Lightweight Steel Framing Houses in Seismic Areas
Behaviour features and needs for Design Technical Regulations

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COLD-FORMED STEEL IN RESIDENTIAL BUILDING

WHY STEEL FRAMED HOUSES?

Because:

• This is a complete sustainable technology – steel structures are 100% recyclable (It takes 6 old cars to produce enough steel for the structure of house);
• It enables the use of high performance thermo-energetic materials for cladding and finishing;
• It is a highly productive and qualitative technology both for fabrication and erection;
• It enables to obtain flexible partitions;
• It enables for further up-grade, modifications and/or development.
WHY STEEL FRAMED HOUSES IN SEISMIC AREAS?

Because:

- Excellent structural performance;
- Appropriate to apply Performance Base Design and enables for safe prediction of behavior under severe actions;
- Good robustness, easy to prevent by design the progressive collapse;
- In case, easy to repair;
- Because their high strength/weight ratios and good redundancy, there are the safest houses, provided it is well designed and executed.

STEEL FRAMED HOUSES:
CLASSICAL OR MODERN APPEARANCE

Cold-formed steel used in:

- Basement walls;
- Floor joists;
- Load bearing walls;
- Non-load bearing walls;
- Roof framing and trusses.

Construction types:

- Stick build;
- Panelized;
- Modular;
- Combination of above.
COLD-FORMED STEEL IN RESIDENTIAL BUILDING

- **Classical Appearance**

![Classical Appearance Image](image1)

COLD-FORMED STEEL IN RESIDENTIAL BUILDING

- **Classical Appearance**

![Classical Appearance Image](image2)
COLD-FORMED STEEL IN RESIDENTIAL BUILDING

• **Classical Appearance**

![Classical Appearance Image]

COLD-FORMED STEEL IN RESIDENTIAL BUILDING

• **Cassette Wall Structure**

![Cassette Wall Structure Image]
COLD-FORMED STEEL IN RESIDENTIAL BUILDING

• **CASSETTE WALL STRUCTURE**

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• **Wall Stud Structure**
COLD-FORMED STEEL IN RESIDENTIAL BUILDING

• Wall Stud Structure

INTRODUCTION TO SEISMIC DESIGN PRINCIPLES

DESIGN PHILOSOPHY:

• Dissipative behaviour;

• Non- or low- dissipative behaviour.
Design Criteria

• Ultimate limit state (collapse prevention)
• Damageability limit state (progressive design criteria in order to limit damage)
• Serviceability limit state

Dissipative structures

• Energy dissipation taking advantage of the post-elastic load bearing capacity of the structure;
• Design codes: earthquake load reduction through “q” factor.
## Behavior factor “q” (pr EN 1998-1)

<table>
<thead>
<tr>
<th>Design concept</th>
<th>Behavior factor “q”</th>
<th>Required ductility class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highly dissipative structures</td>
<td>$q \geq 4.0$</td>
<td>H (high)</td>
</tr>
<tr>
<td>Medium dissipative structures</td>
<td>$2.0 \leq q &lt; 4.0$</td>
<td>M (medium)</td>
</tr>
<tr>
<td>Structures with limited dissipation</td>
<td>$q = 1.5 - 2.0$</td>
<td>L (low)</td>
</tr>
</tbody>
</table>

**Question:** Thin walled steel structures are low dissipative?

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## Definition of the Earthquake Reduction Factor

- **Ductility**
  \[ q_\mu = \frac{F_e}{F_y} \]

- **Structural Overstrength**
  \[ q_s = \frac{F_y}{F_d} \]

- **Total reduction factor without overstrength**
  \[ q = q_\mu \cdot q_R \]

- **Total reduction factor with design overstrength**
  \[ q_{\text{total}} = q_\mu \cdot q_s = q_\mu \cdot q_{\text{Sd}} \cdot q_R \]

- **Total reduction factor with structural overstrength**
  \[ q = q_\mu \cdot q_R \]

- **q_{Sd}** - design overstrength
  \[ q_{Sd} = \frac{\alpha_u}{\alpha_1} \]

- **q_R** - structural overstrength
  \[ q_R = \frac{\alpha_u}{\alpha_l} \]
INTRODUCTION TO SEISMIC DESIGN PRINCIPLES

Cold-formed steel framing for housing are usually made of class 4 or 3 sections e.g.

- Slender sections prone to local buckling;
- Non-plastic;
- Non-dissipative.

Seismic design codes are either restrictive or penalizing the use of these structures in seismic zones.
INTRODUCTION TO SEISMIC DESIGN PRINCIPLES

Earthquake behaviour of a house building

Structural Behaviour of a Building

Floor Diaphragm

Structural Wall Panel

Roof Diaphragm

Ornamental elements Non-structural Walls

Steel Skeleton
- member properties
- stub spacing
- anchoring behaviour

Panel Dimensions
- length to height ratios

Finishing Material
- material properties
- one side, both sides
- type and position of fixings

Boundary Conditions
- corner details for structural panels

Bracing
- bracing typology
- fixing details
- pretensioning

Opening
- dimensions of door and window openings
- position of openings

Bracing Finishing Interaction
- added effect of bracing and exterior finishing

INTRODUCTION TO SEISMIC DESIGN PRINCIPLES

Shear Force Resisting Elements in a Wall Panel

Sheeting Panels

Seem Connectors

Frame

Upper Track

Stud

Blocking

Lower Track

Holdown

Anchor

Sheet-Frame Connectors
An extensive research program was carried out at the “Politehnica” University of Timisoara in order to evaluate and characterize the seismic performance of cold-formed steel framed houses:

- Tests and numerical simulations on shear panels including connection behavior;
- In-situ tests of a house structure in different stages of erection.
### Wall Panel Typologies

#### Resistance, Rigidity and Ductility of Load Bearing Wall Panels with Steel Skeleton

<table>
<thead>
<tr>
<th>Series</th>
<th>Panel Configuration</th>
<th>Cladding</th>
<th>Testing Method</th>
<th>Load Vel. (cm/min)</th>
<th>No. Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>-</td>
<td>-</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>I</td>
<td>Corrugated Sheet LTP20/0.5 (Ext)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>6</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>Corrugated Sheet LTP20/0.5 (Ext)</td>
<td>Gypsum Board (Int)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>6</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>Cross Bracing (Ext-Int)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>Corrugated Sheet LTP20/0.5 (Ext)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>6</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>OSB I</td>
<td>10 mm OSB Panels (Ext)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>OSB II</td>
<td>10 mm OSB Panels (ext)</td>
<td>Monotonic</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

**Total Number of Specimens**: 15
Testing Procedure

- Monotonic test, with loading velocity of 1cm/min;
- From monotonic curve, determination of the Conventional Elastic Limit Displacement ($\Delta_{el}$);
- Cyclic testing followed ECCS recommendation, ($\frac{1}{4}\Delta_{el}$, $\frac{1}{2}\Delta_{el}$, $\frac{3}{4}\Delta_{el}$, $1\Delta_{el}$, $3\times 2\Delta_{el}$, $3\times 4\Delta_{el}$, $3\times 6\Delta_{el}$,...), until significant decrease of capacity.

**Experimental Load-Displacement Curves**

- Characteristic Curves - Series I
- Characteristic Curves - Series II
- Characteristic Curves - Series III
- Characteristic Curves - Series IV
- Characteristic Curves - Series OSB I
- Characteristic Curves - Series OSB II
TESTING ON WALL PANELS

Failure Modes

Steel sheeting to steel framing  
OSB panels to steel framing

CALIBRATION OF SIMPLIFIED HYSTERETIC MODEL

Testing: Characteristics of the Cyclic Behaviour

- Characteristics of the hysteretic behaviour:
  - Nonlinearity;
  - Strong pinching;
  - Low Strength and Stiffness degradation;
  - Performance controlled by connection.
CALIBRATION OF SIMPLIFIED HYSTERETIC MODEL

Testing: Equivalent models

**Method I - Method II**

- $K_0$ up to the value of $F_{el}$ ($0.4F_{max}$);
- $0.1K_0$ rigidity tangent to the experimental curve;
- $F_u$ at intersection;
- $D_u$ at intersection with the downloading branch;
- $D_{400}$ corresponding to $1/400$ rad panel top rotation;
- Initial stiffness ($K_0$), stiffness up to $F_{el}$ ($D_{400}$);
- $F_u$ so as shaded areas are equal;
- $D_u$ at intersection with the downloading branch;

CALIBRATION OF SIMPLIFIED HYSTERETIC MODEL

Testing: Initial Rigidity ($K_0$)

- Because of Lintel, rigidity underevaluated for specimens with openings;

[Graph showing rigidity values for different methods and specimens]
Testing: Ultimate Force ($F_u$)

- Strength degradation for all specimens;
- Higher values for OSB specimens;

Testing: Ductility ($D_{uct} = D_u/D_y$)

- Unexpected corner failure for specimen III-1;
- Lower ductility provided by OSB specimens;
Testing: Overstrength ($F_u/F_{design}$)

- $F_{design}$ at minimum of $2/3F_u$ and $F_{300}$ (1/300 rad);
- Overstrength in the range of 1.2-1.6;
- Values less relevant for panels with openings;

Numerical: Proposed Model


- The panel is replaced by an equivalent brace system with similar hysteretic behavior (DRAIN-3DX);
- Capabilities of the model:
  - Pinching;
  - Non-linearity;
  - NO strength degradation.
CALIBRATION OF SIMPLIFIED HYSTERETIC MODEL

NONLINEAR MODEL

(Richard-Abbott model parameters)

(University of Naples, R. Landolfo et al.)

\[ u = \frac{(k_0 - k_i)d}{1 + \left(\frac{(k_0 - k_i)d}{u_0}\right)^n} + k_i d \]

- Capacity limit of the Fulop’s FE model has been considered based on stabilized envelope curves;
- If few plastic are considered it can be a simple and safe approach to consider strength degradation;

NUMERICAL: COMPARISON OF EXPERIMENTAL TO FE CURVE
**Numerical: Interpretation of Limit States**

- **A** - envelope curve
- **B** - return path

- Yield limit
- Elastic limit

\[
D_{ucl} = \frac{D_u}{D_y}, \quad \text{O}_{strength} = \frac{F_u}{F_{design}}
\]

---

**Numerical: Panel Characteristics for the Analysis**

<table>
<thead>
<tr>
<th>Series</th>
<th>I</th>
<th>II</th>
<th>IV</th>
<th>OSB I</th>
<th>OSB II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Panel Scheme</td>
<td>![Symbol]</td>
<td>![Symbol]</td>
<td>![Symbol]</td>
<td>![Symbol]</td>
<td>![Symbol]</td>
</tr>
<tr>
<td>Initial Rigidity (N/mm)</td>
<td>3446.6</td>
<td>3850.6</td>
<td>1766.3</td>
<td>4197.3</td>
<td>1610.5</td>
</tr>
<tr>
<td>Elastic Limit (F_{eu}) (N)</td>
<td>24086</td>
<td>26566</td>
<td>128670</td>
<td>28942</td>
<td>11850</td>
</tr>
<tr>
<td>Yield Limit (F_{yield}) (N)</td>
<td>33560</td>
<td>39819</td>
<td>26812</td>
<td>48944</td>
<td>33908</td>
</tr>
<tr>
<td>Yield Limit (D_{yield}) (mm)</td>
<td>14.95</td>
<td>15.58</td>
<td>23.78</td>
<td>17.49</td>
<td>27.76</td>
</tr>
<tr>
<td>Ultimate Limit (D_{ult}) (mm)</td>
<td>42.61</td>
<td>57.29</td>
<td>94.35</td>
<td>42.85</td>
<td>65.57</td>
</tr>
<tr>
<td>Duct</td>
<td>4.37</td>
<td>5.54</td>
<td>6.22</td>
<td>3.67</td>
<td>3.11</td>
</tr>
</tbody>
</table>
Numerical: Spectra of EQ Records for Time History Analysis

- Time history analysis using the SDOF FE models;
- Masses of 2, 2.5, 3, 3.5 and 4 t;
- Records scaled from 0.05g to 2g (IDA);

Numerical: Examples of Dynamic Response at Ultimate State

- Main characteristics of the hysteretic behavior are covered;
- Few plastic excursions in ultimate state (3-5);
- The approach with the stabilized envelope curves seems to be safe and reasonable;
Important difference between earthquake level at the elastic and yield limit (overstrength);

Less between yield and ultimate (ductility);
Numerical: Summary of ‘q’ Values

<table>
<thead>
<tr>
<th>Series</th>
<th>I</th>
<th>II</th>
<th>IV</th>
<th>OSB I</th>
<th>OSB II</th>
</tr>
</thead>
<tbody>
<tr>
<td>q₁</td>
<td>2.5</td>
<td>2.7</td>
<td>3.2</td>
<td>2.66</td>
<td>3.78</td>
</tr>
<tr>
<td>q₂</td>
<td>1.46</td>
<td>1.65</td>
<td>2.36</td>
<td>1.38</td>
<td>1.88</td>
</tr>
<tr>
<td>q₃</td>
<td>3.62</td>
<td>3.61</td>
<td>7.65</td>
<td>3.67</td>
<td>6.96</td>
</tr>
</tbody>
</table>

Wall Panel Results

\[ q_\mu = 1.4 \text{ – } 1.6 \]
\[ q_s = 2.2 \text{ – } 2.6 \]
\[ q_d = 3.0 \text{ – } 4.0 \]

The performance of the wall panels is governed by the performance of seam and sheeting to skeleton fasteners.
Tests: Performance criteria based on fastener slip

FO - Fully Operational: elastic deformations and small plastic elongation in fastener holes (drift<0.003);

PO - Partially Operational: local repairs of the sheeting system required against waterproofing (drift<0.015);

SR - Safe but Repairs required: replacement of the sheeting is necessary, but the building is safe against collapse (drift<0.025);

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum slip in fasteners (mm)</th>
<th>Panel top displacement (mm)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-3</td>
<td>0.197</td>
<td>6.71</td>
<td>0.274</td>
</tr>
<tr>
<td>I-3</td>
<td>4.8</td>
<td>29.22</td>
<td>1.197</td>
</tr>
<tr>
<td>IV-2</td>
<td>0.197</td>
<td>7.96</td>
<td>0.326</td>
</tr>
<tr>
<td>IV-2</td>
<td>4.8</td>
<td>44.13</td>
<td>1.808</td>
</tr>
<tr>
<td>IV-3</td>
<td>0.197</td>
<td>8.11</td>
<td>0.332</td>
</tr>
<tr>
<td>IV-3</td>
<td>4.8</td>
<td>42.22</td>
<td>1.730</td>
</tr>
</tbody>
</table>
STEEL-TO-STEEL CONNECTIONS

- Representative for connection typologies used in the wall panels, two types of specimens:
  - corrugated sheet to skeleton (0.417 mm to 1.42 mm) using SD3-T15-4.8×22 (4.8 mm) screws (Series I-TP)
  - corrugated sheet to corrugated sheet (0.417 mm to 0.417 mm) using SL2-T-A14-4.8×20 (4.8 mm) screws (Series I-TP)
- Dimensions according to the ECCS P21, facilitate bearing failure of the thinner sheet;
- With the same materials and in similar conditions as the connections in the panels;
- Two loading velocities:
  - \( V_1 = 1 \text{ mm/min} \) for quasi static loading conditions
  - \( V_2 = 420 \text{ mm/min} \) for high velocity tests.

PERFORMANCE OF CONNECTIONS

FAILURE MODES AND CHARACTERISTIC CURVES FOR “I-TP” SERIES

Failure mode for series I-TP-V1
PERFORMANCE OF CONNECTIONS

FAILRE MODES AND CHARACTERISTIC CURVES FOR “I-TT” SERIES

I-TT-V1

I-TT-V2

PERFORMANCE OF CONNECTIONS

OSB-TO-STEEL CONNECTIONS

• Very inhomogeneous results;
• No conclusion can be drawn;
• OSB connections possess less ductility.
DESIGN CRITERIA FOR CONNECTIONS

- Significant ductility of steel-to-steel connections with the possibility to use multi level design criteria;
- Important overstrength for steel-to-steel connections;
- In case of the OSB-to-steel connections failure is non ductile without possibility to use multi level design criteria;

CONSTRUCTING THE FE MODEL

- The FE model takes into account the:
  - Elastic shear deformation characteristics of the corrugated sheeting (including end distortion);
  - Elastic deformation characteristics of the skeleton;
  - Non-linear deformation characteristics of sheeting-to-framing and seam connections;
  - Non-linear characteristics of hold-down details.
DEFORMATION OF THE FE MODEL

PERFORMANCE OF CONNECTIONS

DESIGN CRITERIA TRANSLATED FROM CONNECTION TO WALL-PANEL LEVEL

- The design criteria for connections determines the criteria for panels
- Because low degradation of strength and stiffness in cyclic, monotonic analysis might apply
In-situ tests of a house structure in different stages of erection

IN-SITU MEASUREMENTS

Structural design based on tested panels

Steel skeleton of the structure

The skeleton with the structural OSB sheeting
Design assisted by testing

\[ E_{s,i} < R_{s,i} \]
\[ R_{s,i} = R_k \cdot L_i \]

where,

- \( E_{s,i} \) = total shear force induced by seismic action on “i” direction (kN);
- \( R_{s,i} \) = total shear wall resistance on “i” direction (kN);
- \( R_k \) = characteristic strength of shear wall (kN/m);
- \( L_i \) = length of shear wall on “i” direction (m).
IN-SITU MEASUREMENTS

Design assisted by testing

Three subsequent erection stages
- Steel framing only;
- Steel framing with OSB panels;
- Steel framing with finishing.

Modal dynamic characteristic compared:
Numerical vs. measures

Transversal:
$L_{gf} = 21.55$ m, $L_{ff} = 17.12$ m

Longitudinal:
$L_{gf} = 17.22$ m, $L_{ff} = 12.42$ m
### Modal parameters obtained by FE modeling

<table>
<thead>
<tr>
<th>Stage</th>
<th>T(_1) (s)</th>
<th>Type*</th>
<th>T(_2) (s)</th>
<th>Type*</th>
<th>T(_3) (s)</th>
<th>Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.44</td>
<td>Tr</td>
<td>0.39</td>
<td>To</td>
<td>0.35</td>
<td>Lo</td>
</tr>
<tr>
<td>2</td>
<td>0.19</td>
<td>Lo</td>
<td>0.18</td>
<td>Tr</td>
<td>0.13</td>
<td>To</td>
</tr>
<tr>
<td>3**</td>
<td>0.33</td>
<td>Lo</td>
<td>0.31</td>
<td>Tr</td>
<td>0.23</td>
<td>To</td>
</tr>
</tbody>
</table>

* Note\(^1\): Tr – transversal, Lo – longitudinal, To - Torsional  
** Note\(^2\): Only the masses changed from Stage 2 to Stage 3

** Mode shapes in Stage 2 (FE modeling):**  
(a) first mode - T\(_1\), (b) second mode - T\(_2\), (c) third mode - T\(_3\)
In-situ measurements (1)

Construction Stage 1 and measurement sensor location on the skeleton of the structure

In-situ measurements (2)

Construction Stage 2 and measurement sensor location
IN-SITU MEASUREMENTS

In-situ measurements (3)

Construction Stage 3 and measurement sensor location

IN-SITU MEASUREMENTS

Modal parameters based on the analysis of ambient vibrations (in different stages “S”)

<table>
<thead>
<tr>
<th>S</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_1$ (s)</td>
<td>$\zeta_1$ (%)</td>
<td>Type</td>
</tr>
<tr>
<td>1</td>
<td>.546</td>
<td>1.2</td>
<td>Lo</td>
</tr>
<tr>
<td>2</td>
<td>.103</td>
<td>3.4</td>
<td>Tr</td>
</tr>
<tr>
<td>3</td>
<td>.101</td>
<td>4.1</td>
<td>Tr</td>
</tr>
</tbody>
</table>

*Note: Vibration shapes: $\text{Tr}$ – transversal, $\text{Lo}$ – longitudinal, $\text{To}$ – Torsional
Conclusions

• Very good 2D behaviour of the wall:
  • Robust, e.g. high redundancy;
  • Moderate ductility available;
  • Performance dominated by connection behaviour.
• Structural design based on calibrated panels, e.g.
  stiffness vs. capacity, confirmed
• Significant contribution of finishing
• Damping ratio of 5% confirmed.

Conclusions: Design Recommendations

• Cold formed steel framed houses will be designed as low dissipative structures
• $q = 1.5-2.0$, and general rules for conceptual seismic design –
  EN 1998-1: regularity, continuity of axes, low stiffness and mass eccentricity, ductile connecting technology
• Elastic design of steel framing according to EN 1993-1.1, EN 1993-1-3, EN 1993-1-8
• Standard detailing
• Prescriptive method design: pre-calibrated wall-stud panel units, numerical or tests for all-steel solutions, tests for composite; calibrated models for connections always.
• Lightweight flooring – dry or light concrete; Lightweight envelope; diaphragm capacity available enabling for box-type behaviour of the building
• Proper foundation design