displacement transducers were placed for measuring the following data:

- beam top displacement – by means of DTF and DTB transducers;
- local rotations, deformations and distortions in the dissipative elements: RBS, web panel of the column, welds (through DBLL, DBLR, DDT1, DDT2, DWL, DWR transducers);
- local slip in pins through DHBL, DVBL, DHBR, DVBR transducers;

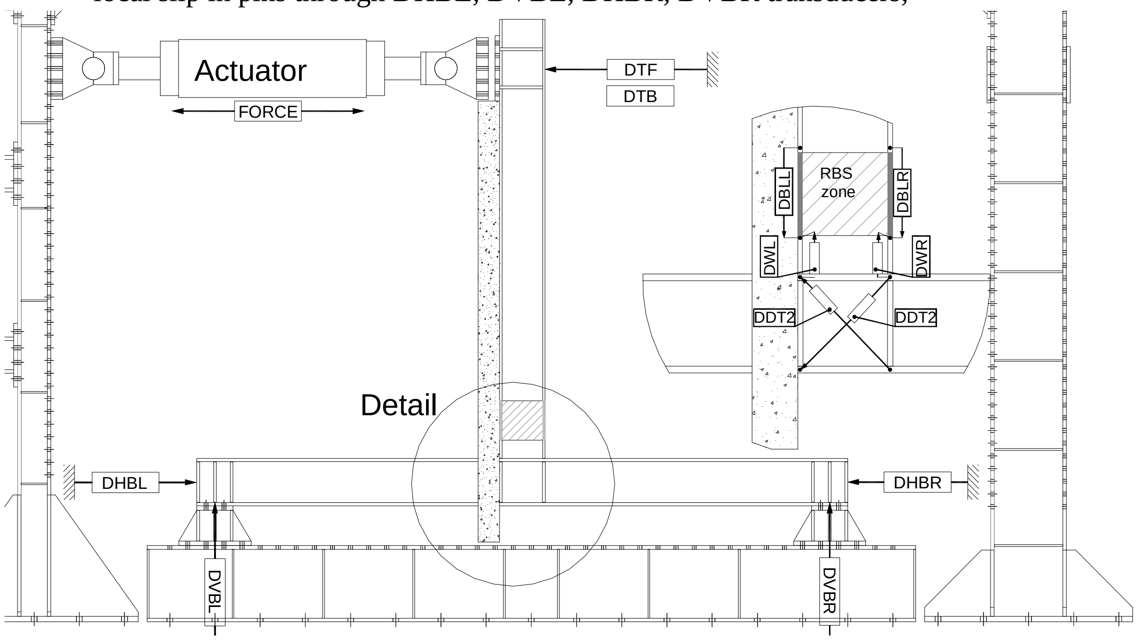


Figure 26. Instrumentation of RBS specimens

The displacement transducers are used for computing different rotations:

- RBS distortion;
- rotation of welded connections;
- column web panel distortion in shear through.

Table 4 presents the identification and characteristic of RBS joint specimens. The main parameters focus on influence of concrete slab and type of loading (monotonic or cyclic).

Table 4. Description of RBS specimens

Specimen	Beam type	RBS	Loading type	Connectors over RBS zone	specimen name
1	steel	Yes	monotonic	No	DB-M
2	steel	Yes	cyclic	No	DB-C
3	composite	Yes	cyclic	No	DB-Comp1
4	composite	Yes	cyclic	Yes	DB-Comp2
5	steel	Yes	cyclic	No	DB-C RLD
6	composite	Yes	cyclic	Yes	DB-Comp RLD

The concrete slab for composite specimens was designed for a full connection (headed stud connectors Φ 19*100mm disposed longitudinally at 95 mm) and with disposition of longitudinal and transversal re-bars as shown in Figure 27: longitudinal bars Φ 12mm spaced at 15 cm tied by transversal stirrups Φ 12mm spaced at 15 cm. The reinforcing of beam-to-column composite joint was verified to SR EN1998-1: Annex C prescriptions.

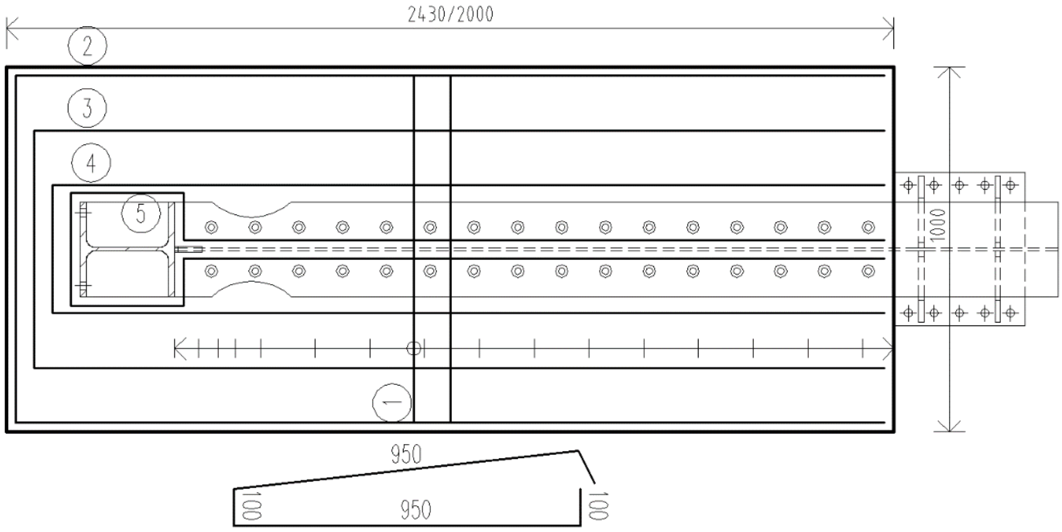
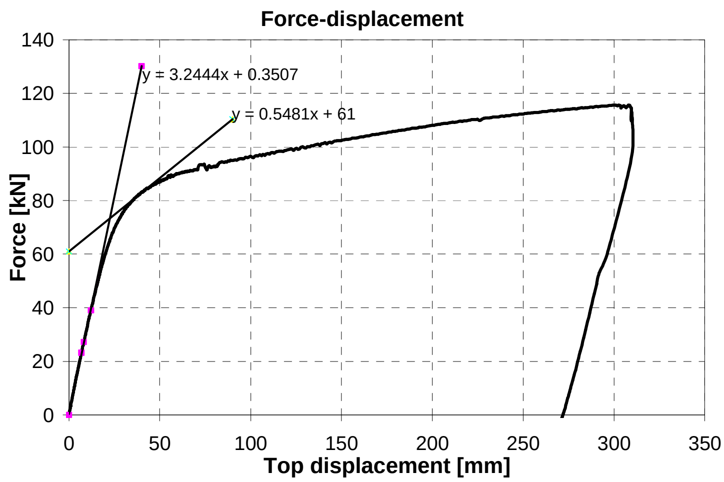


Figure 27. Detailing of reinforcing of concrete slab over beam-to-column joint

- *Results for steel specimens*

The DB-M test was carried out until important plastic deformations were recorded. The termination of the test was practically due to reaching of the actuator displacement limit. Based on the force-top displacement curve the yield displacement δ_y, yield force F_y and the initial rigidity were determined and used for piloting the cyclic specimens. Although in this case the dissipative element is the beam, due to a smaller resistance of the column's material, the plasticization is divided into two components:

- - RBS zone;
- - Column Web Panel (CWP) in shear.



$S_{j,ini}$=3.33 kN/mm
δ_y=20 mm
F_y=60 kN

Figure 28. Load-displacement curve of DB-M specimen

Figure 29 shows a graphical comparison between the total joint rotation (recorded through DTF, DTB transducers) and the local RBS rotation. It results very clear that a large amount of final rotation is assigned to other components than RBS, principally to CWP in shear. However, the total RBS rotation could be judged as high, exceeding 80mrad for RBS solely.

For Figure 29 a force representation was chosen instead of moment (usually met in such representations) for a unitary representation: the level arms which will multiply the force are significantly different in RBS centre (1.96m) and CWP centre (2.28m) respectively. The image in the Figure 29 right shows the behaviour of the plasticized zones in the specimen during the test and the failure pattern, by local buckling of the beam's flange in the RBS.

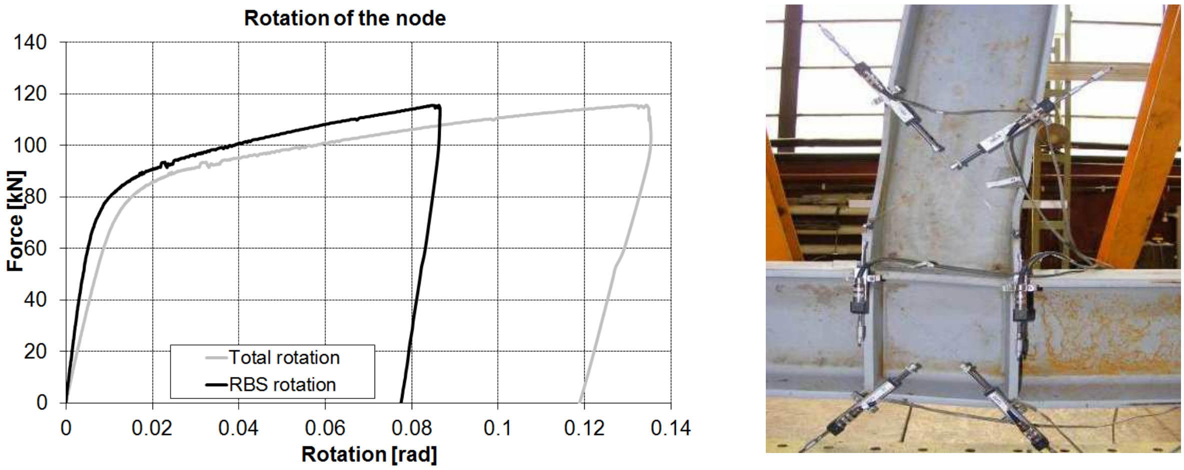


Figure 29. Force-rotation curve of the steel node with reduced beam section and failure mode in case of DB-M specimen

This leads to a very important conclusion in case of RBS DB-M specimen: due to the fact that the steel quality of the column was lower than the one ordered (S275 – received instead of S355 – ordered, according to Table 5), the column web panel became the component with the smallest resistance. In consequence it highly influenced the distribution of plastic deformations within the joint among RBS and CWP.

The DB-C specimen represents a cyclically loaded steel specimen, having identical configuration with the base specimen DB-M. The cyclic loading pattern according to the ECCS protocol was set after determining the yield displacement. The value of δ_y of 20 mm, was considered, as determined from DB-M specimen test results and their interpretation. The global joint rotation shown in Figure 30 could be considered as satisfactory, reaching 80 mrad, with the major observation that this is due primarily to CWP plastification as in case of DB-M specimen. The specimen failure mode was by fracture of the beam-to column flange welds (recorded at an increment of 8x δ_y - third cycle), coincident practically with the maximum actuator stroke limit.

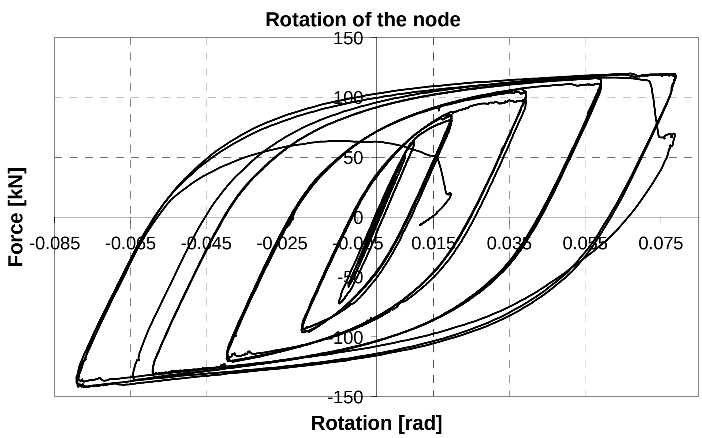


Figure 30. Force – total rotation curve of the DB-C specimen

- *Results for composite specimen*

The composite DB-Comp1 and DB-Comp2 specimens were realised by using similar joint typology as in the case of steel (DB-M and DB-C) specimens. However for the composite specimens the steel beam is connected to a 12 cm slab as detailed above. The main difference between DB-Comp1 and DB-Comp 2 specimens is represented by the steel-concrete interaction above the RBS zone. In case of DB-Comp1 specimen no connectors were disposed over RBS zone while for DB-Comp2 specimen the shear connection is continuous over the top flange of the beam.

The loading of the specimens was satisfactory up to the third cycle for an increment of 8xδy (see Figure 31), when, as in the case of cyclic steel specimen, the welds between the flange of the beam and the flange of the column have cracked. The force-rotation hysteretic curves for the two composite joints are shown in Figure 31 and Figure 33respectively.

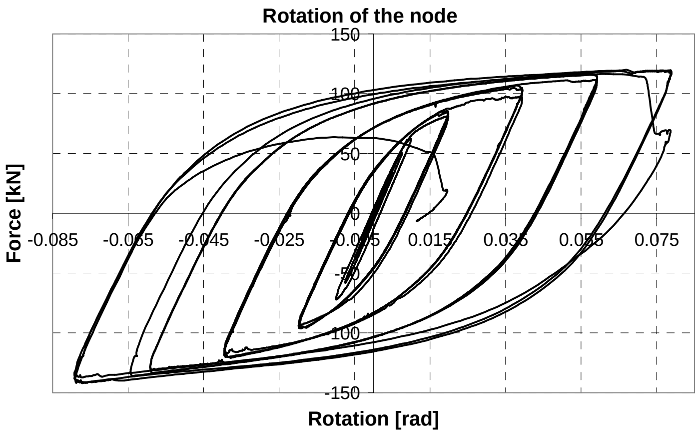


Figure 31. Total force-rotation curve for the DB-Comp1 specimen

The total rotation of the joint (Figure 31) represents practically the sum of CWP and RBS zone rotations. The asymmetry of the global curve is due to RBS zone (Figure 32 left) which developed plastic rotation only for positive bending (concrete in tension) while the plasticization in negative bending was prevented by the higher resistance of composite section. Contrary to this, the CWP rotation is symmetrical (Figure 32-right) without any influence from the concrete slab. The total plastic positive rotation was practically equally shared between the two components (40 mrad) on positive range.

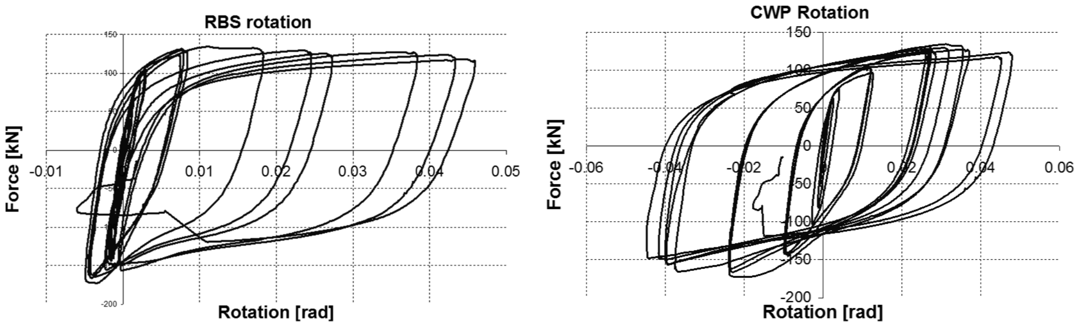


Figure 32. Deduction of RBS and CWP rotations for DB-Comp1

The DB-Comp2 specimen behaved similarly and confirmed the above remarks. However, the cycles are more stable in this case, although the failure was recorded at the same displacement amplitude and by similar weld failure (see Figure 33). In both cases the concrete slab was crushed at the interface with the column flange in compression, while in tension fissures formed starting from column flange corners (see Figure 34). However, the concrete slab was much more affected in case of DB-Comp1 specimen, as shown in Figure 34 a) fact which demonstrates that the steel-to-concrete connection remains effective although shear connectors are not provided near the column.

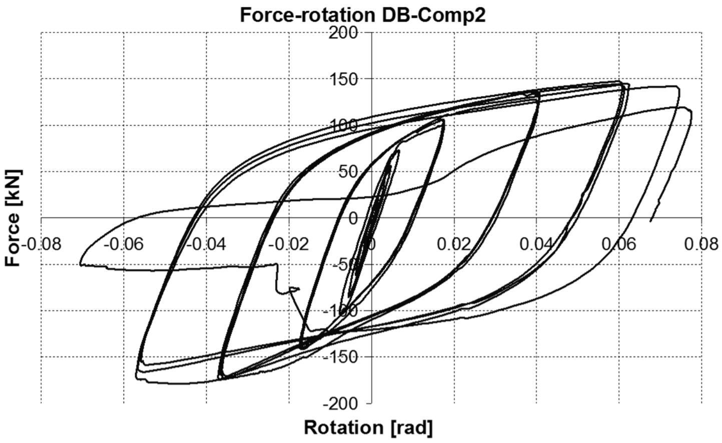


Figure 33. Force-rotation curves for the DB-Comp2 specimen

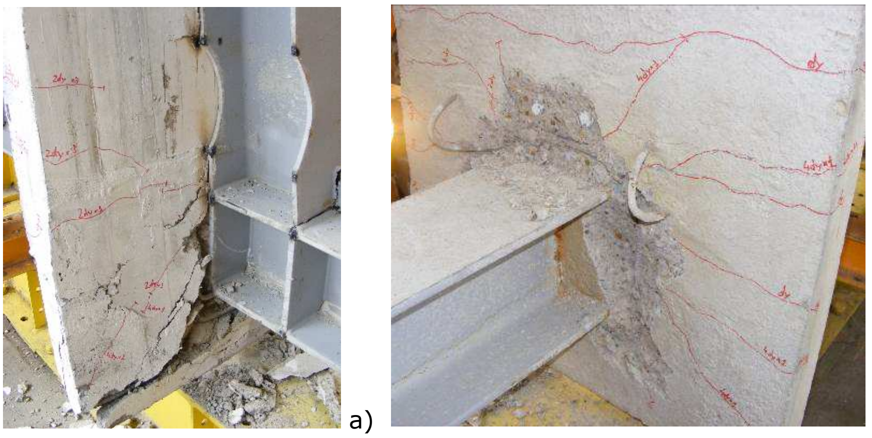


Figure 34. Failure of the specimens DB-Comp1 – a) and DB-Comp 2 – b)

Figure 35 presents as comparison the moment-rotation envelopes for DB cyclic loaded specimens. It becomes obvious that as in case of EBF tests, the global behaviour of composite specimens is very similar, regardless the presence of the connectors over the dissipative zone (RBS).

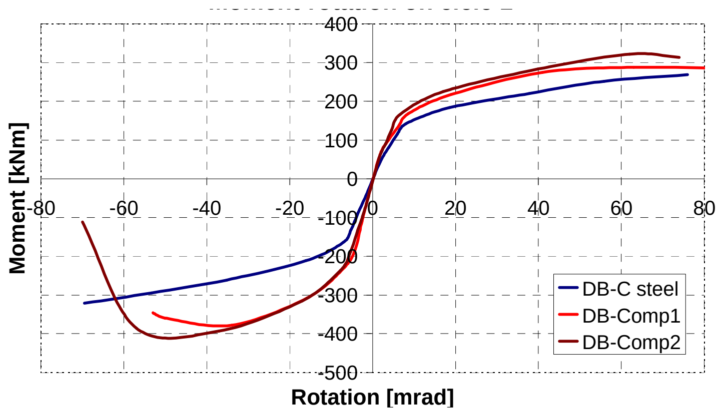


Figure 35. Comparisons of the moment-rotation envelope curves for the tested nodes

In comparison with the steel specimen, the composite ones show higher stiffness, especially on positive behaviour (42.95 kNm/mrad for DB-Comp1 and 36.72 kNm/mrad for DB-Comp2 in regard to 25.38 kNm/mrad for DB-M). The maximum resistance is also significantly higher for composite specimens:

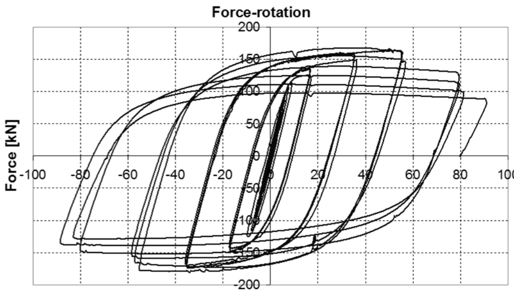
- 321.98 kNm for composite and 268.69 kNm for steel specimens in positive bending;
- 392.90 kNm for composite and 321.67 kNm for steel specimens in negative bending.

For all specimens the ductility was limited by premature failure of the welds.

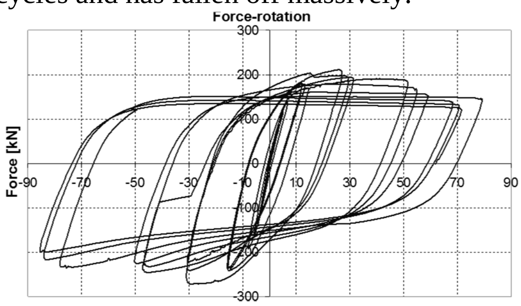
- *Results for specimen with adjusted properties of materials*

Due to the fact that the final purpose of the initial RBS specimens was not fully achieved (dissipation only in the RBS zone), the joint test series was completed by two new specimens, namely DB-C_RLD (steel specimen) and DB-Comp_RLD (composite specimen), both tested cyclically. In this configuration, the beam section was kept (HEA260), while the column was replaced with a HE300B profile in S460 steel grade. As consequence, for this new series of tests the expected failure type was reached in both cases, through ductile plastification of the RBS and gradual reduction of joint resistance. No important plastification of the CWP in shear was recorded. In case of the composite specimen, the upper flange was hindered to buckle due to the presence of the concrete slab. However, its resistance gradually degraded during the plastic cycles by concrete degradation due to fissures recorded parallel to column flange when slab was in tension and respectively by crushing along the fissures already formed in tension when slab in compression.

The charts plotted in Figure 36 show the hysteretic curves in terms of applied force and total joint rotation. The total rotation – greater than 70 mrad in both cases – is due almost exclusively to RBS plasticization. However, high degradation was recorded in the concrete slab around the column zone, in which the concrete was crushed in reversal cycles and has fallen off massively.



a)



b)

Figure 36. Force-rotation curve of steel specimen DB-C_RLD – a) and respectively composite DB-Comp_RLD – b) specimens under cyclic loading.

Figure 37. Failure of DB-C RLD specimen through plastic hinges in RBS

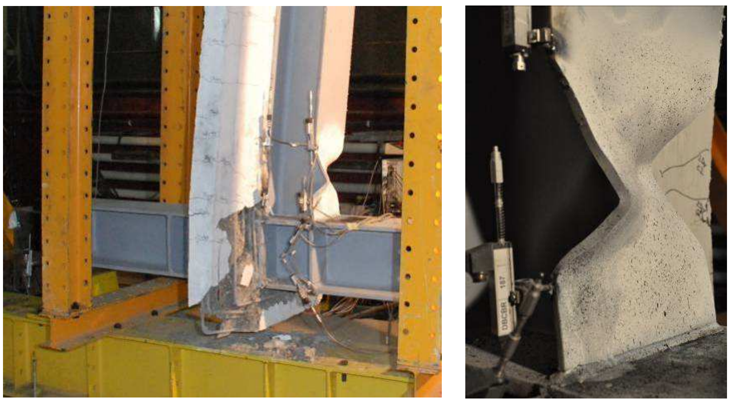


Figure 38. Crushing of concrete and buckling of the compressed flange and for the DB-Comp_RLD specimen.

Comparing the envelope force-rotation curves (see Figure 39) it could be noticed similar rotation capacities (up to 70 mrad) and higher resistances and rigidities for the composite specimen especially in negative bending (slab under compression) as compared to steel bare connection.

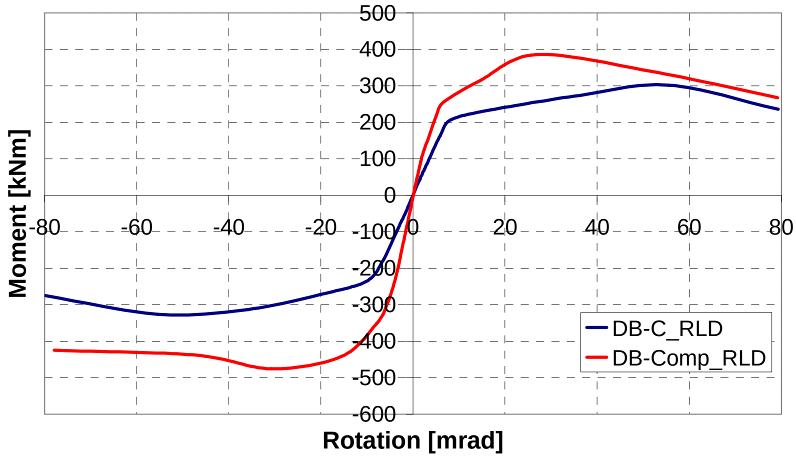


Figure 39. Comparison of the force-displacement envelope curves for DB-C_RLD and DB-Comp_RLD specimens.

- *Conclusions on composite beam-to-steel column joints*

The simple disconnection of the steel beam from the concrete slab over the dissipative zone is not sufficient to ensure a pure steel-like behaviour of the element. The resulted behaviour is practically very close to that of a full-composite specimen;

The composite aspect improves the global resistance and stiffness characteristics of the joint, while maintaining the ductile nature of the solution. However, the composite behaviour should be considered in the design of elements without connectors over the dissipative zone;

A very careful detailing and execution should be applied to beam-to-column joints and links in order to reach the desired levels of ductility and resistance. On the contrary, the steel grade mismatch could change the plasticization order, while the defective execution of welds may lead to the brittle failure of the element;

The reduced beam section solution remains effective in the composite configuration, but the presence of the concrete slab changes the failure mode: the top flange is restrained in buckling, while the concrete slab is degraded by cyclic tension-compression alternating forces.

2.3.2. Behaviour of steel beam-to-composite RHS column joints

The composite RHS column solutions are regarded as very effective in overtaking high loadings in case of MRF or Braced systems. The advantage of the system is also represented by the fact that it develops important resistances on both principal axes and rectangular connections can be easily made. Furthermore, the introduction of high strength steels can lead to diminishes of the geometrical dimensions of columns. However, smaller sections lead to higher inter-storey drifts due to lateral loads. This lack in lateral stiffness can be improved by filling the steel tube with plain or reinforced concrete by obtaining the s-called Column-Filled Tubes (CFT). The critical zones in this case appear in the joining area (beam-to-column joints). With this purpose, the behaviour of steel beam-to-composite RHS column joints was investigated by means of experimental specimens, while the study was further extended by FEM simulations.

The design of the beam-to-column joint specimens, was performed considering the joint dissipation in beams by means of Reduced Beam Sections (RBS specimens) and respectively with

Cover Plates (CP specimens). As basis for definition of the experimental program on beam-column joints, cross-sections from the D-CBF frame designed within HSS-SERF research project were used (see Figure 40), considering two combinations of HSS/MCS:

- CF-RHS 300x12,5 S460 column and IPE 400 S355 beam;
- CF-RHS 250x10 S700 column and IPE 400 S355 beam

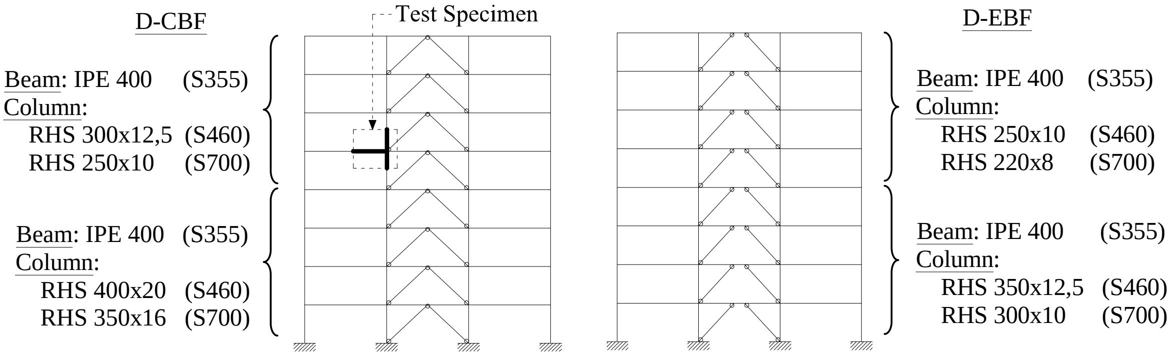


Figure 40. Designed frames with CFT columns.

The design was performed considering the development of the plastic hinge in the beams (see Figure 41a). Further with the bending moment and shear force from the plastic hinge, the welded connections and the components of the joint, i.e. cover plates, external diaphragm (see Figure 41 b) and column web panel (see Figure 41 c) were designed and/or checked so as to comprise an equal or higher capacity in comparison to the fully yielded and strain hardened plastic hinge. Due to the flexibility of the tube walls under transverse forces, the connection solution of the beams and columns within the current research was based on the use of external diaphragms.

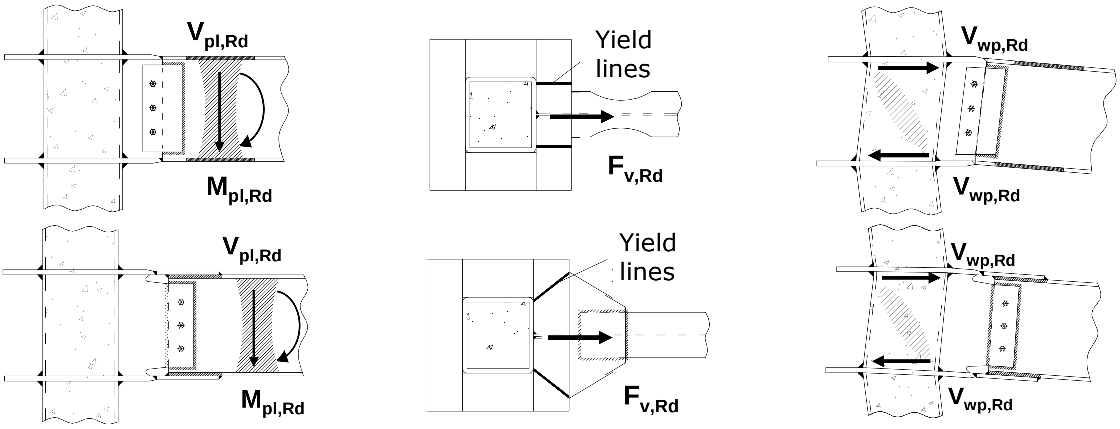


Figure 41. Designed frames with CFT columns.

The experimental program on beam-to-column joints with CF-RHS columns is summarized in Table 5. The variations in the configuration of the joints are given by the two joint typologies (RBS and CP), two steel grades for the rectangular hollow section tubes (S460 and S700) and two intended failure modes (beam and connection zone). In addition, two loading conditions were considered for each beam-to-column joint configuration, i.e. monotonic and cyclic loading procedure.

Considering the two joint typologies (RBS and CP – see Figure 42 a and b) and two steel grades for the tubes (S460 and S700), a number of four beam-to-column joint configurations were designed (see Figure 42 c). Additionally, in order to assess the over-strength of the connection zone and to observe the base components of the joint configurations, tests were considered on the corresponding joints for which the beam was strengthened (Figure 42 d) with the aim to avoid the formation of the plastic hinge in the beam and to force the plastic deformations in the connection zone.

Table 5. Experimental program on beam-to-column joints with CF-RHS columns

Nr.	Specimen name	Column	Beam	Joint type	Loading	Intended failure mode
1	S460-RBS-M	RHS 300x12.5 S460	IPE400 S355	RBS	Monotonic	Beam
2	S460-RBS-C				Cyclic	
3	S700-RBS-M	RHS 250x10 S700	IPE400 S355	RBS	Monotonic	Beam
4	S700-RBS-C				Cyclic	
5	S460-CP-M	RHS 300x12.5 S460	IPE400 S355	CP	Monotonic	Beam
6	S460-CP-C				Cyclic	
7	S700-CP-M	RHS 250x10 S700	IPE400 S355	CP	Monotonic	Beam
8	S700-CP-C				Cyclic	
9	S460-RBS-R-M	RHS 300x12.5 S460	IPE400 S355	RBS	Monotonic	Connection
10	S460-RBS-R-C				Cyclic	
11	S700-RBS-R-M	RHS 250x10 S700	IPE400 S355	RBS	Monotonic	Connection
12	S700-RBS-R-C				Cyclic	
13	S460-CP-R-M	RHS 300x12.5 S460	IPE400 S355	CP	Monotonic	Connection
14	S460-CP-R-C				Cyclic	
15	S700-CP-R-M	RHS 250x10 S700	IPE400 S355	CP	Monotonic	Connection
16	S700-CP-R-C				Cyclic	

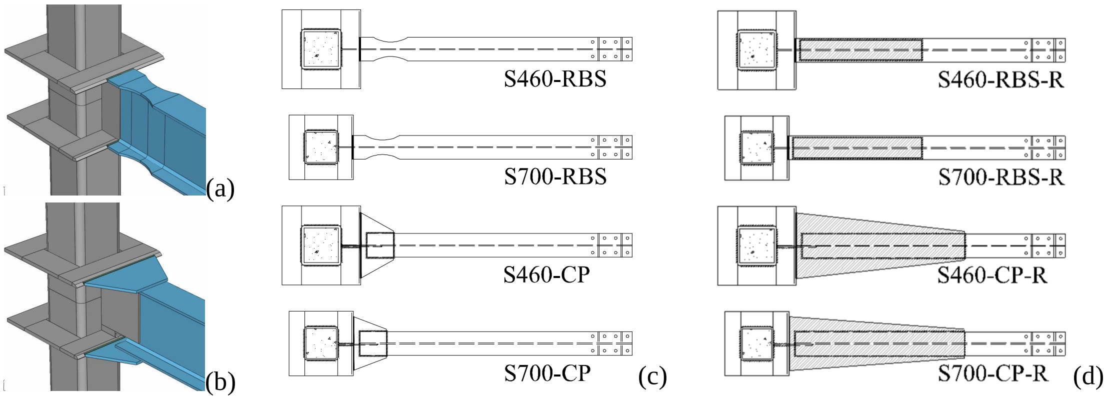


Figure 42. Welded external diaphragm beam-to-column joints with RBS (a), and cover plates (b), designed joint specimens (c), and corresponding joint specimens with reinforced beam (d).

The conceptual scheme and an illustration of the experimental test set-up is shown in Figure 43. A hydraulic actuator connected at the tip of the beam served as loading device. The column was supported at both ends considering a pinned connection. The horizontal and vertical displacements were blocked by the right support, and only the vertical displacements were restrained by the left support. A lateral support system was used to block the out of plane deformations of the beam. Global and local instrumentation was considered for measuring the force in the actuator, the displacement at the tip of the beam, the horizontal and vertical displacement at supports and respectively the deformations within the dissipative zone and the connection zone, as well as in the column web panel.

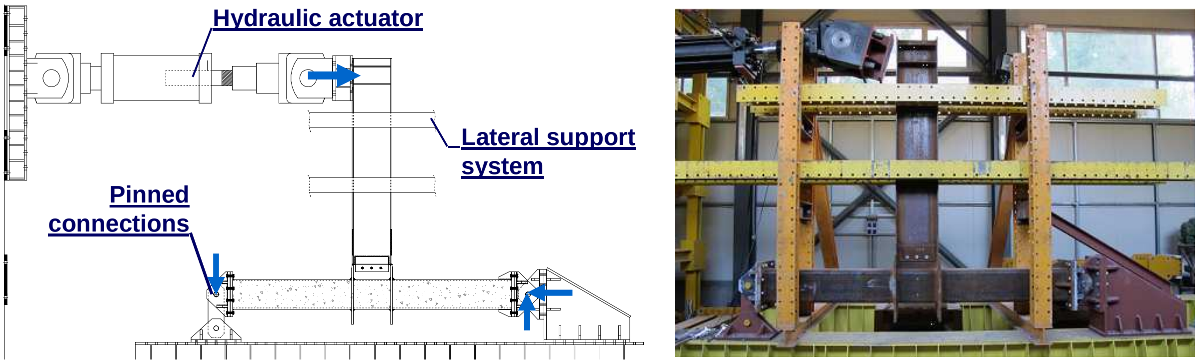
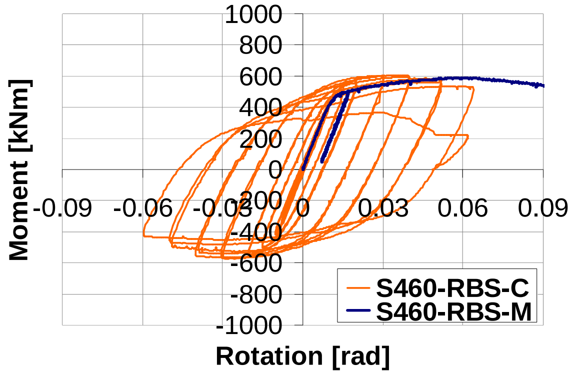


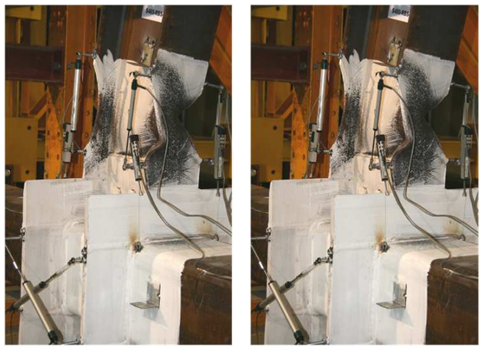
Figure 43. Static disposition and testing set-up.

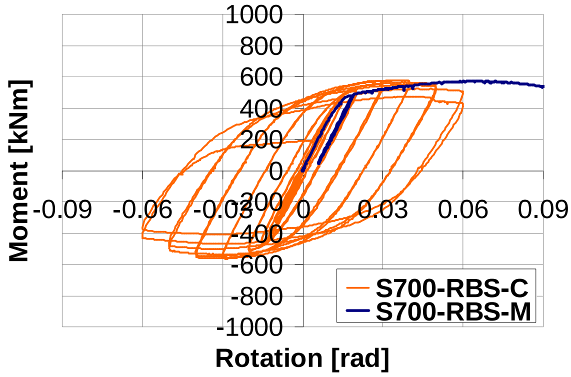
- *Experimental results*

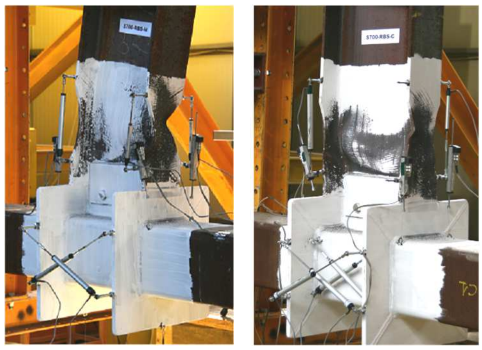
Figure 44 shows the cyclic and monotonic behaviour of specimens as well as their failure modes. For all specimens the first picture represent the failure of monotonic test while the second (right) corresponds to cyclic degradation. The following failure modes were recorded:

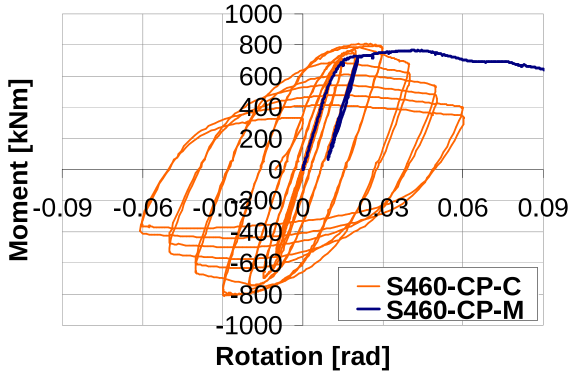
- RBS specimens: The yielding was initiated in the beam flanges within the RBS zone and was followed by large plastic deformations – local buckling of flanges and web under compression;
- joints with strengthened beam flanges RBS-R specimens: yielding was initiated in beam flanges (between the reinforcing plate and external diaphragm) under compression/tension and was followed by yielding of web and external diaphragm. Eventually, the beam flanges fractured in the heat affected zone (HAZ) due to tension forces;
- cover plate (CP-R) specimens: yielding was initiated in the external diaphragm and was followed by local deformations of cover plates under compression/tension and yielding of the column web panel. For the S700-CP-R joints, the yielding was initiated in the compressed cover plate and was followed by failure of the welded connection between plates

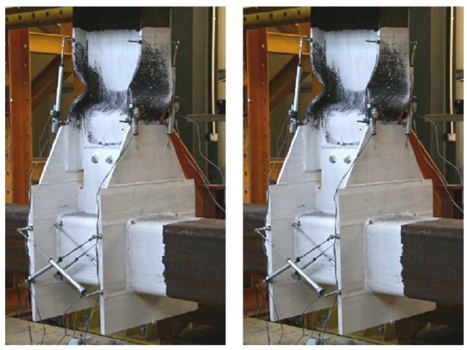












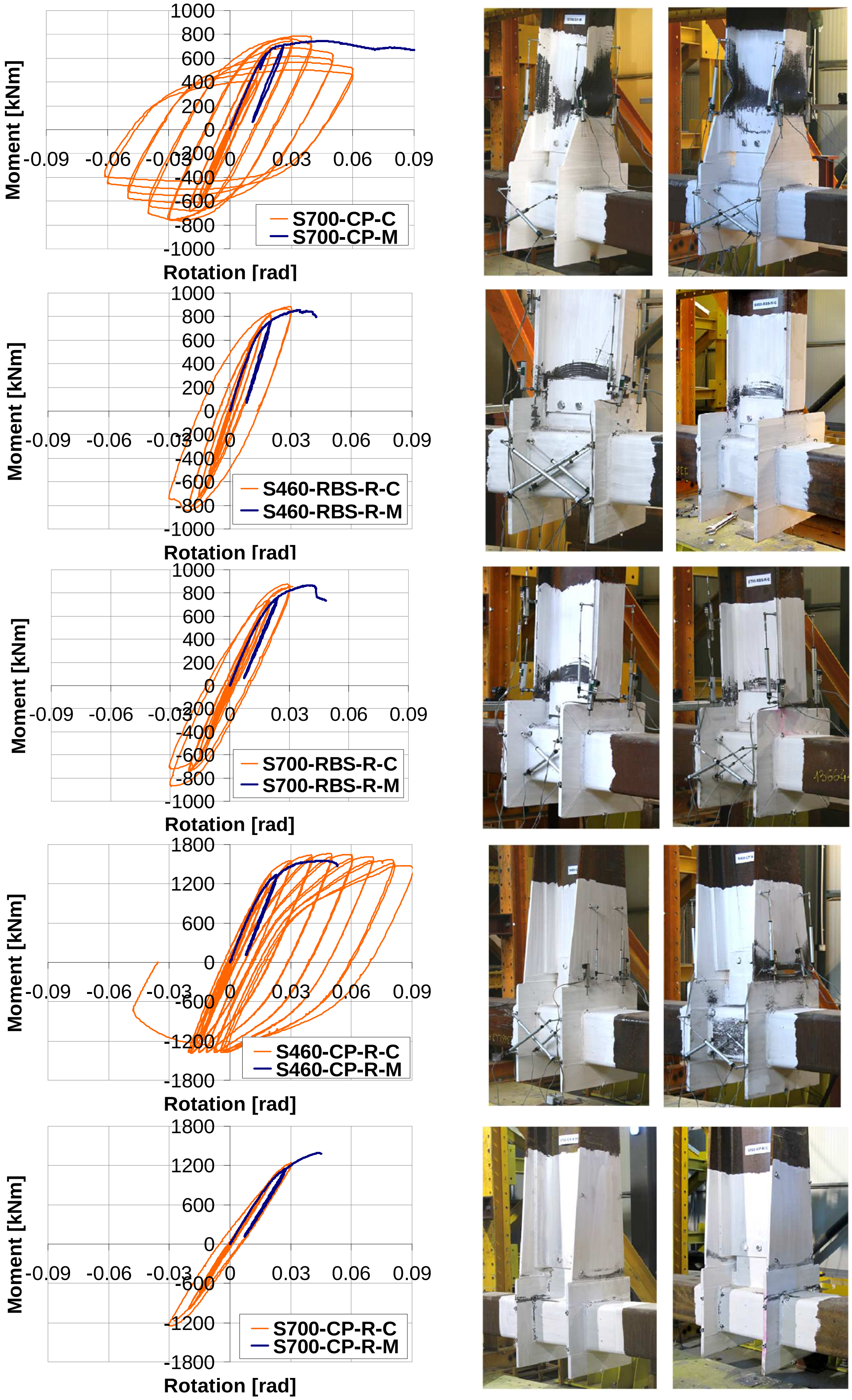


Figure 44. Monotonic and cyclic response of CFT specimens (continued).

- *Evaluation of experimental results – overstrength of the connection zone*

In order to assess the overstrength of the joint and connection zone, a comparison was made between the four designed joints and the corresponding joints with reinforced beam. Figure 45 illustrates as example the overstrength of the RBS joints. The moment-rotation curves were computed at the connection to the external diaphragm, and the overstrength was evaluated corresponding to the yield point and to the maximum capacity. Consequently, the overstrength of the connection zone for the RBS joints (S460 & S700) was evaluated in the amount of 34% and 35% at yield, and respectively 53% and 60% at maximum capacity. The overstrength of the connection zone for the CP joints (S460 & S700) was computed in amount of 55% and 43% at yield, and respectively 101% and 86% at maximum capacity.

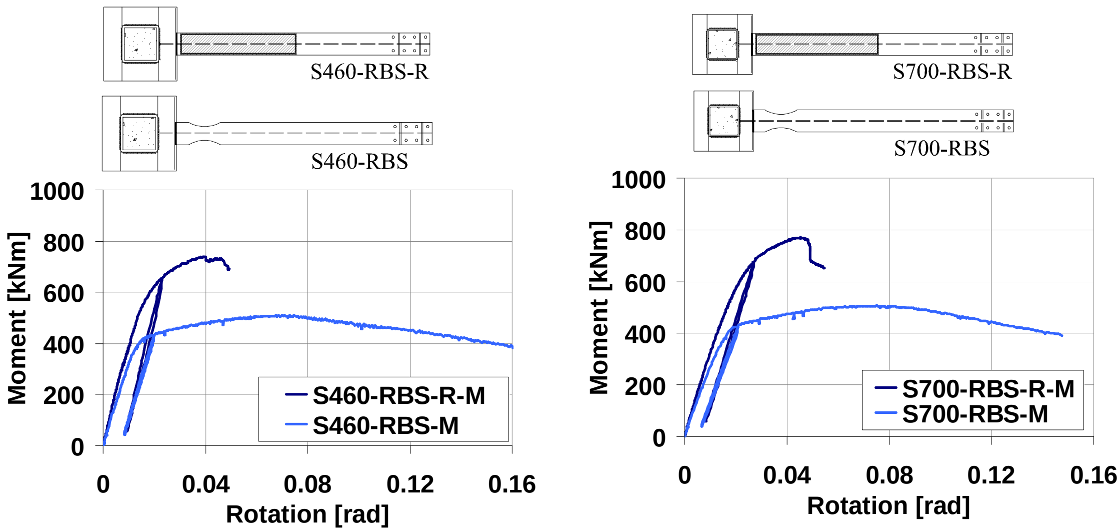


Figure 45. Monotonic and cyclic response of CFT specimens (continued).

- *Evaluation of experimental results – contribution of components to the joint rotation*

The measurements performed during the tests allowed assessing the contribution of the following regions: plastic hinge (dissipative zone), connection (welded connection + external diaphragm), and column web panel. Figure 46 shows the contributions of the connection and column web panel to the overall joint rotation. For the case of RBS and CP specimens these can be appreciated as small in comparison with the rotations due to that of beam plastic hinge.

The deformations within the reinforced beam zone were proved to be low also in cases of CP and RBS reinforced specimens. The main contribution to the overall joint rotation was given by the connection zone (for reinforced RBS joints), and respectively connection and column web panel (for reinforced CP joint). It is to be noted that the measurements from the connection zone included also the deformations of the external diaphragm.

The rotation capacity of the RBS and CP designed joints was evaluated considering the EN 1998-1 criterion for which the reduction of stiffness and capacity is not greater than 20%. Figure 47 show the contribution of components to the joint rotation in case of RBS and CP joint assemblies subjected to the cyclic loading. The main deformations developed in the plastic hinge. The total assembly rotation for which the reduction of stiffness and capacity was not greater than 20% was corresponding to the 50 mrad cycle for RBS joints and to the 40 mrad cycle for the CP joints. The contribution of different components, including the elastic rotation, to the total joint rotation is summarized in Table 6.

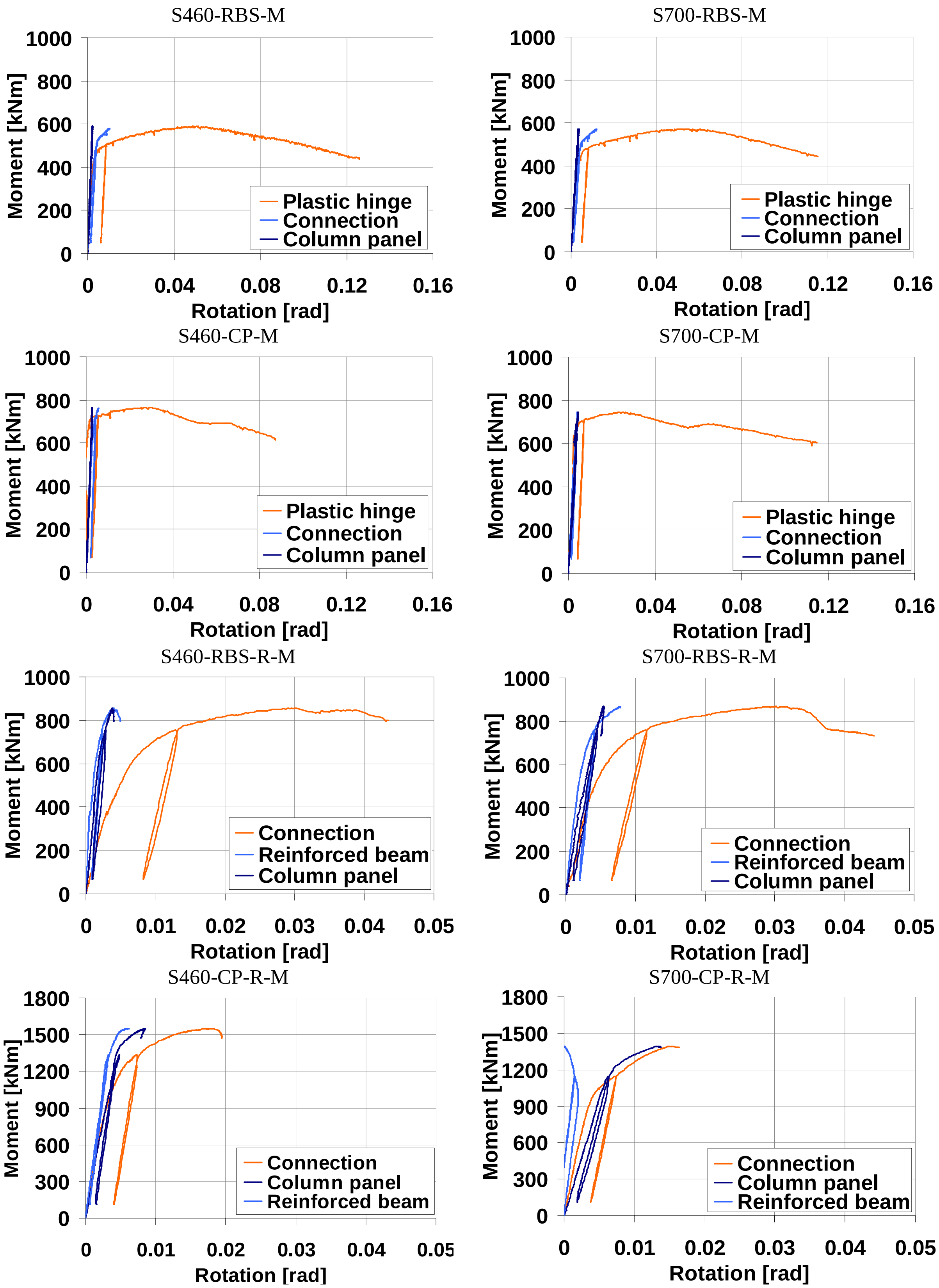


Figure 46. Monotonic and cyclic response of CFT specimens (continued).

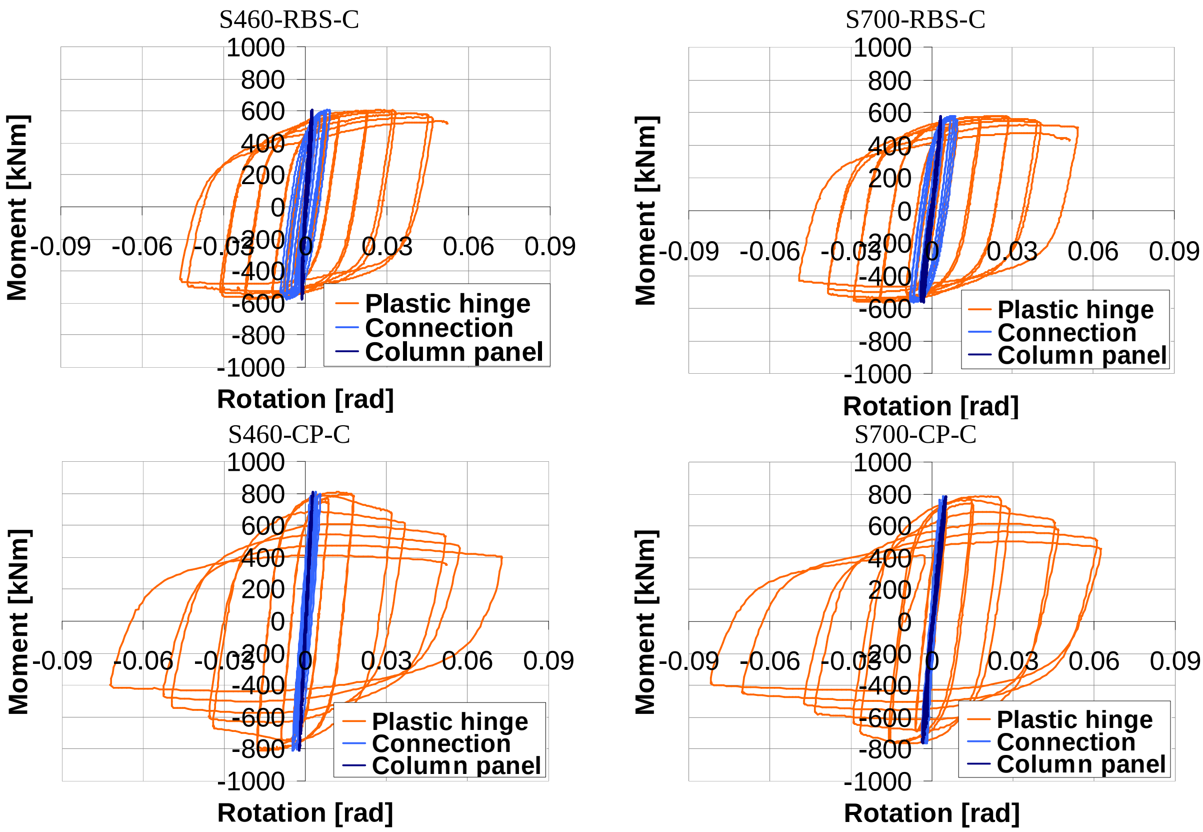


Figure 47. Contribution of components to joint rotation.

Table 6. Contribution of components to the rotation capacity of the joints [mrad]

	S460-RBS	S700-RBS	S460-CP	S700-CP
Total assembly rotation	50	50	40	40
Column web panel	1.1	3.8	1.6	2.1
Connection	6.7	7.6	1.9	3.7
Plastic hinge	43.1	38.2	34.1	29.1
Elastic rotation	3.7	4.5	5.7	6.7

- *Finite element simulations – influence of spatial loading on joints*

In a first step the material model was calibrated based on material sample tests. The pre-test numerical simulations, containing the calibrated material model, allowed assessing for each joint configuration, the stress distribution and plastic strain, as well as the moment-rotation curve, confirming a good design of the joints and the intended failure mechanism. The pre-test numerical simulations are not the subject of the current paper which focuses on the calibration of material model, calibration of the numerical model of the joints, extension of the experimental program showing the influence of a set of parameters, and in a lower extent the validation of joint components.

The numerical investigations were performed with the finite element modelling software Abaqus (2007). All the components of the beam-to-column joints were modelled using solid elements. Due to the large amount of contact surfaces between the concrete core and surrounding steel tube, the Dynamic Explicit type of analysis was considered. For the interaction between the steel tube and the concrete core, a contact law, characterised by normal as well as tangential properties, was defined that allowed the two parts to separate. The load was applied through a displacement control at the tip of the beam and the boundary conditions considered at the ends of the column were consistent with the experimental test set-up, i.e. pinned and simple support. The discretisation of the elements was performed using linear hexahedral elements of type C3D8R.

The comparison between test results and pre-test simulations was observed to be relative close. Consequently, the calibration and refinement of the numerical models was performed using the measured geometry of the specimens. In addition, because the beam was not completely restrained by the out of plane lateral system, a lateral contact was defined (see Figure 48) considering a small gap between flanges and the contact elements. It is to be noted that within the numerical simulations it was not accounted for material fracture or crack propagation, phenomena that were observed at some of the joint specimens. Because in the numerical models of these joints, no fracture developed, the capacity did not suffer as in the experimental tests.

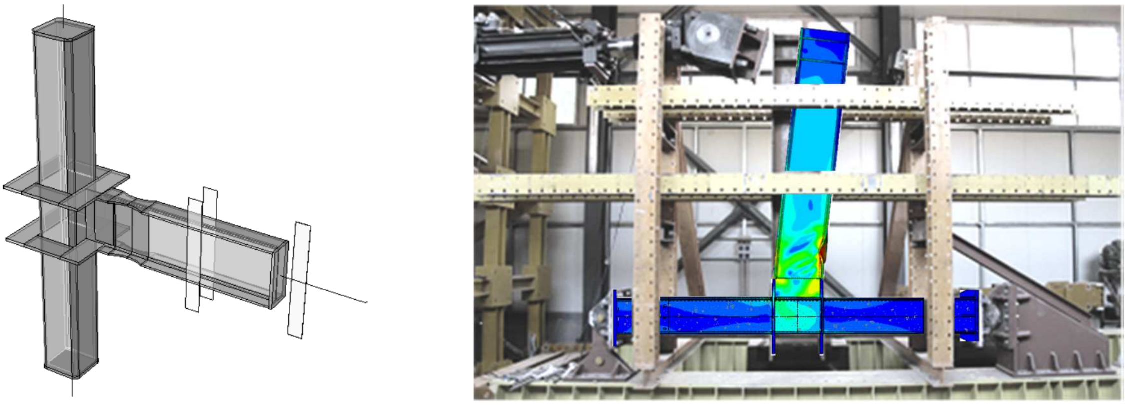


Figure 48. Illustration of the joint model accounting for the out of plane lateral system.

From the calibration, a set of numerical models were obtained which were capable to reproduce with a good accuracy the response of the joints in both moment-rotation curve and failure mechanism, i.e. formation of the plastic hinge in the beam (RBS and CP designed joints) and yielding of components (joints with strengthened beam). Consequently, for each joint configuration, a comparison is shown between test and simulation in terms of moment-rotation curve computed at column centreline. The plastic strain is shown in comparison to the failure mode observed during the test. The result from the calibration of the monotonic tests on beam-to-column joints is shown in Figure 49 for the joints with reduced beam section. The same good agreements were found for the rest of the specimens.

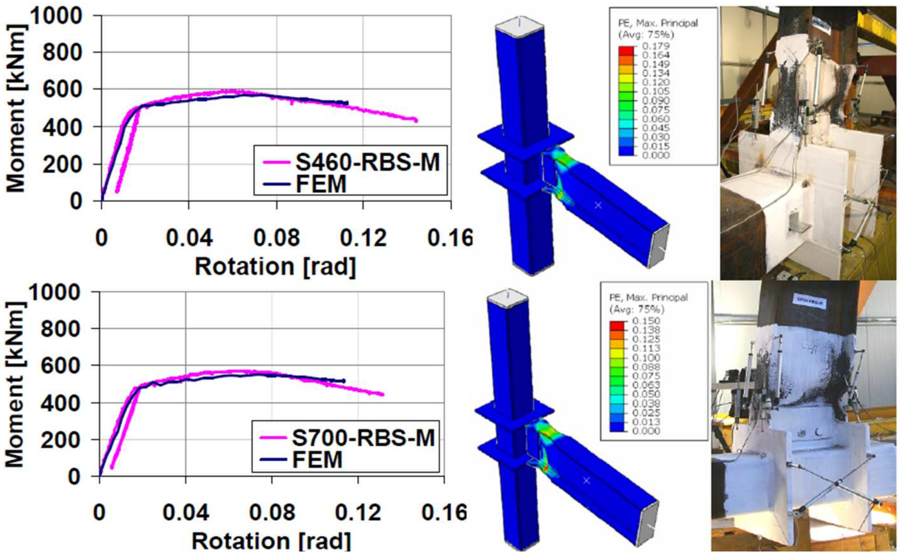


Figure 49. Comparison between test and simulation (example for RBS joints).

The numerical models calibrated based on the joints subjected to monotonic loading were further used for the numerical calibration of the cyclic tests. One of the differences was related to the loading procedure. Consequently, a smooth cyclic loading pattern was used, which was characterized by one cycle for each of the following amplitudes: 10, 15, 20, 30, 40, 50, and 60 mrad. Another difference between cyclic and monotonic analyses was related to the material model. A combined isotropic/kinematic cyclic hardening model was therefore adopted. The input for the material model was represented by the yield strength of the steel part, and in addition the cyclic hardening parameters as given by Dutta et al. (2010), i.e. C_1=42096, γ_1=594.45, Q_∞=60, b=9.71.

The results from the cyclic analysis are shown for each beam-to-column joint configuration in terms of moment-rotation curve (computed at column centreline), von Misses stress distribution and equivalent plastic strain. An illustration of the failure mode, as obtained from the experimental investigations, is shown as well. The comparison between test and numerical simulation is shown in Figure 50-a-b for the RBS joints, in Figure 50-c-d for the CP joints, as example of cyclic calibration. The stress distribution and plastic strain are shown for each joint configuration corresponding to the end of the last cycle. However, based on these calibrated models it is possible to acquire also the stress distribution and the plastic deformations in the joint corresponding to the amplitude of 10, 15, 20, 30, 40, 50 or 60 mrad.

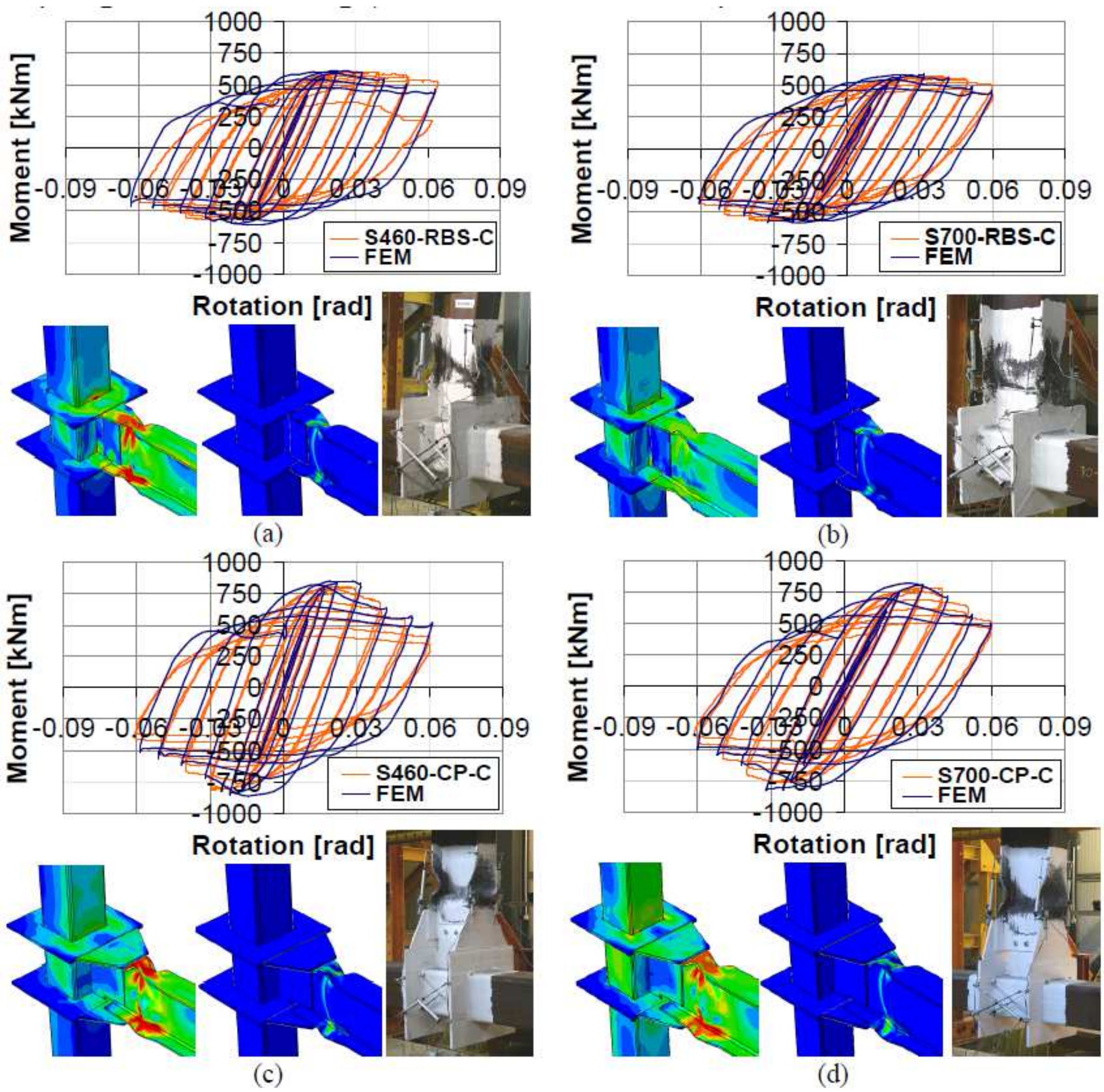


Figure 50. Comparison between test and simulation for cyclic loading (example for RBS and CP specimens).

The results from the numerical investigations of the cyclic tests show a good correlation with the experimental results considering the moment-rotation hysteretic loops as well as the failure mechanism (plastic hinge – buckling of flanges and web of beam in the dissipative zone, yielding of components – failure in the heat affected zone of the reinforced RBS joints, plastic deformations in the external diaphragm and column panel zone).

In order to extend the results of the experimental program, a set of complementary numerical testing cases were considered for the investigation through simulations in order to assess the influence of different parameters on the joint behaviour, such as:

- influence of the concrete core – i.e. the response of the joint without concrete core in comparison to the reference joint with CFT column;
- influence of the axial force – i.e. the response of the joint with axial force in the column ($N=0.5*N_{pl}$) in comparison to the reference model;
- response of the joints with two and respectively four beams welded around the concrete filled tube;
- influence of the external diaphragm.

- *Influence of the concrete core*

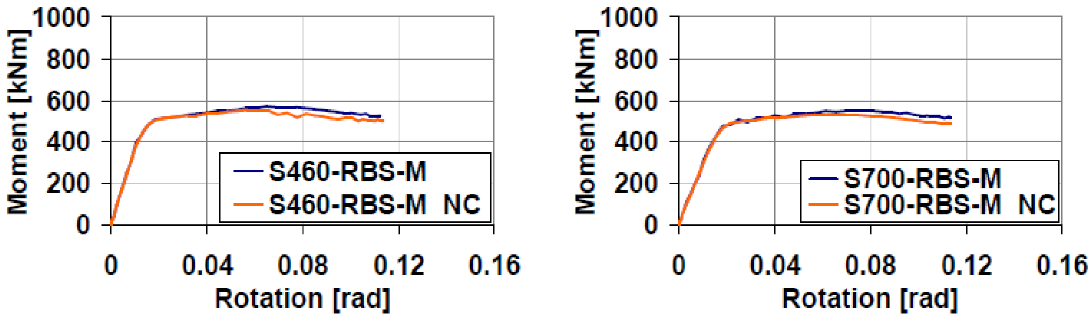


Figure 51. Influence of concrete core – RBS joints.

In order to evaluate the influence of the concrete core – i.e. the response of the joint without concrete core in comparison to the reference joints with composite column (CF-RHS) – a set of additional numerical simulations were performed on the calibrated numerical models from which the concrete part was eliminated (NC → no concrete). The results are shown in terms of moment-rotation curve computed at column centreline. Figure 51 show the results of simulations in case of RBS joints, and Figure 52 the results for CP joints: in these cases, the absence of concrete core did not affect significantly the response. A minor reduction of the capacity was observed.

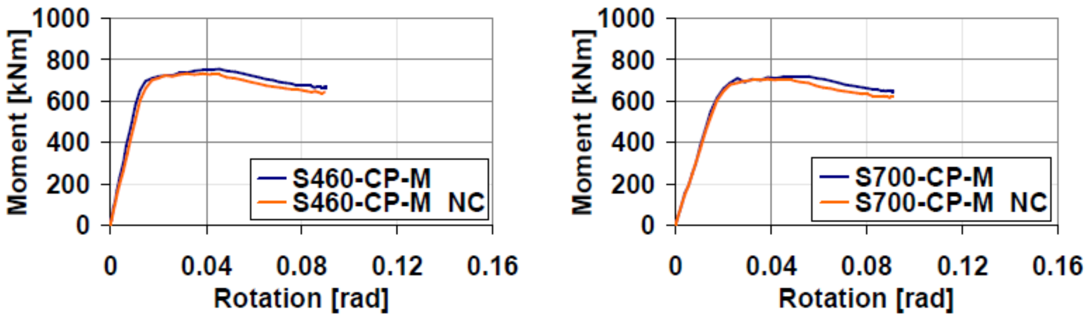


Figure 52. Influence of concrete core – CP joints.

In contrast, for the reinforced joints (extended CP) a significant reduction of capacity can be observed in Figure 53. In these cases, the bending moment and implicitly the shear force in the column web panel were much higher than in the cases shown above. In other words, the shear force in the column web panel exceeded the capacity of the steel tube in shear, and therefore a higher capacity was corresponding to the joints with concrete core.

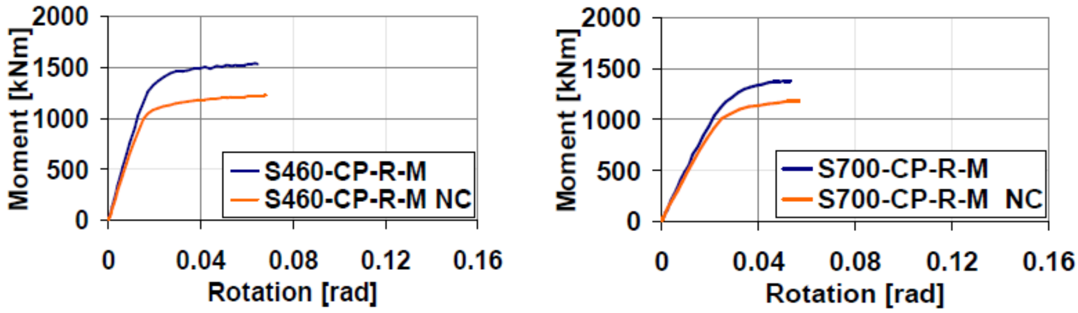


Figure 53. Influence of concrete core – reinforced CP joints.

- *Influence of the axial force*

For assessing the influence of the axial force in the column with respect to the behaviour of the beam-to-column joint, numerical simulations were performed on one of the joints (S460-CP) considering the following cases:

- joint with concrete core and an axial force level in the column corresponding to 50% of $N_{pl,Rd,composite}$;
- joint without concrete core and with an axial force level in the column corresponding to 50% of $N_{pl,Rd,steel}$.

For each of the two cases, in a first step, the column was axially loaded up to the considered level (see Figure 54a and Figure 55a), and in a second step the beam was loaded. The stress distribution and implicitly the failure mechanism of the joint with concrete core are shown in Figure 54b. The corresponding moment rotation curve is compared in Figure 54c with the reference model.

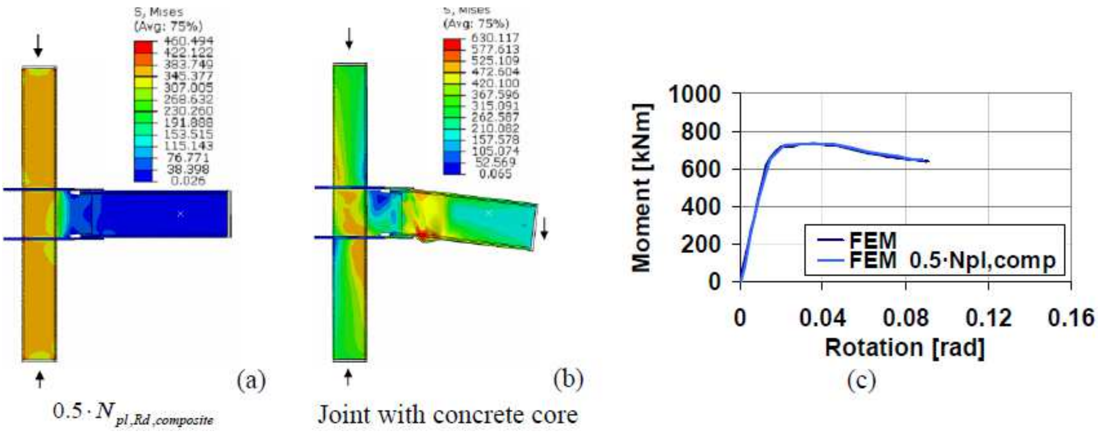


Figure 54. Influence of 0.5·$N_{pl,Rd,composite}$ axial force: S460-CP joint.

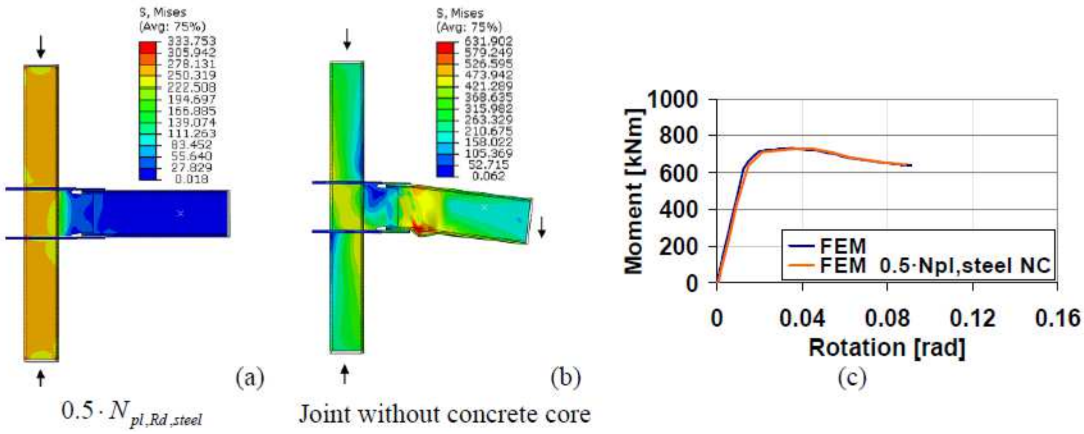


Figure 55. Influence of 0.5·$N_{pl,Rd,steel}$ axial force: S460-CP joint.

It can be observed that the axial force in the column did not affect the response of the joint compared to the reference model, and that the plastic hinge formed in the beam. In addition, the stress distribution and implicitly the failure mechanism of the joint without concrete core are shown in Figure 55b. The moment-rotation curve is compared in Figure 55c with the reference model. In this case it could be concluded too, that the axial force in the column did not affect the response of the joint without concrete core compared to the reference model and that the plastic hinge formed in the beam.

- *Influence of the axial load on beams*

The original experimental tests were performed on single sided beam-to-column joints. It was therefore necessary to assess the behaviour of the joints (including column web panel) in the situation of loading from two or even four sides which corresponds to a more demanding scenario. Figure 56 and Figure 57 show the stress distribution and plastic strain corresponding to the S700-CP joints with- and respectively without concrete core. Consequently, for the case with composite column and two beams (FEM 2G), a reduction of the stiffness can be observed in Figure 58a compared to the test curve, but the failure mode was not affected (see in Figure 56 plastic hinges in the beams). The absence of the concrete core (NC → no concrete) lead to reduction of capacity (Figure 58b) and to yielding of the column panel (Figure 57) – plastic hinges did not form in the beam as for the joint with concrete core.

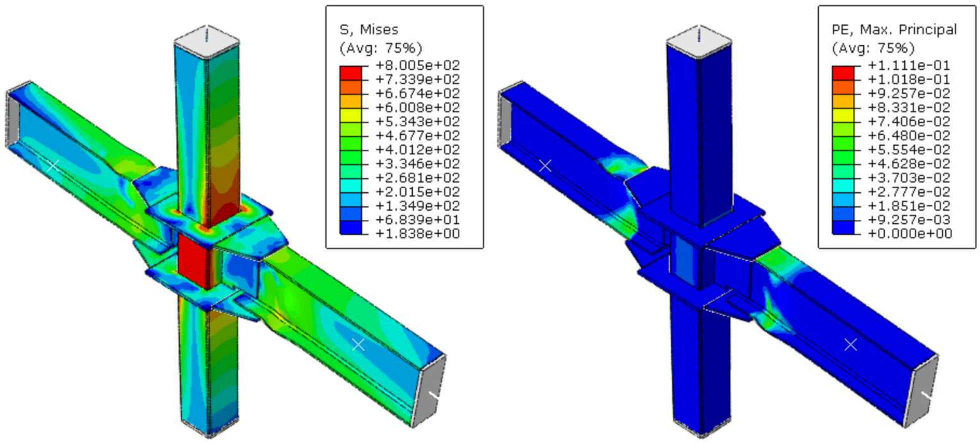


Figure 56. Behaviour of S700-CP joint with beams welded on two sides (CFT).

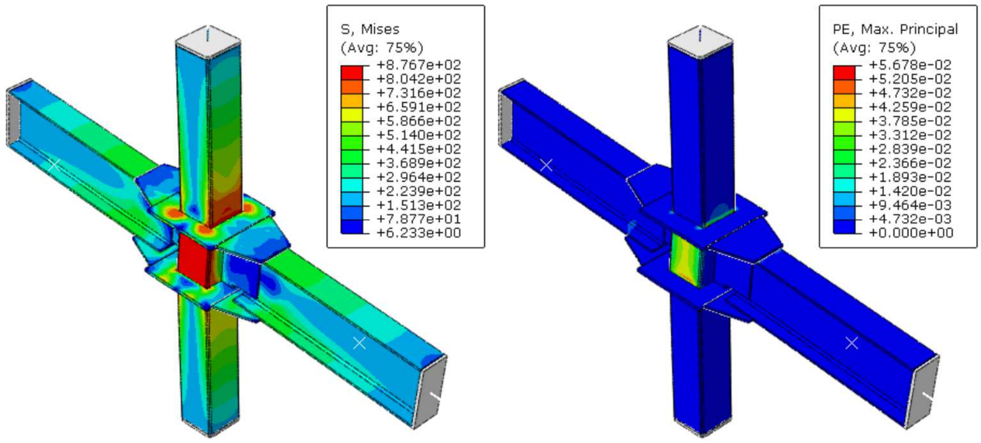


Figure 57. Behaviour of S700-CP joint with beams on two sides (no concrete in tube).

A similar conclusion can be drawn for the joint configuration with reduced beam section: a reduction of the stiffness can be observed also in this case compared to the test curve. The absence of the concrete core did not affect the capacity or the failure mode. Consequently, the plastic hinges are developing in the beams.

The weak-beam/strong-column design concept was therefore confirmed for both the CP and RBS joints, considering the case with beams welded on two sides, and in addition the study showed the contribution of the concrete core to joint behaviour.

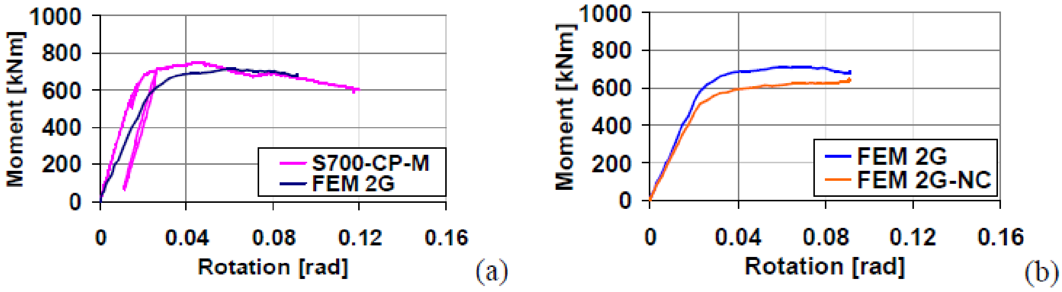


Figure 58. Behaviour of S700-CP joint with beams on two sides (no concrete in tube).

The response of the joints was also investigated considering the case with four beams welded around the concrete filled tube. As reference, the S460-RBS and S460-CP joint models were used. The loading was applied in displacement control at the tip of the beams.

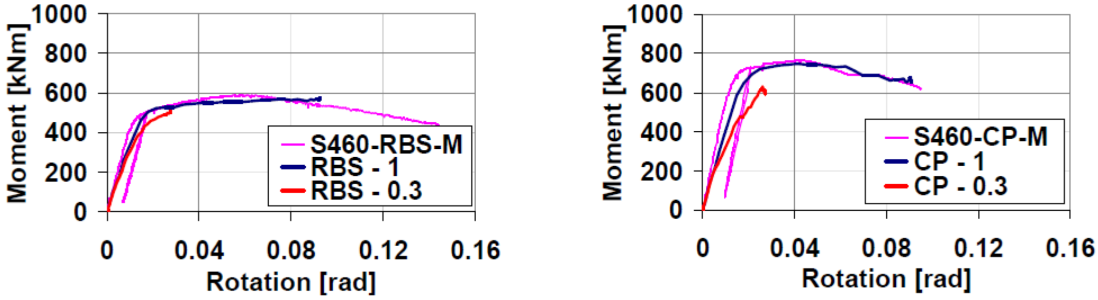


Figure 59. Joints with four beams – Moment-rotation curves.

The load ratio was considered in amount of 100% on one direction and respectively 30% on the other direction. As a result the moment rotation curves are shown in Figure 59 compared to the test results. As it can be observed, on the main loading direction the moment-rotation curve suffered a small reduction of the stiffness but the capacity or the failure mode was not affected. Figure 60 shows the von Misses stress distribution and the plastic strain in the two joint configurations. Therefore, considering the loading of the joints from two directions it can be observed that the joint components (external diaphragm, column web panel) did not suffer plastic deformations.

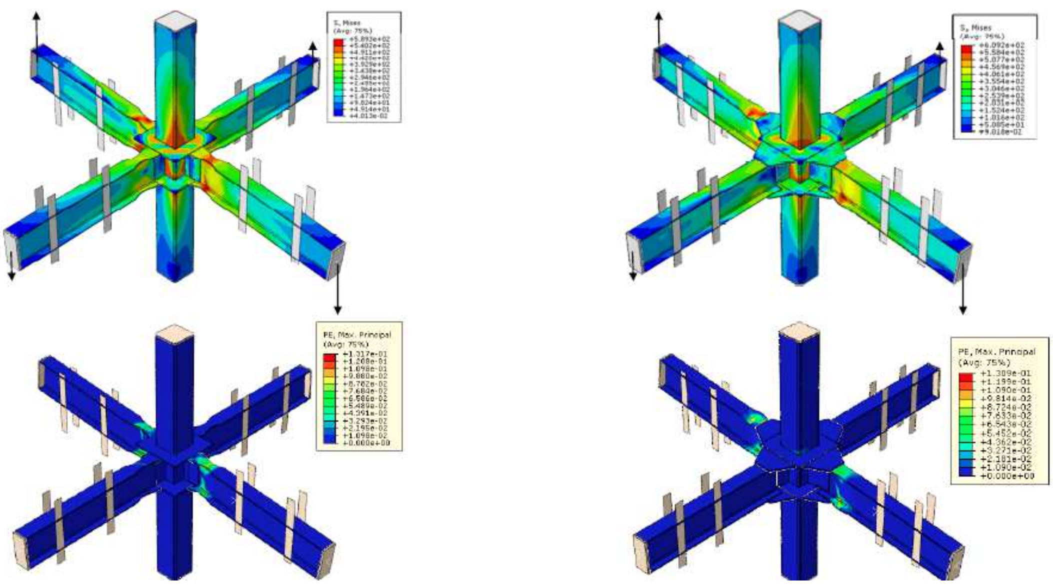


Figure 60. Joints with four beams – von Misses stress distribution and plastic strain.

- *Influence of external diaphragm*

Another parameter studied was the influence of the external diaphragm. Compared to the reference model, i.e. S460-RBS joint characterised by an external diaphragm of 150 mm width, three additional cases were considered in which the width was reduced to 100, 50 and respectively 0 mm.

Figure 61 shows, for each of the three case, the von Misses stress distribution, the plastic strain and the moment-rotation curve which is compared to the reference model. The components of the joint, and therefore also the external diaphragm were designed based on the plastic bending moment and shear force developed in the plastic hinge – and which were affected (increased) by the overstrength factor and the strain hardening factor ($\gamma_{ov}\cdot\gamma_{sh}$=1.25·1.1=1.375), in other words the joint components were designed to possess a certain amount of overstrength with regard to the dissipative zone. It is therefore natural that in the situation in which the steel grade of the beam has a yield strength relative close to the nominal one, and in which the external diaphragm is reduced with an amount covered by the overstrength assumed in the design, the response of the beam-to-column joint is not significantly affected. In the original situation the failure mode was by small plastic deformations in the external diaphragm and at the welded connection to the beam. In contrast, for the joint with external diaphragm of 50 mm width and respectively joint without diaphragm, the moment-rotation curve suffered a significant reduction of stiffness and capacity. Also the stress concentration and the plastic strain moved away from the reduced beam section to the welded connection and particularly in the heat affected zone (HAZ) close to the extremities of the flanges. Thus the load transfer mechanism, according to EN1993-1-8 (2004), from a transverse plate to the side walls of a rectangular hollow section tube by the means of an effective width (smaller than the actual width of the plate) is confirmed.

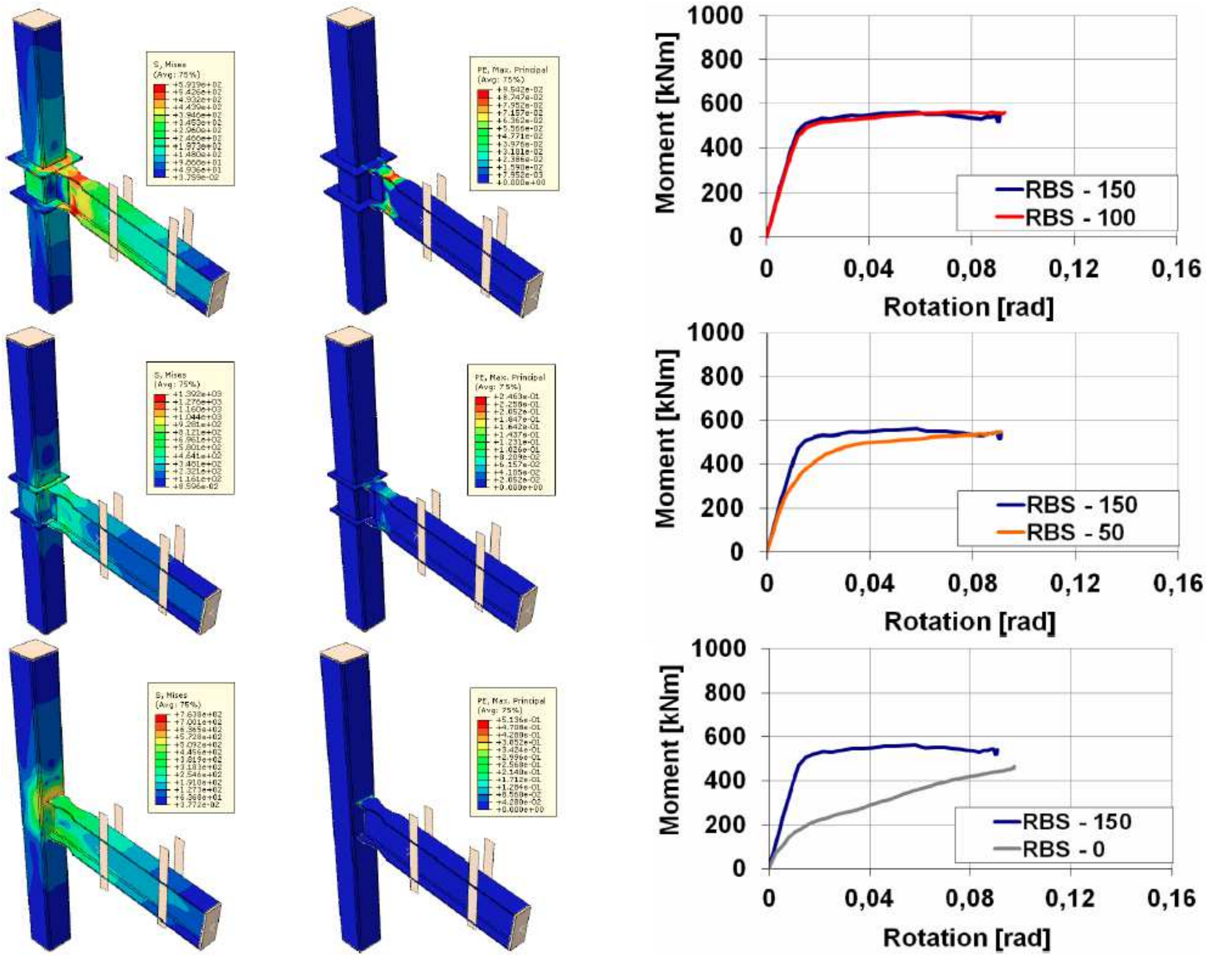


Figure 61. Response of the S460-RBS joint considering the reduction of the external diaphragm from 150 mm to 100, 50 and respectively 0 mm.

- *Conclusions on the behaviour of steel beam-to-composite RHS column joints*

The experimental investigations performed on beam-to-column joints under both monotonic and cyclic loading evidenced a good conception and design of the joints (RBS and CP) justified by the following observations:

- elastic response of the connection zone;
- formation of the plastic hinge in the beam;
- good response of joint detailing and welded connections with one exception (S700-CP-R-M / C) which evidenced weld failure, however corresponding to a force level higher than the design capacity computed using the real material properties.
- a significant overstrength of the welded connections, respectively of the external diaphragm and column web panel was observed. Therefore the overstrength requirements from EN 1998-1 were satisfied.
- for the joints with reduced beam section and with cover plates, the main plastic deformations occurred in the beam (plastic hinge). The contribution of the connection and column panel to the overall joint rotation was small. For the joints with reinforced beam, the main contribution to the overall joint rotation was given by the connection zone including external diaphragm (for strengthened RBS joints), and respectively external diaphragm and column panel (for strengthened CP joint).
- corresponding to cyclic tests performed according to the ANSI/AISC 341 [28] loading protocol, the RBS and CP designed joints evidenced rotation capacities of 50 mrad (RBS joints) and respectively 40 mrad (CP joints) for which the degradation of strength and stiffness were not greater than the 20% limit defined in EN 1998-1.

The extension of the experimental programme by numerical simulations has led to the following conclusions:

- the influence of concrete core was proved to be low for the single sided RBS/CP joints (i.e. reduced load level), but significant corresponding to joints with strengthened beam, respectively to joints with multiple beams (i.e. higher load level);
- the axial force in the column did not affect the response of the joint;
- for the joints with beams welded on two, respectively on four sides, a reduction of the stiffness was observed, but the capacity was not affected, nor the failure mode;
- in relation to the external diaphragm it was shown that a stiff diaphragm is necessary to assure a full strength joint with the development of the plastic deformations in the beam; with a flexible diaphragm or in its absence, a decrease of capacity and development of plastic deformations in the welded connection was evidenced;

2.3.3. Behaviour of composite link elements in eccentrically braces frames

In case of EBF, the seismic input energy is dissipated through of link elements. On the other hand, the use of composite beams (connection of the steel beam with the concrete slab) can improve the overall behaviour of the frame by a higher frame stiffness and higher resistance. However, the current design norms are not very explicit in explaining the influence of composite action. The investigation on link elements was based on experimental tests.

The EBF which was set as the base frame for the test series was adapted specifically for the CEMSIG laboratory conditions. The figure below shows the main structural dimensions and layout. Table 7 describes the EBF specimens which were tested and their configurations, depending on the parameters listed above.

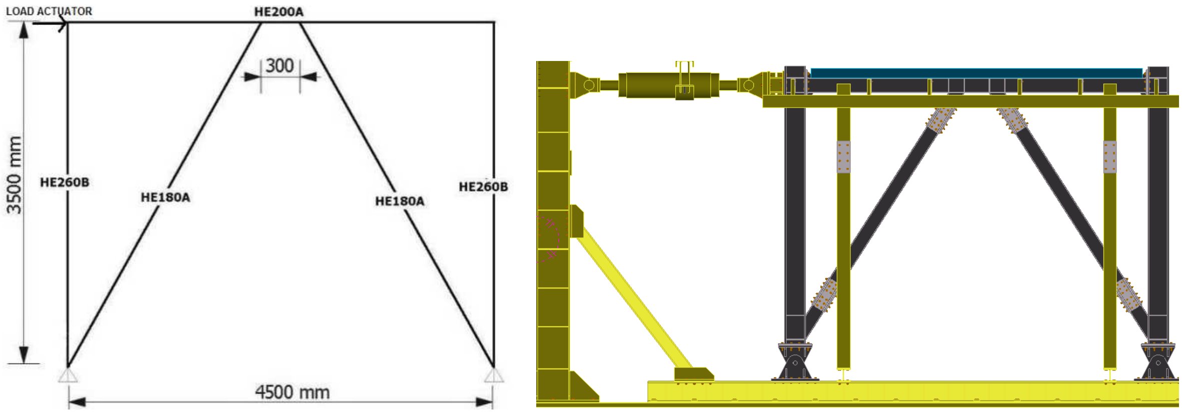


Figure 62. Dimensions and sections of the base EBF frame (static scheme and testing set-up).

Table 7. Description of the tested EBF specimens

Specimen	Beam type	Type of loading	Connectors on the link	Name of the specimen
1	steel	monotonic	No	EBF_M_LF-M
2	steel	cyclic	No	EBF_M_LF-C
3	composite	cyclic	No	EBF_Comp_LF1
4	composite	cyclic	Yes	EBF_Comp_LF2

The lateral load was applied at one end of the top beam through a 100 kN traction/compression actuator. In order to reduce the top force needed for completing the development of the plastic hinge, the tested specimens were pinned at the column base by mechanical hinges. However, as proven by numerical simulations (Goel and Foutch, 1984, Ricles and Popov, 1987, Stratan and Dubina, 2008) the behaviour of the link is not affected by the column-base connection type. Table 8 shows the main mechanical characteristics of the materials.

Table 8. Description of the tested EBF specimens

Element	Profile	Part	Design Class	Yield strength [N/mm²]	Ultimate strength [N/mm²]	Elongation at failure [%]
Beam	HE200A	web	S235	323	475	32.24
		flange		304	434	35.01
Brace	HE180A	web	S355	326	511	29.46
		flange		398	533	36.81
Column	HE260B	flange	S355	283	440	48.06
Concrete Slab	12 cm	-	C20/25	-	23	-

In order to determine the maximum force needed in the experimental tests, the experimental frame was initially modelled through a push-over FE analysis using nominal elastic-plastic characteristics of materials. For this it was considered the most force-demanding situation in which the beam was composite over its full length (Danku, 2011). In this initial modelling stage, the composite beam was modelled using the equivalent cross-section characteristics.

The load cell integrated in the actuator was used to record the applied force. LVDT displacement transducers were used to monitor different absolute and relative deformations or displacements between different components (see details on Figure 63). They have been located in key positions according to the type of specimen:

- global displacements of the frame, measured by DHTL, DHTRF, DHTRB transducers, located at top left and right columns;
- web panel distortion of the link: through DDT1 and DDT2 transducers;
- slip and displacements of the non-dissipative elements:

- absolute displacement of the pin connections: DHBL, DVBL, DHBR, DVBR;
- slip and elongation of the braces and their connections, through DBLL, DBLR, DBrTL, DBrBL, DBrTR, DBrBR transducers;
- deformation and rotation of joints: DBCTL, DBCBL, DBCTR, DBCBR transducers.

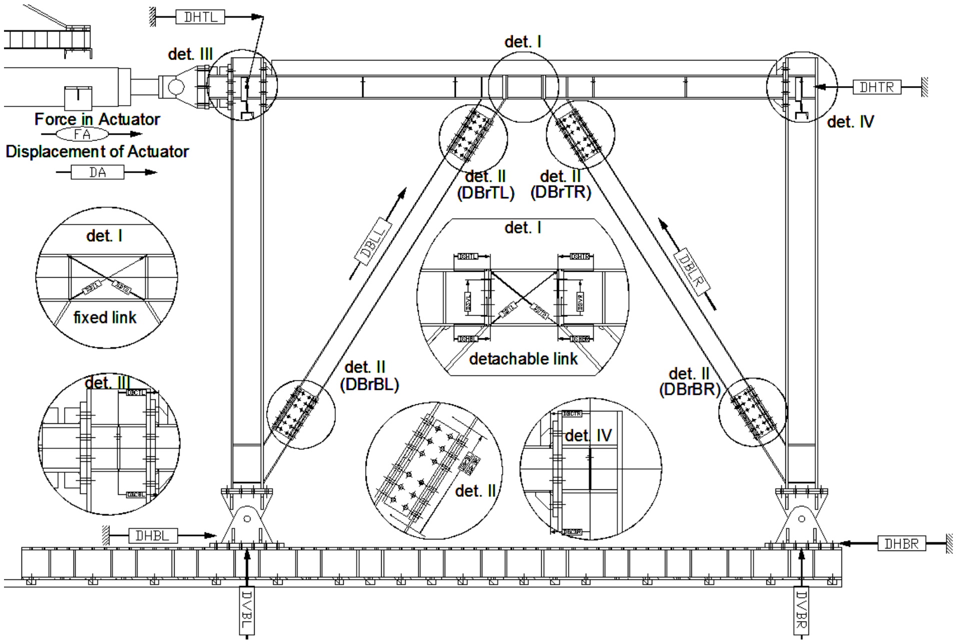


Figure 63. Global instrumentation of the EBF frame

The total link distortion was accounted for by the angle γ as follows (see Figure 64):

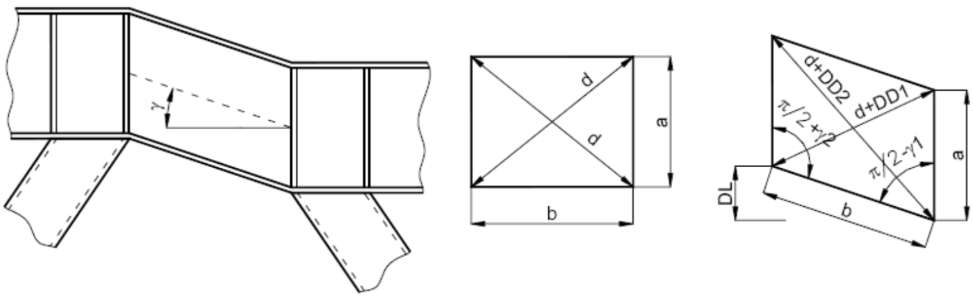


Figure 64. Deformation of the link element and definition of the distortion angle γ

$$\gamma = \frac{\sqrt{a^2 + b^2} \cdot (DD2 - DD1)}{2 \cdot a \cdot b}$$

where DD1 and DD2 are the recordings given by the two diagonals transducers measuring the link distortion. The standard ECCS procedure was applied for loading the monotonic and cyclic tests.

- *Experimental behaviour of steel link element*

As global behaviour, the base steel specimen (EBF_M_LF-M) is characterised as very ductile (see Figure 65), the plastic energy dissipating only in the steel link. It was loaded monotonically, up to failure. The next chart shows the force - top displacement curve of the specimen, where force represents the load induced by the actuator while the top displacement is the average displacement given by top transducers.

The chart from Figure 65 right shows the link distortion – computed according to (1) – versus the shear force of the link. The maximum rotation of almost 280 mrad can be considered as very ductile, this being largely greater than the required values from actual seismic norms, such as P100-1/2006 (2006) and EN 1998-1 (80 mrad) respectively FEMA356 (2000) of 110 mrad corresponding to Collapse Prevention Limit State (CPLS). Other frame members and components such as beam-to-column connections and braces exhibited only elastic behaviour. Their elastic behaviour was demonstrated by recorded deformations. Among all, the braces proved the largest deformation of 5 mm (see Figure 66), from which 4 mm remained permanent. This fact is explained by the slip in the braces splice-connection clearances (2 mm clearance for each connection).

The loading was stopped at very large inelastic link deformations on the descending branch, at a drop of more than 20% of the maximum load, coincident with stroke limitation of the actuator. Figure 67 presents the deformed shape of the link at the end of test.

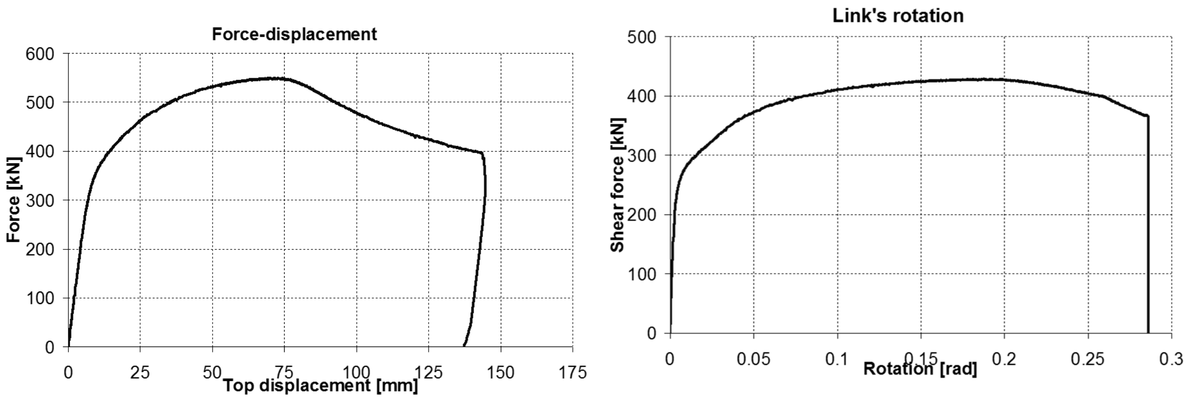


Figure 65. Load-top displacement and shear force-rotation curves for EBF_M_LF-M specimen

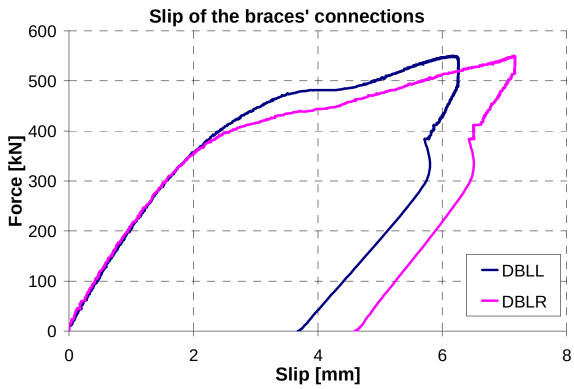


Figure 66. Total elongation of braces – 4mm remanent (specimen EBF_M_LF-M)

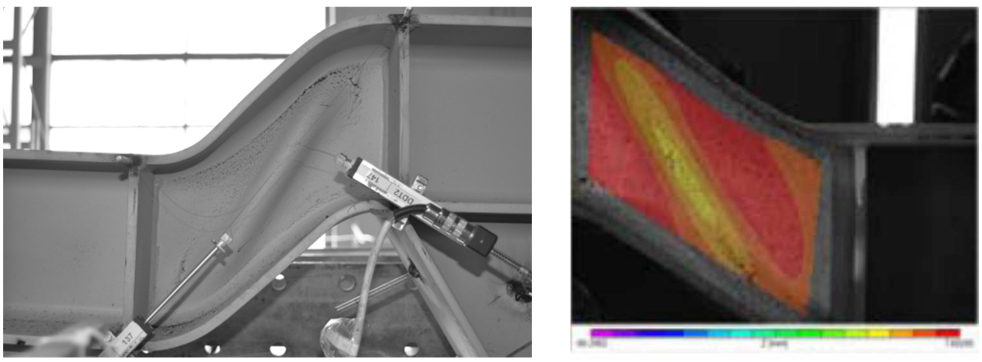


Figure 67. Dissipative link behaviour in shear for specimen EBF_M_LF-M.

The response curves obtained in monotonic loading (EBF_M_LF-M specimen) served for the determination of yielding characteristics used in cyclic loading (according to the ECCS procedure). The following values were determined: S_{ini}=48.20117 kN/mm, F_y=343 kN, d_y=7.9 mm.

The cyclic loading lead to a very small drop in the maximum load (about 20 kN), to a great diminution in the distortion capacity (from 280 to 160 mrad). However, the hysteretic curves present stable cycles, as shown in Figure 68, confirming the recognised good dissipation capacity of EBF link elements.

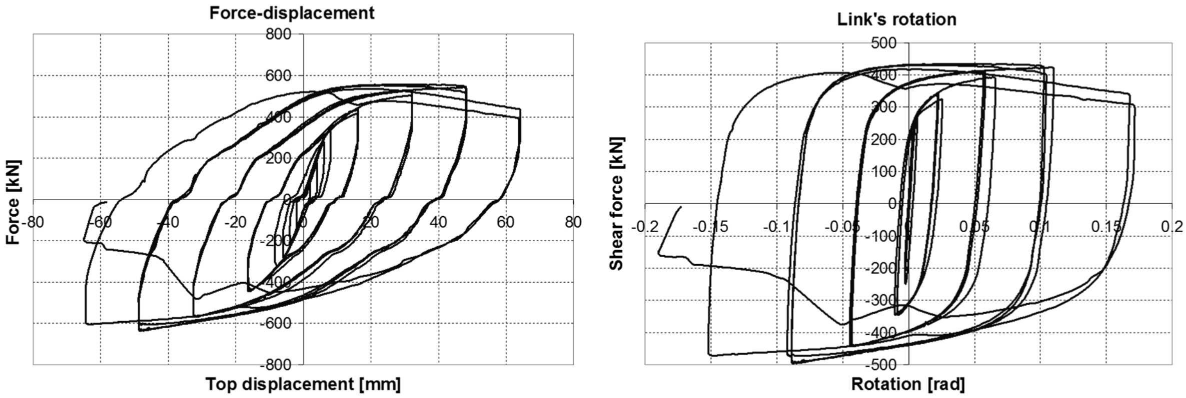


Figure 68. Global load – top displacement and shear force – link distortion response of EBF_M_LF-C specimen

The failure of the specimen (see Figure 69) occurred by complete shear of the web panel of the link, which reached 150 mrad for 20% load loss. Diagonal sinusoidal waves formed alternately during the plastic cycles in the web panel.

Figure 69. Shear failure of EBF_M_LF-C specimen

- *Experimental behaviour of composite link elements*

The EN 1994-1 4 conditions to achieve full-interaction of concrete slab over the brace triangulation, lead to the headed stud disposition shown in Figure 12 b) – 40 Φ 19*100mm over each triangulation, characteristic to composite beam with non-composite link specimen (EBF_Comp_LF1). In case of composite link specimen (EBF_Comp_LF2), the same distance increment (95mm) was kept over the link – see Figure 70 c). The reinforcing of the concrete slab (shown in Figure 70) consisted in top and bottom longitudinal bars Φ 12mm spaced at 15 cm tied by transversal stirrups Φ 12mm spaced at 15 cm.

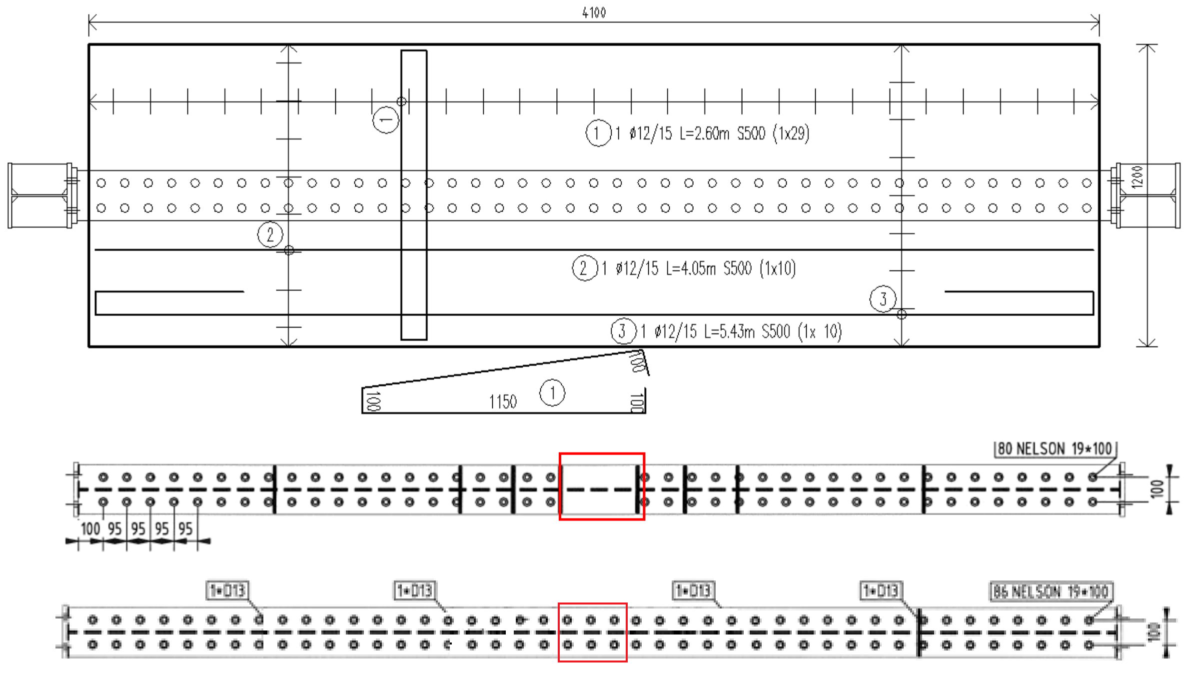


Figure 70. Shear failure of EBF_M_LF-C specimen

The global behaviour of two specimens is presented in Figure 71 and Figure 72 through hysteretic response curves (lateral force – story drift in Figure 71 and shear force – link distortion in Figure 72). It is to be noted that shear force – distortion for LF-Comp 2 specimen is truncated due the deterioration of the displacement transducer's readings, affected by the highly-damaged concrete slab above the link. However, both specimens show similar characteristics in the global behaviour. Only small differences could be found for the main response parameters: resistance, rigidity and ductility – see Table 9 for numerical values. The values of ultimate link distortions arise to 150 mrad in both situations and are considered sufficient to withstand important values if inter-storey-drift deformations of 2 to 3%.

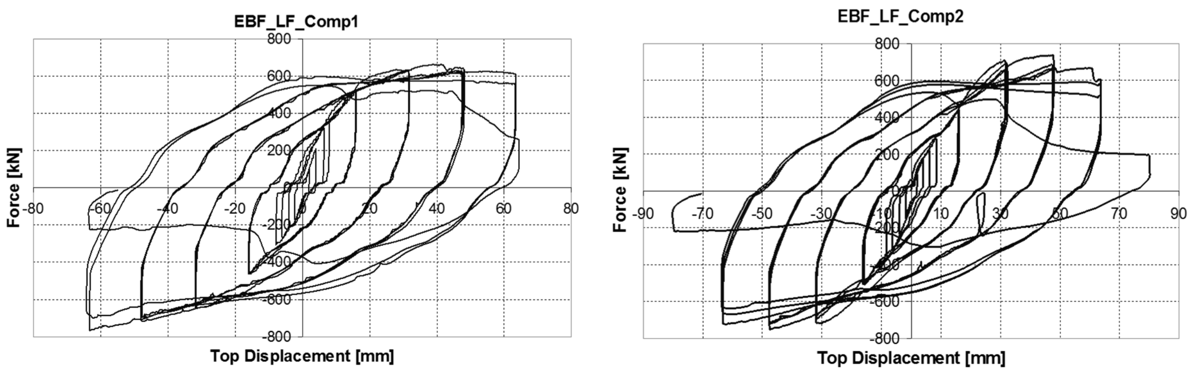


Figure 71. Top load – lateral displacement response hysteretic curves for specimens LF-Comp 1 and LF-Comp 2

Figure 73 presents the failure conditions for links and concrete slab. Both specimens experienced high damages in the link panel zone, practically reeling the web plate. Generally, the damage of the slab was concentrated above the link element developing crack patterns at angles of 45 degrees starting above the link flange corners. No relative deformations (horizontal or vertical) were recorded between the concrete slab and the steel flange. However, the presence of connectors on the link (specimen LF-Comp 2) led to a more important degradation of the concrete slab.

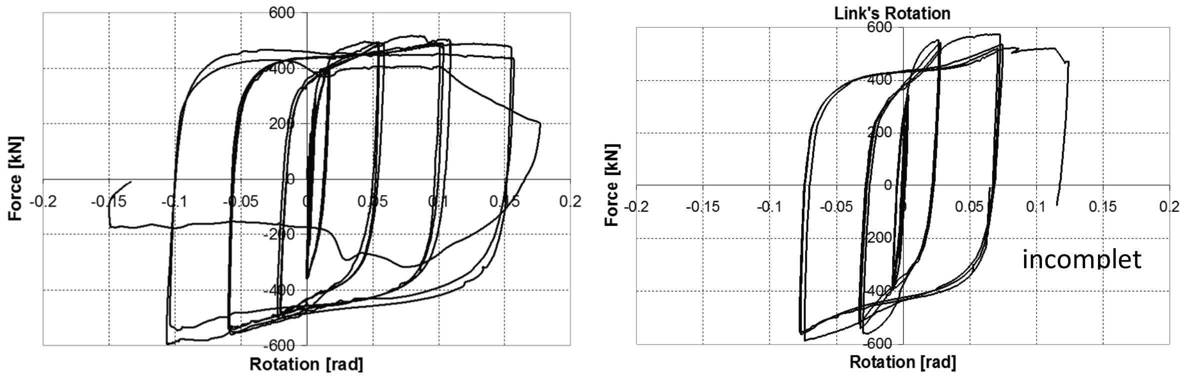


Figure 72. Shear force – link distortion response hysteretic curves for LF-Comp 1 and LF-Comp 2 specimens

Figure 73. Damage of the fixed link and concrete slab for EBF_LF_Comp1 (left) and Comp2 (right) specimens.

- *Appreciation of EBF specimen results - conclusions*

The synthetic results arouse form the EBF specimens are given in Table 9. The following notations were used:

- $S_{j,link}$ – initial rigidity of the positive branch of the envelope curve;
- V_{max} – maximum shear force during testing
- γ at V_{max} – link rotation corresponding to maximum shear force
- γ_{max} – maximum link distortion

Table 9. Description of the tested EBF specimens

Specimen	Initial rigidity $S_{j,link}$ [kN]	Maximum shear resistance V_{max} [kN]	shear distortion γ at V_{max} [mrad]	Maximum shear distortion γ_{max} [mrad]
EBF_M_LF-M	130460	429	187	286
EBF_M_LF-C	74644	495	87	171
EBF_Comp_LF1	123414	598	105	156
EBF_Comp_LF2	152488	587	74	150*

** Estimated value*

Note: all values were computed on the maximum envelope curve (cycle 1)

Figure 74 and Figure 75 present graphically the main differences between specimen results, in the form of envelope curves: lateral force – top displacement respectively shear force – panel distortions on. The charts follow the declared parameters:

- influence of composite slab (Figure 74)
- influence of loading type cyclic/monotonic (Figure 75)

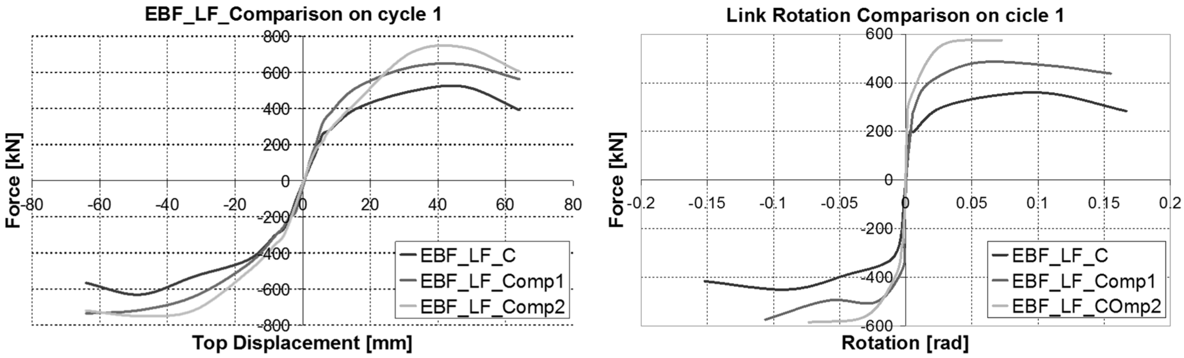


Figure 74. Influence of link typology – monotonic curves on cycle 1.

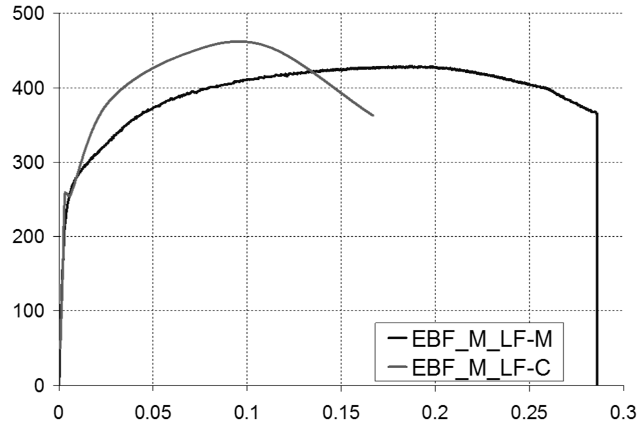


Figure 75. Influence of cyclic loading – monotonic curves on cycle 1 (steel specimens).

The main difference in results regards the resistance of specimens: both composite tests show greater resistances (up to 20%) as compared to steel specimens. Also, the maximum resistances are similar for full and partial composite specimens (Comp_LF2 and Comp_LF1 respectively), which means that the presence of connecters over the link plays only a secondary role in the global behaviour. This is in contradiction to the normative approach (EN 1998), which suggests that a disconnected dissipative zone lead to a behaviour identical to the pure steel behaviour.

Another immediate notice is that all specimens exhibited high levels of link distortions, reaching practically the requirements of modern seismic norms (80-140 mrad) for high ductility structures. In what concerns the initial stiffness, the computed values show that the rigidity of the cyclic envelope is about 30% higher in case of composite specimens in comparison to corresponding steel element.

In case of steel link specimens, the cyclic loading influenced the global behaviour, with respect to the following parameters:

- up to 40% decrease in rigidity;
- 15% increase in ultimate resistance;
- the cyclic specimen reached smaller ultimate rotations, up to 171 mrad, compared to 286 mrad recorded in case of monotonic loading.

However, the most important conclusion of the link experimental study is that the presence of the slab over the link element will influence noticeably the behaviour of the frame, independently of the link connection with the concrete slab.

2.4. Seismic behaviour of composite structures

Since the steel was largely employed in building structures, the use of frames became one of the most used structural typologies in civil engineering, in a first stage as steel framing system and later in composite solutions. In the same time, the effort of engineers and researchers to develop design methods and new technologies was important, leading to advanced solutions as meet in case of steel and concrete composite systems. The European standard EN 1998-1 as well as the Romanian in-use standard for seismic design - P100/2013, indicate similar framed systems for steel and composite steel and concrete solutions.

The Ultimate Limit State checks for design of composite framed structures are quite similar to those used for steel frames located in seismic zones distinguish two different concepts:

- ductile structures;
- structures isolated from seismic loads (base isolated structures).

In the second alternative the structure is conceived in order to avoid the plastic behaviour by using special devices that absorb the seismic input energy and change the fundamental vibration period to values favourable to the structural system.

The first alternative, which represents the usual design way, leads to the use of dissipative systems. Unlike non-dissipative structures which withstand seismic motions by elastic behaviour, the dissipative structures are conceived and designed in such a way that the seismic input energy to be dissipated by plastifications of certain zones, called dissipative zones. These zones dissipate the kinetic energy induced by the seismic motion by a hysteretic behaviour in plastic domain. The development of dissipation mechanisms depends on the structural configuration. On the other hand, the structural non-dissipative elements must be designed in such a way that they will remain in elastic domain. In consequence, they should possess an overstrength in order to resist the maximum efforts transmitted by dissipative elements.

The main types of dissipative frames can be classified in function of the type and nature of dissipative zones. Three main typologies can be named:

- centric braced frames, as shown in Figure 76 a), b), d);
- eccentrically braced frames, example Figure 76 c);
- moment resisting frames, as shown in Figure 76 e).

The dissipative zones of centric braced frames – Figure 76 a), b), d) – is through tensile diagonals, while the compressed diagonals are prone to buckling. However, the dissipative performances of the braces are limited due to their repeated buckling which lead to a certain degradation of cyclic behaviour with the number of plastic cycles. On the other hand, the contribution of compressed braces is diminished with the increase of the slenderness of the element.

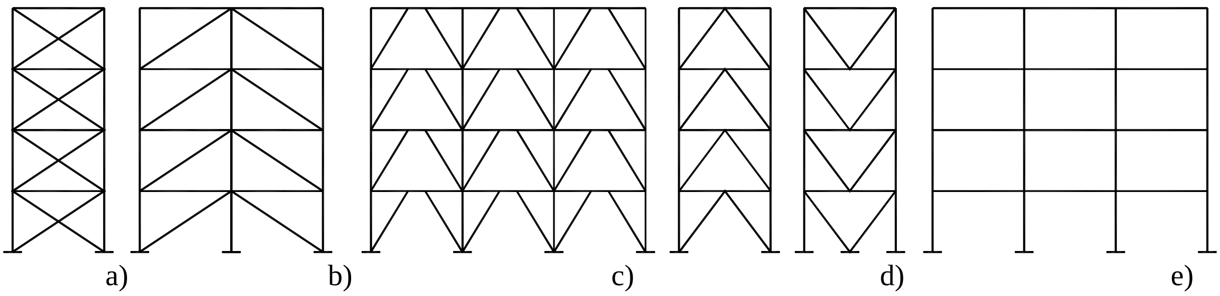


Figure 76. Usual configurations of braced (a-d) and moment resisting frames (e).

The steel and composite steel and concrete structures with eccentric braces represent an interesting alternative to the structural system with centric braces. They are stiffened by the eccentric braces. In this system, each beam is divided in two or more parts, which work differently in case of seismic action. The short part, also called “link”, represents the dissipative element of the beam. In function of the length of this element, the seismic input energy is dissipated through shear elasto-plastic cycles for short links, in bending for long links and respectively shear and bending for intermediate-length links.

The usual steel and composite frame systems with eccentric braces present different dispositions of braces. There can exist “D” type disposition of eccentric braces with a single brace on storey and the link located in one extremity of the beam, “K” type disposition with the link disposed in the central part of the beam as shown in Figure 76 c) and “V” type bracings with links in each extremity of the beam. According to modern seismic norms, the dissipation capabilities of eccentrically braced frames are similar to those of MRFs: both can be employed in design with large values of q-factors, of the order of 6 or even higher.

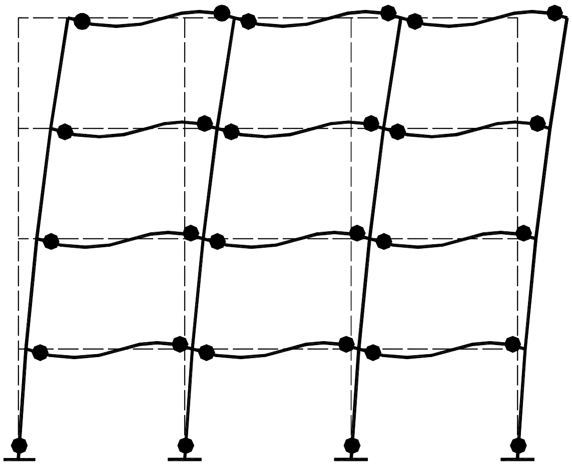


Figure 77. Global MRF failure mechanism by plastic hinges.

The MRF steel and composite systems are used on a large scale for low and moderate-rise buildings. They are able to offer a sufficient energy dissipation capacity due to the large number of dissipative zones. However, in case of higher MRF systems it is hard to make the ULS criteria consistent with the serviceability conditions expressed generally by the limitations within the inter-storey drifts. This is due to the reduction of lateral stiffness with height, although the number of dissipative zones is higher.

The dissipative zones of MRF are characterised by the plastic hinges formation in member elements, first in beam ends and in limiting cases in columns. The most important demands for plastic hinges are in bending and for this reason the dissipation of energy implies a good cyclic hysteretic behaviour with high rotations. In order to globally maximize the capacity of energy dissipation, the design of the structure should conceive a global failure mechanism which allows a maximisation of the plastic energy dissipated as shown in Figure 77.

2.4.1. Seismic behaviour of MRF considering actual response of joints

Usually, the FE modelling of steel Moment Resisting Frames includes the beam and column modelling, the joints being traditionally modelled as pinned or full-strength and rigid. The recent strong earthquakes revealed a series of undesirable failure modes of beam-to-column welded joints of Moment Resisting Frames (MRF). These zones represent the key-points for a ductile seismic response. That is why the joint behaviour needs a particular attention in the usual design of MRF. The extended end-plate bolted connections are traditionally popular in Europe for steel constructions and may be easily adapted to composite systems. However, in many cases of composite MRF, the joints proved to be in fact partial-strength and/or semi-rigid, due to the increased beam resistance. The recent evolution of steel and composite seismic codes relies on the laboratory tests to be sure of suitable behaviour of beam-to-column joints (Eurocode 8). Unless they are rigid and full strength, their behaviour should be integrated by means of appropriate elements (for instance rotational springs or short length finite elements) into the structural design analysis.

The study is concentrated on the results of dynamic simulations performed on two regular 2D of two and four storey frames. In case of beams and columns, refined bar finite elements with fibres are used. The beam-to column joint behaviour is integrated into the structural modelling by means of a sophisticated finite element which models with good accuracy the experimental behaviour of joints. The frames were analysed under severe Romanian Vrancea accelerogram and also two artificial accelerograms by non-elastic dynamic analyses, using the DRAIN 2DX (Prakash et al.) computer code where the above mentioned finite elements are included.

The four-bay composite Moment Resisting Frames on two-storey and four-storey respectively, are shown in Figure 78. The cross-sectional dimensions of the steel columns and composite beams were deduced from an equivalent push-over static analysis, in accordance with EN 1994-1 and P100/2004. The following assumptions were considered for the design:

- a_G=0.35g – the design ground acceleration;
- k_s=1.15 – the soil parameter (soil type C);
- G_k= 31.4 kN/m – the characteristic value of the permanent load;
- Q_k= 12.0 kN/m – the characteristic value of the live load (reduced by a combination coefficient ψ_E = 0.3);
- full shear connection between the steel profile and the concrete slab;
- full-strength and rigid beam-to column joints;
- behaviour factor q=6 (for MRF);
- limit value of the elasto-plastic inter-storey drift for Ultimate Limit State: $d_r \leq 0.02$ h, (h is the storey height).

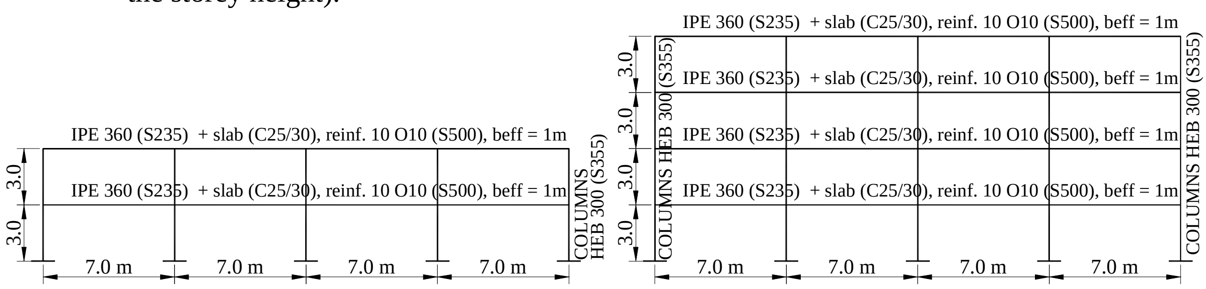


Figure 78. Composite frames layout: Frame 1 (left); Frame 2 (right).

The dimensions of the cross-sections (given in Figure 78) resulted from the elastic static equivalent analysis, as follows:

- in the case of Frame 1, the cross-sectional dimensions were governed generally by the static design under the gravitational loads, the combination of actions for the seismic design situation being not prevailing;
- in the case of Frame 2, the static design under the gravitational loads limited the beams cross-sections, whereas the column cross-sections were governed by the combination of actions for the seismic situation because of the drift limitations (the resistance criterion was largely fulfilled).

- *FE Modelling of Structural Elements*

The above structures were modelled by the help of DRAIN 2DX computer code and subjected to elasto-plastic dynamic analyses with input accelerograms. The fibre finite element (called “element 15” in DRAIN 2DX) may provide a rather refined behaviour in comparison with traditional bilinear elastic-perfectly plastic elements. Obviously, the accuracy of the response depends on the discretisation adopted for the bar length and on the number of fibres considered. Also, the input stress-strain curve for each fibre may affect the response. As the consequence of step-by-step calculation, plastic zones may develop within the element length and cross-section depth.

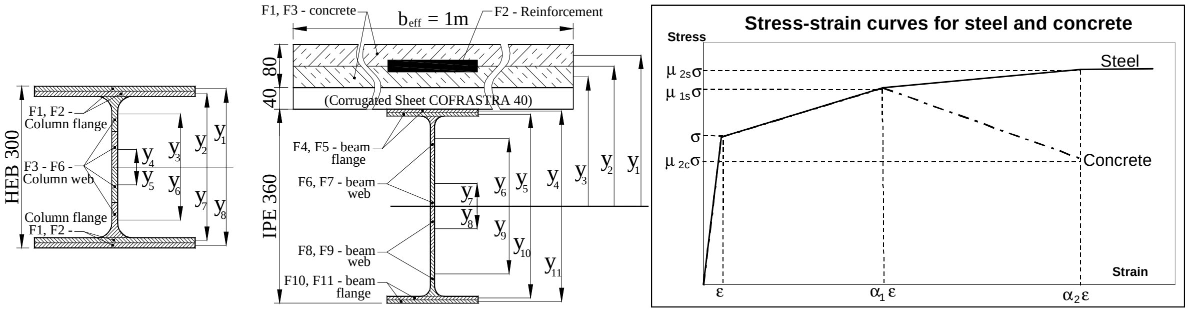


Figure 79. Distribution of fibres for bar elements (column and composite beams) and stress-strain curves for steel and concrete.

Eight and eleven fibres have been used in the case of column and beam elements respectively, as shown in Figure 79. The fibre stress-strain characteristics have been deduced from tensile tests performed on steel samples (beam and column flanges and webs, plus reinforcement) and from compressive tests on concrete cylinders. A general stress-strain diagram may be adopted for both steel and concrete, as shown in Figure 79, provided that appropriate values of elastic limit stress σ, associated strain ε and coefficients (α_i, μ_i) given in Table 1 are used. In fact these values result from direct measures on actual materials, in particular the structural steel grades S235 and S355 for beams and columns respectively, and the steel grade S500 for reinforcing bars. The structural steel is assumed to have the same properties in tension and in compression whereas the tensile resistance of concrete is neglected.

Table 10. Mechanical characteristics of materials for beams and columns

Parameter	Column		Beam		Reinforcement	Concrete
	Flange	Web	Flange	Web		(compression)
ε [μ strain]	2380	1943	1252	1390	3148	1390
σ [N/mm²]	500	408	263	292	661	26
μ_1	1.14	1.17	1.39	1.34	1.15	1.28
α_1	33.6	41.2	63.9	57.6	12.71	1.98
μ_2	1.16	1.23	1.54	1.46	1.22	0.896
α_2	46.9	73.6	153.5	121	30.13	2.88

The elasto-plastic behaviour of the beams and columns in terms of moment-rotation curves is illustrated in Figure 80, in comparison with the bi-linear elastic-perfectly plastic behaviour computed according to Eurocode analytical models, considering the characteristic strength of materials. It should be underlined that the rotation set in horizontal axis corresponds to the integration of the elasto-plastic curvature within the actual extended plastic zone.

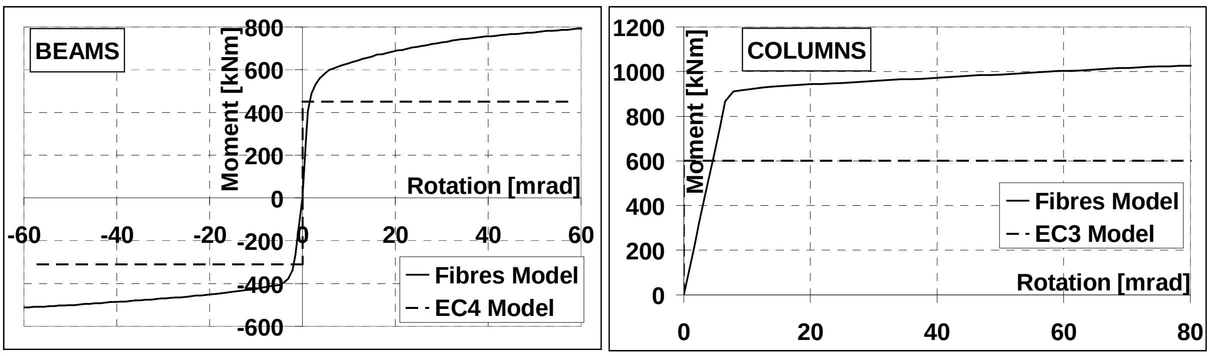


Figure 80. Resulting moment-rotation curves of beams and columns.

- *Beam-to-Column Joints Model*

Generally the modelling of the behaviour of the beam-to-column joints into the structural analysis is a difficult task, because it depends on both the joint typology and material properties. For the presented simulations, a complex “zero-length” finite-element model (Skuber, 1998) is calibrated starting from the actual behaviour of an existing laboratory test (specimen G15). The calibration model of the joint could be found elsewhere (Ciutina, 2003). The experimental test was performed in Laboratory of Structures, at INSA-Rennes, under unsymmetrical cyclic loading, in a series of a testing program (Lachal et al., 2004). The joint consisted in an extended bolted end-plate for the steel part, and had the following characteristics:

- column: HEB 300 (S355);
- steel beam: IPE 360 (S235);
- concrete of class C 25/30;
- composite slab with steel sheeting COFRASTRA 40;
- reinforcement: 10 Φ 10 (S500);
- end-plate: t=15 mm (S235) with 6 high-strength bolts of grade 10.9 and of 22mm diameter used with controlled tightening.

Due to welding deficiency, the failure occurred by cracking of the butt weld between beam flange and end-plate under hogging moment, leading to a clear unsymmetrical moment-rotation diagram of the joint.

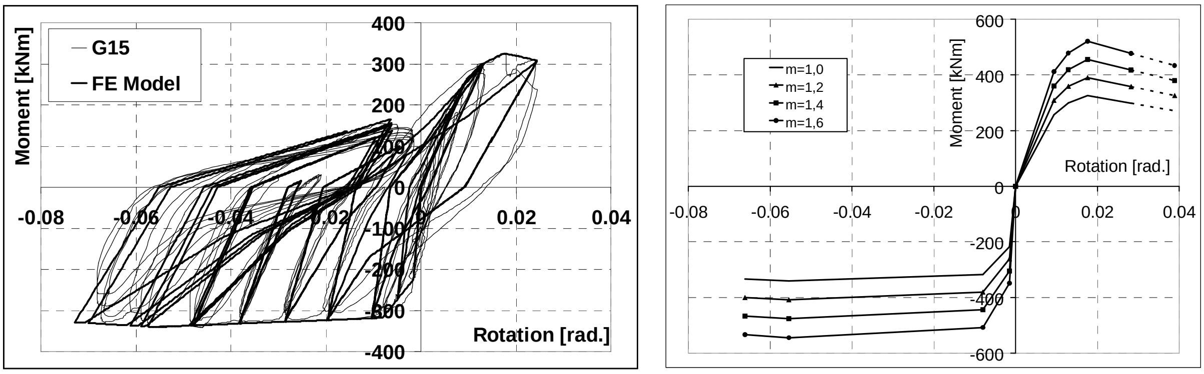


Figure 81. F.E.M. calibration for G15 test and envelope behaviour for different values of m.

The comparison of the modelled curve for the structural analysis to the actual joint behaviour is shown in Figure 81. The model represents a quadri-linear envelope behaviour including a discharging branch. Special factors for cyclic evolution are taken into account, such as: (i) the pinching effect; (ii) the stiffness degradation of unloading branch; (iii) the loss of resistance of repeated cycles having the same rotation range. In addition, the model characteristics are different under sagging and hogging bending moments, being able to simulate the unsymmetrical behaviour. The comparison of the envelope curve of Figure 81 (left) with the beam response curve of Figure 80 shows clearly that the actual joint is partial-strength, with resistance ratios of about 0.6 and 0.8 under sagging and hogging bending moments respectively.

A parameter used in the analyses was the resistance degree of the joints, taken into account by the m parameter (see Figure 81 – right). This parameter multiplies the ordinate of the initial curve (m=1). The values of m considered in the analyses were 1.0, 1.2, 1.4 and 1.6.

- *Accelerograms and response parameters*

Dynamic time-history analyses were performed using the Vrancea 1977 accelerogram (recorded at INCERC Bucharest N-S). The accelerogram and its exact response spectra are shown in Figure 82. For the dynamic analyses, a multiplier λ will be applied in order to scale the acceleration to the required levels of seismic intensity. The accelerogram was scaled by Effective Peak Acceleration (EPA) as this factor was considered as the normalizing factor. As example, in order to scale the accelerogram at a seismic intensity of 0.32g, a factor of 1.33 was applied (EPA for Vrancea accelerogram is 0.24g).

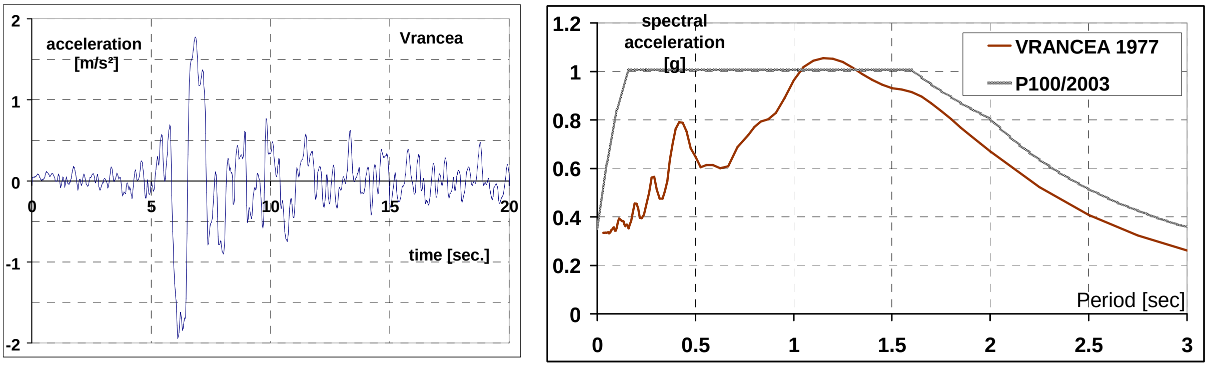


Figure 82. Vrancea accelerogram (left) and its elastic response spectra (right).

As a comparison to above long-period accelerogram two artificial accelerograms determined in such a way that they correspond approximately to EN 1998-1 – soil C spectrum were used. The exact response spectra for these accelerograms are shown in Figure 7. The Romanian territory was divided from the seismic point of view into seven seismic zones for a return period of 100 years. For the analyses, the two structures were hypothetically placed in three seismic zones, denoted by zone A (seismic intensity 0.32g), zone D (seismic intensity 0.20g) and zone F (seismic intensity 0.12g).

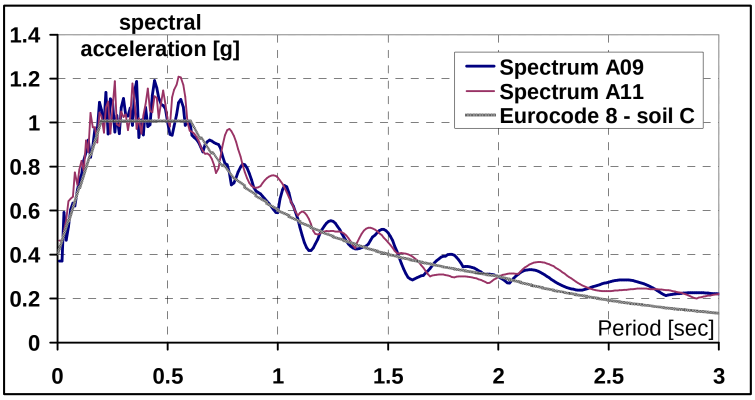


Figure 83. Response spectra for the artificial accelerogams (5% damping).

A first parameter which was varied was the resistance degree of beam-to-column joints as explained in the previous section. An ideal case in which the beam-to-column joints are infinitely rigid and totally resistant was also considered. For all the three seismic zones, the following parameters have been monitored:

- the required elasto-plastic rotations for all the structural elements. These values have been integrated for several segments in the case of fibre elements, respectively monitored in the case of beam-to column joints as a direct result. These values were compared to the values requested by the EN 1998-1, chapter 7, i.e. 35 mrad for the case of Ductility Class High (DCH) and 25 mrad for Ductility Class Medium (DCM), values adopted also in the Romanian seismic norm P100/2004.
- the Inter-storey Drift (ID) values, monitored at each level. They have been computed as a ratio of the relative lateral displacement of the storey and the storey height (expressed in %): (di – di-1)/hstorey. The ID values could also give information about the structural degradation after an earthquake. The valued of the inter-storey drifts were compared to the limiting value of 0.02h (60mm), required by the P100/2004 norm in the case of the ultimate limit state.
- the output behaviour factor (called also the q factor) and the performance factor (η) of the structures, as described in the corresponding section.

- *Numerical Results*

Figure 84 gives information about the structural degradation state in case of the Vrancea accelerogram scaled in order to correspond to the seismic zone A (λ=1.33). There are illustrated the maximum transitory values of (positive and negative) elasto-plastic rotations and inter-storey drifts. The response values (elasto-plastic rotations and ID) of the lower structure – Frame 1 show that even a strong earthquake does not affect much the structure. The joints rotation values of 5 mrad. and ID values under 0.9% could be considered at the limit of the Immediate Occupancy performance criteria according to American Standard FEMA356 (FEMA, 2000) or the Serviceability Limit State according to P100/2004. On the other hand, the higher structure – Frame 2 presents high values of elasto-plastic rotations (of order of 47 mrad – in the joints) and ID values greater than 5%. This means that the frame is far beyond the limit of use, and according to above-mentioned standard it even passes the Collapse Prevention performance criteria.

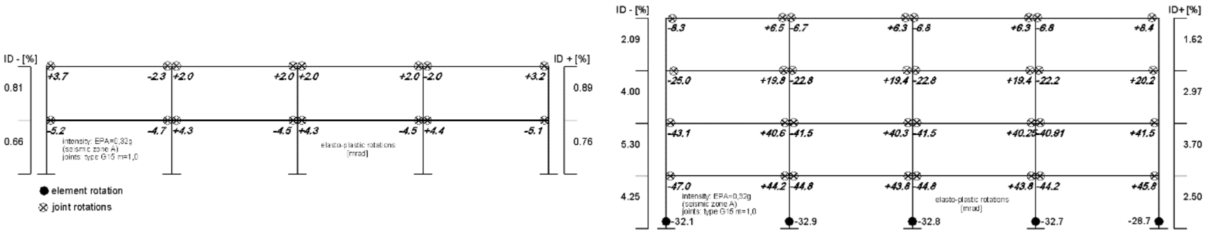


Figure 84. Structural response of frame 1 and 2 for zone A (0.32g) – Vrancea accelerogram.

Nevertheless the higher structures are more affected by an earthquake than the lower ones, but for the studied cases another parameter seems to influence their response, namely the vibration period (1st mode) in regard to the elastic spectra of the accelerogram. The ratio of the elastic spectrum corresponding to the vibration period of the frame 2 (T=0.92s) and the one corresponding to the frame 1 (T=0.47s) is about 1.5 (see Figure 82) fact that explain in part the differences between the two responses in case of Vrancea accelerogram.

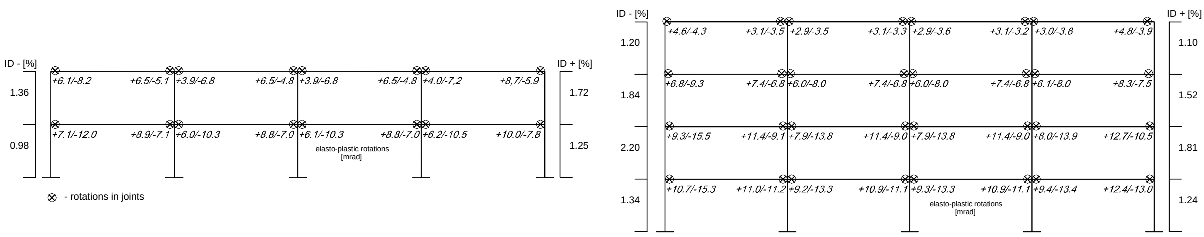


Figure 85. Structural response of Frame 1 (A09 accelerogram) and Frame 2 (A11 accelerogram) for zone A (0.32g).

On the other hand, in the case of the two artificial accelerograms (A09 and A11) the plastic dissipation is more uniformly distributed for both structures in comparison to the case of Vrancea accelerogram – see Figure 85. Although the rotation requirements are moderate under the most severe accelerogram, (here A11), their values indicate however that the joints are plastified:

- 12.8 mrad for frame 1;
- 15.5 mrad for frame 2.

Figure 86 presents globally the maximum values of elasto-plastic rotation and of Inter-storey Drifts for the two structures, separately for zones A, D and F (Vrancea accelerogram). The P100/2004 Ultimate Limit State (ULS) limitation criteria (see paragraph 3.2) have been also figured on the charts.

For Frame 1 the required elasto-plastic rotations remains far under the limits requested by the actual seismic norm, fact that indicate that a q factor not grater than 4 and members of class 2 and 3 corresponding to Ductility Class Medium could be used in design. In the case of the frame 2, the joint required rotations show that Ductility Class High is mandatory at least for zones A to D. The maximum values of the transitory Inter-Storey drifts presented in Figure 86, show also that ULS limitation of 2% criterion is exceeded for the zones A to D for Frame 2, meaning that the deflections given by an equivalent elastic design using perfect rigid and full-strength joints is not safe in regard to the use of partial-resistant and semi-rigid joints.

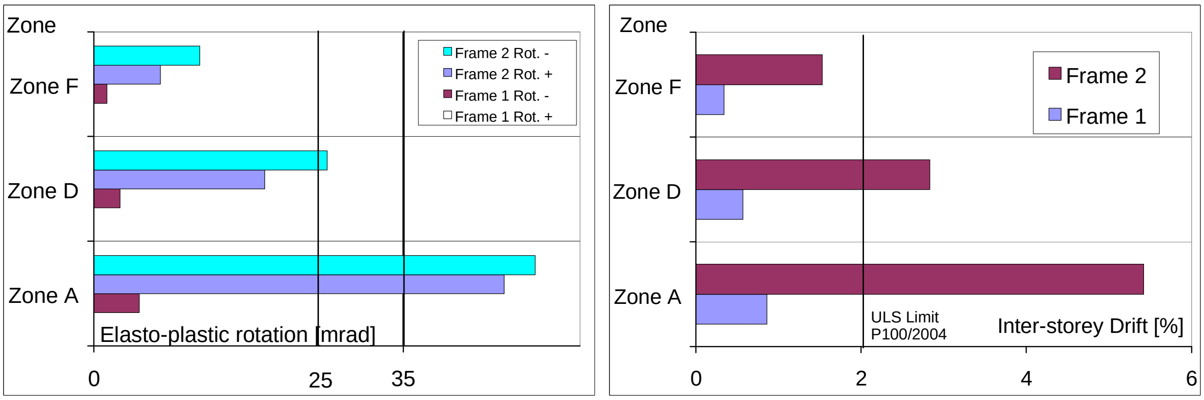


Figure 86. Variation of required elasto-plastic rotation (left) and maximum inter-storey drift (right) in respect to the seismic zone (Vrancea accelerogram).

Figure 87 shows the influence of the response parameters in function of the joint type (accounted by the m factor) in the case of the seismic zone A – Vrancea accelerogram. Generally the increase of m factor has a beneficial influence on the required rotation (joints or elements) as well as on the maximum inter-storey drift, and in this trend the structure having Ideal Rigid (IR) joints presents the lower values of response parameters. Especially in the case of 2nd Frame the values of the elasto-plastic rotation decrease by important amounts, presenting in the ideal case of structure with rigid joints a maximum required rotation of 31 mrad, value that can be easily attained by a usual I steel beam of class 1 or 2. However, this is an ideal case, while the case closest to this situation, having m = 1.6 (with the maximum rotation shared between joints and beams) presents also values of required rotations under 35 mrad.

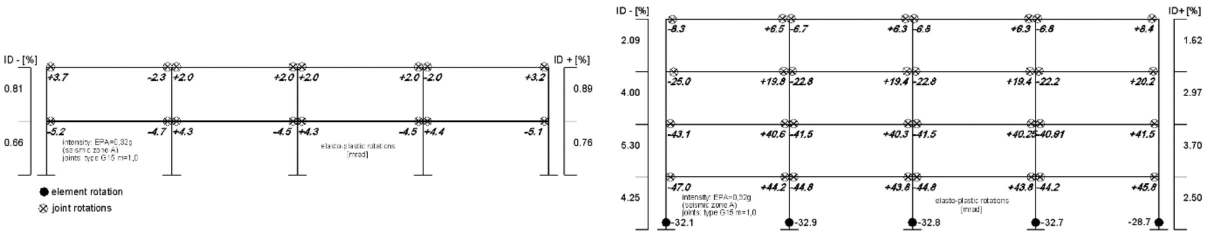


Figure 87. Variation of required elasto-plastic rotation (left) and maximum inter-storey drift (right) in respect to the seismic zone (Vrancea accelerogram)

Figure 87 also shows that for the Frame 2 the ultimate limit state requirement for Inter-storey Drift is not satisfied even in the ideal case with rigid joints. However, this case reduces by half the ID requirement of the same structure with m = 1. These values confirm however the fact that equivalent elastic design of a structure does not necessarily assure a safer response in the post-elastic domain.

The behaviour factor q characterises the plastic dissipation capacity of a structure globally, allowing a design of the structure by reduced static seismic forces. More exactly, this factor is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic, to the minimum seismic forces that may be used in design with a conventional elastic analysis. When using non-linear dynamic analyses, the q factor may be determined as the ratio:

$$q = \frac{(EPA)_u}{(EPA)_e} = \frac{\lambda_u}{\lambda_e}$$

where: - EPA_u represents the effective peak acceleration corresponding to multiplier λu for which the ultimate state of the structure is considered (for instance when the local ductility is reached into a structural element or joint);

- EPA_e represents the effective peak acceleration corresponding to multiplier λe for which the first yielding occurs into the structure.

Value λ_d = 1.00 was considered for the scaling of the accelerograms corresponding to a seismic intensity a_g=0.35g.

On the other hand, the seismic performance factor (η) represents the ability of the structure to resist a certain type of soil motion, as the ratio:

$$\eta = \frac{(EPA)_u}{(EPA)_d} = \frac{\lambda_u}{\lambda_d}$$

Where: - EPA_d represents the effective peak acceleration corresponding to design (in this case equal to the design ground acceleration a_G);

- λ_d the accelerogram multiplier corresponding to EPA_d (in this case equal to 1.00).

Table 11 gives for both frames and for all the accelerograms considered in the dynamic analyses the values of λ_e, λ_u and q considering three different criteria for the structural limit state:

- first, the maximum ground acceleration adopted for the initial design of frames;
- secondly, the attainment of the rotation capacity (under either sagging or hogging bending) in a particular joint of the frame;
- and thirdly, the attainment of the joint rotation capacity equal to 35 mrad (in accordance with EN 1998-1 criterion for structures of high ductility classes) assuming a hypothetical discharging branch under sagging bending up to this value of rotation.

Table 11. Values of λ_e, λ_u (η) and q for different limit criteria.

Structure and seismic action		λ_e	***a_g = 0.35g***		***Joint rotation capacity***		***Theoretical joint rotation capacity***	
			λ_u (η)	*q*	λ_u(η)	*q*	λ_u(η)	*q*
Frame n° 1	A09	0.35	1.00	*2.9*	2.71	*7.7*	3.53	*10.1*
	A11	0.30	1.00	*3.3*	1.70	*5.7*	2.10	*7.0*
	Vrancea	0.50	1.00	*2.0*	1.83	*3.7*	2.40	*4.8*
Frame n° 2	A09	0.26	1.00	*3.9*	1.66	*6.4*	1.94	*7.5*
	A11	0.22	1.00	*4.5*	1.46	*6.6*	1.80	*8.2*
	Vrancea	0.13	1.00	*7.7*	0.59	*4.5*	0.72	*5.5*

Concerning the first criterion, results confirm that generally, the q factor is smaller than 6 due to the fact already mentioned that the seismic action is not significant enough with regard to the frame resistances. A special case is represented by the second frame under Vrancea accelerogram, for which the first plastic rotation forms very early (λ_e = 0.13), theoretically earlier than expected by design.

Using the second criterion, the joint rotation capacity is systematically reached under sagging bending (for value of 24 mrad). It should be noted that the q factor exceeds 6 for the artificial accelerograms. This demonstrates the good dissipative performances of the composite frames which may be classified in high ductility despite the use of partial-strength joints with unsymmetrical behaviour. Nevertheless, these deformations imply the acceptance of inter-storey drifts greater than 0.02h. The limit 0.03h appears more appropriate to be consistent with a q-factor equal to 6. Here also, the long-control period accelerogram of Vrancea leads to values of q which are under the design value of 6.

Finally, if the hypothetical third criterion is considered, the q-factor would range from 7 to 10, provided that the inter-storey drift limitation were increased practically up to 0.05h, as resulted from the results of the analyses. Generally, experimental evidence demonstrates that this latter value of transient inter-storey drift cannot be exceeded without a high risk of structural collapse (FEMA 356 2000).

The values of η shows generally good abilities of structures to withstand considered ground motions, with values ranging between 1.46 and 2.71 in the case of second limit state criterion, and 1.8 to 3.5 in case of third limit state criterion. Of course, a η value greater than 2 may indicate a non-economic design of the structure, but this happens only for the first frame, for which the design was done mainly due to gravitational loads and not due to seismic combination. As for the q factor, the only exception to above mentions is represented by the first frame under Vrancea accelerogram. In this case η takes values smaller than 1, meaning that the structure is not able to overtake the seismic load to which it was designed. Under the circumstances of using partial-resistant and semi-rigid joints, this fact illustrates, very clear that a design made by means of equivalent push-over static analysis is not necessarily safe with regard to the response of the structure under dynamic time-history analyses.

- *Behaviour of MRF considering actual response of joints - conclusions*

As shown by the previous results, the analysed frames proved a good behaviour under the artificial accelerograms, complying with the EN 1998-1 – soil C spectrum, although the joints were semi-rigid and partial-strength. Despite this good behaviour, the same frames behaved differently under the Vrancea accelerogram, characterised by a higher control period.

Table 12. Performance criteria of analysed frames (Vrancea accelerogram).

Limitation criterion		Joints ⇩ / Zone ⇨	Frame 1 F	Frame 1 D	Frame 1 A	Frame 2 F	Frame 2 D	Frame 2 A
ULS Rotation criterion	P100/2004 (EN 1998-1) HDC $\varphi_{\max} \leq 35 mrad$	Ideal Rigid	✓	✓	✓	✓	✓	✓
		m = 1,6	✓	✓	✓	✓	✓	✓
		m = 1,4	✓	✓	✓	✓	✓	X
		m = 1,2	✓	✓	✓	✓	✓	X
		m = 1,0	✓	✓	✓	✓	✓	X
	P100/2004 (EN 1998-1) MDC $\varphi_{\max} \leq 25 mrad$	Ideal Rigid	✓	✓	✓	✓	✓	X
		m = 1,6	✓	✓	✓	✓	✓	X
		m = 1,4	✓	✓	✓	✓	✓	X
		m = 1,2	✓	✓	✓	✓	✓	X
		m = 1,0	✓	✓	✓	✓	X	X
ULS Deformation criterion (Inter-storey Drift ID)	P100/2004 $ID < 2.0\%$	Ideal Rigid	✓	✓	✓	✓	✓	X
		m = 1,6	✓	✓	✓	✓	X	X
		m = 1,4	✓	✓	✓	✓	X	X
		m = 1,2	✓	✓	✓	✓	X	X
		m = 1,0	✓	✓	✓	✓	X	X

Notation: ✓ - criterion satisfied; X – criterion not satisfied

Table 12 gathers the performance criteria for the analysed frames function of the seismic zone and joints resistance in regard to lateral deformation and rotational requirements of the P100/2004 norm in case of Vrancea accelerogram. The following conclusions could be drawn, limited at the element conditions used in this study:

- the degradations in terms of both required elasto-plastic rotation and inter-storey drift, recorded for lower structure (frame 1) are low, even in the case of high seismicity zones, such as the Romanian Zone A. For this case, the values of inter-storey drift remains at the level of serviceability limit state, even using partial-resistant beam-to column joints.
- in case of medium height structures, as is the case of frame 2, the use of partial-resistance joints should be limited to low seismic zones (zones E-F). High values of q-factor (between 4 and 6 according to P100/2004 and EN 1998-1) should be used, in order to have a guaranteed value of the elasto-plastic rotation of the plastic zone of more than 35 mrad.

- in regard to the inter-storey drift criterion, the results show that an equivalent elastic design of a structure does not necessarily assure a safer response in the post-elastic domain. As demonstrated also in above, the partial-resistant joints could influence by large amounts the lateral displacement of a structure, especially at the ultimate state.

The investigation on the q factor shows that the high values of q specified for the composite frames in EN 1998-1 cannot be adopted without considering the inter-storey drift limitation in the sense of larger tolerance. In fact, the use of partial-strength joints does not seem to constitute a handicap, contrary to the unfavourable feeling of usual designers in seismic zones.

In addition, the use of actual joints may lead to a more uniform distribution of the dissipated energy, without requiring a large rotation capacity to classify the structure in high ductility. In conclusion, the treated examples have proved that a rotation capacity of about 25 mrad under sagging bending may be sufficient for moderate seismic zones to reach a q-factor of about 6 while accepting an inter-storey drift of about 0.03h.

2.4.2. Seismic behaviour of dual Moment Resisting + Eccentrically Braced Frames

The modern seismic design codes such as EN1998-1-1 (2003) and P100/1 (2006) allow the Incremental Dynamic Analysis (IDA) as alternative to the method of equivalent lateral forces. However, the IDA methodology applied to steel and composite steel - concrete frame typology assumes the development of plastic hinges in dissipative zones. Various researchers have investigated the response of steel EBFs and dual MRF+EBF frames but in most of the cases disregarding the influence of concrete slab over the overall structural performance. Chao et Goel (2006) propose a method of designing the EBF steel elements based on target displacement using the Performance-Based Seismic Design. The numerical investigation conducted by Özhendekci and Özhendekci (2008) show the difference in elastic and inelastic lateral force distribution in EBFs. The study shows as well the fact that in many cases the plasticization of EBF links is concentrated on a limited number of storeys. Bosco and Rossi (2009) present an interesting numerical work by considering the steel link modelled by three resorts. The authors define a new parameter, called damage distribution capacity factor and use it to characterise the seismic response of EBF structures. Degee et al. (2010) present a parametrical numerical study concentrated on EBFs with vertical short steel links working in shear and composite beams by using nominal material characteristics. Although the main aim of the study is concentrated on the influence of material variability on seismic response of structures, partial conclusions show the good behaviour of the vertical steel links conducting to behaviour factors close to 6 while for design a smaller value was used.

Preliminary tests performed on steel dual MR+EBF (Ricles 1987, Danku 2011) having the steel beams connected to concrete slab (composite beams) show that the composite effect has a favourable influence on the resistance of the plastic hinge while maintaining the high ductility of the systems. Also, the disconnection of steel element from the concrete slab does not assure a similar behaviour to that of steel element alone, but is more close to full-connection situation.

The plastic hinges act as fuses in dissipating seismic input energy through elastic-plastic cycles in bending (for MRF) and shear (for EBF). In current practice, the model defining plastic hinges involves only the use of the steel section to define the plastic behaviour. When considering steel beams composite with the concrete slab, the general tendency is to avoid the connection over the dissipative zone, thus allowing the plastic hinge to fully develop in the steel profile.

A common solution for concentrating the plastic hinge in a desired position is by using Reduced Beam Section (RBS), usually on the beam, in the vicinity of the beam-to-column joint (Plumier, 1990). It implies the reduction of beam section by flange cutting. The procedure is also applied to prevent premature failure of the joint or column panel. The experimental results obtained on dissipative zones of MRF and EBF and described in section 2.3 is used as basis for the calibration of plastic hinge behaviour in such a way to consider the composite effect on the plastic behaviour of the element. These will be further used in nonlinear analyses of time-history and pushover types on frames.

The computer code SAP2000 (CSI 2012) was considered as a rational design tool for required analysis. A parameter in the choice of the analysis software was the large distribution among the steel design engineering community. The hinge definition module developed in version 14 in SAP2000 computer code allows for the definition of various plastic hinge properties, such as:

- M3 bending hinge usually assigned to elements (e.g. beams) - user specifications allowed;
- V2 shear hinge – assigns a hinge to an element working in shear (e.g. short links), - user specifications allowed;
- P axial hinge type can be applied to axially-loaded elements, such as braces. Different hinge properties may be defined in tension and compression;
- P-M2-M3 – axial and biaxial bending moment hinge, used to model the plastic behaviour of columns.

These models are mostly used for steel sections and are symmetric in behaviour by implicit definition (see Figure 88). The model for the plastic hinge includes only the definition of the plastic curve through rigid-plastic behaviour, while the elastic behaviour is integrated in the element model. Different key-points can define plastic deformations related to Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) limit states. However, the implementation and use of a plastic hinge model for composite cross-sections, requires good knowledge on the elasto-plastic behaviour of the structural elements and joints including the influence of alternate loads.

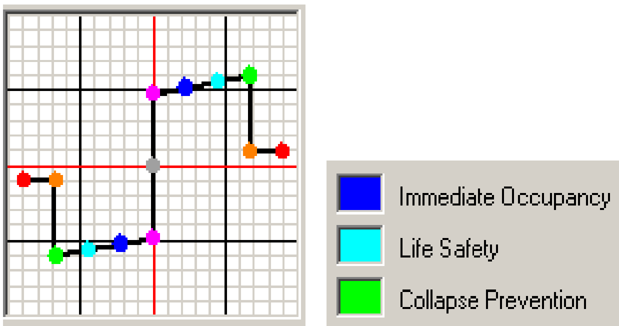


Figure 88. Example of default plastic hinge definition in SAP2000 (SAP 2000, 2012).

A realistic plastic behaviour of a hinge is obtained through definition of the key points that follow the behaviour resulted from experimental envelope curves. Thus the three-linear curves with descending branch shown in Figure 90 were derived on the basis of the experimental envelope curves obtained for bending and shear elements. The experimental tests were concentrated on the behaviour of dissipative zones of dual MRF+EBF (see Figure 90 – left). Testing set-up of beam-to-column and respectively EBF specimens is shown in Figure 90 – middle and right respectively.

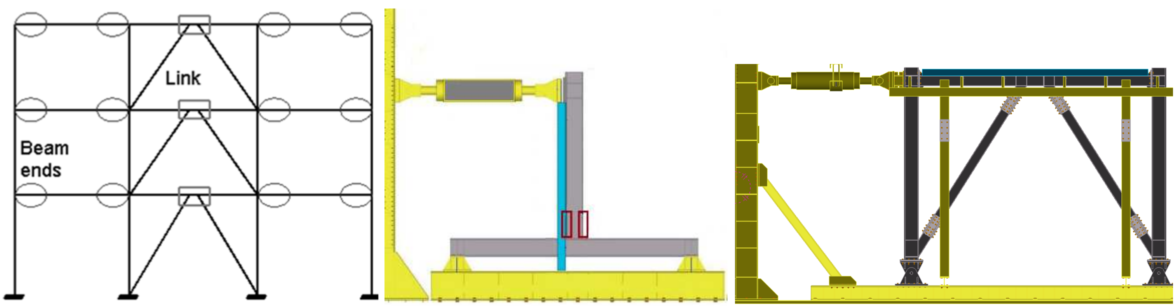


Figure 89. Dissipative zones for MRF+EBF dual configuration (left) and testing set-up for DB and EBF specimens (right).

The experimental results show a significant difference between steel and composite specimens (connected or not over the hinge) for all the monitored parameters: resistance, initial rigidity and ductility. In consequence, the plastic-hinges defined for the composite cross-sections should be asymmetrical with distinct behaviours for positive and negative branches. Figure 90 shows graphically the definition of the key points chosen for the plastic hinges characteristic curves for RBS (moment – rotation) and short EBF links (shear force – distortion) plastic hinges respectively with respect to FEMA recommendations for IO, LS and CP limit states (FEMA 2000). In both link and RBS cases the full-interaction of composite behaviour of the beams was modelled. The quadric-linear numerical models shown in Figure 90 were chosen to follow very closely the experimental results.

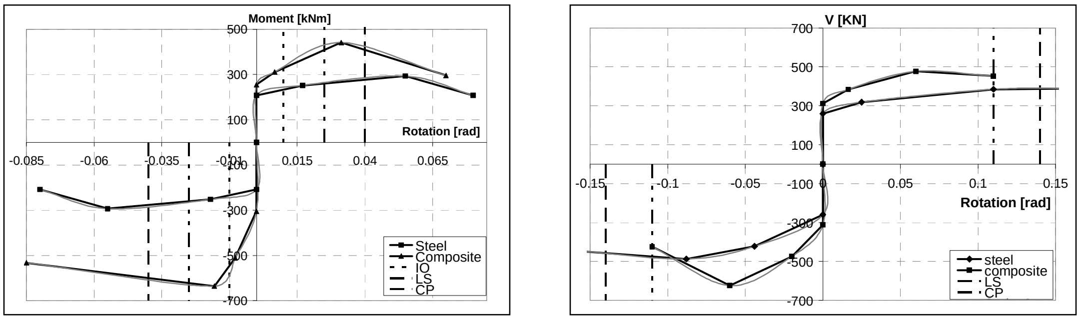


Figure 90. Plastic hinges developed in the RBS (left) and link (right) for steel and composite specimens.

Considering the characteristics for the steel and composite beam and link hinges, the key-points definitions could be made in a normalized manner (see Figure 91), taking as unity the plastic resistance of steel elements: in bending for RBS of beams and respectively in shear for short links.

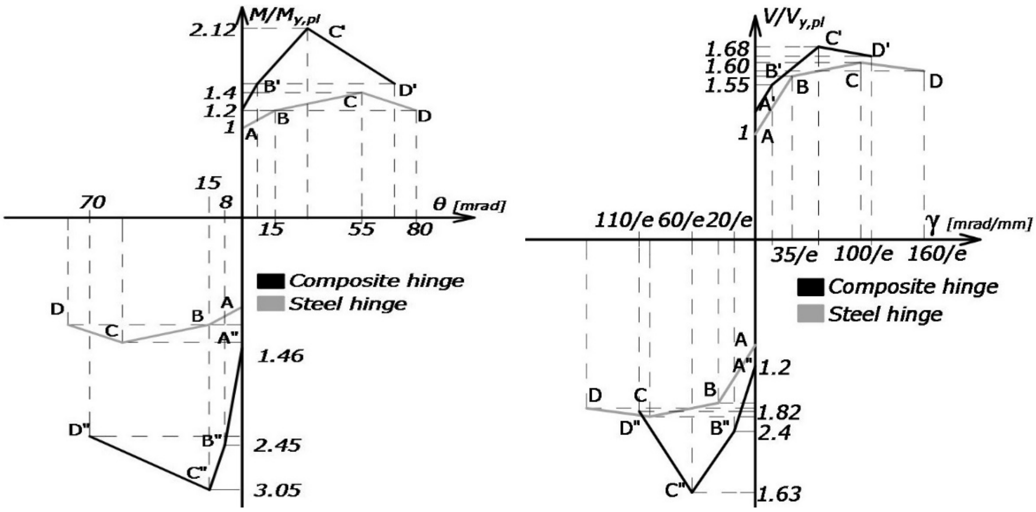
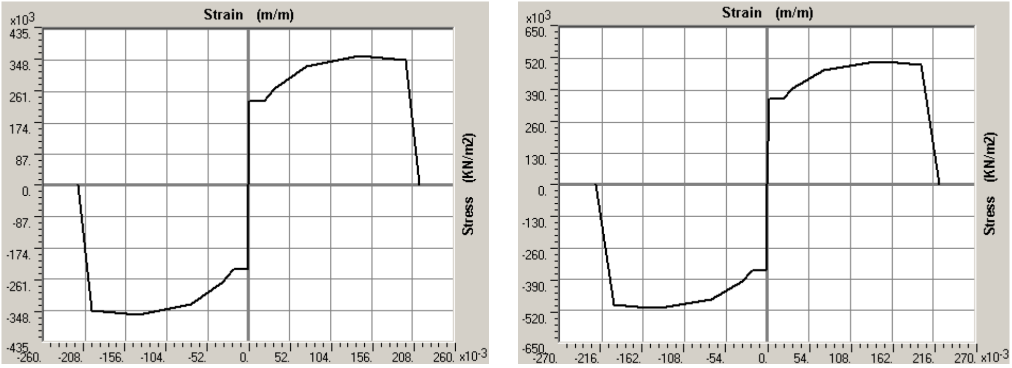


Figure 91. Normalized definition of moment and shear hinges.

The validation of hinge behaviour was performed by integration of hinge definitions on steel subassemblies simulating the tested specimens (presented in Figure 89): experimental EBF (EBF_M_LF-M) and beam-to-column joint (DB-C_RLD specimen) respectively. In order to obtain accurate models, the real stress-strain material definitions were used for elements, as they resulted from traction tests on samples. Multi-linear elastic-plastic material models were used having the nominal characteristics given in 0.



Steel grade S235 - isotropic
E=210000 N/mm^2
f_y=235 N/mm^2
f_u=360 N/mm^2

Steel grade S355 - isotropic
E=210000 N/mm2
fy=355 N/mm2
fu=510 N/mm2

Figure 92. Material definition for steel grades S235 (beams) and S355 (columns and braces).

The lateral force versus top displacement resulted from numerical simulations show good agreement with the experimental response for both joints (DB specimens) and link (EBF specimens) as shown in Figure 93. However, in case of EBF simulations it was necessary to replicate the slip in the brace connections through elastic link elements for obtaining a good accuracy in the initial rigidity of the frame. This considered a linear relationship between force and displacement, 5mm slip corresponding to an axial load of 800kN.

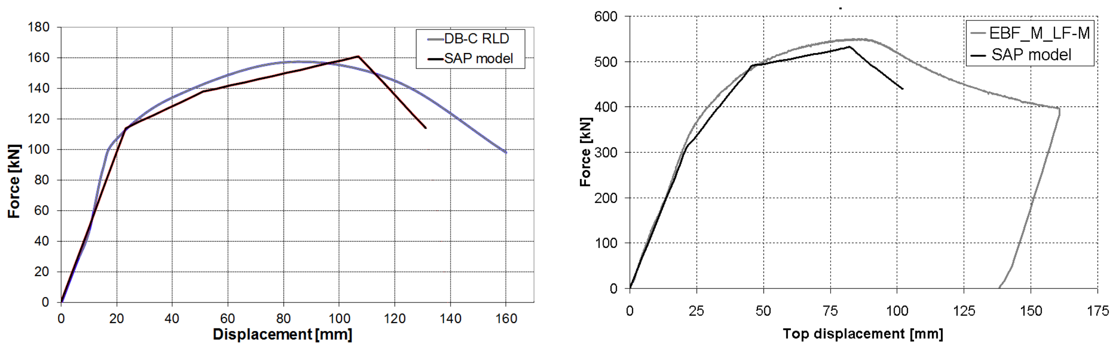


Figure 93. Experimental versus numerical lateral force – top displacement relationships for DB and EBF specimens respectively.

- *Calibration of composite response for beams*

The numerical response of composite steel and concrete beams represents a delicate matter due to the difficulty of assessing differences in material properties, both in elastic and plastic range – rigidity, resistance and deformation to failure / crushing etc. The frame modeling of such elements is even more complicated due to the concentration of sectional properties on longitudinal elements (Zona 2000). Elghazouli (2008) shows that a fiber model for composite beam section with discrete links simulating the connection between steel and concrete element can simulate a good composite behaviour . However, the behaviour of joints should be foreknown.

On the other hand, the research-dedicated software such as Open SEES (2012) or DRAIN 2/3DX (2012) used for modeling composite cross sections in frames analysis are using fiber models with different material characteristics for structural steel, concrete and reinforcement. Yet these programs are difficult to use in current design process.

The SAP 2000 computer software, version 12.0 allows several models for composite elements. The model chosen to simulate the composite beam has been adapted according to the one proposed at

CSI Berkeley in April 2010, and implemented firstly in SAP2000, version 12.0. This has proven to work adequately by comparing the numerical and analytical results, for simple cases of a simply-supported and a double-encased beam. The model uses a bar element for the steel profile and a shell element simulating the concrete layer, while a fixed link is used for simulating the composite connection between steel and concrete. The steel frame element and shell are drawn at the elevations of their centroids while the connection is realized at fixed intervals (see Figure 94).

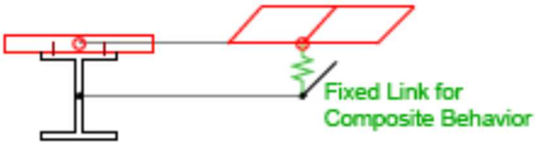


Figure 94. Numerical model used for simulation of composite beams (SAP 2000).

The calibration of composite beams behaviour was performed through push-over analysis on joint (DB_Comp_RLD) and EBF (EBF_LF2_Comp_C1) subassemblies, by comparing the experimental and numerical results. The subassemblies are in fact part of a DUAL MRF+EBF structure (see Figure 93) with spans of 4.5 m for EBF and 6 m for MRF. The concrete slab considered a thickness of 12 cm and two reinforcement layers (Φ12mm distanced at 15 cm), according to the characteristics of the experimental tests. The considered slab width for both MRF and EBF of 120 mm was greater than the one required by EN 1998-1 (2x0.1L) and En 1994-1 (2xL0/8) in case of MRF. However, this value is close to L/4 which is considered by Huang (2012) as appropriate for accurate response of composite frames.

The concrete material was defined based on the results obtained on standard compressive tests on cube samples, using the Mander model (CSI 2012) and, further adapted for the actual resistances, the model being shown in Figure 96.

The model has been adapted for satisfying the requirements needed for modelling a complete interaction in longitudinal shear between steel and concrete. For this purpose, the links have been placed at a maximum distance of 30 cm one from the other. This distance resulted as adequate in modelling the complete interaction. The shell element section was considering multi-layered non-linear elements in which the concrete is represented by the shell layer while the reinforcements are represented by two membrane layers with elastic-plastic behaviour.

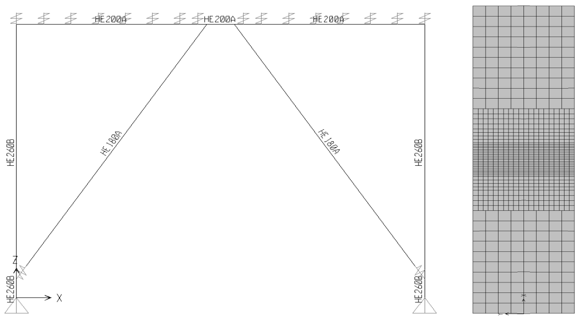
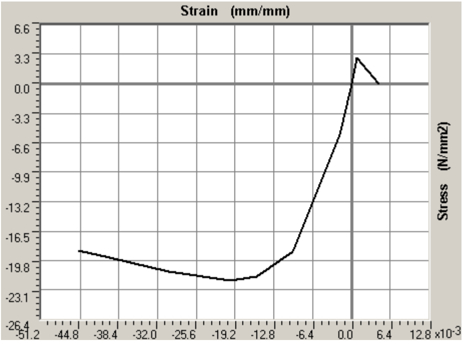


Figure 95. Numerical model used for simulation of composite beams (SAP 2000).

The concrete slab was modeled by considering two mesh sizes: a larger mesh of 15x15 cm was used for the spans, where no plasticization was expected, while in the potentially plastic region a finer mesh (2.5x6 cm) was considered (see Figure 95). The shear hinge behaviour of link was defined as shown in Figure 90 and Figure 91 to follow the experimental response. The comparison of numerical and experimental results is given under the form of force – top displacement curves in Figure 97 and proves an accurate response of numerical models for both EBF and DB specimens.



Concrete material definition
Concrete class C20/25 – isotropic
E=30000 N/mm2
f_{ck}=22 N/mm2

Figure 96. Numerical model used for simulation of concrete material in composite beams.

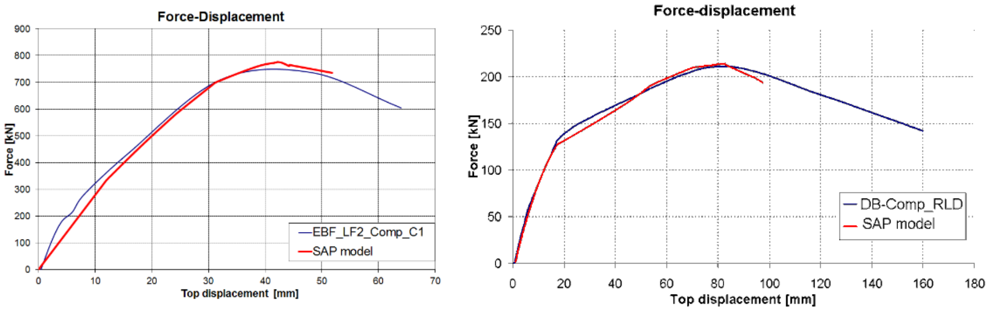


Figure 97. Experimental and numerical push-over curves obtained for EBF and DB subassemblies.

- *Case-studies, design and monitored parameters*

The numerical investigation consisted in monitoring the nonlinear response of eight 2D structures, designed according to current European seismic regulations. The structures were considered with steel columns while the beams were designed in two distinct conditions: steel and composite resulting in this way a total number of 16 structures. The systems considered for analyses were three EBF, two MRF and three dual EBF+MRF structures. The study was considering a wide variety of structures, from low-rise 4 story structures, up to 8 and 12 story configurations.

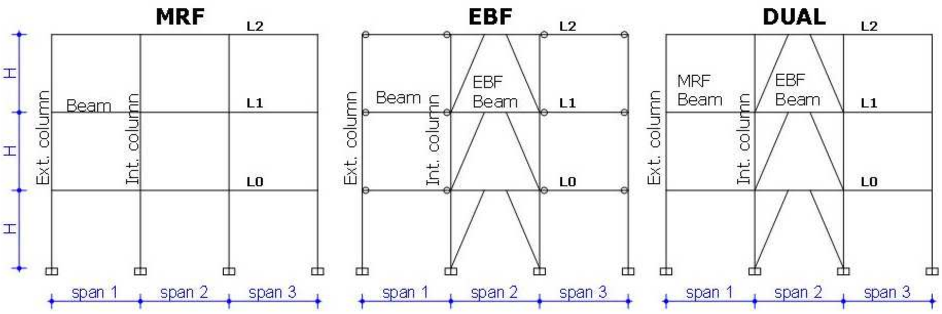


Figure 98. Lay-out and notations for frames.

The structural elements were designed according to EN 1993-1-1, EN 1994-1-1 and EN 1998-1. The gravitational loads considered, were uniformly distributed on floors: dead load: 4 kN/m^2 and live load: 3.5 kN/m^2 (including partition walls). The masses were computed according to the seismic combination 1.35G+1.5Q and concentrated in structural joints. For the seismic design, an equivalent elastic spectral analysis was considered. The spectrum considered is characteristic to the city of Bucharest, and has the control period T_c=1.6s and the peak ground acceleration a_g=0.24g. The value of the q factor was taken equal to 6 (table 6.2 from EN 1998-1) corresponding to high ductility class structures. The non-ductile elements were designed considering the values of $1.1\gamma_{ov}\Omega$ as 2.5 for EBF and dual frame and 3.0 for MRF.

The structural configurations are given in Figure 98. All the frames represent façade frames isolated from hypothetical square structures. Notations for different elements are given, in function of structural typology. The description of spans and element sections are given in Table 13.

Table 13. Main data for analysed frames

Frame number	Frame name	Nr. of levels	Level height [m]	Spans [m]	MRF Beams	EBF Beams	Composite beams	Columns	Braces
F1S	DUAL - 4S	5	4, 4x3.5	8, 6.5, 8	IPE450	IPE360	no	HE400B	L0-L1: HE200B L2-L4: HE180B
F1C	DUAL - 4C	5	4, 4x3.5	8, 6.5, 8	IPE450	IPE360	yes	HE400B	L0-L1: HE200B L2-L4: HE180B
F2S	EBF - 4S	5	4, 4x3.5	8, 6.5, 8	IPE400	L0: HE240A L1: HE260A L2: HE240A L3: HE220A L4: HE200A	no	ext:HE300B int:HE400B	L0-L3: HE200B L4: HE180B
F2C	EBF - 4C	5	4, 4x3.5	8, 6.5, 8	IPE400	L0: HE240A L1: HE260A L2: HE240A L3: HE220A L4: HE200A	yes	ext:HE300B int:HE400B	L0-L3: HE200B L4: HE180B
F3S	DUAL - 8S	9	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6-L8: HE220A	no	ext:HE400B int:HE500M	L0-L5: HE240B L6-L8: HE200B
F3C	DUAL - 8C	9	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6-L8: HE220A	yes	ext:HE400B int:HE500M	L0-L5: HE240B L6-L8: HE200B
F4S	EBF - 8S	9	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6-L8: HE220A	no	ext:HE400B int:HE500M	L0-L5: HE240B L6-L8: HE200B
F4C	EBF - 8C	9	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6-L8: HE220A	yes	ext:HE400B int:HE500M	L0-L5: HE240B L6-L8: HE200B
F5S	EBF - 6S	7	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6: HE220A	no	ext:HE360B int:HE450M	HE200B
F5C	EBF - 6C	7	3.5	7, 6, 7	HE280A	L0-L2: HE300A L3-L4: HE280A L5: HE260A L6: HE220A	yes	ext:HE360B int:HE450M	HE200B
F6S	DUAL - 12S	13	3.5	6, 6, 6	IPE400	L0-L2: IPE400 L3-L5: IPE360 L6-L8: IPE330 L9: IPE300 L10: IPE270 L11-L12:	no	HE450M / HE450B	L0-L2: HE300A L3-L5: HE260A L6: HE240A L7-L9: HE220A L10-L11: HE200A
F6C	DUAL - 12C	13	3.5	6, 6, 6	IPE400	L0-L2: IPE400 L3-L5: IPE360 L6-L8: IPE330 L9: IPE300 L10: IPE270 L11-L12:	yes	HE450M / HE450B	L0-L2: HE300A L3-L5: HE260A L6: HE240A L7-L9: HE220A L10-L11: HE200A
F7S	MRF - 5S	6	3.5	7.5, 7.5, 7.5	IPE500	-	no	HE600B / HE600M	-
F7C	MRF - 5C	6	3.5	7.5, 7.5, 7.5	IPE500	-	yes	HE600B / HE600M	-
F8S	MRF6 - 5S	6	3.5	6, 6, 6	IPE400	-	no	HE500B / HE500M	-
F8C	MRF6 - 5C	6	3.5	6, 6, 6	IPE400	-	yes	HE500B / HE500M	-

** Note: The design was considering the dissipative elements (beams and links) of the steel grade S235, while the non-ductile elements (EBF braces and columns) of S355. The only exception to that are the frames F4S/C of 8 stories for which the EBF beams were made of S355.*

The non-linear response of the structures was monitored through push-over and incremental dynamic analyses with accelerograms. The push-over analysis gives valuable information concerning the non-linear response of the structures. For the numerical study the non-linear push-over analyses were used for the confirmation of the plastic failure mechanism considered in design and also to verify it the structures can reach the target displacements for serviceability and ultimate limit conditions. The N2 method (Fajfar 2000) was used for this verification.

The Incremental Dynamic Analyses (IDA) were used for a complete characterization of the non-linear structural response. Seven earthquakes were considered, all of them recorded from Vrancea source. Details on these recordings could be found elsewhere (Danku 2011):

- One recording from Vrancea 1977 earthquake (INCERC N-S);
- Three recordings from Vrancea 1986 earthquake (INCERC N-S, EREN 10W and Magurele N-S);
- Three recordings from Vrancea 1986 earthquake (INCERC N-S, Magurele N-S and Armeneasca S3E).

The recordings were scaled initially on the design peak ground acceleration corresponding to Bucharest seismic conditions ($a_{gdesign}$=0.24g). The intensities were then increased up to the reaching of a failure point by an acceleration multiplication factor (λ). The failure (ultimate) criteria for the analysed frames were considered at the attainment of one of the following points:

- Development of a structural mechanism;
- Reaching the maximum rotation capacity in a hinge (according to FEMA356 document);
- Reaching the maximum allowable inter-story drift limit (3% was considered as the limiting criterion).

The IDA results show the realistic behaviour of structures indicating the structural response under severe ground motions and the dissipation capacity of the structure. Several parameters were monitored:

- the inter-storey drift values for different limit state conditions such as serviceability (0,12g) and ultimate (0,24g). The inter-storey drifts were expressed as percentages of storey heights;
- the plastic rotations developed in dissipative elements for different levels of ground acceleration (ultimate and serviceability). Their values were compared to norm conditions and experimental results;
- the behaviour factor, expressed as the ratio between the ground motion intensity corresponding to failure (λ_u) and the multiplier corresponding to the development of first plastic hinge (λ_1): $q=\lambda_u/\lambda_1$.
- the efficiency of the building η defining the ability of the structure to withstand a certain earthquake was computed by $\eta= a_{gu}/a_g$ (ultimate to design ground accelerations). In the conditions in which the accelerograms were scaled to a_g, the efficiency is equal to the multiplier of the accelerograms in ultimate conditions: $\eta= \lambda_u$.

The elements were modelled considering elastic-plastic behaviour of materials, and plastic hinges definitions described in second section.

- *Response of the 9 storey DUAL structure*

The 9 storey DUAL frame is considered a typical structure that combines the advantages of MRF and EBF for a convenient height. The convention of rotations shown in Figure 99 was used for representing the damages in the structural elements. The values are related to the maximum rotation reached in experimental investigations for link elements in EBF and in RBS in MRF. The red colour will indicate the results obtained by structures with composite beams, while blue will depict results obtained with steel beams.

Figure 100 presents the development of the plastic hinges resulted from the push-over analysis for different levels of top lateral displacement: 90 mm (situation corresponding to the serviceability seismic intensity); 180 mm and 230mm as displacement corresponding to ultimate limit state conditions. As it could be noted, first plasticization occurs in the lower-level links while the MRF beam ends (RBS) are plastifying for higher displacements in RBS zones (hogging bending only).

Link hinge	Rotation [mrad]				Max. Value
Pattern					166 mrad
Steel	0-40	40-110	110-150	>150	
Pattern					115 mrad
Composi	0-15	15-60	60-110	>110	

RBS hinge	Rotation [mrad]				Max. Value
Pattern					85 mrad
Steel	0-15	15-55	55-80	>80	
Pattern					80 mrad
Composite	0-5	5-30	30-70	>70	

Figure 99. Levels of response rotation for links and RBS.

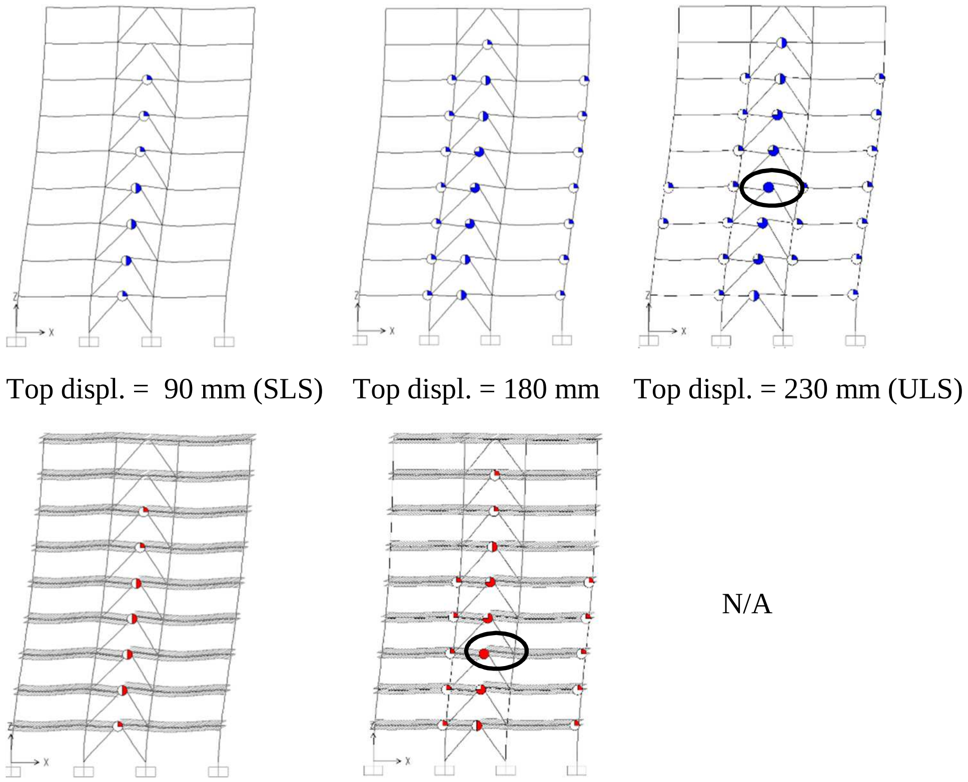
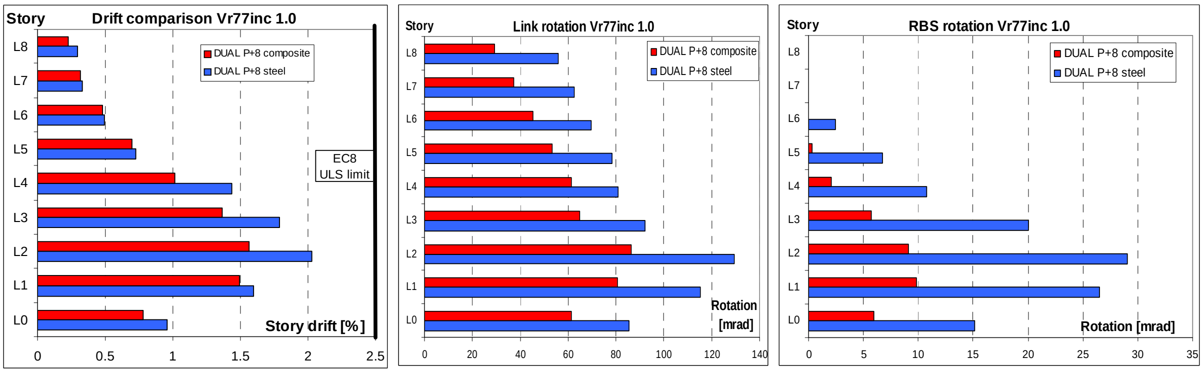


Figure 100. Push-over response of nine-storey structure.

In both cases (steel and composite), the SLS deflection conditions are reached by plastic hinges formed only in link elements. The ultimate limit state corresponds to the exhaustion of rotation capacity of links, recorded in the mid-stories. It is to be noted that at the ULS stage, the RBS have only initial plasticization under bending. In these conditions, a replacement of the EBF link element may restore the building in a recovery state after a strong seismic motion. In case of the composite beams the plastic hinges tend to develop later and have smaller rotation values. Consequently, the composite solution leads obviously to a stiffer structure. Due to this fact the structure with composite beams does not reach its target displacement at ULS. Even in these conditions, the overall performance of the structure with composite beams is appreciated as satisfactory, no damage occurring in the non-dissipative elements.

Figure 101 presents the results of dynamic analyses performed on the DUAL-8 M(C) structures for the design condition in case of Vrancea 77 INCERC N-S accelerogram. Among the seven accelerograms considered Vrancea 77 ground motion represents the most destructive. The accelerogram was scaled for corresponding to the peak ground acceleration of 0.24g (λ=1.0) used in design. The charts given in Figure 101 present the maximum values (envelopes) recorded during the quake action for RBS rotation, link distortion as well as the inter-story drift (expressed as percentage of storey height) for each storey.

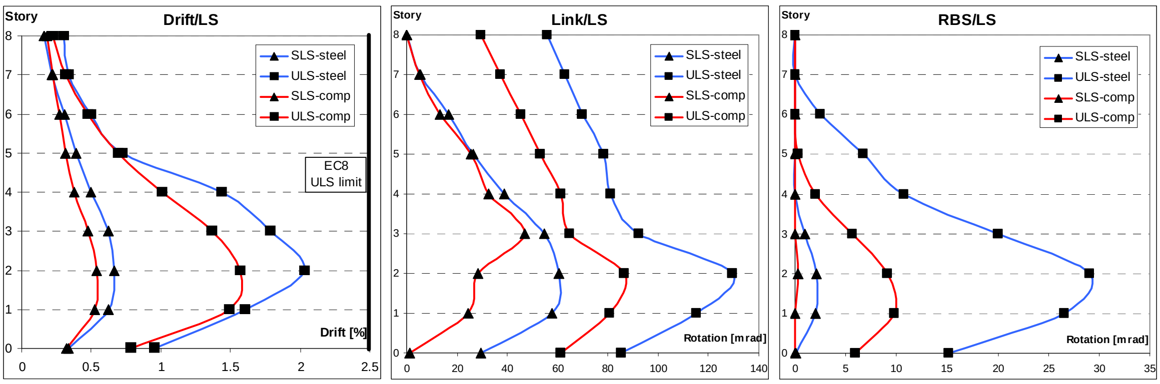


a) inter-storey drift b) plastic link rotation c) plastic rotation in RBS

Figure 101. Maximum values recorded for Vrancea 77 accelerogram.

The largest inter-story drift is recorded at the 2nd level, but the maximum value (roughly 2%) does not exceed the limit given by P100/2006 and Eurocode 8. The stiffer character of the composite frame leads in this case to a maximum drift of about 75% of that of steel. The link exhibits high ductility, reaching rotation values in excess of 110 mrad for steel and 80 mrad for composite structure respectively. The plastic hinge formation in RBS was recorded after the formation of the plastic hinges in link elements. The development of hinges in RBS was up to the 6th floor in the steel configuration and 5th floor in case of composite frames. However, in this case the values are noticeably higher for the steel beams in comparison to the composite ones.

Figure 102 illustrates the difference between the SLS and ULS conditions for the same parameters: envelope inter-storey drift and plastic rotations in link elements and RBS zones. The SLS condition is defined by a half design ground acceleration (λ=0.5). The results at SLS show very smaller differences between the steel and composite frames in what concerns the drift values and RBS zone rotation. However, for lower levels in the structure, there are significant differences in link rotations: 20 mrad for composite, up to 40 mrad for steel frame, respectively. The stiffer character of the composite frame is preserved.



a) inter-storey drift b) plastic link rotation c) plastic rotation in RBS

Figure 102. Differences in SLS and ULD conditions for Vrancea 77 accelerogram.

Figure 103 and Figure 104 present the same recorded parameters in ultimate limit state conditions for all seven accelerograms. The values of rotations recorded in plastic hinges of links for the composite structure appear to be more evenly distributed along the building height. Considering the norm limitations for drift and plastic rotation at ULS, the composite structure could be considered safe, with all the required values for drift and rotations under the norm limiting values. This is also valid in case of steel frame with only two exceptions: the link distortion for Vr77 and 86 exceeds the required experimental value of 110 mrad. As a general conclusion it could be noticed that the plastic rotations demands in case of composite frames is significantly smaller than in the case of steel frame.

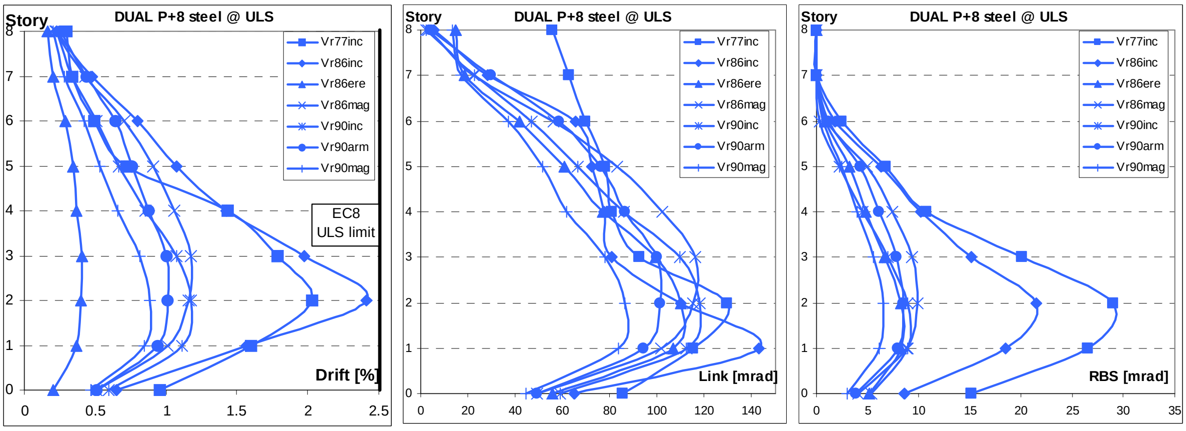


Figure 103. Results recorded at ULS for steel DUAL 8S frame.

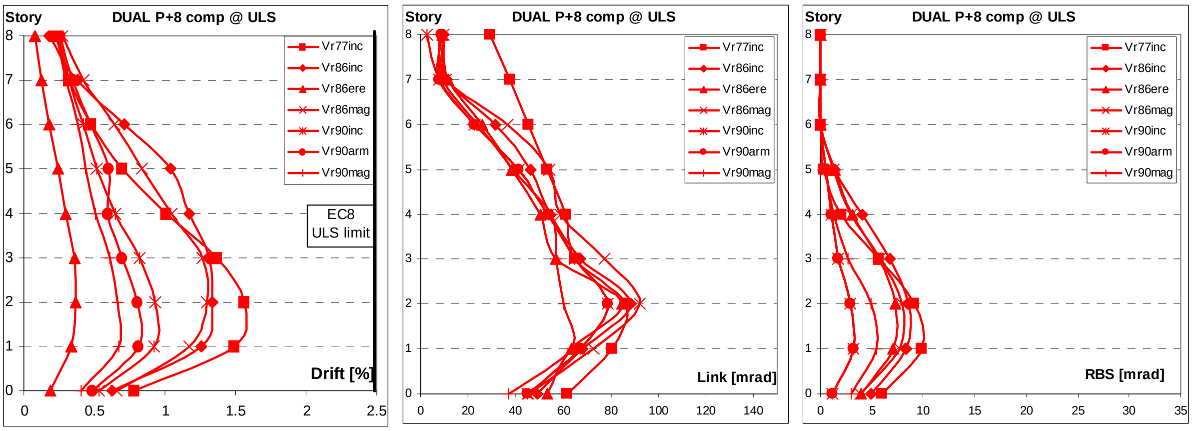


Figure 104. Results recorded at ULS for composite DUAL 8C frame.

- *Incremental dynamic analysis results*

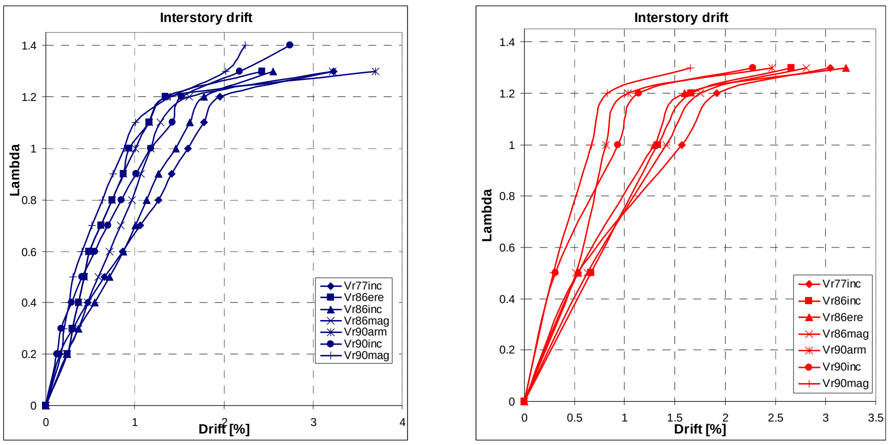


Figure 105. IDA response representations for the DUAL 8 story steel and composite structure, respectively.

The results of IDA are presented under the form of charts plotting the accelerograms multiplier (λ) against the top inter-story drift (envelope value). Figure 105 shows the comparison between the global responses of the 8 story structure in the steel and composite beams solution under 7 earthquake recordings from Vrancea source. The IDA were performed by step-by-step incrementing of the acceleration level while monitoring the top displacement value up to the point when a failure criterion was reached. The apparition and development of plastic hinges was also monitored.

The results show very clearly that in case of frame with steel beams the first plastic hinges are formed at smaller values of λ multipliers than in the case of frames with composite beams, while the failure increment (either by attainment of failure mechanism or reaching the maximum rotation in one of the hinges) has similar values. On the other hand the steel structure exhibits higher seismic overstrength factors presenting a safety reserve of at least 20% (λ=1.2). From this point the deformations for both structures are amplified.

- *Differences in response: EBF and DUAL structures*

The difference in seismic design of EBF and DUAL EBF+MRF is made by the presence of moment-resisting structure, which must withstand a lateral seismic force proportional to its stiffness (clause 6.10.2 of EN 1998-1). Consequently it is to be expected that a dual configuration will be stiffer that the equivalent EBF. Table 14 presents in comparison the drift and rotation requirements for the two types of structures, in the design situation (λ=1.0) for each earthquake recording. Also, in Figure 106 are presented the IDA top-drift versus λ multiplier curves for three selected accelerograms. The results show that not in all the cases the DUAL structures are stiffer than EBF frames and in a general manner other parameters, such as the shape of spectrum governs the particular behaviour of the frame. Also, it can be noticed that in both cases, the drift, link distortion and RBS rotation are smaller for the composite configuration, confirming the conclusions drawn up to this point.

Table 14. Comparison in monitored parameters for EBF and DUAL 8 structures

Results at ULS for EBF - 8 structure						
Quake	Drift requirement [%]		Link rotation [mrad]		RBS rotation [mrad]	
	Steel	Composite	Steel	Composite	Steel	Composite
Vr77inc	1.52	1.27	146	110	---	---
Vr86inc	1.47	0.92	136	96	---	---
Vr86ere	1.36	0.85	125	61	---	---
Vr86mag	1.21	0.72	108	64	---	---
Vr90inc	1.03	0.93	92	53	---	---
Vr90arm	0.97	0.93	85	58	---	---
Vr90mag	0.83	0.53	72	59	---	---

Results at ULS for DUAL - 8 structure						
Quake	Drift requirement [%]		Link rotation [mrad]		RBS rotation [mrad]	
	Steel	Composite	Steel	Composite	Steel	Composite
Vr77inc	1.59	1.56	129	86	29	9
Vr86inc	2.41	1.89	143	128	21	7
Vr86ere	1.11	1.04	110	92	8	7
Vr86mag	0.70	0.62	116	98	9	7
Vr90inc	1.17	0.93	117	110	8	4
Vr90arm	1.00	0.81	101	91	8	3
Vr90mag	0.88	0.66	86	79	6	5

Note: all steel beams in DUAL configuration were designed using steel quality S235, while all beams in EBF configuration are steel quality S355.

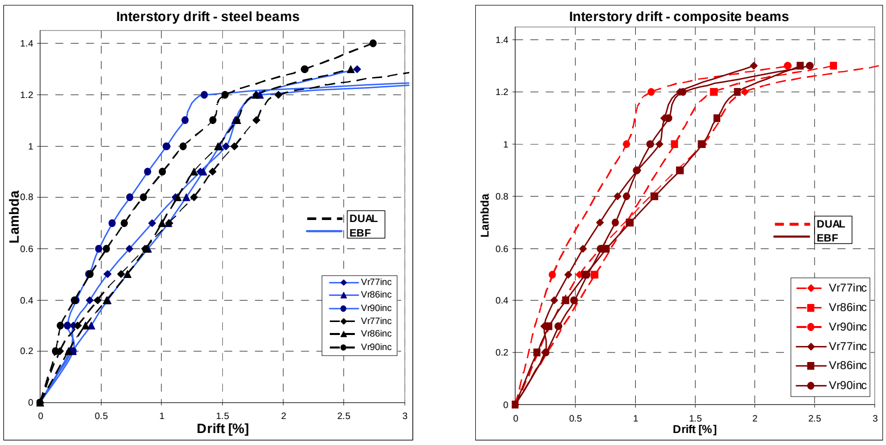


Figure 106. IDA response representations for DUAL and EBF 8 story steel and composite structures.

- *Investigation on behaviour factor and seismic efficiency*

Table 15 shows the values of q (behaviour factor) and η (seismic efficiency) computed as described above. The results are given for six representative frames. Due to the fact that the accelerograms used in analyses were initially pre-scaled to the design seismic intensity, the η factor is equal in this case to the ultimate value of accelerograms multiplier – λ_u. The values listed in Table 15 represent the average values computed for all the seven accelerograms considered for analyses. The exact values of the q factors for each structure and accelerogram could be found elsewhere (Danku 2011). The q values resulted show a good agreement with the value used in design (q=6) for EBF and DUAL type frames but for pure steel structures only. These are in coherence with the values found by other researchers (Rossi 2007, Degee, 2010). However, smaller values are found for composite structures. This fact could be explained by the higher rigidity offered by composite beams. Although the ultimate accelerograms multipliers are about the same for steel and composite frames, higher λ values are required for first plastic hinge in composite beams. In consequence, the overall q values are smaller than in case of steel structures.

The values of the seismic efficiency proves a global minimum reserve for all structures (steel and composite) greater than 20% which could be judged as safe for the current design. The final value of η does not depend on structural typology (steel/concrete, DUAL/MRF/EBF), but only on the real response under different accelerograms.

Table 15. Accelerogram multipliers, q factor and structural efficiency.

Structure	Configuration	q_{avg}	η_{avg}
F1 DUAL - 4	Steel	5.5	1.4
	Composite	3.9	1.2
F6 DUAL - 12	Steel	5.8	1.4
	Composite	4.9	1.2
F7 MRF - 5	Steel	5.8	1.5
	Composite	5.8	1.5
F5 EBF - 6	Steel	5.9	1.2
	Composite	3.6	1.2
F4 EBF - 8	Steel	6.5	1.3
	Composite	5.8	1.2
F3 DUAL - 8	Steel	5.8	1.4
	Composite	4	1.3

Note: q_{avg} and η_{avg} represent the average values of all earthquake recordings

- *Investigation on the overstrength factor*

The overstrength Ω factors are used in the design of non-ductile elements (columns for example) and represent the minimum ratio between loading level of ductile elements under seismic combination condition to the plastic capacity of the element section (clauses 6.6.3 and 6.8.3 of EN 1998-1). The Ω factor should be unique on the structure. The usual values of the $1.1\gamma_{ov}\Omega$ product for design range between 2 and 4. For the structures under consideration a value of 2.5 was used for EBF and 3 for MRF. The seismic analyses performed on accelerograms for the equivalent elastic analysis show that the Ω values are increasing from bottom elements to top ones, which means that the top dissipation elements are usually not very stressed in seismic conditions, leading to high values equal to 7 or 9. However, this conclusion is coherent with the order of formation of plastic hinges. In consequence, the minimal values of the overstrength factor Ω are always taken from the lower storeys.

Table 16 gives the overstrength factor values for the structures under analysis, computed separately where the case for MRF, EBF and overall DUAL values. The smallest values were resulting for MRF with values close to 1.0 both for steel and composite structures. This fact proves the efficiency of RBS solution. The final values of overstrength product $1.1*\gamma_{ov}*\Omega$ (between 1.5 and 2) is smaller than the values used in design.

Table 16. Overstrength factor values.

Structure	Ω - EBF		Ω - MRF		Ω - structure		$1.1*\gamma_{ov}*\Omega$	
	steel	composite	steel	composite	steel	composite	steel	composite
F1 DUAL - 4	2.16	2.22	2.83	2.94	2.16	2.22	2.97	3.06
F3 DUAL - 8	1.52	1.55	1.39	1.72	1.39	1.55	1.91	2.13
F6 DUAL - 12	1.57	1.58	1.32	1.39	1.32	1.39	1.81	1.91
F2 EBF - 4	1.52	1.62	---	---	1.52	1.62	2.09	2.23
F5 EBF - 6	1.83	2.07	---	---	1.83	2.07	2.51	2.84
F4 EBF - 8	1.53	1.67	---	---	1.53	1.67	2.11	2.30
F8 MRF - 5	---	---	1.08	1.18	1.08	1.18	1.49	1.62
F8 MRF - 5	---	---	1.04	1.30	1.04	1.30	1.43	1.78

However, in case of simple EBF structures the overstrength factors are greater, the maximum values being 1.8 for steel and 2.1 for composite frames. This leads to average $1.1*\gamma_{ov}*\Omega$ values of 2.5 for steel structures (identical to the ones prescribed) and 2.8 for composite.

For DUAL frames, there have been computed Ω factors for both MR and EBFs. In this situation the MRF beams are less stressed as compared to simple moment resisting frames, this being proved by higher Ω values. On the other hand, smaller values are obtained for EBF of dual frames in comparison to pure EBF. This leads to the conclusion that the seismic-induced efforts are redistributed compared to the case of pure MRF and EBF frames, namely from MRF to EBF. In order to be coherent with the modern seismic norms, a single Ω factor should be used for the entire structure. In consequence the lowest Ω value should be considered from all the values computed for MR and EB elements. For our applications the values found could be divided in two cases:

- for low-rise buildings (e.g. 4 storey structures) the $1.1*\gamma_{ov}*\Omega$ products given in norms are smaller than the obtained values. This could lead to under-dimensioned non-ductile elements (columns and braces in this case);
- for medium-rise buildings (8 to 12 storeys) the $1.1*\gamma_{ov}*\Omega$ product is in the limits of the values used in design.

The composite frames have greater values for both Ω and total $1.1*\gamma_{ov}*\Omega$ products. This is however against the existing prescriptions, which guarantees the same values for steel and composite frames.

- *Seismic behaviour of dual MRF + EBF - Conclusions*

The Incremental Dynamic Analyses (IDA) has proven the fact that structures where the interaction between steel and concrete was modelled have had a different behaviour from the bare steel ones. The low-rise steel structures (4 stories, 5 stories) have shown higher drift and rotation requirements than the similar frames modelled with composite beams. For the high-rise structures, with a higher vibration period, the increase in strength and rigidity induced by the composite effect leads also to smaller rotations in links and RBS.

The numerical results indicate that the main plastic deformation requirements in case of DUAL (MRF+EBF) structures are to be found mainly in the link elements (with values around 100 mrad for the design situation) and with some contribution from the RBS (values around 20 mrad for the ULS). From this point of view, the value proposed by EN 1998-1, § 6.8.2. of 80 mrad for the short links in EBF becomes insufficient for the DUAL frames.

The values for the behaviour factor q, obtained for the analysed steel structures, are close to the prescribed design values (e.g. q=6 for DUAL frames), confirming the good dissipation capacity of these systems. It is to be noted that, following the numerical analyses, for the composite structures it resulted behaviour factors with smaller values than for the same structures with steel beams (e.g. for the DUAL frames, between 4 and 5, with respect to 6).

By analysing the seismic efficiency – η, it resulted that the analysed structures have strength and ductility reserves of about 20-40% compared to the design requirements.

2.4.3. Robustness of steel frames with composite steel and concrete beams

The term structural robustness is used to identify the quality of a structure of being indifferent to local failure, in which small damage causes only a similarly small change in the structural behaviour. A robust structure has the ability to redistribute load when a load carrying member experience a loss of strength or stiffness and exhibits ductile rather than brittle global failure modes. A robust structure does not mean one which is over-sized. The ability to resist damage is attained through consideration of the global structural behaviour and failure modes so that the effects of a localised structural failure can be decreased by the ability of the structure to redistribute the load elsewhere, and so that the effects of the initial failure are limited.

Significant area of research in structural engineering in the last years has been structural robustness. The need for robustness rest in the fact that structural design codes are based mainly on the design of structural members or the consideration of the member failure modes. Furthermore, design codes and engineers may not always include all relevant design situations or relevance for the integrity of the overall structural performance. Robustness is a property which is not only associated with the structure itself, but needs to be considered as a product of several factors: risk, redundancy, ductility, consequences of structural component and system failures, variability of loads and resistances, dependency of failure modes, performance of structural joints, occurrence frequency of extraordinary loads and environmental exposures, strategies for monitoring and maintenance, emergency preparedness and evacuation plans.

In order to minimize the possibility of failures, as those mentioned above, many building codes consider the need for robustness in buildings and provide methods to obtain robustness. In fact, in all modern building codes, it is possible to recognize the statement (in this or a slightly different form): „total damage resulting from an action should not be disproportional to the initial damage caused by this action".

Progressive collapse resistance is a measure of the structural robustness and relies primarily on resistance of key elements, continuity between elements and ductility of elements and their connections. The different nature and intensity of extreme loading events make difficult the development of explicit design requirements for such design situations. A better strategy is to limit the extent of damage in case of such events, so that the progressive collapse is not initiated. However, there are concerns that primary load carrying members would not be able to develop the required tie forces because of the significant deformation demands. Some structural features may improve the load redistribution capacity and thus, may reduce the deformation demands. One example is the floor system that considers the interaction between concrete slab and steel beams. In the experimental study (Astaneh et al., 2001), the ability of a typical steel structure to resist progressive collapse in the event of the loss of a column due to a blast was investigated. The findings suggested that a retrofit scheme in which cables are added to the side of beams could be used to develop larger catenary action with a higher factor of safety. The increase in capacity achieved by this scheme was confirmed by a different test in which horizontal cables were placed in the floors and on the top flange of the girders along the exterior column line (Tan and Astaneh, 2003). Other studies were conducted to Stevens et al., 2009), (Alashker et al., 2010), (Dubina and Dinu, 2012). The results showed that the effect of this interaction is favourable especially for less redundant structural systems.

- *Robustness analysis – reference building*

The case study building has a four-bay, four-span, and six-story steel structure with moment frames in both directions as shown in Figure 107. The bays and spans each measure 8.0 m and the stories are 4.0 m high each. The structure was designed with non-composite and composite steel beams. For the first structure, no interaction between the steel beams and the concrete slab was considered (denoted as steel model). For the second structure, secondary beams were designed as composite section with full shear connection using shear connectors along the entire length (denoted as composite model I), see Figure 107. A third model, also composite, has been studied, with shear connectors on secondary beams but also on the main beams (denoted as composite model II). Composite beams had shear connectors at top flange on one row at 200 mm interval (Figure 107).

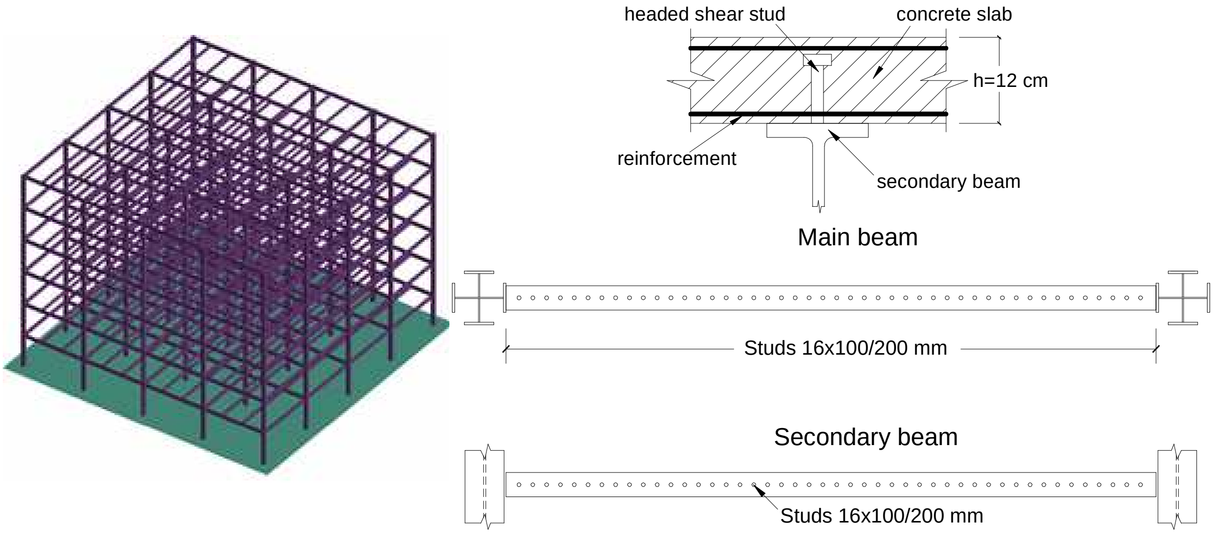


Figure 107. General view of the multi-storey frame building; details of the floor structure: cross section of the composite secondary beam and distribution of connectors for main beam and secondary beam

The structures were designed to the effect of gravity loads (permanent and variable actions) and lateral loads (wind and seismic actions). The dead and live loads considered are 4.0 kN/m^2 each and the reference wind pressure was 0.5 kN/m^2. The structures were located in a low-seismicity area, characterized by a design ground acceleration a_g of 0.08 g, and a control period TC of 0.7 s. It should be noted that, the seismic intensity and the response spectrum used in design were those given in the Romanian Seismic Code, P100-1/2006. The high dissipative structural system was considered using a behaviour factor q of 6.5. An inter-story drift limitation of 0.008 of the story height was used for the seismic design at the serviceability limit state. Persistent and seismic design situations were used for the design of members and connections, using the relevant parts of Eurocodes. No particular accidental design situation was considered in design. In order to overtake the bending on two directions, the columns sections are Malta cross sections, made up of two HEB 450 profiles. The design conducted to girders were made of IPE 400 hot rolled profiles and secondary beams IPE 330 for the non-composite floor structure and IPE 270 for the composite floor structure. A reinforced concrete slab of 12 cm was considered with a 2.66 m span between the floor beams. The slab reinforcement includes welded wire mesh of Φ6/166mm x Φ6/166mm.

- *Experimental Models and FE analysis on sub-structures*

A 3D assembly of the reference building was considered for FE and experimental analysis and scaled down to 1:2.5, ensuring that the support and connection conditions are equivalent to those in the reference building. The scaled assemblies are two-bay two-span structures with the total size of 6.0m x 6.0m and 1.5m height. The distance between the column lines is 3m in both directions and the height of the column is 1.5m, excepting the middle perimeter columns which are 3m (1.5m + 1.5m). In order to replicate the constraints within the full structure, it was necessary to model the interaction between the assembly model and the adjacent structure. Thus, lateral restraints made from circular hollow section elements were introduced at the level of the floor beams as well as at the top side of the middle perimeter columns as shown in Figure 108.

In case of scaled sub-structure the columns are also in Malta Cross shape but from HEB 260 profiles but with flange widths reduced to 130 mm. The main beams are IPE 220 while the secondary beams are of IPE 200. Columns were designed as rigid base connections (Figure 109 a). The bolted end plate beam to column connections (see Figure 109 b) are rigid and full strength. The thickness of the end plate is 20mm and the bolts are 10.9 grade bolts in 20 mm diameter. A reinforced concrete slab of 8 cm was considered with a 1.5m span between the floor beams. The slab reinforcement includes welded wire mesh of Φ6/200mm x Φ6/200mm.

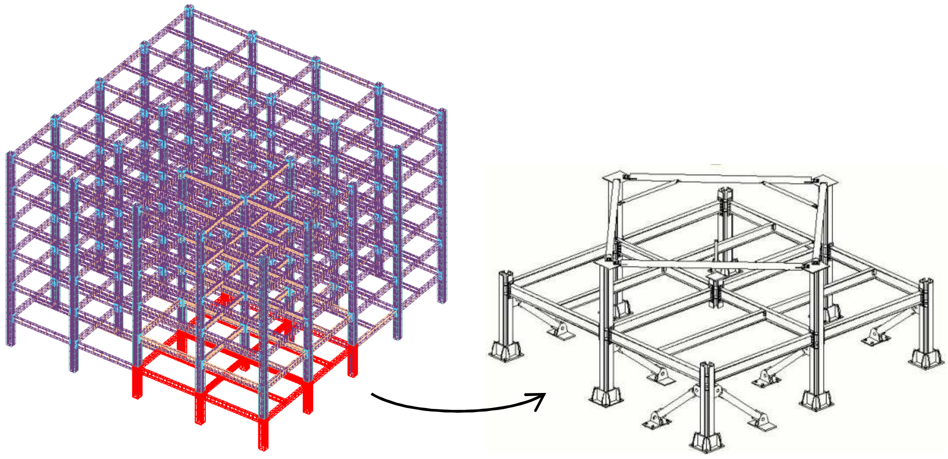
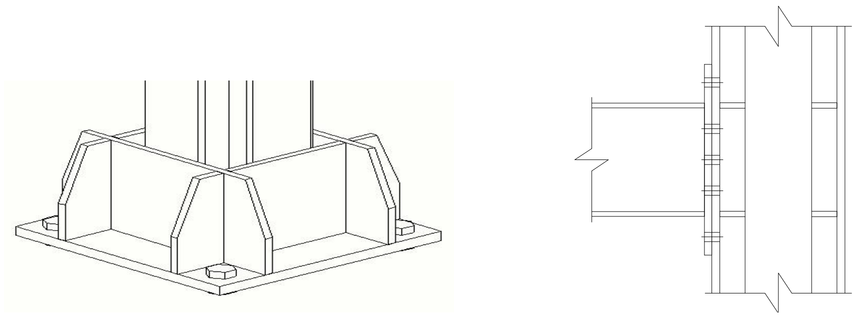


Figure 108. Extraction of 3D assembly model (right) from the reference multi-storey frame building (left)



a) b)

Figure 109. a) Column base connection; b) beam to column connection

- *Numerical simulations and results*

Non-linear static analyses were employed for the evaluation of the structural behaviour following the removal of a column. The loading consisted of application of a gradually increased load on top of the central column. The way of introducing the vertical load, i.e. by pushing down the central column, will be also used for the experimental tests. Material models were based on the characteristic properties. For defining the plastic behaviour of the steel elements (including bolts and shear studs), isotropic hardening models were used by defining yield stress and plastic strain data (Fig. 5a-b). For concrete slab, a constitutive model based on tensile cracking and compressive crushing has been defined (Figure 110 c-d). The parameters for concrete damage plasticity modelling of concrete C25/30 were taken from (Dinu, Dubina, 2012). Figure 111 shows the numerical model for steel structure, Figure 111 b-c shows the distribution of shear connectors on the beams for composite models and Figure 111 d shows the detailed view of the beam to column joint.

Figure 112 displays the deformed shape for the steel model and for the composite floor II model. Figure 113 shows the vertical force vs. vertical displacement curves for the three models. The model with shear connectors only on secondary beams (composite floor I) has approximately the same initial stiffness and yield capacity as the steel model, but after the plasticity starts to develop, the capacity is larger and the ultimate capacity is almost 25% larger than for the steel model.

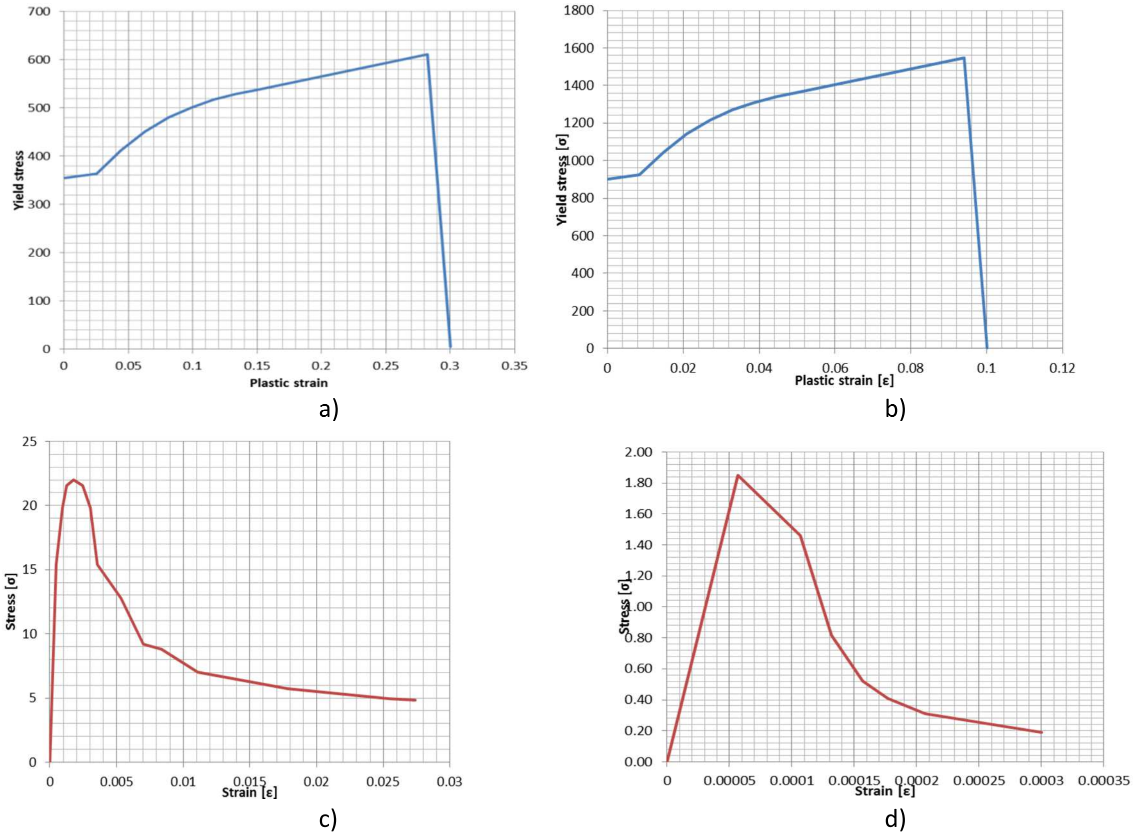


Figure 110. Isotropic hardening model data used for defining the nonlinear plastic behaviour for steel elements and shear studs (a) and for bolts (b)

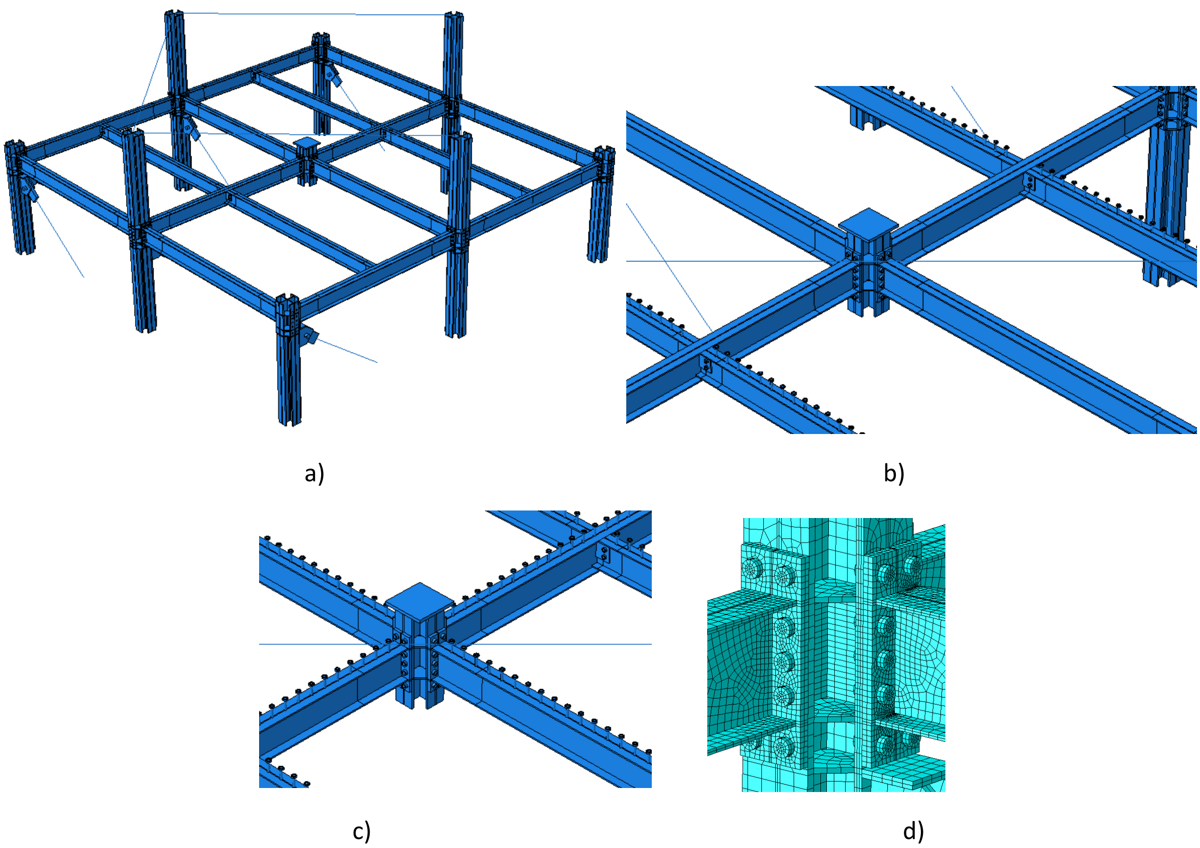


Figure 111. a) 3D model on pure steel; b) shear connectors on secondary beams for composite model I; c) shear connectors on beams for composite model II; d) detailed view of FE joint mesh

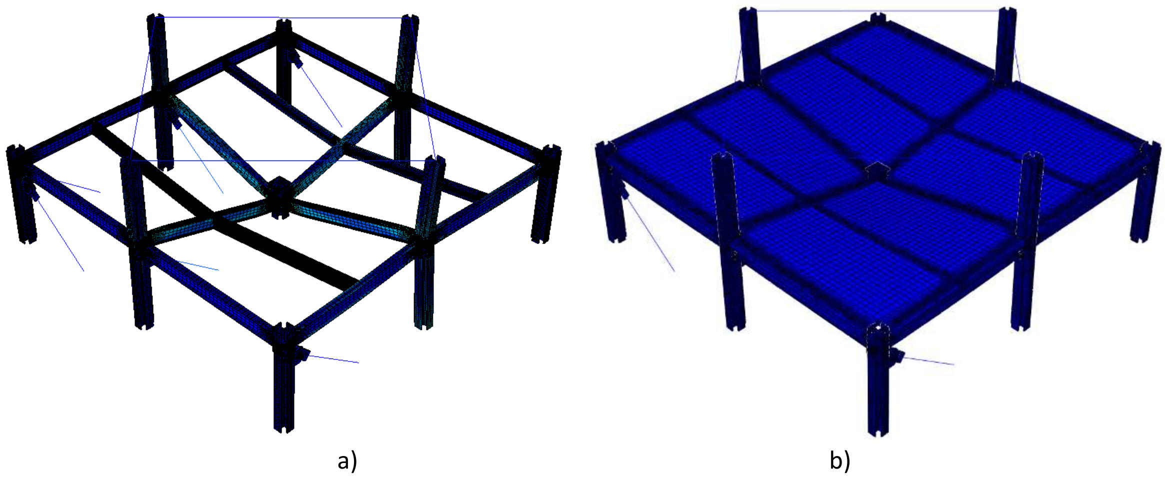


Figure 112. a) Deformed shape of the structure: a) steel model; b) composite floor II

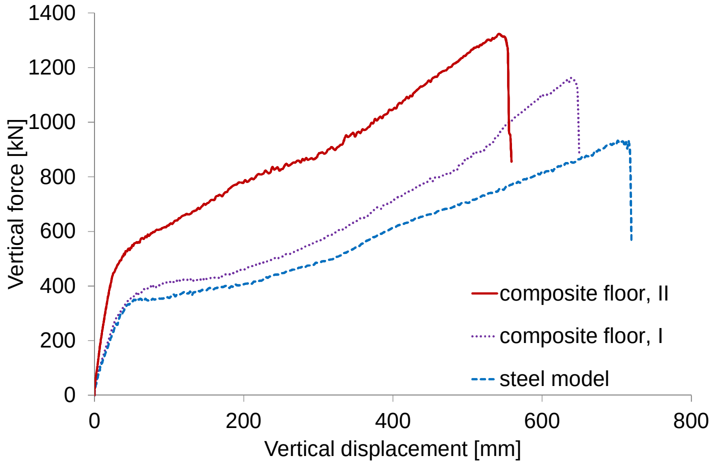


Figure 113. Comparison of force-displacement curve for steel model, composite model I and composite model II

Figure 114 shows the contribution of the flexural and catenary action to the total response of the structure for the steel model. As seen from the figure, the load in the initial phase is resisted entirely by the flexural action (denoted as F). The plastic rotation in beam associated to this stage is 0.033 rad. At larger deformation and rotation stage, the load is mainly resisted by catenary action (denoted as C) and the ultimate rotation of the beam is 0.24 rad. Between these margins, the contribution is shared between flexural and catenary actions (denoted as F+C) and the maximum rotation is 0.117 rad. The same results may be plotted for the composite models.

- *Robustness of steel frames with composite steel and concrete beams - conclusions*

Three models with different floor systems have been studied. The conclusions of the study are summarised below:

- composite beam models showed an increase of the ultimate capacity but a reduction of the ultimate displacement compared to the steel model;
- the contribution of the concrete slab to the ultimate resistance of the structure increases when shear studs are welded on main and secondary beams;
- the catenary forces develop under large displacements and lead to a significant increase of the ultimate resistance;
- The research program is continuing with experimental tests on similar 3D assembly specimens. The tests will provide data for the validation of the numerical models under column loss scenario.

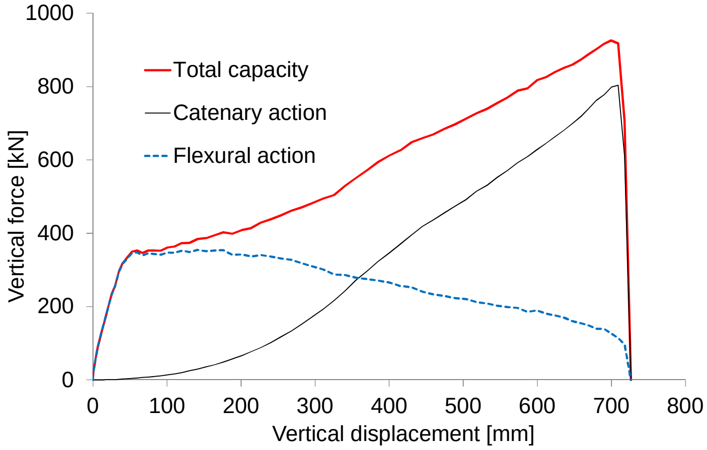


Figure 114. Catenary and flexural resistance for the steel model

3. SUSTAINABLE DEVELOPMENT OF BUILDINGS

Sustainable development is a concept often used in the past decade and has become a leitmotiv for the industry in presenting their products. However most times, showcasing “eco” products is generally based on a simplistic analysis, often compared to similar products and most of the times including only the production phase into the analysis of the product, without taking into account the environmental impact during its life-time and at the end of life.

Nevertheless, in terms of thinking based on a purely economic analysis, the "environment" factor is only a secondary criterion of choosing a product. Therefore, without a coherent penalty policy for products that have a major impact on the environment, an economic analysis integrating the "environment" factor cannot be reached.

There are examples of successful policies for such integration of the environmental factor in the case of cars (EURO - European emission standards), or appliances (by categories of pollution and energy consumption). Currently, more than ever, and significantly more in the near future than at present, products/processes must be designed so as to preserve our existing resources and minimize environmental impact.

Primary factor contributing to environmental degradation is the energy consumed in all stages of production and operation of products (processing, transport, use, including disposal). The Chartered Institute of Building, UK shows that about 45% of the energy generated is used in order to power and maintain buildings, and 5% to construct them (CIOB, 2008).

The heating, lighting and cooling of buildings directly through the burning of fossil fuels (gas, coal, oil) and indirectly through the use of electricity is the primary source of carbon dioxide and accounts for half of all global warming gas emissions. In conclusion, energy consumption in building sector could be considered as the principal culpable for the environmental impact. The mission of the strategy called "sustainable development" is represented by the environmental impact mitigation and monitoring and it has become one of the world priorities for present and near future.

Sustainable development in constructions can be achieved only through innovation at the technological and conceptual level. The process is clearly multi- and interdisciplinary. To build sustainable, one can base on performance conceptual models (functional, safe, neutral or low environmental impact), using materials with good mechanical characteristics and sustainable features (recyclable and low embedded consumption of primary resources and energy), applying constructive systems and related technologies (security, flexibility, low energy consumption, minimal impact to the environment).

Energy use throughout the entire service life of the building, known as operational energy, is one of the most important keys in the construction sector. Regarding buildings, thermal performance and energy efficiency, respectively, have an important economic, social and environmental impact. Designing a building for a lifetime, established by design rules to 50-100 years, can no longer be made by ignoring its impact on the environment, including its construction and exploitation, respectively, through both resource consumption and its effects - in the construction phase - as well as during operation. Integrating a sustainable development, a building and its associated area must meet the following requirements during building and operation: (i) effective choice of location, (ii) design in terms of durability of construction, (iii) material selection, (iv) construction, (v) waste management, (vi) energy and water efficiency, (vii) indoor air quality, (viii) use, (ix) dismantling, (x) reuse of components, (xi) recycling. The impact of all of these requirements should be integrated into an overall life cycle impact (Dubina et. al., 2007a, b). Considering the huge amount of energy and materials used in construction, environmental impact is increasingly seen as a prerequisite of the design process. Moreover, this should be considered in all phases of construction, as Michlmair and Maydl (2008) showed the influence of selected materials and the construction stage is overestimated compared to the use and maintenance stages, highlighting the importance of life-cycle design of buildings.

At a global level, there exists an interaction among energy, economic growth and sustainable development. This is known as the "energy trilemma" and involves three parameters: energy consumption, economic development and environmental impact (Kahn, 1992). As a matter of fact, it is very difficult to exit of this vicious circle, and the implicit decision is taken on the principle: pick two, ignore the third (Andersson, 2008).

Starting from its definition, according to which the economic growth represents the result of a real process of energy-and matter- transformation by using living labour it can be simply deduced that energy consumption and economic development go hand-in-hand. In this sense, the environmental impact is not only out of the equation but is negatively affected, as shown by E. Altvater in his treaty (2008) according to which the economic growth violates the conditions of ecological sustainability. Adversely, the strategies that take into account a sustainable development will have a negative impact on one or both of the other two parameters.

However, the conflicts between the three parameters could be generally solved by compromises (conflicts are often the beginning of compromise). For this purpose, the interaction of parameters is of a prime importance in finding solutions. The general tendency nowadays is to rely on economic growth considering the environmental impact, tending to minimise the traditional energy use. That is why a field which is in full progress nowadays and encouraged by all the governments is the research on the "green energy". This kind of energy can have favourable benefits for the environment while keeping the economic growth.

3.1. Environmental constrain in design of buildings

In the traditional spirit of building, there is a frequent dispute that leads to a so-called "adequate design". It includes two conditions that are somehow in contradiction:

- the accomplishment of design criteria related to safety and functionality. It implies aspects related to resistance and stability under severe conditions of loading, the architectural, thermal, acoustic and hydro-insulation demands etc. that could affect the internal comfort of the inhabitants;
- the achievement of an economic structure. This criterion may influence not only the choice of a structural system, but also the choice of building envelopes and other non-structural components. Mainly, it is the beneficiary who takes the principal decisions about investing more in the construction phase thus reducing the costs during exploitation or reciprocally.

In the holistic approach, applying an integrated performance design, the environmental impact should be considered as a third set of constraints. Taking into account the fact that the first condition of safety and functionality represents a necessary condition for a building, it results that the design should be based on the following philosophy: *among the solutions that assure the safety and functionality of a building, one should chose those conducting to a minimum cost and a lower impact on environment*.

Sustainable development in construction assumes not only the implications in design options (the choice of building materials etc.) and the architectural aspect, but also should contain considerations on energy efficiency or the disposal scenario. Most of the results derived from environmental impact analyses carried on buildings clearly show the fact that the dominant impact factors are related – directly or indirectly – to the energy use.

3.1.1. Phases of an LCA

Life Cycle Assessment (LCA) is the most appropriate way to determine the environmental impact of products or processes. Despite numerous studies about the approaches for the buildings sustainability assessment, there is a lack of a worldwide accepted method of quantifying the environmental impact (Braganca et al., 2007).

LCA is a methodology for assessing environmental aspects and the potential associated with a product by:
- compiling an inventory of data input and output of a system;
- assessing potential environmental impacts associated with the input and output;
- interpretation of results in relation to objectives.

LCA studies the potential impact of products, through raw material acquisition through production process, use and disposal (cradle-to-grave). General categories of environmental impact to be considered include the use of resources, human health and environmental consequences (ISO 14040, 2006). In this context, there have been defined a number of impact categories for which their contributions due to buildings can be calculated. These represent the LCA indicators. By performing an LCA analysis one can obtain quantitative information about building contribution to climate change and resource depletion. Subsequently these results can be compared with the results of other similar buildings (keeping the same boundary conditions). Research aims to evaluate and increase the overall efficiency of a project, as the problem that arises in the selection of a variant of a life cycle of a building is to choose an optimal LC. In order to evaluate the overall efficiency of a project it is necessary to identify selection criteria. Banaitiene et al. (2008) presented a methodology for the multivariant design and multiple criteria analysis technique of the life cycle of a building.

In an LCA analysis the principle is relatively simple: for every stage of life cycle are investigated quantities of material and energy use and the emissions associated with these processes. Emissions (the most publicized being CO2 emissions) are then multiplied by characterization factors proportional to their power, each having a different impact on the environment. Often, one is chosen as a reference emission and the result is given as an equivalent to the impact of the reference substance (Glaumann and Kahraman, 2009). The number of equivalent summed for each impact category can then be normalized and weighted to arrive at an aggregated (final) result. Different calculation tools may use different characterization factors and different emission data if the production and combustion processes vary. Also different calculation tools may use different normalization and aggregation methods, leading to different results. As it has been pointed out by da Silva et al. (2007), a basic LCA tool may include a generic database with information on emissions for a number of construction materials, processes and types of energies. It is preferable that this information is extracted from the EPD (Environmental Product Declarations). Sophisticated LCA tools need access to international databases, specific for each product/process/material. The necessary input information to the various stages of the life cycle is given in Figure 115.

To make an LCA assessment some key elements are needed. Although there is no single way to conduct a LCA, the LCA is expected to include the following phases:
- definition of the purpose and scope of application;
- inventory analysis;
- impact assessment;
- interpretation of results.

To define the purpose and domain of application, one should define a functional unit (to which the environmental impact relates) and boundary conditions (conditions included in the evaluation) according to the aim of the study. There should be included at least two stages of the life cycle, for example the production of construction materials, and the operational phase, to justify the life-cycle approach.

Definition of functional unit is particularly important when compared to other products (buildings). According to CEN Technical Committee 350 (EN 15643, 2011) it is recommended that it shall be called "functional equivalent".

The steps to follow in conducting a LCA analysis are shown in Figure 115 – right (ISO 14042, 2006). The inventory analysis is the process of compiling information necessary for evaluation. In a subsequent step, is carried out the life cycle impact assessment (LCIA - Life Cycle Impact Assessment). To carry out the LCIA are required a few items:

- Selection of impact categories, category indicators and characterization models;
- LCI results classification;
- Calculation of indicators results by category (characterization).

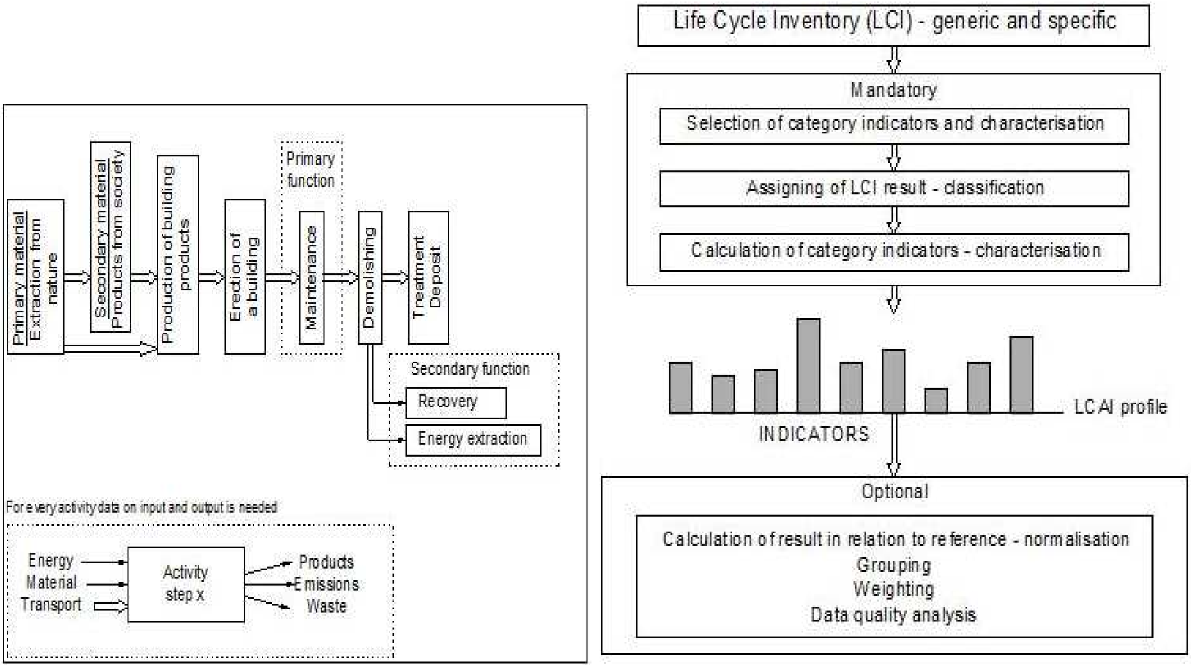


Figure 115. Illustration of life cycle stages of a building and the data for the LCA (Glaumann and Kahraman, 2009) and actions carried for a LCA according to ISO 14042 (2006).

Life cycle costing (LCC) is a tool for performance evaluation of the total cost of a product during a period, including purchase, operation, maintenance and cost of disposal. LCC analysis consists in evaluation of various options to achieve client objectives, in which the alternatives differ not only in initial costs, but include subsequent operational costs. LCC is the focus in the current international trend of financial distribution to ensure efficient construction and built assets. The current trend in sustainable development is to minimize the initial cost and environmental impact.

The building components inventory used in LCA can be also used for LCC but requires additional information for conversion in generic currencies, such as €/Mj and €/kg. The main advantage of LCC analysis is that one can study the price for entire life cycle for various construction products, respectively the construction itself and choice of a suitable design solution. For an accurate assessment there should be considered different scenarios for life cycles.

Due to the fact both LCA and LCC are based on life cycle, assuming a life cycle of materials and construction processes, they can be combined to provide together a potential life cycle cost and environmental impact. Their combination can be used for:

- choice of alternative technical solutions;
- identification of a technical solution that meets environmental targets at lower cost;
- recalculation of environmental impact costs;
- evaluation of investment.

Therefore LCC and LCA can be used for joint causes in a broader assessment process in which each process could be the entry basis for the other.

3.1.2. Current normative framework

By the European Integration, Romania must implement and apply the European standards, including those referring to Sustainable Design of Buildings. At European level within the European Committee for Standardisation (CEN) is operational the Technical Committee 350 (TC350) which provides the legislative framework on sustainable development in the building sector. CEN TC 350 has delivered constantly a considerable quantity of advanced and coherent information in the domain of Sustainable Development of Buildings. Also, the committee has developed measuring and parameterising instruments regarding the environmental and social impact of buildings, making a distinction between the production, building, use and end-of-life stages for buildings. The corresponding TC in Romania dealing with this subject is TC 343 within ASRO. The CEN TC350 documents will be gathered in European regulations, present drafting norms being provisional Europeans standards (pre-standards), such as:

- CEN/TR 15941:2010 - Sustainability of construction works - Environmental product declarations - Methodology for selection and use of generic data;
- EN 15643-1:2010 - Sustainability of construction works - Sustainability assessment of buildings - Part 1: General framework;
- EN 15643-2:2011 - Sustainability of construction works - Assessment of buildings - Part 2: Framework for the assessment of environmental performance;
- EN 15643-3:2012 - Sustainability of construction works - Assessment of buildings - Part 3: Framework for the assessment of social performance;
- EN 15643-4:2012 - Sustainability of construction works - Assessment of buildings - Part 4: Framework for the assessment of economic performance;
- EN 15804:2012 - Sustainability of construction works - Environmental product declarations - Core rules for the product category of construction products;
- EN 15942:2011 - Sustainability of construction works - Environmental product declarations - Communication format business-to-business;
- EN 15978:2011 - Sustainability of construction works - Assessment of environmental performance of buildings - Calculation method.

These documents are based on the family of international standards ISO 14000. In 2006, ISO published a second edition of the LCA standards. ISO 14040 – Environmental Management – Life-cycle Assessment – Principles and Framework, together with ISO 14044 – Environmental Management – Life-cycle Assessment – Requirements and Guidelines (ISO 14044, 2006).

ISO standard 14040 (2006) describes the principles and framework for LCA. It provides an overview of the practice and its applications and limitations. It does not describe the LCA technique in detail, nor does it specify methodologies for the individual components of the LCA (goal and scope definition, inventory, impact assessment, and interpretation). Because the standard must be applicable to many industrial and consumer sectors, it is rather general. Nonetheless, it includes a comprehensive set of terms and definitions, the methodological framework for each of the four components, reporting considerations, approaches for critical review, and an appendix describing the application of LCA.

ISO standard 14044 (2006) specifies requirements and provides guidelines for LCA. It is designed for the preparation, conduct, and critical review of life-cycle inventory analysis and provides guidance on the impact assessment and interpretation phases of LCA and on the nature and quality of data collected.

3.1.3. Use of steel-intensive buildings

Steel as construction material plays an important role as component for buildings and engineering structures, and it has a wide range of applications. On the other hand, steel is the most recycled material and from the total production in the world, almost half is obtained from waste material.

Steel construction sector has a great deal to offer to the sustainable development. Like other industrial activities, steel industry works continuously to improve in terms of sustainability. The following guiding principles for sustainable constructions can be emphasized (2003):

- understand what sustainable development means for you, you clients and customers;
- use whole-life thinking, best value considerations and high quality information to inform your decision making;
- design for flexibility to extend building lifetimes and, where possible, further extend the life of buildings by renovation and refurbishment;
- design and construct with maximum speed and minimum disruption around the site;
- design to minimize operational impacts (e.g. energy use);
- design for demountability, to encourage future re-use and recycling of products and materials;
- engage organizations within your supply chain about sustainability development;
- select responsible contractors who have embraced sustainable development principles.

Burstrand (2001) presents the reasons in choosing steel framing from an environmental point of view:

- light gauge steel framing is a dry construction system without organic materials. Dry construction significantly reduces the risk of moisture problems and sick building syndrome;
- seel, gypsum and mineral wool are closed cycle materials;
- every material used in light gauge steel framing (steel, gypsum and mineral wool) can be recycled to 100%;
- it is possible to disassemble the building components for re-use;
- steel framing means less energy consumption during production than equivalent housing with a framework of concrete poured on-site;
- steel framing only uses about a fourth of the amount of raw materials used for equivalent masonry-concrete homes;
- less waste means a cleaner work site and a low dead weight of building components ensures a good working environment;
- low dead weight leads to reduced transport needs.

In what concerns the building envelopes they should be energy-positive, adaptable, affordable, environmental, healthy, intelligent and durable. The envelope is essential in ensuring the comfort and energy savings of the system taken as a whole. Envelopes have to address the following problems:

- to provide an exterior layer which has to balance the needs of protection from the elements, visual value and economics;
- the thermal insulation has to be compatible with the exterior layers and mechanically fastened to the structure;
- optionally, there could be a cavity space for ventilation;
- the interior components of the envelope containing the finishing surfaces can be permeable or not. The consistency of this last layer has a great influence on the indoor air quality. The trend today is to use gypsum board and vapour barrier on studs.

Regardless of the system / wall assembly, the envelope has to provide as much as possible a uniform "wrapping" of the steel structure in order to avoid and/or control thermal bridging. This aspect is most important, as all condensation problems start from here.

3.2. Sustainability of steel-intensive buildings

The examples presented below show the impact environmental analysis of some steel intensive buildings. Both internal and external analysis were considered: internal analysis for the evaluation of the macro-components with a large impact and (ii) external analysis type as comparisons with other equivalent systems. The candidate was involved in the LCA evaluation and not necessarily in the structural design.

3.2.1. Case study I - Constantin family house

The structure for this private single-family house was built in 2005 in Ploiesti (Fülöp and Dubina, 2004 a, b). There are two main characteristics of the built house, namely the lightweight-steel framing and the architectural solution constrained by the site shape (Figure 116). From the architectural point of view, the main challenge of the project was to fit this private house on an irregularly shaped lot of only 168m2.

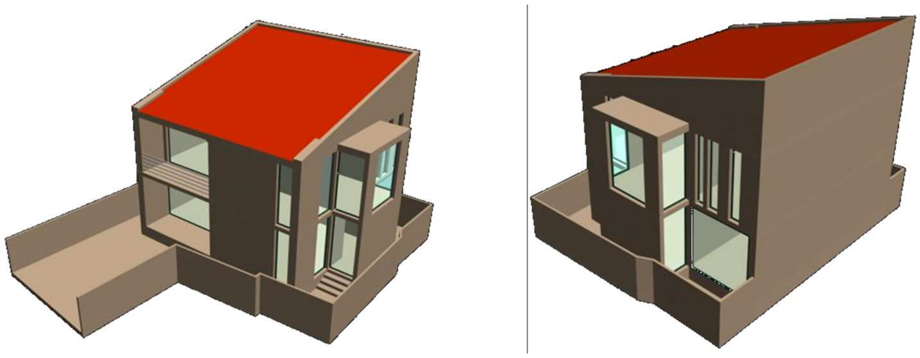
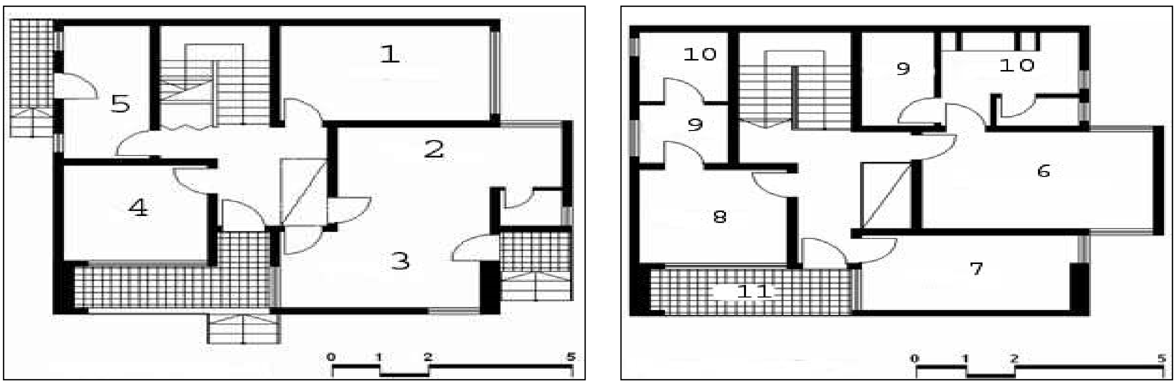


Figure 116. 3D view of Constantin family house

The resulting cube-like building measured in the end 84m^2 of built area on each of the two floors (Figure 117). Due to the proximity of the buildings on the adjacent properties, the next difficulty consisted in finding the balance between the right amounts of views, natural light and privacy. Because the building seats on the property limit, it was impossible to provide window openings on that side. This was also one of the reasons for the roof being made with a single slope, the height of the building ranging from 9m to 10.5m. Each floor has approximately 2.75m height with a roof slope of 30°. Two skylights located above the staircase and the hallway, were placed to provide a light shaft, in order to enhance the centre.



Lower floor plan: 1. dining room; 2. kitchen; 3. living room; 4. den; 5. laundry

Upper floor plan: 6. master bedroom; 7. library; 8. bedroom; 9. dressing; 10. bathroom; 11. loggia

Figure 117. a) Steel skeleton of the structure; b) Skeleton with structural OSB sheeting

Isolated foundations under columns were realized linked by foundation beams. The floor at ground level (concrete slab of 10cm) was realized over a bed of dense soil and a layer of compacted gravel. Figure 118 shows the structure in two different stages: (a) the finished steel skeleton, (b) the steel skeleton together with all load bearing OSB panels attached while Figure 120 presents final exterior and interior views of the completed house. Each floor has approximately 2.75m height with a roof slope of 30°. Two skylights located above the staircase and the hallway, were placed to provide a light shaft, in order to enhance the centre.

(a) finished steel skeleton

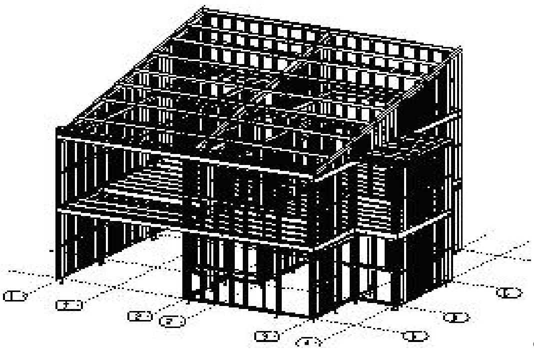
(b) steel skeleton together with all load bearing OSB panels attached

Figure 118. The structure during construction

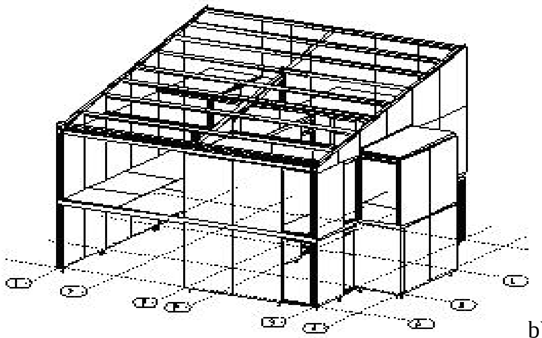
- *Analysis systems and boundary conditions*

In case of thin-walled cold-formed structural system, the structural skeleton is made of light-gauge C shaped profiles (C150/1.5) spaced at intervals of 600mm, with a thickness of 1.5mm, fixed with 4.8mm diameter self-drilling screws Figure 119 a). The height of the cross-section of profiles is 150mm, which governed the thickness of the walls. In order to withstand horizontal actions and to provide stiffness and strength the walls were stiffened using 12 mm thick Oriented Strand Boards (OSB) provided on both sides of the structural walls (Figure 119 b).

The load bearing beams in the slab are C200/1.5 profiles disposed at 600mm intervals, this distribution resulting from the condition of controlling the vibrations of the floor rather than from strength conditions. Roof purlins resulted as Z150/1.5 profiles spaced at intervals of 1200mm. The floor diaphragms were designed to be based on the same principle of covering with OSB. No concrete was used on the slab.



a)



b)

Figure 119. The structure during construction

(a) General view

(b) Interior view – staircase

Figure 120. The structure during construction

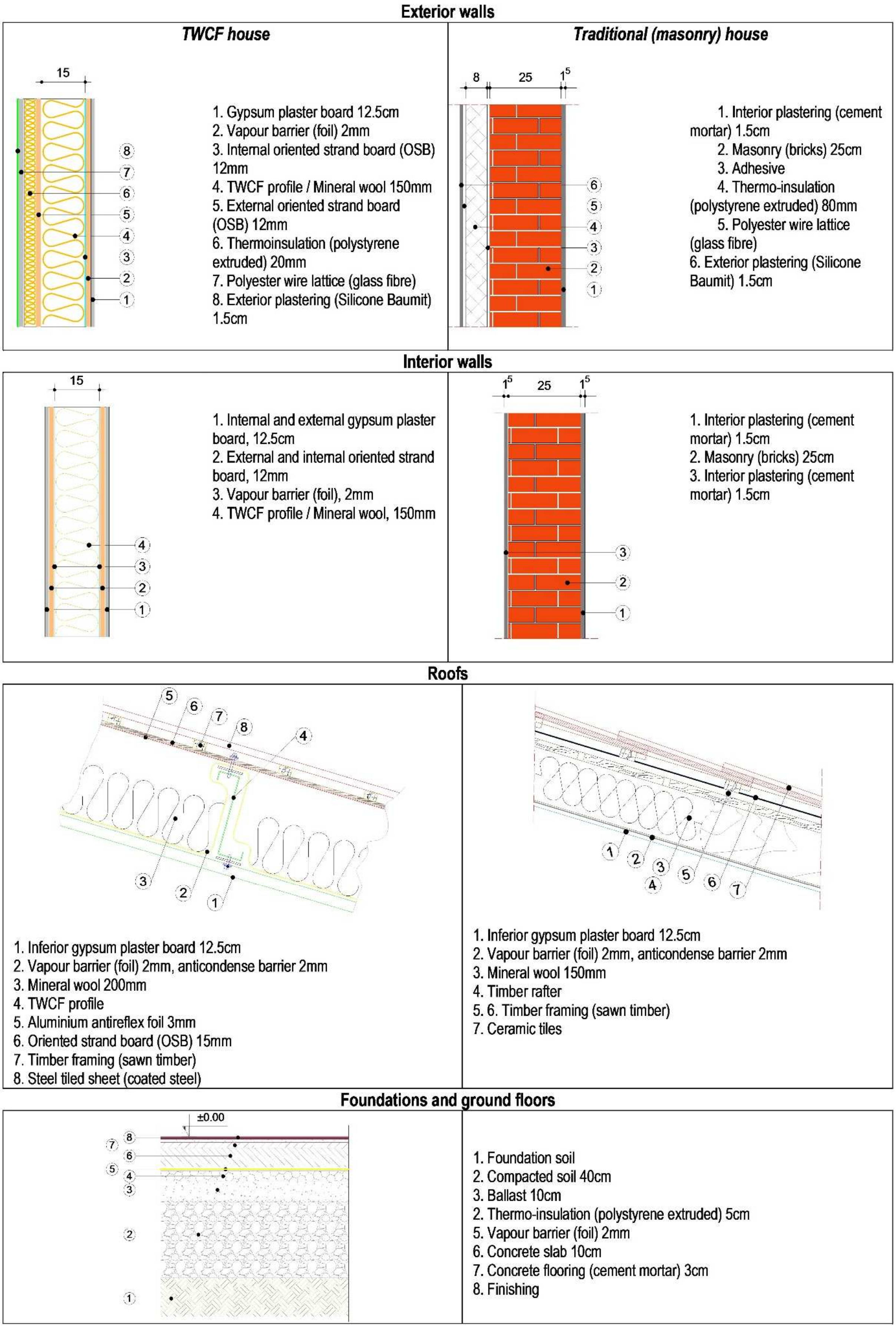


Figure 121. Layers used for structural components

Similar studies (Santos et al., 2010) showed through LCA studies that the environmental performance of light-weight steel houses is greater than that of traditional ones. Therefore, in order to have a real environmental impact comparison, a masonry structure has been designed following the above-described architectural plans (Ciutina et al., 2009; Ciutina and Ungureanu, 2009). The structural system is composed by masonry walls of 25cm, concrete floor, continuous foundations and timber roof structure.

Due to the different thicknesses of walls used in the two solutions, the main preoccupation was to obtain the same inside volume for all rooms. The design considers the same location of the building and the same imposed and climatic loadings. Both solutions (classic and cold-formed) were designed for a service life of 50 years, according to Basis of structural design, EN1990.

Figure 121 presents the layers used for the distinct components of the house: exterior walls, interior walls, first floor, roof and foundations (including the concrete ground floor).

The comparative life cycle analysis was performed using the SimaPro software (SimaPro 7, 2008), a general and comprehensive tool, widely used in environmental design and LCA, which uses the Ecoinvent database (Ecoinvent Centre, 2000). In the analyses were included the material production, construction, end-of life for these materials as well as a maintenance scenario for a life-time period of the house of 50 years. In order to set the input elements (inventory), both for simplifying the model and analysis time-saving, the inventory analysis has been done according to system boundary conditions. According to this, several aspects were considered:

- all identical components and materials which are identical as dimensions and weights for both design situations were left out of comparison. They practically bring the same input and output in analysis (for example wall painting or the floor finishing). Including here are the doors/windows and electrical or heating systems;
- the transportation was not taken into account, although the values (especially the weights) are much smaller in case of steel cold-formed house. However, these may be introduced at any time in comparison if site is set and materials providers are known;
- the domestic use of the building (water/gas/ electricity use), was not integrated herein, as this comparison was focused on the construction stage;
- the energy used for construction purposes (such as cranes and other technological machinery) were not integrated in comparison.

It is to be noticed that due to the lack of information for the Romanian processes and materials, the mean European values were used for the inventory instead.

- *Environmental impact results – construction stage*

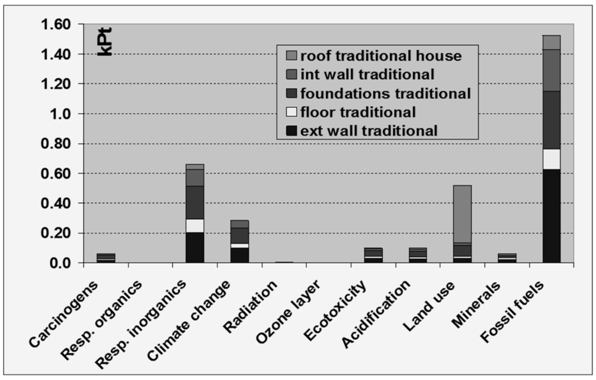
For LCA calculation of building, the input materials have been considered according to the constructive elements: (1) exterior walls; (2) interior walls; (3) flooring system; (4) roof system; (5) foundation-infrastructure.

Figure 122 to Figure 124 present the environmental impact by considering the above input data for construction phase but disregarding the common materials and processes according to the boundaries described in previous paragraph. All the results are given in “Eco-indicator points” (Pt) (Eco-indicator 99, 2000), which express the total environmental load of a product or process, based on data from a life cycle assessment, in order to have unitary and comparable outcomes. The method used for impact analysis is Eco-indicator ‘99.

Figure 122 – left gives the environmental impact for the traditional masonry house for the construction process. One could realize that the major impact is for fossil fuels, as these resources are used for the fabrication of building materials at all levels. Of these, one can highlight brick (the material used for walls) which passes through a kilning process. Also, important values of impact are recorded for inorganic respiratory emissions, climate change substances and land use.

Figure 122 – right presents the result of the same stage ranked per constructive elements. The major impact corresponds now to exterior walls and foundations. Both constructive elements are high consumers of resources but also have a great impact on human health (each with more than 300 points). However, not the same observations could be stated about roofing system, for which the major environmental impact is on eco-system quality, mainly due to high quantity of wood used. Analysing the metallic house in the same manner, the same impact categories are predominant as in the case of traditional case (fossil fuels, respiratory inorganic emissions and land use). The results ranked per constructive elements show that also the external walls and the foundations have the lead on the environmental impact.

In a direct comparative impact analysis, higher impact values result for traditional home for most of the impact indicators (categories), as could be seen in Figure 123. Although in the case of carcinogens and ecotoxicity the impacts have comparable values, large differences could be noticed in case of land use (2.5 times larger the environmental impact of steel house), fossil fuels, respiratory inorganic substances and climate change (at least 1.5 times greater for traditional house).



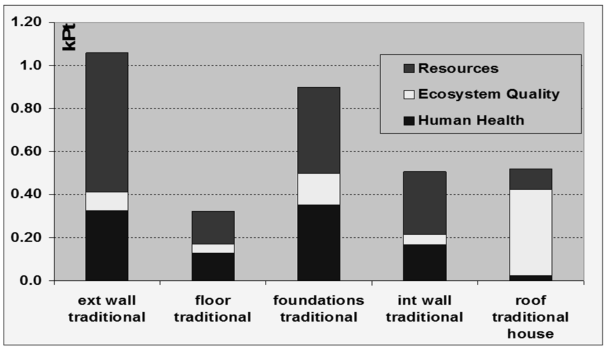
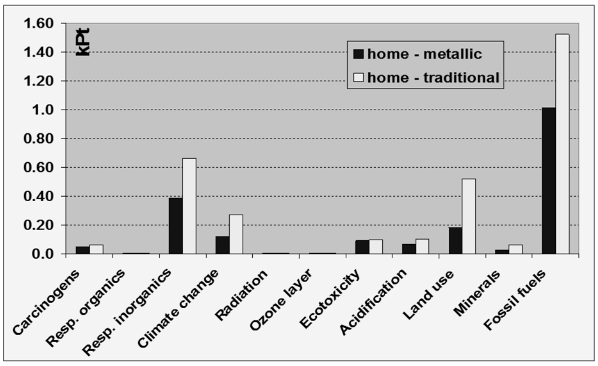


Figure 122. Environmental impact for traditional house – construction phase weighting and impact per constructive element



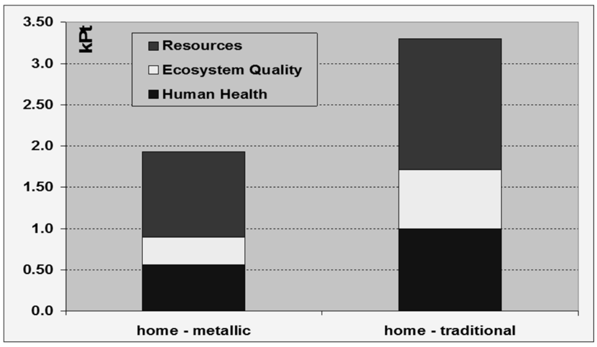


Figure 123. Comparison on environmental impact for metallic and traditional house: weighting and global score

The results presented above on impact (or damage) categories are aggregated into a single score (Figure 124), leading to an overall score of 1931 points for the metallic house, with 41% smaller than the global score of the traditional house (3300 points).

- *Environmental impact results – Impacts of the construction stage and end-of-life*

It is certain that the building process is not complete without an end-of-life for the materials if considering the life-cycle approach. Normally the final destination of waste building materials represents a problem in every country, but may differ even within a country, from zone to zone. There are materials that could be reused in the form they are for the same purpose (ballast for example), others that could be reused for other less important purposes (e.g. crushed concrete as street bed-layer), and others that need waste treatment (incineration) or used simply as land-fill.

For the purpose of our study, the end-of-life of integrated materials was thought according to present conditions in Romania for recycling, reuse and disposal. The end-of life scenarios for the main building materials considered in the analysis were based on inquiry of site engineers about the present conditions in Romania, which are summarized in Table 17.

Table 17. End-of life for building materials.

Building material	Reuse [%]	Recycling [%]	Burn [%]	Landfill [%]
Steel – steel profiles, steel tiled sheets	---	100	---	---
Steel – reinforcement	---	80	---	20
Bricks, ceramic tiles	---	---	---	100
Wood	35	---	65	
Ballast	70	---	---	30
Concrete, mortar	---	---	---	100
Other inert materials	---	---	---	100
Other combustible materials	---	---	100	---

In these conditions, the results ranked on impact categories are given in Figure 124. Obviously, the same impact categories (fossil fuels, respiratory inorganic emissions and land use) are the most employed and all of these prove smaller impact values for steel dwelling. Ranked in a single score (Figure 124) it results that in the life-cycle comparison for the construction and disposal scenarios, the gap between the two situations is larger than in the case of construction process only (comparison of Figure 124 to Figure 123). One of the causes that leads to the real advantage of steel home is the steel recycling which practically introduces positive scoring in the impact assessment, this way the scoring including recycling being even smaller than that including construction process only.

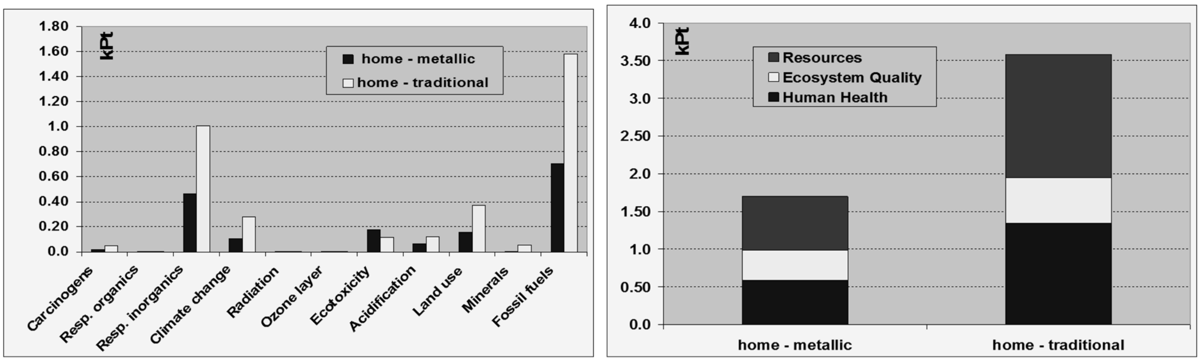


Figure 124. Comparison on environmental impact for metallic and traditional house: weighting and global score – construction phase and EOL

- *Environmental impact results – Integrating the maintenance of buildings*

Regardless the chosen constructive system, a building needs maintenance works during its lifecycle. These works could be of different types and function of this type could be more or less expensive and represent a very important part of the building life-cycle.

The integration of maintenance works for a structure is difficult to make, because the predictions that could be made in advance may not correspond to reality. However, in order to complete the life-cycle of buildings under consideration, the following prediction (pre-planned maintenance) was made for a house, thought for a standard life-time of 50 years:

i) In case of the traditional house:

- nine internal decorations (once at 5 years);
- three external decorations (once at 12.5 years);
- three changes for bathroom/kitchen sanitary: sandstone, sanitary furniture, internal plaster-board etc. (once at 12.5 years);
- one change of the electric and heating system (once at 25 years);
- one change of the roofing system (wood and cover) (once at 25 years);
- one change of the exterior thermo-system (once at 25 years).

ii) In case of the metallic house:

- nine internal decorations (once at 5 years);
- three external decorations (once at 12.5 years);
- three changes for bathroom/kitchen sanitary: sandstone, sanitary furniture, internal plaster board etc. (once at 12.5 years);
- one change of the electric and heating system (once at 25 years);
- ne change of the steel tiled sheeting (once at 25 years);
- one change of the thermo-system (once at 25 years).

Important to notice is the fact that, in case of steel structure, only the steel skeleton remains (theoretically) unchanged, while all other elements are changed once in 50 years. For traditional house, the maintenance reduces here at the level of plastering for walls, thermo-system and part of the roof-wood supporting. For both cases no maintenance was considered for infrastructure. In addition, for the life-cycle assessment, the same conditions for disposal at the end-of life have been considered in accordance to the previous explanations (see Table 17).

Figure 125 presents the impact deduced only for maintenance process. As the quantities introduced for analyses are smaller than that from the building process, the final scores are also smaller. In terms of impact categories, the same categories (fossil fuels, respiratory inorganic substances and land use) integrate more than 85 % of the total scores. In case of maintenance of steel house, the eco-toxicity is also a mentionable category to total score.

All the values resulted only from maintenance and cumulated in a single score (Figure 125) will conduct to about 1000 eco-points in the case of metallic house and respectively 600 eco-points for the traditional house. This is somehow in contradiction with the values derived for the construction process, where the ratio is reversed in the favour of the metallic house. The explanation for this derives from the facts that for the steel house the wall, floor and roof layers are replaced (remaining practically only the steel skeleton) while in the case of traditional house, all the brick, concrete and main wood frame remain in the original form.

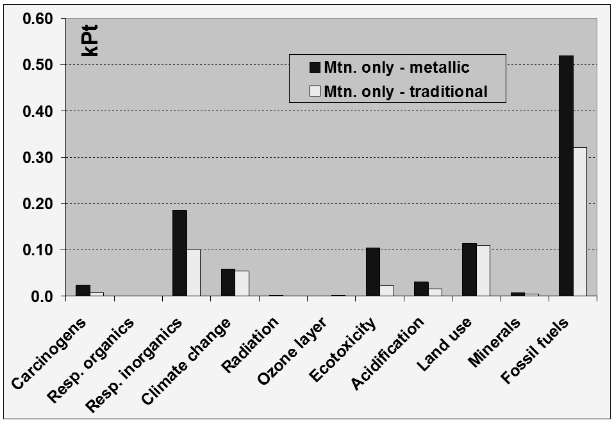


Figure 125. Comparison on environmental impact for metallic / classic house - maintenance only

- *Life-cycle results integrating the construction, disposal and maintenance*

Figure 126 shows the environmental impact on described stages of the life-cycle of buildings as a direct sum of:

- the construction stage;
- the disposal at the end-of-life for different materials;
- the maintenance of the building for a life-time period of 50 years.

As a general trend, the following impact categories are most affected (see Figure 126):

(i) fossil fuels and respiratory inorganic substances, climate change: mainly due to the manufacturing processes which require large quantities of energy, which further on affect directly the fossil fuel reserves. These processes contribute in high extent to the emissions of inorganic substances and climate change gasses;

(ii) land use: due to damages to land use (wood exploitation, ballast pits etc).

As a general rule, the differences obtained in the life-cycle analysis including maintenance follow the trend observed in case of building process itself, namely almost all the environmental impact categories are greater for the traditional home.

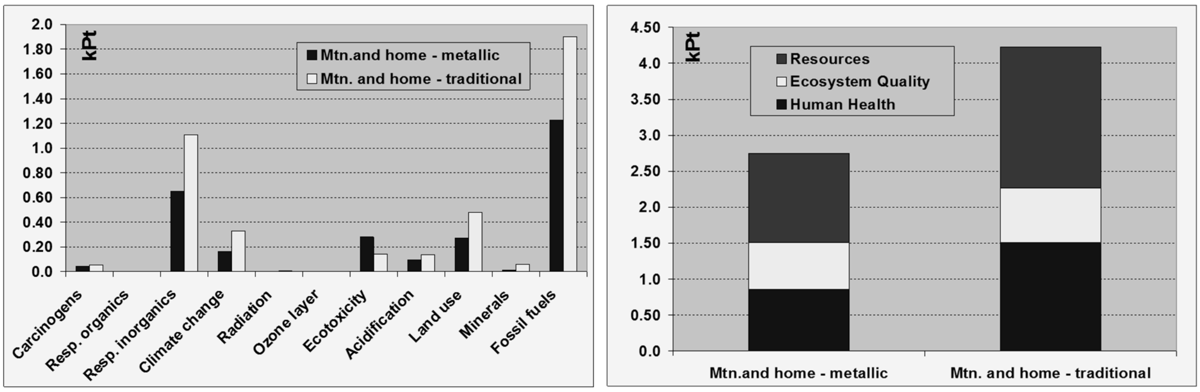


Figure 126. Life-cycle comparison on environmental impact for metallic and traditional house: weighting and global score – construction, including maintenance and end-of-life

In a single score analysis (Figure 126 - right), and taking into account the boundary conditions as explained before, the metallic house (2750 eco-points) present an important advantage in front of traditional home (4220 eco-points). Obviously, many parameters (such as national or regional peculiarities, climatic zones or distance from the material distributors) may affect these results.

- *LCA results - Conclusions*

A modern design of buildings should integrate aspects related to their sustainability in addition to the economic, safety and functionality requirements. Within the purpose of the study a series of boundary conditions have been set according to the scope and goal of the LCA analysis. The conclusions of the study can be summarised below:

- in all the stages the global and individual main category scoring of steel structure is significantly smaller than that of traditional house;
- a proven advantage of steel housing is represented by steel recyclability which brings positive impact at the end-of-life stage. This is in contrast to the modern (hollow) bricks that cannot be recovered once that they are plastered;
- the maintenance process is however more disadvantageous for steel dwelling, the change of thermal system includes also the change of all the wall layers. This is directly reflected in the environmental impact;
- as a main conclusion of the study, it could be stated that steel dwellings with cold-formed structural elements represents a good alternative to masonry houses, not only with respect to structural requirements but also in considering the environmental impact assessment.

Nevertheless, the above conclusions are drawn on the limits of the study described. Several other parameters (such as transportation or the operational energy) may change the result ratios in the comparison.

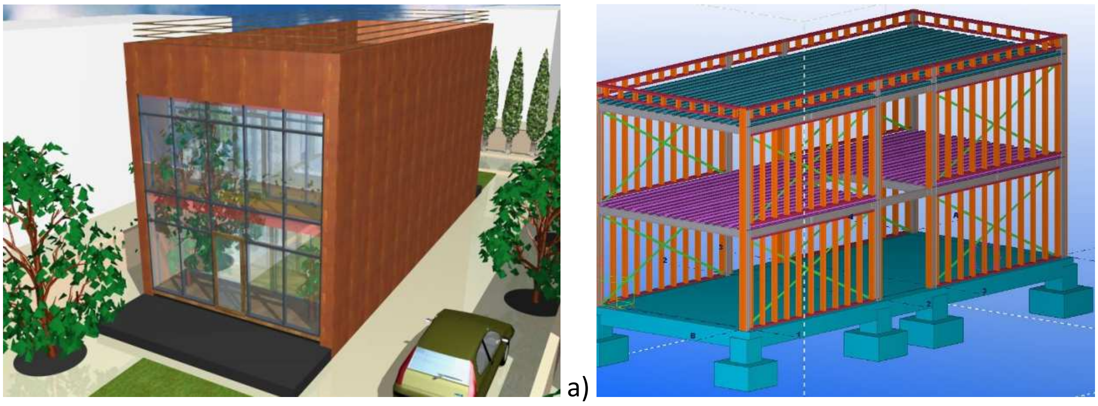
3.2.2. Case study II – Affordable house

In case of affordable housing, the main target of designers is to find a good relation between cost and comfort. Lately, a lot of new materials appeared on the market and are proposed by researchers and companies within this scope, in many cases without a real track record for their behaviour in time. The solution presented here relies on usual building materials, but applying innovative systems at several levels:

- the application of industrial building technologies in domestic housing (residential application) provides a fast erection and fabrication time;
- the system is modular in such a way that at any time the owner can add a new module, both by vertical and/or horizontal addition;
- the structural system made of lightweight steel frames (with hot-rolled profiles, cold-formed steel studs plated by Oriented-Strand Boards – OSB) assures the lightness of the house and the proper response to climatic and seismic loading.

- *Affordable house – system description*

The architectural concept relies on the development of a rectangular footprint of 5.60m X 13.40m, which gives a first module of 75m^2, for one level unit. In fact, the house represents a two storey building, with flat roof, resulting in a gross built area of 150m^2 (from which the usable area is 124.41m^2). Figure 127 presents the 3D view of the house, while Figure 128 presents the ground and upper floor plans.



a) b)

Figure 127. 3D view of the house: (a) architectural view; (b) structural layout.

The construction system shown in Figure 129 and consists of a hot-rolled framed steel structure with a secondary structure made from cold-formed steel studs system. The envelope could be realised by various systems (e.g. OSB plating). The floor is realised on light concrete topping on trapezoidal steel deck. A characteristic of the house is the double glazed loggias with PVC or aluminium frames. Isolated foundations and the ground slab are used from cast in place reinforced concrete. The system could offer the choice for flat or pitched roof.

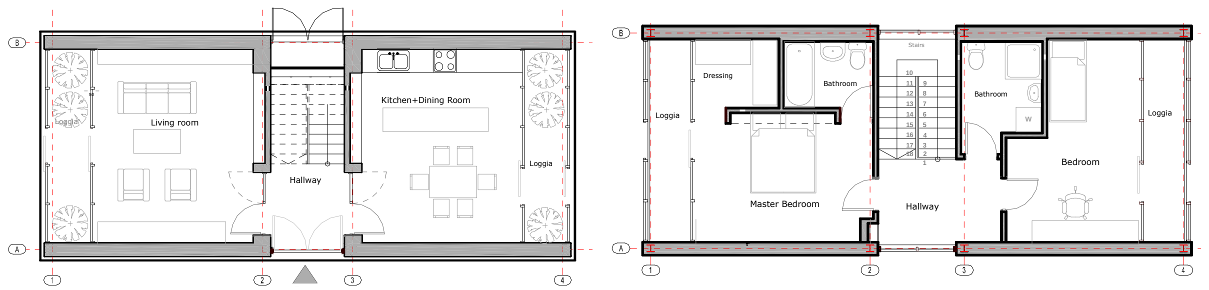


Figure 128. The ground floor and the upper floor.

The achievement of thermo-energetic efficiency was another goal set by the design team. Several factors were taken into consideration such as: (i) indoor temperature and air quality; (ii) thermal insulation of walls and floors; (iii) moisture protection; (iv) different heating and cooling systems; (v) passive ventilation, shading and buffer zone by the use of double glazed loggias and (vi) skylights used to enhance cross ventilation in case of one level house.

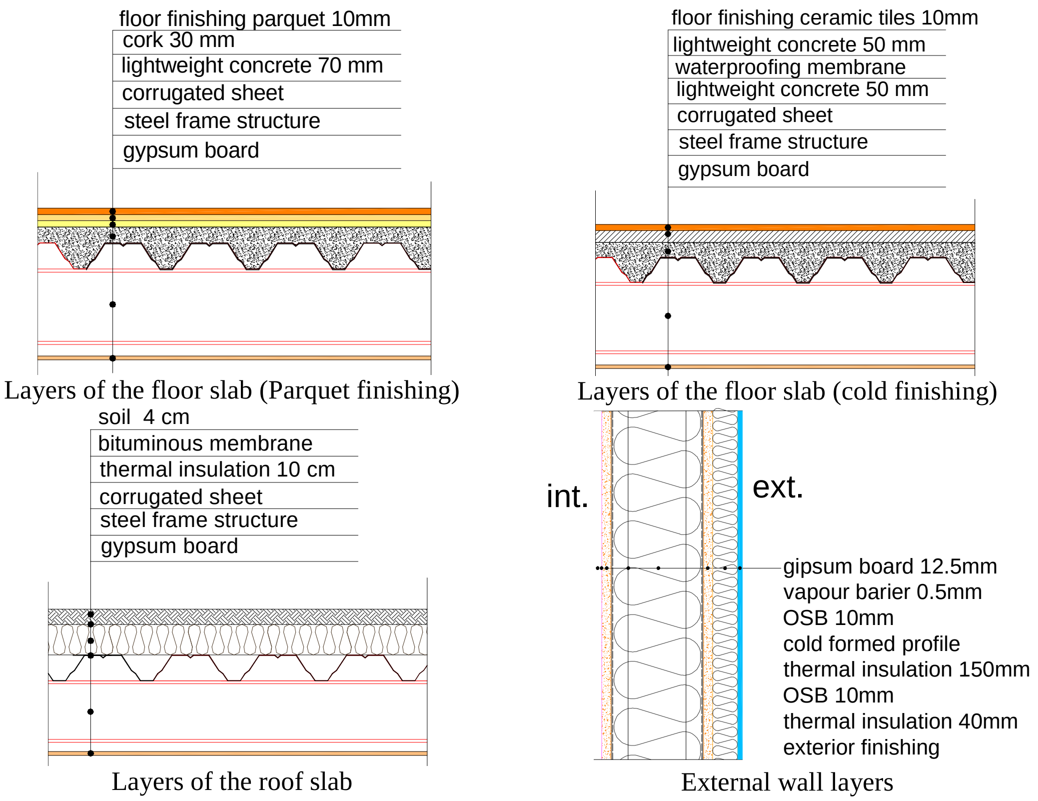


Figure 129. Main structural components of building with steel structure.

For a fast fabrication and erection of the structural and non-structural system, a steel framing solution was chosen like in case of larger buildings. The main structural skeleton is made by hot-rolled profile (European profiles HEA for columns, IPE profiles for beams). Transversally there are disposed four moment-resisting frames (two end frames and two intermediary frames), linked longitudinally by shear walls to resist horizontal forces from wind and earthquake. Columns are fixed in the base on transversal direction and pinned longitudinally. The transversal girders are semi-rigidly connected to columns while the longitudinal girders are pin-ended. The horizontal longitudinal loads are overtaken by shear walls made by cold-formed steel studs sheeted with OSB of 10mm on both sides.

The design of structural elements was made according with EN 1993-1-1, EN 1993-1-3, EN 1993-1-8 and Romanian seismic design code P100-1/2006, both for ULS and SLS. In order to control the floor vibrations an allowable deflection of L/350 was considered.

In what concerns the seismic response, the performance of shear walls is crucial. Test results of full-scale wall panels, made by cold-formed wall-stud skeleton and different cladding arrangements, commonly used for residential buildings, tested in the CEMSIG Laboratory at Politehnica University of Timisoara (Fulop et al., 1 and 2) have been used as reference values to check the available shear strength of walls. In this way the secondary elements bring their contribution. Figure 129 presents the components of the main structural elements.

- *Affordable house – environmental impact*

The modern approach of design should include information about environmental impact of buildings. In this purpose, the next step in design is a comparative life-cycle analysis of the above-described house for various solutions. In this purpose the building was designed in four different solutions, each of them having its own building system as follows:

(1) *Hot-rolled steel frames:* moment resisting frames on transversal directions, while on longitudinal direction the structure is stiffened by cold-formed shear walls sheeted with OSB. The floor structure is made by light concrete topping on trapezoidal steel deck. The infrastructure is composed by isolated foundation connected with foundation beams.

(2) *Cold-formed steel framing structure*. In this case, the construction system consisted of:

- isolated foundations under columns linked by foundation beams and a concrete floor slab;
- cold-formed steel profiles for the framed skeleton structure (C150 cold-formed sections);
- secondary structure made of cold-formed steel studs (C150 profiles spaced at 600mm) on which OSB is laid at interior and exterior, in order to stiffen the steel framing for lateral loads including wind and seismic loads;
- floor structure made of OSB on trapezoidal steel deck;
- envelope system and double glazed loggias with aluminium frames;
- flat roof (thermal and hydro – insulations) laid on trapezoidal steel sheeting.

(3) *Wood framing structure* which is realised on the same principle as the cold-formed system. In this case the wood skeleton is extended to roofing. The secondary structure and envelope is built on the same principle as in the case of the cold-formed house.

(4) *Masonry structure.* This is the classic Romanian solution of dwelling, with hollow-brick walls of 25 cm thickness and polystyrene thermal-insulation. The infrastructure system is assured by continuous foundations under the walls and a solid concrete slab. The flooring and roofing is also a reinforced concrete slab of 13 cm.

All four structures have been designed following the above-described architectural plans. For all of these building systems a detailed analysis was performed and complete lists of materials were derived. The design considers the same location of the building and the same imposed and climatic loading. The buildings were designed in such a way to achieve similar indoor environment. The layers for the main components (exterior walls and intermediate floors) are presented in Figure 130.

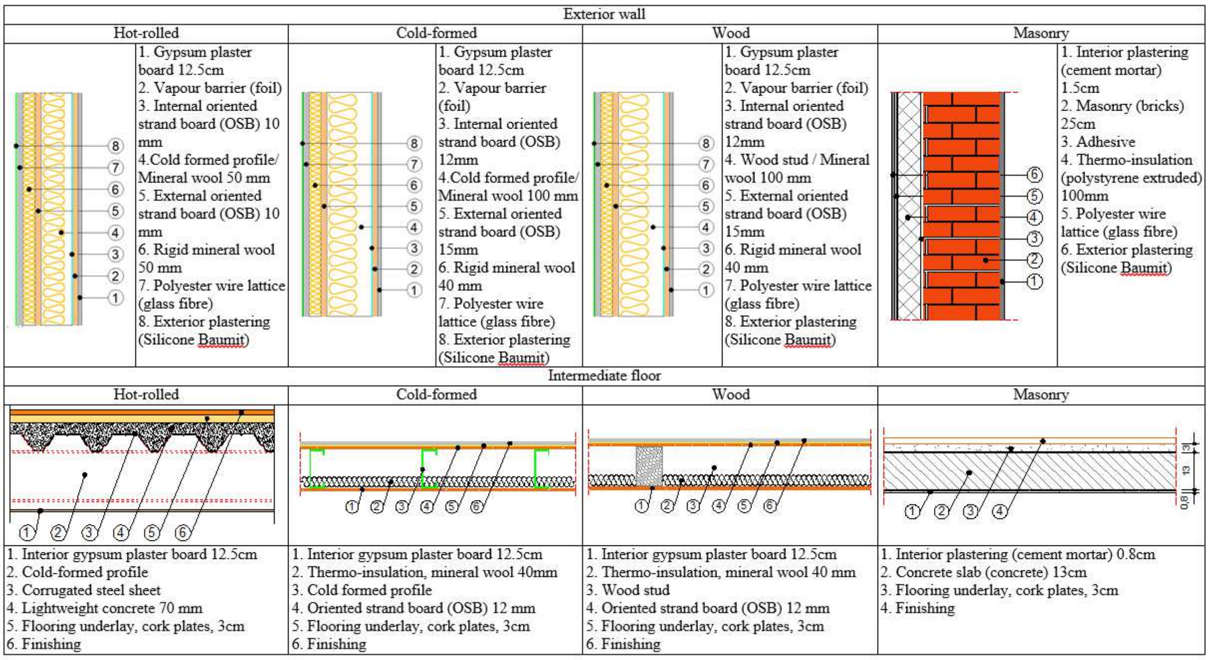


Figure 130. Main components of the structure (examples): exterior walls and intermediate floors

The comparative LCA analysis was performed using the SimaPro tool. In analyses there were included the material production, construction, end-of life of materials as well as a maintenance scenario for a life-time period of the house of 50 years. In order to set the input elements (inventory), both for simplifying the model and time-saving, the inventory analysis has been done according to system boundary conditions, in which several aspects were considered:

- the electrical and heating systems were left out of comparison as they bring the same impact in the analyses;

- transportation was not taken into account, although the values (especially the weights) are different from system to system;

- domestic use of the building (water / gas / electricity use), was not integrated herein by considering that these values are similar;

- energy used for construction purposes (such as cranes and other technological machinery) was not integrated in comparison.

- *Affordable house – Life Cycle Inventory*

For LCA analysis, the constructive elements were considered according to the design material lists and stratification. In order to have an easier input of constitutive materials in the analysis tool, they were gathered in assemblies, as listed below:

- constitutive materials on infrastructure: concrete and reinforcing bars in foundations, gravel, sand layer, polyethylene foil, extruded polystyrene, concrete and reinforcing bars for ground floor slab;

- constitutive materials on superstructure – according to the construction system: (1) hot-rolled profiles and light concrete topping on trapezoidal steel deck; (2) cold-formed steel members for studs and trapezoidal steel deck; (3) wood for studs and floors; (4); masonry and concrete floors;

- materials considered for secondary structure: cold-formed steel studs only for the house made by hot-rolled profiles;

- materials integrated in enclosures: OSB interior and exterior sheeting, thermal insulation for internal and external walls, hydro-insulations (polyethylene foils) etc. for wood and steel solutions; thermo-system for masonry house;

- materials used for finishing: finishing on exterior (stucco) and internal (acrylic paint) walls, gypsum plasterboards on walls and ceilings or internal plastering, interior doors, glazed walls, internal cold (ceramic tiles) and warm (laminated flooring) floor finishing etc.

For each structural system, the material quantities for each assembly were derived from the structural design or geometric data of the house. A brief image of the main building material weights used is shown in Figure 131.

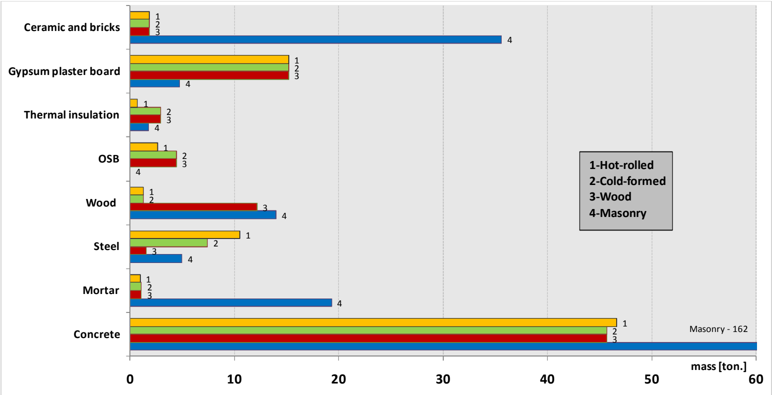


Figure 131. Calculated quantities of the main materials for construction stage.

- *Affordable house – Maintenance scenario*

Regardless the chosen constructive system, a building needs maintenance works during its life-cycle. These works could be of different types and in consequence could be more or less expensive and represent a very important aspect of the building life-cycle. In some cases, the building materials could be changed several times during the building life.

The integration of maintenance works for a structure is difficult to make in the initial stage, because the predictions that could be made in advance may not correspond to a future reality. However, in order to complete the life-cycle of buildings under consideration, the following prediction (considered as a pre-planned maintenance) was made, for each house, for a standard life-time of 50 years:

- 9 internal decorations (every 5 years);
- 4 changes of internal finishing (every 10 years);
- 4 changes of roof hydro-insulation (every 10 years);
- 3 external decorations (every 12.5 years);
- 3 changes for bathroom/kitchen sanitary: sandstone, sanitary furniture etc. (every 12.5 years);
- 1 change of the electric and heating system (every 25 years);
- 1 change of the roofing system (every 25 years);
- 1 change of the thermo-system (every 25 years).

In order to remain consistent with the previously explained boundary conditions, the main building material quantities used for maintenance were computed and represented in Figure 132.

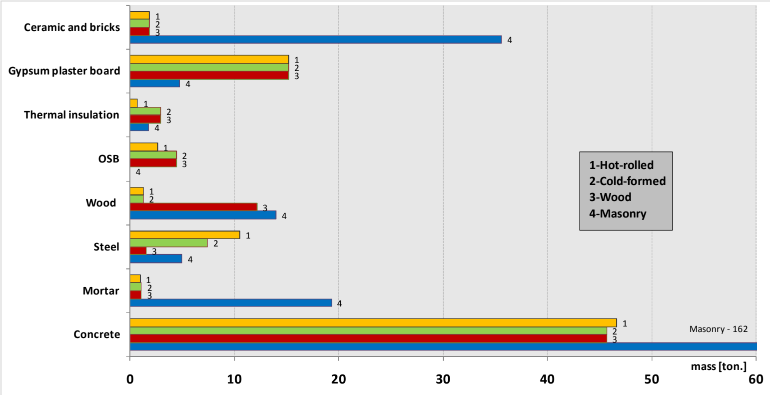


Figure 132. Calculated quantities of the main materials for maintenance stage

It is important to notice the fact that, in case of the steel and wood structures, only the skeleton remains unchanged, while all other elements are changed at least once in 50 years. In case of traditional house, the maintenance reduces here at the level of plastering for walls, thermo-system and part of the roofing system. No maintenance was considered for the infrastructure. However, in order to complete the life-cycle assessment, the same conditions for disposal at the end-of-life have been considered for both construction and maintenance stages (see Table 18).

Table 18. End-of life for building materials.

Building material	Reuse [%]	Recycling [%]	Burn [%]	Landfill [%]
Steel – steel profiles, steel tiled sheets	---	100	---	---
Steel – reinforcement	---	80	---	20
Bricks, ceramic tiles	---	---	---	100
Structural timber – wall studs	20	---	80	---
Timber for formworks	60	---	40	---
OSB	40	---	60	---
Ballast	80	---	---	20
Concrete, mortar	---	---	---	100
Other inert materials	---	---	---	100
Other combustible materials	---	---	100	---

- *Affordable house – Environmental Impact for the Construction Stage*

The environmental impact assessment was made by considering the above input data for construction phase but disregarding the common materials and processes according to the boundaries described in the previous paragraph. All the results are given in “eco-points” (Eco-indicator’99) in order to have unitary and comparable outcomes. The method used for impact analysis is Eco-indicator’99.

Considering an environmental impact analysis on different impact categories, the highest impact values result for the masonry home, as could be seen in Figure 133 – left. This fact is also confirmed by the single-score result (see Figure 133 – right), in which the best impact (2364 points) is obtained in case of cold-formed house, about 40% smaller than the global score of the masonry house (3864 points).

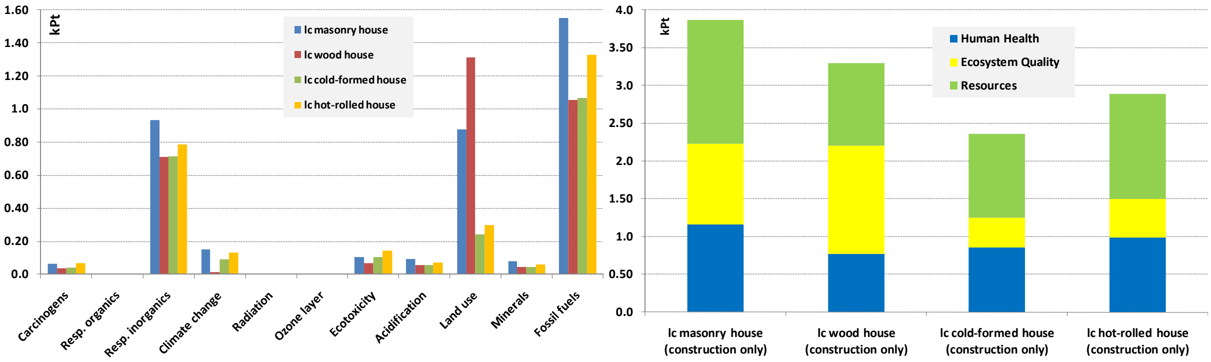


Figure 133. Comparison on environmental impact for the construction stage only: weighting and single score respectively

It must be noticed the fact that for all building systems, about half of the total score is given by the fossil fuels, used generally in processing of materials. In case of wood and masonry house a large amount of impact is due to the land use mainly tributary to the wood quantity employed. With this exception, all the impact categories are led by masonry home.

- *Affordable house - Environmental Impact of the Construction Stage and End of Life*

The life-cycle comparison for construction and disposal scenario conduct to similar results to those for the construction stage only (comparison of Figure 134 with Figure 133). Ranked in a single score (see Figure 134 - right) it results that the masonry house affects the environment about 1.3 times more than that constructed on hot-rolled steel or wood skeleton, and 1.6 times more than that constructed on cold-formed steel, respectively. The same impact categories (fossil fuels, respiratory inorganic substances and land use) bring the greatest impact.

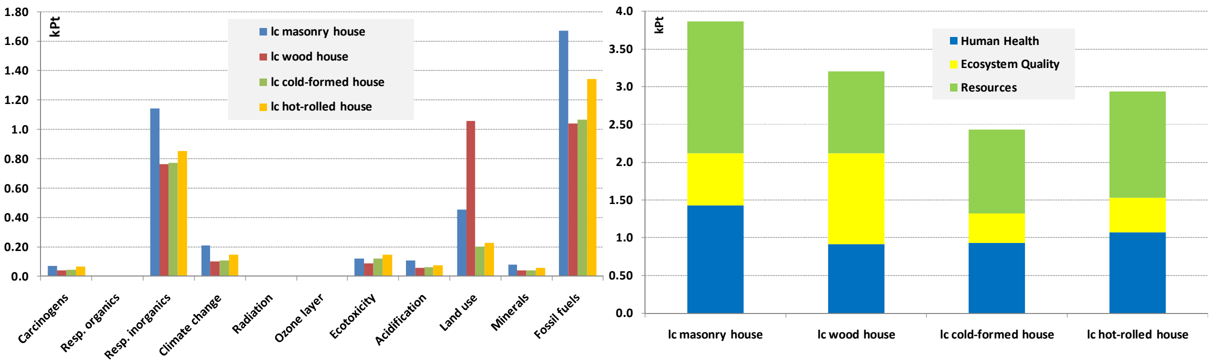


Figure 134. Comparison on environmental impact for the construction and end-of-life stage: weighting and single score respectively

- *Affordable house - Environmental Impact Considering Maintenance*

The final scores for the maintenance stage vary in both amount and distribution for different impact categories than those obtained in the construction stage or end-of-life. In terms of impact categories, the same categories (fossil fuels, respiratory inorganic substances and land use) integrate more than 85% of the total scores, as shown by Figure 135 - left.

All the values resulted from maintenance only and, cumulated in a single score (see Figure 135 - right), lead to small final differences for all the four systems considered. However, there could be noticed a difference from 3500 eco-points in the case of metallic and wood houses to 3300 eco-points for the masonry dwelling. This is somehow in contradiction with the values derived for the construction process, where the ratio is reversed in the favour of the metallic houses. The explanation for this derives from the facts that for the steel and wood houses, the wall, floor and roof layers are totally replaced (practically remain only the steel/wood skeleton) while in the case of traditional house, the structure (bricks, concrete) remains as built. Moreover, it could be easily observed that maintenance plays a major role in terms of environmental impact.

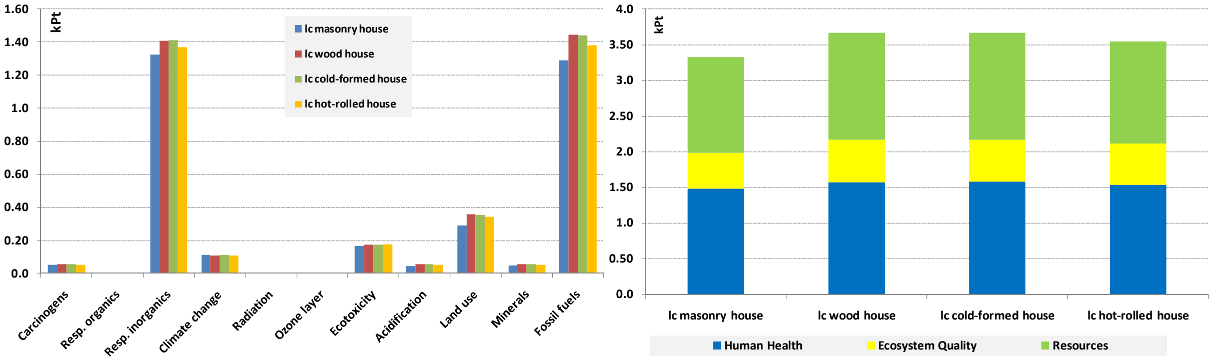


Figure 135. Comparison on environmental impact for the maintenance stage only: weighting and single score respectively

- *Affordable house - Environmental Impact for Life-Cycle*

Figure 136 presents the scoring of all the stages of the life-cycle of buildings as a direct sum of the construction stage, the maintenance of building for a life-time period of 50 years and the disposal at the end-of-life for the constitutive materials.

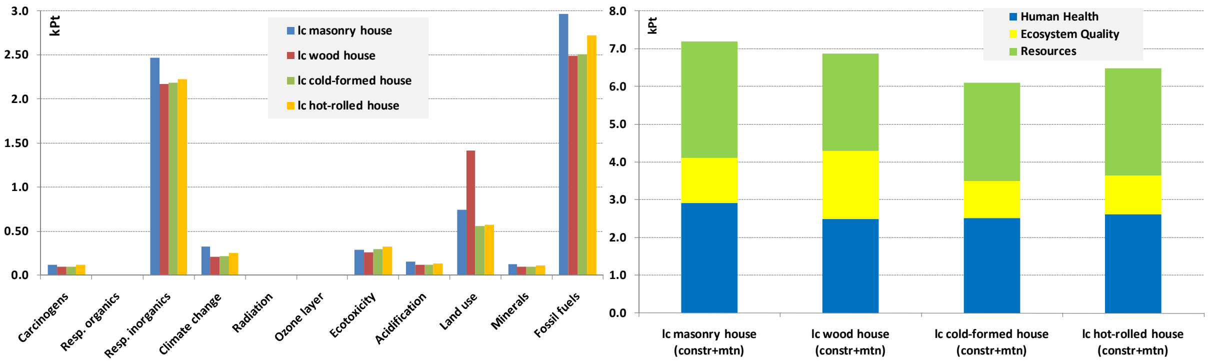


Figure 136. Life-cycle comparison on environmental impact (single score) for construction, including maintenance and end-of-life: weighting and single score respectively

As a general trend, the following impact categories are most affected, as Figure 136 shows:

(i) fossil fuels and respiratory inorganic substances, climate change: mainly due to the manufacturing processes which require large quantities of energy, which further on affect directly the fossil fuel reserves. These processes contribute in high extent to the emissions of inorganic substances and climate change gasses;

(ii) land use: due to damages generated to the land (wood exploitation, ballast pits etc.).

Generally, the differences obtained in the life-cycle analysis including maintenance follow the trend observed in case of construction stage only, namely almost all the environmental impact categories are greater for the masonry house. In a single score analysis (see Figure 136 - right), and taking into account the boundary conditions as explained before, the steel houses (6096 and 6481 eco-points, respectively) present an important advantage in front of masonry house (7192 eco-points), while the score for the wood house is about the average value corresponding to the impact of the other three solutions. Of course, many parameters (such as national or regional peculiarities, climatic zones or distance from the material distributors) may affect these results. From this point of view one has to observe the trends and not the values given by the analyses.

- *Affordable house - conclusions*

The life-cycle impact analysis performed shows the following aspects:

- the steel framing solutions (both hot-rolled and cold-formed framings) represent a good alternative to the classic masonry house, both from safety and sustainability points of view;

- all the framing solutions present a better environmental impact for construction stage and life-cycle analyses on building materials in comparison to the classic masonry house;

- the maintenance process of steel and wood solutions is more complex than that of masonry house. More, it could be easily observed that maintenance plays a major role in terms of environmental impact;

- there are two impact categories which lead the global impact score of analysed systems: (i) fossil fuel due mainly to the processing of raw building materials and (ii) land use, in case of houses using much wood in building process. In fact these impact categories show practically the directions that should be followed for achieving a limited environmental impact.

3.2.3. Case study III – Multi-storey residential building

This case-study represents a block of flats built in 2007 in Timisoara, Romania. The keys for this kind of structure are built-in flexibility and energetic efficiency. The main structure is made of steel profiles with light floors. Column-free and free floor slabs are the optimum answer to allow users to optimally reconfigure internal areas and this generally means long-span solutions. Some innovative design solutions have been used in this project, i.e. thermo-energetic cladding system, gas-electric energy supply system etc. An intensive study has been carried out in order to choose the correct cladding solution. Several types of cladding systems have been analysed, i.e. walls with or without cavity. Based on the chosen solution, interesting data related to the performance of cladding system have been collected during the 2008/2009 autumn/winter season.

- *Multi-storey residential building – system description*

Architectural views, structure during erection and final view of the erected building are presented in Figure 137. The keys for this kind of structure are built-in flexibility and energetic efficiency. Column-free and free floor slabs are the optimum answer to allow users to optimally reconfigure internal areas that means long-span solutions.

The builder/developer aimed at providing superior levels of comfort at a reasonable cost. Considering this requirement, the design was directed to fulfil three main objectives: (1) to minimize heat loss through the envelope; (2) to ensure high levels of physical well-being; (3) to equip the building with an energy saving heating system.

Figure 137. Structure during erection and final view of the erected building.

The city of Timisoara is located in a moderate seismic risk region. In what concerns the climate, Timisoara city is located in the temperate continental moderate climate region which characterizes the Southern-Eastern part of the Panonic Depression. General climatic features consist of various and irregular weather conditions. The average annual temperature is of 10.6°C while the hottest month of the year is July (21.1°C). Being predominantly under the influence of North-western maritime air masses, the precipitations that occur in Timisoara are far more numerous than those from the Romanian Plain. The average 592mm annual amount is reached due to the rich May, June and July precipitations (34.4% of the total yearly amount).

Given the wide variations of seasonal temperature levels, as described above, and the cumulated effects of: overheating of the south and west facades in summer and heat loss due to the wind-chill effect on the north / north-west sides of the facade in winter, special attention was paid first of all to the passive energy saving measures.

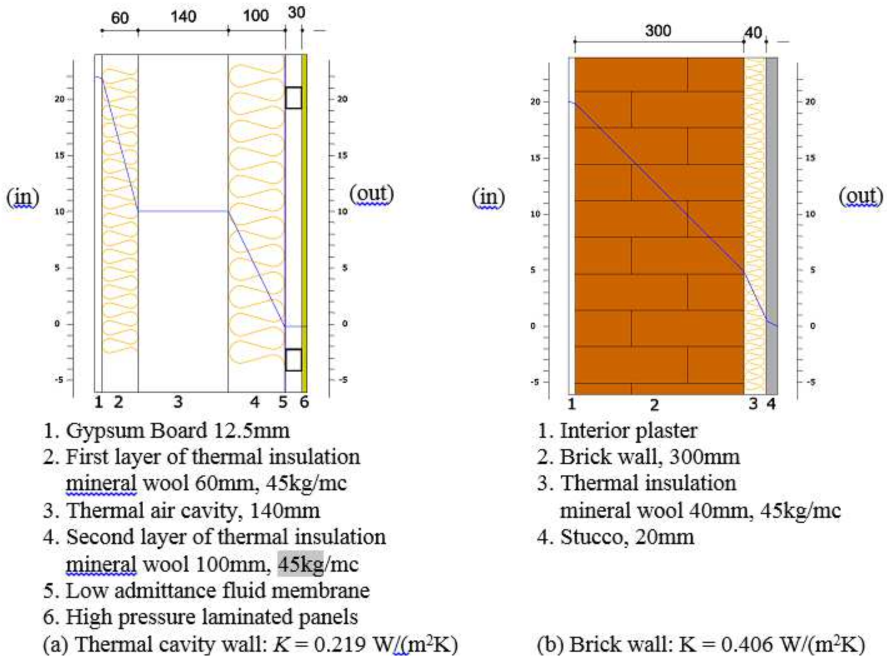


Figure 138. Importance of adequate insulation.

The envelope design was rationalised, as permitted by site conditions and functional parameters:

- glazing was essentially restricted to the short facades [east and west], protected from the afternoon sun by deep loggias. Windows and exterior doors are double glazing with stratified wood framing;

- the long facades, facing North and South are mostly solid envelope, conceived as a thermal cavity system wall.

Figure 138 – left presents the actual layers for the cladding (in/out). Figure 138 also illustrates the importance of adequate insulation both as thickness and position in the wall assembly; the comparison was made with a brick wall. The combined effect of insulation thickness 60+100mm and the thermal buffer produced by the 140mm air layer, provides a high level of insulation for this climatic zone, both in summer and winter (K = 0.219W/m^{2}K). By comparison, a brick wall with 60mm insulation, has K = 0.406W/m^{2}K.

Figure 139 presents some pictures with the envelope during erection. The materials used in the building store moisture for a very limited period of time. Thermal insulation (basaltic mineral wool with density of 45kg/m^3) allows for constant vapour migration. In order not to trap the moisture in the rooms, the vapour barrier layer under the gypsum board was eliminated, allowing for the free vapour migration through the wall to the exterior. Given the gradual migration of vapour through the thermal cavity wall, the conditions for condensation are practically eliminated.

Figure 139. Envelope during erection.

In order to ensure a high level of physical well-being for the occupants, the following set of conditions has to be kept under control:

- control of average surface temperature of enclosing elements and room temperature;
- control of relative humidity and room air temperature;
- control of floor and roof temperatures;
- control of air movement around occupants and room temperature;
- control of acoustical influences.

The high level of thermal insulation combined with the moisture control benefits of the thermal cavity wall, address in a satisfactory manner the set of control measures enumerated above.

The solution chosen for the heating system tries to make use of the energetic performances of the building, not only by means of production and distribution, but also by the heating time (is important to heat up only what we need and when we need). The technical solution consists in the production of the thermal agent in a gas heated boiler and, in the same time in a CHP (combined heat and power unit). The distribution is made exclusively through the interior of the building, and the dispersion of the heat is done by convectors placed in the ceiling, which ensure a massive heat exchange (heating or cooling), in a short amount of time. The use of the CHP unit, which simultaneously produces hot water at 90°C and electricity, leads to a substantial reduction of costs, as the in-house produced electricity is cheaper than the electricity available from the distribution network. The hot water is stored in a tank, that can use thermal agent from the boiler/ CHP/ boiler + CHP/ solar panels/ heat pump/ electrical.

Some areas in the vicinity of the windows or the floors in the bathrooms are fitted with an intelligent electrical heating system, integrated in the floor. This has the advantage of being cost efficient, safe in exploitation, flexible in configuration, and can be controlled via the internet.

In order to prove the thermal capacities of the building, ambient measurements were taken at the beginning of 2009 over a period of about two months (January and February), the coldest for this location, considered as indicative for the whole period in which the building is heated. Two sets of temperature reading sensors were placed on the North and South facades of the building, in order to measure the interior, the wall cavity and the exterior temperatures. The positions of the sensors correspond to the living room area of the apartments, with a volume of approximate 120m^3. It has to be added that at the time the measurements were taken, the apartments were not occupied and, as a result, the contribution of human produced humidity in the room was not present.

The measurements have shown that moisture content in the building during the heating season tends to be low as long as no fresh air supply is provided. During the heating period, humidity levels in the building rise to 30-34% after short natural ventilation periods. When inhabited, the humidity level is adjusted to reach 45-55% at 20°C, through natural ventilation, human produced humidity and/or with the help of humidifiers if required. It becomes evident that by removing the vapour barrier under the gypsum board, vapour migration through the envelope is accelerated. This factor combined with the mineral wool characteristics (of not storing moisture) are the key elements for moisture control. The wall cavity has been continuously monitored. After two winter seasons with high variations in temperature and humidity, no particular problems were recorded.

- - *Multi-storey residential building – Evaluation of environmental impact*

The environmental impact for the block of flats was performed at the level of construction stage only. The analysis was performed using the SimaPro software, a general and comprehensive tool, widely used in environmental design and LCA, which uses the Ecoinvent database. As mentioned before, in the analysis were included the material production and construction stage. The inventory analysis has been done according to the system boundary conditions. According to this, several aspects were considered:

- no finishing were taken into account (for example wall painting, the floor finishing, doors, windows and electrical or heating system);
- the transportation was not taken into account;
- the domestic use (water/gas/electricity use) of the building was not accounted for;
- the energy used for construction purposes (such as cranes and other technological machinery) were not integrated in the comparison.

For the environmental impact calculation of the building, the input materials have been considered according to the constructive elements: (1) exterior walls; (2) interior walls; (3) intermediate floors; (4) terrace; (5) infrastructure.

In order to have an easier input of construction materials in the LCA tool, there have been computed average values for the weight of materials. These have been estimated for each type of constructive element as follows: the total weight of materials (resulted from the material lists) was divided to the total area of constructive element (in sqm). In this way, the final result represents an aggregate average per square meter of constructive element. This represents in fact the inventory analysis.

The following figures present the environmental impact by considering the above input data for construction phase but disregarding the materials and processes according to the boundaries described previously. All the results are given in “Eco-indicator points” (Pt), which express the total environmental load of a product or process, based on data from a life cycle assessment, in order to have unitary and comparable outcomes. The method used for impact analysis is Eco-indicator‘99.

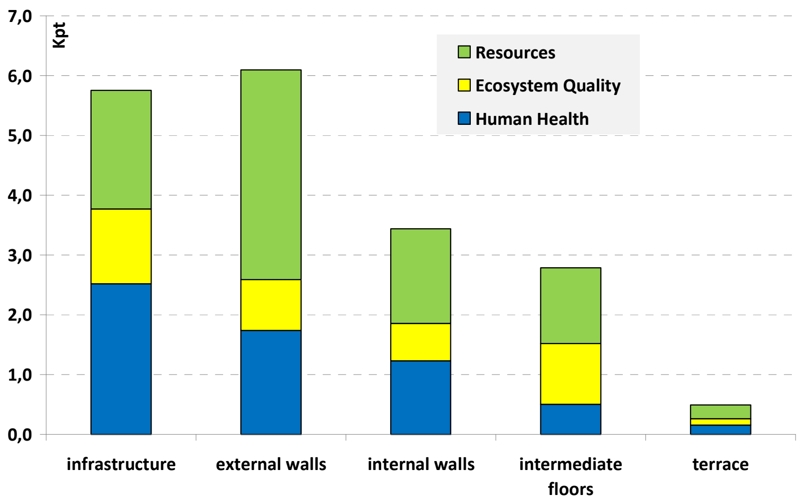


Figure 140. Environmental impact per constructive element.

Figure 140 presents the impact for the block of flats for the construction process ranked per constructive elements. The major impact corresponds to exterior walls and infrastructure. These constructive elements are high consumers of resources, but also have a great impact on human health.

Figure 141 presents the impact deduced only for construction stage. One could realise that for the structure the major impact comes from fossil fuels, as these resources are used for the fabrication of building materials at all levels. Also, important values of impact are recorded for inorganic respiratory emissions and ecotoxicity. The results presented above on impact (or damage) categories are aggregated into a single score (see Figure 141 - right), leading to an overall score of 18560 points.

Finally it can be observed that the major impact corresponds to exterior walls, followed by the infrastructure and interior walls. These constructive elements are high consumers of resources but also have a great impact on human health.

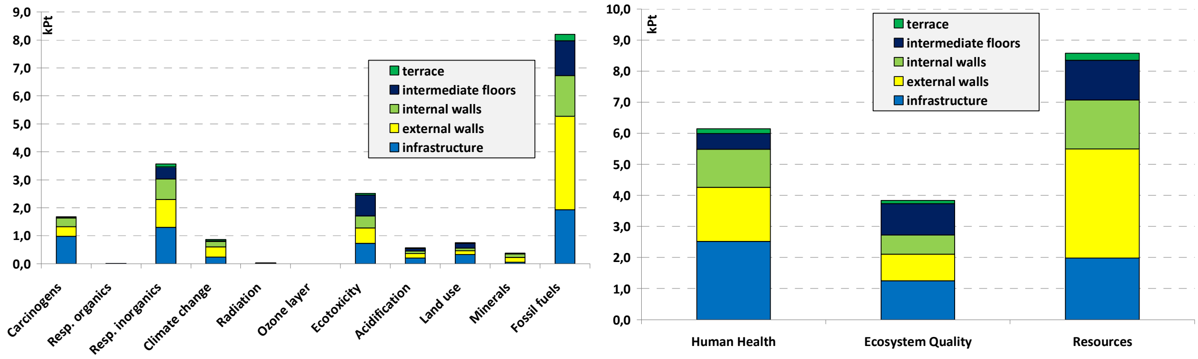


Figure 141. The environmental impact for the block of flats: weighting and single scores respectively.

- *Multi-storey residential building – Conclusions*

The building represents a complete sustainable technology of high performance thermo-energetic materials used for cladding and finishing. It enables to obtain flexible partitions and allows for further up-grade, easy modifications and/or development.

The steel main frame allows for: (1) high design and construction safety standards; (2) larger spans; (3) layout flexibility; (4) faster fabrication and erection times; (5) high solution diversity for flooring and envelope. In what concerns the physical well-being for the occupants, a set of parameters can be kept under control such as inside average temperature, relative humidity and room air temperature, air movement or acoustic insulation.

The environmental impact analysis shows that the major impact corresponds to exterior walls, followed by the infrastructure and interior walls. These constructive elements are high consumers of resources but also have a great impact on human health.

3.3. Retrofitting of existing building stock

3.3.1. Integrated design of existing buildings

The aim of the Integrated Design (ID) of buildings is to incorporate in usual architectural, structural and technical design additional requirements related to sustainability. Hence, the ID of buildings maximizes the overall life-cycle response through structural, economic and environmental performances. The way in which the social (structural, comfort, architectural etc.), economic and environmental performances are assessed is rather a complicated matter due to the complex definition of a building, but, nevertheless, in literature it is largely accepted that ID of buildings is characterized by the following key attributes (Landolfo, 2012):

- it is a methodology oriented on the Life-Cycle Approach (LCA);
- the ID represents a multi-performance based design approach;

- the evaluation of safety and serviceability, durability, life-cycle costs and environ-mental impact is based on quantitative design procedure.

The main problem in performing an ID based on LCA is related to the long-life period of buildings in comparison with other products. The difficulties arise due to the uncertainties occurred during the use period and end-of-life.

The ID of existing buildings is even more complicated due to the restrictions imposed by the existent building conditions. The sustainable retrofitting solutions should be adapted to the initial conditions of structures and adjust them in order to fulfil the updated regulations and requirements (Ungureanu, 2012). As a consequence, the modern design regulations for structural and thermal retrofitting of buildings integrate new concepts in the evaluation and design of buildings: design performance objectives, acceptance criteria linked to performance level, sustainability issues and analytical techniques for performance assessment. The rating systems (LEEDS, BREAM, SBTOOL etc.) could offer a measure of building performance both prior and post-retrofitting. However in case of existent buildings an ID solution should integrate both structural and thermal resistance aspects. It is obvious that independent interventions are deficient. An integrated retrofitting solution should be based on the following basic criteria (adapted after Dubina et al., 2008) – other specific criteria can be added in function of the specificity of the project:

A. Structural aspects:

- Capability to achieve requested structural performance objective (only after building structural evaluation);
- Solution compatibility with the actual structural system;
- Adaptability to change of design actions (including seismic if necessary);
- Adaptability to change of building partitioning.

B. Technical and comfort aspects:

- Reversibility of intervention;
- Durability;
- Operational;
- Comfort (thermal, phonic, space);
- Functionally and aesthetically compatible and complementary to the existing building;
- Technical support (Codification, Recommendations, Technical rules);
- Availability of material/device;
- Quality control.

C. Economical aspects:

- Costs (Material Fabrication, Transportation, Erection, Installation, Maintenance, Preparatory works).

D. Environmental aspects:

- Measures to lower the operational energy;
- Use of ecological and friendly materials.

However, in many interventions there are additional issues that may be incorporated, depending on the project specificity. The integration of specific aspects related to safety, comfort and sustainability in the retrofitting design of old buildings is often done under the form of margins of safety through which the design intervention should be judged. During last years, important steps were made in development of quick methods in establishment of buildings vulnerabilities. However, methodologies for integrated design (for both new and old buildings) are usually complicated and hard to apply in practice.

Figure 142 presents the relation between the processes of evaluation, design and construction, as a conceptual framework applied to structural retrofitting of existing masonry and concrete residential buildings. It represents in fact an iterative process in any phase and it is oriented towards the achievement of desired performance level.

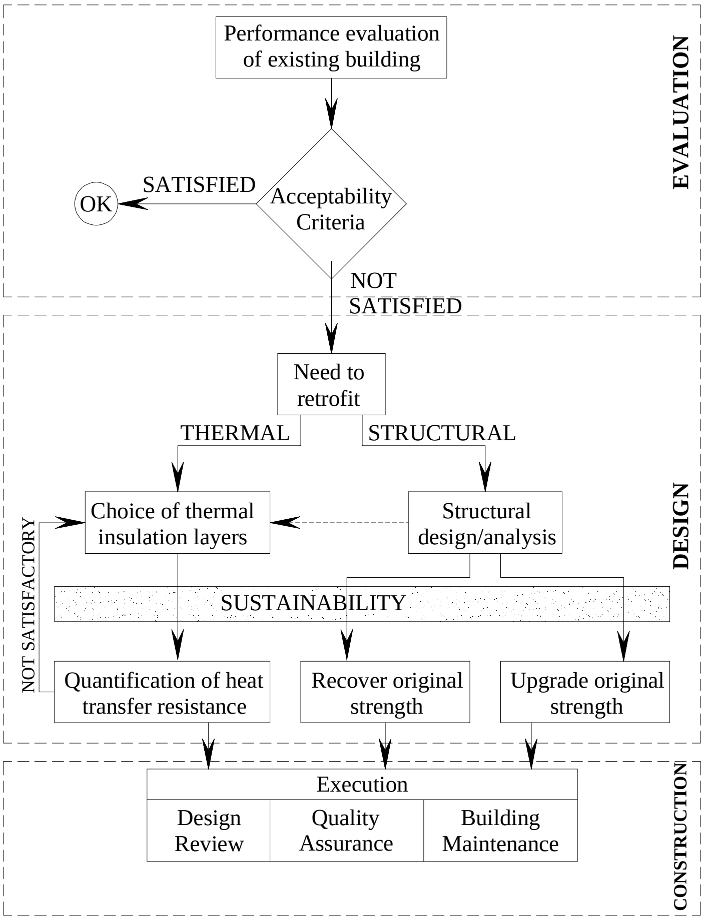


Figure 142. The environmental impact for the block of flats: weighting and single scores respectively.

The *evaluation* phase is based primordially on the criteria listed above and should have a decisional purpose. The *design* phase could comprise one or both thermal and structural aspects, but in any case, the given solution should be an integrated one: the retrofitting solution must be compatible mechanically, physically and chemically) with existing structural system / envelope. In this phase, the sustainability should act as a filter in decision taking.

In a normal process of *construction*, the execution represents only the basic purpose. Design reviews should be considered in each case in which the design solutions are not applicable to site reality while the quality assurance should confirm the conformity with the project. The life-through building care should be assured through maintenance: repairing, renovation or restorations.

It is obvious that the final retrofitting solution represent a multi-criterial issue. The design solutions for walls should consider features that assure integrated solutions such as (adapted after Plank & Dowling 2003):

- assurance of required levels of structural performance;
- assurance of thermal and moisture protection;
- design for demountability and use of recyclable materials;
- use of eco-friendly materials;
- select responsible contractors who have embraced sustainable development principles;
- design for flexibility to extend building lifetime;
- use the whole-life thinking.

In case of building walls, the retrofitting solution should be considered in at least three parameters, based on the three sustainability pillars:

Environment ---> use of eco-friendly materials in a life-cycle assessment

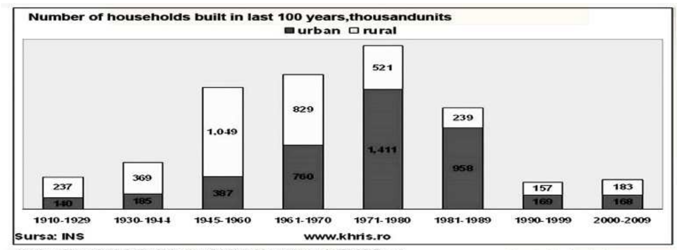
Economic ---> life-cycle costing
Social ---> assurance of safety and comfort conditions through modern design norms

Similar to Life-Cycle Assessment methods, the final results could be aggregated into final scores, by crediting each parameter (n.b. Environment / Social / Economic) with different percentages or impact factors. Although it represents in fact a controversial procedure, once the parameters are set the final decision could be taken in an engineering way.

3.3.2. Romanian building stock

According to the Romanian national census (INSSE, 2003) in 2002, the building stock in Romania com-prised a total of 4,834,063 buildings. From these:
- 4,605,412 were individual buildings (dwelling type);
- 129,893 – coupled type buildings and
- 83,799 as block of flats (apartment house type).

The last typology, although it represents only 1.7% from the total building stock, they are housing some 7,821,169 people, which is more than one third of the total Romanian population. The major part of these buildings is located in urban areas. According to the same census, the block of flats structures are built predominantly between 1960 and 1990 (68294 units) and have as primary resistance structures concrete (57431 units) and respectively various types of masonry (26368 units).



Number of households built with large prefabricated concrete panels "BLOCK" type

TOTAL	de 1910	1910-1929	1930-1944	1945-1960	1961-1970	1971-1980	1981-1989
57431	93	123	239	634	4822	22355	25352
24533	742	976	2341	3397	6667	6059	3497
81.964	835	1.099	2.580	4.031	11.499	28.414	28.859

Households	Area (m)	
8.071.912	303.300.454	RURAL
4.234.173	159.234.576	URBAN
3.021.122	105.245.605	BLOCKS
71.35 %	66.09 %	

Figure 143. Statistical data regarding the building stock according to the 2002 census.

The building system at that time was using standardised structural typologies, applied for a large number of block units all across the country (Niculescu 1961). Some advantages were derived from the use of large precast reinforced concrete panels, such as the industrialisation of the building process, short erection time, easy adaptation to all climatic loads etc. Over a period of time, the typologies of precast reinforced concrete buildings presented major differences 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) (BIT 24/1967);
- architectural criteria: surface (built square metres, inhabitable area, and usable area); built space, facades, space configuration, access definition (BIT 8/1968; 7/1969;6/1970; 4/1971);
- energy demands and CO2 emissions due to the various finishing materials and the stratifications of the envelopes;
- finishing criteria: (thermal, water-proof and noise insulation);
- engineering: (differences in structural and seismic performance according to the period when the prefabricated units were erected).

The collective dwellings in Romania made of large precast reinforced concrete panels were executed in three major stages and different typologies were used depending on the state decrees issued at that particular time, as well as due to the evolution of design and of the urban systematization stages: 1932-1975, 1975-1982 and 1982-1989 respectively.

The major dysfunctions that concern these neighbourhoods, regardless of the structural and energy consumption capacity of the units, refer rather to their lack of retrofitting, the degradation of the urban aesthetics, the interior space partitioning, the relatively small public green areas and space among units. Moreover, in time, the urban space was diminished by inhabitants that turned large green areas into parking lots. Structurally, the reduced interior surfaces of the flats have lead the inhabitants to creating openings in the structural partition walls in order to redesign the interior space and also to extend their living area, by closing the balconies. Since the '70, several studies have shown the inefficiency of balconies and loggias in apartment buildings and the transformation, in terms of functionality, in residual spaces (e.g. storage areas). On the other hand, in many particular cases, the apartment extensions by means of closing existing balconies areas has proved inefficient in terms of the useful space and the living comfort.

3.3.3. Structural and architectural retrofitting

Apart of the external deficiencies of the inhabitable space such as location, accessibility, circulation and parking places, facilities and so on, the main deficiencies of large precast concrete collective living buildings are the *inadequate living space*, post-building interventions, and deficient thermal and hydro insulations, facts that lead to lack of comfort of the inhabitants.

The principal ways of updating the living spaces of such buildings is to include an integrated action of structural and thermal rehabilitation in order to retrim the buildings to modern interior acceptable levels. The reconfiguration of locative spaces with the primary purpose of extension can be done through:

- coupling (merging) of two apartments on horizontal;
- coupling (merging) of two apartments on vertical;
- overcladding.

The reconfiguration of internal spaces by practicing holes in their diaphragms must be done in such a way to keep the strength and stability of the overall structure in order to resist actual horizontal and vertical loads. This should be proven by detailed structural analyses which should demonstrate that the modified structure can overtake the design loads evaluated to actual norms.

The case-studies performed within the INSPIRE research grant have considered three typologies of precast collective building types with large prefabricated concrete elements and considered typical for the period of their construction:

- IPCT T744R project type, specific to the period 1962 – 1975;
- IPCT T770 project type, specific to the period 1975 – 1982;
- IPCT T1340 project type, specific to the period 1982 – 1990.

The study of optimisation of the internal spaces considered in total five interventions on structural systems, considering both methods (coupling of apartments and overcladding):

- coupling of apartments:
    - coupling of apartments on horizontal: structural system IPCT T744R
    - coupling of apartments on vertical: structural system IPCT T744R
- overcladding:
    - overcladding for structural system IPCT T744R with structural system of hot-rolled steel profiles
    - overcladding for structural system IPCT 770 with structural system of thin-walled steel elements
    - overcladding for structural system IPCT 1340 with structural system of hollow sections

The structural systems were analysed in several steps, in order to check the structural safety in different stages:

- analysis of the original structures: the main purpose of this analysis is to check the structural response under the actual loading norms. The checks of the elements were performed in accordance to norms specific to concrete elements subjected to gravitational and seismic loads;
- analysis on modified structures: in this case were considered the structures affected by openings in case if merging apartments or overcladded structures. Both seismic and gravitational loads are considered. As result, special zones with necessities of consolidation can result;
- analysis on modified and strengthened structures: they are necessary for the confirmation of chosen strengthening solutions and to ensure the composite action between additional strengthening elements and existing structural ones.

- *Analysis of original structures*

For all the three structural cases (IPCT project types 744R, 770 and 1340) the structural analyses have proven the structural abilities in overtaking the gravitational and seismic loads. The following norms were used for verifications:

- CR 0-2005 Cod de proiectare Bazele proiectării structurilor în construcţii SR EN 1991-1-2004
- CR 2-1-1.1-2005 "Cod de proiectare a construcţiilor cu pereţi structurali de beton armat"
- CR 1-1-3-2005 "Cod de proiectare. Evaluarea acţiunii zăpezii asupra construcţiilor"
- NP 082-04 "Cod de proiectare. Bazele proiectării şi acţiuni asupra construcţiilor.
- Cod de proiectare seismică – P100-1/2006 (P100-1/2011)

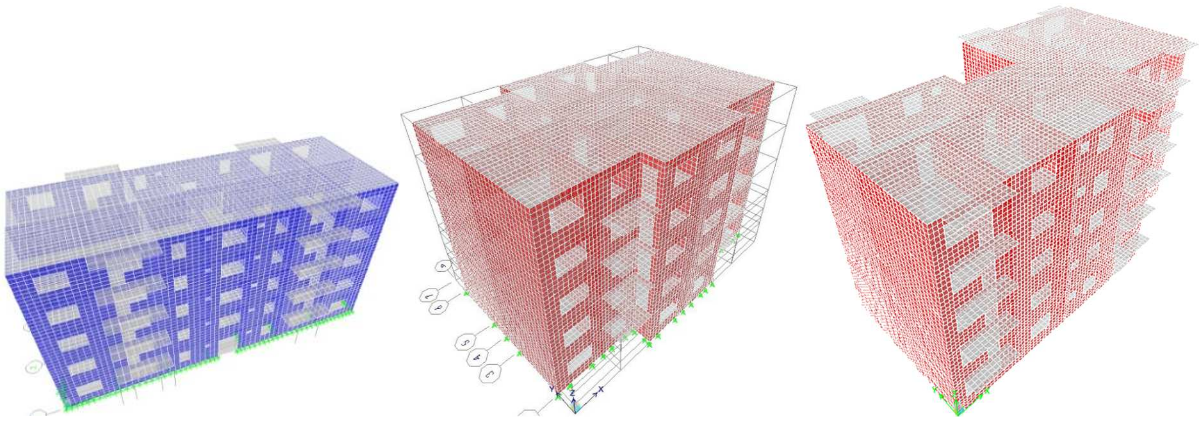
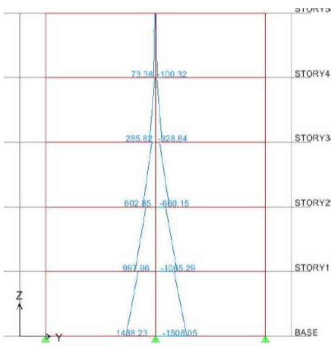


Figure 144. 3D shell models in ETABS for project types 744R, 770 and 1340.

The performance of the original structure was evaluated with the ETABS computer code, by using 3D analyses based on shell finite elements (see Figure 144). The thickness of the shell elements modelling horizontal and vertical diaphragms was equal to the thickness of the load bearing layer. 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 specific to Timisoara location. The first modes of vibration are translational, longitudinal and respectively torsional.

The main checks regarded the diaphragm panels subjected to maximum bending and axial loads as shown in Figure 145, as well as the panels in which openings will be created by carving. The diaphragm panel checks was complemented by the following checks:

- internal core web subjected to compression;
- design to shear;
- design of horizontal reinforcement in the joints;
- check of the reinforcing coefficient.



Nr.etaj	N [kN]	M [kNm]
Etaj 4	-219.98	100.32
Etaj 3	-464.78	328.84
Etaj 2	-742.73	660.15
Etaj 1	-1048	1065.29
Base	-1393.6	1508.05

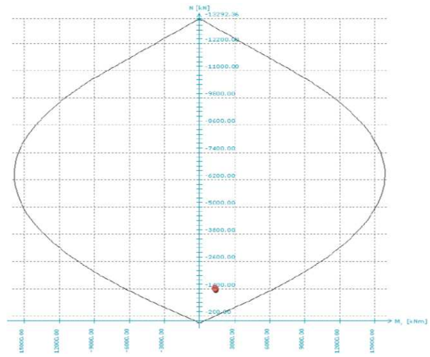
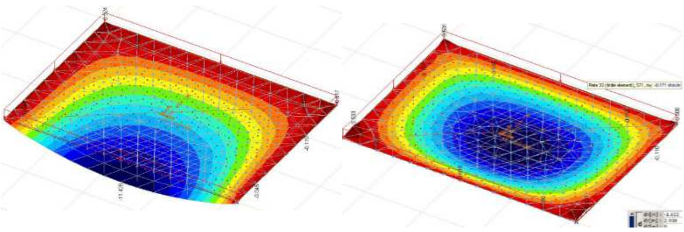


Figure 145. Bending moment diagram, maximum values of forces and check on N-M interaction diagram for diaphragm in Axis A for 744R IPCT project type.

The horizontal slab diaphragms have been checked to bending. Considering that one of the solutions of merging apartments relies on cutting slabs, for the installation of as internal staircase the concerned horizontal diaphragm panel was analysed (see 0).

The structural checks confirmed for all three cases considered in analyses that the elements correspond to actual norms, without supplementary strengthening. Moreover, they possess significant reserves of resistance.



Direction moment	Value moment [kNm]		Direction moment	Allowable Moment[kNm]
M_x	11,428	<	$M_{cap,x}$	11,48
	-0,524			
M_y	3,90		$M_{cap,y}$	6,23
	-0,53			

Figure 146. Bending diagrams and checks for diaphragm panel P42-21 for 744R IPCT project type

- *Analysis of modified structures*

The solution of merging two apartments on horizontal implies the perforation new openings in the existing structure, namely in vertical diaphragms of the living-rooms. In this way, the new configuration of the system will be with two apartments on level, and double living areas. The new openings are considered with a width of 135 0mm in the central diaphragm of axis B. The chosen solution in analysis implies the creation of openings at each level, solution which is considered as most dangerous in the eventuality of accepting this intervention.

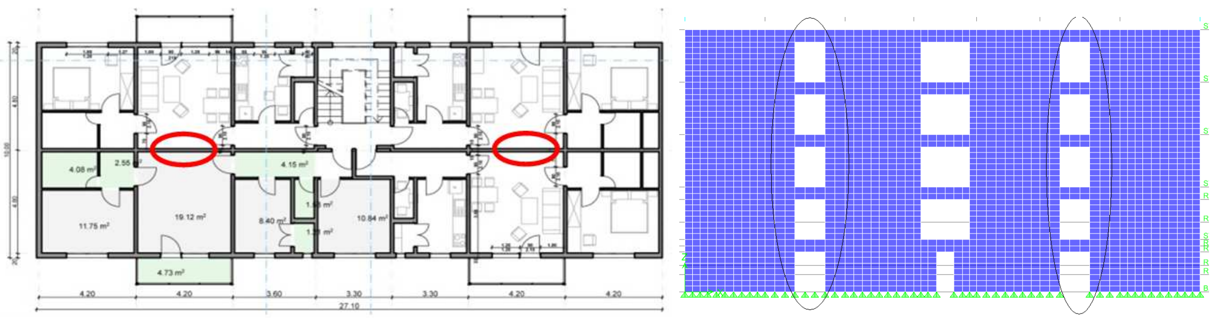


Figure 147. Detail of intervention (plan and elevation) – intervention in mentioned area.

The structural analysis on modified structure confirmed that the forces in the original diaphragms (without interventions) are similar to the values registered in the case of initial structure. However, important differences are recorded in the diaphragm axis 2, which accommodated the access holes, mainly in the seismic combination – see Figure 148. In this case the checks for local verification of the diaphragm web in compression, of transversal and horizontal reinforcement, as well as the reinforcement coefficient are beyond the safety limits. For this reason, consolidation measures were considered/

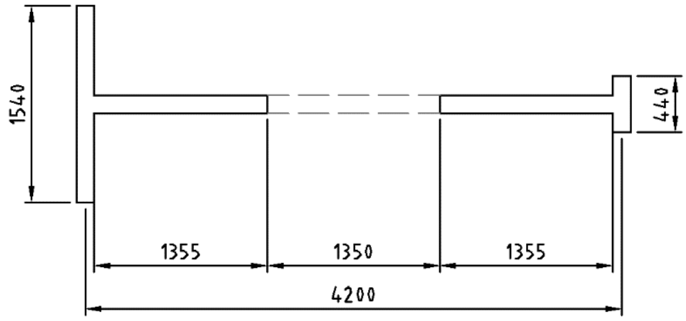


Figure 148. Diaphragm section after intervention.

There are two solutions that were proposed for local consolidation of diaphragm openings:

- considering steel elements bordering (see Figure 149 a): the solution consists of a steel frame of two columns and a connecting beam, made by “U” profiles which frames the opening. The profiles are linked to the concrete elements by connecting bolts passing from one part of the diaphragm to the other. The columns are considered pinned at the bottom part in concrete slab;

- by concrete jacketing (see Figure 149 b): the solution is based on composite action between the existing and new element. For the wall opening shown in Figure 148, is required a jacketing with a thickness of 100 mm on each side of the wall.

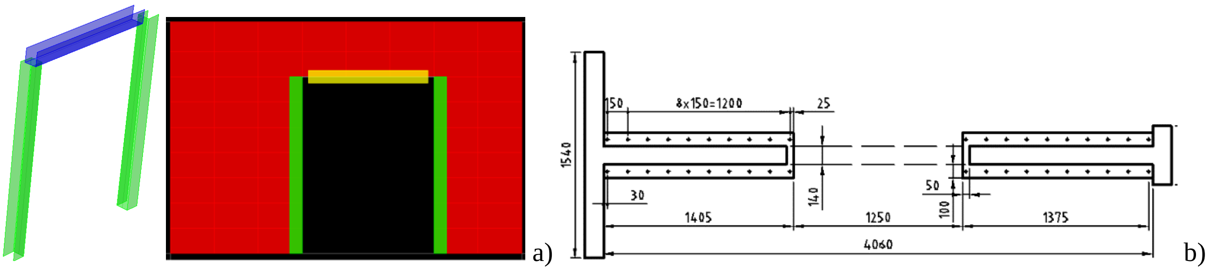


Figure 149. Consolidation methods for vertical diaphragms: with steel frames (a) and by concrete jacketing (b).

In the case of vertical merging of apartments, there are needed cuts in horizontal slab diaphragms that may allow the access through a staircase. The cuts are 1200x1200mm and their disposition in thought for the living room. The cuts are performed between second and first storeys, respectively forth and third storeys, as shown in Figure 150. At the end are realised four apartments on level, at each two storeys, with a double living area as compared to usual apartments.

The structural analysis carried on the modified structure demonstrated that the force differences in vertical diaphragms are insignificant as compared to the initial structure. However, important changes are recorded at the level of the cut horizontal diaphragm panel (panel P42-21, as shown in Figure 150). Both the resistance and deflection criteria are exceeded. In consequence, two strengthening solutions were investigated:

- by increasing the reinforcement percentage: the solution considers the addition of steel strips on both sides of the concrete slab (see Figure 151). For the connection with the concrete slab, anchoring bolts are disposed with the main role of shear connectors between existing and new added elements. For the cut slab panel two steel strips 5x25mm are disposed on each direction of the cut;

- by reinforcement using polymeric fibres: for bordering the cut, polymeric fibre fabrics are disposed in the zones in which the existent reinforcing is not sufficient. According to the resistance analyses 6 fabric strips are needed on longitudinal direction of the slab panel.

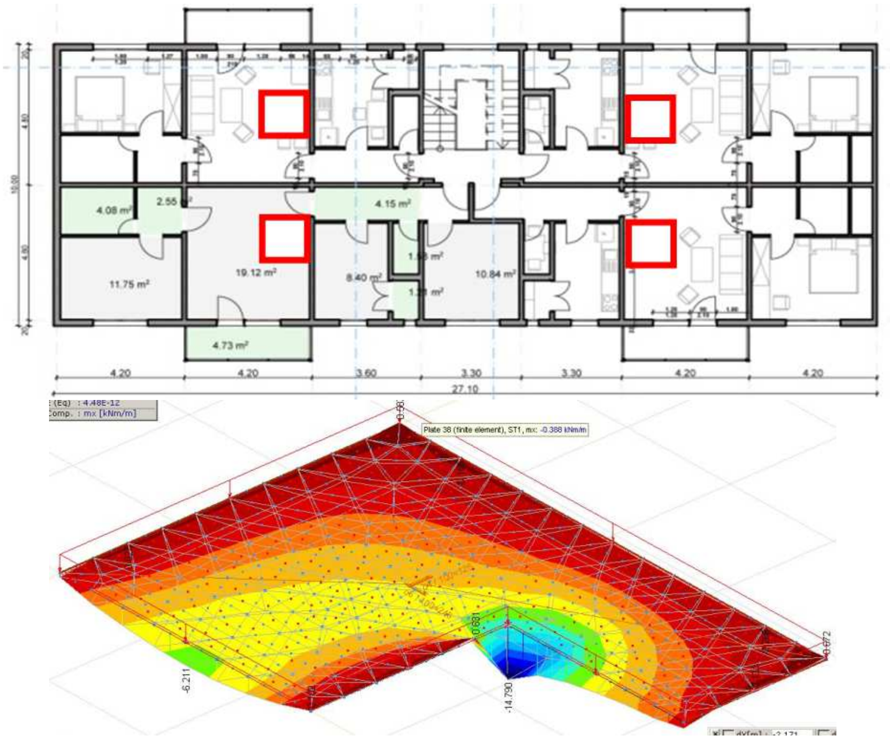


Figure 150. Detail for a unitary intervention for the cuts in slab diaphragms and the force diagram in the case of a diaphragm panel.

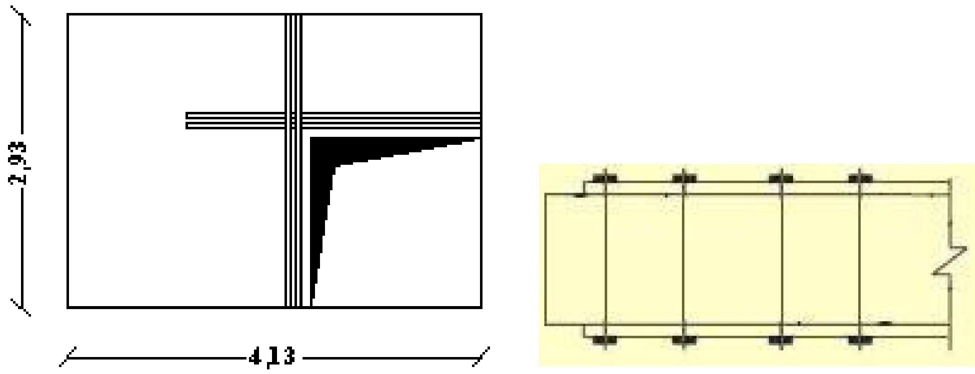


Figure 151. Addition of supplementary reinforcement (steel strips) and their bolt anchoring.

- *Overcladding solutions*

1. Solution considering steel frames on hot-rolled sections

The overcladding solution considering hot-rolled steel elements was analysed for the project type 744R building and considers transversal moment resisting frames and respectively braced longitudinal frames (see Figure 152). The inverted V bracings allow keeping five free spans for the installation of windows in living rooms and bedrooms, while for braced spans the application of smaller windows is still possible, similarly to the kitchen windows at inferior storeys.

The frame beams are haunched and are fixed on columns. On transversal direction the stability and stiffness of the structure is assured by the diaphragm effect. The gable frames are not braced. Longitudinal stability and stiffness is assured through vertical longitudinal braces. The spatial conformity is obtained through horizontal bracing and respectively roof bracing systems. The structural analysis was considering two different situations: columns fixed in existing structure, respectively columns pinned in old concrete building.

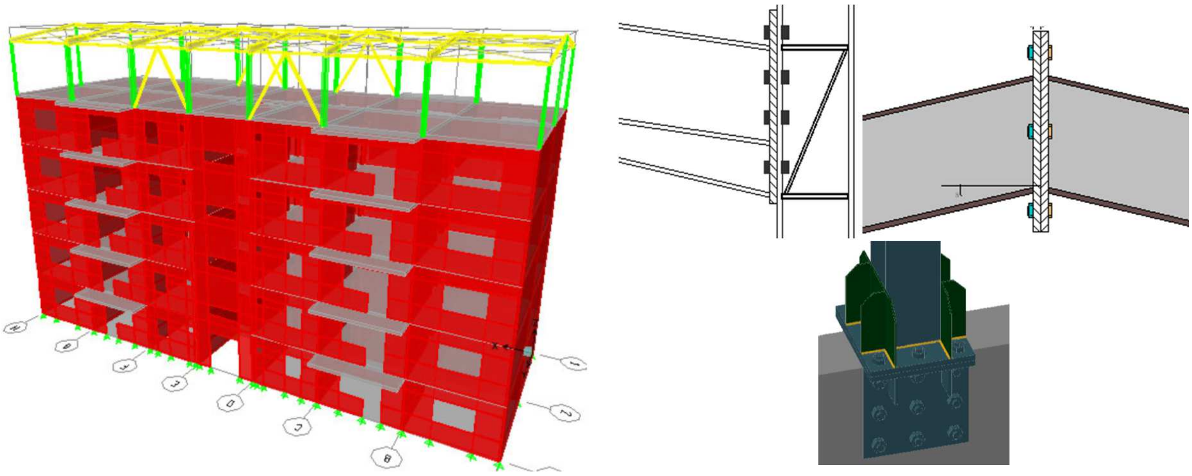


Figure 152. Overcladding of 744R IPCT project type (solution with steel hot-rolled profiles): global view and connections.

The analysis of steel structure was considering the climatic and seismic loads characteristic to the city of Timisoara, lead to the following sections of steel elements (structural steel S235): columns HEB 180, transversal beams IPE 220 (with 140mm haunches near column connection), inverted V braces from RHS 80x80x4, front columns HEB 100 and roof braces Φ26mm. These dimensions are practically identical for both structural systems: fixed / pinned. However, a few differences exist between pin basis of the columns and fixed ones:

- double vibration period;
- higher lateral deflections (within the norm limits);
- increase of the bending moment values at the top level of the column (5%);
- increase of the bending moment values at ridge (25%)

Though, the pinned connection is considered simpler as this transmits only axial and shear forces to existing structure.

The analysis of concrete structures both in original and overcladded situation demonstrates that for the overcladded situation the bending moment values are higher than those obtained in the original state. However, they are inferior to resistance values (for the corresponding axial load). Accordingly there could be concluded that the concrete structure can withstand the additional loads due to overcladding. Also, it should be mentioned that in an initial stage the structure is discharged of the hidro-insulation characteristic layers (membranes, slope concrete etc.) and the loads due to overcladding are of the same magnitude.

On the other hand, comparative analyses led to the conclusion that the overcladding steel structure is not influenced on the design position: on ground or on the top of existing concrete structure. This conclusion results from the forces recorded at the level of principal structural elements which are almost identical, as well as the values of the vibration periods.

2. Solution considering lightweight cold-formed steel frames

The main advantage of using an overcladding solution made by thin-walled cold-formed elements is the lightness, easy erection and simple transportation of structural elements. The case-study was performed on 770 IPCT project type.

The initial design consisted in checking the bearing capacity of the original structure in overtaking the actual design loads. The results confirmed that the building possesses important reserves of resistance. In consequence, in the next step the overcladded structure was analysed. In this case, too, was considered that the top layers characteristic to hidroinsulation and slope concrete were removed.

For this case-study the overcladding structure is made on transversal MRF and concentrically braced frames on longitudinal direction (see Figure 153). The overcladding columns are made of two C Lindab sections back-to-back disposed at 80mm. The columns are axially disposed over the axis of exterior diaphragms. Transversal beams are made also on C sections back-to-back disposition while the longitudinal ones are from two overlapped C-shapes forming a box profile. Longitudinally it was adopted the X braces made on steel circular rods. All steel elements are on S355.

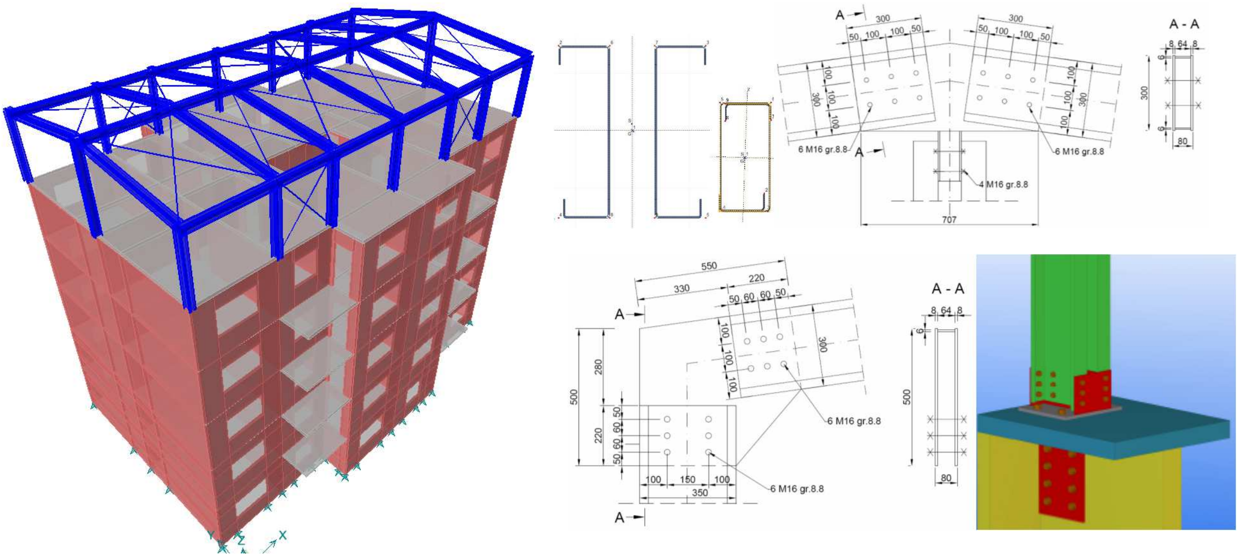


Figure 153. Overcladding of 770 IPCT project type (solution with lightweight cold-formed steel elements): global view and connections.

The analysis for the steel structure fixed at the base was realised for the climatic loads characteristic to the city of Timisoara lead to following sections of steel elements (steel class S355): columns 2C350x3, beams 2C300x3, longitudinal beams 2C100x1, roof braces of steel rods Φ16.

In case of the structural analysis of the structure pinned at the base, the structural elements remain identical to the ones listed above as they present reserves of bending resistance at their upper parts. However, the vibration periods are higher up to values of 0.565 seconds for the principal vibration mode, which conducts to a higher transversal flexibility. Nevertheless, the values of the lateral deflections under seismic conditions remain under allowable values.

The comparative results of the analyses made for the concrete structure show that a pinned steel LWCF solution induces similar values of internal forces as the fixed one. The only problem remains at the level of fixing the columns in the concrete structure, solution that is simpler for the pinned case. Moreover, the analyses revealed that the internal forces for the overcladded structure are smaller than in its original configuration, as the LWCF overcladding solution is lighter than the removed layers from the roof terrace.

The analyses of the steel structure confirmed the fact that there are not recorded noticeable differences between the cases of design on the ground or design on the top of the concrete structure. This is based on the high rigidity of the existing concrete structure which practically follows the ground motion. In consequence the ground design is identical to the design on the top of existing building.

3. Solution considering hollow section profiles

The use of hollow sections for single-storey buildings allows the realisation of moment-resisting frames in both directions. In consequence, the spans are free for the disposal of windows. The case-study considered the 1340 IPCT project type with a non-regular shape in plan. The steel structure follows this disposition and consequently it results a structure translated in the axis 5. The structure is composed of seven MR transversal frames. Three of the frames are 10.20m wide, two of 10.80m while the other three are 13.80m. The roof is pitched, resting on steel beams (IPE profiles) with a double slope of 15%.

In this case-study too, the initial analysis of existing structure confirmed that it possesses important reserves of resistance under actual climatic conditions specific to the city of Timisoara. In a second phase was designed the overcladding structure and also checked of the overcladded concrete building. Also, it was considered that before the positioning of the new steel structure, the existing structure was cleaned of the old hydro-insulation layers and slope concrete. The overcladding structure was studied in two hypotheses: fixed and respectively pinned on the concrete building.

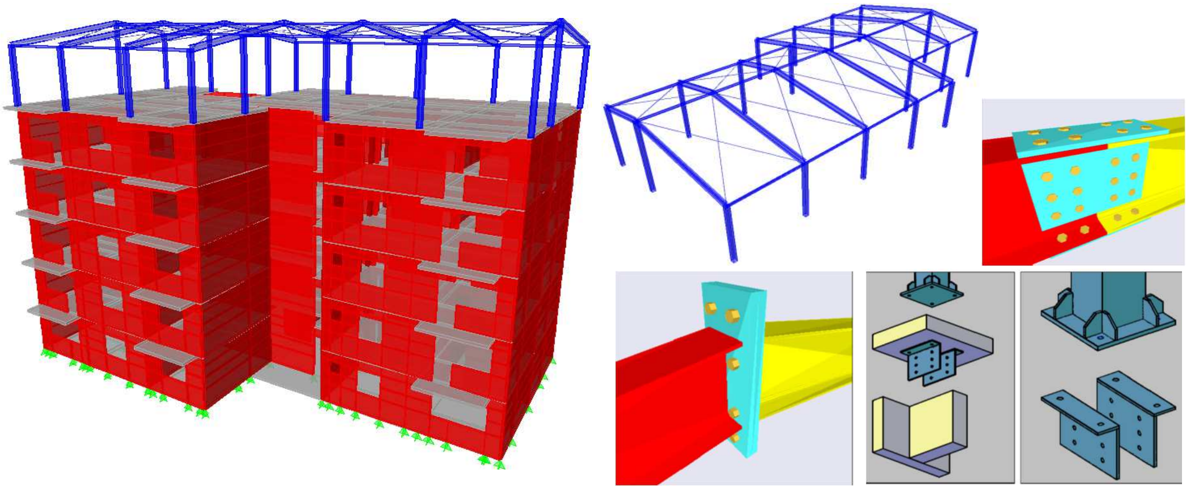


Figure 154. Overcladding of 1340 IPCT project type (solution with steel hollow sections elements): global view and connections.

The design of structural elements for the structure with fixed columns leads to RHS cross-section 200x200x8 for columns, IPE 270 cross-section for beams and HEB 100 longitudinal beams. All elements are in steel class S235.

Similar with previous cases, identical column cross-section resulted for case with pinned base. However, due to increased bending values in beam-to-column joints, the beam cross-section increases from IPE 270 to IPE 330. On the other hand, the comparative analyses for the steel structure (fixed or pinned on the ground or on the top of existing building), show that the steel design is not influenced on the position of the steel structure: top of the building or on the ground. The values of the internal forces are quasi-identical for the two situations.

The results of the structural analyses made on the concrete structure (original and overcladded structure) certifies that the global effect of overcladding leads to a slight diminishing of internal forces on diaphragms, mainly due to elimination of hydro-insulation and slope concrete layers at top level. Comparing the internal forces at the basis of the concrete structure in function of connection between steel and existing concrete structure (pinned or fixed) it was shown no important differences are recorded. The only noticeable difference is at the connection level of steel structural elements on the concrete diaphragms. The connection details are simpler for pinned case.

- *Structural and architectural retrofitting solutions - conclusions*

The interior repartitioning of concrete residential buildings can improve the comfort of inhabitants in such buildings. The apartment coupling horizontally or vertically will result in new internal configurations and can offer improved interior space, new types of flats, with implications at larger scale on the local community, such as the decrease of densification in urban areas. These types of interventions can also be used to revitalise parts of the city and contribute to cities urban regeneration.

Structurally, both types of interventions are possible but attention should be given to local detailing: (i) when cuts in the vertical diaphragms are performed, these should be reinforced by additional steel frames or concrete jacketing; (ii) if cuts are made on horizontal diaphragms, additional reinforcement near the cut-outs is needed.

The structural analyses performed on 744R, 770 and 1340 IPCT project types made on large precast concrete panels led to the following main conclusions:

- all considered typologies possess important reserves of resistance, even for the case of increased loads characteristic to modern norms, including seismic;
- the structures allow the reconfiguration of internal spaces by merging apartments on vertical or horizontal, by performing new cuts in existing diaphragms in a controlled way;
- when merging horizontal apartments, the cut diaphragm must be strengthened by steel frames or concrete jacketing;
- when merging apartments on vertical, the cuts in horizontal diaphragms for internal staircases must be strengthened. This can be done by additional reinforcement using steel strips or by using fibre polymeric solutions bond on concrete elements;
- overcladding of existing buildings is possible by considering light steel framing solutions. Prior installation, the removal of existing hydro and slope concrete layers should be done;
- the design of the steel frames can be performed on the top of existing concrete building or on the ground. It is recommended that the steel structure to be pinned on the existing concrete building due to simpler connecting details.

3.3.4. Thermal retrofitting solutions

Nevertheless, the main problem of the existing Romanian building stock is represented by the low thermal efficiency of their envelopes. Taking into account that Romania has predominantly a continental climate, this issue lead to human discomfort during cold and warm seasons, as well as to large amount of energy dissipation.

Figure 155 shows the principal typologies of walls, with their description, benefits and drawbacks. The main types of wall systems were using the following resistance typologies:

- Resistance walls in plain brick masonry. Masonry with vertical hollows and stanchions since 1980s.
- Concrete diaphragms plated with different types of thermo-insulation (aerated concrete blocks / mineral wool);
- Reinforced concrete frames with in-fill walls: masonry of plain bricks, bricks with vertical hollows or aerated concrete blocks.

The $R_{0,ef}$ value presented in Figure 155 represents the thermal resistances to heat flow, expressed in [m^2K/W] units. This is computed as the sum of thermal resistances to heat flow of all wall layers (exterior to interior). A very important change in thermal insulation of buildings was performed in 1984, when at the national level it was imposed an energy saving programme and the solutions adopted for buildings were changed. This year represent also a turning point in the Romanian standards regarding thermal insulation (Dan et al. 2007).

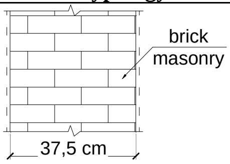
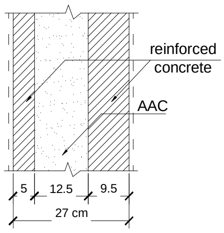
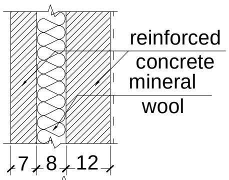
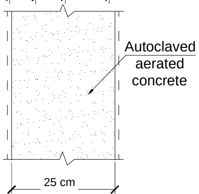
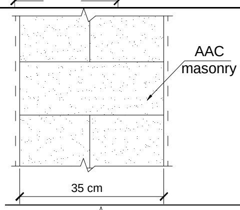
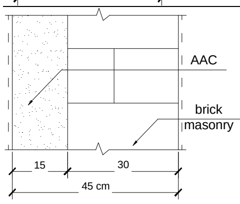
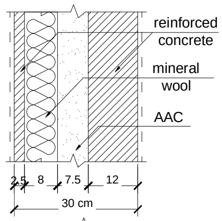
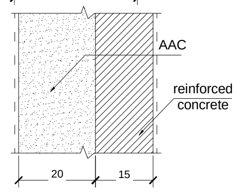
YR	Typology	Description	BENEFITS (+)	DRAWBACKS (–)
1960-1984		1. Exterior plastering 2. Masonry plain bricks of 37.5 cm / bricks with vertical hollows 30cm 3. Interior plastering	- fire resistance: good behaviour - vertical diaphragm action: good behaviour if the masonry correctly executed - thermo-insulation: very low - $R_{0,ef}=0.57 / 0.54\ m^2K/W$	- prone to thermal-bridging - local deterioration due to humidity - environmental impact: preliminary studies indicate very high environmental impact due to heat loss
		1. Exterior plastering 2. 1st layer reinforced concrete 5cm 3. Plain aerated concrete masonry 12.5 cm 4. 2nd layer reinforced concrete 9.5cm 5. Interior plastering	- fire resistance: good behaviour - vertical diaphragm action: good response - thermo-insulation: very low - $R_{0,ef}=0.57\ m^2K/W$	- prone to thermal-bridging - local damages in bad executed joints - environmental impact: preliminary studies indicate very high environmental impact due to heat loss
		1. Exterior plastering 2. 1st layer reinforced concrete 3. Mineral wool 4. 2nd layer reinforced concrete 5. Interior plastering	- fire resistance: good behaviour - vertical diaphragm action: good response - thermo-insulation: very low - $R_{0,ef}=0.93\ m^2K/W$	- local damages in bad executed joints - environmental impact: preliminary studies indicate high environmental impact
		1. Exterior plastering 2. Plain aerated concrete masonry 25 cm 3. Interior plastering	- fire resistance: good behaviour - resistance: assured through frame behaviour - thermo-insulation: very low - $R_{0,ef}=0.68\ m^2K/W$	- prone to thermal-bridging - local deterioration due to humidity - environmental impact: very high due to heat loss
1984-1994		1. Exterior plastering 2. Plain aerated concrete masonry 35/45 cm 3. Interior plastering	- fire resistance: good behaviour - resistance: assured through frame behaviour - thermo-insulation: fair - $R_{0,ef}=1.84/2.43\ m^2K/W$	- prone to thermal-bridging - local deterioration due to humidity - environmental impact: fair
		1. Exterior plastering 2. Plain aerated concrete masonry 15/45 cm 3. Masonry plain bricks of 30 cm 4. Interior plastering	- fire resistance: good behaviour - vertical diaphragm action: good behaviour if the masonry correctly executed - thermo-insulation: low - $R_{0,ef}=1.38\ m^2K/W$	- prone to thermal-bridging - local deterioration due to humidity - environmental impact: high environmental impact due t heat loss
		1. Exterior plastering 2. Mineral wool 3. Plain aerated concrete masonry 4. Reinforced concrete	- fire resistance: good behaviour - vertical diaphragm action: good behaviour - thermo-insulation: fair - $R_{0,ef}=1.63\ m^2K/W$	- prone to thermal-bridging - local damages in bad executed joints - environmental impact: low due to heat loss through wall joints
		1. Exterior plastering 2. Mineral wool 3. Plain aerated concrete masonry 4. Reinforced concrete	- fire resistance: good behaviour - vertical diaphragm action: good behaviour - thermo-insulation: fair - $R_{0,ef}=1.61\ m^2K/W$	- prone to thermal-bridging - local damages in bad executed joints - environmental impact: low due to heat loss through wall joints

Figure 155. Design details and characteristics of wall typologies between 1960 and 1994.

The first Romanian normative requirements regarding heat transfer were given in 1960s for exterior walls, flat roofs and floors over basement. Table 19 presents briefly the variation in required thermal resistance of building envelope elements.

Table 19. Romanian normative requirements for thermal resistance of envelope elements (values in m^2K/W).

Year	Standard	Ext. walls	Flat roofs	Floor over basement
1962	6472-61	0.76	0.96	0.82
1984	NP15-84	1.20	1.55	1.08
1997	C107/3-1997	1.09	1.46	1.25
2010	C107/3-1997	1.80	5.00	2.90

During the last 50 years the R values have been changed more than forty times in new and changed issues of national standards, but always increasing values were required. Thus, the heat transfer requirement increased 2.5 times for external walls, 5 times for flat roofs and 3.5 times for floors and basements.

Unfortunately the changes in the heat transfer requirements were not followed by the up-dating of the envelopes of the existing building stock. This is in fact the main reason of having in present a very large building stock that do not fulfil the actual norms regarding the heat flow transfer.

The usual stratifications of collective residential buildings (as shown in Figure 155) are far from assuring the actual thermal resistance requirements. Table 20 shows the thermal resistances for the exterior walls in comparison with the normative requirements (ASRO, 2010). It results that in many cases the existing layers can assure at most half of the required thermal resistance. Moreover, the thermal insulation layer is in many cases out-dated or damaged due to ageing.

In consequence, an ID for retrofitting such structures should include as a necessity the thermal upgrading of exterior walls. The compatibility of the thermal retrofitting solution with the existing structure must be assured.

Table 20. Thermal resistances in of external walls in different periods (values in m^2K/W).

Typology	Thermal resistance of wall	Requirement by norm	Requirement by actual norm (C107/3-2010)
Type II	0.57	0.76 (NR 6472-61)	1.80
Type III	0.93	0.76 (NR 6472-61)	
Type VI	1.38	1.20 (NP 15/84)	
Type VII	1.63	1.20 (NP 15/84)	

- *Retrofitting solutions*

Figure 156 shows eight solutions used for thermal retrofitting the concrete external walls through external over-cladding. The original stratification chosen as case-study in the pre-sent paper is type II, according to Figure 155, which corresponds to the T770 collective building standardized project, built in the 1975-1985 period. The solutions name is given by the insulation material used or by the name of the company offering the integrating solution.

First of the solutions proposed represents the common solution of thermal rehabilitation of buildings, through a thermo-system assured by a polystyrene layer (100 mm in this case) and exterior plastering adherent on a glass fibre. The next five solutions are similar as conception, by considering a thermal insulation layer on cellulose, glass, mineral or rig-id basaltic wool and polyurethane foam respectively. The external siding – plastering, polyvinyl or brick siding – is fixed either directly on the thermal insulation layer either on thin walled cold-formed profiles fixed on concrete wall.

The last two solutions represent integrated solutions offered by specialized companies on external wall siding: the Ruukki solution is a sandwich panel with embedded thermal insulations while in the case of siding cassettes offered by Lindab the thermal insulation layer is chosen by the owner. The two systems are fixed on steel profiles, pre-fastened on the concrete bed.

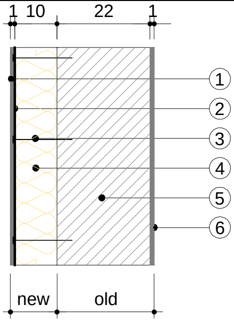
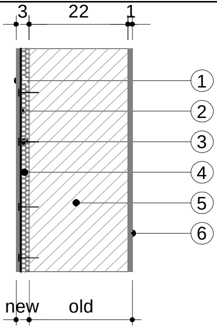
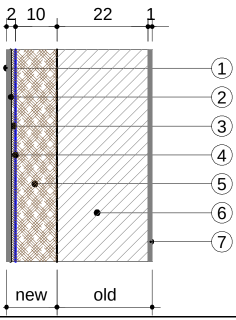
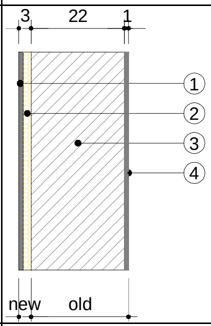
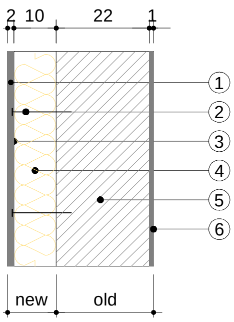
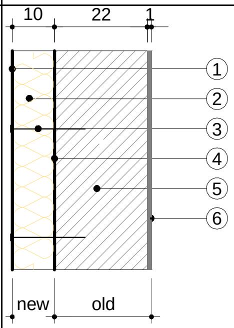
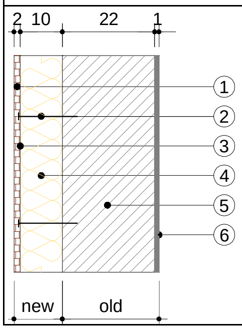
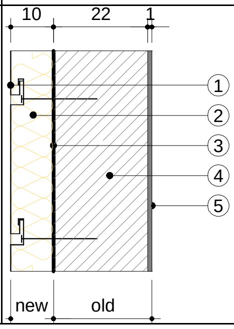
<table>
<tr><td>Solution 1 – “Polystyrene</td><td>Solution 5 – “Rigid foam”</td></tr>
<tr><td>
1. Exterior plastering
2. Glass fibre wire lattice
3. Fixing screws (steel screws and plastic fixings)
5. Thermal insulation: extruded polystyrene 10cm
6. Reinforced concrete
7. Interior plastering</td>
<td>
1. Exterior plastering
2. Glass fibre wire lattice
3. Fixing screws (steel)
4. Thermal insulation: Plastispan HD and Durofoam HD 10cm
5. Reinforced concrete
6. Interior plastering</td></tr>
<tr><td>Solution 2 – “Cellulose”</td><td>Solution 6 – “Polyurethane Foam”</td></tr>
<tr><td>
1. Exterior plastering
2. Glass fibre wire lattice
3. Oriented Strand Board
4. PVC vapour barrier
5. Thermal insulation: untied cellulose fibre 10cm
6. Reinforced concrete
7. Interior plastering</td>
<td>
1. Acrylic paint
2. Thermal insulation: polyurethane foam 10cm
3. Reinforced concrete
4. Interior plastering</td></tr>
<tr><td>Solution 3 – “Glass Wool”</td><td>Solution 7– “Ruukki”</td></tr>
<tr><td>
1. Polyvinyl exterior plates
2. Fixing screws (steel)
3. Ventilation void
4. PVC vapour barrier
5. Thermal insulation: Glass wool 10cm
6. Reinforced concrete
7. Interior plastering</td>
<td>
1. Exterior sandwich steel plate
2 Thermal insulation: Basaltic wool 10cm
3. Interior sandwich steel plate
4. Fixing screws (steel)
6. Reinforced concrete
7. Interior plastering</td></tr>
<tr><td>Solution 4 – “Mineral Wool”</td><td>Solution 8– “Lindab”</td></tr>
<tr><td>
1. External brick siding
2. Fixing screws (steel)
3. Ventilation void
4. Thermal insulation: Mineral wool 10cm
5. Reinforced concrete
6. Interior plastering</td>
<td>
1. Lindab steel cassette
2 Thermal insulation: Basaltic wool 10cm
3. PVC vapour barrier
6. Reinforced concrete
7. Interior plastering</td></tr>
</table>

Figure 156. Solutions for thermal retrofitting the existing concrete walls.

- *Multi-criterial analysis and choice of solution*

Three criteria were considered for the final selection of thermal retrofitting solution, corresponding to the sustainability pillars:

- social: *thermal insulation* resistance. It measures the ability of each solution of offering adequate internal comfort, while its implication is also on the building energy use. The values given in the Table 21 consider the thermal resistances of existent and additional layers, but disregard the existent thermal insulation layer which in many cases is not efficient. All the solutions were adjusted to offer values close to the thermal resistance normative requirements (1.80 m2k/W);

- economic: the *economic assessment* is considered through integrated costs for materials, mechanical fasteners and labour. The values offered in Table 21 are computed according to the current economic situation in Romania per square meter of retrofitted wall;

- environment: the *environmental impact* was considered through Life-Cycle Impact Analysis (LCIA) on new added materials, per square meter of wall using the SimaPro computer tool. The values are given in eco-points, by considering the EcoIndicator 99 as method of analysis. The LCIA inventory was constituted from average weights of materials divided to the total covered area. For the life cycle approach the production and end-of life of materials was considered. For the end-of-life, a reasonable scenario for reuse, recycling and disposal of materials was envisaged (see Table 22). Figure 157 shows graphically the LCIA results for all seven solutions considered.

It is to be noticed the fact that the safety criterion was not included in analysis as this was assured for all solutions (excluded by boundary conditions).

Table 21. Thermal, costing and environmental impact estimations for retrofitting solutions.

Sol. No.	Sol. Name	Effective thermal resistance [m^{2}k/W]	Price [eur/sqm]	Environmental impact [Pt]
1	Polystyrene	2.04	35.36	2.26
2	Celluloses	1.95	20.51	0.99
3	Glass Wool	2.09	45.43	0.68
4	Mineral Wool	2.10	37.10	0.62
5	Rigid Foam	2.05	24.57	0.85
6	Polyurethane Foam	2.34	31.64	0.65
7	Ruukki	2.08	21.88	0.55
8	Lindab	1.98	53.41	1.73

Table 22. End-of-life scenario for main building materials.

Material	*Reuse [%]*	*Recycling [%]*	*Burn [%]*	*Disposal - Landfill [%]*
Steel materials	--	100	--	--
Wooden materials	30	--	70	--
Concrete, mortar	--	--	--	100
Other inert materials	--	--	--	100
Other combustible materials	--	--	100	--

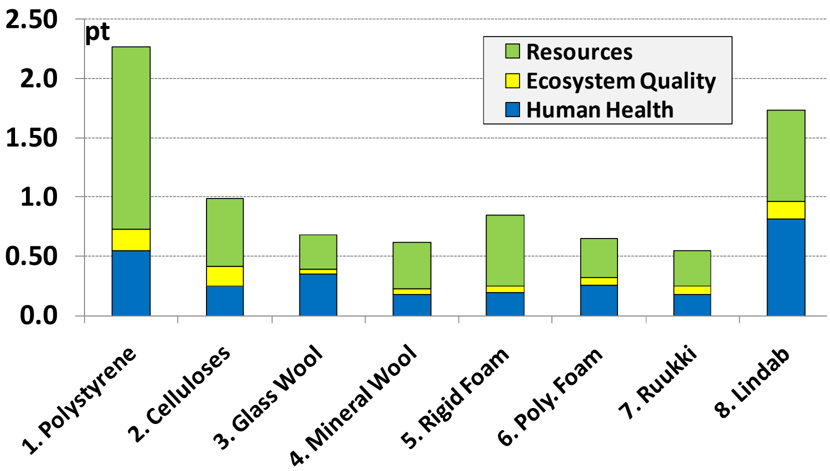


Figure 157. LC environmental impact for retrofitting solutions (tool: SimaPro, Method: Ecoinventor 99)

- *Multi-criterial analysis and choice of solution - selection of solution by considering indicator-oriented methods*

The choice of solution through indicator-oriented method represents the easiest way of selecting among solutions. The willingness to pay is the most used factor on selecting goods on the economic market. Solutions 2, 6 and 7 represent in turn the best choices considering individual indicators: price, thermal resistances and environmental impact respectively. In consequence, judging strictly on one of these criteria, one of the solutions is chosen.

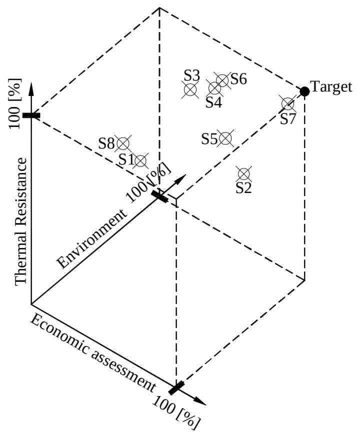
- *Multi-criterial analysis and choice of solution - selection of solution by considering multi-axial representation method*

The solution selection through multi-axial representation considers an axis for each individual indicator. The representation is possible for three indicators but the solution remains valid even for more indicators. The first step of the method is the normalization of results: the solutions having the best performance in regard to a certain indicator is maximized to 100% while the rest of indicators are normalized to this value in percentages as shown in Table 23.

The second step is the computation of the distance to an ideal target, defined by the point of maximum coordinates (100,100,100). This can be easily done by computing the vector between the real coordinated points and the ideal target through the square root of sum of squares. Figure 158 shows the 3D representation by considering the three indicators considered in the case-study. Also, the figure shows the computed distances to the target.

Table 23. Normalization of indicator values

Sol No.	Th. Resistance		Price		Environmental Impact	
	[m^{2}k/W]	Normalisation	[€/sqm]	Normalisation	[pt]	Normalisation
1. Polystyrene	2.04	86.90	35.36	58.00	2.26	24.42
2. Celluloses	1.95	83.41	20.51	100.00	0.99	55.93
3. Glass Wool	2.09	89.08	45.43	45.15	0.68	80.70
4. Mineral Wool	2.10	89.51	37.10	55.28	0.62	88.89
5. Rigid Foam	2.05	87.56	24.57	83.48	0.85	65.09
6. Polyurethane Foam	2.34	100.00	31.64	64.82	0.65	84.53
7. Ruukki	2.08	88.95	21.88	93.74	0.55	100.00
8. Lindab	1.98	84.33	53.41	38.40	1.73	31.91



Sol. No.	Distance to target
1. Polystyrene	87.45
2. Celluloses	47.09
3. Glass Wool	59.17
4. Mineral Wool	47.26
5. Rigid Foam	40.57
6. Polyurethane Foam	38.43
7. Ruukki	12.70
8. Lindab	93.15

Figure 158. Tri-axial representation and distance to target for solutions 1-8

The smallest distance to the target is obtained for the seventh solution (12.70) which by far is better than other thermal rehabilitation systems. It could be noticed that the first solution which is largely used in real rehabilitation projects is almost the furthest to the target point. The bad scoring is due to the bad environmental impact and also the price.

- *Multi-criterial analysis and choice of solution - Selection of solution by considering characterization factor method*

The method is based on using characterization factors in accordance to the importance of a specific indicator in the final decision choice. The factorized values multiply the normalized values which are finally added in a final score (aggregated value). The highest value represents the best score. The difficulty of the method is finding the right characterization factors reflecting the importance of indicators. Usually a block of experts can be consulted for finding adequate factor values. One solution is by considering factors in unitary ratios.

Table 24 shows the factorized values and final scores using the following characterization factors for specific the three indicators considered:

- c_t = 0.45 for the thermal resistance
- c_p = 0.30 for the economic assessment
- c_e = 0.25 for the environmental impact

Table 24. Factorized values and final scores for different solutions

Solution No.	Thermal Resistance		Economic assessment		Environmental impact		*Final score*
	Normalised value	Factorised value	Normalised value	Factorised value	Normalised value	Factorised value	
1. Polystyrene	86.90	39.10	58.00	17.40	24.42	6.11	*62.61*
2. Celluloses	83.41	37.54	100.00	30.00	55.93	13.98	*81.52*
3. Glass Wool	89.08	40.09	45.15	13.54	80.70	20.18	*73.80*
4. Mineral Wool	89.51	40.28	55.28	16.58	88.89	22.22	*79.09*
5. Rigid Foam	87.56	39.40	83.48	25.04	65.09	16.27	*80.72*
6. Polyurethane Foam	100.00	45.00	64.82	19.45	84.53	21.13	*85.58*
7. Ruukki	88.95	40.03	93.74	28.12	100.00	25.00	*93.15*
8. Lindab	84.33	37.95	38.40	11.52	31.91	7.98	*57.45*
Ch. Factor	*x 0.45*		*x 0.30*		*x 0.25*		

According to this method again the seventh solution with an integrated sandwich panel, presents the best scoring. This fact is due primarily to an integrated optimization of the solution, combining several parameters such as:

- adequate thermal insulations, adjusted through the width of the insulation layers;
- good pricing due to the intensive industrialized processes;
- relatively low environmental impact by using high recyclable materials (such as steel).

However, good results are obtained also for the polyurethane foam and cellulose solutions, which might be considered as good alternatives for thermal insulation.

- *Thermal retrofitting solutions - conclusions*

One of the immediate needs for assuring the thermal comfort of habitants and also the efficient use of operational energy is the thermal insulation. In an integrated retrofitting design, the techniques employed in thermal insulation should consider the conditions of the existing structure. Moreover, by considering a sustainable design, the environmental criterion should be present in the decisional matrix, among other economic and technical criteria.

The choice of a multicriterial analysis in thermal retrofitting of old buildings is a decisional task which may employ several methods. Usually, the indicator-oriented methods are deficient due to ignorance of non-considered criteria. This is why other methods, such as the multi-axial representation or the characterisation factor method could clear the choice for a solution.

In case of the study-case, the steel-intensive solutions such as sandwich panels can offer a good solution for thermal rehabilitation of concrete structures. The reliability of solution is due to the use of highly recyclable materials and economic advantages due to the industrialized processes used in manufacturing.

C. SCIENTIFIC, PROFESSIONAL AND ACADEMIC FUTURE DEVELOPMENT PLANS

The development plans of the candidate on scientific, professional and academic direction consider the basis already investigated on composite steel and concrete structures and the sustainable development of buildings. However, the candidate does not exclude other development directions in the field of structural engineering linked to steel and concrete elements.

C1 - SCIENTIFIC

In this moment the CEMSIG Laboratory of the Department of Steel Structures and Structural Mechanics at the Politehnica University of Timisoara has an impressive experimental capability, being able to test real-scale building sub-assemblies and building structures under static and dynamic loads. The intention of the candidate is to further develop the CEMSIG facilities in the field of structural engineering both experimental and numerical through acquisition of modern facilities and strong modelling software. The plans for scientific development of the candidate mainly regard the research directions listed below.

- *Application of slim-floor decking in seismic regions*

The Slim Floor Beam (SFB) system represents a modern technique of flooring successfully used in buildings located in non-seismic areas. It presents several advantages as compared to usual floor systems. However, for the integration of these systems in the seismic zones, additional research should be performed. The solutions must assure adequate levels of resistance and ductility in case of SFB systems used in seismic-resistant structures. The DUAL type steel moment resisting frames MRF + braced systems with SFB configurations can be thought as combining efficiently the architectural freedom and seismic resistance. However, even in this case the MRF should withstand a fraction of the seismic lateral action. In this equation the beam-to-column joining will play the key-role in the Dual configuration behaviour. Firstly, the MRF SFB-to-column joints as they should possess good dissipation capacity to lateral loads.

In order to develop applicable design procedures for safe use of slim-floor solutions in seismic dissipative structural systems the solutions should be investigated experimentally, on key-subassemblies and sub-structures, completed in a second step by numerical analyses on structures for the confirmation of structural behaviour in a macro-level.

- *Investigation of performances of steel-concrete composite framed structural systems to strong seismic motions specific to Romanian territory*

The investigation of the composite structures with connection between the concrete or the masonry part and the steel structure is of scientific interest nowadays, due to the economic reasons resulted from the practical application of the solution. However, the applicability of such structural systems in the seismic conditions of the Romanian territory must be proved. Based on previous experimental studies performed within the CEMSIG laboratory, the solutions must now be extrapolated towards global structural response. The study should be based on numerical analyses on structural systems (braced and unbraced frames) with different heights and bays and an initial design on different Romanian seismic zones. The analyses must consider dynamic analyses with accelerograms and elasto-plastic behaviour of the structural elements. Of course, the accelerograms should correspond to the Romanian seismic territory.

- *Dual frames with removable dissipative components*

The investigations already performed by the candidate within the CEMSIG team on the steel and composite link elements have shown that the EBFs are systems of high dissipation in case of strong seismic motions. Current seismic design philosophy is based on dissipative structural response, which implicitly accepts damage to the main structure and significant economic losses. Repair of the structure is often impeded by the permanent (residual) drifts of the structure. Hence, a more rational design approach in the context of sustainability is by reducing the repair costs and downtime of a structure hit by an earthquake. These objectives can be attained through removable dissipative members and re-centring capability of the structure.

The concepts of removable dissipative members and re-centring capability are to be implemented in dual structures, obtained by combining a stiff dissipative subsystem (EBFs or SPSWs) with a flexible elastic subsystem (MRFs). The bolted links in EBFs and bolted shear walls in Shear Panel Walls SPSWs are intended to provide the energy dissipation capacity and to be easily replaceable, while the more flexible MRFs would provide the necessary re-centring capability to the structure. The validation of the proposed solution is to be realised through pseudo-dynamic tests of large scale models of dual eccentrically braced frames with vertical links (D-EBFs) and dual steel plate shear walls (D-SPSWs). The research will demonstrate the feasibility of the proposed concept (by re-centring capability of dual structures with removable dissipative members), clearing the route toward implementation into design practice. Additionally, the overall seismic performance of DEBFs and D-SPSWs will be validated.

- *Sustainable investigations on optimal solutions for external wall and floor systems applied to new and existing buildings*

In case of modern housing, the trend is for using lightweight building solutions with intensive-industrialised materials and components. For this reason, the investigation on wall and floor solutions for buildings could provide both structural and functional advantages. The main purpose is to obtain wall and floor technical solutions compatible with fast-track construction, providing structural robustness and simultaneously functional criteria in terms of thermo-energetic efficiency, vibration control, and as well as global sustainability parameters.

The study should be focused on lightweight construction, based on the intensive use of metal and/or composite solutions and dry techniques. Low and medium-rise buildings are considered for which resistance and robustness are assured by:

- diaphragm action of walls and floors for low-rise buildings;
- rigid cores in case of medium-rise buildings added to the resistance of lateral walls;
- catenary action capacity of floors, enabling for redistribution and alternative paths of stresses in case of local failures due to accidental actions.

New-building and rehabilitation solutions for ancient wooden floors and masonry/concrete walls should be considered. The objectives are to improve the original structural state for achieving a required global structural response, thermo-energetic efficiency and building durability.

- *Functional Refurbishment of Residential Buildings with Structure on Large Reinforced Concrete Panels*

As shown by the already presented studies, the Romanian comfort of habitation in urban areas intensively constructed with residential apartment type buildings is at a low level due to the deficiencies in (i) urban management (circulation and parking spaces, accessibility, comfort) and (ii) interior spaces of habitable units: underdimensioned habitable space, low thermal comfort and energetic efficiency, non-unitary cost construction interventions, deficient hydro insulations etc. Moreover, the constructive systems of the large reinforced concrete precast panel apartment buildings present a rigid interior partitioning. Although during time, there have been developed different partitions within standardised projects these do not satisfy the modern requirements for flexibility of the habitable space. From the structural point of view the situation of these buildings has been aggravated by the partial interventions of owners and in consequence a large number of buildings present increased vulnerability to current or accidental loading conditions.

The large reinforced concrete precast panel apartment buildings have been executed in the 1960-1990 period throughout standardized projects. For this reason the functional refurbishment solutions for the habitable spaces should also be based on standardized interventions applied to the main building typologies. The integrated solutions should include structural aspects (e.g. coupling two apartments, integration of some adjacent access-ways) and thermal refurbishment solutions. Some other aspects should be integrated:

- realisation of an integrated intervention methodology on different building typologies;
- national applicability maps for proposed solutions;
- technical strengthening details applicable to load bearing elements;
- integrated intervention solutions considering structural and thermal refurbishment of the envelope compatible with the original envelope systems;
- economical evaluation of solutions, both structural and thermal;
- evaluation of the environmental impact on the environment.

C2 - PROFESSIONAL

In parallel with the research and academic activity, the candidate developed also his initial formation of civil engineer. In his activity, the candidate participated as chief designer or member in the design team in the design of about twenty steel and composite steel and concrete structures. The candidate received twice, as member in the design team, the ECCS award „European Steel Design Awards 2003 (BancPost Timisoara) and 2007 (Bucharest Tower International) for the steel structures realised in Romania. Following this direction, the candidate has in intention to continue the design activities for special buildings with steel or steel and concrete structures, using in this way his gained expertise in this field.

Figure 159. ECCS awards received by the candidate.

Furthermore, continuing the direction of implementation of Eurocode systems in Romania, the candidate will sustain the ASRO activities of translating, realisation of national annexes or reviewing the European documents. In this moment, the candidate is member of the ASRO eCommittee SC 4 for the revision of Eurocode 4 at European level. This activity is considered for a medium term run, as in 2018 the Eurocode norms will be issued in a revised version. As member in the European Convention for Structural Steelwork (ECCS), the candidate is in touch with the activities of European engineers and researchers in the fields of composite steel and concrete structures and sustainable development of steel buildings as national representative within technical committees TC11 and respectively TC 14.

As member in Romanian professional engineering associations: AICPS (Association of Design Engineers) and APCMR (Romanian Association of Constructional Steelwork), the candidate disseminated his findings within engineering community through presentations in national or local conferences and workshops as well as publications in professional journals. The opinion of the candidate is that these kinds of actions are beneficial for the engineering community and should be encouraged as the academic community should be a support and provide valuable information to the society, based on the research findings.

C3 - ACADEMIC

The candidate activates from the beginning of his career (1997) in Academic environments in the main domain of civil engineering. Currently, the candidate is lecturing Composite Steel and Concrete Structures at bachelor and master levels as well as subjects related to Sustainable Development of Buildings.

In the perspective of a further habilitation in conducting Ph.D. students, the candidate will promote the integration of doctoral students in the academic activities, including research and academic. For this purpose, funding of the research is regarded as the key-issue.

One of the main academic development plans of the candidate regard the homogenisation of the two topics in the curricula of the bachelor and master students in Civil Engineering. This issue is almost achieved in case of the courses on Composite Structures, by one basic course in the fourth year and a course on Advanced Design of Composite Structures for master level. In case of Sustainable Development, since two years there exists the course of Environmental Design of Buildings introduced in the third year –Romanian studies (bachelor) but this should be implemented also for English and German studies. This introductive course must be harmonised with the courses held for the master studies.

In this moment the candidate administrates for Politehnica University an ERASMUS Mundus Master Course (full name: Sustainable Constructions under natural hazards and catastrophic events - SUSCOS). The candidate has in intention the continuation of good relations with other national and European entities through both research and didactic activities. In this direction the regular ERASMUS activities can represent an initiation for further academic activities, including research. Present and past Erasmus contacts coordinated by the candidate: Technical University of Prague, University of Coimbra, Universite Blaise-Pascal Clermont-Ferrand and INSA-Rennes, France.

Nevertheless, based on the good relations already existent between the CEMSIG team and the technical universities from Cluj, Bucharest and Iasi, the candidate considers that the national cooperation should and could be strengthened through research grants and academic activities (student competitions, workshops and summer-courses).

Continuing the didactic actions beyond the regular activities, the candidate will help bachelor, master and doctoral students in organising workshops and summer-courses, sustaining both enrichment of the achievements and also student’s social life. This activity is considered as important by the candidate as this contributes to the personal development of further specialists.

References

Abaqus (2007) Analysis User's Manual I-V. Version 6.7. USA: ABAQUS, Inc., Dassault Systèmes.

Alashker Y., El-Tawil S., Sadek F., 2010. "Progressive collapse resistance of steel–concrete composite floors". Journal of Structural Engineering, 136(10):1187–96.

Altvater E., Growth, Productivity Employment, and Ecological Sustainability – a „Globalization Trilemma", www.memo-europe.uni-bremen.de/

, Growth, Productivity Employment, and Ecological Sustainability – a „Globalization Trilemma", www.memo-europe.uni-bremen.de/

Andersson J.O., Aspects on welfare and globalization, Dialogues on Sustainable Paths For The Future, Editor J. Kohl, FFRC eBOOK 7/2008, ISBN 9789515645555.

Aribert, J-M. & Lachal, A., 2000 Moment Resistant Connections of Steel Frames in Seismic Areas. Design and Reliability (Editor F.M. Mazzolani), E&FN SPON, London, ISBN 0-415-23577-4 Chapter. 4.3. Cyclic behaviour of Shear Connectors.

Astaneh-Asl A., Jones B., Zhao Y., Hwa, R., 2001. "Floor catenary action to prevent progressive collapse of steel structures". Report No. UCB/CE-Steel-03/2001, Department of Civil and Environmental Engineering, University of California, Berkeley, USA.

Banaitiene N., Banaitis A., Kaklauskas A., Zavadskas E.K., (2008), Evaluating the life cycle of a building: A multivariant and multiple criteria approach, Omega, 36, 429-441.

Beck H., Reuter M., "Powder actuated fasteners in steel construction", Steel Construction Calendar 2005 - Stahlbau Kalender 2005 (http://www.us.hilti.com/fstore/holus/LinkFiles/Stahlbaukalender.pdf)

Beck H., "Nailed shear connection in composite tube columns", Second European Conference on Steel Structures, Prague, 26-29 May 1999, pp. 565−568.

Beck H., "Nailed shear connection in composite tube column with Hilti X-DSH 32 P10 – Installation instructions for test samples at University of Timisoara", Hilti AG., 22.11.2010.

BIT 24/1967. Blocks of flats. Trends in urban residential buildings design. No. 24/1967 (Technical Informational Journal – in Romanian)

Bosco M. and Rossi P.P. (2009), "Seismic Behaviour of Eccentrically Braced Frames", Engineering Structures ISSN 0141-0296 31(3), Pages 664–674.

Braganca L., Mateus R., Koukkari H., (2007), Assessment of building sustainability, Proc. of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering, Lisbon, Portugal, September 13-15, 0.3-0.12.

Bursi, O.& Ballerini M., 1997 Quasi-static cyclic and pseudo-dynamic tests on composite substructures with softening behaviour 5th International Colloquium on Stability and Ductility of Steel Structures, Nagoya, Japan.

Burstrand, H. Light-gauge Steel Framing for Housing; Swedish Institute of Steel Construction. Publication No. 170, 2001.

C107/1-3/1997 – Romanian standard for the computation of global thermal insulation coefficients for civil buildings, ASRO, 1997.

Chao S-H., Khandelwal K., and El-Tawil S., Ductile Web Fracture Initiation in Steel Shear Links, Journal of Structural Engineering ASCE ISSN 0733-9445, pp 1192-1200, Aug. 2006

Chen C.C., Lin N.J., “Experimental investigation of flange plate connections between concrete-filled tube column and steel beam”, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 476, 2004.

Cheng C.T., Chan C.F., Chung L.L., “Seismic behaviour of steel beams and CFT column moment-resisting connections with floor slabs”, Journal of Constructional Steel Research 63, p.p. 1479–1493. (doi:10.1016/ j.jcsr.2007.01.014), 2007.

Ciutina A., 2003 : Assemblages et comportement sismique de portiques en acier et mixtes acier - beton : experimentation et simulation numerique, PhD thesis, INSA Rennes, Octobre 2003.

Ciutina A., Ungureanu V., Grecea D., Dubina D., (2009), Family house: cold-formed steel framing and OSB cladding vs. masonry – Case study, Proc. of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering, Timisoara, Romania, 23-24 October, 373-390.

Ciutina A., Ungureanu V., (2009), Family house: cold-formed steel framing and OSB cladding vs. masonry – LC Energy approach, Proc. of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering, Timisoara, Romania, October 23-24, 391-400.

Ciutina, A., Ungureanu, V., Grecea, D., Dubina, D. Family house: cold-formed steel framing and OSB cladding vs. masonry – Case study. Proceedings of Workshop Sustainability of Constructions: Integrated approach to life-time structural engineering. COST C25 Publication (ISBN 978-973-638-428-8). Timisoara, Romania, 2009, 373-390.

Ciutina, A., Ungureanu, V., Dubina D. & Dinu F. Integrated design of buildings Proceedings of the International Conference: Sustainability of Constructions – Towards a better built environment COST C25. Innsbruck, 3-5.02.2011, ISBN 978-999957-816-0-6.

Clifton C., Bruneau M., MacRae G., Leon R., Fussell A., (2011). Steel Structures Damage from the Christchurch Earthquake of February 22, 2011, NZST, Bulletin of the New Zealand Society for Earthquake Engineering, Vol.44, No.4, pp.297-318.

CSI – Computers and Structures Incorporation, (2012) SAP 2000 Overview. Online at http://www.csiberkeley.com/sap2000 2012

Dan, D., Stoian, V., Nagy-Gyorgy, T. & Daescu, C. 2007. Energy efficiency of old and new buildings in Romania. Proceedings of the First Workshop: Sustainability of Constructions, Integrated Approach to Life-time Structural Engineering, Lisbon, Portugal, 13-14-15 September 2007. 2.84-2.91.

Danku, G., “Study of the Development of Plastic Hinges in Composite Steel-Concrete Structural Members Subjected to Shear and/or Bending”, PhD Thesis, Politehnica University of Timisoara, 2011.

Degée, H., Lebrun, N., Plumier A. (2010), “Considerations on the Design, Analysis and Performances of Eccentrically Braced Composite Frames under Seismic Action”, Proceedings of SDSS 2010 Conference – Stability and Ductility of Steel Structures, 337-344.

Dubina D., Dinu F., 2012. “Collapse prevention design of multistorey steel building frames under extreme action”. Nordic Steel Construction Conference, 05-07 September 2012, Oslo, Norway, ISBN 978-82-91466-12-5, pp. 1-14.

Dubina D., Ungureanu V., Mutiu M., (2007a), Sustainable building structures for housing, Int. Conf. on Sustainable Buildings 2007: Sustainable Construction. Materials and Practices, Lisbon, Portugal, 12-14 September, Published by IOS Press, 2, 1096-1103.

Dubina D., Ungureanu V., Mutiu M., (2007b), Sustainable mixed building technologies applied to residential buildings: some Romanian examples, Proc. of the first Cost C25 Workshop: Sustainability of Constructions, Integrated Approach to Life-time Structural Engineering, Lisbon, Portugal, September 13-15, 3.93-3.102.

Dubina, D., Dogariu, A., Stratan, A., Bordea, S., 2008 Performance Based Seismic Assessment of Buildings for Evaluation of Retrofitting Systems Efficiency. Interim report on contract no. RFSR-CT-2007-00050 Steel Solutions for the Seismic Retrofit and Upgrade of Existing Constructions, February 2008.

Ecoinvent Centre. Ecoinvent data v1.3. Final reports Ecoinvent 2000 No. 1-15. Swiss Centre for Life Cycle Inventories. Dubendorf. www.ecoinvent.ch, 2004

Elghazouli A.Y., Castro J.M., Izzuddin B.A., Seismic performance of composite moment-resisting frames, Engineering Structures, ISSN 0141-0296, Volume 30, Issue 7, July 2008, Pages 1802–1819

EN 1994-1-1, 2004. Eurocode 4: Part 1.1 – Design of composite steel and concrete structures. Brussels: CEN, European Committee for Standardisation, Final Version, September 2004 .

EN1998-1-1, Eurocode 8, Design of structures for earthquake resistance - Part 1, General rules, seismic actions and rules for buildings, CEN, European Committee for Standardization, 2004.

European Convention for Constructional Steelwork, Technical Committee 1, TWG 1.3 – Seismic Design, No.45, 1986, Recommended Testing Procedures for Assessing the Behaviour of Structural Elements under Cyclic Loads.

Fajfar, P., (2000) "A nonlinear analysis method for performance-based seismic design" Earthquake Spectra, ISSN: 8755-2930 No. 16(3), 573-592.

Feldman, M., Gesella, H. & Leffer, A. 2004 The Cyclic Force-Slip Behaviour of Headed Studs under Non-Static Service Loads – Experimental Studies and Analytical Description Composite Constructions in Steel and Concrete V, Ed. Leon, T. and Lange, J., ASCE 2004.

FEMA 356 / November 2000, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, D.C. November 2000.

Fink A., “Das Momentenrotationsverhalten von Verbundknoten mit Verbundstützen aus Rechteckhohlprofilen”, Diploma-work, Institute for Steel and Timber Structures, University of Innsbruck, 1997.

Fukumoto T., Morita K., “Elastoplastic Behaviour of Panel Zone in Steel Beam-to-Concrete Filled Steel Tube Column Moment Connections”, Journal of Structural Engineering, Vol. 131, No. 12, December 2005, ASCE, ISSN 0733-9445/2005/12 pp. 1841–1853, 2005.

Fülöp L.A., Dubina D., (2004a), Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading. Part I: Experimental research, Thin Walled Structures, 42, 321-338.

Fülöp L.A., Dubina D., (2004b), Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading, Part II: Numerical modelling and performance analysis, Thin Walled Structures, 42, 339-343.

Glaumann M., Kahraman I., (2009), Basic life time considerations in design of buildings and constructions, Proc. of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering, Timisoara, Romania, October 23-24, 333-353.

Goel, S.C., Foutch, D.A., “Preliminary Studies and Test Results of Eccentrically Braced Full-Size Steel Structure”, US-Japan Cooperative Earthquake Research Program Utilizing Large Scale Test Facilities, Proceedings of the 16th Joint Meeting on Wind and Seismic Effects, UJNR, May 15-18, Maryland, 1984.

Hanswille G., Beck H., Neubauer T., “Design concept of nailed shear connections in composite tube columns”, Proceedings of the International Symposium on Connections between Steel and Concrete

Hilti AG, Prüfbericht nr. 172268, Prüfobjekt: Hilti Nägel X-DSH 32 P10 Special, Auftragsnummer 555236, 25-27.05.1998.

Huber G., “Semi-continuous beam-to-column joints at the Millennium Tower in Vienna, Austria”, Steel and Composite Structures, Vol. 1, No. 2, pp. 159-170, 2001.

Huang, Y., Yi, W., Nie, J. (2012), “Seismic Analysis of CFST Frames Considering the Effect of the Floor Slab”, Journal of Steel and Composite Structures, ISSN 1229-9367, 13(4), 397-408.

Hosain, M.U. & Pashan, A. 2004 Channel Shear Connectors in Composite Beams: Push-out Tests Composite Constructions in Steel and Concrete V, Ed. Leon, T. and Lange, J., ASCE 2004.

INNSE 2003. The official census regarding population and dwellings made on March 18-27, 2002. The National Institute of Statistics, Romania. http://www.insse.ro/cms/files/ RPL2002INS/index_eng.htm.

ISO 14040, (2006), Environmental management – Life cycle assessment – Principles and framework, International Organization for Standardization, Geneva, Switzerland.

ISO 14042, (2006), Environmental management – Life cycle assessment – Life cycle impact assessment, International Organization for Standardization, Geneva, Switzerland.

ISO 14044, (2006), Environmental management – Life cycle assessment – Requirements and guidelines, International Organization for Standardization, Geneva, Switzerland.

Johnson, R.P., Willmington, R.T., “Vertical Shear in Continuous Composite Beams”, Proceedings of the Institution of Civil Engineers, 53, Sept., 189 189-205, 1972.

Khan, A.M., Energy Systems: End Uses. Proceedings of Workshop and Conference on Global Change and Environmental Considerations for Energy System Development. Asian Institute of Technology, Bangkok, Thailand, 1992.

Lachal A., Aribert J.M. and Ciutina A, 2004: Seismic Performance Of End-Plate Moment Resisting Composite Joints, Composite Construction in Steel and Concrete V, Conference, South Africa, July 18 – 23, 2004.

Larcher T.Z., “Versuche zur Krafteinleitung der Trägerauflagerkräfte bei Hohlprofilverbundstützen mit Setznägeln”, Diploma-work, Institute for Steel and Timber Structures, University of Innsbruck, 1997.

Liang Q., Uy B., Bradford M., Ronagh H., Strength Analysis of Steel-Concrete Composite Beams in Combined Bending and Shear, Journal of. Structural Engineering ASCE, ISSN 0733-9445, 131(10), pp 1593-1600, Oct. 2005.

Marececk, J., Samec, J. & Studnicka, J. 2005 Perfobound Shear Connector Behaviour Proceedings of EUROSTEEL 2005 4th European Conference on Steel and Composite Structures, Maastricht, 2005 Ed. Hoffmeister, B. and Hechler, O.

Michlmair M., Maydl P., (2008), Life-cycle design of residential buildings appearance and reality of ecological assessments, Proceedings of "Life-Cycle Civil Engineering", Venerra, Italy, June 10-14, 2008, 397-402.

Morino S., Tsuda K., “Design and Construction of Concrete-Filled Steel Tube Column System in Japan”, Earthquake Engineering and Engineering Seismology, Vol. 4, No. 1, pp. 51−73, 2003.

Niculescu, D.D. 1961. The construction of residential buildings with large panels. Bucharest: Technical Publishing House

Okazaki T., Arce G., Ryu H.-C. and Engelhardt, M.D., Experimental study of local buckling, overstrength, and fracture of links in eccentrically braced frames, , Journal of. Structural Engineering ASCE, ISSN 0733-9445, 131(10), pp 1526-1535, Oct. 2005.

Özhendekci D. and Özhendekci N. (2008), “Effects of the Frame Geometry on the Weight and Inelastic Behaviour of Eccentrically Braced Chevron Steel Frames”, Journal of Constructional Steel Research ISSN 0143-974X 64(3), 326–343.

Park J.W., Kang S.M., Yang S.C., “Experimental studies of wide flange beam to square concrete-filled tube column joints with stiffening plates around the column”, Journal of Structural Engineering, Vol. 131, No. 12, December 2005, ASCE, ISSN 0733-9445 / 2005/12 pp. 1866–1876, (DOI:10.1061/ (ASCE)0733-9445(2005)131:12(1866)), 2005.

P100-1-2006: P100-1 Cod de proiectare seismic„ — Partea I — Prevederi de proiectare pentru cladiri, indicativ P 100-1/2006, Romanian Institute of Standardization, 2006.

Plank, R.; Dowling, J. Sustainable Constructions. Steel Buildings, 1st ed.; The British Constructional Steelwork Association Ltd, British Library, Publication No. 35/03, 2003.

Plumier A., Doneux C. Seismic Behaviour and Design of Composite Steel Concrete Structures, European Comission – Training and Mobility of Researchers » - Programme Inovative Concepts in Seismic Design Project ICONS, mai 2001.

Plumier, A. and Doneux, C., European developments of seismic design guidelines for composite steel concrete structures, Proceedings of International Conference of Steel and Composite Structures ICSCS'01, Pusan, Korea, 2001.

Plumier A., Doneux C., Bouwkamp J.G., Slab design in connection zone of composite frames, proceedings of the 1998 Annual Technical Session & Meeting of the Structural Stability Research Council, 21-23 September 1998, Atlanta.

Plumier A. (1990) “New idea for safe structure in seismic zone’, Proceedings of IABSE symposium on mixed structures including new materials, 431–36

Prakash V., Powell G.H. and Campbell S., 1993: DRAIN 2DX – Base Program Description and User Guide, Berkeley University of California, 1993.

Prakash V., Powell G., (2012) DRAIN 2DX – computer manual and software. Online at, http://nisee.berkeley.edu/elibrary/getpkg?id=DRAIN2DX .

Ricles J. M., Popov E. P. Experiments on eccentrically braced frames with composite floors, Earthquake Engineering Research Centre, University of California, Berkeley, California, 1987.

RILEM Publications SARL, Print-ISBN: 2-912143-25-X, e-ISBN: 2351580346, Stuttgart, Germany, September 10-12, pp. 1056-1065, 2001.

Rodriguez, J. 2010. Overcladding Alternatives. Retrived from: http://construction.about.com.

Rossi P.P., Lombardo A., (2007) “Influence of the link overstrength factor on the seismic behaviour of eccentrically braced frames”, Journal of Constructional Steel Research, ISSN 0143-974X Nr. 63 (2007) 1529–1545.

Santos P., Mateus D., da Silva L.S., Rebelo C., Gervásio H., Correia A., Ferreira H., Santiago A., Murtinho V., Rigueiro C., (2010), Affordable houses (Part II): Functional, structural and technological performance, Proc. Of First International Conference on Structures and Architecture (ICSA2010), Guimarães, Portugal, July 21–23, 1298-1306.

Skuber P., 1998: Element 14 User guide: Non-linear modelling of connections in moment resisting frames, University of Ljubljana, Slovenia, 1998.

Shayanfar M.A., Barkhordari M.A., Rezaeian A.R., Experimental study of cyclic behaviour of composite vertical shear link in eccentrically braced frames, Steel and Composite Structures, ISSN 1229-9367 Vol. 12, No. 1 pp13-29, 2011

Shin K.J., Kim I.J., Oh Y.S., Moon T.S., "Behaviour of welded CFT column to H-beam connections with external stiffeners", Engineering Structures 26 (2004) pp. 1877–1887, (doi:10.1016/j.engstruct.2004.06.016), 2004.

SimaPro 7, (2008), Software and Database Manual, Amersfoort, The Netherlands, Online at: www.pre.nl .

Simões R., da Silva L.S., Cruz P., Cyclic behaviour of end-plate beam-to-column composite joints, Steel and Composite Structures, ISSN 1229-9367, Vol. 1, No. 3 (2001) 355-376, 2001.

da Silva L.S., Grecea D., Krigsvoll G., Gervasio H., Blok R., Aktuglu Y., (2007), LCA databases (EPD vs Generic data), Proc. of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering, Lisbon, Portugal, September 13-15, 0.13-0.22.

Stevens D. J., Marchand K. A., McKay A. E., 2009. "Revision of the tie force and alternate path approaches in the DoD progressive collapse design requirements". Proceedings of the 2009 Structures Congress, April 30-May 2, 2009, ASCE, Austin, TX.

Stratan, A., Dubina, D., Removable bolted links for eccentrically braced frames, Proceedings of the International Symposium "Urban Habitat Constructions under Catastrophic Events", Malta, 22-23 October 2008, COST Action C26, Editors: Mazzolani, Mistakidis, Borg, Byfield, De Matteis, Dubina, Indirli, Mandara, Muzeau, Wald, Wang, p. 181-186. ISBN 978-99909-44-42-6, 2008.

Tan S., & Astaneh-Asl A., 2003. "Cable-based retrofit of steel building floors to prevent progressive collapse". University of California, Berkeley, USA.

The Regents of the University of California, (2012) OpenSees computer code. Online at http://opensees.berkeley.edu .

Ţuca, I., Dubina, D., Ciutina, A., Ungureanu, V., Solutions for thermal renovation of precast concrete wall panels – Case study, Proceedings of the International Conference: Sustainability of Constructions – Towards a better built environment COST C25. Innsbruck, 3-5.02.2011, ISBN 978-999957-816-0-6.

UIA 2009. Copenhagen Declaration, Declaration on Sustainability and Cultural Diversity, Approval by the UIA World Congress in June 2008, www.uia-architectes.org

Yuan H., Weijian Y., Rui Z., Ming X., "Behaviour and design modification of RBS moment connections with composite beams", Engineering Structures 59 (2014) 39–48

Yurisman, Budiono B. Moestopo M., Suarjana M., Behaviour of Shear Link of WF Section with Diagonal Web Stiffener of Eccentrically Braced Frame (EBF) of Steel Structure, ITB Journal of Engineering Science ISSN 1978-3051 Vol. 42, No. 2, 2010, 103-128.

Wang W., Chen Y., Li W., Leon R.T., "Bidirectional seismic performance of steel beam to circular tubular column connections with outer diaphragm", Earthquake Engineering and Structural Dynamics (2010), (DOI: 10.1002/ eqe.1070), 2010.

Zona A., Barbato M., Conte J., (2008) "Nonlinear Seismic Response Analysis of Steel–Concrete Composite Frames", Journal of Structural Engineering ASCE, ISSN 0733-9445 134(6), 986–997.