HABILITATION THESIS

Contributions to the recent research advances in cold-formed steel structures and sustainability of steel framed buildings

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HABILITATION THESIS

"Contributions to the recent research advances in cold-formed steel structures and sustainability of steel framed buildings"

(a) ABSTRACT

Present thesis summarises the research activity of the candidate after defending the PhD Thesis at The "Politehnica" University of Timişoara and confirmed by The Ministry of Education and Research, on the basis of Order no. 3876, dated 19.05.2004.

The research activity and achievements presented here are developed in two main thematic directions. The first one is *"Thin-walled cold-formed steel members"*, which continues and diversify with new subjects, the topic of the PhD Thesis.

It should be noted that the activity of the candidate in the field of thin-walled cold-formed steel structures (18 years of research in this field), from the beginning, from November 1994, until the defending of PhD Thesis, and for the post-thesis period, is in line with the fields of research of Steel Structures Research School of Timişoara. It may be underlined the long tradition of the Timişoara School in the field of thin-walled cold-formed steel structures – more than 40 years of activity.

The new subjects of research in the post-thesis period can be divided in two big classes, each of them related to the following aspects:

Theoretical contributions:

- Post-elastic strength of thin-walled cold-formed steel members. Plastic mechanisms for members in compression and bending;
- Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections;
- Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed steel members;
- Behaviour of cold-formed steel perforated pallet rack sections in compression.

Innovative structural systems:

- *Wall Stud Modular System* (WSMS) – bungalow type buildings used as shops, offices, for industrial purposes, for housing or school facilities;
- *Hall Type Framed Structure* (HTFS) – are used as small and medium size single storey industrial buildings and storehouses;
- *Penthouse Framed Structure* (PFS) – for refurbishment and restructuring by vertical addition of new storey the existing buildings;
- *Affordable Houses* (AH).

For these innovative structural systems both technical and environmental performances have been studied.

The main achievements and results are presented in detail in Chapter (b-i): *Scientific, professional and academic achievements.*

The second direction, entitled *"Sustainability of construction"*, is a new field of research. It should be underlined that in Romania, in general, and in the construction sector, in particular, not too much work has been done in the field of sustainability. The work done by the candidate and by the team from CMMC Department of the Civil Engineering Faculty from Timişoara represents, probably, the only scientific work in this field in Romania related to the construction sector. The activity started in 2006, when the candidate was appointed as member of Management Committee in the COST C25 action: "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*" and vice-chairman of Working Group WG3: "*Life-time Structural Engineering*" of COST C25 action. Since 2008 the candidate is member of Technical Committee TC14 "*Sustainability and Eco-efficiency of Steel Building*" of the European Convention for Constructional Steelwork (ECCS). More than 30 papers have been published in this field.

Three case studies of sustainable mixed building technologies, which combine steel in the framing and different materials for cladding, roofing and flooring, in order to obtain highly performance thermo-energetic properties are presented in this thesis:

Case study 1. The aim of the first case study is a comparative life cycle analysis (LCA) between a cold-formed steel framing house and an identical one made of masonry structural walls and concrete floor, according to Romanian traditional houses. The impact analyses were performed at different levels: (i) construction, (ii) construction and end-of-life, (iii) life cycle including maintenance and, (iv) the life cycle including maintenance and consumable goods.

Case study 2. This case study presents an affordable house, with innovative structure/envelope solutions, enabling flexible floor plans and modular construction, faster fabrication and erection times and high solution diversity for floors and envelope. The main features of the architectural and technical solution are presented together with a comparative LC analysis for the above house, designed in various solutions, i.e.: (1) hot-rolled steel framing – the innovative structure; (2) cold-formed steel framing; (3) timber framing, and (4) masonry structural walls and concrete floor. Several aspects are presented, in order to underline the impact for different stages during the life cycle: (a) construction stage only; (b) construction stage and final disposal of materials; (c) construction stage, disposal and maintenance.

Case study 3. The case study presents a block of flats built in 2007 in Timisoara, Romania. The main structure is made of steel profiles with light floors. Some innovative design solutions have been used in this project. Particular attention has been paid to the aspects related to design and detailing, as well as solutions for cladding, including structural features and thermo-energetic performance. A comparative study, related to the environmental impact analysis for the construction stage only, for the structure designed in two solutions: (a) steel framed structure and (b) reinforced concrete framed structure is presented.

In what concerns the future research and development plans of the candidate, related to the fields of research presented above, the following research topics will continue or will be developed:

A) Thin-walled cold-formed steel members

- To extend the database for plastic mechanisms for members in eccentric compression;
- Codification of imperfections for cold-formed steel members and frames;
- Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections restrained by sheeting;
- Cold-formed steel beams of screwed corrugated webs;
- Influence of residual stresses on the perforated pallet rack members due to the cold-reducing process;
- The behaviour of pallet rack systems under seismic action;
- Modular / affordable / emergency steel intensive houses.

B) Sustainability of construction

- Sustainable design, climate change and the built environment;
- Structural glass;
- Retrofitting of existing prefabricated concrete residential buildings to reduce the primary energy use and GHG emissions.

A short description of each topic has been done in Chapter (b-ii): *Scientific, professional and academic future development plans.*

Finally, it have to be underlined that the active role of the candidate will continuously increase by participation with new research topics to international conferences and papers published in specialised journals. Also, as members of Technical Committee TC7 "*Cold Formed Thin Walled Sheet Steel in Building*" and Technical Committee TC14 "*Sustainability & Eco-Efficiency of Steel Construction*" of the European Convention for Constructional Steelwork (ECCS), the candidate will continue the collaborative work with researchers in the field.

(b) ACHIEVEMENTS AND DEVELOPMENT PLANS

(b-i) Scientific, professional and academic achievements

1. Introduction

Present thesis summarises the research activity of the candidate after defending the PhD Thesis at The "Politehnica" University of Timişoara and confirmed by The Ministry of Education and Research, on the basis of Order no. 3876, dated 19.05.2004.

It should be noted that the activity of the candidate in the field of thin-walled cold-formed steel structures (18 years of research in this field), from the beginning, from November 1994, until the defending of PhD Thesis, and for the post-thesis period, is in line with the fields of research of Steel Structures Research School of Timişoara. It may be underlined the long tradition of the Timişoara School in the field of thin-walled cold-formed steel structures – more than 40 years of activity.

The research activity and achievements presented here are developed in two main thematic directions. The first one is *"Thin-walled cold-formed steel members"*, which continues and diversify with new subjects, the topic of the PhD Thesis entitled "*Contributions to the lateral-torsional buckling of thin-walled cold-formed beams".*

The new subjects of research in the post-thesis period can be divided in two big classes, each of them related to the following aspects:

Theoretical contributions:

- Post-elastic strength of thin-walled cold-formed steel members. Plastic mechanisms for members in compression and bending;
- Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections;
- Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed steel members;
- Behaviour of cold-formed steel perforated pallet rack sections in compression.

Innovative structural systems:

- *Wall Stud Modular System* (WSMS) - bungalow type buildings used as shops, offices, for industrial purposes, for housing or school facilities;
- *Hall Type Framed Structure* (HTFS) – are used as small and medium size single storey industrial buildings and storehouses;
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For these innovative structural systems both technical and environmental performances have been studied.

The second direction, entitled *"Sustainability of construction"*, is a new field of research. It should be underlined that in Romania, in general, and in the construction sector, in particular, not too much work has been done in the field of sustainability. The work done by the candidate and by the team from CMMC Department of the Civil Engineering Faculty from Timişoara represents, probably, the only scientific work in this field in Romania related to the construction sector. The activity started in 2006, when the candidate was appointed as member of Management Committee in the COST C25 action: "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*" and vice-chairman of Working Group WG3: "*Life-time Structural Engineering*" of COST C25 action. Since 2008 the candidate is member of Technical Committee TC14 "*Sustainability and Eco-efficiency of Steel Building*" of the European Convention for Constructional Steelwork (ECCS). More than 30 papers have been published in this field.

Articles constituting the habilitation thesis

This is a survey of the results constituting my habilitation thesis. It is based on the following articles.

1. Kotełko M., **Ungureanu V.**, Dubina D., Macdonald M. (2011). "Plastic strength of thin-walled plated members—Alternative solutions review". Thin Walled Structures, 49(5), pp. 636-646, (DOI: 10.1016/j.tws.2010.09.007) (2 ISI citations).
2. **Ungureanu V.**, Kotełko M., Mania R.J., Dubina D. (2010). "Plastic mechanisms database for thin-walled cold-formed steel members in compression and bending". Thin Walled Structures, 48(10-11), pp. 818-826, (DOI: 10.1016/j.tws.2010.01.004) (12 ISI citations).
3. Bambach M.R., Rasmussen K.J.R., **Ungureanu V.** (2007). "Inelastic behaviour and design of slender I-sections in minor axis bending". Journal of Constructional Steel Research, 63(1), pp. 1-12, ISSN 0143-974X (DOI: 10.1016/j.jcsr.2006.03.001).
4. Dubina D., **Ungureanu V.** (2010). "Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections". Thin-Walled Structures, 48(10-11), pp. 866-871. ISSN 0263-8231 (DOI: 10.1016/j.tws.2010.04.003) (3 ISI citations).
5. Georgescu M., **Ungureanu V.** (2010). "Improved design model for thin-walled cold-formed purlins continuously connected to sandwich panel roofing". Proceedings of the First International Conference on Structures and Architecture – ICSA 2010, 21-23.07.2010, Guimaraes, Portugal, ISBN 978-0-415-49249-2, pp. 153-154+CD. (MATHnetBASE).
6. Dubina D., **Ungureanu V.**, Stratan A. (2008). "Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed members". Proceedings of the 5th International Conference on Thin-Walled Structures: Recent Innovations and Developments, 18-20.06.2008, Gold Coast, Australia, Vol. 1, ISBN 978-1-74107-239-6, pp. 387-394.
7. Crișan A., **Ungureanu V.**, Dubina D. (2012). "Behaviour of cold-formed steel perforated sections in compression. Part 1 - Experimental investigations. Part 2 - Numerical investigations". Proceedings of the 6th International conference on Thin-Walled Structures: Recent research Advances and Trends. 5-7 September 2011, Timisoara, Romania, Vol. 2, ISBN: 978-92-9147-102-7, pp. 795-812.
8. Dubina D., **Ungureanu V.**, Ciutina A., Tuca I., Mutiu M. (2010). "Sustainable detached family house - case study". Journal of Steel Construction. Design and Research. Vol. 3, pp. 154-162, ISSN 1867-0520 (Wiley Online Library).
9. Dubina D., **Ungureanu V.**, Ciutina A., Mutiu M., Grecea D. (2010). "Innovative sustainable steel framing based affordable house solution for continental seismic areas". Proceedings of the First International Conference on Structures and Architecture – ICSA 2010, 21-23.07.2010, Guimaraes, Portugal, ISBN 978-0-415-49249-2, pp. 367-368+CD (MATHnetBASE).
10. **Ungureanu V.**, Ciutina A., Tuca I., Dubina D., Grecea D., Fulop L. (2011). "Structural and Environmental Performance of Steel Framed Houses: A Case Study". Summary report of the Cooperative Activities. Cost Action C25. Volume 1: Integrated approach towards sustainable constructions, ISBN 978-99957-816-1-3, pp. 395-428.

2. Thin-walled cold-formed steel members

National and international experience of the candidate related to the topic
"Thin-walled cold-formed steel members"
(Post-PhD Thesis period)

Member of technical boards:

- Member of Technical Committee TC7 "Cold-Formed Thin-Walled Sheet Steel in Building" of the European Convention for Constructional Steelwork (ECCS);
- Member of AICPS – Romanian Association of Structural Engineers;
- Member of APCMR – Romanian Association of Steelwork Producers;
- Member of APK – Association pour la Promotion de l'Enseignement de la Construction Acier, France.

Conference committees:

- Scientific Secretary of the International Conference on Thin-Walled Structures: *Recent research advances and trends*, Timisoara, Romania, 5-7 September 2011;
- Scientific Secretary of the International Conference in Metal Structures: *Steel – A New and Traditional Material for Building*, Poiana Braşov, Romania, 20-22.09.2006;
- Member of Scientific Committee of The 6th International Conference on Coupled Instabilities in Metal Structures, 03-05 December 2012, Glasgow, UK;
- Chairman of Technical Session: Thin-walled Beams and Columns. The 5th International Conference on Thin-Walled Structures: Recent Innovations and Developments, Gold Coast, Australia, 18-20 June 2008.

Research projects:

- SkyClad/29.01.2007. *Consulting and technical assistance in handling the specific problems of designing of thin-walled cold-formed steel structures according to EN1993-1.3*. Beneficiary: Sky Clad Ltd., Ireland. Total value: 3000€ (coordinator of research team).
- 65/01.07.2009. *Experimental testing on storage racking components*. Beneficiary: SC Dexion HI-LO Storage Solutions SRL. Total value: 18000€ (member research team).

Invited papers and courses:

- Invited lecturer at AKKON Steel Structures System Inc.**,** 28.11.2008**,** Cerkezkoy – Tekirdag**,** Turkey (Lecture: *Technical solutions end erection technology for light gauge building steel framing of cold-formed steel sheathing based envelops.* Authors: **Viorel Ungureanu** & Dan Dubina).
- Invited paper: *Ultimate Capacity of Cold-Formed Pitched Roof Portal Frames*, at The 9th Constructional Steelwork Day – TUCSA, 27.10.2008, Istanbul, Turkey. Authors: Dan Dubina, **Viorel Ungureanu**, Aurel Stratan & Zsolt Nagy.

Reviewer in ISI journals:

- Thin-Walled Structures (www.elseviere.com)
- Engineering Structures (www.elseviere.com)
- Advances in Engineering Software (www.elseviere.com)
- Journal of Structural Engineering – ASCE (http://ascelibrary.org/sto/)

Books:

- Dubina D., Ungureanu V., Landolfo R. (2012): *Design of Cold-formed Steel Structures. Eurocode 3: Design of Steel Structures. Part 1-3 Design of cold-formed Steel Structures*. ISBN-13: 978-3-433-02979-4, Ernst & Sohn, A Wiley Company, Berlin, 512 pages.
- Ungureanu V. (2006): *Construcţii metalice uşoare din profile de oţel formate la rece. Probleme speciale de calcul*. ISBN 13-978-973-638-279-6, Editura Orizonturi Universitare Timişoara, 160 pages.
- Dubina D., Ungureanu V., Zaharia R., Nagy Zs. (2004): *Calculul şi proiectarea construcţiilor din profile metalice cu pereţi subţiri formate la rece. Volumul I*. ISBN 973-86509-4-1, Editura AMM, Colecţia Lindab, Bucureşti, 256 pages.

Member in PhD Juries related to the "Thin-walled cold-formed steel members" topic

- Member in the PhD Jury of Andrei Crişan: "*Buckling strength of cold formed steel sections applied in pallet rack*". The "Politehnica" University of Timisoara, Civil Engineering Faculty, September 2011.
- Member in the PhD Jury of Lucian-Dumitru Gîlia: "*Structural performances of cold-formed girders made of lipped channel sections for flanges and corrugated web*". The Technical University of Cluj-Napoca, Civil Engineering Faculty, March 2012.

Scientific articles with international partnership

1. Kotelko M., **Ungureanu V.**, Dubina D., Macdonald M.: *Plastic strength of thin-walled plated members—Alternative solutions review*. Thin-Walled Structures. 49 (2011), 636–644 (DOI: 10.1016/j.tws.2010.09.007, Citations: 2).
2. **Ungureanu V.**, Kotelko M., Mania R.J., Dubina D.: *Plastic mechanisms database for thin-walled cold-formed steel members in compression and bending*. Thin-Walled Structures. 48(10-11), 2010, 818-826 (DOI: 10.1016/j.tws.2010.01.004, Citations: 12).
3. **Ungureanu V.**, Dubina D., Kotełko M., Mania R.J.: *Plastic strength of thin-walled plated members - alternative solutions review, Part I. Numerical analysis*. Proceedings of the XIIth Symposium on Stability of Structures (ISBN 83-914019-6-0). Zakopane, Polonia, 07-11.09.2009, 451-460.
4. **Ungureanu V.**, Dubina D., Kotełko M., Mania R.J.: *Plastic strength of thin-walled plated members - alternative solutions review, Part II. Numerical vs. experimental comparisons*. Proceedings of the XIIth Symposium on Stability of Structures (ISBN 83-914019-6-0). Zakopane, Polonia, 07-11.09.2009, 461-470.
5. **Ungureanu V.**, Kotelko M., Mania R.J., Dubina D.: *Plastic mechanisms database for thin-walled cold-formed steel members in compression and bending*. Proceedings of the Fifth Conference on Coupled Instabilities in Metal Structures – CIMS 2008, Sydney, Australia, 23-25 June 2008. Vol. 1, ISBN: 978-0-646-49439-5, 189-197.
6. Bambach M.R., Rasmussen K.J.R., **Ungureanu V.**: *Inelastic behaviour and design of slender I-sections in minor axis bending*. Journal of Constructional Steel Research. 63(1), 2007, 1-12 (DOI: 10.1016/j.jcsr.2006.03.001).
7. **Ungureanu V.**, Bambach M.: *Plastic strength of thin-walled I-section members subjected to minor axis bending*. Proceedings of the International Conference in Metal Structures: *Steel – A New and Traditional Material for Building*. Poiana Brasov, Romania, 20-22 September 2006, Eds D. Dubina & V. Ungureanu, Taylor & Francis Group / Balkema, ISBN 10:0-415-40817-2 (ISBN 13: 0-415-40817-2), 251-256.
8. Dubina D., **Ungureanu V.**, Rondal J.: *Numerical modelling and codification of imperfections for cold-formed steel members analysis.* Steel and Composite Structures – An International Journal, Vol. 5, No. 6, 2005, 515-533.

9. Dubina D., **Ungureanu V.**, Rondal J.: *Numerical Modelling and Codification of Imperfections for Cold-formed Steel Members Analysis*. Proceedings of The Second International Conference on Steel & Composite Structures, Seoul, Korea, 2-4 September 2004, ISBN 89-89693-12-8-98530, pp. 132 + CD, pp. 206-223.
10. Rondal J., Dubina D., **Ungureanu V.**: *Imperfections and computational modelling of cold-formed steel members*. International Colloquium „Recent Advances and New Trends in Structural Design", 7-8 May 2004, Timişoara, Romania, Ed. Orizonturi Universitare, ISBN 973-638-119-6, pp. 209-220.

Editor of international proceedings and journals

1. Dubina D., Ungureanu V. (Guest editors): Special Issue of Thin Walled Structures: Recent research advances on thin-walled structures. Publication date: October 2012, ISSN 0263-8231.
2. Dubina D., Ungureanu V. (editors): Proceedings of the 6th International Conference on Thin-Walled Structures: Recent Research Advances and Trends. Volume 1 + 2. 5-7.09.2011, Timisoara, Romania, ISBN: 978-92-9147-102-7, 1064 pp.
3. Dubina D., Ungureanu V. (editors): Steel – A New and Traditional Material for Building (Proceedings of the International Conference in Metal Structures, Poiana Braşov, Romania, 20-22.09.2006). Taylor & Francis Group / Balkema, London, UK, 2006, ISBN 10:0-415-40817-2 (ISBN 13: 0-415-40817-2), 648 pp.

2.1 Introduction

The field of *Thin-walled cold-formed steel members*, among other topics including steel, is a traditional research subject of the CEMSIG Research Centre from the “Politehnica” University of Timisoara. The Research Centre for Mechanics of Materials and Structural Safety – CEMSIG (http://cemsig.ct.upt.ro/) is a RTD (Research and Technical Development) unit of the "Politehnica" University of Timisoara, at the Faculty of Civil Engineering, Department of Steel Structures and Structural Mechanics. The research centre was established in 1999. In 2001 CEMSIG was qualified as Research Centre of Excellence by the National University Research Council (CNCSIS). In 2006 CEMSIG was again qualified as Research Centre of Excellence.

The research activity in the field of thin-walled cold-formed steel members (TWCFSM) has been and continues to be focused on several topics:

- Evaluation of the effective geometrical characteristics of thin-walled members;
- The interaction between overall and local/distortional buckling and interactive buckling curves for cold-formed steel members;
- Experimental evaluation of the ultimate resistance of centrically and eccentrically compression members;
- Experimental calibration of stiffness of bolted joints in cold-formed steel trusses;
- Post-elastic strength of thin-walled cold-formed steel members;
- Design assisted by testing of seismic resistant cold-formed steel framed houses;
- Design assisted by testing of pitched-roof cold-formed steel portal frames considering the actual behaviour of joints;
- Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections on intermediate supports;
- Behaviour of cold-formed steel perforated pallet rack sections in compression.

In fact, the members of TWCFSM team are the authors of former and actual related Design Codes and Good Practice Guides in Romania and not only.

In the ‘80s the CMMC team was involved together with the colleagues from INCERC, Bucharest Construction Institute and IPROMET in drafting the first technical recommendations to design the steel structures of thin-walled cold-formed sections – P.54-80 (*Instrucţiuni tehnice pentru proiectarea construcţiilor din profile de oţel cu pereţi subţiri, formate la rece*. Buletinul Construcţiilor vol. 8/80). In 1983, CMMC team was involved to draft the first standard for thin-walled cold-formed sections, i.e. STAS 10108/2-83: *Steel Structures. Design of steel structures of thin-walled cold-formed sections (Construcţii din oţel. Calculul elementelor din oţel alcătuite din profile cu pereti subtiri, formate la rece).*

In 1997, a new Romanian design code aligned to ENV1993-1-3:1996 was drafted by a team including "Politehnica" University of Timisoara (Dan Dubina, Mircea Georgescu, Raul Zaharia), Romanian Academy – Timişoara Branch (Viorel Ungureanu) and INCERC Timişoara (Adriana Moldovan), i.e. NP012-1997: *Normativ pentru calculul elementelor din oţel cu pereţi subţiri formate la rece (Design code for steel structures made of thin-walled cold-formed steel sections)*. Buletinul Construcţiilor vol. 15/1998, pp. 3-183.

On the same period two seminars and associated volumes of worked examples were prepared at the CMMC Department, together with colleagues from other universities from abroad, in the frame of TEMPUS JEP 4502 Project, i.e.:

1. *Seminar on Eurocode 3-Part. 1.3: Cold-formed gauge members and sheeting*, TEMPUS JEP 4502, Timişoara, 03.07.1995.Editura Klidarithmos C. Books, Greece, Eds. D. Dubina, I. Vayas, 1995, ISBN 960-332-038-2, 274 pages.
2. *EUROCODE 3 – Exemple de calcul (Design of Steel Structures; EUROCODE 3-Worked Examples)* (bilingual edition). Timişoara, Romania, Eds. D. Dubina, J. Rondal & I. Vayas, 1997, ISBN 963-04-8383-1, 287 pages.

Other two important events, were devoted to coupled instabilities and light gauge metal structures, were the Advanced Courses organised by CISM Udine in October 1996 under the co-ordination of Professor Rondal and in 2002 under the co-ordination of Professors Rondal and Dubina. An outstanding team of lecturers, among them Professors Gioncu and Dubina from Timisoara, contributed with high-level lectures to these courses. The lectures are published in two volumes by Springer-Verlag, i.e.:

- Rondal J (Ed.) *Coupled instabilities in metal structures. Theory and practical aspects.* New York: Springer-Verlag. Series: CISM International Centre for Mechanical Sciences, Number 379, 1998.
- Rondal J., Dubina D. (Eds.). *Light Gauge Metal Structures Recent Advances*. New York: Springer-Verlag. Series: CISM International Centre for Mechanical Sciences, Number 455, 2005.

In recent years, starting with the integration in EU, the Eurocodes providing a common approach for the design of buildings and other civil engineering works and construction products had to be translated into Romanian and to create the National Annexes containing all Nationally Determined Parameters. Concerning the field of thin-walled cold-formed members (not only steel), the following codes have been translated by CEMSIG team:

- Eurocode 3: Design of steel structures – Part 1.3: General rules – Supplementary rules for cold-formed members and sheeting + National Annex.
- Eurocode 3: Design of steel structures – Part 1.4: General rules – Supplementary rules for Stainless steels + National Annex.
- Eurocode 3: Design of steel structures – Part 1.5: Plated structural elements + National Annex.
- Eurocode 3: Design of steel structures – Part 1.6: Strength and Stability of Shell Structures + National Annex.
- Eurocode 9: Design of aluminium structures – Part 1.4: Cold-formed structural sheeting + National Annex.

Several books related to the field of thin-walled cold-formed steel members in the national or international circuit have been published by the members of CEMSIG Research Centre, i.e.:

- Dubina D., Ungureanu V., Zaharia R., Nagy Zs.: *Calculul şi proiectarea construcţiilor din profile metalice cu pereţi subţiri formate la rece*. Volumul I. Editura AMM, Colecţia Lindab, Bucureşti 2004, ISBN 973-86509-4-1, 256 pages (in Romanian).
- Ungureanu V.: *Construcţii metalice uşoare din profile de oţel formate la rece. Probleme speciale de calcul*. Editura Orizonturi Universitare Timişoara, 2006, ISBN 10-973-638-279-6 (ISBN 13-978-973-638-279-6), 160 pages (in Romanian).
- Vayas I, Dubina D (2004). *Cold-formed steel structures*. Kleidarithmos Publishing House, Athena (in Greek).
- Santos P., Simoes de Silva L., Ungureanu V. (2012): *Energy efficiency of light-weight steel-framed buildings*. Technical Committee 14: Sustainability and Eco-efficiency of Steel Constructions, No. 129/2012, Published by ECCS – European Convention for Constructional Steelwork, ISBN: 978-92-9147-105-8, 175 pages.
- Dubina D., Ungureanu V., Landolfo R. (2012): *Design of Cold-formed Steel Structures. Eurocode 3: Design of Steel Structures. Part 1-3 Design of cold-formed Steel Structures*. ISBN-13: 978-3-433-02979-4, Ernst & Sohn, A Wiley Company, Berlin, 634 pages.

Last but not least, several international conferences have been initiated or organised by the members of Steel Structures and Structural Mechanics Department (CMMC Department) of the "Politehnica" University Timişoara.

Every three years since 1973, the Department of Steel Structures and Structural Mechanics of the "Politehnica" University in Timisoara, has organized an international conference dedicated to steel structures. In 1982 the conference included the *East-European Session of the SSRC International Colloquium on the Stability of Steel Structures*. Throughout these years, the organization of the International Conference in Metal Structures, ICMS, has continued, and the

11th edition (ICTW 2006) has been organised in association with the 2006 ECCS Annual Meeting, and the location has been changed to Poiana Brasov.

One of the most important contributions of the Timişoara Stability Research School to international co-operation in the field was the initiation, together with MSM Department of the University of Liege, of *the International Conferences on Coupled Instabilities in Metal Structures*. The first one, *CIMS'92*, took place in Timisoara on 10–12 October 1992. 60 authors from 19 countries presented 61 contributions that were published in a Special Issue of *Thin-Walled Structures*, in two parts, with about 600 pages. The Special Issue had J. Rondal, D. Dubina & V. Gioncu as Guest Editors (*Coupled instabilities in metal structures.* Special issue of Thin-Walled Structures Journal, 1994:19(2-4), 1994:20(1-4)).

The second conference, CIMS'96, held in Liege on 5-7 September 1996, was characterised by a larger participation – 166 authors from 23 countries presented 62 contributions published in a volume of 596 pages, edited by the same team (J. Rondal, D. Dubina & V. Gioncu, Eds.). This conference follows the successful series of CIMS conferences, Lisbon (2000), Rome (2004) and Sydney (2008). The sixth international conference on Coupled Instabilities in Metal Structures, CIMS 2012, will be held in December at Strathclyde University, Scotland, UK.

Although it is not connected directly to structural stability analysis and cold-formed steel members, it has to be emphasised that the series of *STESSA International Conferences*, devoted to the behaviour of steel structures in seismic areas, was also initiated and started from Timisoara, as a fruitful co-operation with Naples University. The first STESSA conference dedicated to *Steel Structures in Seismic Areas* was held in 1994 in Timişoara. The next editions of the Conference were held in Kyoto (Japan, 1997), Montreal (Canada 2000), Naples (Italy, 2003), Yokohama (Japan, 2006), and Philadelphia (United States, 2009), Santiago de Chile (2012).

In September 1999, the 1st Session of the 6th SDSS Colloquium was organized in Timişoara within the series of SSRC colloquia dedicated to the *Stability and Ductility of Steel Structures*.

The highly successful series of *International Conferences on Thin Walled Structures* started in December 1996 in Glasgow continuing with the ones in Singapore (1998), Krakow (2001), Loughborough (2004) and Brisbane (2008). The 6th edition has been organized in Timisoara, Romania, in September 2011. The Proceedings of the ICTWS 2011 Conference, organized in two volumes, contain 115 papers, by 224 authors from 33 countries of Africa, Asia, Australia, Europe, North America and South America.

It have to be underlined the active role of Professor Dan Dubina and Assoc. Professor Viorel Ungureanu as members of Technical Committee TC7 "*Cold Formed Thin Walled Sheet Steel in Building*" of the European Convention for Constructional Steelwork (ECCS).

Finally, it can be observed the long tradition of Steel Structures Research School of Timisoara, of more than 40 years of activity in the field of *Thin-walled cold-formed steel structures*.

It should be noted that the activity of the candidate in the field of thin-walled cold-formed steel structures, from the beginning of research activity until the defending of PhD Thesis, and for the post-thesis period, is in line with the fields of research of Timisoara Stability Research School. For Timisoara Stability Research School, the field of research on thin-walled cold-formed steel structures is worldwide recognised.

2.2 Post-elastic strength of thin-walled cold-formed steel members. Plastic mechanisms for members in compression and bending [2.7, 2.8]

Introduction

Short members of thin-walled cold-formed (TWCF) steel sections in compression and bending fail by forming local plastic mechanisms [2.1, 2.2]. Due to the local buckling, its

behaviour in the post-buckling range displays large local elasto-plastic and plastic deformations of the cross-section walls. Subsequently, yield lines occur and the plastic mechanism of failure is being formed.

For a slender member, multiple single buckling modes, with *m-1*, *m*, *m+1* half-wavelengths, may interact in a first form of the interaction, characterised by an unstable post-critical behaviour. It is very important to know that such an interaction provides a localisation of the buckling patterns, because the localised mode has always a more pronounced unstable slop than periodical one. The second interaction occurs between the stable post-critical overall buckling mode with an unstable post-critical localised buckling one and yields to a very unstable post-critical behaviour. This interaction produces a great erosion of the critical load due to geometrical imperfection, and characterises the behaviour of TWCF members.

For a slender member, when the localized buckling mode occurs prior to the overall one, the collapse, due to the interaction between these two modes, is always characterised by local plastic mechanism failure. In fact, the interaction occurs between the overall mode, which corresponds to an elastic non-linear behaviour of the TWCF steel members and the local plastic buckling of the component walls [2.3]. This kind of behaviour is confirmed both by tests and numerical simulations.

The local plastic mechanism depends on the type of the cross-section and slenderness of component walls, as well as the stress state, and can be significantly influenced by imperfections. A proper identification of the geometry of a local plastic mechanism is crucial for the correct evaluation of the post-buckling rigid-plastic curve (*failure curve*), which subsequently results in a correct estimation of the load-carrying capacity of the member, as well as of the energy absorption at collapse [2.4, 2.5].

The aim of this chapter is to create a database of plastic mechanisms of failure for TWCF steel members subject to compression and/or bending [2.7, 2.23]. Such a database will be useful both in the prediction of the load-carrying capacity and collapse behaviour (including energy absorption) of these structural members.

The plastic mechanisms approach is based on two basic methods [2.6], namely the *energy method (work method)* and the *equilibrium strip method*. A discussion and comparison of results obtained using both methods in the analysis of collapse behaviour of TWCF steel members has been carried out by Zhao [2.6].

Using the energy method we apply the *Principle of Virtual Velocities* of the general following form:

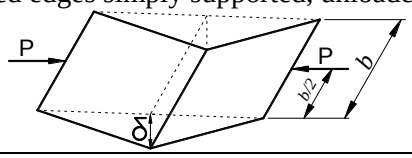
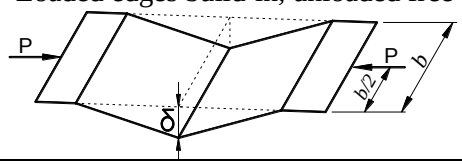
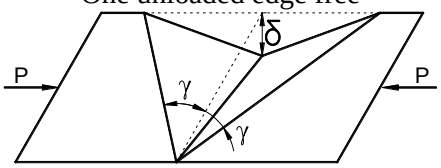
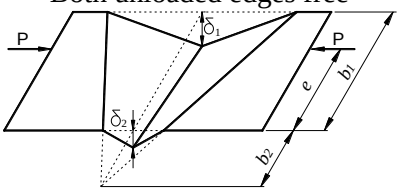
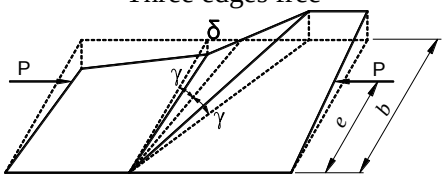
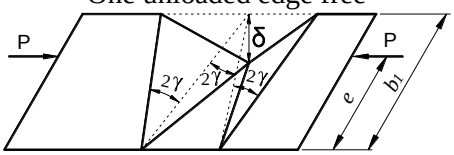
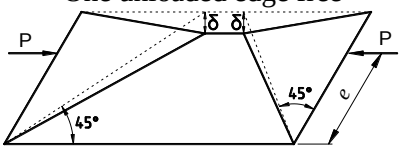
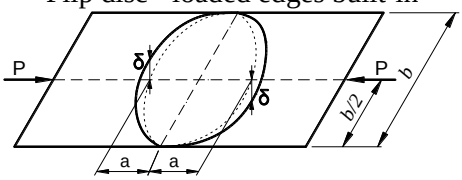
$$P \cdot \dot{\delta} = \int_V \sigma_{ij} \dot{\varepsilon}_{ij}^{p} \ (\beta, \chi) dV \tag{2.1}$$

where δ is the global generalized displacement, $\dot{\delta}$ is the rate of change of the global generalized displacement, β is the vector of kinematical parameters of the plastic mechanisms, χ is the vector of geometrical parameters of the plastic mechanisms, $\dot{\varepsilon}_{ij}^{p}$ is the strain rate tensor. As a result, it will be obtain a load – deformation relation, the graphical representation of which will be termed below as a *failure curve*. The intersection of the elastic curve and the rigid-plastic one can be used to estimate the ultimate strength of a short member.

A starting point of the plastic mechanism analysis, by means of both methods mentioned above, is determination of the geometrical model of the mechanism. Thus, a database of potential failure plastic mechanisms for TWCF members seems to be useful in the process of generalization of existing solutions and creation of new solutions for certain TWCF members.

Plastic mechanisms of plates subjected to non-uniform compression under mixed boundary conditions

Table 2.1: Basic plastic mechanisms of plate elements subjected to different boundary conditions

No.	Mechanism	Load – deflection relation
1	Loaded edges simply supported, unloaded free 	$P=\sigma_y bt\left[\sqrt{(2\delta/t)^2+1}-\frac{2\delta}{t}\right]$
2	Loaded edges build-in, unloaded free 	$P=\sigma_y bt\left[\sqrt{(\delta/t)^2+1}-\frac{\delta}{t}\right]$
3	One unloaded edge free 	$P=\frac{\sigma_y tb}{2}\left[\sqrt{\left(\frac{2\delta}{k_1 t}\right)^2+1}-\left(\frac{2\delta}{k_1 t}\right)+\frac{k_1 t}{2\delta}\ln\left\{\sqrt{\left(\frac{2\delta}{k_1 t}\right)^2+1}+\frac{2\delta}{k_1 t}\right\}\right]$ $Pe=\frac{\sigma_y t^3 b^2 k_1^2}{12\delta^2}\left[\sqrt{\left[\left(\frac{2\delta}{k_1 t}\right)^2+1\right]^3}-1-\left(\frac{2\delta}{k_1 t}\right)^3\right]$
4	Both unloaded edges free 	Relation for mechanism no. 3 $P=P_1-P_2$ $Pe=P_1e_1-P_2e_2$
5	Three edges free 	Relation for mechanism no. 3 but k_1 replaced by k_2
6	One unloaded edge free 	Relation for mechanism no. 3 but k_1 replaced by k_3
7	One unloaded edge free 	Relation for mechanism no. 5 with $\gamma = 45°$
8	Flip disc - loaded edges built-in 	$P=\frac{\sigma_y tb}{6}\left[1-\frac{2\delta}{t}+\sqrt{\left(\frac{2\delta}{t}\right)^2+1}-\right.$ $\left.-\frac{6\delta}{t(1+4a^2/b^2)}+4\sqrt{\left\{\frac{3\delta}{2t(1+4a^2/b^2)}\right\}^2+1}\right]$
$k_1=1+\sec^2\gamma$	$k_2=\sec^2\gamma$	$k_3=2\sec^2\gamma$

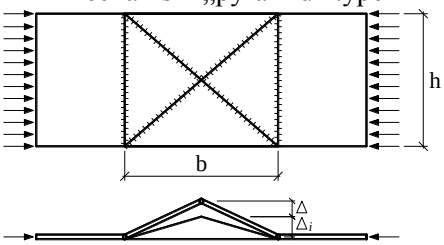
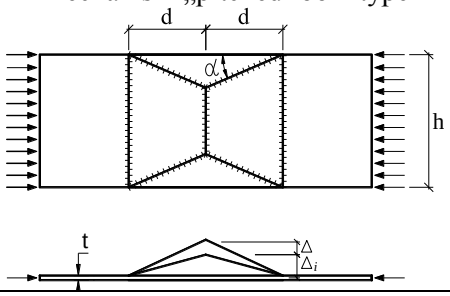
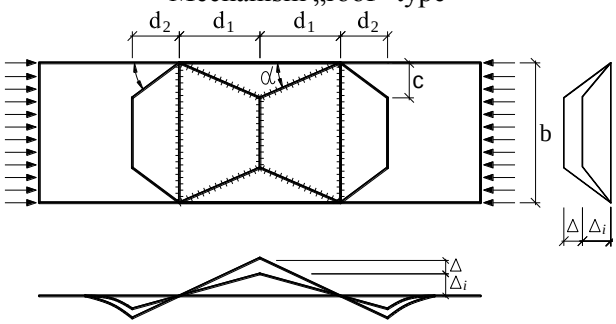
Experimental investigations carried out by numerous researchers on plate and TWCF members, prone to uniform or eccentric compression, indicated that, in such elements, one can

distinguish some simple plastic mechanisms termed by Murray & Khoo as *basic mechanisms* [2.1]. Simultaneously, experimental research performed, among others by Murray, confirmed that even a very complex plastic mechanism may be treated as an assembly of some simple basic mechanisms. Thus, Murray & Khoo proposed 8 basic mechanisms for plate elements subjected to eccentric compression and also under mixed boundary conditions and derived relations representing current load in terms of the plate deflection in the stage of failure. Those mechanisms are also described in details by Królak [2.9]. They are presented in Tables 2.1 and 2.2.

Plastic mechanisms of plates subjected to uniform compression

Contrary to the plates subjected to mixed boundary conditions, plates prone to uniform compression and under symmetrical boundary conditions can develop two basic types of plastic mechanisms, namely the flip disc mechanism described above (no. 8 in Table 2.1) and the mechanism termed in literature as “pitched roof”. The latter was described by many researchers, among others Kato [2.10], Korol & Sherbourne [2.11], as well as Mahendran [2.12]. Modifications of this mechanism have been elaborated by Rondal & Maquoi [2.13] and Kragerup [2.14]. These mechanisms were discussed in details by Ungureanu et al. [2.7]. Table 2 shows the geometry of these mechanisms together with the relations for current load in terms of deflection at the stage of failure. The relations take into account initial imperfections.

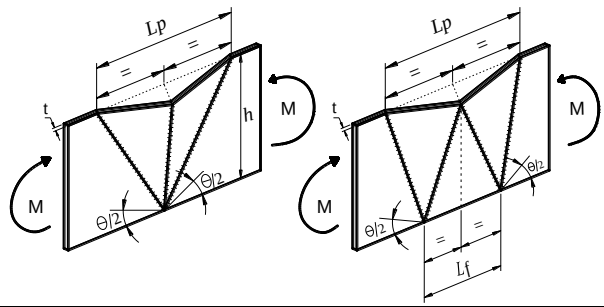
Table 2.2: Basic plastic mechanisms of plates under uniform compression subject to symmetrical boundary conditions

No.	Mechanism	Load – deflection relation
1	Mechanism „pyramid” type 	$\frac{F}{F_p}=\frac{\sigma}{\sigma_y}=\frac{1}{2}\left\{1-2\frac{\Delta+\Delta_i}{t}+\left[1+4\left(\frac{\Delta+\Delta_i}{t}\right)^2\right]^{1/2}\right\}$
2	Mechanism „pitched roof” type 	$\frac{F}{F_p}=\frac{\sigma}{\sigma_y}=1-0.355\frac{\Delta+\Delta_i}{t}+0.056\left(\frac{\Delta+\Delta_i}{t}\right)^2-$ $-0.003\left(\frac{\Delta+\Delta_i}{t}\right)^3$
3	Mechanism „roof” type 	$\frac{F}{F_p}=\frac{\sigma}{\sigma_y}=\left(1-2\frac{c}{b}\right)\left\{\left[\left(1+r^2\right)\left(\frac{\Delta+\Delta_i}{t}\right)+1\right]^{1/2}-\right.$ $\left.-(1+r)\left(\frac{\Delta+\Delta_i}{t}\right)\right\}+\frac{c}{b}\left\{\left[\frac{4\left(1+r^2\right)}{k^2}\left(\frac{\Delta+\Delta_i}{t}\right)^2+1\right]^{1/2}-\right.$ $\left.-\frac{2(1+r)}{k}\left(\frac{\Delta+\Delta_i}{t}\right)\right\}+\frac{1}{\frac{2(1+r)}{k}}\frac{1}{\frac{\Delta+\Delta_i}{t}}\cdot$ $\ln\left\{\left[\frac{4\left(1+r^2\right)}{k^2}\left(\frac{\Delta+\Delta_i}{t}\right)^2+1\right]^{1/2}-\frac{2(1+r)}{k}\left(\frac{\Delta+\Delta_i}{t}\right)\right\}$

Basic plastic mechanisms of plates subjected to in-plane bending

Basic mechanisms of plate elements subjected to in-plane bending can be treated as components of more complex plastic mechanisms in thin-walled beams or beam-columns. Experimental tests indicate that there are two main basic mechanisms in this case. They are shown in Table 2.3 as mechanisms T1 and T2. The problem of plastic mechanisms of separated plates subject in-plane bending was solved by Feldman [2.15] as wells as Park & Lee [2.16]. These mechanisms were discussed in detail by Ungureanu et al. [2.7].

Table 2.3: Basic plastic mechanisms of plates under in-plane bending

No.	Mechanisms	Load – deflection relation
1	Mechanism T1 and T2 	$\frac{M}{M_p}=\frac{1}{6}k^2\left(\frac{t}{\Delta+\Delta_i}\right)^2\left\{\left[1+\frac{4}{k^2}\left(\frac{\Delta+\Delta_i}{t}\right)^2\right]^{3/2}-1-\frac{8}{k^3}\left(\frac{\Delta+\Delta_i}{t}\right)^3\right\}$
2		Load –deflection curve (failure curve) can be obtained in numerical procedure based on the energy method [2.19]

Plastic mechanisms of TWCF steel members

Theoretical models of 3D plastic mechanisms in channel-section columns were originally elaborated by Murray & Khoo [2.1]. Among them a true mechanism (three-hinge flange mechanism), which develops due to axial compression and/or bending (flanges deflect laterally towards the free edge –Table 2.4-1) has been developed. If a channel or a lip channel is subject to in-plane bending, in the web plane, a web mechanism can develop [2.4]. In case of a lipped channel-section member the additional local three-hinge mechanisms in lips should be incorporated into the model (see Table 2.4-2). Based on these two models proposed in [2.4], Ungureanu et al. [2.7] used the plastic mechanism theory in order to determine the ultimate load for members in bending and also for interactive buckling of TWCF members.

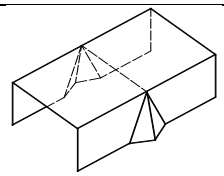
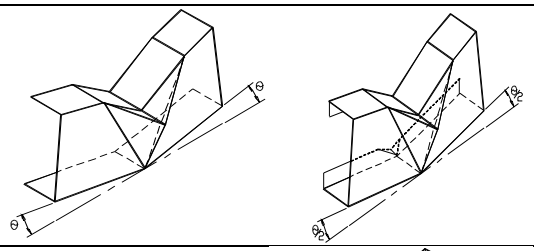
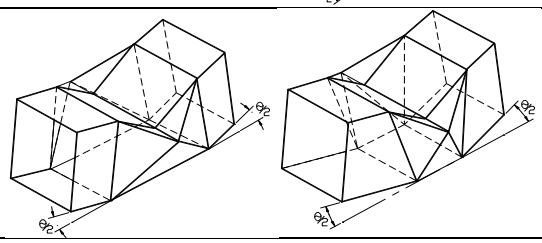
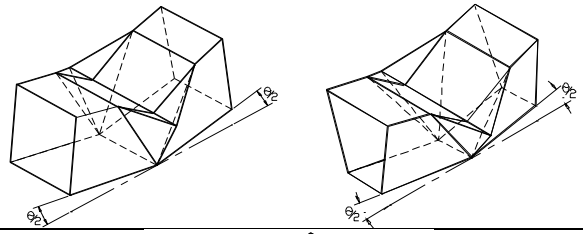
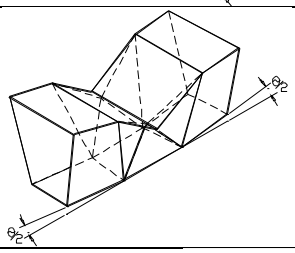
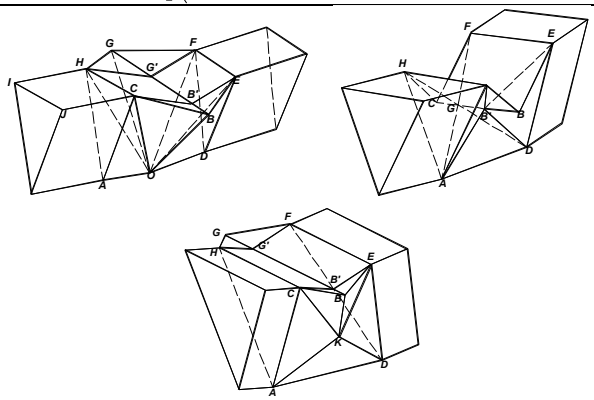
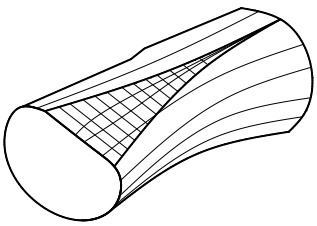
The first theoretical model of plastic mechanism in thin-walled rectangular tube subject to bending was elaborated by Kečman [2.17] on the basis of results of tests carried out on steel tubes and using a *true mechanism* model in association with the rigid-plastic theory. The Kečman's model has taken into account local rolling mechanisms and travelling hinges (see Table 2.4-3). Later, Kotełko [2.18] investigated both trapezoidal and rectangular box-section steel beams of relatively low width-to-thickness ratio (less than 80) subject to pure bending. The experimental tests indicated the plastic mechanisms developed in beams under investigation to be in fact *quasi-mechanism*, particularly in the initial phase of failure. On the basis of experimental work, theoretical models of plastic mechanisms in rectangular cross-section and trapezoidal cross-section were elaborated (see Table 2.4-4 and 2.4-5). Detailed geometrical relations of plastic mechanisms are given in [2.18].

On the basis of tests performed on non-metallic models, three different theoretical, plastic true mechanisms were elaborated by Kotełko & Królak [2.19] for beams with triangular cross-section (see Table 2.4-6).

Based on experimental work and past theoretical investigation from literature for rigid plastic mechanism of circular hollow steel tubes under pure bending, Elchalakani et al. [2.20, 2.21] have proposed two local plastic mechanisms, i.e. *star* and *diamond* shapes (Table 2.4-7).

Bambach et al. [2.22] present a number of inelastic design methods that may be used to predict the strength of slender I-sections in minor axis bending. Among them a plastic mechanism for I-sections in minor axis bending was proposed.

Table 2.4: Basic plastic mechanisms in TWCF beams

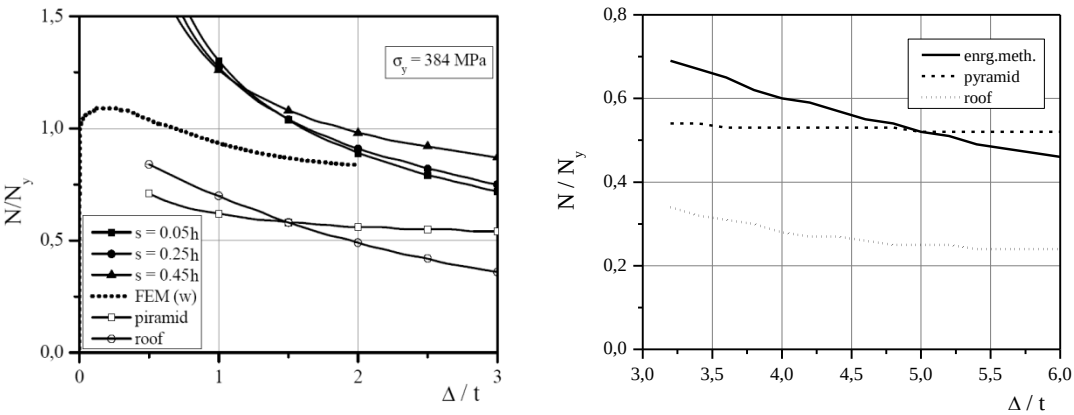
No.	Beam cross-section	Mechanism	References
1	Channel (bending in the flange plane)		Murray & Khoo [2.1] Strip equilibrium method applied
2	Channel (bending in the web plane)		Kotełko [2.4] Energy method applied
3	Rectangular or trapezoidal box-section		Kecman [2.17], Kotełko [2.18] Energy method applied
4	Rectangular or trapezoidal box-section		Kotelko [2.18] Energy method applied
5	Trapezoidal box-section		Kotelko [2.18] Energy method applied
6	Triangular cross-section		Kotełko & Królak [2.19] Energy method applied
7	Circular hollow section		Elchalakani, Grzebieta & Zhao [2.20], [2.21] Energy method applied

Energy method vs. Equilibrium strip method. Case study: rectangular steel plate [2.8,2.24,2.25]

The case study was performed for two subjects: a steel thin plate under uniform compression and steel channel section column under axial compression. The analytical results were verified using numerical analyses performed with ANSYS vs. 10.

For the plate under uniform compression, the analytical solutions based on formulae given in Table 2.2 for mechanisms type "pyramid" and "pitched roof" have been compared with the solution based on the energy method for the "pitched roof" mechanism according to [2.8]. These curves, together with FE loading path, are shown in Figure 2.1. The FE results concern the plate simply supported at all edges. Failure curves obtained by the energy method correspond to three different values of the mechanism parameter *s* (see Table 2.2-2). The failure curve for the pitched-roof mechanism obtained by the equilibrium strip method was evaluated for the mechanism parameter α= 30º. The failure curve for the "pyramid" type mechanism was also obtained using the equilibrium strip method (see Table 2.2-1).

Figure 2.1(a) shows the loading paths up to the relative deflection of magnitude 3. Figure 2.1(b) represents the comparison of analytical solutions only, for the same plate, but for large relative deflections, beyond 3. The continuous line represents the energy method solution for the "pitched roof" mechanism (*s*=0.25*h*), while the two dotted lines represent the equilibrium strip method solution for "pyramid" type and "pitched roof" mechanisms, respectively.



a) b)

Fig. 2.1: Loading paths and failure curves of rectangular steel plate of dimensions 1600×800×50 [mm]; $\Delta_i = 0$. a) $0 < \Delta/t < 3$, b) $\Delta/t > 3$ [2.8]

Figure 2.2 represents the exemplification of the upper bound approach consisting in the compilation of the FE extrapolated post-buckling path and the failure curve obtained for the "pitched roof" mechanism using the energy method. The qualitative agreement of the FE unloading path with the analytical failure curve is relatively good. The discrepancy between the upper bound ultimate load and FE maximum load amounts about 18%.

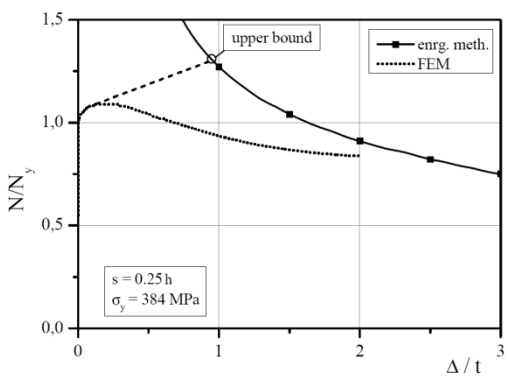


Fig. 2.2: Loading paths of rectangular steel plate of dimensions 1600×800×50 [mm] (pitched roof mechanism for energy method – *s*=0.25*h*) [2.8]

In the case of a thin plate under uniform compression the analytical equilibrium strip underestimates strongly the load carrying capacity in comparison with the energy analytical solution and also with FE results. A relatively good agreement of two analytical methods applied is obtained for large deflection only.

Load-carrying capacity estimation – comparative study and results review [2.8, 2.24, 2.25]

In this paragraph the comparative study and results review of load-carrying capacity estimation is limited to stub columns subjected to uniform compression only. The theoretical and experimental results are compared with those obtained from strength predictions given by design codes.

Kołakowski & Kotełko [2.26] present results of the upper-bound estimation (UBELC) in the case of plain channel section subjected to uniform compression. The pre- and post-buckling path is obtained using the asymptotic method, while the failure curve obtained on the basis of the energy method. The same approach has been used in this study. The subject of investigation was also a plain channel section. The UBELC estimation was performed using failure curves obtained on the basis of both energy method and equilibrium strip method. In Figure 2.3 the load-shortening path of steel channel-section is presented. The pre- and post-buckling is obtained using the asymptotic method [2.26]. Two intersection points are indicated (UBELC1 and UBELC2), representing the upper-bound estimation of the load-capacity. Figure 2.4 shows load-shortening paths (three failure curves) for the same channel-section, in the range of large deformations (large shortening), e.g. in the final stage of collapse. Two curves were derived from the energy method, for the rigid-perfectly plastic material (r-p-p) and material with strain-hardening (sh) in the plastic range, respectively. The curve indicated as "yield str" was obtained from the equilibrium strip method. The results obtained on the basis of the energy method are overestimated. However, a discrepancy of UBELC is relatively low (see Figure 2.3).

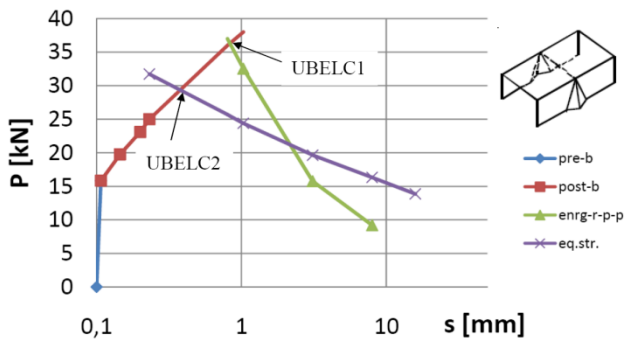


Fig. 2.3: Load-shortening path of steel plain channel section in compression.
b = 25, b_1 = 50, t = 1 [mm], σ_y = 300 MPa, σ_{ult} = 350 MPa, E = 2.10^5 MPa, E_t = 3000 MPa

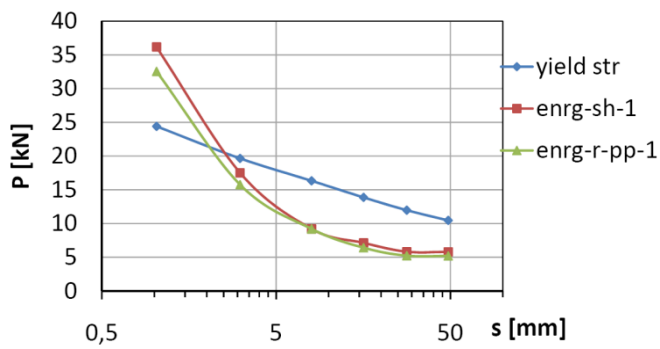


Fig. 2.4: Load-shortening path (failure curves) of steel plain channel section in compression.
b = 25, b_1 = 50, t = 1 [mm], σ_y = 300 MPa, σ_{ult} = 350 MPa, E = 2.10^5 MPa, E_t = 3000 MPa

Concluding remarks

The local rigid-plastic model describes properly the behaviour of TWCF stub columns and short beams. The main problem is to identify correctly the type of plastic mechanism to be used in analysis. In fact, when local buckling firstly appears, it always changes into a local plastic mechanism when the member fails. This is why the local plastic mechanism model naturally describes the short member behaviour, which is not the case of effective width model. This model is consistent with the real phenomenon and is confirmed by test results and advanced elastic-plastic FEM analysis.

The ultimate strength in interactive buckling of TWCF members can be regarded as an interaction between a localized plastic mode with and overall elastic one. The plastic-elastic interactive model naturally describes the phenomenon of the interactive buckling of TWCF members.

Plastic mechanism method enables to obtain bending moment-plastic rotation curves of TWCF steel sections and, on this basis, to evaluate their ductility and performance in seismic resistant structures. The ductility and rotation capacity of the studied members are limited. The numerical results sustain to classify thin-walled cold-formed steel structures as low-dissipative. This method can be also used to evaluate the energy absorption.

In the case of a thin plate under uniform compression the analytical equilibrium strip method underestimates strongly the load carrying capacity in comparison with the energy analytical solution and also with FE results. A relatively good agreement of two analytical methods applied is obtained for large deflection only.

Generally, the energy method delivers an estimation of the load carrying capacity higher than the equilibrium strip method, except at the final stage of failure (for large deflection). It coincides with Flockhart et al. [2.27] research results but indicates much more substantial discrepancies between two methods under investigation. Thus, further research into a realistic application of both analytical methods for different plated structures should be continued.

2.3 Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections [2.28]

Introduction

In practice, multi-span purlins of cold-formed Z-sections with overlaps and bolted connections over internal supports are very popular owing to their high structural efficiency, but also to their low transportation cost with effective stacking and highly productive installing.

Traditional design methodology considers the lapped bolted zone does not affect the continuity of purlin and the bending moments and shear forces are correspondingly obtained. The strength and stability checking for this zone, for bending, shear or local transverse forces, considering single or coupled effects, are performed taking a homogeneous section, of which properties are calculated as sum of the two component sections.

However, the actual behaviour of such a purlin is different because (1st), the lapped zone does not work like a homogenous section, and (2nd), the usual fastening system does not provide the continuity of purlin between single and lapped sections. Obviously, the traditional model is really optimistic, both for ULS and SLS design criteria.

In the last years, extensive studies have been conducted on the structural behaviour of lapped Z-section purlins [2.28, 2.29, 2.30]. Summarizing, their conclusions are:

- the failure of such purlins usually occurs at the edge of the lap zone by the local buckling of compression flange;
- additionally, the failure of purlins are influenced by the shear buckling of the web of single section at the edge of the lap.

Figure 2.5 [2.31] shows there is no shear buckling of the web at the failure of section, but it is the crushing of the flange and web crippling, that is caused by the coupling of two effects i.e.

- the high compression stress concentration around the bolt holes in the web;

- the locking of flange-into-flange in compression zone, which initiate the crushing of internal flange and, in the presence of compression stress concentration, induces the web crippling.

Based on these observations and considering the semi-continuous connections between single and lapped sections the ULS criterion is regarded as an interaction between bending moment and web crippling in the single section at the edge of the lap.

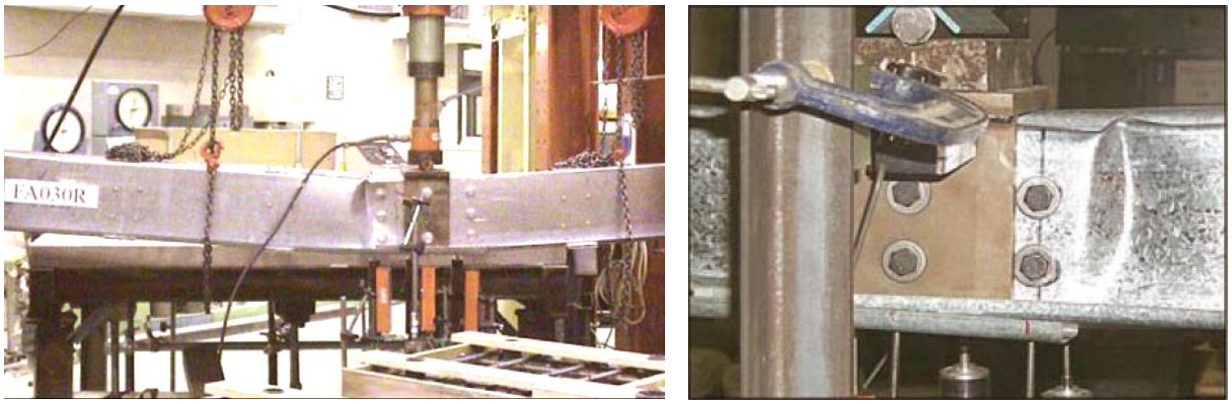


Fig. 2.5: Typical mode of failure – section failure at the end of lap [2.31]

Bending – web crippling interaction

The results of experimental tests by Ho & Chung [2.31] are used to compare the interaction between bending and shear force (BSF), with the one between bending and web crippling (BWC). The following equations have been used to find the ultimate force for the specimen of the shape shown in Figure 2.6. Details are presented in [2.28, 2.29, 2.30].

BSF	**BWC**
$M_{Ed}/M_{c,Rd} \leq 1$ (2.2)	
$V_{Ed}/V_{c,Rd} \leq 1$ (2.3)	$F_{Ed}/R_{w,Rd} \leq 1$ (2.5)
$\left(\frac{M_{Ed}}{M_{c,Rd}}\right)^2 + \left(\frac{V_{Ed}}{V_{c,Rd}}\right)^2 \leq 1$ (2.4)	$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}} \leq 1.25$ (2.6)

where:
M_{Ed} – is the design value of the bending moment;
V_{Ed} – is the design value of the shear force;
F_{Ed} – is the transverse force;
$M_{c,Rd}$ – is the design moment resistance;
$V_{c,Rd}$ – is the design shear resistance;
$R_{w,Rd}$ – is the local transverse resistance of the web.

$M_{c,Rd}$, $V_{c,Rd}$ and $R_{w,Rd}$ are calculated using the relevant formulas of EN1993-1.3 [2.32]. It has to be noted that the geometrical characteristics of the sections are taking into account the effective geometrical characteristics considering both the local buckling effect and influence of bolt holes. F_{Ed} was evaluated as the sum of bearing forces in bolt holes.

The comparison between test results and the ultimate forces corresponding to BSF and BWC interaction, shown that the difference between P_{BSF} and P_{BWC} ranging from 3% to 13% (related to P_{BSF}), but, in many cases, P_{BWC} values fit better with experiments.

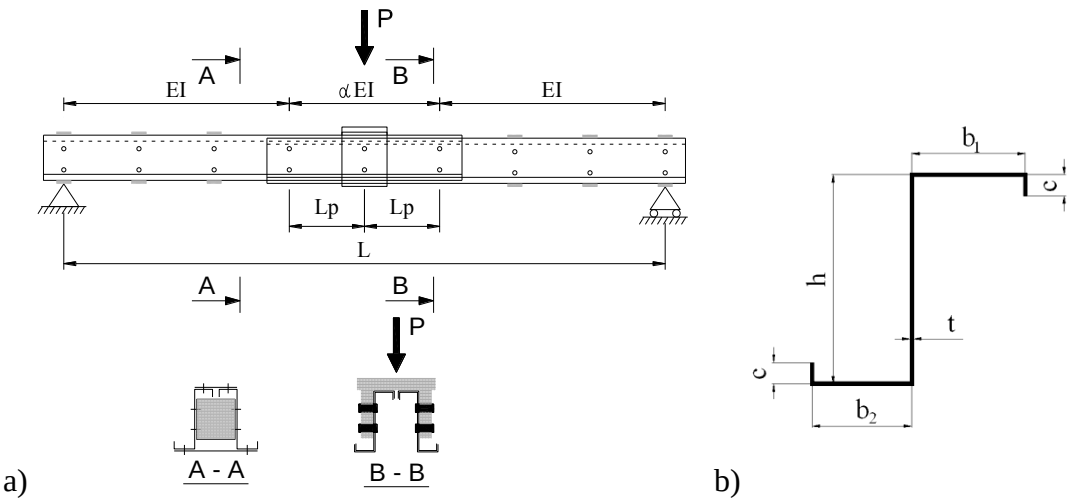


Fig. 2.6: a) General test setup of one point load test [2.31]; b) dimensions of cross-section

These tests have been used by authors to calibrate an advanced FE model. The comparison for one of the specimens is shown in Figure 2.7. With this model the behaviour of other sections than those tested by Chung and co-authors [2.31], connected each-other with two or three bolts have been simulated and the results are shown in Table 2.5. The failure mechanism involving the interaction between compression of flange due to bending and web crippling due to local transverse stress due to bearing of bolts and locking of flanges was always confirmed.

Table 2.5. Numerical vs. interactive BWC results

Test	Dimensions $h\times b_1\times b_2\times c\times t$ (mm)	L (mm)	$2L_p$ (mm)	No. of bolts/end lap	f_y (N/mm^2)	E (kN/mm^2)	P_{ANSYS} (kN)	P_{BWC} (kN)
Z150	150×47×41×16.8×1.2	1800	900	2	350	210	17.32	15.21
Z150	150×47×41×16.8×1.2	1800	1200	2	350	210	25.52	23.63
Z250	250×74×66×23.7×1.5	1800	900	3	350	210	30.21	28.15
Z250	250×74×66×23.7×1.5	1800	1200	3	350	210	34.87	33.39

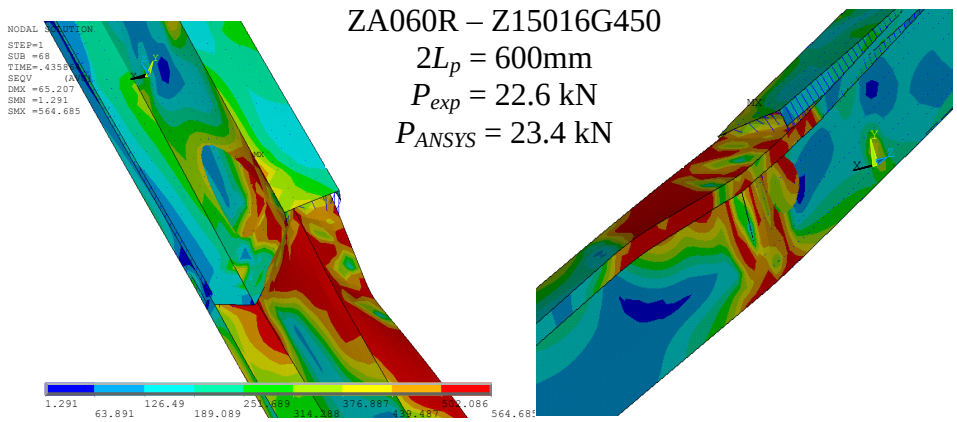


Fig. 2.7: FE model for calibration; experiment vs. numerical results

Flexural rigidity of purlin at the edge of lapped length

To discuss this matter a two span lapped Z-section purlin is considered, fixed with bolted cleats on the rafter and fastened with three bolts at the end of each lap (see Figure 2.8). The lapped connections of the type shown in Figure 2.8a are always semi-rigid and partial restraint. Figure 2.8b presents the calculation model for this situation. The source of flexural deformability are the bearing work of bolts into the web of section, the web distortion due to stress concentration around bolt holes, the flexural deformation of flanges and the shear deformation of the web.

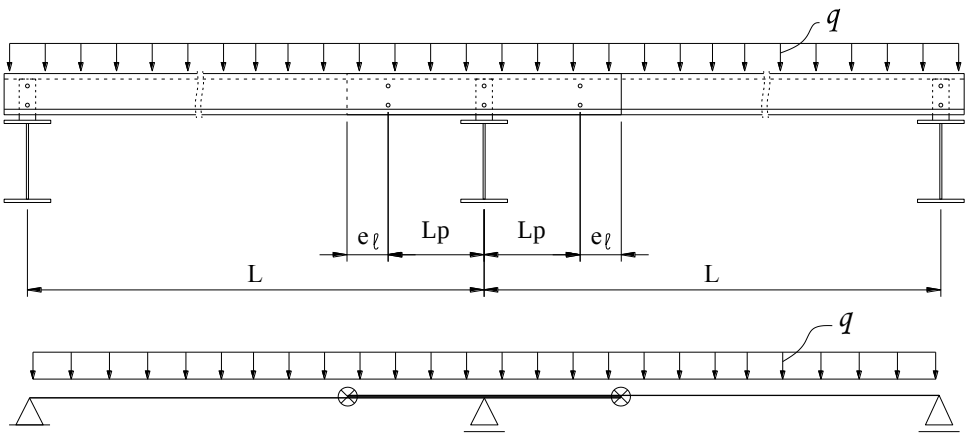


Fig. 2.8: Two span lapped Z purlin: a) real detailing; (b) semi-continuous model

Zaharia & Dubina [2.33] proposed a formula to evaluate flexural stiffness of bolted cold-formed steel sections, to be applied for truss structures. This formula, together with the other components, can be adapted for the present case if the situation of Figure 2.9 is considered.

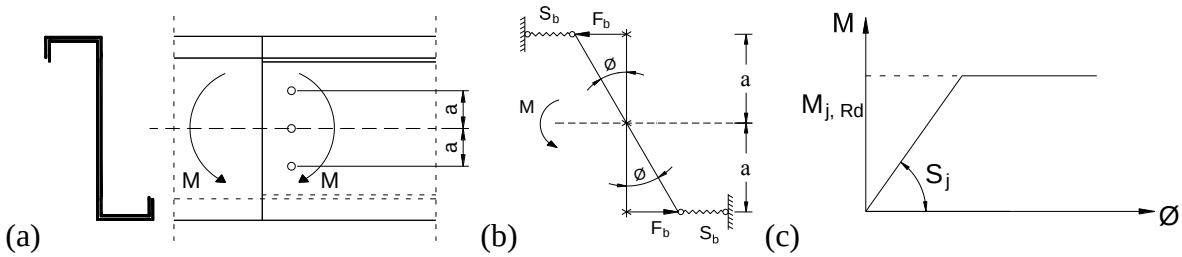


Fig. 2.9: Semi-continuous bolted lapped Z connection a) edge connection detailing; b) calculation model; c) Moment-rotation curve [2.28]

Numerical analysis

A two span semi-continuous purlin of L = 7.0m has been taken. The dimensions of purlin section have been obtained as average of the measured dimensions of the tested specimens (e.g. h = 153.5mm; b_1 = 69.95mm, b_2 = 59.75mm, c = 18.4mm, t = 1.54mm), with E = 204kN/mm^2 and f_y = 547 N/mm^2. Six different lap lengths are considered.

The results of analysis are shown in Table 2.6, where:

$q_{BSF,r}$ = is the load carrying capacity calculated for BSF interaction on the intermediate support;

$q_{BSF,l}$ = is the load carrying capacity calculated for BSF interaction at the edge of lapped length of single Z-section;

$q_{BWC,l}$ = is the load carrying capacity calculated for BWC interaction at the edge of lapped length of single Z-section;

q_{BSP} = is the load carrying capacity calculated for maximum span moment.

Table 2.6. Numerical comparison for a two span semi-continuous purlin

L (mm)	$2L_p$ (mm)	q_{BSP} (kN/m)	$q_{BSF,l}$ (kN/m)	$q_{BWC,l}$ (kN/m)	$q_{BSF,r}$ (kN/m)
7000	180	1.65	2.85	2.52	5.17
7000	300	1.91	2.00	1.88	3.53
7000	600	2.20	1.71	1.68	2.71
7000	900	2.31	1.78	1.74	2.52
7000	1400	2.41	2.07	1.95	2.39
7000	2800	2.57	4.04	3.48	2.19

Except the extreme cases of smallest and largest lapped length, in the other cases the dominant design criterion results from BWC interaction. For lap length of 1400mm, which is quite usual in practice, the difference is of 6%, which for this section cannot be neglected.

In practice, two common roof systems, consisting of purlins with top roof sheet or purlins with both top and bottom sheets are applied. In the last case, the restraining provided by sheeting avoids, in general, the lateral-torsional buckling. In the first case, lateral-torsional buckling is possible, even the restraining effect by top sheeting is acting on the almost all length of the compression flange. Therefore, to check the resistance considering the lateral-torsional buckling of a multi-span Z-purlin with bolted lapped connections, the semi-continuous model will be used. Considering the interaction between in-plane bending moment with lateral bending moment calculated at the level of free flange in compression, for the single section at the edge of the lap, the design checking for stability will be performed according to the formulas given by EN1993-1-3:

$$\frac{M_{y,Ed}}{\chi_{LT} W_{eff,y}} + \frac{M_{fz,Ed}}{W_{fz}} \leq f_{yb} / \gamma_{M1} \tag{2.7}$$

where:
$M_{y,Ed}$ = is the design value of the in-plane bending moment;
$M_{fz,Ed}$ = is the bending moment in the free flange due to the lateral load;
$W_{eff,y}$ = is the effective section modulus of the cross-section for bending about the *y-y* axis;
W_{fz} = is the gross elastic section modulus of the free flange plus the contributing part of the web for bending about the *z-z* axis.

Eqn. (2.7) applies both for gravity and uplift loads. In this report, the numerical application is limited to the first case, the gravity load. The results obtained with this interaction formula are shown in Table 2.7, expressed in terms of load carrying capacity (e.g. $q_{LT,l}$, $q_{LT,r}$). $q_{LT,l}$ and $q_{LT,r}$ are the load carrying capacity corresponding to the lateral-torsional buckling at the edge of the lap and intermediate support. The results in Table 2.7 are consistent with those in Table 2.6, e.g. except the shortest and the longest lap cases, in all others the design is given by lateral-torsional buckling strength at the edge of the lap.

Table 2.7: Numerical comparison for a two span semi-continuous purlin with top roof sheet

L (mm)	$2Lp$ (mm)	q_{BSP} (kN/m)	$q_{LT,l}$ (kN/m)	$q_{BSF,r}$ (kN/m)
7000	180	1.65	1.93	3.41
7000	300	1.91	1.44	2.48
7000	600	2.20	1.22	1.97
7000	900	2.31	1.26	1.82
7000	1400	2.41	1.44	1.72
7000	2800	2.57	2.51	1.60

Finally, it can be concluded that the lap edge single section was found to be the most critical point for design in the case of this purlin typology, and the interaction of bending moment with web crippling is the relevant criterion. In the case of laterally unrestrained purlins (or not enough restrained), lateral-torsional buckling might become the relevant design criterion; in this case the critical section is the edge of the lap, too.

2.4 Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed steel members

Introduction

The global behaviour of cold-formed steel portal frames of bolted joints were studied experimentally by Lim [2.34], Dundu & Kemp [2.35], and Kwon et al. [2.36]. All these studies provided evidence of the crucial importance of joint performance on the global response of frames, which are semi-rigid and in almost all cases partial strength [2.37].

An experimental program on ridge and eaves joints was carried out at the “Politehnica” University of Timisoara. Detailed results on joint behaviour are reported in [2.38]. Based on experimental results, a calculation procedure based on the component method of EN1993-1.8 [2.39] was adapted to cold-formed joints [2.40]. Joint stiffness and moment capacity, obtained using the component method, are used to develop a joint model for global structural analysis.

Secondly, in order to characterize the behaviour for global structural analysis, two full-scale tests on cold-formed steel pitched-roof portal frames with bolted joints were performed, with the primary objective to assess their performance under horizontal (seismic) loading [2.41] and to validate the joint model used in the global analysis. In the case of the first test (C1) only lateral loading was applied. For the second test (C2), gravity loading corresponding to seismic design situation (permanent and a 0.3 fraction of the snow load) was applied, followed by increasing lateral load up to failure. Total gravity loading amounted to 31.2kN per frame, and was applied using 30 corrugated steel sheets laid on the purlins.

Related to the experimental tests on full scale frame units, several numerical and analytical procedure to evaluate the ultimate design capacity of these frames were proposed [2.42]. Results of experimental investigations together with numerical simulations and comparison with analytical predictions are presented.

In order to assess the influence of connection stiffness and post-buckling resistance, three frame models, presented in Figure 2.10 were analysed [2.41, 2.42]. A nonlinear static analysis under increasing lateral load, performed with SAP2000 computer code, was applied to the models and the results were compared to experimental ones.

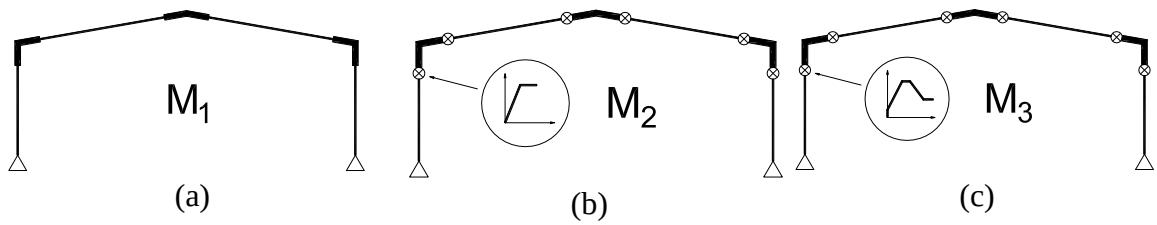


Fig. 2.10: Considered structural models: (a) rigid connections – M1, (b) elastic-perfectly plastic connections – M2 and (c) degrading connections – M3

The first model (M1, see Figure 2.10(a)) was a conventional model, where connections were considered rigid. Nominal geometrical characteristics were used to model members. Finite dimensions of brackets were taken into account. Local buckling of members was modelled by rigid-plastic hinges located at the extremities of cold-formed members. Analytically determined moment capacity (M_c=117.8kNm) was considered.

The second model (M2, see Figure 2.10(b)) was obtained from model M1 by adopting an elastic perfectly-plastic model of the joint. Initial stiffness (K_{iniC}=5224kNm/rad) and moment capacity (M_c=117.8kNm).

In the case of M3 model (see Figure 2.10(a)), the elasto-plastic model of the joint was enhanced considering the stiffening effect due to the wedging and friction between the cold-formed profiles and the bracket in the early stage of loading. Also, the softening branch in the post-elastic stage was considered. Value of the "slipping" moment M_s was estimated based on experimental results, as 15% from the connection moment capacity. Following the initial rigid

behaviour, connection model consists of an elastic response at the initial stiffness K_{iniC} (determined using component method), up to the connection moment capacity M_C.

Analytical prediction of design capacity

As analytical prediction of design capacity, three methods were proposed to estimate the ultimate resistance [2.42] at the edge of bracket-to-rafter lap (e.g. the first local buckling location) of that section under design loads, i.e.:

- the "effective width" model, $F_{eff,Rd}$, based on EN1993-1-3 procedure;
- the "plastic" model, based on the local plastic mechanism, $F_{pl,Rd}$;
- the "interactive" model, which considers the coupling effect between web crippling, at the edge of the bracket-to-member lap, and the bearing in the bolt holes in flanges, $F_{wb,Rd}$.

In Figures 2.11(a) and 2.12(a) the three lines, corresponding to the three methods are presented comparatively with tests and numerical simulations. As expected, the "interactive" model, fit better with tests and numerical analysis (M3 model).

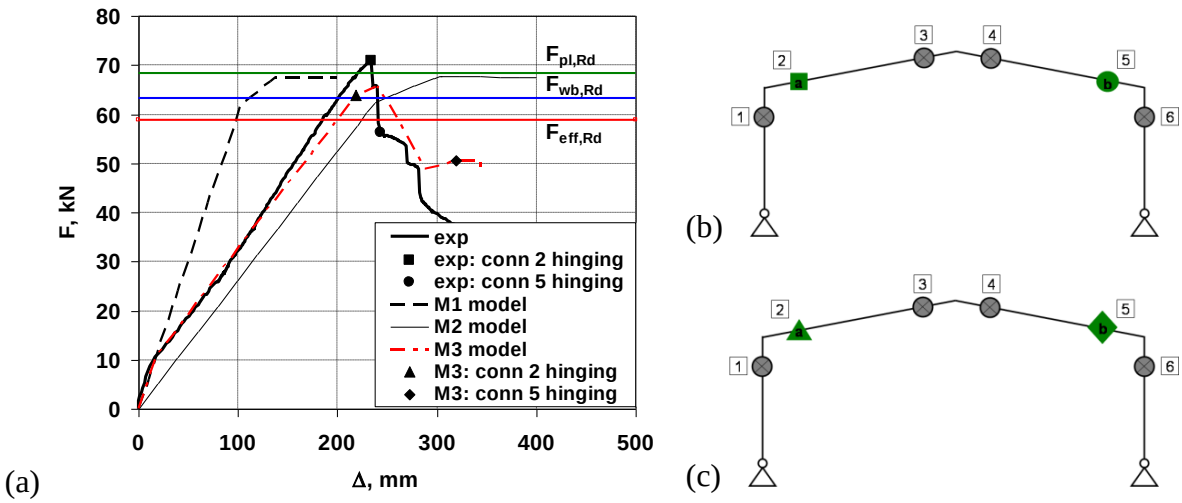


Fig. 2.11: Frame C1: experimental vs. numerical lateral force - deformation curves (a); position of local buckling observed experimentally (b) and in the numerical model (c)

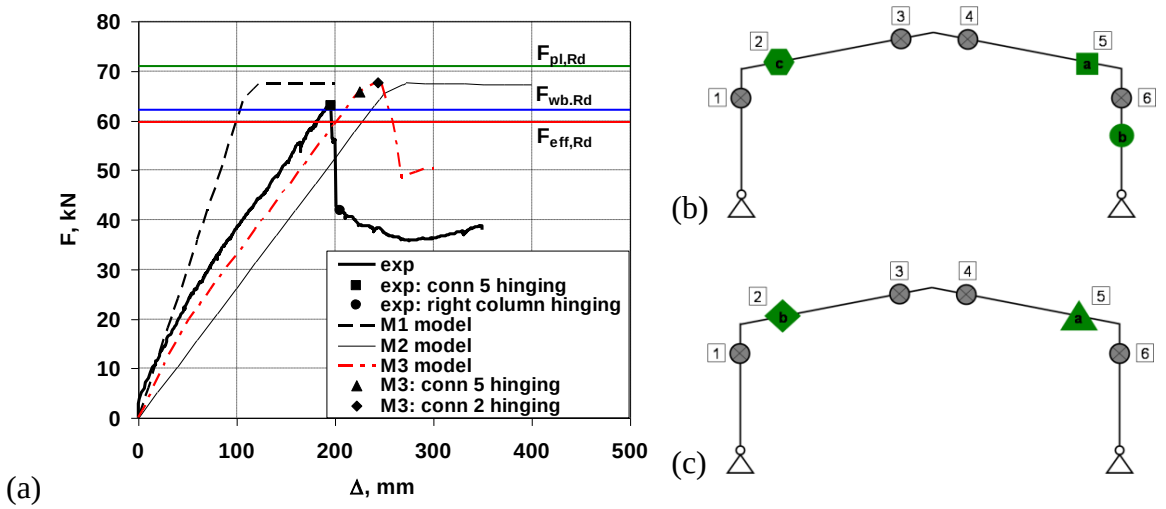


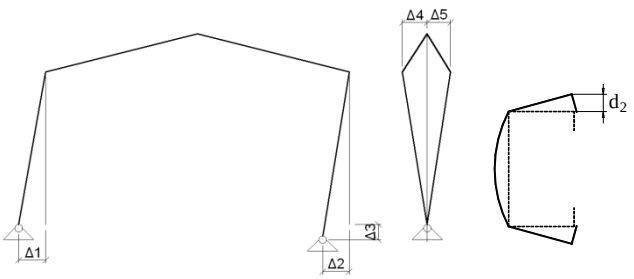
Fig. 2.12: Frame C2: comparison of experimental and numerical lateral force - deformation curves (a), and position of local buckling observed experimentally (b) and in the numerical model (c)

Imperfections sensitivity analysis of pitched roof cold – formed steel portal frames [2.43]

A GMNIA FE model was calibrated based on experimental tests [2.43]. One frame only from the set of two was considered in the analysis (i.e. a plane frame with partial lateral restraints). FE models have been prepared for each experimental test, e.g.: (1) one for the first experimental test (C1), where only lateral loading (seismic effect) was applied at left eaves up to failure and, (2) another one for the second experimental test (C2), where constant gravity loading was applied, simulating the dead and snow loads corresponding to seismic load combination, followed by the lateral load up to failure. ABAQUS/CAE v.6.8 was used for these numerical simulations.

The features of the FE model used in the study are: (1) *4-node shell element (S4R)* used to model the cold-formed members; (2) *3D solid elements (C3D4)* to model the brackets at the eaves and ridge; (3) *nonlinear spring elements* to model the effect of purlins (two springs are used to model the lateral restrains introduced by purlins of type Z150/1); (4) *contact elements* to model the gap between the back-to-back cold-formed lipped channels. The mesh size for the shell elements was around 24×24mm. For the analyses the connections were assumed to be rigid. The material properties for thin-walled cold-formed elements, determined from coupon tests, are: yield strength of 486N/mm^2, ultimate tensile strength 553N/mm^2, Young's modulus E=210000N/mm^2 and a measured thickness minus zinc coating of 2.93mm. Based on tests results, the material has been introduced by means of bilinear isotropic elastic-perfectly plastic model. Global imperfections for frames, local imperfections for individual members and imperfections at the level of cross-section have been taken into account. Details about numerical simulations are presented in [2.43].

Few imperfections were measured on site, and only for C2 tested frames; they are synthetically presented in Figure 2.13. There are no measured imperfections for frame C1 and no measured imperfections at the level of members for both tested frames. However, for numerical analyses, global and local imperfections have been considered according with: (1) tolerances proposed by EN1090-2 [2.44] or (2) equivalent imperfections for structural analyses proposed by EN1993-1-1 [2.45]. The following types of imperfections have been taken into account: a) global imperfections for frames; b) local imperfections for individual members (in- & out-of-plane imperfections); c) imperfections at the level of cross-section.



Frame C2.1	Frame C2.2
Δ_1 = 16mm	Δ_1 = 0mm
Δ_2 = 20mm	Δ_2 = 1mm
Δ_3 = 6mm	Δ_3 = 11mm
Δ_4 = 4.76mm	Δ_4 = 3.82mm
Δ_5 = 3.59mm	Δ_5 = 4.67mm

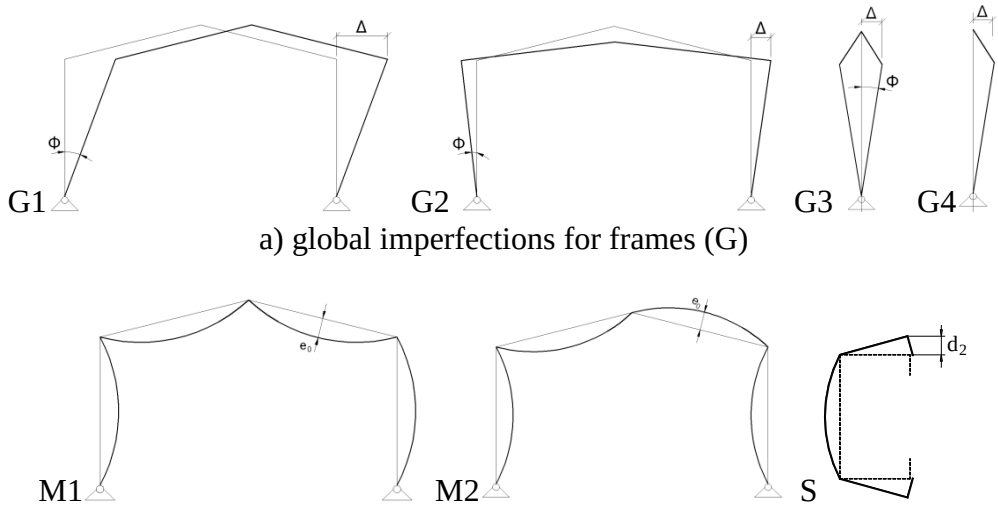
d_2 = 1.9mm

Fig. 2.13: Measured imperfections for frames C2

Based on preliminary analyses, in order to emphasize the importance of imperfections shape, only the imperfections presented in Figure 2.14 were considered. These are summarized in Table 2.8 too. It should be observed that the measured imperfections are covered by the imperfections proposed by the codes.

Table 2.8. Values of imperfections according to EN1090-2 and EN1993-1-1

	Frame	Columns in-plane	Columns out-of-plane	Cross-section
EN1090-2	Δ = h/500	±h/750	±h/750	d_2 = t
[mm]	8.2	5.5	5.5	3
EN1993-1-1	$\phi = \phi_o \cdot \alpha_h \cdot \alpha_m$	±h/250	±h/200	d_2 = t
[mm]	20.5	16.8	21	3



a) global imperfections for frames (G)

b) local imperfections (M) for individual members (in- & out-of-plane);
c) section imperfection (S)

Fig. 2.14: Types of imperfections for GMNIA FE model

Table 2.9 shows the sensitivity of frame C1, subjected only to a horizontal load applied to left eaves, to different types of imperfections (single or coupled) according with the shapes presented in Figure 2.14. The sizes of imperfections for this parametric study were taken according with EN1090-2, and are given in Table 2.8. It should be observed that the analyzed frame is not sensible to imperfections, the difference being less than 3%.

Table 2.9. Sensitivity to imperfections of frame C1

Type of imperfection	No imperf.	G1	G2	G3	G4	M1	M2	S	G1M1S in plane
F_{max} (kN)	73.932	73.364	73.366	72.859	72.169	73.179	73.192	72.611	72.634

On the following, a more complex combination of imperfections was considered in order to compare the numerical model with the experimental one, both for frame C1 and frame C2. This combination of imperfections was considered according to the types of imperfections measured on site for frame C2; it is the *G1M1S* model which combines the in- and out-of-plane imperfections, taken according with EN1090-2 and EN1993-1-1 (see Table 2.8). Figure 2.15 displays the experimental and numerical curves. A good agreement at the level of ultimate force for frame C1 can be observed.

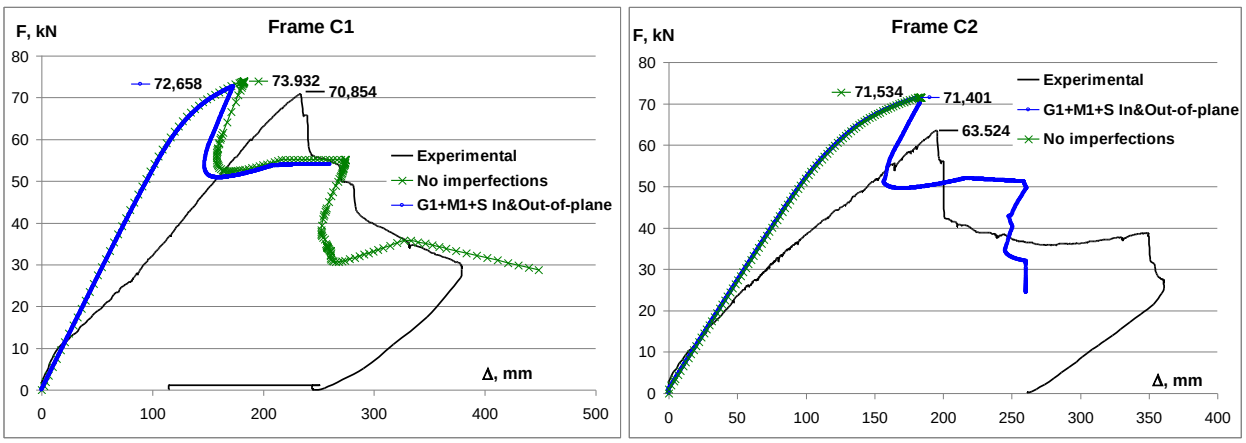


Fig. 2.15: Experimental vs. numerical G1M1S curve

For frame C2, the difference in ultimate force can be explained by the fact that in the experimental test the horizontal load has been applied with an eccentricity of 25mm, which increased the influence of compression force in right side column, and due to the fact that there were no lateral restraining at column flanges by side rails, it induced at the end a localized instability form (see Figure 2.16(b)). This imperfection was not considered in the numerical model. The difference in slope of the experimental and numerical curves is due to the fact that in the present models the eaves and ridge connections were considered fully rigid. The values of ultimate horizontal loads are summarized in Table 2.8.

(a) (b)

Fig. 2.16: (a) C1 frame: global view; (b) C2 frame: global view

Table 2.10. Frames C1 and C2: comparison of experimental and numerical ultimate forces

	Experimental F_u (kN)	EN1090-2 imperfections $F_{EN1090-2}$ (kN)	EN1090-2 imperfections $F_{EN1993-1-1}$ (kN)
Frame C1	70.85	72.658	72.629
Frame C2	63.52	71.401	71.316

Figure 2.17 shows the deformed shape of nonlinear elastic-plastic (*G1M1S*) model and local plastic mechanism formed at the edge of bracket-to-rafter lap for Frame C1, similar with the experimental one presented in Figure 2.16(a).

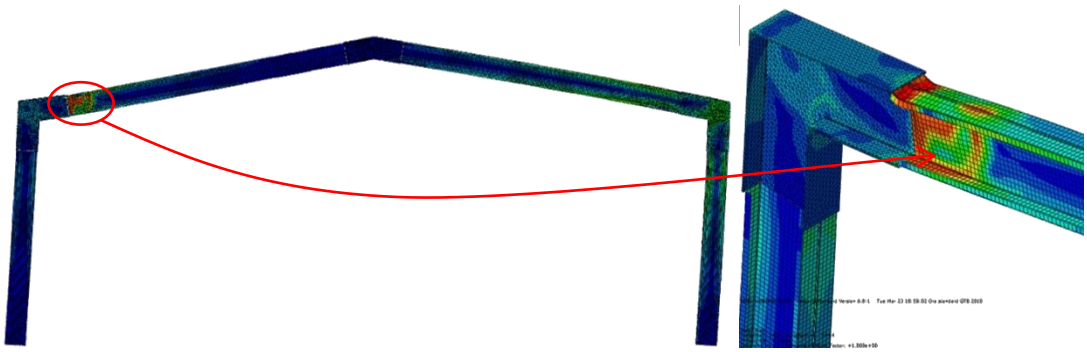


Fig. 2.15: Frame C1: nonlinear elastic-plastic model and local plastic mechanism

For the analysed frames, designed for current load conditions in Romania (heavy snow, moderate earthquake), the results prove they are less sensitive to imperfections.

LEA and in-plane GMNIA for General Method of EN1993-1-1

The General Method of EN1993-1-1 [2.45] may be used where the other methods of EN1993-1-1 do not apply. It allows the verification of the resistance to lateral and lateral-torsional buckling for structural components. Overall resistance to out-of-plane buckling for any structural component can be verified by:

$$\frac{\chi_{op}\alpha_{ult,k}}{\gamma_{M1}} \geq 1.0 \tag{2.8}$$

where: $\alpha_{ult.k}$ is the minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross-section of the structural component considering its in plane behaviour without taking lateral or lateral-torsional buckling into account however accounting for all effects due to in plane geometrical deformation and imperfections, global and local, where relevant; χ_{op} is the reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$, to take account of lateral and lateral-torsional buckling; γ_{M1} is the safety coefficient ($\gamma_{M1} = 1$).

The global non dimensional slenderness $\bar{\lambda}_{op}$ for the structural component should be determined from:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \tag{2.9}$$

where: $\alpha_{cr.op}$ is the minimum amplifier for the in plane design loads to reach the elastic critical resistance of the structural component with regards to lateral or lateral-torsional buckling without accounting for in plane flexural buckling. In determine $\alpha_{cr.op}$ and $\alpha_{ult.k}$, Finite Element analysis may be used.

Both linear eigen buckling (LEA) and in-plane nonlinear elastic-plastic (GMNIA) analyses have been applied for C1 and C2 frames, in order to validate the General Method. The imperfections taken according to EN1090-2 for in-plane GMNIA analyses are those given in Table 2.8. Table 2.11 presents the results obtained via General Method, for the safety coefficient $\gamma_{M1} = 1$, and shows that, at least for the case of these frames (stiff enough), it gives good results.

Table 2.11. Frames C1 and C2: General Method

	GMNIA in-plane $\alpha_{ult,k}$	LEA $\alpha_{cr,op}$	$\bar{\lambda}_{op}$	χ_{op}	$\chi_{op}\alpha_{ult,k} / \gamma_{M1} \geq 1$
Frame C1	1.224	3.990	0.554	0.859	1.051
Frame C2	1.191	4.148	0.536	0.868	1.033

Conclusion remarks

The connection with both flange and web bolts is semi-rigid but full-resistant. Therefore design of light-gauge portal frames of the type analysed in this chapter need to account for connection flexibility.

The critical section for usual frames is at the edge of lap between the connecting bracket and rafter. For this section the reference design resistance has to be obtained, and for that the best estimation was provided to be the one based on the interaction between web crippling effect, at the edge of the lap, and the bearing in the outer bolt holes.

Frames tested experimentally under horizontal loading only, and horizontal and gravity loadings have been studied using GMNIA FE model. For numerical simulations the imperfections were taken according with the tolerances specified EN1090-2 and the provisions of EN1993-1-1. Different types of imperfections were considered. For the analysed frames,

designed for current load conditions in Romania (heavy snow, moderate earthquake), the results prove they are less sensitive to imperfections.

In the next step, the General Method of EN1993-1-1 was applied to the analysed frames and the obtained results encourage to recommend to use this approach for such a type of structures because it is simply enough and effective.

2.5 Behaviour of cold-formed steel perforated pallet rack sections in compression

Introduction

All structures are in reality imperfect. The imperfections refer to cross-section and member geometry, to residual stresses and to yield strength distribution across the section, to supporting conditions of the members and to load introduction. Excepting the last two types of imperfections, which are of mechanical type, a lot of work has been done to analyse, classify and codify the material and geometrical imperfections [2.46-2.51].

It was observed the different nature of imperfections, associated with the slenderness of component walls, leads to different instability behaviour of cold-formed sections compared to hot-rolled ones [2.46]. As a consequence, specific buckling curves should be provided for cold-formed steel sections instead of using European buckling curves obtained for hot-rolled ones.

Due to the local and distortional instability phenomena, and their coupling with overall buckling modes, the post-critical behaviour of thin-walled cold-formed steel members is highly non-linear, being very difficult to be predicted using analytical methods. Numerical non-linear analysis can be successfully used to simulate the real behaviour of cold-formed steel sections. Initial imperfections as equivalent sine shapes, with half-wave lengths corresponding to relevant buckling modes are used as geometric non-linearity. Rasmussen & Hancock [2.52] and Schafer & Peköz [2.47] proposed numerical models, to generate automatically geometrical imperfection modes into the non-linear analysis. To define the relevant sine imperfection modes, Schafer et al. [2.53] used the probabilistic analysis in order to evaluate the frequency and magnitude of imperfections.

Related to numerical models and methods applied in the simulation, two general reports, presented in two editions of *Coupled Instability in Metal Structures* conferences, CIMS 1996 and CIMS 2000, by Rasmussen [2.54] and Sridharan [2.55], reviewed the main contributions and milestones in the progress at the date. They concluded the most used computational models are the ones applying the semi-analytical and spline finite strip and the finite element methods. At CIMS 2008, summarizing the advances and developments of computational modelling of cold-formed steel elements, Schafer [2.56] emphasized that the primarily focus is the use of semi-analytical finite strip method, considering the implementation of the constrained finite strip method (cFSM) [2.57]. This method allows for discrete separation of local, distortional and global deformations, and collapse modelling using shell finite elements.

A good alternative to that is the application of modal decomposition via Generalised Beam Theory (GBT), method which achieved a significant development in the last decade by works of the Lisbon team led by Camotim [2.58], which makes possible to select the deformation modes to be considered in the analysis.

Camotim et al. [2.59] summarise the main concepts and procedures involved in performing a GBT buckling analysis together with the development and numerical implementation of a GBT-based beam finite element formulation, which includes local, distortional and global deformation modes and can handle general loadings. Camotim & Dinis [2.60] have performed extended numerical studies, using FEM and GBT, to study the elastic post-buckling behaviour of cold-formed steel columns affected by mode interaction phenomena involving distortional buckling, namely local/distortional, distortional/overall (flexural-torsional) and local/distortional/overall mode interaction and also sensitivity to imperfections of thin-walled cold-formed steel members.

Loughlan et al. [2.61] analysed the behaviour of lipped channel profiles in compression considering the local-distortional interaction, including material yielding and yield propagation to ultimate conditions and then to elastic-plastic unloading. The effects of geometric imperfections were also considered in the numerical simulations.

Based on numerical simulations Dubina & Ungureanu [2.50, 2.51] have systematically studied the influence of size and shape of sectional geometrical imperfections and the erosion of theoretical buckling strength on the behaviour of cold-formed steel plain and lipped channel sections., both in compression and bending.

However, despite this numerical progress and, even if there is an important number of existing investigations devoted to the effect of holes on cold-formed steel members, there is not yet an analytical design procedure for pallet rack columns to be accepted by the professional community. In what concerns the possibility to apply numerical methods used, at this moment GBT and FSM cannot model members with perforated walls, except if using an equivalent thickness; in such circumstances FEM remains the only approach available to model perforated walls, but with the price of a costly work.

Casafont et al. [2.62] present an investigation on the use of the Finite Strip Method to calculate elastic buckling loads of perforated cold-formed storage rack columns. Due to the fact that holes cannot be directly modelled with FSM, the concept of the reduced thickness of the perforated strip was applied to take into account their effect. A formulation was presented for the reduced thickness that has been calibrated with loads obtained in eigen buckling FEM analyses. Bonada et al. [2.63] presented three numerical methodologies to predict the compression load carrying capacity of cold-formed steel rack section without perforations. The three methodologies allow for different imperfection shapes. The first one uses the critical mode shape (the first buckling mode). The second corresponds to an iterative methodology in which the shape that leads to the lowest ultimate load is used. These two first methodologies use exclusively the finite element method (FEM). The third one combines the finite element analysis with the generalised beam theory (GBT) in order to determine the modal participation of the FEM buckling mode and generate a particular combined geometric imperfection.

Besides stability problems, the material changes due to cold-forming influences the ultimate capacity of pallet rack upright sections. Armani et al. [2.64] investigated, by numerical simulations, the effects of local changes of the material properties due to the strain-hardening associated with cold-forming and the role of the initial geometrical imperfections when the uprights are subject to axial load.

Present paper presents the numerical approach for the study of buckling modes interaction (distortional and overall) for pallet rack members in compression. A numerical imperfection sensitivity study was conducted in order to determine the maximum erosion of critical bifurcation load due to mode coupling, imperfections and perforations. Using the ECBL approach [2.65] the maximum value of erosion was computed and based on its value, a corresponding α imperfection factor, in order to adapt the actual European buckling curves for cold-formed pallet rack sections.

Experimental program

An intensive experimental study on pallet rack uprights in compression has been carried out at the “Politehnica” University of Timisoara. The experimental program was extensively presented by the authors in [2.66].

Both perforated and unperforated section specimens have been tested, of calibrated lengths for: stub columns (s) [2.68]; upright member specimens for distortional buckling (u) [2.68]; specimens of lengths equal with the half-wave length for distortional buckling (d); specimens of lengths corresponding to interactive buckling range (c). Two cross-sections of the same typology but different sizes, RS125×3.2 and RS95×2.6, have been considered, of perforated-to-brut cross-section ratios (A_N/A_B) of 0.806 and 0.760, respectively. Their brut and perforated (i.e. net)

sections are shown in Figure 2.16 together with the perforations details. The pitch is 50mm for both studied sections.

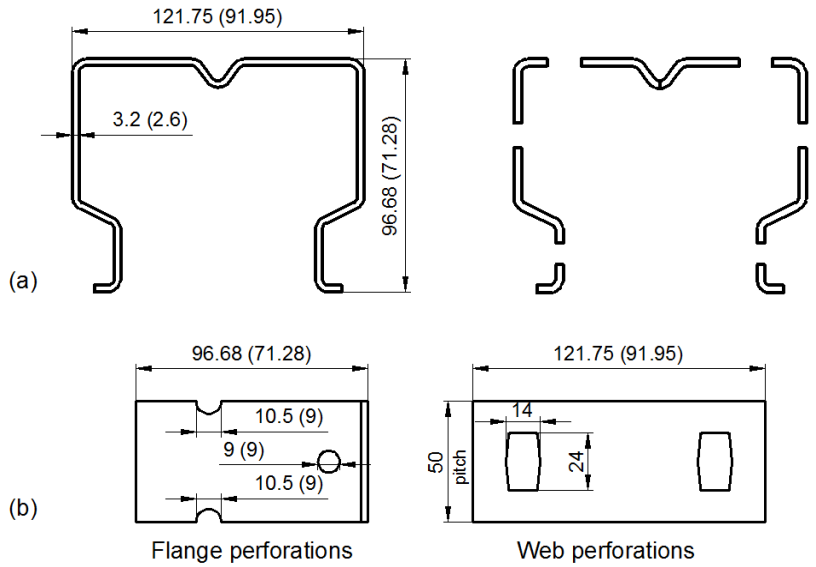


Fig. 2.16: (a) Brut and perforated specimen cross-section; (b) perforation details

The test setup was the same for all tested specimens. The test setup for stub column test is presented in detail in Figure 2.17. The ball bearing was positioned on the symmetry axis of the cross-section in between the position of gross and the minimum cross-section centres of gravity. Additional restraints were foreseen for specimens of lengths corresponding to interactive buckling range (c) in order to restrain the torsion.

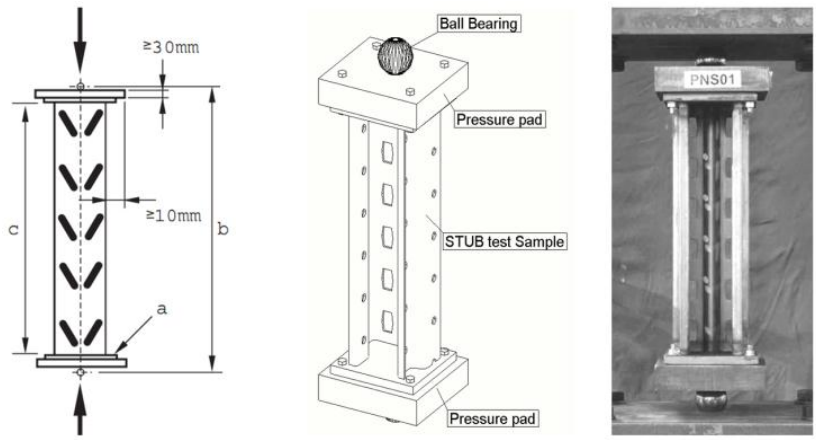


Fig. 2.17: Stub column test setup

Table 2.12 presents the failure modes for each type of the tested specimen/section. The following notations were used: S – Squash, DS – symmetrical distortional buckling, FT – flexural-torsional buckling, F – flexural buckling.

Table 2.12. Failure modes for tested sections

Section / Test type	RS95×2.6 brut	RS95×2.6 perforated	RS125×3.2 brut	RS125×3.2 perforated
Stub (s)	S	S/DS	DS	DS
Distortional (d)	DS	DS	DS	DS
Upright (u)	F or FT	F or FT	DS	DS
Interactive buckling (c)	DS+F or DS+FT	DS+F or DS+FT	DS+F or DS+FT	DS+F or DS+FT

Additional experimental tests have been done in order to determine the mechanical properties of the material. A set of samples were tested from the base material. Due to cold-forming process of the cross-section, the material properties are modified. New series of tests on coupons cut over the cross-section of specimens without perforations was done for both types of sections, in order to determine the increase of yield strength, ultimate tensile strength and residual stresses [2.66]. Figure 2.18 shows, as an example, the measured values of yield strength and residual stresses distributions for RS125×3.2 brut cross-section, as percent of yield strength of base material.

In what concerns the geometric imperfections, all tested specimens were measured. Two types of imperfections were recorded, i.e. (a) sectional and (b) global [2.66]. The sectional geometric imperfections range for RS125×3.2 cross-section, between –3.10mm … +1.64mm, while for RS95×2.6 cross-section between –2.93mm … +2.74mm. Similar values for this type of imperfection were mentioned by Schafer & Peköz [2.47] in their studies. The global imperfections, represented by the mid span deflections, on both *y* and *z* direction, were obtained matching the measured deflection with a half-wave sine equivalent (see Figure 2.21). The maximum recorded values of global imperfections in *z* direction were *L*/1416 for RS95×2.6 cross-section and *L*/1651 for RS125×3.2 one, while in *y* direction the maximum values were found to be less than *L*/3500 for both sections, with and without perforations. The measured global imperfection (overall sinusoidal imperfections) are significantly lower than the less conservative value, of *L*/1000, proposed by ECCS Recommendation [2.69] and considered for European buckling curves. On the other hand, the corresponding tolerance accepted by EN1090-2 [2.44] is *L*/750.

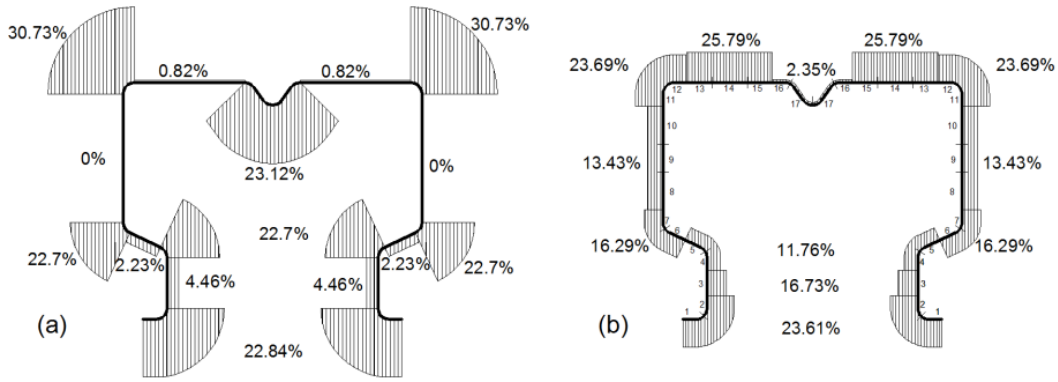


Fig. 2.18: (a) Yield strength distribution (%f_y); (b) Residual stress distribution (%f_y) represented on the compressed side of the strip (RS125 brut cross-section)

Numerical model calibration and validation

Numerical models applied to simulate the behaviour of studied sections, have been created using the commercial FE program ABAQUS/CAE. The numerical models were calibrated to replicate the physical experimental tests. Rectangular 4-noded shell elements with reduced integration (S4R) were used to model the thin-walled cold-formed members. In order to create a reliable mesh and to account the holes present along the specimen's length a mesh size of about 5×5mm was chosen. In the calibration process it was found that the influence of residual stress is small (less than 3%) and their effects will be ignored further in the analysis [2.67].

The base plates and pressure pads were modelled using RIGID BODY with PINNED nodes constraints. The reference point for the constraints was considered the centre of the ball bearings (55 mm outside the profile), in the gravity centre of the cross-section. For numerical simulations, the specimens were considered pinned at one end and simply supported at the other one. For the pinned end, all three translations together with the rotation along the longitudinal axis of the profile were restrained, while the rotations about maximum and minimum inertia axes

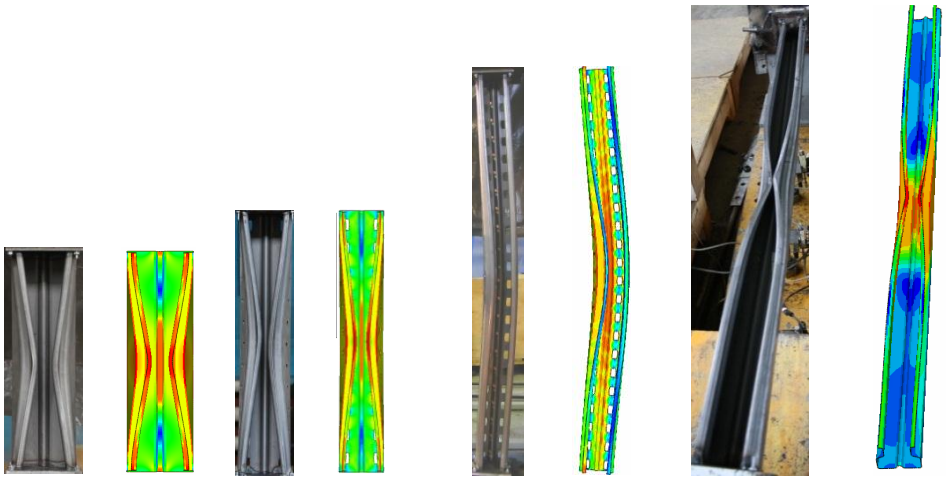
were free. For the simply supported end, the translations along section axis and the rotation about longitudinal profiles axis were restrained, while the rotations about major and minor inertia axis together with longitudinal translation were allowed. For the tested specimens the rotation about longitudinal axis was prevented by friction, while in numerical model the rotation was restrained, to remove rigid body displacements.

The analysis was conducted into two steps. The first step consists into an eigen buckling analysis (LBA), in order to find a buckling mode or combination of buckling modes, affine with the relevant measured imperfections. After, imposing the initial geometric imperfection, obtained as a linear combination of eigen buckling modes from the previous step, a GMNIA analysis with arc-length (static, Riks) solver was used to determine the ultimate capacity of pallet rack members in compression. A unit displacement was applied at the simply supported end, incremented during the analysis, in order to simulate a displacement controlled experimental test.

It must be underlined that for all considered numerical models, the failure modes were in accordance with the failure modes observed in experimental tests (see Figure 2.19). The calibrated numerical model was validated against experimental tests for all tested sets of profiles. Table 2.13 presents the values of ultimate load from numerical simulations and the experimental ones for all types of members ((s), (u), (d), (c)), for both RS125×3.2 and RS95×2.6 cross-sections, with and without perforations.

Table 2.13. Ultimate load [kN] – Experimental vs. FEM

RSBs125×3.2		RSNs125×3.2		RSBs95×2.6		RSNs95×2.6	
EXP	FEM	EXP	FEM	EXP	FEM	EXP	FEM
487.05	486.13	411.02	422.98	338.88	335.15	274.33	272.01
RSBd125×3.2		RSNd125×3.2		RSBd95×2.6		RSNd95×2.6	
EXP	FEM	EXP	FEM	EXP	FEM	EXP	FEM
440.79	440.78	394.62	397.04	325.10	331.05	262.67	255.47
RSBu125×3.2		RSNu125×3.2		RSBu95×2.6		RSNu95×2.6	
EXP	FEM	EXP	FEM	EXP	FEM	EXP	FEM
386.72	384.40	347.26	344.00	279.65	285.96	223.33	231.89
RSBc125×3.2		RSBc125×3.2		RSBc95×2.6		RSBc95×2.6	
EXP	FEM	EXP	FEM	EXP	FEM	EXP	FEM
317.89	316.67	293.62	292.9	220.29	220.26	168.88	177.11
(s) Stub columns; (d) Specimens of lengths equal with the half-wave length of distortional buckling; (u) Upright member specimens; (c) Specimens of lengths corresponding to interactive buckling range. N/B – perforated/brut							



RSBs 125×3.2 RSNd 125×3.2 RSNu 95×2.6 RSBc95×2.6

Fig. 2.19: Failure modes – Experimental vs. FE models

Based on the results obtained from numerical simulations, it can be noted that from the point of view of maximum load, the numerical model is able to accurately replicate the experimental tests. For specimens with increased length, where global and sectional imperfections are of same importance, a more complex imperfections measurement is recommended. The measurements should allow the decomposition of geometric imperfections into sectional and global components that can afterwards be used to reconstruct the initial deformed shape.

Imperfection sensitivity analysis. Determination of coupling point using ECBL approach

The interactive buckling approach based on ECBL method was largely presented in [2.65]. The principle of this method is summarized here only. Assuming the two theoretical simple instability modes that couple, in a thin-walled compression member, are the Euler bar instability mode, $\overline{N}_E = 1/\overline{\lambda}^2$ ($\overline{\lambda}$ = relative member slenderness) and the distortional instability mode described by means of the reducing factor of area $\overline{N}_D$. The resulting eroded curve for coupled instability mode is $\overline{N}(\overline{\lambda}, \overline{N}_D, \psi)$ (see Figure 2.20).

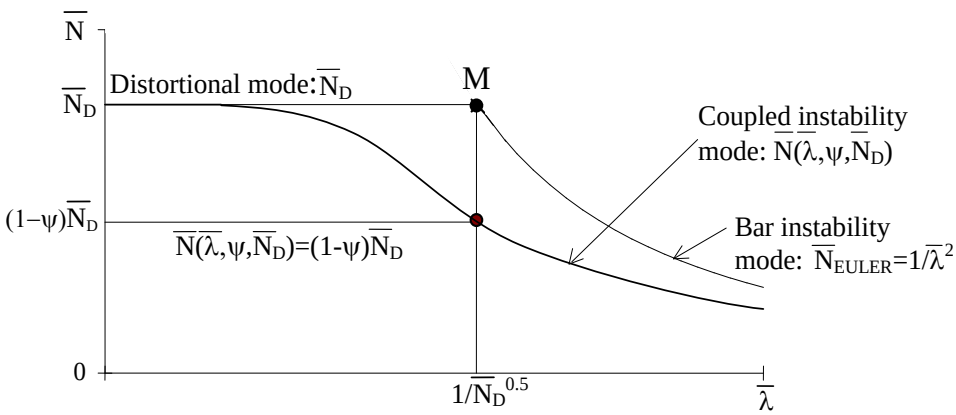


Fig. 2.20: The interactive buckling model based on the ECBL theory

The maximum erosion of critical load, due both to the imperfections and coupling effect, occurs in the interaction point, M ($\overline{\lambda} = \sqrt{1/\overline{N}_D}$) where, the erosion coefficient ψ is defined as:

$$\psi = \overline{N}_D - \overline{N} \tag{2.9}$$

in which $\overline{N}(\overline{\lambda}, \overline{N}_D, \psi)$ is the relative interactive buckling load and $\overline{N}_D = N_D / f_y A$; A = is the cross-section area; N_D = is the ultimate capacity corresponding to distortional buckling; $\overline{N} = N / f_y A$ is the relative axial load; N = is the axial load.

If $\overline{\lambda} = \sqrt{1/\overline{N}_D}$ is introduced, it results an imperfection factor corresponding to distortional-global buckling:

$$\alpha = \frac{\psi^2}{1-\psi} \cdot \frac{\sqrt{\overline{N}_D}}{1-0.2\sqrt{\overline{N}_D}} \tag{2.10}$$

Equation (2.10) represents the new formula of α imperfection factor which should be introduced in European buckling curves in order to adapt these curves to distortional-overall interactive buckling.

The coupling point between distortional (D) and global (F) buckling modes is determined following the ECBL approach as shown in Figure 2.19. On this purpose, FE analyses were performed to simulate the influence of different types of imperfections in the coupling point. Because the interest is to observe the erosion of critical bifurcation load, the ECBL approach is applied considering the distortional critical load, obtained for the relevant section by an eigen buckling analysis (Linear Buckling Analysis (LBA) using ABAQUS), in interaction with Euler buckling of the corresponding bar member. Table 2.14 shows the reference values for critical and ultimate sectional loads obtained numerically and experimentally for the studied sections.

Table 2.14. Sectional capacity and distortional buckling load

Section	RSN125×3.2	RSN95×2.6
Length [mm]	600	500
Distortional buckling load* ($N_{cr,D}$) [kN]	370.48	340.78
Distortional ultimate load** ($N_{D,u}$) [kN]	388.35	---
Stub ultimate load*** ($N_{S,u}$) [kN]	407.79	279.27
Squash load**** (N_{pl}) [kN]	480.94	286.72
* distortional buckling load determined using LBA; ** experimental failure load corresponding to "distortional" specimens – mean values; *** experimental failure load corresponding cu stub column specimens – mean values; **** $N_{pl}=A\cdot f_y$		

Table 2.15 presents the lengths corresponding to the theoretical interactive buckling loads (e.g. in the point of $\overline{\lambda}=\sqrt{1/\overline{N}_D}$, $\overline{N}_D=\overline{N}_{cr,D}$) determined via the ECBL approach, in the interactive buckling point, M, for each section.

Table 2.15. Lengths corresponding to the theoretical interactive buckling

Profile	$N_{cr,D}$ [kN]	N_{pl} [kN]	$\overline{N}_D$	Coupling length [mm]
RSN125	370.48	480.94	0.770	2559
RSN95	340.78	286.72	1.000	1667

It can be observed that for RS95N cross-sections, the critical load corresponding to distortional buckling is greater than the cross-section squash load. In this case the $\overline{N}_D$ value has to be limited to 1.00. Based on this limitation, for RS95 section, with and without perforation, there is no classical interactive buckling, but we could speak about a local plastic – elastic buckling interaction.

Imperfection sensitivity study [2.72, 2.73]

On the following, the study focuses on the sensitivity to imperfections of pallet rack sections in compression, having the member length equal to the interactive buckling length, established according to ECBL approach, presented in the previous subchapter.

An imperfection sensitivity analysis was conducted in order to identify the most critical imperfection or combination of imperfections. Figure 2.21 shows the types of geometrical imperfections considered in the analysis, i.e. distortional (d ±), flexural about the minor axis (f ±), and coupling of these two (f ± d ±). Also, load eccentricities, located on the axis of symmetry, were taken into consideration, with different amplitudes. In case of flexural-torsional buckling (FT), both initial deflection and initial twisting imperfection (ft) were considered together, according to Australian Standard AS4100 [2.71].

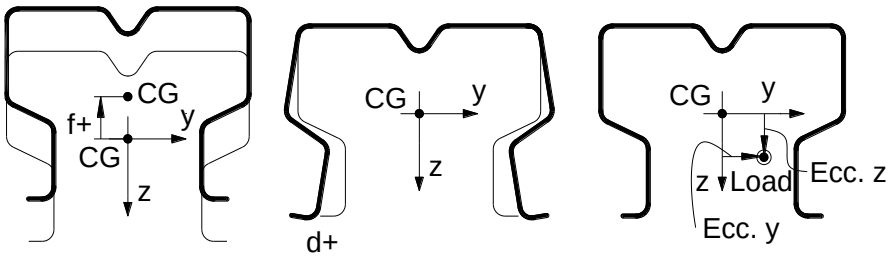


Fig. 2.21: Example of considered simple imperfections (f and d)

Detailing, the imperfections used for this study are: distortional symmetric imperfection (ds), distortional asymmetric imperfection (da) (only for RSN125×3.2 section), flexural bow imperfection about the minor inertia axis (f), loading eccentricities on both axes (independent and coupled, i.e. EY, EZ, EY-EZ) and flexural-torsional imperfection (ft). The distortional imperfection, symmetric and asymmetric, was scaled to 0.5*t*, 1.0*t* and 1.5*t*, the flexural bow imperfection was scaled to *L*/750, *L*/1000 and *L*/1500, while the flexural-torsional imperfection was considered in accordance with the provisions of Australian Standard [2.71]. The loading eccentricities were varied on both sectional axes (±2mm; ±4mm; ±6mm), independently and together, as an oblique eccentricity.

As observed in Table 2.15, for RS95×2.6 section there is no classical buckling mode interaction. Further, the present imperfection study will be focused on RSN125×3.2 only. Table 2.16 presents the considered simple imperfections, sectional, global and loading eccentricities for RSN125×3.2 section together with ψ erosion coefficient and α imperfection factors for simple imperfections [2.72, 2.73].

Table 2.16. ψ erosion coefficients and α imperfection factors for simple imperfections

Imperfection	RSN125×3.2			Imperfection	RSN125×3.2	
	ψ	α			ψ	α
ds – 0.5 *t*	0.236	0.078		EZ -6	0.313	0.152
ds – 1.0 *t*	0.339	0.185		EZ -4	0.272	0.108
ds – 1.5 *t*	0.398	0.280		EZ -2	0.210	0.059
da – 0.5 *t*	0.152	0.029		EZ +2	0.216	0.063
da – 1.0 *t*	0.245	0.085		EZ +4	0.255	0.093
da – 1.5 *t*	0.321	0.162		EZ +6	0.285	0.121
f – *L*/750	0.240	0.081		EY-EZ 0	0.157	0.031
f – *L*/1000	0.216	0.063		EY-EZ +6	0.321	0.162
f – *L*/1500	0.181	0.043		EY-EZ +4	0.276	0.112
ft	0.240	0.081		EY-EZ +2	0.215	0.063
EY +2	0.169	0.037		EY-EZ -2	0.223	0.068
EY +4	0.196	0.051		EY-EZ -4	0.270	0.106
EY +6	0.224	0.069		EY-EZ -6	0.307	0.145

In Table 2.16 can be easily observed that, for simple imperfections, symmetric distortion imperfection and major axis eccentricities give higher values for erosion coefficient than those corresponding to flexural and flexural-torsional imperfections.

Table 2.17 presents the coupled imperfections considered for the RSN125×3.2 section, i.e. f – *L*/750, ds – 0.5*t*; f – *L*/750, ds – 1.5*t*; f – *L*/1500, ds – 0.5*t* and f – *L*/1500, ds – 1.5*t*, combinations coupled with various types of eccentricities [2.72, 2.73]. It is easy to observe that the combination (f – *L*/750, ds – 1.5*t*) of imperfections is the most critical one. However, statistically is not recommended to combine all imperfections to cumulate their negative effects, because their random compensation.

Table 2.17. ψ erosion coefficients and α imperfection factors for coupled imperfections

Imperfection	ψ	α	ψ	α	ψ	α	ψ	α
	f – L/750, ds – 0.5t		f – L/750, ds – 1.5t		f – L/1500, ds – 0.5t		f – L/1500, ds – 1.5t	
EY 2	0.339	0.185	0.440	0.368	0.302	0.139	0.422	0.328
EY 4	0.342	0.189	0.442	0.373	0.305	0.142	0.423	0.330
EY 6	0.346	0.195	0.443	0.375	0.310	0.148	0.425	0.334
EZ 6	0.425	0.334	0.493	0.510	0.411	0.305	0.483	0.480
EZ 4	0.404	0.292	0.479	0.469	0.384	0.255	0.467	0.436
EZ 2	0.376	0.241	0.461	0.420	0.350	0.201	0.447	0.385
EZ -2	0.279	0.115	0.413	0.309	0.174	0.039	0.387	0.260
EZ -4	0.194	0.050	0.374	0.238	0.228	0.072	0.326	0.168
EZ -6	0.240	0.081	0.276	0.112	0.264	0.101	0.261	0.098
EY-EZ 0	0.240	0.081	0.440	0.368	0.301	0.138	0.421	0.326
EY-EZ 6	0.430	0.345	0.495	0.517	0.414	0.311	0.485	0.486
EY-EZ 4	0.406	0.295	0.480	0.472	0.386	0.258	0.467	0.436
EY-EZ 2	0.377	0.243	0.462	0.422	0.351	0.202	0.447	0.385
EY-EZ -2	0.280	0.116	0.413	0.309	0.182	0.043	0.387	0.260
EY-EZ -4	0.218	0.065	0.376	0.241	0.247	0.086	0.330	0.173
EY-EZ -6	0.271	0.107	0.298	0.135	0.289	0.125	0.285	0.121

A precise framing for coupled instabilities is very important in order to choose a suitable design strategy. For weak and moderate interaction class, simple design methods based on safety coefficients can be used. In case of strong and very strong interaction, special design methods must be developed [2.65].

It can be observed that for the case of RSN125×3.2 pallet rack section, the computed erosion can classify the section into medium up to very strong interaction, depending on the considered imperfection.

Conclusions

Both experimental tests and numerical simulations have proven the negative influence of both interaction between distortional and overall buckling and geometrical imperfections on the ultimate capacity of perforated pallet rack sections in compression in the interactive range, especially for the case of sections analysed in this paper.

The ECBL approach is an excellent method that allows for the evaluation of ψ erosion coefficients and α imperfection factors, as result of interactive buckling. It applies for the interaction of sectional (local or distortional buckling) with global (flexural or flexural-torsional) instability modes, using a limited number of experimental tests.

In order to reduce the number of experimental tests, a rational sensitivity analysis done using calibrated and validated numerical models can be used in order to determine the most detrimental imperfections to be considered for the numerical modelling. Moreover, using correctly calibrated numerical models, ECBL is a perfect method to perform a sensitivity analysis and to obtain the maximum erosion coefficient and corresponding imperfection factor for a given section, with or without perforations.

Performing a sensitivity analysis for RSN125×3.2 cross-section, it is easy to observe that for uncoupled imperfections, symmetric distortion imperfection and major axis eccentricities give higher values for erosion coefficient than those corresponding to flexural and flexural-torsional imperfections. For the case of coupled imperfections, it is easy to observe that (f – L/750, ds – 1.5t) combination of imperfections is the most critical one. However, statistically is not recommended to combine all imperfections to cumulate their negative effects, because their random compensation.

In conclusion, related to the imperfection scenarios to be adopted in numerical simulations, it is compulsory to be estimated by means of reliability analysis, in order to get results for a given failure probability. On this purpose, future research should be done in order to find values for reliability index that could be associated with the erosion classes of mode interaction.

2.6 Innovative structural systems

Introduction

Cold-formed steel sections are used more and more as structural members. Compared with hot-rolled ones, there are important technical advantages of these sections, but there are also some structural problems, mainly due to their slenderness, which impose limits of using such sections in heavy buildings and, particularly, in buildings located in seismic zones.

As a fruit of co-operation between LINDAB Constructions Ltd. Company of Romania, BRITT Ltd. Consulting and Design Company, Department of Steel Structures and Structural Mechanics of the "Politehnica" University of Timisoara and Laboratory of Steel Structures of Romanian Academy, Timisoara Branch, and taking into account for the increased market demand for quickly and easy-to-erect building facilities for small and medium size enterprises, three cold-formed steel structural systems for small and medium size residential and non-residential buildings have been developed. The candidate has been deeply involved in research, design and prefabrication process. It should be mentioned here that the examples presented in this paragraph represent the first structures built in Romania using thin-walled cold-formed steel members.

These structural systems, based on Lindab components, were designed to be used in heavy snow and seismic zones, like South-Eastern Europe and Balkans. The buildings are fully made by cold-formed steel sections, using bolted connections. Prefabricated units can be obtained. The diaphragm effect of roof sheeting panels and some vertical braces are used to resist against horizontal actions [2.77].

Three types of cold-formed steel buildings made by LINDAB profiles have been designed. The structures are different as destination and conception, i.e.:

- *Wall Stud Modular System (WSMS)* – bungalow type buildings, are used as shops, offices, for industrial purposes, for housing or school facilities;
- *Hall Type Framed Structure (HTFS)* – are used as small and medium size single storey industrial buildings and storehouses;
- *Penthouse Framed Structure (PFS)* – for refurbishment and restructuring by vertical addition of new storey the existing buildings.

Another innovative structural system included here is *Affordable Houses* (AH). Even if the main structure is made by hot-rolled profiles, the secondary structure is made by thin-walled cold-formed steel profiles, considering the diaphragm effect of roof sheeting panels and longitudinal walls to resist against horizontal actions [2.78, 3.43, 3.44]. Technical and environmental performances are presented in Chapter 3 – Case study 2.

On the following, the three types of cold-formed steel buildings, the general design principles and building technologies are presented together with some specific design detailing and accompanied with examples of application. For some of these innovative structural systems both technical and environmental performances have been studied.

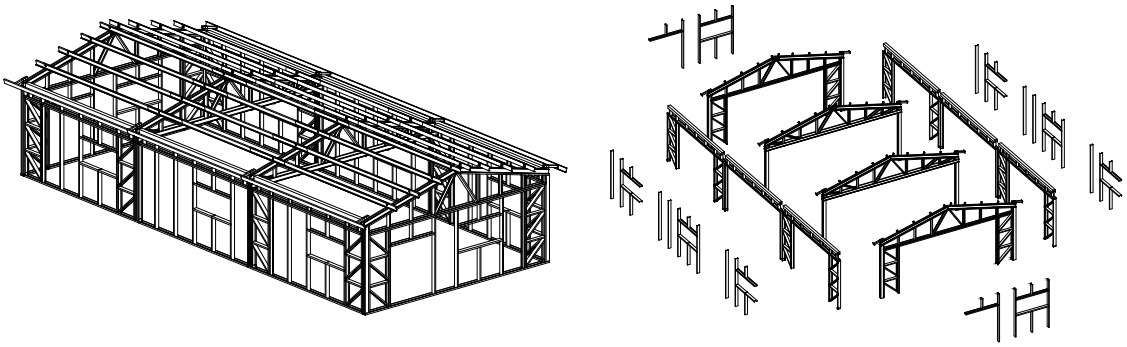
Wall Stud Modular System (WSMS) [2.79, 2.80, 2.81]

The main idea in designing the WSMS was to combine the "wall stud" system, used for steel framed housing, with a "light roof" solution, used for industrial hall type buildings. Figure 2.11 shows the basic components of the system. The 3D model comprises main frames, with built-up sections in columns and trusses, the wall-stud panels and the roof structure with purlins

and sheeting panels (diaphragm effect is considered), able to carry both vertical and horizontal loads. The “wall stud” panels are adaptable, according to the shape and dimensions of openings. All structural and non-structural components are made of cold-formed steel LINDAB sections.

The structure is designed by modules, to be installed bay-by-bay, using the so-called “stick” technique, easier to transport and to build pre-erected units (panels, trusses) on the site. Prefabrication in modules is also possible. The façade as well as the choice of foundation are decided by the customer in co-operation with the retailer and according to technical specifications given by designer.

Table 2.18 contains the characteristics of snow and seismic load data and main dimensions of the actual WSMS version, while Figure 2.12 shows the theoretical weight performances (steel consumption for the main frame structure, i.e. without purlins and adaptable wall panels) of the system [2.80].



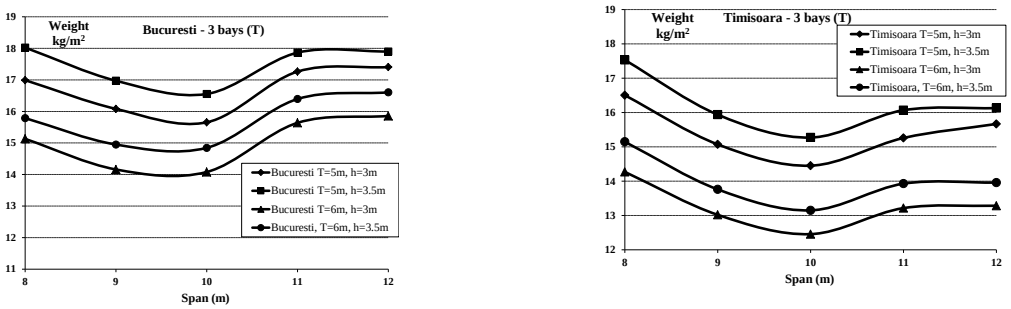
a) b)

Fig. 2.11: a) Main frame structure for a WSMS; b) Basic components of a WSMS

Table 2.18. Main dimensions of the actual WSMS and the characteristics values for actions

Region	Main dimensions			Design snow load g_z (kN/m^2)	Seismic parameters*
	Span S (m)	Bay B (m)	Eaves Height H (m)		
Bucharest	8-12	5 (6)	3 (3.5)	3.12	$\alpha = 1$; $k_s = 0.2$ ($T_c = 1.5$s); $\psi = 1$
Timisoara	8-12	5 (6)	3 (3.5)	1.8	$\alpha = 1$; $k_s = 0.2$ ($T_c = 1.0$s); $\psi = 1$

(*) According to Romanian Seismic Code P100-92 [2.84] the seismic force is calculated with the following formula $S_r = c_r \cdot G = \alpha \cdot k_s \cdot \beta_r \cdot \psi_r \cdot \varepsilon_r \cdot G$, where α is a coefficient of the building class, k_s is the coefficient depending on the seismic zone, β_r is the dynamic amplification coefficient in the rth vibration mode, ψ_r is coefficient of reduction of seismic effects = $1/q = 1$, ε_r is coefficient of equivalence between the actual system and system with a single degree of freedom, corresponding to the rth natural vibration mode, and G is resultant of gravity loads for the building.



a) b)

Fig. 2.12: Weight performances of the system

Table 2.19 displays the main dimensions, the location and the real weight (main frame structure) of some WSMS units erected in Romania, while Figures 2.13 to 2.16 are showing some representative images for these buildings.

Table 2.19. Main dimensions, location and the weight of some WSMS units

Name of structure	Dimensions S×(nB)×H (m)	Location	Weight (kg/m^2)
ROMRECYCLING Bucharest, Office building	10×(2×6)×3	Bucharest	20.1
JASMIN Satu-Mare, Storehouse for Dr. Oetker	10×(3×6)×3	Satu-Mare	20.6
AQUABIS Bistrita-Nasaud, Storehouse	10×(2×6)×3	Bistrita-Nasaud	21.9
FLAMINGO Bucharest, Storehouse	10×(3×6)×3	Bucharest	20.2
TERANITA Giurgiu, Storehouse	10×(4×6)×3	Giurgiu	21.3
ALCEDO Timisoara, Storehouse	9.65×16.75×3.9	Timisoara	23.8
BIOLACT Oradea, Storehouse	10×(7×5.4)×3	Oradea	23.2

Fig. 2.13: Main frame structure of BIOLACT building

Fig. 2.14: ROMRECYCLING structure during erection

Fig. 2.15: JASMIN building

Fig. 2.16: ALCEDO building

Hall Type Modular System (HTMS) [2.77, 2.81]

For eaves height greater than 3.5m the WSMS becomes inefficient to be used. For such situations, which may appear quite often in practice, a cold-formed steel structural system based

on pitched roof portal frames made by built-up "C" sections (Johnston type [2.82]) was developed. The main components of this system are shown in Figure 2.17.

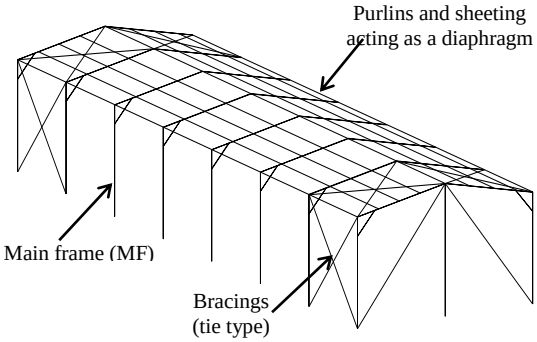


Fig. 2.17: Main frame structure and basic components of a HTMS

Several industrial buildings, using Johnston built-up cold-formed sections have been designed by the candidate accompanied by several research papers [2.77, 2.81, 2.83]. Two examples are presented in Table 2.20.

Table 2.20: Main dimensions, location and the weight of some HTMS units

No.	Name	Location	Type of building	Main dimensions			Weight (kg/m^2)
				L [m]	B [m]	H [m]	
1	ARBEMA Hall	Arad	Single story house	12	42	7.00	31
2	POPET Hall	Vaslui	Single story house	12	36	4.28	21

Due to the fact the cold-formed steel sections are of class 4, the structures are considered to work in the elastic range, e.g. non-dissipative, and correspondingly, the $\psi = 1/q$ coefficient is taken equal to 1, when the seismic force is evaluated according with Romanian Seismic Design Code P100-92 [2.84]. The structures were designed accounting for diaphragm action of sheeting [2.77]. In Figure 2.18 are shown the steel consumption curves for some usual dimensions of a building located in Bucharest.

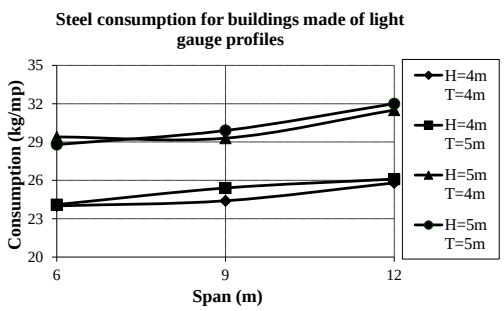


Fig. 2.18: Steel consumption curves (location: Bucharest)

Two relevant examples of application of HTMS system are the ARBEMA Brewery thermal power station building located in Arad City, in the west part of Romania, near to Timisoara and POPET Hall in Vaslui, located in the east part of Romania (see Table 2.20 and Figures 2.19 and 2.20).

Fig. 2.19: HTMS structure in Arad City

Fig. 2.20: HTMS structure in Vaslui City

Penthouse Framed Structures (PFS) [2.85, 2.86]

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

A cold-formed steel structure made by built-up members of Johnston type [2.82], by LINDAB C sections, and special bolted connections to design very efficient penthouses above the existing masonry or RC buildings was proposed. The new floor, because the existing terrace (RC slab or timber structure) usually has not enough load-bearing capacity to carry the additional structure, with its dead and life loads, is made by a steel castellated beam grid.

The structure described below represents an extension of the production capacity belonging to ALCATEL-DATATIM Company of Timisoara [2.85, 2.86].

By project initial data, an extension of the existing production space was required by:
a) erection of a ground floor structure over the existing courtyard;
b) erection of a penthouse superstructure over the buildings;
c) erection of a staircase in the space of the access gallery, to provide access from downstairs to the first floor level;
d) extension of the social outbuilding also by penthouse structure.

The main characteristics of penthouse component buildings are given in Table 2.21.

Table 2.21: Main characteristics of penthouse component buildings

Name	Destination	Deck Level [m]	Eaves Height [m]	Built Area [m^2]	Built Volume [m^3]
Penthouse over buildings	Production	+5,70	4,00	1066	4264
Social outbuilding	Social	+5,00	3,00	85	255
Greenhouse	Social	±0,00	8,00	90	720
Stair case + Stairs	Access to first floor	±0,00	9,00	36	320

Due to the fact Timisoara is a seismic territory the penthouse structure had to be light enough in order to avoid a heavy and expensive strengthening of existing RC structure, not properly designed according to seismic design criteria. However, the RC structure was supplementary braced.

A thin-walled cold-formed structure connected exclusively by bolts was proposed by the authors of this paper for the penthouse superstructure. The roofing and cladding panels are of LINDAB trapezoidal steel sheet, with suitable thermal insulation. As interface structure, a metallic grid built of castellated steel beams interconnected by HSFG bolts and connected to the concrete structure by cruciform steel elements was provided. The floor of the penthouse is a composite steel-concrete deck [2.86].

In order to avoid to overload the existing structure with bending moments from horizontal actions (earthquake and wind), the column supports at both ends are pinned and, correspondingly, the structure had to be properly braced. The diaphragm effect of sheeting in roof structure was also considered.

Figure 2.21 shows the structure during erection and after the roof envelope was completed.

Fig. 2.21: Cold-formed steel structure during the erection and after the roof envelope was completed

The structure was designed on the basis of a 3D static and dynamic analysis. Technical performances proving the efficiency of this structure are shown in Table 2.22.

A very important advantage of this structural solution, in the particular conditions of narrow building site, was the easy erection. Practically, the structure was raised, stick by stick, on the deck of new floor, and there the trusses and frames were assembled and lifted with a light autocrane.

Table 2.22. Technical performances

Characteristic	Value	Comments
Maximum transversal top drift Maximum longitudinal top drift Eigen period on transversal direction Eigen period on longitudinal direction	19.3 mm 36.8 mm 0.388 sec 0.690 sec	Seismic action
Steel weight of the skeleton Steel weight of purlins and rails TOTAL steel weight of structure	17 kg/m^2 9 kg/m^2 26 kg/m^2	

Figure 2.22 shows a view of the erection procedure and the corresponding building façade nearly completed. Some specific details for the Alcatel-Datatim penthouse structures are shown in Figures 2.23 to 2.24.

Fig. 2.22: View of the erection procedure and corresponding building façade

Fig. 2.23: Eaves connection

Fig. 2.24: Ridge connection

Fig. 2.24: Base-column connection

Conclusions

Light gauge steel structures are more and more attractive in the recent years. They are characterised by a good price-quality ratio and, very important, they can be built very quickly. Within the actual technological and economical contest, construction industry needs methods to build faster and to obtain better quality "products". Cold-formed steel framed constructions can be a solution to satisfy this need, due to easy prefabrication, reduced erection time, and possibility of high-precision quality control.

In a seismic territory like Romania, the use of light gauge steel structures has the advantage of a reduced mass, and they are recommended to be used. Even this structures are considered non-dissipative and have to be considered in the elastic range (q=1), generally the dominant load combination for design is the fundamental one, except for the case of bracings and column-base anchor bolts.

Structural systems based on the use of cold-formed built-up double C sections, of Johnston type, together with some particular bolted connection details, show very good performances.

Care has to be taken for the behaviour of truss connections which are of semi-rigid type. The effect of semi-rigidity can be a benefit but also a source for supplementary stresses.

The examples presented here are the first structures built in Romania using thin-walled cold-formed steel members. They show the large applicability of cold-formed steel sections, to very different kind of buildings using either, standardised WSMS, specific HTMS or PFS structures, each complying with specific requirements, and, not in the last time, the possibility of obtaining a good price-quality ratio, easy and short time erection.

2.7 Scientific and technical contributions of the author to the actual state-of-knowledge

Regarding the subjects presented above and based on the studies done by the author and the obtained results, the personal contributions include:

1. *Post-elastic strength of thin-walled cold-formed steel members. Plastic mechanisms for members in compression and bending;*
   - Description of a database for plastic mechanisms of thin-walled cold-formed steel members in compression and bending, in order to obtain the ultimate strength of members;
   - Proposal of a method using plastic mechanisms to obtain bending moment-plastic rotation curves of TWCF steel sections and, based on this basis, to evaluate their ductility and performance in seismic resistant structures.

2. *Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections*;
   - Identification of a more realistic failure mode involving the interaction between compression flange due to bending and web crippling due to local transverse action induced by bolts in bearing and locking of flanges;
   - Proposal of an analytical procedure to evaluate the rotational rigidity of a lapped connection between Z-sections which accounts for global shear and bending and for local bearing in the web of sections around the bolt holes;
   - Proposal of an new interactive analytical model, i.e. bending moment – web crippling, at the end of overlapping.

3. *Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed steel members;*
   - Identification of the critical section for usual frames at the edge of lap between the connecting bracket and rafter (i.e. the first local buckling location) and proposal of three analytical methods to estimate the ultimate resistance at the edge of bracket-to-rafter lap, i.e.: (1) the "effective width" model, based on EN1993-1-3 procedure; (2) the "plastic"

model, based on the local plastic mechanism; (3) the "interactive" model, which considers the coupling effect between web crippling, at the edge of the bracket-to-member lap, and the bearing in the bolt holes in flanges;

- Development of a numerical simulation programs carried out on full-scale pitched roof cold-formed steel portal frames of back-to-back lipped channel sections with bolted joints in order to evaluate the influence of different type of geometrical and structural imperfections on the structural stability performance of these structures.

4. *Behaviour of cold-formed steel perforated pallet rack sections in compression.*
   - Development of a sensitivity analysis done using calibrated and validated numerical models in order to determine the most detrimental combinations of imperfections to be considered for numerical simulations, in the interactive buckling range;
   - Evaluation of the erosion of critical bifurcation load as a result of the coupling effect and geometric imperfections and the computation of the corresponding imperfection coefficients for specimens having the length in the interactive buckling range.

Innovative structural systems:

- Application of knowledge and lessons learned in the research activity on the design of three types of structures made of thin-walled cold-formed steel members, i.e.: (1) *Wall Stud Modular System* (WSMS); (2) *Hall Type Framed Structure* (HTFS); (3) *Penthouse Framed Structure* (PFS);
- It should be mentioned that at the beginning of 2000's these applications represented the first structures built in Romania, using thin-walled cold-formed steel members;
- Realisation of a pilot application, development and implementation of a series of application for *Wall Stud Modular System* (WSMS) at LINDAB Romania. Is should be underlined here that *Wall Stud Modular System was the* bronze medallist at the EXPO CONSTRUCT, Bucharest, 14-18 March 2001.

3. Sustainability of Constructions

National and international experience of the candidate related to the topic "Sustainability of Constructions"

Member of technical boards:

- Member of Management Committee (representing Romania) of COST C25 action: "*Sustainability of Constructions – Integrated Approach to Life-time Structural Engineering*" and vice-chairman of Working Group WG3: "*Life-time Structural Engineering*" of COST C25 action;
- Member of Technical Committee TC14 "*Sustainability and Eco-efficiency of Steel Building*" of the European Convention for Constructional Steelwork (ECCS);
- Member of International Association for Life-Cycle Civil Engineering – IALCCE.

Conference committees:

- Chairman of Workshop: *Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*. The 9th Management Committee and 8th Working Group Meetings of Cost C25, Timisoara, Romania, 23-24.10.2009;
- Chairman of Technical Session: *Sustainability*, of International Conference "Steel Structures: Culture & Sustainability 2010", 21-23 September 2010, Istanbul, Turkey;
- Member of Scientific Committee of *International Symposium Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*, 11 May 2009, Naples, Italy;
- Member of Scientific Committee of *International Conference "Steel Structures: Culture & Sustainability 2010"*, 21-23 September 2010, Istanbul, Turkey;
- Member of Scientific Committee of *International Conference: Sustainability of Constructions – Towards a better built environment*. 03-05 February 2011, Innsbruck, Austria.

Research projects:

- RFSR-CT-2010-00027: „*SB_Steel – Sustainable Building Project in Steel*". Period: 2010-2013. Beneficiary: RFCS – Research Fund for Coil and Steel, EU. Total value: 140000 EUR (coordinator of PUT research team).
- ERA-NET 3-002/2011: *"INSPIRE – Integrated strategies and policy instruments for retrofitting buildings to reduce primary energy use and GHG emissions"*. Period: 2010-2012. Beneficiary: UEFISCDI – Romania. Total value: 180000 EUR (coordinator of PUT research team).
- AF2009: *Affordable House Project*. Beneficiary: ArcelorMittal Liege Research. Total value: 25000€ (member of PUT research team).

Invited papers and courses:

- Invited paper: *Sustainable Design of Steel Framed Houses*, at The 9th Constructional Steelwork Day – TUCSA, 27.10.2008, Istanbul, Turkey. Authors: Dan Dubina, **Viorel Ungureanu**, Mihai Mutiu & Daniel Grecea.
- Invited paper: *Steel structures & Sustainability*, at The 10th Constructional Steelwork Day – TUCSA, 05.11.2009, Istanbul, Turkey. Authors: **Viorel Ungureanu**, Adrian Ciutina, Daniel Grecea & Dan Dubina.

- Two lectures at the *BEST Summer Course Home Alone – Green Edition: Sustainability of Built Environment under Effects of Climate Change*, 22.07-02.08.2010, Timişoara, http://courses.ro/sc10/academics.html (30 students from 20 countries):
  - *Course C2. Sustainable development: Framework & Backgrounds (2h, Dan Dubina, **Viorel Ungureanu**, Daniel Grecea);*
  - *Course C4. Environmental Impact of Constructions: LCA and LCC (4h, **Viorel Ungureanu**, Adrian Ciutina, Daniel Grecea).*

Books / Books chapters:

- P. Santos, L. Simoes de Silva, **V. Ungureanu**: *Energy efficiency of light-weight steel-framed buildings*. Technical Committee 14: Sustainability and Eco-efficiency of Steel Constructions, No. 129/2012, Published by ECCS – European Convention for Constructional Steelwork, ISBN: 978-92-9147-105-8, 175 pages.
- **Ungureanu V.**, Ciutina A., Tuca I., Dubina D., Grecea D., Fulop L.: *Structural and Environmental Performance of Steel Framed Houses: A Case Study.* In: Summary report of the Cooperative Activities. Cost Action C25. Volume 1: Integrated approach towards sustainable constructions. ISBN: 978-99957-816-1-3, 2011, pp. 395-428.
- **Ungureanu V.**, Dogariu A., Dubina D., Ciutina A., Landolfo R., Mazzolani F.M., Portioli F.: *Maintenance, repair and rehabilitation of buildings.* In: Summary report of the Cooperative Activities. Cost Action C25. Volume 2: Integrated approach to life-time structural engineering. ISBN: 978-99957-816-2-0, 2011, pp. 235-281.
- Dubina D., **Ungureanu V.**: *Sustainable technologies for maintenance of structures.* In: International Training School: COST C25 Sustainability of Construction: A Life Cycle Approach in Engineering, 26.07-01.08.2010, Malta, Faculty of the Build Environment, University of Malta, ISBN 978-9993209188, 2010, pp. 321-429.

Scientific articles with international partnership

1. Grodzicka E., Dehan V., Koukkari H., Zukowska E.A., Dincă A., Marcu I., Ciutina A., **Ungureanu V.**: Competitiveness and sustainability of steel-intensive solutions. In: Concepts and Methods for Steel Intensive Building Projects. Technical Committee 14: Sustainability and Eco-efficiency of Steel Constructions, No. 130/2012, Published by ECCS – European Convention for Constructional Steelwork, ISBN 978-92-9147-106-5, pp. 181-194.
2. Portioli F., Cascini L., Landolfo R., **Ungureanu V.**: *Sustainable refurbishment, deconstruction and demolition: Application of MCDM method*. Summary report of the Cooperative Activities. Cost Action C25. Volume 2: Integrated approach to life-time structural engineering. ISBN: 978-99957-816-2-0, 2011, pp. 317-326.
3. Portioli F., Cascini L., **Ungureanu V.**: *Multi-criteria decision making methods in refurbishment, deconstruction and demolition of existing structures*. Proceedings of the International Symposium: COST C25 Sustainability of Construction: Integrated Approach to Life-time Structural Engineering, Malta, 23-25.07.2010, ISBN 978-9993209171, pp. 177-184.
4. Bragança L., Koukkari H., Blok R., Gervásio H., Veljkovic M., Plewako Z., Landolfo R., **Ungureanu V.**: *Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*. Proceedings of Improving the Quality of Suburban Building Stock, COST Action TU0701 – Mid Term Conference, Malta, 7 – 8 May 2010, ISBN 978-99932-0-880-8, pp. 601-614.
5. Portioli F., **Ungureanu V.**: *Demolition and deconstruction of building structures*. Proceedings of Seminar: Sustainability of Construction: Integrated Approach to Life-time Structural Engineering, 6-7 October 2008, Dresden, Germany. ISBN: 978-3-86780-094-5, pp. 4.124-4.131.

Editor of international proceedings and journals

1. Braganca L., Koukkari H., Blok R., Gervasio H., Veljkovic M., Borg P.R., Landolfo R., **Ungureanu V.**, Schaur C. (editors): *Proceedings of the International Conference: Sustainability of Constructions – Towards a better built environment*. 3-5 February 2011, Innsbruck, Austria, ISBN: 978-999957-816-0-6, COST Publication, 582 pp.
2. Braganca L., Koukkari H., Landolfo R., **Ungureanu V.**, Vesikari E., Hechler O. (editors): *Summary report of the Cooperative Activities. Cost Action C25. Volume 2: Integrated approach to life-time structural engineering*. ISBN: 978-99957-816-2-0, COST Publication, 398 pp.
3. Braganca L., Koukkari H., Blok R., Gervasio H., Veljkovic M., Plewako Z., Landolfo R., **Ungureanu V.**, Silva L.S. (editors): *Proceedings of the Workshop Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*. Timişoara, Romania, 23-24.10.2009. Cost Action C25. Editura Orizonturi Universitare, ISBN: 978-973-638-428-8, COST Publication, 412 pp.
4. Braganca L., Koukkari H., Blok R., Gervasio H., Veljkovic M., Plewako Z., Landolfo R., **Ungureanu V.**, Silva L.S., Haller P. (editors): *Proceedings of the Second Workshop Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*. Dresden 6-7 October 2008. Cost Action C25. ISBN: 978-3-86780-094-5, COST Publication, 462 pp.
5. Braganca L., Koukkari H., Blok R., Gervasio H., Veljkovic M., Plewako Z., Landolfo R., **Ungureanu V.**, Silva L.S. (editors): *Proceedings of the First Workshop Sustainability of Constructions. Integrated Approach to Life-time Structural Engineering*. Lisbon, 13-15 September 2007. Cost Action C25. Published by Multicomp, Lda. ISBN: 978-989-20-0787-8, COST Publication, 304 pp.

3.1 Introduction

Sustainable development has become in the last decade the conceptual framework for almost all areas of social and economic life. One possible definition is "creating the highest quality construction possible while the lowest impact on the environment" [3.1]. Sustainable development was intended at first to be a solution to the environmental crisis caused by intense industrial exploitation of resources and to the continue environmental degradation.

Sustainability is one of the greatest challenges of the modern world. A very well-known definition of this concept was given by *World Commission on Environmental and Development* in 1987: “Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.” Sustainability includes environmental, economic and social aspects which contribute to a durable development of the society [3.2]. In this way, the concept has expanded the quality of life in its complexity, encompassing economic and social aspects in addition to environmental factors. In recent years a fourth pillar – the cultural one – has been defined, considered the key to a truly sustainable development. It refers to a global paradigm shift, involving both humanity as a whole and each of us. In this respect the cultural dimension is understood in its widest sense. The cultural pillar thus includes the educational and institutional dimensions.

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 [3.3]. The use of the buildings and all construction related activities generate more than 40% of all CO_2 emissions, use about 40% of the produced energy and consume more than 40% of the material resources used in the society. 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.

Construction sector play an important role to sustainable development of the world and national economies. Sustainable construction has different approaches and different priorities in various countries. Some of them identify economic, social and cultural aspects as part of their sustainable construction, but it is raised as a major issue only in a few countries.

Sustainable construction can be regarded as a subset of sustainable development and contains a wide range of issues, i.e.: re-use of existing built assets, design for minimum waste, minimizing resource and energy use and reducing pollution.

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).

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.

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 and 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.

Improving energy efficiency in buildings can reduce [3.4]:

- whole-life costs;
- depletion of non-renewable fuels, e.g. oil, gas and coal;
- global warming from the burning of carbon-based fossil fuels.

To understand how the impacts of buildings in construction and use can be reduced, it is important to distinguish between embodied and operational energy. Embodied energy is the energy required to manufacture and transport the materials and for the means of construction and to carry out the building process. 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.

It is widely acknowledged that operational energy consumption in intensively used and heavily populated buildings far outweighs the embodied energy. For example, research undertaken by the Steel Construction Institute [3.5] found that for an air-conditioned office building over a 60-year design life, the ratio of embodied to operational energy is around 1:10 (see Figure 3.1).

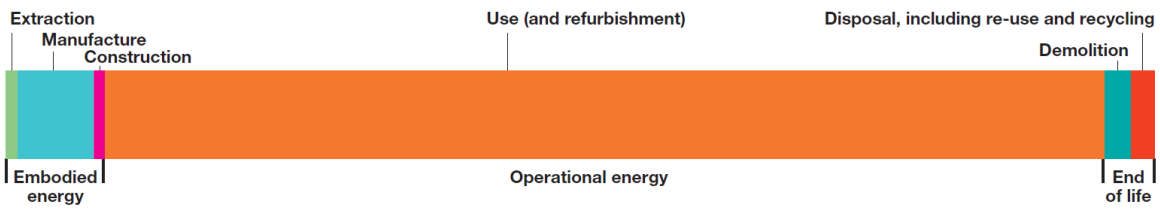


Fig. 3.1: Energy life cycle for an office building over 60 years [3.4, 3.5]

The message is clear – in such buildings, designers can make the greatest contribution to sustainable construction by concentrating their efforts on reducing operational energy. Greater energy efficiency in buildings is generally achieved by a combination of the following measures [3.4]:

- reducing primary heat losses through the building envelope;
- reducing cooling loads;
- introducing energy saving measures in the operation of the building, e.g. energy efficient electrical appliances;

- installing energy creation systems, such as photovoltaic panels and combined heat and power plants;
- improving natural lighting.

Another important aspect is related to landfilling of building waste, which is no longer acceptable or affordable and, in some cases, legal. The challenge for designers is to minimise wastage through all construction stages. For example, each year, construction and demolition activities in the UK generate around 90 million tonnes of waste [3.6]. Wastage occurs in the production of construction products, during the construction process, throughout the life of the building and at its end-of-life. In the past, little thought was given to the disposal of construction waste and frequently it was simply landfilled (see Figure 3.2).

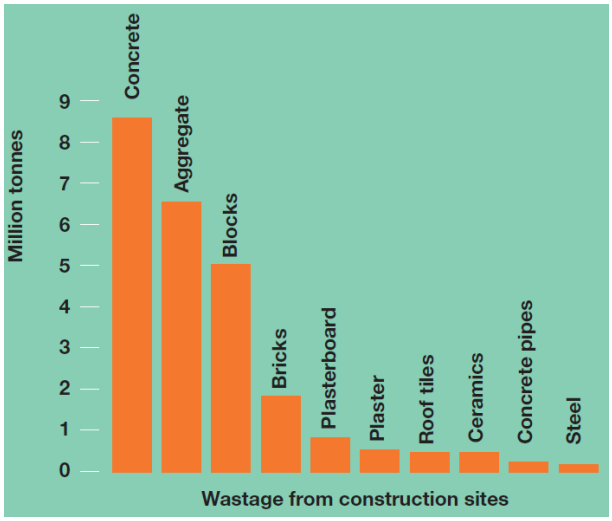


Fig. 3.2: Wastage from construction sites [3.4, 3.6]

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 [3.7, 3.8].

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 & Maydl [3.9] showed. They underlined that 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. Life Cycle Assessment (LCA) is the best way to determine the environmental impact of products or processes. Despite various studies about the approaches for the buildings sustainability assessment, there is a lack of a worldwide accepted method of quantifying the environmental impact [3.10].

The steel construction sector has a great deal to offer sustainable development. Like other industrial activities, steel construction work for continuously improvement in terms of sustainability. The following guiding principles for sustainable constructions can be emphasized [3.11]:

- 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.

Steel, as material for construction, plays an important role as component of building and engineering structures, and it is used in a wide range of applications. Steel construction means using steel as the building's main framing material. Steel is also common in the building envelope, fasteners, building services, substructures and as concrete reinforcement. On the other hand, steel is the most recycled material and, actually, from the total production in the world, almost half is obtained from waste material. It is, perhaps, interesting to know that steel collected from three recycled cars can be converted in a structure for a cold-formed steel framing house. Compared to today's average construction practice, modern steel construction offers [3.12]:

- material efficiency – resulting in less natural resource usage, less transports, less emissions and less energy usage;
- ultra-high recyclability – resulting in less natural resource usage, less waste and less emissions;
- quality and durability – resulting in many sustainability favours;
- dry and lean construction – resulting in less health hazards, less waste, less energy usage, less emissions and a better work environment.

Like other industrial activities steel construction has direct relations to several important sustainability issues, like energy and waste, and further improvements are to be done continuously.

3.2 Life Cycle Assessment (LCA)

Life Cycle Assessment (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 [3.13]. 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. [3.14] 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 CO_2 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 [3.15].

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.

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 3.3.

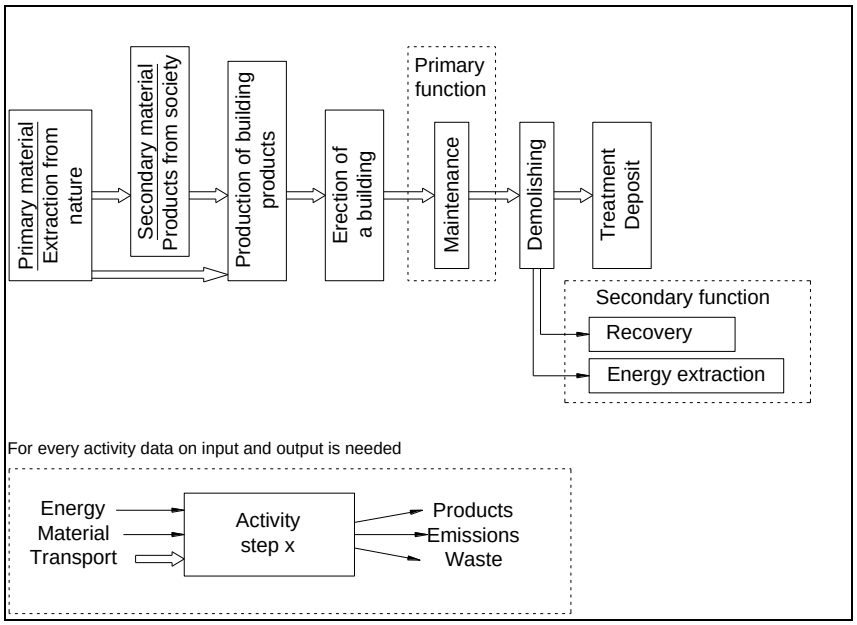


Fig. 3.3: Illustration of life cycle stages of a building and the data for the LCA [3.15]

3.2.1 LCA assessment phases

3.2.1.1 Context

To make an LCA assessment there are needed some key elements. Although there is no single way to conduct a 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 [3.16] it is recommended that it shall be called "functional equivalent".

The steps to follow in conducting a LCA analysis are shown in Figure 3.4 [3.17].

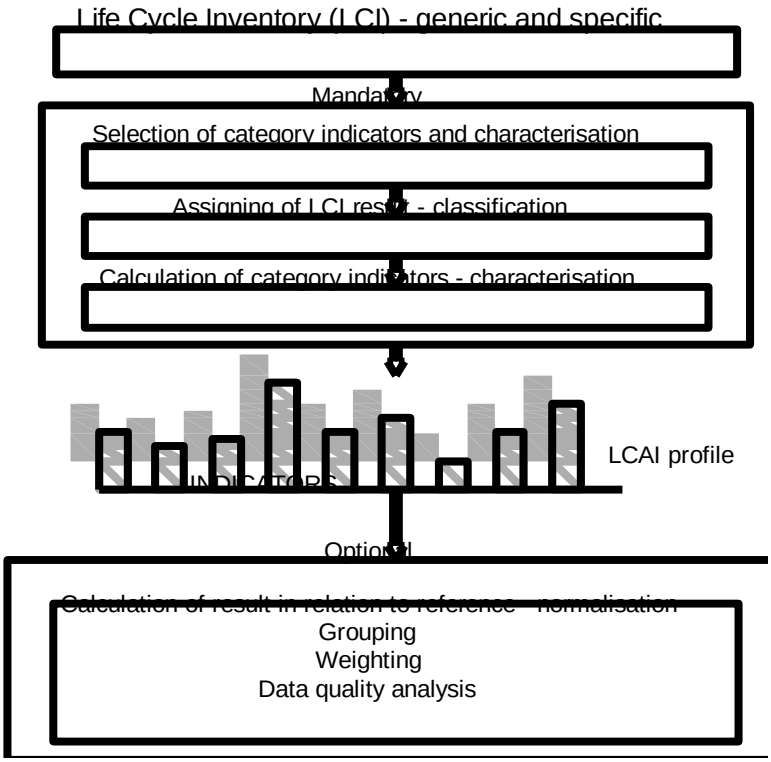


Fig. 3.4: Illustrating the actions carried out for the life cycle assessment [3.17]

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).

3.2.1.2 LCC (Life Cycle Cost) evaluation

Life cycle cost (*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.

3.2.1.3 LCA and LCC Integration

Due to the fact both LCA and LCC are based on life cycle, assuming a life cycle for 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.

3.2.2 The current legal framework

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 TC350 documents will be gathered in European regulations, present drafting norms being provisional Europeans standards (pre-standards), such as:

- prEN 15643: Sustainability of construction works – Assessment of buildings [3.16];
- prEN 15804: Sustainability of construction works – Environmental product declarations [3.18];
- prEN 15978: Sustainability of construction works – Assessment of environmental performance of buildings [3.19].

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* [3.13], together with ISO 14044 – *Environmental Management – Life-cycle Assessment – Requirements and Guidelines* [3.20].

ISO standard 14040 [3.13] 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 [3.20] 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.

3.3 Sustainable steel framed houses in Romania – Case studies

In recent years, steel framed houses have become a choice for house construction in many European countries, including Romania. Compared with traditional solutions, the properties of the steel skeleton can be exploited to take both technical and economic advantages from lightness of structures, ease of prefabrication, speed of erection and enhanced quality.

On the following, three case studies of sustainable mixed building technologies, which combine steel in the framing and different materials for cladding, roofing and flooring, in order to obtain highly performance thermo-energetic properties are presented.

Case study 1. The aim of the first case study is a comparative life cycle analysis between a cold-formed steel framing house and an identical masonry one. The house under evaluation represents an existing single-family house, built in 2005 in Ploiesti, Romania. The structural skeleton is made of light-gauge C shaped profiles sheathed with OSB plates. In order to complete the environmental impact comparison, the same house has been designed with masonry structural walls and concrete floor, according to Romanian traditional houses. The impact

analyses were performed at different levels: (i) construction, (ii) construction and end-of-life, (iii) life cycle including maintenance and, (iv) the life cycle including maintenance and consumable goods.

This research was developed in the frame of COST C25 action: "Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering", in which I was involved as member of Management Committee and Vice-chairman of Working Group WG3: "Life-time Structural Engineering".

Case study 2. Another important issue raised in recent years was to find the most economic dwelling building systems. The users, architects and engineers have all been searching for the best solution. In this context, steel-framed houses have become a choice in many European countries, including Romania. One of the issues raised in the case of the new systems concerns their environmental impact. This case study presents an innovative structure/envelope solution, enabling flexible floor plans and modular construction, faster fabrication and erection times and high solution diversity for floors and envelope. The main features of the architectural and technical solution are first presented. In the second part, a comparative life cycle analysis for the above house, designed with various solutions is presented. For this purpose, the building was designed with four different solutions, each having its own structural system, as follows: (1) hot-rolled steel framing – the innovative structure; (2) cold-formed steel framing; (3) timber framing, and (4) masonry. Several aspects are presented, in order to underline the impact for different stages during the life cycle: (a) construction stage only; (b) construction stage and final disposal of materials; (c) construction stage, disposal and maintenance.

This research was developed in the frame of Affordable House Project, project granted by ArcelorMittal Network, in which I have been involved as member of the research team.

Case study 3. The case study presents a building in the context of sustainability, i.e. a block of flats built in 2007 in Timisoara, Romania. The main structure is made of steel profiles with light floors. Some innovative design solutions have been used in this project, i.e. thermo-energetic cladding system, gas-electric energy supply system etc. Particular attention has been paid to the aspects related to design and detailing, as well as solutions for cladding, including structural features and thermo-energetic performance. A comparative study, related to the environmental impact analysis for the construction stage only, for the structure designed in two solutions: (a) steel framed structure and (b) reinforced concrete framed structure will be presented.

3.3.1 Case study 1 – Cold-formed steel framing house vs. traditional (masonry) house

Steel framed houses have become a choice for house construction. Compared with traditional solutions, the properties of the light-gauge steel skeleton can be exploited to take both technical and economic advantages from lightness of structures, ease of prefabrication, speed of erection and enhanced quality. In what concerns the sustainability of such systems, one may think that they could have an important environmental impact, taking into account the types of materials used: steel for the skeleton, also steel for roof tiles, highly processed wood – Oriented Strand Boards (OSB) for wall boarding and so on.

This is the reason why a comparison between the "traditional" house (masonry walls, concrete floor and timber roof structure) and the "thin-walled cold-formed" (TWCF) house is welcome in terms of sustainability. Similar studies [3.21] showed through LCA studies that the environmental performance of light-weight steel houses is greater than that of traditional ones.

The house under investigation represents a private single-family house, built in TWCF solution, in 2005 in Ploiesti, Romania [3.22]. There are two main characteristics of the built house, namely the lightweight-steel framing and the architectural solution constrained by the site shape (see Figure 3.5).

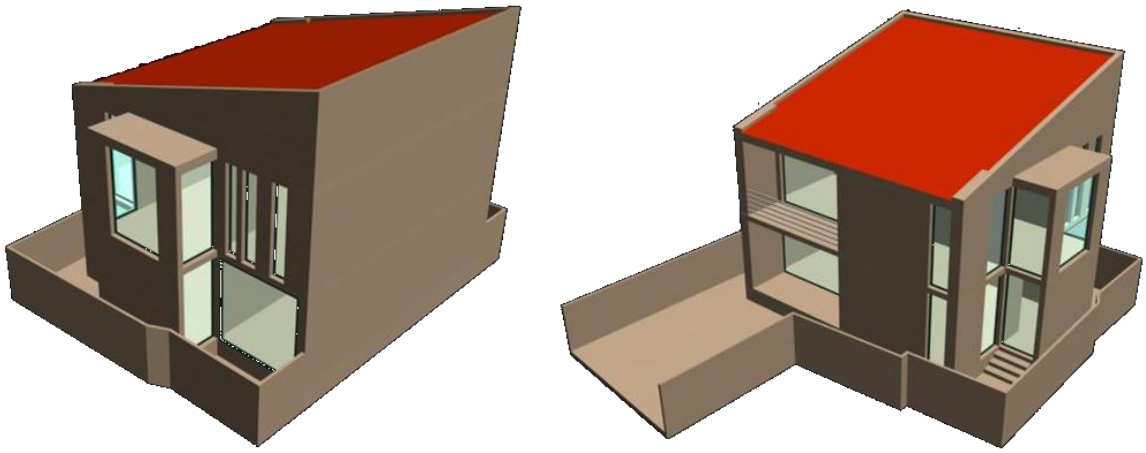
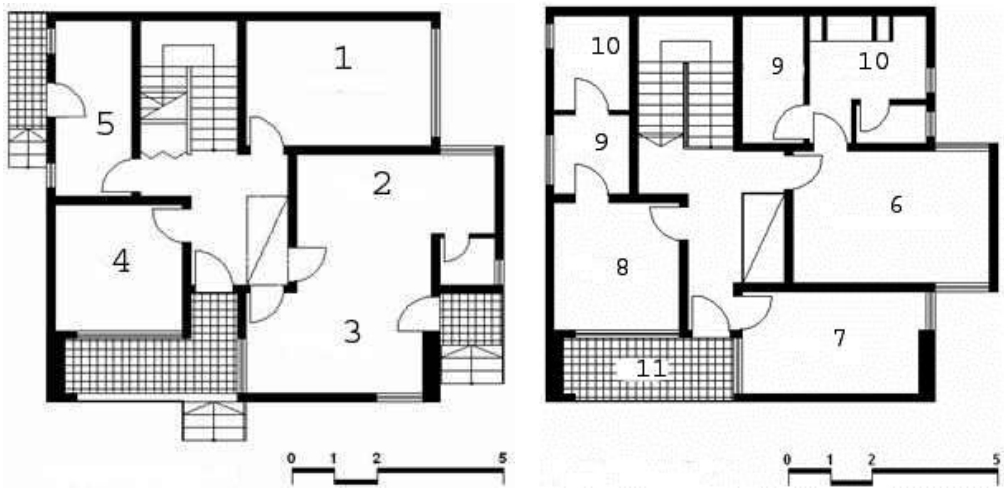


Fig. 3.5: 3D views of Constantin's family house [3.22]

From the architectural point of view, the main challenge of the project was to fit the house on an irregularly shaped lot of only 168m^2. The resulting cube-like building measured at the end 84m^2 of built area on each of the two floors (see Figure 3.6). 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 using a single sloped roof, the resulting 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 in order to provide a light shaft, in order to enhance the central part.



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. logia
Fig. 3.6: First and second floor plan of Constantin's house

3.3.1.1 Description of TWCF 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 (see Figure 3.7). The height of the cross-section of profiles is 150mm, dimension that governed the thickness of the walls. In order to withstand horizontal actions and to provide stiffness and strength the walls were stiffened using 12mm thick Oriented Strand Boards (OSB) provided on both sides of the structural walls, as Figure 3.8 shows [3.22].

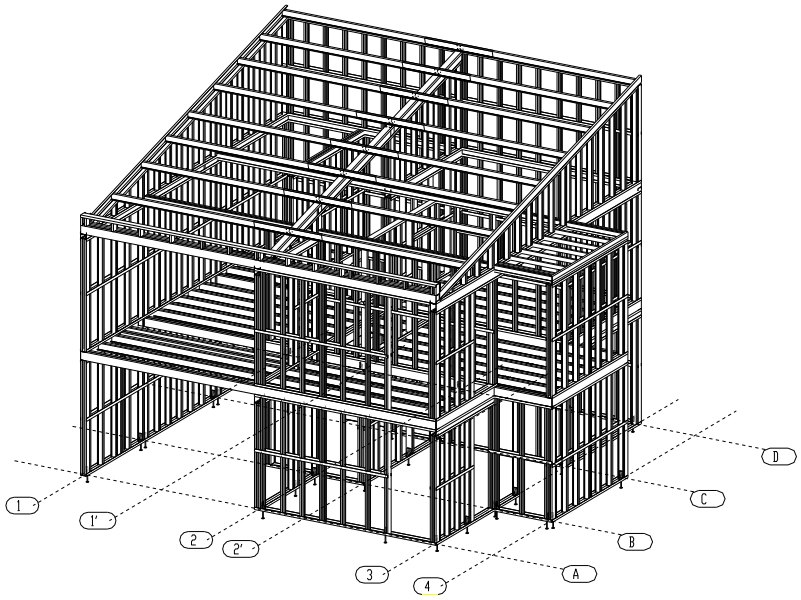


Fig. 1.7: Steel skeleton of the structure

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.

The isolated foundations under columns were linked by foundation beams. The floor at ground level (concrete slab of 10cm) was cast over a bed of dense soil and a layer of compacted gravel.

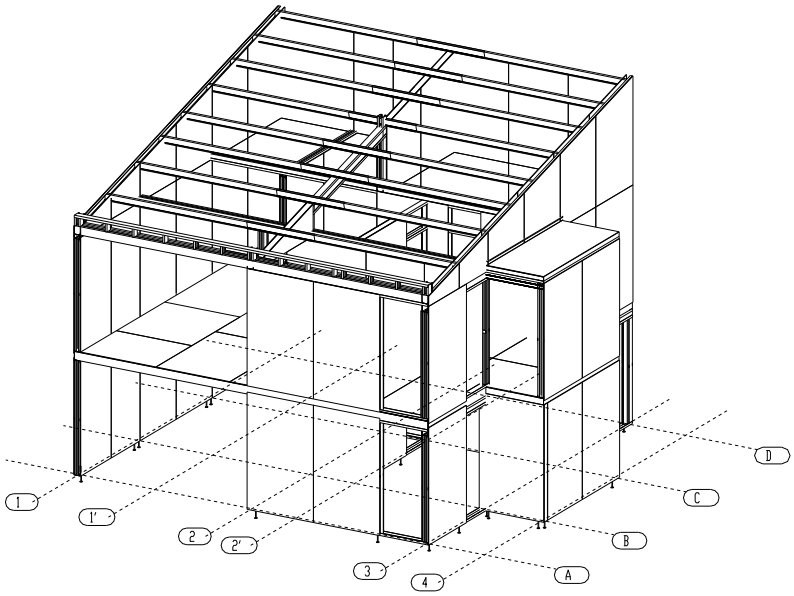


Fig. 3.8: Skeleton with structural OSB sheeting

Figure 3.9 shows the structure in two different stages: (a) the finished steel skeleton and (b) the steel skeleton stiffened by load bearing OSB panels. Figure 3.10 presents final exterior and interior views of the completed house.

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

Fig. 3.9: The structure during construction

(a) General view (b) Interior view – staircase

Fig. 3.10: Constantin's family house – completed house

3.3.1.2 Description of traditional Romanian dwelling

In order to have a real environmental impact comparison, a masonry structure has been designed following the above-described architectural plans [3.23]. Due to differences of wall thicknesses 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. The resulted structural system is composed by masonry walls of 25cm, concrete floor, continuous foundations and timber roof structure covered with ceramic tiles, as presented in Figure 3.11.

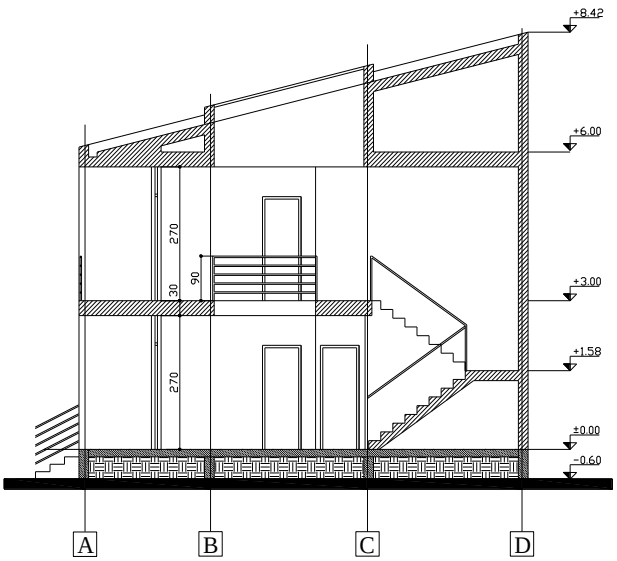


Fig. 3.11: Vertical section of the masonry structure

3.3.1.3 Structural design of TWCF structure

The structure is designed considering the climatic and seismic conditions of the city of Ploiesti, Romania. The loads used for the design of the structure were taken according to Romanian codes, actually aligned to relevant Eurocodes.

The self weight of the structure was evaluated at: 0.45kN/m^2 for the roof, 0.70kN/m^2 for the slab, 0.60kN/m^2 for exterior walls and 0.20kN/m^2 for internal walls. The other loading conditions were determined according to the Romanian standards. Imposed load on the slab was taken as 1.50kN/m^2, snow load on the roof as 1.20kN/m^2 and wind load on the surface exposed to maximum pressure as 0.40kN/m^2. The earthquake design of the building was done for a peak ground acceleration (PGA) of 0.25g allowing no reduction of the seismic action by choosing a behaviour factor q = 1. This condition is required by national regulations, which do not allow for reduction of seismic forces in case of light-gauge steel structures; the rules being independent of the structural scheme. The combination of actions was done according to EN1990:2002 [3.32].

The design of structural elements (columns, beams) was made according with EN1993-1-1 [3.24], EN1993-1-3 [3.25], EN1993-1-8 [3.26] and Romanian seismic design code P100-92 [3.27], both for ULS and SLS. To control the floor vibrations, a deflection of L/400 was considered reasonable for a family house.

The structural skeleton is made of light-gauge C shaped profiles framing. Columns are pinned at the base on both directions. In order to withstand horizontal actions from wind and earthquake and to provide stiffness and strength, the walls were stiffened using 10mm thick OSB provided on both sides of the structural walls. The floor diaphragms were designed to be based on the same principle of covering with OSB.

One of the problems arising during the design of such a structure is the evaluation of the load bearing capacity and of the rigidity of the sheathing system of walls and slabs. The case of the slabs can be covered by general provisions (ex. some detailing conditions) due to the low level of stresses, but the realistic analysis for the walls is crucial. In what concerns the seismic response, the performance of shear walls is crucial.

A large experimental and numerical investigation campaign has been carried out at the “Politehnica” University of Timisoara, in order to evaluate and characterize the behaviour of this type of structures [3.28, 3.29, 3.30]. First, the stiffness, resistance and ductility of cold-formed steel shear walls have been evaluated via laboratory tests and numerical simulations. A particular attention was paid to the influence of the fastening system [3.33].

For the 3D analysis, the rigidity induced by the sheathing has been replaced by equivalent cross bracings and considering the frame to be pinned. Rigidity criteria and test results on wall specimens have been used to calibrate the area of equivalent braces. The masses were computed from the earthquake combinations (1×self-weight + 0.4×live load + 0.3×snow load) according to Romanian standards in 2004. The first three vibration modes of the structure have been simulated using FE analysis, and the corresponding periods of vibration are: T_1 = 0.38s, T_2 = 0.35s and T_3 = 0.26s.

Based on experimental results, a very simple method was applied to calculate the efficiency of shear wall panels. Due to the configuration of the structure, the analysis can be divided following the two principal directions, i.e. transversal and longitudinal ones [3.34]. In transversal direction it is considered that seismic action is resisted by the transversal shear walls, while longitudinally by the longitudinal shear walls.

Besides the structural walls, the behaviour of a house is influenced, at least in the early stage of the shaking, by the non-structural elements which usually is not accounted for in the design. The non-structural finishing elements have a structural contribution that changes the vibration modes and the damping properties of the structure. In conclusion, the main modal characteristics have been determined in three subsequent stages of construction and analysed in order to evaluate the seismic performance of the building as a whole and used to quantify the influence of the non-structural elements which are not accounted for in the modelling [3.31].

For that purpose, the dynamic properties for small amplitude vibrations of the building have been studied by direct measurements in three distinct stages of construction:

- Stage 1: only the self-weight of the steel structure has to be taken into account;
- Stage 2: the steel skeleton together with all load bearing OSB panels fixed;
- Stage 3: the completed building with all finishes, before being handed to the owner.

The measured results were better than those obtained by calculation, which means that the design procedure is safe enough. It has to be emphasised that the mounting of the stiffening OSB panels not only increased the rigidity of the structure considerably, but the direction of the weakest response has been also changed. It is interesting to observe that at this construction stage, the damping ratio has also increased considerably. At the finished stage, no important change of the vibration properties can be observed. This means that the supplementary mass introduced with the finishing is counterbalanced by the stiffness increase generated by these finishing elements. Another conclusion is that the damping ratio $\xi = 0.05$ is reasonable estimate (even if slightly un-conservative) for light-gauge steel houses.

3.3.1.4 Structural design of traditional (masonry) structure

The masonry structure has been designed following the above-described architectural plans, considering the same location of the building and the same imposed and climatic loadings. Both buildings (TWCF and masonry) were designed in such a way to achieve similar indoor environment.

For the design of the structure the following input data was considered:

- number of storeys: 2;
- structural system: confined masonry structural walls, made of hollow bricks;
- reinforced concrete slab acting as a rigid diaphragm;
- importance class (according to P100/2006): class III, normal importance ($\gamma_1 = 1.0$);
- behaviour factor: $q = 2.00$;
- characteristics of the seismic action for Ploiesti: $a_g = 0.28$g ; $T_C = 1.0$ sec.

According to the design by the Romanian codes CR6-2006 [3.35] and P100/2006 [3.27], but also from constructive considerations, the following structural elements resulted:

- concrete stanchions having cross-section of 25×25cm, reinforced with four bars having the diameter of 12mm and stirrups of 6mm diameter at 15cm. They were disposed according to the following rules: (i) free edges of each structural wall element; (ii) each intersection of walls; (iii) within the walls at a maximum distance of 5m; (iv) at both sides of any wall opening with an area of more than 1.5m^2;
- horizontal confining elements placed in the plane of the wall at every floor level (elevations of 3.00 and 6.00m), realized as concrete beams having cross-section of 25×25cm, reinforced with four bars having the diameter of 12mm and stirrups of 8mm diameter at 15cm;
- floors: designed as reinforced concrete slabs having the maximum span of 3.1m reinforced with steel wire meshes;
- for all the concrete elements, the class used was C16/20;
- bricks: mark 100 laid with M5 mortar.

3.3.1.5 Evaluation of environmental impact

The scope of the study was the integration of environmental impact into design of buildings, by means of a case-study. The example compares the environmental impact for a single-family house, designed in two different situations: the existing thin-walled cold-formed house and the same house designed in masonry solution – the so-called traditional house. The impact analyses were performed at different levels: (i) construction, (ii) construction and end-of-

life, (iii) life cycle including maintenance and, (iv) the life cycle including maintenance and consumable goods [3.23, 3.39].

The comparative life-cycle analysis was performed using the SimaPro software [3.36], a general and comprehensive tool, widely used in environmental design and LCA, which uses the Ecoinvent database [3.37]. 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.

For LCA calculation of the buildings, 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 3.12 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).

Table 3.1 presents the resulted quantities of materials for the construction stage, per square meter. The stratification of different constructive elements could be also followed in the same table.

It should be underlined the fact that the above values contain generic weights as the materials are gathered all-together. As an example in case of external walls of the traditional house the concrete resulted from the stanchions, although they are present only at each 2.5m and at corners.

Table 3.2 gives the total surfaces for the constructive elements, computed by excluding all the openings (doors, windows, technological or staircase opening etc.).

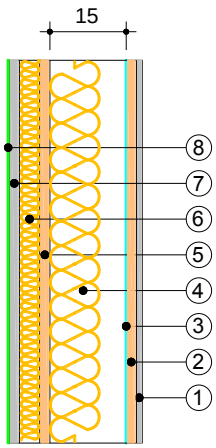
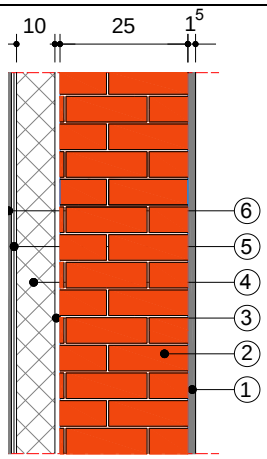
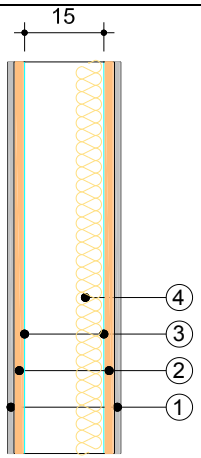
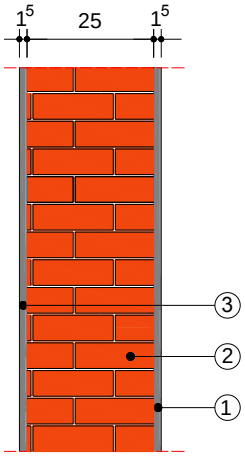
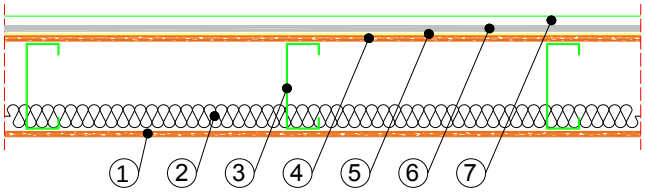
TWCF house		Traditional (masonry) house	
Exterior walls			
	1. Gypsum plaster board 12.5cm 2. Vapour barrier (foil) 2mm 3. Internal oriented strand board (OSB) 12mm 4. TWCF profile / Mineral wool 100mm 5. External oriented strand board (OSB) 12mm 6. Thermoinsulation (polystyrene extruded) 20mm 7. Polyester wire lattice (glass fibre) 8. Exterior plastering (Silicone Baumit) 1.5cm		1. Interior plastering (cement mortar) 1.5cm 2. Masonry (bricks) 25cm 3. Adhesive 4. Thermo-insulation (polystyrene extruded) 100mm 5. Polyester wire lattice (glass fibre) 6. Exterior plastering (Silicone Baumit) 1.5cm
Interior walls			
	1. Internal and external gypsum plaster board, 12.5cm 2. External and internal oriented strand board, 12mm 3. Vapour barrier (foil), 2mm 4. TWCF profile / Mineral wool, 50mm		1. Interior plastering (cement mortar) 1.5cm 2. Masonry (bricks) 25cm 3. Interior plastering (cement mortar) 1.5cm

Fig. 3.12: Layers used for structural components

Floors

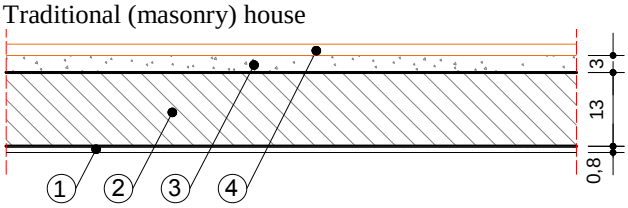
TWCF house



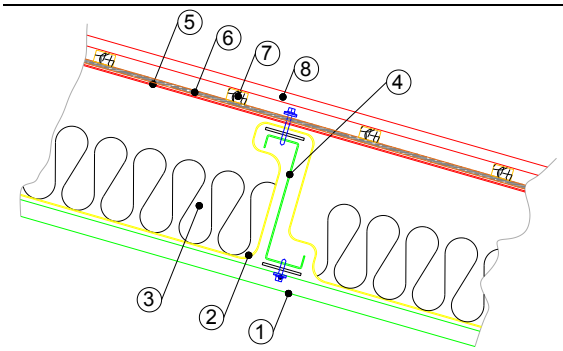
1. Interior gypsum plaster board 12.5cm
2. Thermoinsulation, mineral wool 50mm
3. TWCF profile
4. Oriented strand board (OSB) 15mm
5. Phonoinsulation foil 3mm
6. Oriented strand board (OSB) 12mm
7. Finishing

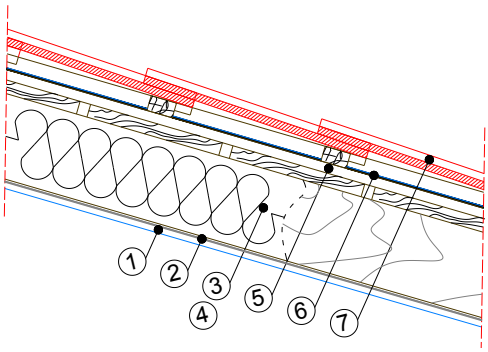
1. Interior plastering (cement mortar) 0.8cm
2. Concrete slab (concrete) 13cm
3. Concrete flooring (cement mortar) 3cm
4. Finishing

Traditional (masonry) house



Roofs

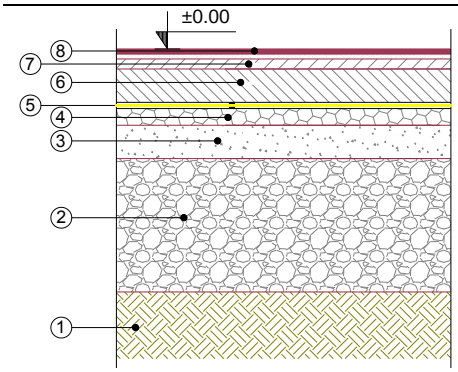




1. Inferior gypsum plaster board 12.5cm
2. Vapour barrier (foil) 2mm, anticondense barrier 2mm
3. Mineral wool 180mm
4. TWCF profile
5. Aluminium antireflex foil 3mm
6. Oriented strand board (OSB) 15mm
7. Timber framing (sawn timber)
8. Steel tiled sheet (coated steel)

1. Inferior gypsum plaster board 12.5cm
2. Vapour barrier (foil) 2mm, anticondens barrier 2mm
3. Mineral wool 180mm
4. Timber rafter
5. 6. Timber framing (sawn timber)
7. Ceramic tiles

Foundations and ground floors



1. Foundation soil
2. Compacted soil 40cm
3. Ballast 10cm
2. Thermo-insulation (polystyrene extruded) 5cm
5. Vapour barrier (foil) 2mm
6. Concrete slab 10cm
7. Concrete flooring (cement mortar) 3cm
8. Finishing

Fig. 3.12: Layers used for structural components (continued)

Table 3.1. Calculated quantities of materials for construction stage

Constructive element	Constitutive materials	Use for traditional house	Constitutive materials	Use for TWCF house
Exterior walls	Interior plastering (cement mortar) Masonry (bricks) Cement mortar Thermoinsulation (polystyrene extruded)	1.5cm (21kg) 25cm (207.5kg) 69.19kg 100mm (3.5kg)	Gypsum plaster board Vapour barrier (foil) Mineral wool 100mm Internal oriented strand board (OSB)	12.5cm (9.15kg) 2mm (0.1kg) 4.5kg 12mm (7.7kg)

	Polyester wire lattice (glass fibre) Exterior plastering (Silicone Baumit) Concrete (stanchions, lintels) Reinforcing	$1m^2$ (0.16kg) 1.5cm (4.2kg) 66.54kg 8.14kg	External oriented strand board (OSB) Thermoinsulation (polystyrene extruded) Polyester wire lattice (glass fibre) Exterior plastering (Silicone Baumit) Steel cold-formed profile	12mm (7.7kg) 20mm (0.7kg) $1m^2$ (0.16kg) 1.5cm (4.2kg) 16.32kg
Interior walls	Masonry (bricks) Interior plastering (cement mortar) Cement mortar Concrete (stanchions, lintels) Reinforcing	25cm (207.5kg) 1.5cmx2 (42kg) 69.19kg 66.54kg 8.14kg	Internal and external gypsum plaster board Vapour barrier (foil) Mineral wool External and internal oriented strand board Steel cold-formed profile	12.5cm x 2 (18.3kg) 2mmx2 (0.20kg) 50mm (2.25kg) 12mm x2 (15.36kg) 18.33 kg
Floor	Concrete flooring (cement mortar) Concrete slab (concrete) Reinforcement (steel) Interior plastering (cement mortar)	3cm (66kg) $1m^2$ (312kg) 16.8kg 0.8cm (11.2kg)	Oriented strand board (OSB) Phonoinsulation foil 3mm Thermoinsulation Mineral wool Inferior gypsum plaster board Oriented strand board (OSB) Steel cold-formed profile	12mm (7.68kg) 0.1kg 50mm (2.25kg) 12.5cm (9.15kg) 15mm (9.6kg) 13.61kg
Roof system	Timber framing (sawn timber) Ceramic tiles Mineral wool Vapour barrier (foil) Anticondens barrier Inferior gypsum plaster board	32.7kg 43kg 180mm (8.1kg) 2mm (0.1kg) 2mm (0.135kg) 12.5cm (9.15kg)	Steel tiled sheet (coated steel) Steel cold-formed profile Timber framing (sawn timber) Anticondens barrier Oriented strand board (OSB) Mineral wool Aluminium antireflex foil Vapour barrier (foil) Inferior gypsum plaster board	5kg 19.09kg 5.16kg 2mm (0.135kg) 15mm (9.6kg) 180mm (8.1kg) 3mm (0.1kg) 2mm (0.1kg) 12.5cm (9.15kg)
Foundation / Infrastructure	Ballast – compacted Concrete Reinforcement Thermoinsulation (polystyrene extruded)	10cm (540kg) 1752kg 19.12kg 50mm (1.75kg)	Ballast – compacted Concrete Reinforcement Thermoinsulation (polystyrene extruded)	10cm (320kg) 1078kg 13kg 50mm (1.75kg)

Table 3.2. Computed surfaces for different constructive elements (m^2)

Constructive element	Traditional house	Metallic house
Exterior walls	216.25	215.10
Interior walls	134.27	126.11
Floors	86.13	80.60
Roof system	100.11	92.54
Foundation	95.36	91.28

Boundary conditions

In order to set the input elements (inventory analysis), 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 (water/gas/electricity use) of the two buildings is considered to be similar, as the buildings were designed in such a way to achieve similar indoor environment;
- 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 for the Construction Stage

In order to have an easier input of construction materials in LCA tool used (SimaPro 7), 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 used for SimaPro.

Figure 3.13 and Figure 3.14 present the contribution flow of processes for the construction stage to the environmental impact in the form of process trees for TWCF and masonry structural systems respectively.

Figures 3.15-3.17 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 the previous paragraph. All the results are given in “Eco-indicator points” (Pt) [3.38], 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 3.15 presents the impact for the traditional house for the construction process ranked per constructive elements. The major impact corresponds to exterior walls and foundations. Both constructive elements are high consumers of resources but also have a great impact of human health (each with more than 900 points). However, not the same observations could be said about roofing system, for which the major environmental impact is on eco-system quality, mainly due to high quantity of wood.

Analysing the TWCF house in the same manner (see Figure 3.16), the results ranked per constructive elements show that also the same constructive elements (external walls and the foundations) have the lead on the environmental impact.

In a direct comparative impact analysis, for traditional house, higher impact values for most of the impact indicators (categories) result, as could be seen in Figure 3.17. One could realise that for both structures the major impact is for 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, climate change substances and land use. 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 2 times greater for traditional house).

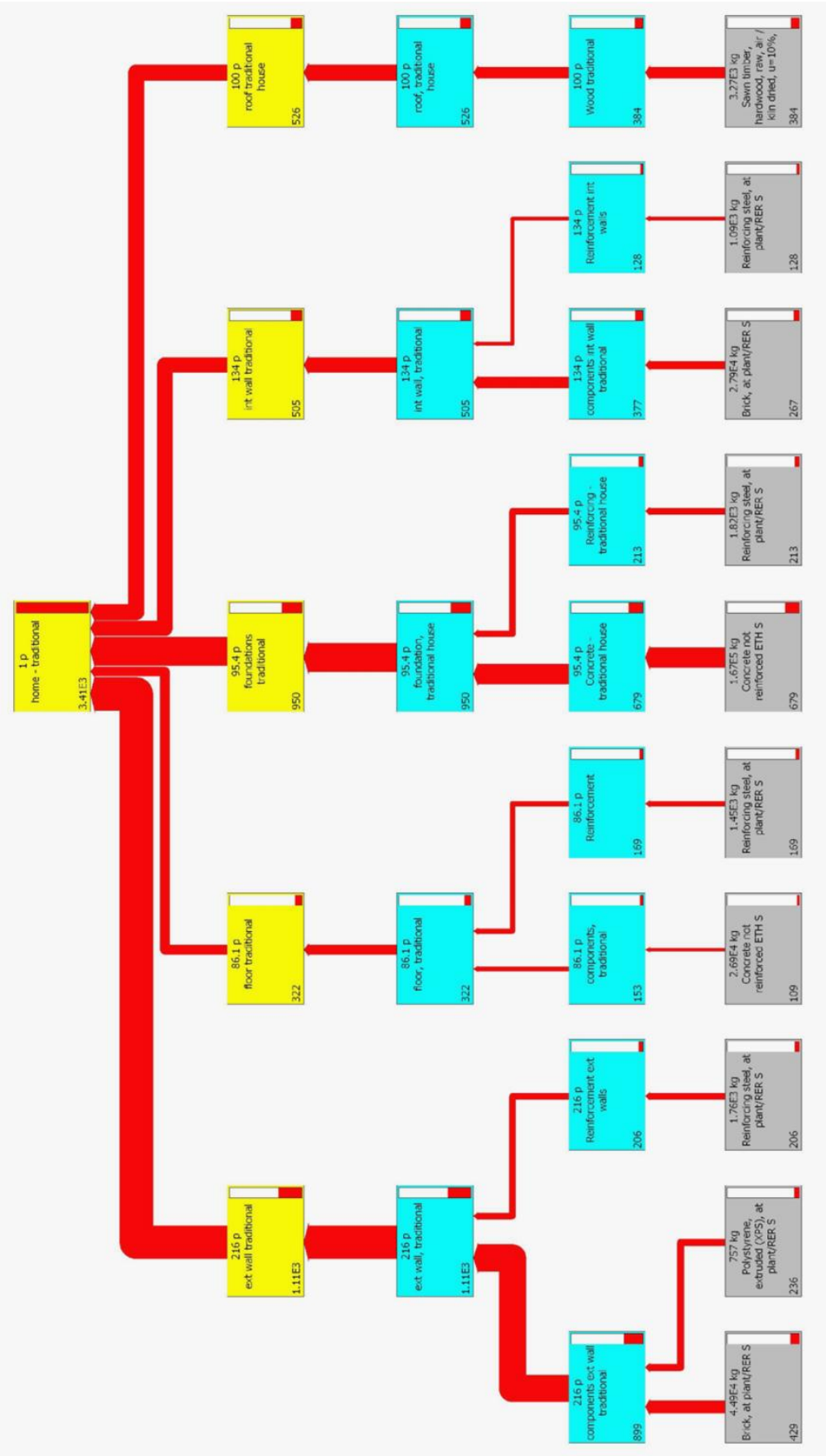


Fig. 3.13: Process tree for construction stage of the traditional house

The results presented in Figure 3.17, on impact (or damage) categories, are aggregated into a single score (inner figure), leading to an overall score of 1626 points for the metallic house, less than half of the global score of the traditional house (3409 points).

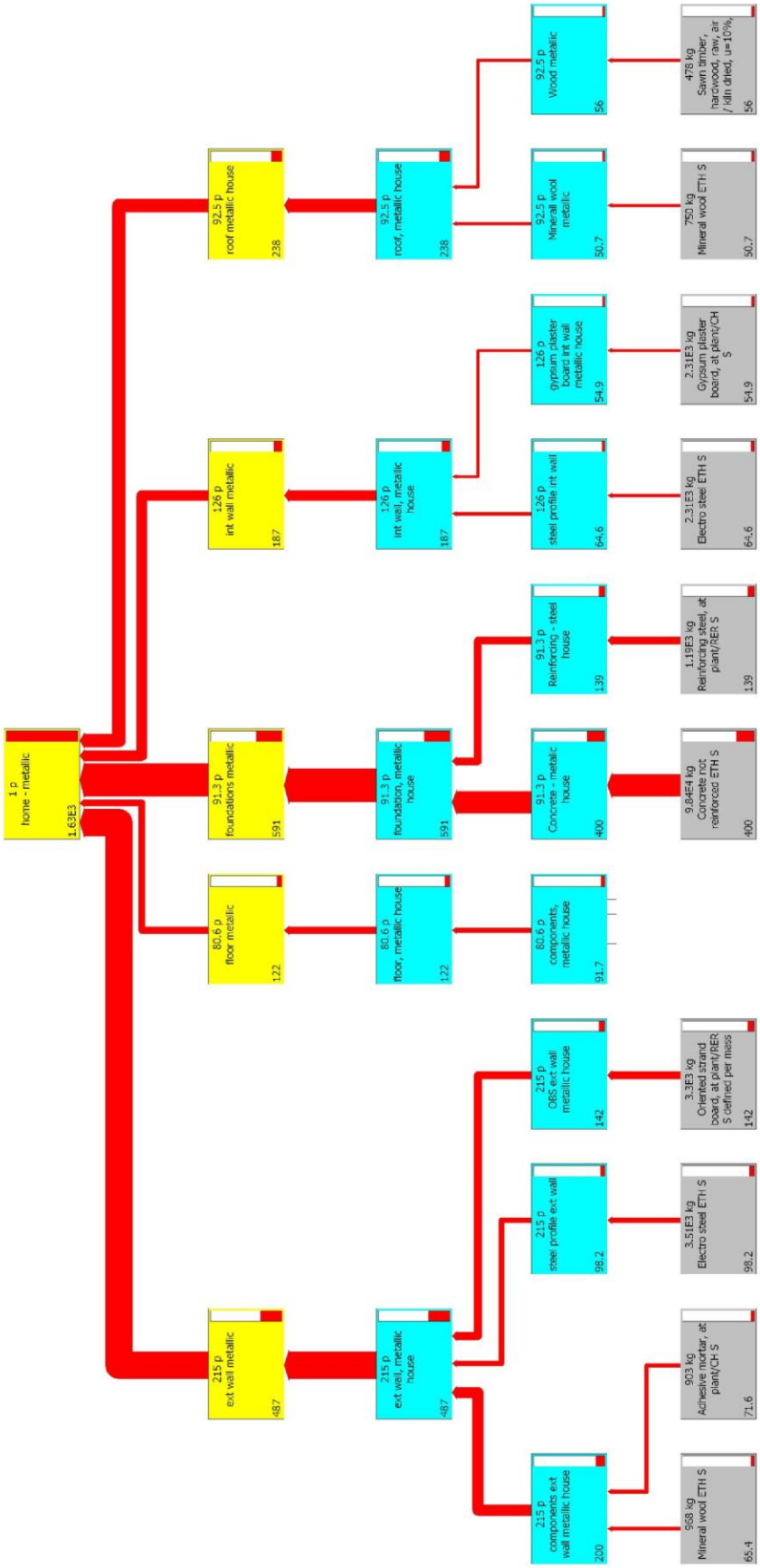


Fig. 3.14: Process tree for construction stage of the metallic house

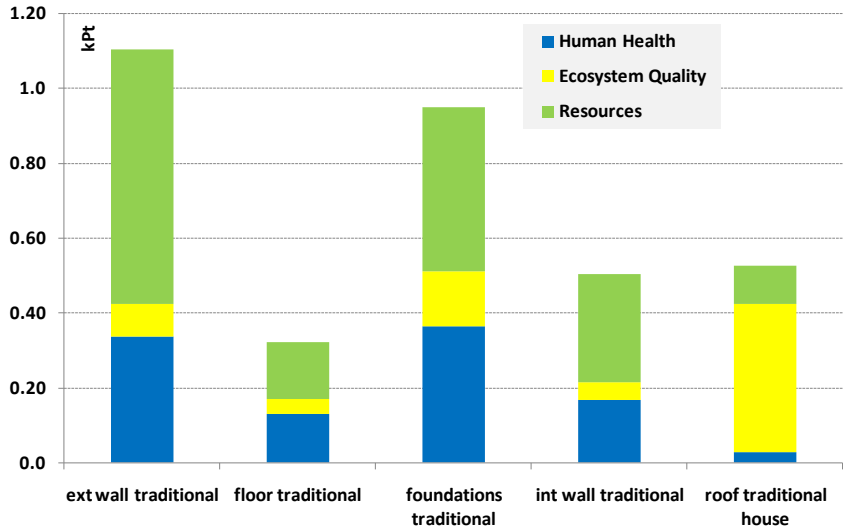


Fig. 3.15: Environmental impact per constructive element for traditional house – construction phase

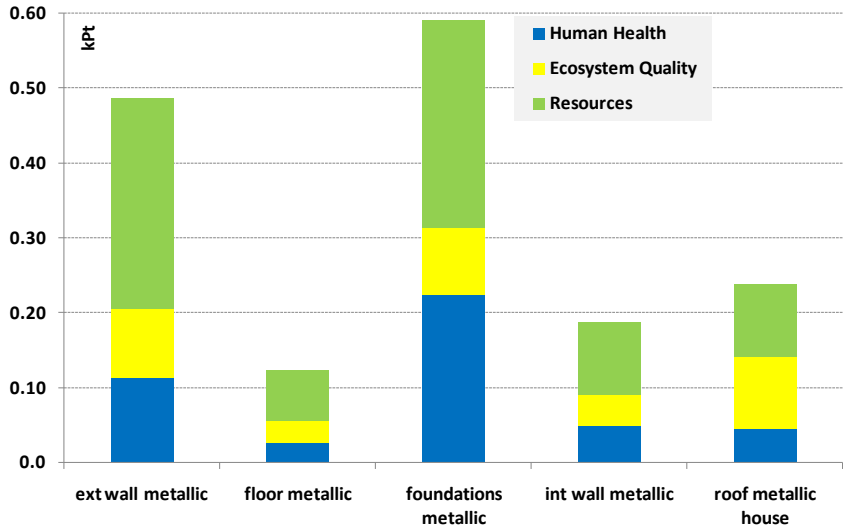


Fig. 3.16: Environmental impact per constructive element for metallic house – construction phase

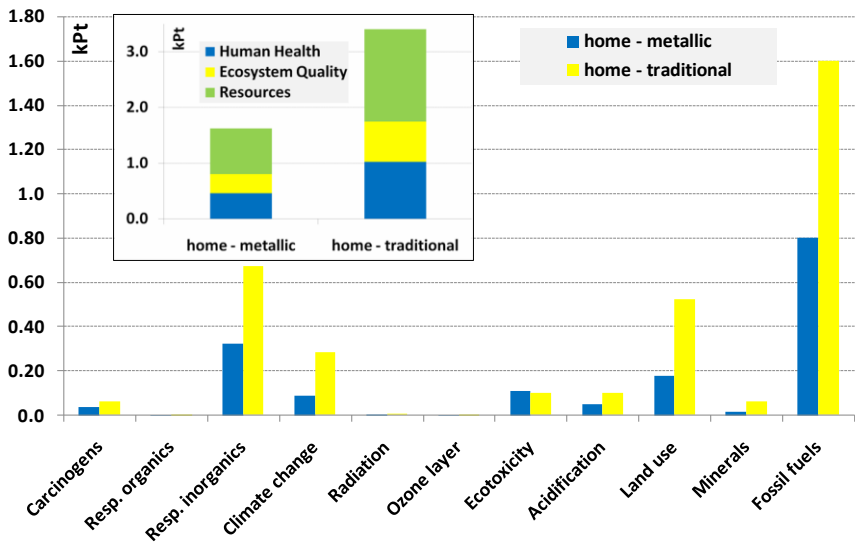


Fig. 3.17: Comparison on environmental impact for metallic and traditional houses (weighting)

Environmental Impact Considering the Construction Stage and End-of-Life (Recycle, Reuse and Disposal)

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 (low-cycling - 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 summarised in Table 3.3.

Table 3.3. 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	---
OSB	50	---	50	---
Gypsum plaster boards	30	---	---	70
Ballast	70	---	---	30
Concrete, mortar	---	---	---	100
Other inert materials	---	---	---	100
Other combustible materials	---	---	100	---

In these conditions, ranked in a single score (see Figure 3.18) it results that in the life-cycle comparison for the construction and disposal scenarios, the gap between the two situations is greater than in the case of construction process only (comparison of Figure 3.15 and Figure 3.16). As could be noticed, for both structural systems, the end-of-life scenario brings a negative impact (global scores being now 1779 points for TWCF and 3986 points for masonry one respectively) although some benefits are obtained by the reuse of certain materials (wood, ballast, OSB and gypsum plaster boards).

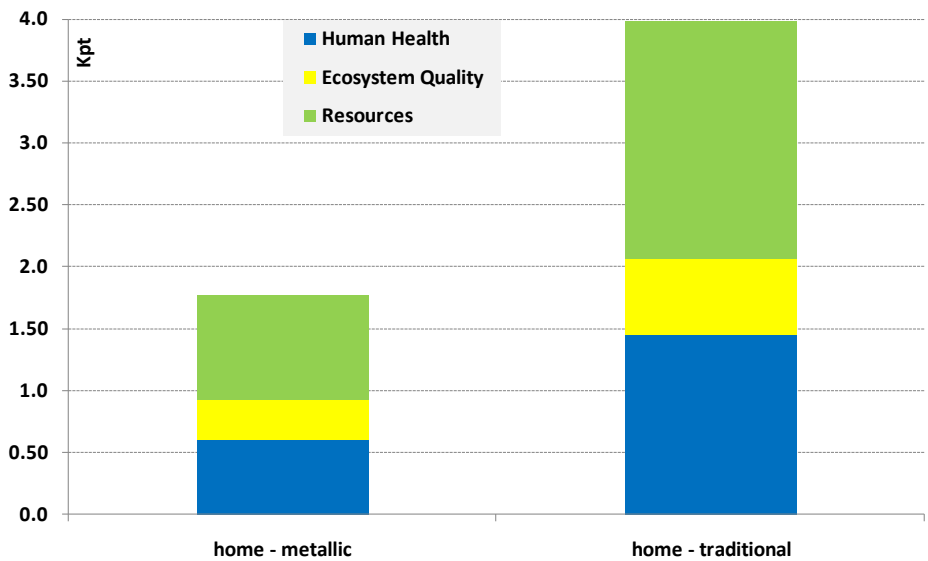


Fig. 3.18: Comparison on environmental impact for metallic and traditional house (global score) - construction phase and end-of life

Integration of the Maintenance of Buildings

Regardless the chosen constructive system, a building needs maintenance works during its life-cycle. These works could be of different types and, function of this type, could be more or less expensive and they 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 and adjacent internal plaster-board (once at 12.5 years);
- one change of the electric and heating system (once at 25 years);
- one change of the steel tiled sheeting (once at 25 years);
- one change of the thermo-system for interior walls, including OSB and plaster-board panels, on one face of the wall (once at 25 years);
- one change of the thermo-system and external OSB panels for external walls (once at 25 years).

Important to notice is the fact that, in case of steel structure, only the steel skeleton and some OSB and plaster-board panels remain unchanged, while all other elements are changed once in 50 years. Of course this is the worst scenario and, different scenarios can be proposed too. 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 3.3).

Figure 3.19 presents the impact deduced only for maintenance process. As the quantities introduced for analyses are smaller than that from the building process, the global scores are also smaller. In terms of impact categories, the same indicators (fossil fuels, respiratory inorganic substances and land use) integrate more than 85% of the global scores. In case of maintenance for steel house, the eco-toxicity is also a mentionable category to global score.

All the values resulted only from maintenance and cumulated in a single score (see Figure 3.20) will conduct to about 670 eco-points in the case of TWCF house and respectively 615 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 TWCF house. The explanation for this derives from the facts that for the steel house one side of walls, floor and roof layers are replaced while in the case of traditional house, all the brick, concrete, wall plastering and main wood frame remain in the original form.

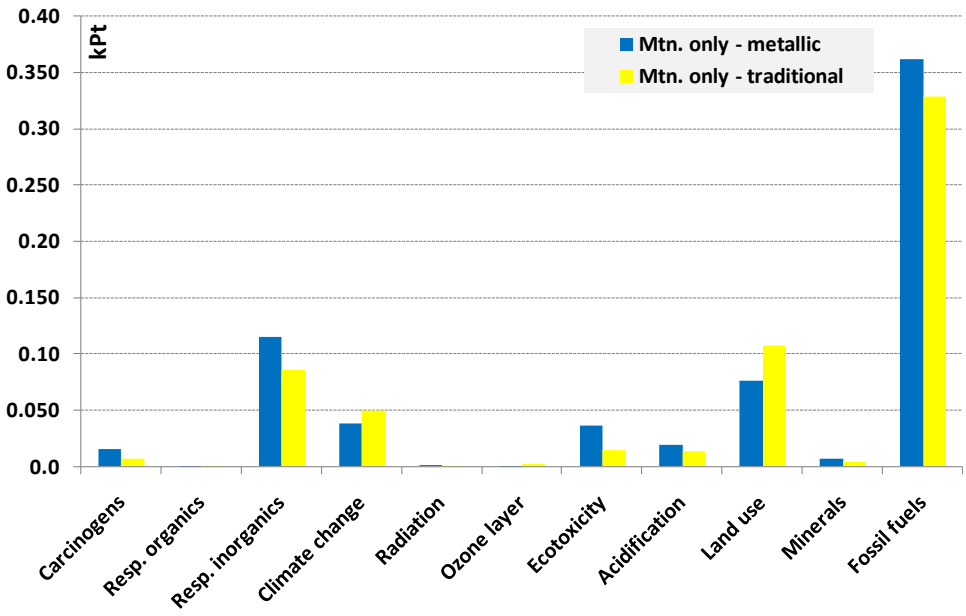


Fig. 3.19: Comparison on environmental impact for metallic and classic house - maintenance only

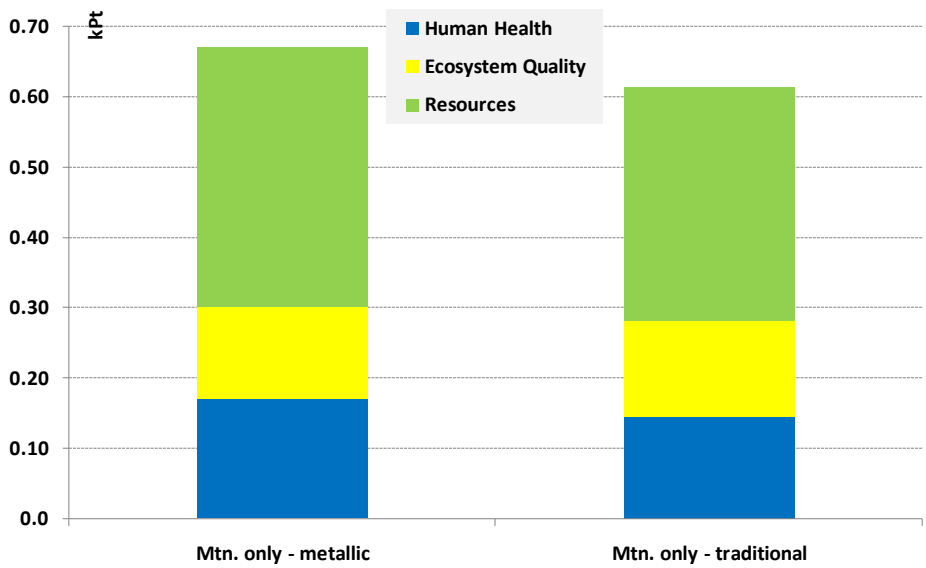


Fig. 3.20: Comparison on environmental impact for metallic and classic house – maintenance only (global score)

Figure 3.21 and Figure 3.22 present the above – described stages from 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 3.21):

(i) *fossil fuels, respiratory inorganic substances and climate change*: mainly due to the manufacturing processes which require large quantities of energy and 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.

In a single score analysis (see Figure 3.22), and taking into account the boundary conditions as explained before, the metallic house (2450 eco-points) presents an important advantage in front of traditional one (4600 eco-points). Of course, many parameters (such as national or regional peculiarities, climatic zones or distance from the material distributors) may affect these results.

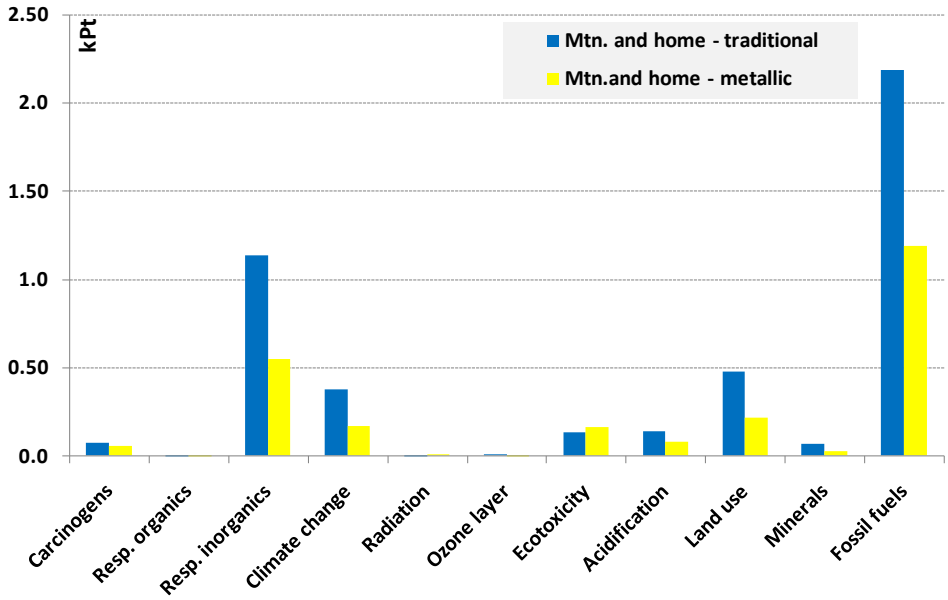


Fig. 3.21: Life-cycle comparison on environmental impact for metallic and traditional house (weighting) for construction, including maintenance and end-of-life

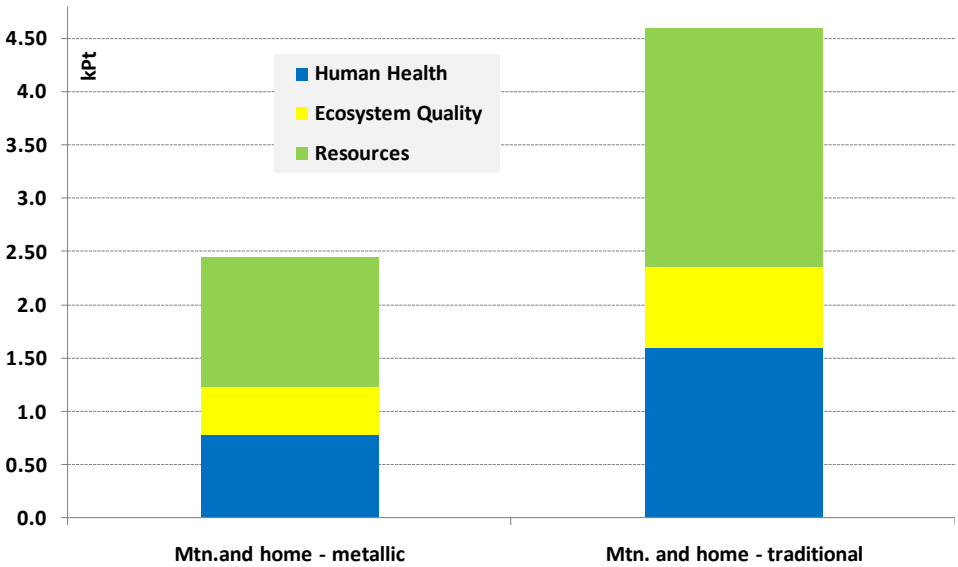


Fig. 3.22: Life-cycle comparison on environmental impact for metallic and traditional house (weighting) for construction, including maintenance and end-of-life

3.3.1.6 Life-cycle including consumable goods

In order to complete the life-cycle assessment of the house, the consumable goods (electricity, water etc.) should be estimated. In our case, the estimation for these goods was made for a standard life-time period of 50 years.

The energy considered in this study is used for heating, cooling processes, domestic use of electrical devices, lightning and so on. The values taken into consideration in our comparison are based on mean computation for power consumption per capita or per surface, while in the case of heating/cooling there have been considered the particular characteristics of each structural system.

Taking into account that both systems were designed in order to accomplish the same interior thermal and comfort conditions and are conceived for same use in electric power and water, they lead practically to the same need of consumable goods.

Three main components were considered as consumable goods, as follow [3.23, 3.40]:

- water, used for domestic purposes, as cold and hot. The hot water is prepared at home;
- electricity, used for lighting, electrical devices, cooling system;
- natural gas, employed in heating the house during cold months and for hot water preparation.

Calculation of consumable goods. Estimation of the annual natural gas requirement

As already mentioned, the natural gas was considered the source used for heating and hot water preparation. For this purpose, first of all, it is necessary an estimation of the quantity of heat required during one year. This was performed according to the Romanian standards SR1907-1:1997 [3.41] and SR1907-2:1997 [3.42], as function of the interior and exterior temperatures and construction elements.

Practically, for each distinct component delimiting the house rooms (walls – including windows and doors, ceiling and floors), there have been computed the resistance to thermal transfer. Then, as function of outside temperature (monthly mean) and the required inside temperature, the required heat quantity was computed.

The resistance to thermal transfer was computed by the following equation:

$$Q = Q_T[1 + (A_0 + A_c)/100] + Q_i \tag{3.1}$$

where:

Q_T represents the thermal flux through the delimiting elements (walls, windows and doors in steady regime);

Q_i is the required heat for warming the air flow, cooled due to doors and windows opening, from exterior to interior temperature;

A_o North-South orientation addition;

A_c addition for cold-surfaces compensation.

The thermal flux through the delimiting elements was computed by:

$$Q_T = A \cdot (m/R) \cdot \Delta t \cdot C_M + Q_p \tag{3.2}$$

in which:

A represents the area of the delimiting element;

R is the element resistance to thermal transfer;

C_M is a coefficient given function of the relative mass of the building, taken as 0.94;

m the thermal massivity of material (computed in function of the thermal inertial index);

Δt represents the interior-exterior thermal difference;

Q_p is the heat transmitted through the floor.

For our study, the thermal resistance was computed for each type of delimiting component. Table 3.4 gives, as an example, the way of calculation of the thermal resistance for the external wall of the traditional house. From the wall materials, there have been considered those having

an important conductivity (mortar, polystyrene and masonry bricks). The stratification of this wall has been presented previously (see Figure 3.30).

Table 3.4. Computation of thermal resistance (example for an exterior wall)

Exterior wall (25 cm)							
Name	*Thermal conductivity*	*Thermal assimilation*	*Element width*	*Thermal resistance*	*Total thermal resistance*	*Thermal inertial index*	*Thermal massivity*
	l (W/mk)	s (W/m^{2}k)	d (m)	RO (m^{2}k/W)	R=RI+RE+RO (m^{2}k/W)	D	m
Cement mortar	0.9330	10.08	0.03	0.0323	3.0626	0.3252	
Hollow brick	0.330	7.64	0.25	0.7576	0.0420	5.7879	
Polystyrene	0.044	0.30	0.10	2.2727	0.1250	0.6818	
				3.0626	3.2296	6.7949	1.00
R = 3.2296							

Table 3.5 summarises the thermal resistances of all elements/components, computed as above, function of the system stratification. The thermal insulation resistance of main thermo-resistant elements (external walls and roofing system) shows similar values, fact that allows the assumption of using the same amount of heat for both systems (assumption proved by actual calculation).

Table 3.5. Thermal resistances per elements for traditional vs. metallic house

No.	*Element name*	*Thermal resistance for traditional house (m^{2}k/W)*	*Thermal resistance for metallic house (m^{2}k/W)*
1	Exterior wall	3.2296	3.1187
2	Interior wall	0.9461	1.4185
3	Floor on ground	2.7724	2.7724
4	Floor over 1st storey	0.6462	1.6911
5	Floor over 2nd storey	4.5464	4.5464

The heat requirement has been computed taking into account three assumptions: (i) in the houses are living 3 persons; (ii) the external mean temperatures were considered for the city of Ploiesti, location in which the structure was build and (iii) the interior ambient temperature was considered 20°C. On these presumptions, it results a daily requirement for heating. Table 3.6 presents, as an example, the heating requirement for one day in January, for the first floor. The total heat requirements, for both houses, are given in Table 3.7, in watts.

Table 3.6. Heat requirement for first floor in January example of computation

Wall, floor name	*Orientation*	*Width*	*Height*	*Surface*	*To subtract*	*In calculus*	*R''*	*m*	*Δt*	*Q''*
		[m]	[m]	[m^2]	[m^2]	[m^2]	[m^{2}k/W]		[°C]	[W]
FP1	1	11.05	2.7	29.84	13	16.9	3.2296	1.034	22	164.26
FP2	2	11.05	2.7	29.835	3.51	26.3	3.2296	1.034	22	182.47
LE3	3	9.45	2.7	25.52	9.54	16	3.2296	1.034	22	110.47
LV4	4	9.45	2.7	25.52	6.26	19.3	3.2296	1.034	22	133.19
G+U				32.27	0	32.3	0.667	1.000	22	1,044.86
F				90.46	0	90.5	2.7724	1.069	16	557.94
PP				90.46	0	90.5	0.6462	1.133	0	-
TOTAL										**2,145.21**

Table 3.7. Heat requirement (watts) for one year

Month	*No. days.*	*Traditional house*		*Metallic House*	
		Heat req./ day	*Heat req./ month*	*Heat req./ day*	*Heat req./ month*
January	31	3,996.501	2,973,396.74	4,070.78	3,028,658.71
February	29	3,434.891	2,390,683.99	3,502.78	2,437,933.13
March	31	2,659.99	1,979,032.77	2,710.63	2,016,708.07
April	30	1,636.914	1,178,577.92	1,670.66	1,202,878.70
October	31	1,636.914	1,217,863.86	1,670.66	1,242,974.66
November	30	3,229.963	1,915,193.00	2,710.63	1,951,652.97
December	31	3,585.165	2,667,362.80	3,644.35	2,711,392.96
TOTAL (Q_{inc})			**14,322,111.08**	**TOTAL (Q_{inc})**	**14,592,199.21**

The required energy for heating water was computed in accordance to the following formula:

$$Q_{ac} = m \cdot c_p \cdot \rho \cdot (t_{ac} - t_{ar}) \text{ [kJ]} \tag{3.3}$$

where:
m is the water flow rate in kg/hour;
c_p the thermal capacity of water;
ρ the water density (1000 kg/m^3);
$t_{a.c.}$-$t_{a.r.}$ the hot (45ºC in mean) and cold (10ºC in mean) water temperature.

The required volume of hot water per day was fixed at 110 l for each inhabitant, leading to a total amount of 330 l/house/day.

Resulted total heat requirement for a house is then given by:

$$Q_{tot} = Q_{ac} + Q_{inc} \tag{3.4}$$

where:
Q_{inc} is the required energy for heating.

The total heat requirement Q_{inc} computed for the traditional house is 14322 kilowatt and for the metallic house is 14592 kilowatt; the difference being less than 2%, further on, they would be considered as being the same value (14322 kilowatt).

The gas requirement is then computed by:

$$M = Q_{tot} / q_{gas} \tag{3.5}$$

where, q_{gas} is the thermal power of gas taken as 8.5 mc/kW.

Using equation (3.4), it results the following values for the total requirements of energy and its translation in gas volume burnt in domestic gas heating power station (see Table 3.8).

Table 3.8. Annual heat requirement and gas equivalent

Q_{inc} [kilowatt]	14322
Q_{ac} [kilowatt]	4902
Q_{tot} [kilowatt]	19224
M [m^3]	**2262**

Estimation of the annual domestic electrical power

In order to compute the annual domestic power consumed by the inhabitants, it was considered the following configuration of the electric board (see Table 3.9):

Table 3.9. Electric board scenario

No.	Circuit No.	Consumer denomination	P_i [kW]	P_f [kW]	K_u	cos. Φ	tg. Φ	CHARGE		
								Active	Reactive	Apparent
								P_a	Q_{nec}	S
								[kW]	[kVar]	[kVA]
1	TD1.1	1st floor lightning	1.00	1.00	0.60	0.80	0.75	0.6	0.19	0.8
2	TD1.2	2nd floor lightning	1.00	1.00	0.60	0.80	0.75	0.6	0.19	0.8
3	TD1.3	Sockets – 1st floor	2.50	2.50	0.60	0.80	0.75	1.5	0.49	1.9
4	TD1.4	Sockets – 2nd floor	2.50	2.50	0.60	0.80	0.75	1.5	0.49	1.9
5	TD1.5	AC Split	3.00	3.00	0.60	0.80	0.75	1.8	0.58	2.3
6	TD1.6	Washing machine	1.50	1.50	0.60	0.80	0.75	0.9	0.29	1.1
TOTAL			**8.50**	**11.50**				**6.9**	**2.2**	**8.6**

Taking into account the temperature fluctuation during the summer months, it was considered a monthly consumption of 200 kWh for summer months respectively 120 kWh for cold months. The difference is due to the use of Air Conditioning Split.

Consequently, the annual electrical power consumption is:

$$W = 6 \times 120 + 6 \times 200 = 1920 kWh \tag{3.6}$$

resulting for the entire life of house of 50 years, a value of 96000 kWh.

Estimation of the annual water used

Considering the house inhabited in mean by 3 persons, the cold and hot water requirement was considered as 20 m^3 per month (the warming of the hot water was integrated into the annual gas estimation). These values resulted also by considering the national values. The annual water consumption results as:

$$C = 12 \times 20 = 240 m^3 \text{ per year (12000 m}^3\text{ for the entire life of the house)} \tag{3.7}$$

Life-cycle analysis including energy scenario

According to the calculation presented in the previous paragraph, the following quantities of consumable goods – computed for a lifetime period of 50 years – have to be introduced supplementary in the LCA analyses (see Table 3.10). No disposal scenario was considered for the consumables.

Table 3.10. Consumable goods used for LCA

Consumable	*Quantity*
Water [m^3]	12000
Gas [m^3]	113100
Electricity [kWh]	96000

Figure 3.23 presents the impact on environment for these consumable goods. In terms of impact categories, the highest impact is taken by fossil fuels, fact that is normal in regard to the elements considered (gas, electricity and water). The other impact categories are negligible in

regard to fossil fuels category. It has to be noted that in the energy scenario was not included any solution of using renewable energy, as these could diminish the fossil fuel requirement.

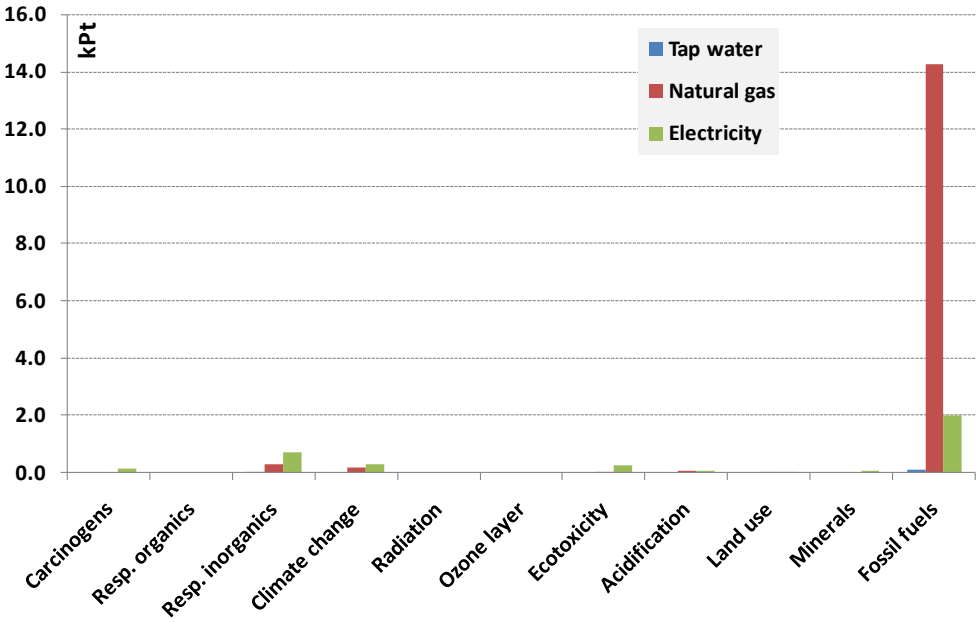


Fig. 3.23: Environmental impact (weighting) – consumable goods only

Observing the share of impact categories for different consumables considered: gas, electricity and water, it could be drawn very easily the conclusion that the gas affects largely the fossil fuel category.

Comparing now the environmental impacts produced by consumable goods relative to the building process including maintenance (see Figure 3.24), it could be easily observed that, by far, the consumable goods take the highest impact (more than 7.5 times in case of metallic house and 4 times more in case of traditional house). These results are both considered for the entire life-time period of a house (50 years). It can be seen that the difference arises due to the use of fossil fuels affecting the resources category and not due to human health damage substances or eco-system quality. As a main comment, it should be said that since the energy consumption for heating take the lead in the consumable goods impact and depends on the thermal resistance of envelope elements, these ratios could be diminished by better insulation.

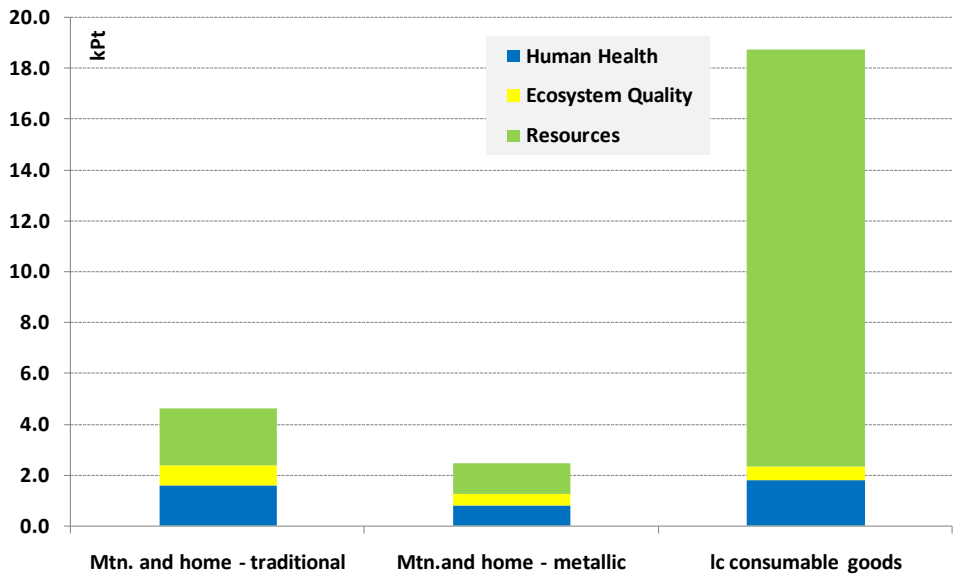


Fig. 3.24: Single score comparison of environmental impact for building process and consumable goods

In a direct conclusive comparison, by considering the building process and consumable goods, according to the analysis performed in the conditions described herein, the total global impact score for a life-time period of 50 years is by 10% smaller in case of the metallic structure (see Figure 3.25).

In conclusion it could be stated that a metallic house could represent a good alternative to the usual traditional Romanian house in terms of environmental impact.

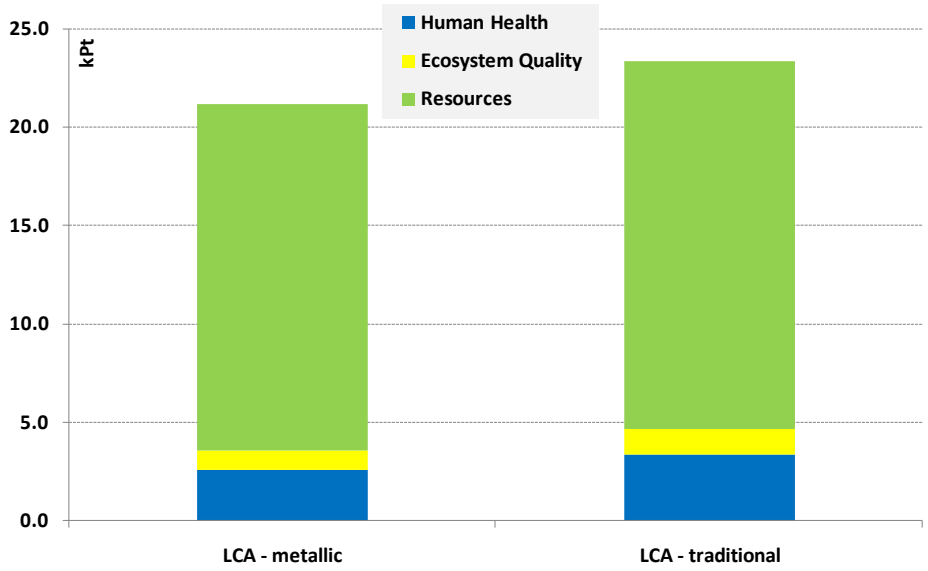


Fig. 3.25: LCA comparison of environmental impact (single score) for metallic and traditional houses for building process and consumables

3.3.1.7 Concluding remarks for case study 1

A modern design of buildings should integrate aspects related to their sustainability in addition to the economic, safety and functionality requirements. The study presents the practical aspects related to the environmental impact analysis of an existing dwelling made with light-steel structural elements. In order to have a base of comparison of the investigated solution, a comparison with a traditional solution was done, for this purpose a similar masonry house being designed.

Within the purpose of the study, a series of boundary conditions have been set. In order to have a good overview of the LCIA analyses, the results are presented in different stages:

- by considering the constructions process only;
- construction process integrating the end-of-life scenario;
- construction process, end-of-life and maintenance.

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;
- the results presented show very clear that the major impact on environment is not due to the building itself (including maintenance), but to the energy consumption during its

use. Furthermore, impact category that takes the major impact is represented by the use of the fossil fuels;

- 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.

The constructive system used for TWCF steel house, has been proved to be an efficient alternative not only for the building process, but also for the consumable goods used during the life-time.

3.3.2 Case study 2 – Sustainable steel framing based affordable house solution

In case of affordable houses, the basic preoccupation of designers is to find a good relation between cost and comfort. A supplementary parameter could be in some cases the erection time. That is why the architects and engineers try to find solutions which rely on usual building materials but applying innovative systems [3.43, 3.44].

In this context, an innovative structure-envelope solution is proposed by:

- application of industrial building technologies in dwelling building systems (residential application) providing a fast erection and fabrication time. The basic assumption is that an affordable house should rely on standard details and common technologies, available to most of the builders, instead of experimenting with new materials without track record;
- developing of a modular systems in such a way that at any time the owner can add a new module, both by vertical and/or horizontal addition, with high solution diversity for flooring and envelope;
- designing, in terms of structural performance, both the walls and the floors based on stress skin design. It is well known that Oriented Strand Board (OSB) panels used for walls and profiled steel sheeting used as floor decking are very effective as shear diaphragms. Provided it is positively attached to the secondary members and main frames by mechanical fasteners or welding, it is extremely reliable and predictable and may be confidently used as a structural component;
- using structural systems made of lightweight steel frames, hot-rolled profiles or wood framing assures the lightness of the house and the proper response to climatic and seismic loading.

All these improvements should be judged in comparison to the classic building system built in masonry and concrete, which represents the traditional house in Romania.

In a modern design approach the environmental impact should also be integrated in the development process. Only in this way the new systems could be in line with the approach of sustainable development of construction works. On the following, architectural and technical main features of the proposed solution are displayed and structural performance and sustainability scoring are presented.

3.3.2.1 Description of structural system [3.43, 3.44]

The architectural concept relies on the development of a rectangular footprint of 5.60m×13.40m, which gives a first module of 75m^2, for one level unit. The dwelling is a two

storey building, with terrace roof, having a gross built area of 150m^2 and a usable area of 124.41m^2. Figure 3.26 presents the 3D view of the house, while Figure 3.27 presents the ground and upper floor plans. A further development of the house is possible by adding a new module by horizontal addition, extending in this way the living area.

The proposed construction system, as shown in Figure 3.26, consists of: (1) hot–rolled framed steel structure; (2) secondary structure – cold–formed steel studs (or wood studs); (3) various envelope systems; (4) floor structure – light concrete topping on trapezoidal steel deck; (5) double glazed loggias with PVC or aluminium frames; (6) foundations and slab / cast in place reinforced concrete; (7) terrace roof or pitched roof.

The achievement of thermo-energetic efficiency was another goal set by the design team. Several factors were taken into consideration: (1) Indoor temperature and air quality; (2) Thermal insulation; (3) Moisture protection; (3) Different heating and cooling systems; (4) Passive ventilation and shading; (5) The glazed terraces act as a buffer zone; (6) Skylights will be used to enhance cross ventilation for the one level house.

(a)

(b)

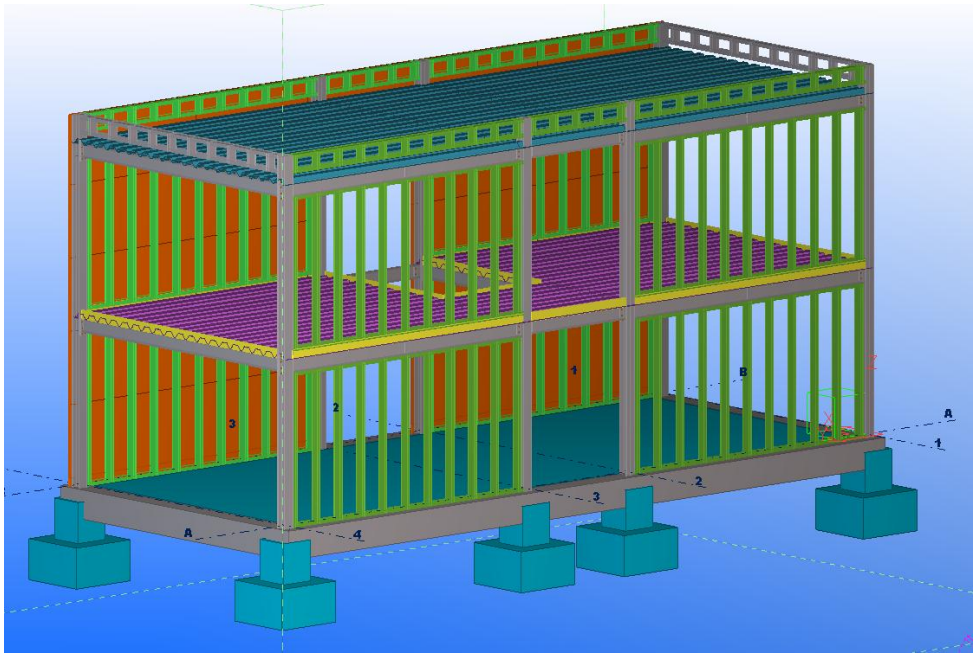


Fig. 3.26: 3D view of the house: (a) architectural layout; (b) structural layout

Fig. 3.27: (a) The ground floor and (b) the upper floor

3.3.2.2 Structural design

The structure is designed considering the climatic and seismic conditions of the city of Timisoara, Romania. The loads used for the design of the structure are taken according to Romanian codes, actually aligned to relevant Eurocodes.

Main structural skeleton is made by hot-rolled profile (European profiles HEA for columns, IPE profiles for beams). The main steel structure is a two storey portal frame structure on transversal direction, while longitudinally are used shear walls to resist horizontal forces from wind and earthquake. Columns are made by hot-rolled profiles and are rigid at the base on transversal direction and pinned on longitudinal direction. The transversal girders are made by hot-rolled profiles of semi-rigid beam-to-column connections while the longitudinal girders are pin-ended. To take over the horizontal longitudinal loads, shear walls made by cold-formed steel studs sheeted with 10mm OSB on both sides are used.

The design of structural elements (columns, beams) was made according with EN1993-1-1 [3.24], EN1993-1-3 [3.25], EN1993-1-8 [3.26] and Romanian seismic design code P100-1/2006 [3.27], both for ULS and SLS. To control the floor vibrations, a deflection of L/400 was considered reasonable for a family house.

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 in residential buildings, tested in the Laboratory of Steel Structures of the “Politehnica” University of Timisoara, Romania [3.28, 3.29], have been used as reference values for checking the available shear strength of walls.

The experimental program was based on six series of full-scale wall tests, of 3600mm × 2440mm, with different exterior cladding arrangements, i.e. corrugated sheet, OSB, bracings and no cladding - straps on both sides of the frame. Each series consisted of identical wall panels, tested under both monotonic and cyclic lateral loads. The shear-resistance of cold-formed steel wall stud panel specimens sheeted with 10mm OSB under the cyclic loads was found of 69844 N. For a 3.6m panel length this leads to a value of 19.40 kN/m.

Based on experimental results, a very simple method can be applied to calculate the efficiency of shear wall panels. Due to the fact that the structure is very regular, with planes of symmetry on both axes and distinct load path, the analysis can be divided following the two principal directions, i.e. transversal and longitudinal ones. In transversal direction the seismic action is resisted by the portal frames, while longitudinally by the shear walls (see Figure 3.28).

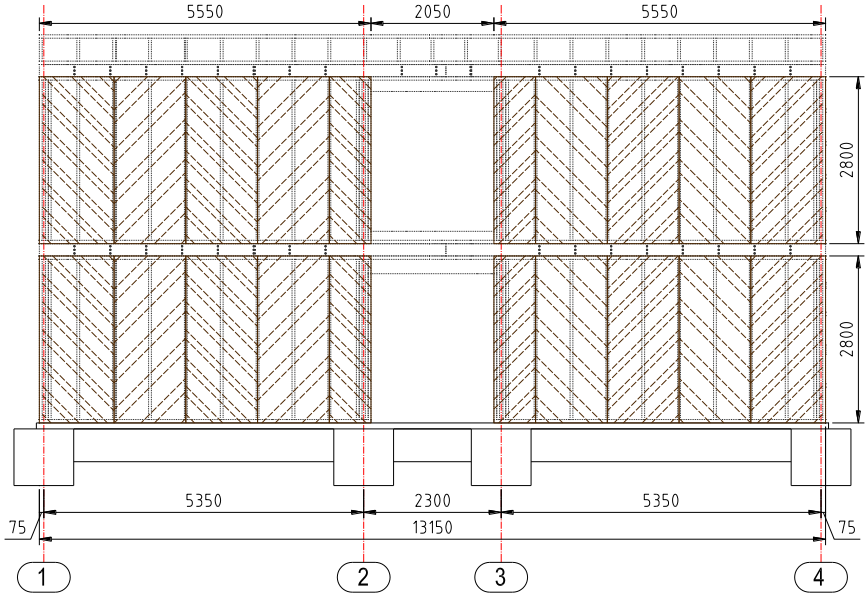


Fig. 3.28: The shear walls on longitudinal direction

In a first step, the total mass of the structure is evaluated, at the level of the first floor and at the roof level, taken into account the loads and safety coefficients, considered in the special combination [3.45]:

$$\sum_{j=1}^{n} G_{k,j} + \gamma_I A_{E,k} + \sum_{i=2}^{m} \psi_{2,i} Q_{k,i} \tag{3.8}$$

where:
$A_{E,k}$ – Characteristic value of seismic action;
$\psi_{0,i}$ – Factor for combination value of the variable action i;
Q_i – 0.4 for snow load and live load.

It results: the total mass at the floor level M_{floor} = 28.2 t and the total mass at the roof level M_{roof} = 20.1 t. Accordingly, the total seismic force applied on the structure is:

$$S_e(T) = \gamma_I \cdot a_g \cdot \beta(T) \cdot g \cdot M = 1 \cdot 0.16 \cdot 3.0 \cdot 9.81 \cdot (28.2 + 20.1) = 227.43\,\text{kN} \tag{3.9}$$

Considering that the deformation is linear in regard with the height, the horizontal load is distributed to both levels as follows:

$$S_1 = S_r \cdot \frac{z_1 \cdot G_1}{\sum (z_j \cdot G_j)} = 227.43 \cdot \frac{2.8 \cdot 28.2}{(2.8 \cdot 28.2 + 5.8 \cdot 20.1)} = 91.84 \text{ kN}$$
$$S_2 = S_r \cdot \frac{z_2 \cdot G_2}{\sum (z_j \cdot G_j)} = 227.43 \cdot \frac{5.8 \cdot 20.1}{(2.8 \cdot 28.2 + 5.8 \cdot 20.1)} = 135.60 \text{ kN} \quad (3.10)$$

The total seismic force corresponding to both levels are:

$$T_{floor} = S_1 + S_2 = 227.43 \text{ kN}; \; T_{roof} = S_2 = 135.60 \text{ kN} \quad (3.11)$$

Considering the experimental reference capacity of 19.40 kN/m, the length of walls necessary to provide the shear capacity results as:

$$L_{floor} = \frac{T_{floor}}{P_{calc}} = \frac{227.43}{19.40} = 11.73 \text{ m} < 21.40 \text{ m};$$
$$L_{roof} = \frac{T_{roof}}{P_{calc}} = \frac{136.12}{19.40} = 7.02 \text{ m} < 21.40 \text{ m} \quad (3.12)$$

3.3.2.3 Life-cycle analysis and environmental impact [3.43, 3.44]

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.

The layers for the main components (exterior walls and intermediate floors) are presented in Figure 3.29.

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.

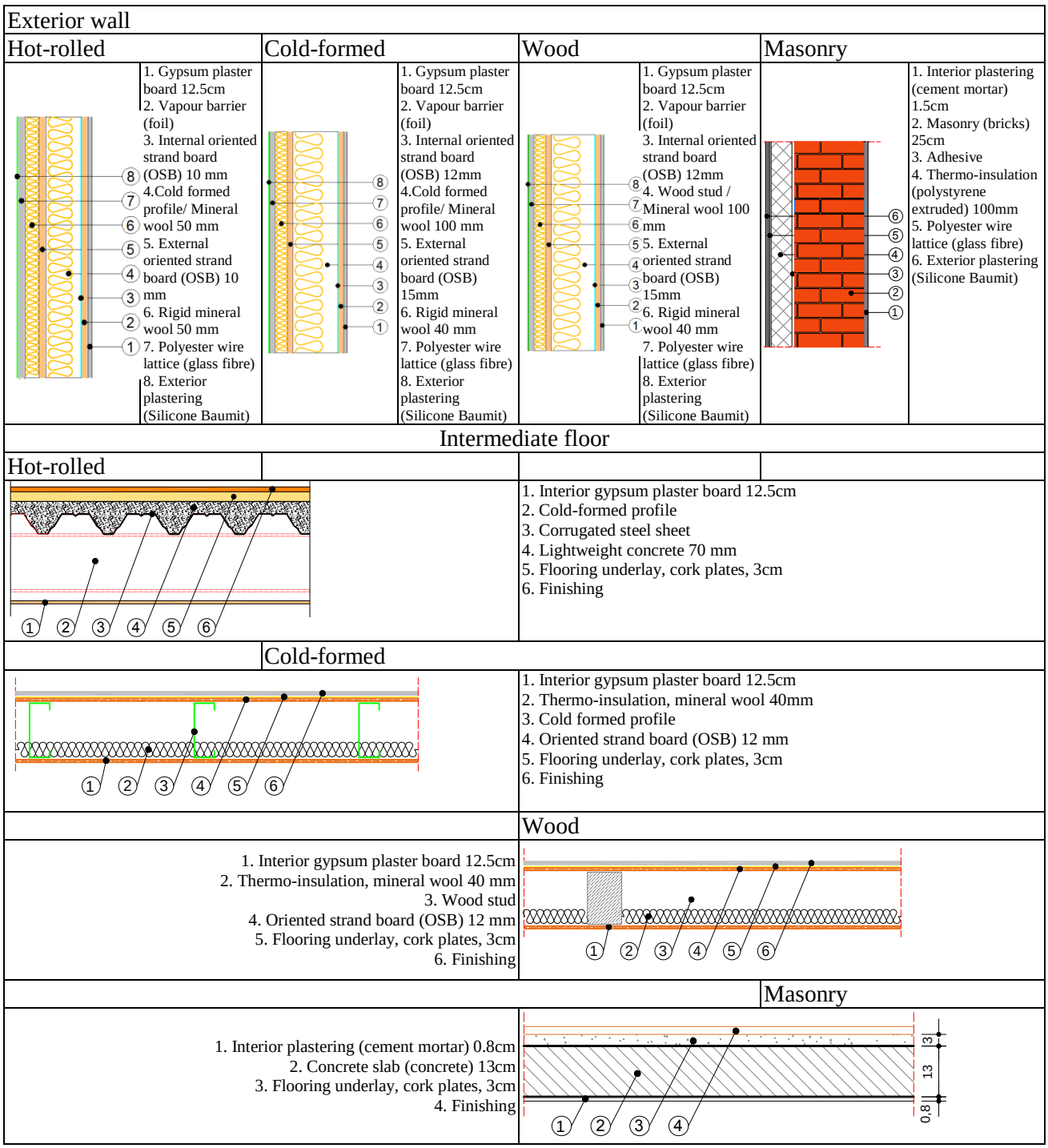


Fig. 3.29: Main components of the structure (examples): exterior walls and intermediate floors

System Boundaries

The comparative LCA analysis was performed using the SimaPro [3.36] 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.

Life Cycle Inventory (Input Data)

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 3.30.

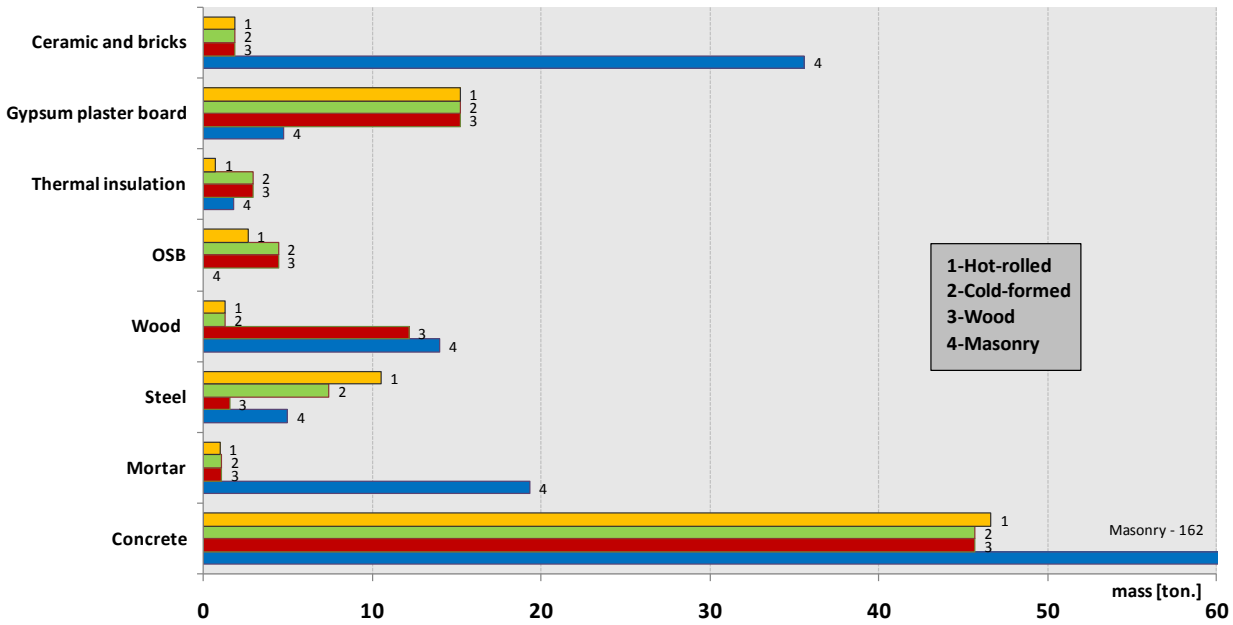


Fig. 3.30: Calculated quantities of the main materials for construction stage

Maintenance

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 (the designed life-cycle for such a category of construction in Romania is between 50 and 100 years according with CR0-2005 code [3.45] – a 50 years life-time was chosen as appropriate):

- 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 3.31.

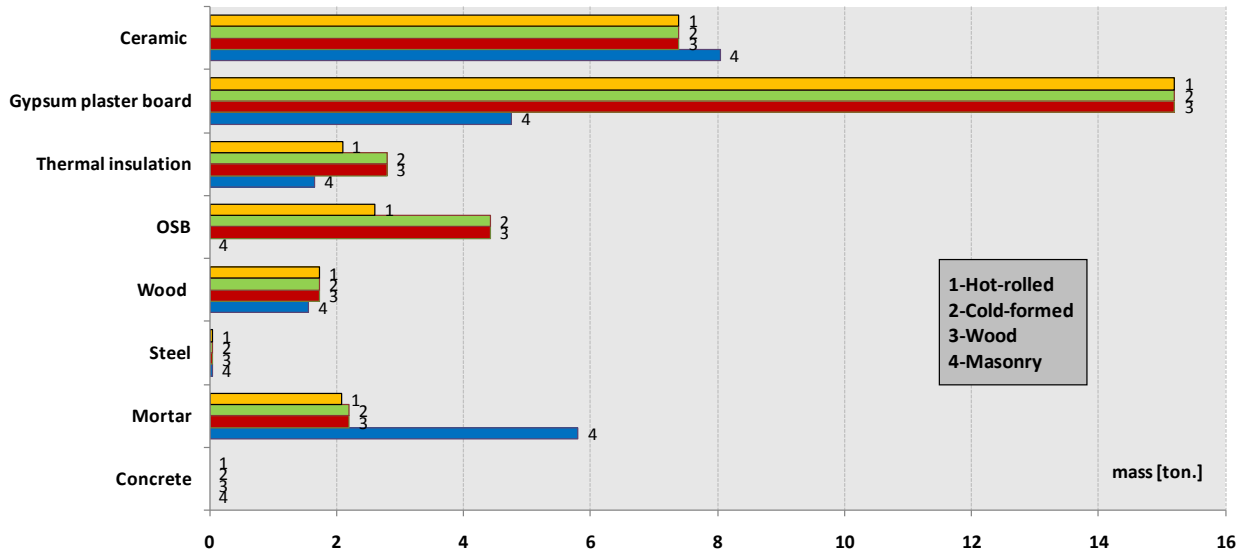


Fig. 3.31: Calculated quantities of the main materials for maintenance stage

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 3.11).

Table 3.11. 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	---

End-of-life (recycle, reuse and disposal)

In a life-cycle approach it is clear that the building process is not complete without an end-of-life estimation for the materials. 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. For the purpose of present study, the end-of-life of integrated materials was thought according to present conditions in Romania for recycling, reuse and disposal. Relevant data for main building materials, according to the inventory are summarised in Table 3.11.

3.3.2.4 Results of the Life-cycle analysis

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 [3.38].

Considering an environmental impact analysis on different impact categories, the highest impact values result for the masonry home, as could be seen in Figure 3.32. This fact is also confirmed by the single-score result (see Figure 3.33), 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).

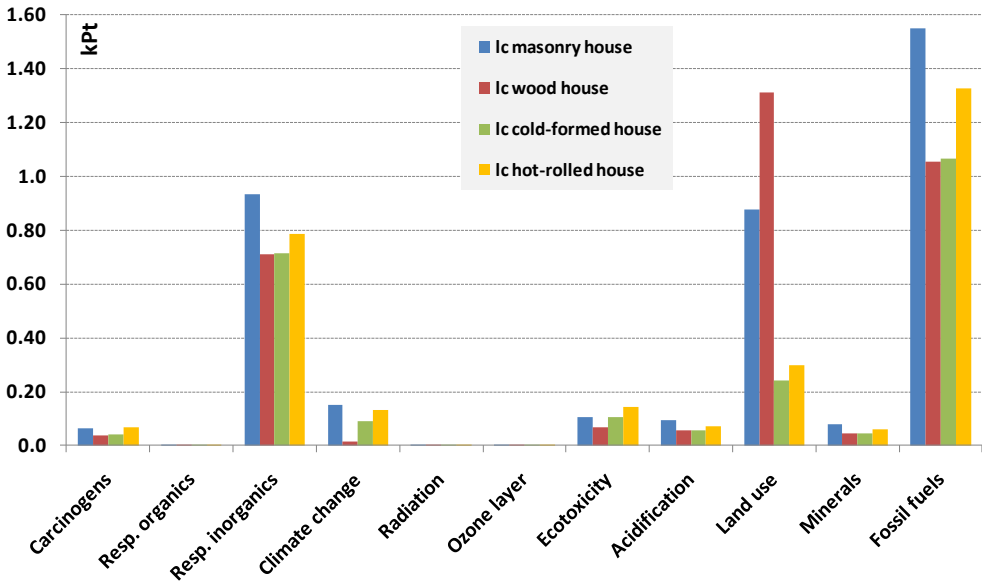


Fig. 3.32: Comparison on environmental impact for the construction stage only (weighting)

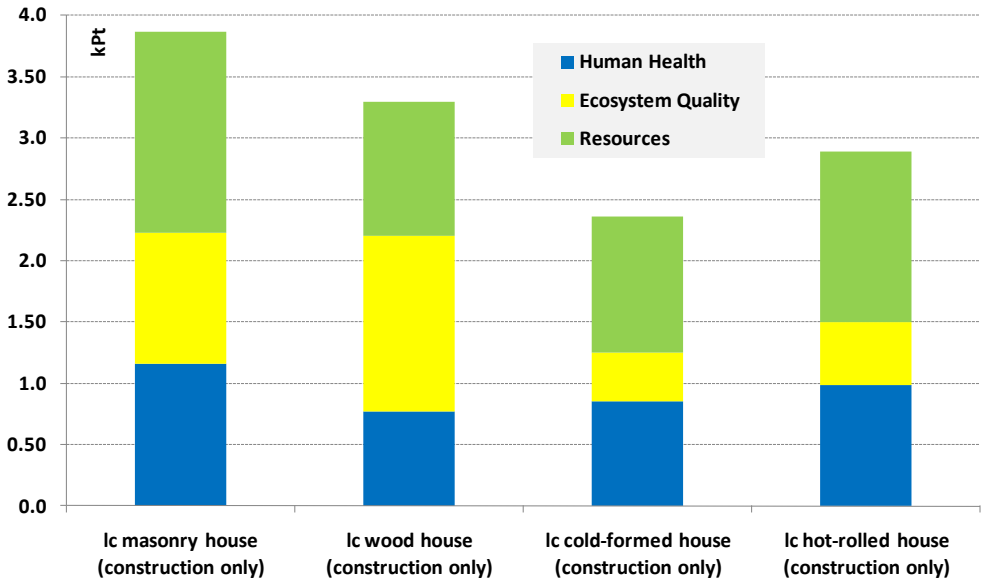


Fig. 3.33: Comparison on environmental impact for the construction stage only (single score)

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.

Environmental Impact Considering the Construction Stage and End-of-Life (Recycle, Reuse and Disposal)

The life-cycle comparison for construction and disposal scenario conduct to similar results to those for the construction stage only (comparison of Figures 3.34 and 3.35 with Figures 3.32 and 3.33 respectively). Ranked in a single score (see Figure 3.35) 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.

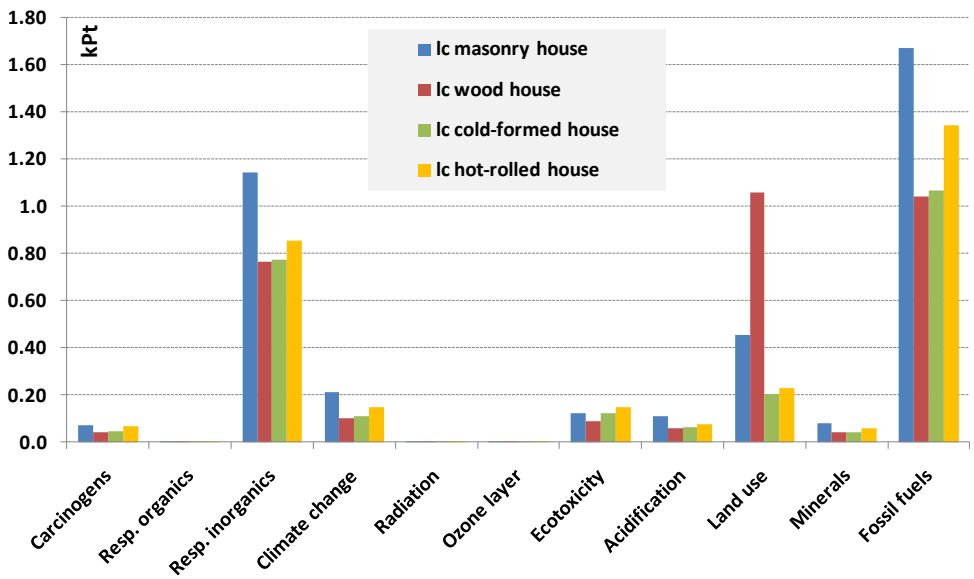


Fig. 3.34: Comparison on environmental impact for the construction and end-of-life stage (weighting)

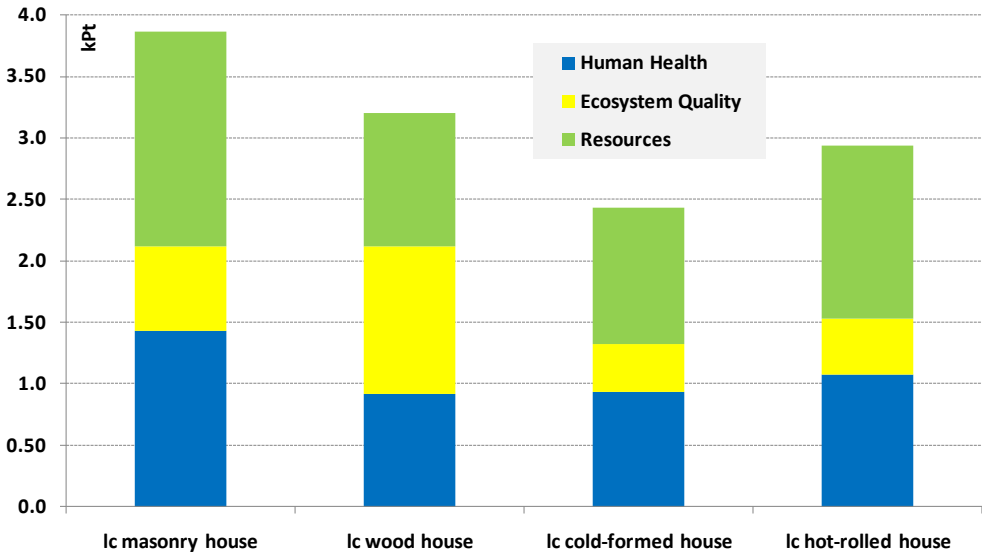


Fig.3.35: Comparison on environmental impact for the construction and end-of-life stage (single score)

Environmental Impact Considering the Maintenance of Buildings

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 3.36.

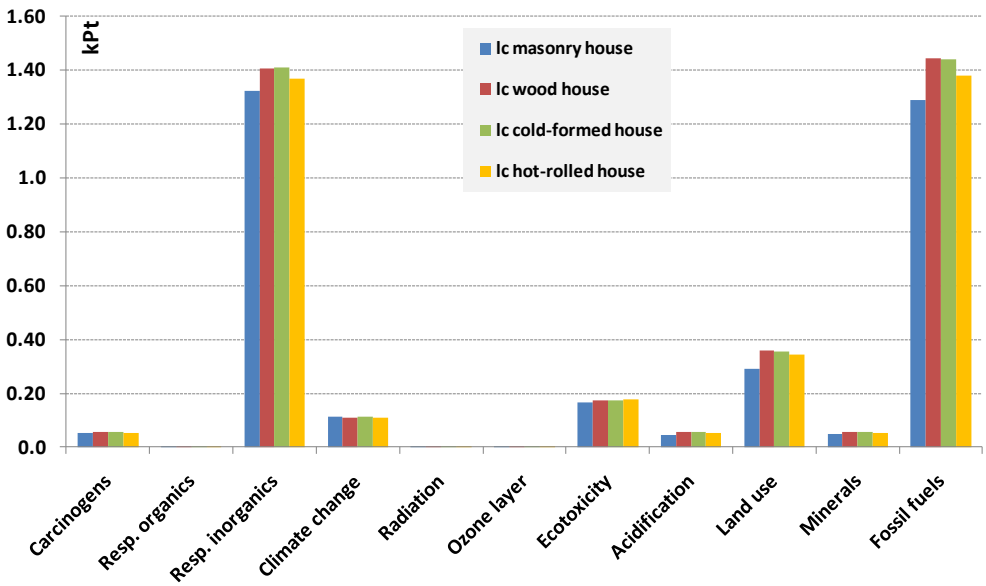


Fig. 3.36: Comparison on environmental impact for the maintenance stage only (weighting)

All the values resulted from maintenance only and, cumulated in a single score (see Figure 3.37), 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.

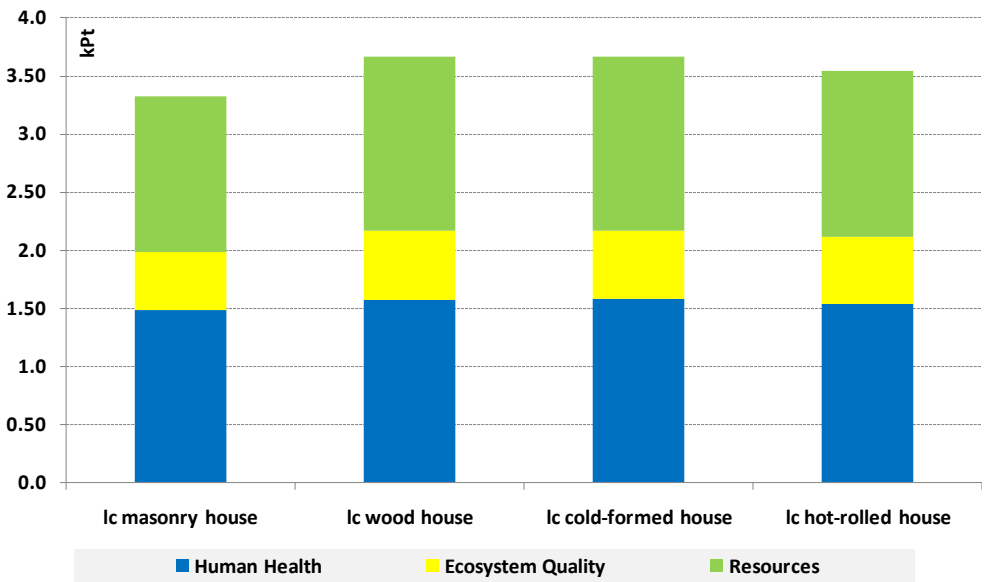


Figure 3.37. Comparison on environmental impact for the maintenance stage only (single score)

Environmental Impact Integrating the Maintenance of Buildings and End-of-Life

Figures 3.38 and 3.39 present 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.

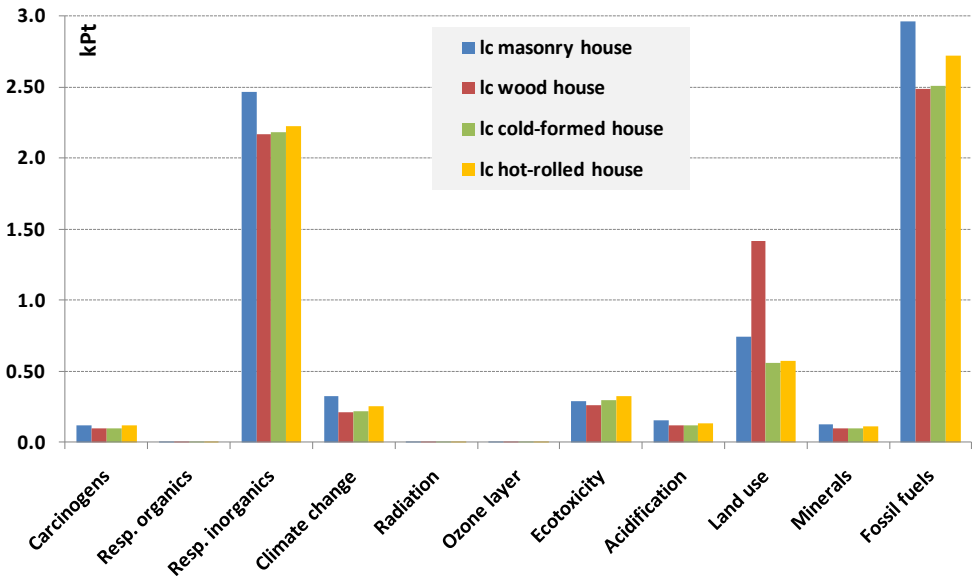


Fig. 3.38: Life-cycle comparison on environmental impact (weighting) for construction, including maintenance and end-of-life

As a general trend, the following impact categories are most affected, as Figure 3.38 shown:

(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.

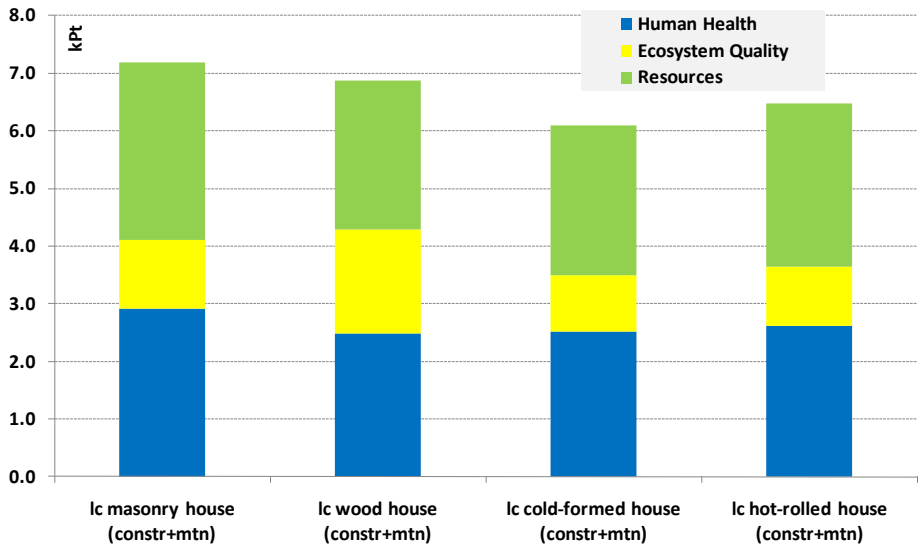


Fig. 3.39: Life-cycle comparison on environmental impact (single score) for construction, including maintenance and end-of-life

In a single score analysis (see Figure 3.39), 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.

3.3.2.5 Concluding remarks for case study 2

The case study presents an innovative structure-envelope solution, enabling flexible floor plan and modular construction, faster fabrication and erection times and high solution diversity for flooring and envelope. The following advantages of this structural system can be underlined:

(1) functional and aesthetical architecture;
(2) modularity and adaptability;
(3) enhanced structural performance, particularly for the seismic action (practically complete prevention of collapse);
(4) cost effective;
(5) low-to-moderate environmental impact compared with traditional technologies (e.g. masonry and/or concrete).

In the second part, a detailed environmental impact analysis on a family house designed in four different building systems was performed: hot-rolled framing solution, cold-formed steel framing, wood structure and masonry structure. The analyses show 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.

3.3.3 Case study 3 – Sustainable block of flats

3.3.3.1 Description of structural system and envelope

The structure is a block of flats built in 2007 in Timisoara, Romania [3.46, 3.47]. Architectural views, structure during erection and final view of the erected building are presented in Figure 3.40. 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 that means long-span solutions.

Fig. 3.40: Structure during erection and final view of the erected building

In the *Building Envelope Technology Roadmap* guide [3.48], the vision for 2020 is that building envelopes will be energy-positive, adaptable, affordable, environmental, healthy, intelligent and durable. This roadmap focuses on residential buildings, including new and existing low-rise multifamily dwellings as well as townhouses and single family detached homes. The envelope is essential in ensuring the comfort and energy savings of the system taken as a whole. All envelopes have to address some of 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 layer and mechanically fastened to the structure;
- optionally there could be a cavity space for ventilation;
- the interior component 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 structure in order to avoid and/or control thermal bridging. This aspect is of high importance, as all condensation problems start from here.

In building the high-end three story residential building, 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.

It should be noted that 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 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:

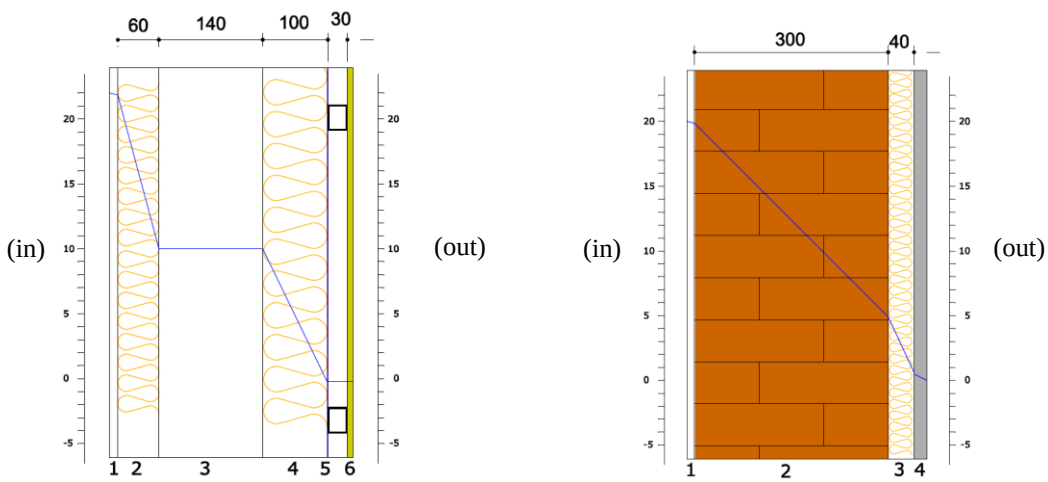
- overheating of the south and west facades in summer and,
- heat loss due to the wind-chill effect on the north/north-west sides of the facade in winter;

special attention was paid first of all to the passive energy saving measures.

The envelope design was rationalised, as permitted by site conditions and functional parameters [3.46, 3.47]:

- 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 frames;
- the long facades, facing north and south are mostly solid envelope, conceived as a thermal cavity system wall.

In Figure 3.41(a) are presented the actual layers for the cladding (in/out). Figure 3.41 illustrates the importance of adequate insulation both as thickness and position in the wall assembly; the comparison was made with a brick wall.



1. Gypsum Board 12.5mm
2. First layer of thermal insulation mineral wool 60mm, 45kg/mc
3. Thermal air cavity, 140mm
4. Second layer of thermal insulation mineral wool 100mm, 45kg/mc
5. Low admittance fluid membrane
6. High pressure laminated panels

(a) Thermal cavity wall: K = 0.219 W/(m^{2}K)

1. Interior plaster
2. Brick wall, 300mm
3. Thermal insulation mineral wool 40mm, 45kg/mc
4. Stucco, 20mm

(b) Brick wall: K = 0.406 W/(m^{2}K)

Fig. 3.41: Importance of adequate insulation

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 3.42 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, conditions for condensation are practically eliminated.

Fig. 3.42: 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.

Ambient measurements

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 [3.46, 3.47]. 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.

The heating system

From technical point of view, the chosen solution tries to make use of the energetic performances of the building, not only by means of production and distribution, but also by another parameter, i.e. the heating time, because it is very 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 usage 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.

3.3.3.2 Environmental impact

One of the requirements in an integrated design of the building [3.49]. For this reason, the above-descried structure is analysed for the evaluation of the evaluation impact in two situations [3.50, 3.51]:

- in a first stage, by comparing the structural system of the built structure to an equivalent reinforced concrete one. This comparison is limited to structural systems only, considering that the in-fill walls and the other non-structural elements could be identical and;
- secondly, for the built structure a complete environmental impact analysis is made, by considering all the building materials in order to observe the constructive elements that request the greatest environmental impact on building.

For both analyses the material production and the construction stage were included. The inventory used in analyses (performed by the help of SimaPro computer code [3.36]) was determined on the basis of the list of materials resulted by resistance design and adjusted in accordance to the following boundary conditions:

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

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 [3.38].

Concerning the first issue, Figure 3.43 presents the results of the direct comparison (on impact categories) between the analysed structural system and an equivalent reinforced concrete framed structure. It should be underlined the fact that this analysis includes exclusively the materials that are part of the structural system. For the built structure the main parameters are the steel demand (48kg/m^2/floor), concrete (0.1m^3/m^2 of lightweight floor and 0.44m^3/m^2 normal concrete for foundation). In the case of the concrete structure, the main materials of the building are the reinforcement (54kg/m^2/floor) and concrete (0.44m^3/m^2 normal concrete on total usable area). The inventory includes also the foundations and the roofing systems.

The values from above related to steel consumption (48kg/m^2/floor for steel structure and 54kg/m^2/floor for reinforced concrete structure) show clear that in seismic regions including moderate ones, the steel consumption is comparable. Also, the execution and erection time are faster in case of steel structures.

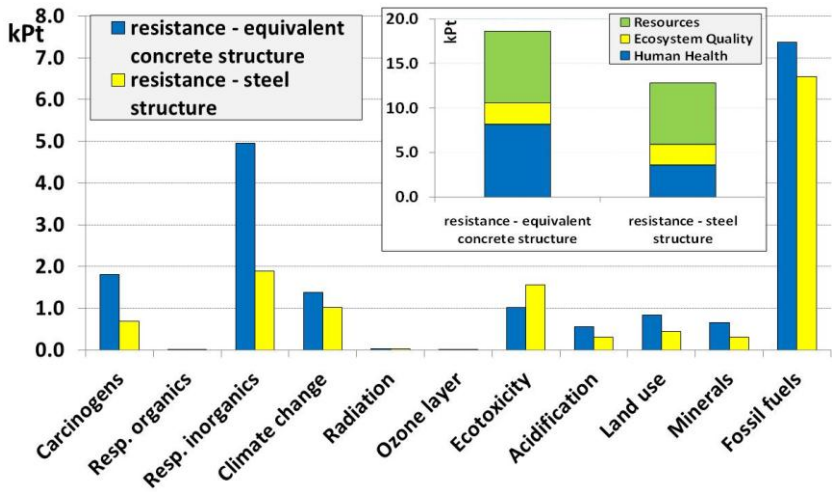


Fig. 3.43: The environmental impact comparison for steel and concrete structural solutions

It is easy to see that the concrete structure is having a larger environmental impact for almost all the impact categories (with the only exception – ecotoxicity) and leads to a global score (the inner figure) larger by about 50% than that of the steel structure. The above results should be interpreted as trends and not by the exact figures, and show very clearly the fact steel structures can have good environmental performances when compared to concrete solutions.

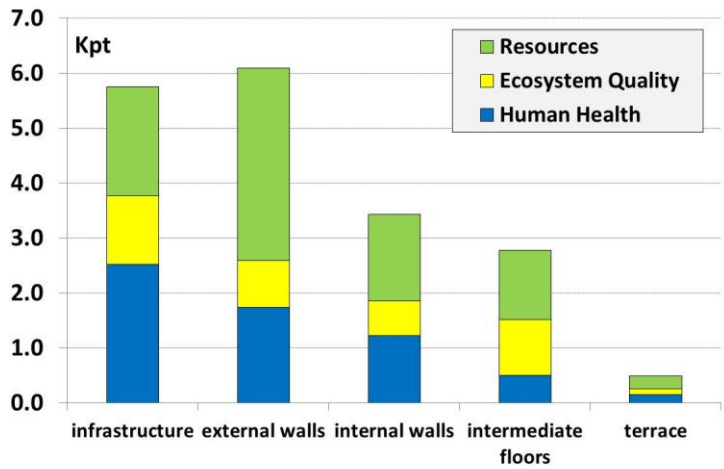


Fig. 3.44: Environmental impact per constructive element

Concerning the construction process ranked per constructive elements, Figure 3.44 presents the environmental analysis results considering the following elements: (1) exterior walls; (2) interior walls; (3) flooring system; (4) terrace; (5) foundation-infrastructure. The results show clearly that the major impact corresponds to exterior walls and infrastructure. These constitutive elements are high consumers of resources and have a great impact on human health. This analysis refers only to the steel solution, for which al materials and quantities exist.

3.3.3.3 Concluding remarks for case study 3

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: (1) inside average temperature; (2) relative humidity and room air temperature; (3) air movement; (4) acoustic insulation.

In the second part, an environmental impact analysis was performed for the block of flats, for the construction stage only and a comparison at the structural level, i.e. steel vs. reinforced concrete.

Comparing the structural systems one can see that the concrete structure is having a larger environmental impact for almost all the impact categories and leads to a global score 50% higher than for steel structure. More, the steel consumption (steel vs. reinforced concrete structure), for this type of structure located in a moderate seismic area is comparable. Also, the execution and erection time are faster in case of steel structures. Concerning the environmental impact per constructive element, in case of steel structure, the analysis shows that the major impact corresponds to exterior walls, followed by the infrastructure and interior walls.

3.4 Scientific and technical contributions of the author to the actual state-of-knowledge

Regarding the subjects presented above and based on the studies done by the author and the obtained results, the personal contributions include:

- According to candidate knowledge, the works done in the field of *Sustainability in constructions* represent the first LCA applications done in Romania, related to the construction sector;
- Analysis of three case studies of sustainable mixed building technologies, which combine steel in the framing and different materials for cladding, roofing and flooring, in order to obtain highly performance thermo-energetic properties;
- One of the innovative structural system developed is *Affordable House* (AH). Even if the main structure is made by hot-rolled profiles, the secondary structure is made by thin-walled cold-formed steel profiles. This innovative structural system is based on modular progression in such a way that at any time the owner can add a new module, both by vertical and/or horizontal addition, with high solution diversity for flooring and envelope. The structural system uses the application of industrial building technologies in dwelling building systems providing a fast erection and fabrication time.
- Development of comparative life cycle analysis (LCA) for each case study, which prove that steel structures represent a good alternative to masonry houses, not only in respect to structural requirements but also in considering the environmental impact assessment.

(b-ii) Scientific, professional and academic future development plans

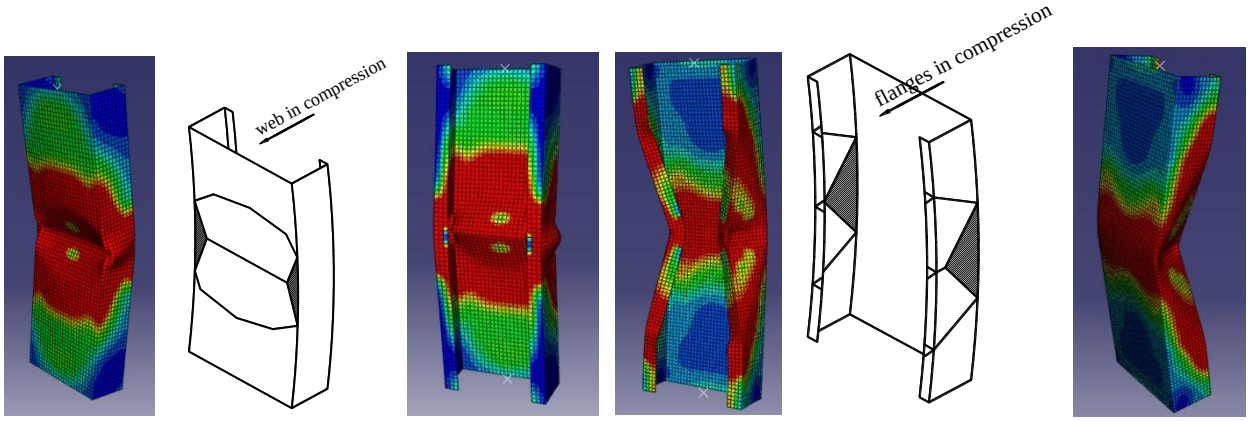
As presented in (b-i), the researches done by the candidate are developed in two main thematic directions.

The first one is related to *"Thin-walled cold-formed steel members"*, which continue the long tradition of more than 40 years of activity of Steel Structures Research School of Timisoara and remain the main topic for the candidate (18 years of research in this field).

The second direction, entitled *"Sustainability of construction"*, is a new field of research. It should be underlined that in Romania, in general, and in the construction sector, in particular, not too much work has been done in the field of sustainability. The work done by the candidate and by the team from CMMC Department of the Civil Engineering Faculty from Timisoara represents, probably, the only scientific papers and practical applications, in this field, in Romania. The activity started in 2006, when the candidate was appointed as member of Management Committee in the COST C25 action: "*Sustainability of Constructions - Integrated Approach to Life-time Structural Engineering*" and vice-chairman of Working Group WG3: "*Life-time Structural Engineering*" of COST C25 action. Since 2008 the candidate is member of Technical Committee TC14 "*Sustainability and Eco-efficiency of Steel Building*" of the European Convention for Constructional Steelwork (ECCS). More than 30 papers have been published in this field.

A) In what concerns the future research and development plans of the candidate, related to the field of research on *"Thin-walled cold-formed steel members"*, the following research topics will be continued or will be developed:

1) *To extend the database for plastic mechanisms for members in eccentric compression* – single sections (e.g. U-, C-, Z- sections) [2.7] or more complicated ones (e.g. two back-to-back lipped channel sections connected with stitches, Z-purlins restrained by sheeting). Figure below presents some on-going research in order to propose plastic mechanism for thin-walled cold-formed steel members subjected to eccentric compression.



Plastic mechanisms of members subjected to compression and bending about minor axis

2) *Codification of imperfections for cold-formed steel members and frames*:
   - to continue and to extend the work of identification and codification of local and global imperfection for individual members, started in [2.74], especially for pallet rack sections;
   - to extend the studies on imperfections sensitivity of pitched roof cold-formed steel portal frames [2.75], i.e. (a) global imperfections for frames; (b) local imperfections for individual members (in- & out-of-plane imperfections); (c) imperfections at the level of cross-section.
   - to provide a set of reliable and easy-to-use guidelines aimed at both (i) fabricators (to limit and control the manufacturing tolerances), (ii) constructor (to limit and control the

erection tolerances) and (iii) clear definition of member initial state that must be included in a FE analysis;

3) *Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections restrained by sheeting*.

The behaviour of multi-span cold-formed Z-purlins with bolted lapped connections has been studied in detail by Ungureanu & Dubina [2.28, 2.29, 2.30].

The structural behaviour of cold-formed steel purlins restrained by sheeting is very complex. The cold-formed steel purlins restrained by sheeting shows a complex structural behaviour characterised by a non-linear load displacement response differing substantially from that obtained based on the analysis of the isolated purlin.

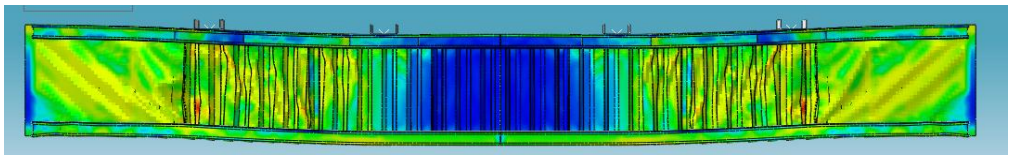
The purpose of the future research is to study experimentally and numerically the behaviour of multi-span cold-formed Z-purlins under gravity loads, where at the inner support the bending, torsion and lateral distortion of the free flange are influenced by the overlapping, bolts, sheeting and by the connections between the purlin and the sheeting. In a similar way, but more simply is to study after the behaviour under uplift loads which involves bending, torsion and lateral distortion in the span.

A parametric study will be performed to see the influence of the above parameters on the behaviour of cold-formed steel purlins restrained by sheeting. It is well known that the lapped connections of such purlins are always semi-rigid and partially resistant. A method to determine the semi-rigid behaviour will be proposed.

4) Cold-formed steel beams of screwed corrugated webs.

Corrugated web girders are relatively new structural system. Increased interest of this solution was observed for the main frames of single-storey steel buildings. The thin corrugated web, affords a significant weight reduction of these beams, compared with hot-rolled or welded ones. In previous developed solutions the flanges are flat plates, welded to the sinusoidal web sheet, demanding for specific welding technology. The girder's flanges provide mainly flexural strength of girder with low contribution from the corrugated web, which provides the girder's shear capacity. Failure of the web occurs by steel yielding or web buckling. Lateral-torsional buckling of the girder and local flange buckling of corrugated web represents other possible failure modes.

The future research proposes to investigate and validate a NEW technological solution of such system: (1) the web is made by trapezoidal cold-formed sheet; (2) the flanges are built-up cold-formed members; (3) the connections between flanges and web and web-to-web is made by self-drilling screws or by a by a new welding technology called Cold Metal Transfer (CMT).



An experimentally study will be proposed in order investigated and to demonstrate the feasibility of structural solution and to identify the components and failure mechanisms.

A parametric study based on numerical models will be developed in order to validate an analytical model.

The following aspects are proposed to be investigated:

- to identify the failure modes: failure of the web occurs by steel yielding, web buckling (local or distortional or their interaction), lateral-torsional buckling of the girder and local flange buckling of corrugated web, separately or in interaction, behaviour of screwed connections between flanges and webs and seam fasteners between corrugated sheets;

- in order to obtain a complete solution, experimental investigations will be carried out on beam-to-column connections and beam-to-beam connections at the ridge, to identify their capacity in terms of strength and stiffness and to choose the proper one;
- the influence of openings in the web will be investigated.

A standard series of solutions will be proposed to be applied in portal frames, either with steel or RC columns, or as secondary light beams.

5) Influence of residual stresses on the perforated pallet rack members due to the cold-reducing process.

It must be specified that the steel strips used for cold-forming of perforated pallet rack sections is obtained, very offend, by cold-reducing. In this case, the yield strength might increase significantly (i.e. from 320-350 N/mm^2, for the raw steel coil, up to 500-610 N/mm^2 for the base strips used to form the steel sections). The ultimate tensile strength is also increased by cold-reducing process. Together with the increase in yield and tensile strength, the material ductility is significantly reduced (i.e. form 30% down to 14-15%, even less).

Due to cold-forming process of the cross-section, the material properties are modified. The corners areas are more affected by plastic deformations and the changes in material properties are more pronounced in corners areas than for flat ones.

Research in order to determine experimentally the increase of yield strength, ultimate tensile strength and the distribution of residual stresses is proposed. The biggest impact is for the case of connections. Recalibration of design formulas for connections of thin-walled cold-formed members, especially in case of perforated pallet rack members is necessary. The use of existing equations is overoptimistic for connections in steel having low ductility, leading to unconservative estimates for the tension capacity.

6) The behaviour of pallet rack systems under seismic action.

Pallet rack systems are not regular buildings but a very particular form of steel construction. They differ from buildings in terms of their use, the loads that are supported, the geometrical dimensions and the components used in their construction. These components are normally thin-gauge cold-formed steel profiles and, in the case of uprights, are typically continuously perforated.

It should be mentioned that the new Code of Practice issued in September 2010 [2.76] needs several improvements. The following aspects are proposed to be studied, i.e.:

- component tests are still needed with the aim of characterizing the behaviour of both beam-to-upright and base connections, in order to allow a correct interpretation of full scale tests as well as in order to calibrate numerical models;
- cyclic testing of the above components;
- correct definition of q-factors to be adopted in seismic design of racks.

7) Modular / affordable / emergency steel intensive houses

In recent years, users, architects and engineers have all been searching for the most economic dwelling building systems. In this context, steel-framed houses have become a choice in many European countries, including Romania. The idea is to find an innovative structure / envelope solution, enabling flexible floor plans and modular construction, faster fabrication and erection times and high solution diversity for floors and envelope.

Based on the experience learned in *Affordable House Project*, as member of Romanian team, in the frame of ArcelorMittal Network, between 2008-2009, several types of steel intensive houses are proposed to be studied, i.e. *modular / affordable / emergency steel intensive houses*.

Using thin-walled cold-formed steel members or hot-rolled profiles, such houses can be developed to be affordable and to protect against damage from earthquakes, tornados, fires and floods, to be extremely durable, comfortable, healthy and safe.

In this context, several ideas are proposed to be investigated:

- the application of industrial building technologies in dwelling building systems (residential applications) achieving fast erection and fabrication times. The basic assumption is that an affordable / emergency house should rely on the standard details and common technologies available to most builders instead of experimenting with new materials with no track record;
- the development of a modular system in such a way that at any time the owner can add a new module, both vertically and/or horizontally, with a high solution diversity for floors and envelope;
- the design – in terms of structural performance – of both the walls and the floors based on stressed skin technology. It is well known that using oriented strand board (OSB) panels for walls and profiled steel sheeting as the floor decking results in very effective shear diaphragms. Provided they are positively attached to the secondary members and main frames by mechanical fasteners or welding, they are extremely reliable and predictable, and may be confidently used as structural components;
- the use of structural systems made from lightweight steel frames, hot-rolled sections or timber framing, which assures the lightness of the house and the proper response to climatic and seismic loading.

Finally, it have to be underlined that the active role of the candidate in the field will continuously increase by participation with new research topics to international conferences and papers published in specialised journals. Also, as members of Technical Committee TC7 "*Cold Formed Thin Walled Sheet Steel in Building*" of the European Convention for Constructional Steelwork (ECCS), the candidate will continue the collaborative work with researchers in the field.

It should be underlined the last collaborative work done in the frame of TC7 "*Cold Formed Thin Walled Sheet Steel in Building*" of ECCS, i.e. the volume published in 2012: *Design of Cold-formed Steel Structures. Eurocode 3: Design of Steel Structures. Part 1-3 Design of cold-formed Steel Structures*. Authors: Dubina D., Ungureanu V., Landolfo R., Ernst & Sohn, A Wiley Company, Berlin.

B) In what concerns the future research and development plans of the candidate, related to the field of research on *"Sustainability of construction"*, the following research topics will be developed:

1) Sustainable design, climate change and the built environment

Climate change impacts the construction sector in various ways. In this context, the construction sector includes not only construction chemistry, materials technology and building science, but also planning, construction techniques and urban climate.

The challenge is to identify which component is most vulnerable to changes in climate, how to keep the safety and durability of structures and how to respond to these changes. Many challenging questions are raised by scientists, policy makers, industry etc. For example, what do we know about climate change and its impacts on the construction sector? Which technologies are needed to respond to future climate conditions without neglecting ecological, social, and economical aspects?

Some of the climatic factors which affect the behaviour of buildings are:

- Extreme weather conditions in terms of *wind, snow and rain*. These extreme conditions lead to increase of static and dynamic loads;
- *Temperature*. Maximum and minimum changes will affect heating, cooling and air conditioning costs. Frequency of cycling through freezing point will affect durability of materials.
- *Relative humidity*, which will affect condensation and associated damages;

- *Radiations*. Increase may affect need for solar glare control. More intense sunshine and associated ultraviolet levels;
- *Heavy storm.* Increase will affect the need for weather tightness, risk of water ingress, effectiveness of air conditioning, energy use, risk of roof failures;
- *Soil drying / Precipitation*. Increase will affect water tables and could affect foundations in clay soils.

The following topics in the context of sustainability and climate change will be addressed:

- Identification of longer-lasting, optimized properties, ecological, economical, and easy-to-handle materials to reduce materials and energy input;
- Innovative building technologies and concepts which help to prevent damage caused by increasing climate extremes and provide indoor environments with high energy and comfort performance;
- Innovation of materials technology with focus on mechanical properties, processibility, and serviceability of materials to reduce deformation and damage of infrastructure and building.

Existing buildings have been designed for a given life-time. It is important to predict the new life time, probably shorter than the estimated one, to find the best strategy of intervention and to find the best materials and technologies to be applied. For new buildings the key will be the use of new longer-lasting and eco- materials together with details for adaptive technologies.

2) *Structural glass*

The increasing demand in modern architecture for more slender and lighter structures requires the use of new construction materials. Glass, a material that has been used for a long time in windows as a filling material, has much to offer in this regard due to its very high compressive strength and transparency. For this reason, there is a growing trend to extend the use of glass sheets to load carrying elements such as columns, beams and panels. Due to their high slenderness and high compressive strength, such elements tend to fail because of instability (i.e. column buckling, lateral torsional buckling or plate buckling). At the moment little knowledge exists about the load carrying behaviour of glass structural elements, and existing design methods for other materials (i.e. steel) have been found to be unsuitable for direct transfer to the design of glass panels. Connections present a special challenge when designing with structural glass, but several different forms of connections have been successfully demonstrated in construction.

The glazing façades can be classified into two main types, namely framed glazing and frameless glazing façade systems. The unitized curtain wall system is a more contemporary framing method which comprises a glass vision panel and spandrel panel mounted in a prefabricated aluminium frame. Alternatively a new contemporary frameless glazed façade system is available which provides transparency and improved aesthetics, known as point fixed or bolt fixed glazing curtain wall system. Point fixed glazing systems are often connected with bolts to steel support structures, (which are exposed architectural elements) to combine structural stability with aesthetic expression.

The two structural concepts are proposed to be analysed in interaction with the main structure, especially under seismic actions:

- point support concept - the glass panel is attached to the substructure by bolted connections at corners;
- linear support concept - the glass panel is attached to the substructure by continuous supporting on the four sides;

The local behaviour of the connection devices and the global behaviour of the glass panel under in-plane shear force will be studied by means of experimental investigations, numerical

modelling and parametric analyses. Additionally, the thermal performance analysis of glazed facades using EnergyPlus will be analysed.

3) *Retrofitting of existing prefabricated concrete residential buildings to reduce the primary energy use and GHG emissions*

Concrete residential buildings present the largest retrofitting challenge in Eastern European countries, including Romania, and one of the best opportunities to substantially improve energy efficiency of residential buildings on the large scale in the EU. The scope is the renovation of prefabricated concrete residential with special focus on technological (e.g. architecture, building physics, safety, energy, etc.), economic, institutional and policy settings.

The following aspects are proposed to be studied:

- reorganisation of internal spaces including addition of new balconies, without affecting the safety of structure;
- over-cladding and over-roofing – the use of light steel structure;
- the use of external elevators;
- the use of intelligent facades to reduce the primary energy use and GHG emissions.

Finally, it have to be underlined that the active role of the candidate in the field will continuously increase by participation with new research topics to international conferences and papers published in specialised journals. Also, as members of Technical Committee TC14 "*Sustainability & Eco-Efficiency of Steel Construction*" of the European Convention for Constructional Steelwork (ECCS), the candidate will continue the collaborative work with researchers in the field.

It should be underlined the last collaborative work done in the frame of Technical Committee 14: Sustainability and Eco-efficiency of Steel Constructions of ECCS, i.e. the volume published in 2012: *Energy efficiency of light-weight steel-framed buildings*. Authors: Santos P., Simoes de Silva L., Ungureanu V., No. 129/2012, Published by ECCS, ISBN: 978-92-9147-105-8.

Also it should be mentioned here the participation in two international projects as responsible person for "Politehnica" University of Timisoara:

1. RFSR-CT-2010-00027: „SB_Steel – Sustainable Building Project in Steel". Period: 2010-2013. Beneficiary: RFCS - Research Fund for Coil and Steel, EU.
2. ERA-NET 3002/2011: „INSPIRE - Integrated strategies and policy instruments for retrofitting buildings to reduce primary energy use and GHG emissions". Period: 2010-2012. Beneficiary: MECTS.

(b-iii) References

Chapter 2: Thin-walled cold-formed steel members

[2.1] Murray N.W., Khoo P.S. (1981). *Some basic mechanism in the local buckling of thin-walled steel structures*. Int. Journal of Mechanical Sciences, 23(12), pp. 703-713.

[2.2] Kotelko M., Lim T.H., Rhodes J. (2000). *Post-failure of box sections beams under pure bending (an experimental study)*. Thin-Walled Structures, Vol. 38, pp. 179-194.

[2.3] Ungureanu V., Dubina D. (2004). *Recent research advances on ECBL approach. Part I: Plastic-elastic interactive buckling of cold-formed steel sections*. Thin-Walled Structures, 42(2), pp. 177–94.

[2.4] Kotełko M. (2004). *Load-capacity estimation and collapse analysis of thin-walled beams and columns – recent advances*. Thin-Walled Structures, 42(2), pp. 153-175.

[2.5] Dubina D., Ungureanu V. (2000). *Elastic-plastic interactive buckling of thin-walled steel compression members*. Proc. of 15th Int. Conference on Cold-Formed Steel Structures, St. Louis, USA, pp. 223 – 237.

[2.6] Zhao X.-L. (2003). *Yield line mechanism analysis of steel members and connections.* Progress in Structural Engineering and Materials, Vol. 5, pp. 252-262.

[2.7] Ungureanu V., Kotelko M., Mania R.J., Dubina D. (2010). *Plastic mechanisms database for thin-walled cold-formed steel members in compression and bending*. Thin-Walled Structures. 48(10-11), pp. 818-826.

[2.8] Kotelko M., Ungureanu V., Dubina D., Macdonald M. (2011). *Plastic strength of thin-walled plated members—Alternative solutions review*. Thin-Walled Structures. 49(5), pp. 636–644.

[2.9] Królak M., Młotkowski A. (1996). *Experimental analysis of post-buckling and collapse behaviour of thin-walled box-section beam*. Thin-Walled Structures, 26 (4), pp. 287-314.

[2.10] Kato B. (1965). *Buckling strength of plates in the plastic range*. International Association of Bridge and Structural Engineering, Vol. 25, pp. 127-141.

[2.11] Korol R.M., Sherbourne A.N. (1972). *Strength prediction of plates in uniaxial compression. Journal of Structural Division*, Vol. 98, ST 9, pp. 1965-1986.

[2.12] Mahendran M. (1997). *Local plastic mechanisms in thin steel plates under in-plane compression*. Thin-Walled Structures, 27(3), pp. 245-261.

[2.13] Rondal J., Maquoi R. (1985). *Stub-column strength of thin-walled square and rectangular hollow sections*. Thin-Walled Structures, Vol. 3, pp. 15-34.

[2.14] Kragerup J. (1982). *Five notes on plate buckling*. Technical University of Denmark, Department of Structural Engineering, Series R, No. 143.

[2.15] Feldman M. (1994). *Zur Rotations Kapazitat von I-Profilen statisch und dynamisch belastung Trager*. PhD Thesis, RWTH Universitat Aachen, Germany.

[2.16] Park M.S., Lee B.C. (1996). *Prediction of bending collapse behaviours of thin-walled open section beams*. Thin-Walled Structures, 25(3), pp. 185-206.

[2.17] Kecman D. (1983). *Bending collapse of rectangular and square section tubes*. International Journal of Mechanical Sciences, 25(9-10), pp. 623-636.

[2.18] Kotelko M. (1996). *Ultimate load and post-failure behaviour of box-section beams under pure bending*. Engineering Transactions, 44(2), pp. 229-251.

[2.19] Kotelko M., Krolak M. (1993). *Collapse behaviour of triangular cross-section girders subject to pure bending*. Thin-Walled Structures, 15(2), pp. 127-141.

[2.20] Elchalakani M., Zhao X.L., Grzebieta R.H. (2002). *Plastic mechanism analysis of circular tubes under pure bending*. Int. J. of Mechanical Sciences, 44(6), pp. 1117-1143.

[2.21] Elchalakani M., Grzebieta R.H., Zhao X.L. (2002). *Plastic collapse analysis of slender circular tubes subjected to large deformation pure bending*. Advances in Structural Engineering – An International Journal, 5(4), pp. 241-257.

[2.22] Bambach M.R., Rasmussen K.J.R., Ungureanu V. (2007). *Inelastic behaviour and design of slender I-sections in minor axis bending*. Journal of Constructional Steel Research. 63(1), pp. 1-12.

[2.23] Ungureanu V., Kotelko M., Mania R.J., Dubina D. (2008). *Plastic mechanisms database for thin-walled cold-formed steel members in compression and bending*. Proceedings of the Fifth Conference on Coupled Instabilities in Metal Structures – CIMS 2008, Sydney, Australia, 23-25 June 2008. Vol. 1, ISBN: 978-0-646-49439-5, pp. 189-197.

[2.24] Ungureanu V., Dubina D., Kotełko M., Mania R.J. (2009). *Plastic strength of thin-walled plated members - alternative solutions review, Part I. Numerical analysis*. Proc. of the XIIth Symposium on Stability of Structures, ISBN 83-914019-6-0. Zakopane, Polonia, 07-11.09.2009, pp. 451-460.

[2.25] Ungureanu V., Dubina D., Kotełko M., Mania R.J. (2009). *Plastic strength of thin-walled plated members - alternative solutions review, Part II. Numerical vs. experimental comparisons*. Proc. of the XIIth Symposium on Stability of Structures, ISBN 83-914019-6-0. Zakopane, Polonia, 07-11.09.2009, pp. 461-470.

[2.26] Kołakowski Z., Kotełko M. (2002). *Coupled instability related collapse behaviour of channel-section beam-column*. Proc. of 3rd Int. Conf. on Coupled Instabilities in Metal Structures CIMS'2000 (Lisbon, September 2000), Imperial College Press, pp. 205-212.

[2.27] Flockhart C.J., et al. (1999). *Comparison of upper-bound rigid-plastic yield line mechanism analysis by the energy and equilibrium strip method*. Proc. of the 2nd Australasia Congress of Applied Mechanics, Canberra, Paper R053.

[2.28] Dubina D., Ungureanu V. (2010). *Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections. Thin-Walled Structures*. 48(10-11), pp. 866-871.

[2.29] Ungureanu V., Dubina D. (2008). *Behaviour of continuous purlins of lapped cold-formed Z-sections and bolted on intermediate supports*. Proceedings of the 5th European Conference on Steel and Composite Structures – EUROSTEEL 2008, Graz, Austria, 3-5 September 2008. Vol. A, ISBN: 92-0147-000-90, pp. 75-80.

[2.30] Dubina D., Ungureanu V. (2008). *Behaviour of multi-span purlins of bolted lapped cold-formed Z-sections*. Proceedings of the Fifth Conference on Coupled Instabilities in Metal Structures – CIMS 2008, Sydney, Australia, 23-25 June 2008. Vol. 1, ISBN: 978-0-646-49439-5, pp. 507-514.

[2.31] Ho HC, Chung KF (2004). *Experimental investigation into the structural behaviour of lapped connections between cold-formed steel Z sections*. Thin-Walled Structures, vol. 42, pp. 1013–33.

[2.32] EN1993-1-3. *Eurocode 3 – Part 1-3: Supplementary rules for cold-formed thin gauge members and sheeting*. European Committee for Standardization, Brussels, 2006.

[2.33] Zaharia R, Dubina D (2006). *Stiffness of joints in bolted connected cold-formed steel joints*. Journal of Constructional Steel Research, Vol.62, No.3, pp.195-203.

[2.34] Lim J.B.P. (2001). *Joint effects in cold-formed steel portal frames*. University of Nottingham, PhD Thesis.

[2.35] Dundu M., Kemp A.R. (2006). *Strength requirements of single cold formed channels connected back-to-back*. Journal of Constructional Steel Research, 62(3), pp. 250-261.

[2.36] Kwon Y.B., Chung H.S., Kim G.D. (2006). *Experiments of cold-formed steel connections and portal frames*. Journal of Structural Engineering, 132(4), pp. 600-607.

[2.37] Dubina D., Stratan A., Ciutina A., Fulop L., Nagy Zs. (2004). *Performance of ridge and eaves joints in cold-formed steel portal frames*. The 17th Int. Specialty Conference, Orlando, Florida, USA, pp. 727-742.

[2.38] EN1993-1-8 (2005). *Eurocode 3: Design of steel structures. Part 1-8: Design of joints*. European Committee for Standardization, Brussels, Belgium.

[2.39] Nagy Zs., Stratan A., Dubina D. (2006). *Application of component method for bolted cold-formed steel joints*. Int. Conference on Metal Structures – ICMS 2006 "Steel – A new and traditional material for building", Poiana Brasov, Romania, pp. 207-215.

[2.41] Dubina D., Stratan A., Nagy Zs. (2007). *Full – scale testing of cold-formed steel pitched-roof portal frames of back-to-back channel sections and bolted joints*. The Sixth International Conference on Steel and Aluminum Structures, Oxford, UK, pp. 931-939.

[2.42] Dubina D, Ungureanu V, Stratan A (2008). *Ultimate design capacity of pitch-roof portal frames made by thin-walled cold-formed members*. The 5th International Conference on Thin-Walled Structures: Recent Innovations and Developments, Gold Coast, Australia, Vol. 1, pp. 387-394.

[2.43] Dubina D., Ungureanu V., Nagy Zs., Nunes L., Pernes P. (2010). *Imperfections sensitivity analysis of pitched roof cold-formed steel portal frames*. Proceedings of the International Colloquium on Stability and Ductility of Steel Structures, SDSS'Rio 2010, Rio de Janeiro, Brazil. 8-10.09.2010, ISBN 978-85-285-0137-7, pp. 929-936.

[2.44] EN1090-2. *Execution of steel structures and aluminium structures. Technical requirements for the execution of steel structures*. European Committee for Standardization, Brussels, 2008.

[2.45] EN1993-1-1. *Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*. European Committee for Standardization, Brussels, 2005.

[2.46] Rondal J. (1988). *Thin-walled structures – General Report*, Stability of steel structures, Budapest, Hungary, Akademiai Kiado, pp. 849–66.

[2.47] Schafer B.W., Peköz T. (1998). *Computational modelling of cold-formed steel characterising geometric imperfections and residual stresses*. J. Constructional Steel Research, 47(3), pp. 193-210.

[2.48] Abdel-Rahman N., Sivakumaran K.S. (1997). *Material properties models for analysis of cold-formed steel members*. J. Struct. Eng., ASCE, 123(9), pp. 1135-1143.

[2.49] Moen C.D., Igusa T., Schafer B.W. (2008). *Prediction of residual stresses and strains in cold-formed steel members*. Thin-Walled Structures, 46, pp. 1274–1289.

[2.50] Dubina D., Ungureanu V., Rondal J. (2005). *Numerical modelling and codification of imperfections for cold-formed steel members analysis*. Steel and Composite Structures, Vol. 5, No. 6, pp. 515-533.

[2.51] Dubina D, Ungureanu V. (2002). *Effect of imperfections on numerical simulation on instability behaviour of cold-formed steel members*. Thin-Walled Structures, 40(3), pp. 239-262.

[2.52] Rasmussen K.J.R., Hancock G.J. (1988). *Geometric imperfections in plated structures subject to interaction between buckling modes*. Thin-Walled Structures, 6, pp. 433-452.

[2.53] Schafer B.W., Grigoriu M., Peköz T. (1998). *A probabilistic examination of the ultimate strength of cold-formed steel elements*. Thin-Walled Structures, 31(4), pp. 271-288.

[2.54] Rasmussen K.J.R. (1996). *Numerical simulations and computational models in coupled instabilities*. Proc. of the 2nd Int. Conf. on Coupled Instabilities in Metal Structures, CIMS'96, pp. 45-60.

[2.55] Sridharan S. (2000). *Numerical simulation and computational models for coupled instabilities*. Proc. of the 3rd Int. Conf. on Coupled Instabilities in Metal Structures, CIMS'2000, pp. 61-72.

[2.56] Schafer B.W. (2008). *Computational modelling of cold-formed steel.* Proc. of the 5th Int. Conf. on Coupled Instabilities in Metal Structures, CIMS'2008, pp. 53-60.

[2.57] Adany S., Schafer B.W. (2008). *A full modal decomposition of thin-walled, single-branched open cross-section members via constrained finite strip method*. J. Constructional Steel Research, 64(1), pp. 12-29.

[2.58] Bebiano R., Pina P., Silvestre N., Camotim D. (2008). *GBTUL – Buckling and vibration analysis of thin-walled members*. DECivil/IST, Technical University of Lisbon, http://www.civil.ist.utl.pt/gbt.

[2.59] Camotim D., Basaglia C., Silvestre N. (2010). *GBT buckling analysis of thin-walled steel frames: A state-of-the-art report*. Thin-Walled Structures, 48(10-11), pp. 726-743.

[2.60] Camotim D., Dinis P.B. (2010). *Coupled instabilities with distortional buckling in cold-formed steel lipped channel columns*. Thin-Walled Structures, 48(10-11), pp. 771-785.
[2.61] Loughlan J., Yidris N., Jones K. (2010). *The failure of thin-walled lipped channel compression members due to coupled local-distortional interactions and material yielding*. Thin-Walled Structures, 2012 (article in press).
[2.62] Casafont M., Caparrós F., Pastor M., Roure F., Bonada J. (2011). *Linear buckling analysis of perforated steel storage rack columns with the finite strip method*. Proc. of the 6th Int. Conf. on Thin Walled Structures, Timisoara, Romania, Vol. 2, pp. 787-794.
[2.63] Bonada J., Casafont M., Roure F., Pastor M.M. (2012). *Selection of the initial geometrical imperfection in nonlinear FE analysis of cold-formed steel rack columns*. Thin-Walled Structures, 51, pp. 99-111.
[2.64] Armani P., Baldassino N., Zandonini R. (2011). *Study of the response of uprights of pallet racks under compression*. Proc. of the 6th Int. Conf. on Thin Walled Structures, Timisoara, Romania, Vol. 2, pp. 772-778.
[2.65] Dubina D. (2011). *The ECBL approach for interactive buckling of thin-walled steel members*. Steel & Composite Structures, 1(1), pp. 75-96.
[2.66] Crisan A., Ungureanu V., Dubina D. (2012). *Behaviour of cold-formed steel perforated sections in compression. Part 1 – Experimental investigations*. Thin-Walled Structures (article in press).
[2.67] Crisan A., Ungureanu V., Dubina D. (2012). *Behaviour of cold-formed steel perforated sections in compression. Part 2 – Numerical investigations*, Thin-Walled Structures (article in press).
[2.68] EN15512:2009: *Steel static storage systems – Adjustable pallet racking systems – Principles for structural design*, CEN, Brussels, 2009.
[2.69] *European Recommendation for the Design of Light Gauge Steel Members*, ECCS, Brussels, 1978.
[2.70] EN1090-2:2008. *Execution of steel structures and aluminium structures - Part 2: Technical requirements for steel structures*, CEN, Brussels, 2009.
[2.71] AS4100-1990: *Australian Standard: Steel Structures*, Homebush, Australia.
[2.72] Ungureanu V., Dubina D. (2012). *Sensitivity to imperfections of perforated pallet rack sections*. Stability of Structures XIII-th Symposium. 17-21 September 2012, Zakopane, Poland (in print).
[2.73] Dubina D., Ungureanu V., Crisan A. (2012). *Experimental evidence of erosion on critical interactive distortional – overall buckling load*. Journal of Structural Engineering – ASCE (article in press).
[2.74] Dubina D., Ungureanu V.: *Effect of imperfections on numerical simulation of instability behaviour of cold-formed steel members*, Thin-Walled Structures, 40(2002), 239-262.
[2.75] Dubina D., Ungureanu V., Nagy Zs., Nunes L., Pernes P.: *Imperfections sensitivity analysis of pitched roof cold-formed steel portal frames*. Proceedings of the International Colloquium on Stability and Ductility of Steel Structures, SDSS'Rio 2010, Rio de Janeiro, Brazil. 8-10.09.2010, ISBN 978-85-285-0137-7, pp. 929-936.
[2.76] FEM 10.2.08 – *Recommendations for the design of static steel racks in seismic conditions*, version 1.00, European Racking Federation, September 2010.
[2.77] Dubina D., Fülöp L., Ungureanu V., Nagy Zs. (2000). *Cold-formed Steel Structures for Single Storey Buildings*. International Conference on Steel Structures of the 2000's, 11-13 September 2000, Istanbul, Turkey, pp. 191-196.
[2.78] Dubina D., Ungureanu V. Mutiu M. (member of PUT research team) (2009). *Affordable House Project*. Beneficiary: ArcelorMittal Liege Research.
[2.79] Dubina D., Ungureanu V., Fülöp L., Zaharia R. (2005): *Research Advances in Cold-formed Steel Design:Contributions of "Timisoara School*. Sesiunea Ştiinţifică Festivă prilejuită de împlinirea a 60 ani de către Dl. Prof. Dr. Ing. Radu Băncilă, Editura Solnes, ISBN 973-729-035-6, pp.118-135.

[2.80] Dubina D., Ungureanu V., Dinu F., Nagy Zs. (2004). *Structură modulară din profile de oţel formate la rece pentru clădiri civile şi industriale*. Construcţii civile şi industriale, anul V, nr. 20, pp. 21-25.

[2.81] Dubina D., Ungureanu V., Nagy Zs. (2002). *Lightweight steel structures using Lindab cold-formed sections for residential and non-residential buildings*, Proceedings of LSCE 2002 - International IASS Symposium on Lightweight Structures in Civil Engineering, 24-28.06.2002, Warsaw, Poland, pp. 667-676.

[2.82] Johnston B.G. (1971). *Spaced steel columns*. Journal of Structural Division, ASCE, Vol. 97, No. ST5, pp. 1465-1479.

[2.83] Dubina D., Zaharia R., Ungureanu V. (2002). *Behaviour of built-up columns made of C sections connected with bolted C stitches*, Proceedings of the Stability and Ductility of Steel Structures – SDSS 2002, Budapesta, Ungaria, 26-28 September 2002, pp. 179-186.

[2.84] P100-92: *Code for Aseismic Design of Residential Buildings, Agrozootechnical and Industrial Structures*. Ministry of Public Works and Territory Planning, Romania, 1992.

[2.85] Dubina D., Ungureanu V., Georgescu M., Fülöp L. (2001) *Innovative cold-formed steel structure for restructuring of existing RC or masonry buildings by vertical addition of supplementary storey*. The 3rd International Conference on Thin-Walled Structures, Cracow, Poland, 5-7 June 2001, pp. 187-194.

[2.86] Dubina D., Georgescu M., Ungureanu V., Dinu F. (2000). *Innovative cold-formed steel structures for one storey penthouse superstructure of DATATIM-ALCATEL industrial building*. The 9th Int. Conference on Metal Structures – ICMS'2000, Timisoara, Romania, 19-22 October 2000, pp. 318-326.

Chapter 3: Sustainability of Constructions

[3.1] Egmond – de Wilde de Ligny E., Blok R., Kahraman I. (2009). *Achieving sustainable building and construction. Sustainable construction industry versus sustainable building design*. Sustainability of Constructions, Integrated Approach to Life-time Structural Engineering, COST C25, Proceedings of Workshop, Ed. Orizonturi Universitare, Timisoara, Romania, pp. 298-317.

[3.2] Brundtland G. (1987). *Our Common Future: The World Commission on Environment and Development.* Oxford: Oxford University Press.

[3.3] CIOB – The Chartered Institute of Building 2008. *Sustainability and Construction*. www.ciob.org.uk.

[3.4] Corus (2006). *Sustainable steel construction. The design and construction of sustainable buildings*. www.corusconstruction.com

[3.5] The Steel Construction Institute. *A comparative environmental life cycle assessment of modern office buildings*.

[3.6] VIRIDIS (2002). *The construction industry mass balance: resource use, wastes and emissions*.

[3.7] Dubina D., Ungureanu V., Mutiu M. (2007). *Sustainable building structures for housing.* Proceedings of the International Conference on Sustainable Buildings 2007: Sustainable Construction. Materials and Practices. Lisbon, Portugal, September 12-14. IOS Press, vol. 2, pp. 1096-1103.

[3.8] Dubina D., Ungureanu V., Mutiu M. (2007). *Sustainable mixed building technologies applied to residential buildings: some Romanian examples.* Proceedings of the First Cost C25 Workshop: Sustainability of Constructions, Integrated Approach to Life-time Structural Engineering. Lisbon, Portugal, September 13-15. IOS Press, pp. 3.93-3.102.

[3.9] 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, pp. 397-402.

[3.10] Braganca L., Mateus R., Koukkari H. (2007). *Assessment of building sustainability.* Proceedings of the First COST C25 Workshop: Sustainability of Constructions, Integrated Approach to Life-time Structural Engineering. Lisbon, Portugal, September 13-15, IOS Press, pp. 0.3-0.12.

[3.11] Plank R., Dowling J. (2003). *Sustainable Constructions. Steel Buildings*. BSCA, Publication No. 35/03.

[3.12] The Steel Construction Sector Sustainability Committee (2003). *Sustainable Steel Construction. Building a Better Future*, UK.

[3.13] ISO 14040 (2006), *Environmental management – Life cycle assessment – Principles and framework*, International Organization for Standardization, Geneva, Switzerland.

[3.14] 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, pp. 429-441.

[3.15] Glaumann M., Kahraman I. (2009). Basic life time considerations in design of buildings and constructions, *Proceedings of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering. Timisoara, Romania, October 23-24*, Orizonturi Universitare, pp. 333-353.

[3.16] prEN15643:2009. *Sustainability of construction works – Assessment of buildings*, European Committee for Standardisation, Brussels, Belgium.

[3.17] ISO14042:2006. *Environmental management – Life cycle assessment – Life cycle impact assessment*, International Organization for Standardization, Geneva, Switzerland.

[3.18] prEN15804:2008. *Sustainability of construction works - Environmental product declarations*, European Committee for Standardisation, Brussels, Belgium.

[3.19] prEN15978:2009. *Sustainability of construction works – Assessment of environmental performance of buildings*, European Committee for Standardisation, Brussels, Belgium.

[3.20] ISO14044:2006. *Environmental management – Life cycle assessment – Requirements and guidelines*, International Organization for Standardization, Geneva, Switzerland.

[3.21] 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 the First International Conference on Structures and Architecture (ICSA2010). Guimarães, Portugal, 21–23.06, pp. 1298-1306.

[3.22] Ungureanu V., Dubina D. (2006). *Case Study: Constantin's Family House, Ploiesti, Romania* (SP024a-EN-EU). Access Steel (www.access-steel.com).

[3.23] Ungureanu V., Ciutina A., Tuca I., Dubina D., Grecea D., Fulop L. (2011). *Structural and Environmental Performance of Steel Framed Houses: A Case Study*. Summary report of the Cooperative Activities. Cost Action C25. Volume 1: Integrated approach towards sustainable constructions, ISBN 978-99957-816-1-3, pp. 395-428.

[3.24] EN1993-1-1:2005. *Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*. European Committee for Standardization, Brussels, Belgium.

[3.25] EN1993-1-3:2006. *Eurocode 3 – Part 1-3: Supplementary rules for cold-formed thin gauge members and sheeting*. ECS, Brussels, Belgium.

[3.26] EN1993-1-8:2006. *Eurocode 3: Design of steel structures, Part 1-8: Design of joints*. European Committee for Standardisation, Brussels, Belgium.

[3.27] P100-1:2006. *Cod de proiectare seismica – Partea I – Prevederi de proiectare pentru cladiri*. Monitorul Oficial al Romaniei, Partea 1, No. 803 bis.

[3.28] Fülöp L.A., Dubina D. (2004). *Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading. Part I: Experimental research*. Thin Walled Structures, 42(2), pp. 321-338.

[3.29] Fülöp L.A., Dubina D. (2004). *Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading, Part II: Numerical modelling and performance analysis*. Thin Walled Structures, 42(2), pp. 339-343.

[3.30] Dubina D. (2008). *Behaviour and performance of cold-formed steel framed houses under seismic action.* Journal of Constructional Steel Research, 64(7-8), pp. 896-913.
[3.31] Dubina D., Fulop L.A., Aldea A., Demetriu S., Nagy Zs. (2006). *Seismic performance of cold-formed steel framed houses.* STESSA 2006 – Behaviour of Steel Structures in Seismic Zones. Taylor&Francis/Balkema, London, pp. 429-437.
[3.32] EN1990:2002. *Basis of structural design*. European Committee for Standardisation, Brussels, Belgium.
[3.33] Fülöp L., Dubina D. (2006). *Design criteria for seam and sheeting-to-framing connections of cold-formed steel shear panels.* Journal of Structural Engineering – ASCE. Vol. 132, No. 4.
[3.34] Fülöp L.A. (2003). *Contributions to the optimization of structural systems for one family houses with steel structures (in Romanian).* PhD Thesis, The "Politehnica" University of Timisoara.
[3.35] CR6-2006. Cod de proiectare pentru structuri din zidărie. Monitorul Oficial al Romaniei, Partea 1, Anul 174, No. 807 bis.
[3.36] SimaPro 7. *Software and database manual*. Amersfoort, The Netherlands, 2008, www.pre.nl.
[3.37] Ecoinvent Centre. *Ecoinvent data v1.3, Final reports Ecoinvent, 2000*, No. 1-15, Swiss Centre for Life Cycle Inventories, Dubendorf, On line at: www.ecoinvent.ch.
[3.38] Eco-indicator'99, 2000. *Eco-indicator 99 – Manual for Designers*, On line at: www.pre.nl/download.
[3.39] Ciutina A., Ungureanu V., Grecea D., Dubina D. (2009). *Family house: cold-formed steel framing and OSB cladding vs. masonry – Case study, Proceedings of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering. Timisoara, Romania, October 23-24*, Orizonturi Universitare, pp. 373-390.
[3.40] Ciutina A., Ungureanu V. (2009). *Family house: cold-formed steel framing and OSB cladding vs. masonry – LC Energy approach, Proceedings of COST C25 Workshop Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering. Timisoara, Romania, October 23-24*, Orizonturi Universitare, pp. 391-400.
[3.41] SR1907-1:1997. *Heating plants. Design heat requirements computation for buildings. Computation specifications*.
[3.42] SR1907-2:1997. *Heating plants. Design heat requirements computation for buildings. Design conventional indoor temperatures*.
[3.43] Dubina D., Ungureanu V., Ciutina A., Tuca I., Mutiu M. (2010). *Sustainable detached family house - case study*. Journal of Steel Construction. Design and Research. Vol. 3, pp. 154-162, ISSN 1867-0520.
[3.44] Dubina D., Ungureanu V., Ciutina A., Mutiu M., Grecea D. (2010). *Innovative sustainable steel framing based affordable house solution for continental seismic areas*. Proceedings of the First International Conference on Structures and Architecture – ICSA 2010, 21-23.07.2010, Guimaraes, Portugal, ISBN 978-0-415-49249-2, pp. 367-368+CD.
[3.45] CR0-2005. *Basis of structural design*. Ministry of Transport, Construction & Tourism. 2230/27 December 2005, Bucharest, Romania.
[3.46] Arghirescu C., Mutiu M., Dubina D., Ungureanu V. (2009). *Sustainable block of flats: a Romanian example*. Proceedings of the 1st International Exergy, Life Cycle Assessment, and Sustainability Workshop & Symposium (ELCAS), Nisyros, Greece, 4-6 June 2009, pp. 141-148.
[3.47] Ungureanu V., Ciutina A., Dubina D. (2011). *Life cycle analysis of a steel framed building in Romania*. Proceedings of the Final COST C25 Conference: Sustainability of Constructions – Towards a better built environment, Innsbruck, Austria, 3-5 February 2011, pp. 341-350.
[3.48] Energy Efficiency and Renewable Energy. U.S. Dept. of Energy. *Building Envelope – Technology Roadmap*. http://evanmills.lbl.gov/pubs/buildings-roadmap/envelope_roadmap.pdf.

[3.49] Ciutina A., Ungureanu V., Dubina D., Dinu F. (2011). *Integrated design of buildings, Proceedings of the Final COST C25 Conference: Sustainability of Constructions – Towards a better built environment*, Innsbruck, Austria, 3-5 February 2011, pp. 235-247.

[3.50] Ungureanu V., Ciutina A., Dubina D. (2011). *Sustainable steel framed building – Case study*. Proceedings of the 6th European Conference on Steel and Composite Structures. August 31 – September 2, 2011, Budapest, Hungary, pp. 1971-1976.

[3.51] Ungureanu V., Ciutina A., Dubina D. (2011). *Analyses of sustainability and environmental impacts of steel framed buildings – example from practice in Romania*. Proceedings of the 9th Nordic Symposium on Building Physics. Volume 3. 29.05-02.06.2011. Tampere, Finland, pp. 1347-1354.