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Ductility and Seismic Performance of Thin-walled Cold-formed Steel Structures

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Abstract

Cold-formed steel structures are usually made with thin-walled sections of class 3 or 4 steel. Traditionally these structures, being slender, are considered non-ductile and plastic design is not allowed. As a consequence, only a reduction factor q=1 and elastic design can be applied. This paper summarizes recent research in the field and demonstrates that thin-walled steel structures can be considered as "low dissipative" according to the EN 1998-1 classification, which means a *q* factor value of 1.5 to 2 may be used for seismic design.

Keywords: steel sections, cold-formed, thin-walled, local instability, ductility, cyclic loading

1. Introduction

Cold-formed steel structures are usually made with thin-walled sections, of class 4 steel or, at most, class 3. Compared with hot-rolled sections (of class 1 or 2), they are characterized by reduced post-elastic strength and, as a consequence, by reduced ductility (e.g., they do not have sufficient plastic rotation capacity to form plastic hinges).

The European specific design rules for cold-formed steel design have no recommendations for seismic design of these structures. In the North American Specification (AISI, 2001), the provisions in Section G, "Design of cold-formed steel structural members and connections for cyclic loadings," refers to fatigue rather than to seismic behavior. The 2003 draft of the Australian/New Zealand Standard (Revision of AS/NZS 4600:1996) for Cold-formed Steel Structures in Section 6 "Fatigue" includes similar provisions as AISI 2001. Therefore, actual cold-formed steel design codes do not contain specific recommendations for seismic design of cold-formed steel structures.

EN 1998-1 does not specifically mention the use of thin-walled steel sections for seismic resistant structures. However, it provides low dissipative (e.g. low ductility) structures with a behavior factor *q* of values from 1.5 to 2.0 (see Table 1). Assuming that these structures are made by "elastic" sections (e.g., class 3 or class 4), a *q* factor greater than 1.0 can be justified by overstrength and structural redundancy. The question the present paper attempts to answer is whether or not thin-walled structures can be classified as "low dissipative."

2. Elastic-plastic Behavior of Thin-walled Sections in Post-critical Range

2.1. Post-critical strength of thin plates and sections

The behavior of an ideal and an actual plate is shown in Fig. 1. Examining the path of an ideal plate stress-deflection curve, it can be observed:

1. In the pre-critical range $\sigma > \sigma_{cr}$, the plate has linear behavior characterized by a plane stress state;
2. When the critical stress point is reached, $\sigma = \sigma_{cr}$, the plate suddenly loses its rigidity (see Fig. 1c) and a significant increase in deflection occurs;
3. In the post-critical range $\sigma_{cr} < \sigma < f_y$, the behavior continues to remain elastic and due to "membrane lag" effect, a stabilizing action occurs from which a post-critical stress reserve is available. This "membrane lag" is the explanation for the non-linear elastic behavior within this range;
4. When the first yield is reached in the point $\sigma = \sigma_{pl}$ the curve changes the curvature, and the plate starts its elastic-plastic behavior. In the domain $\sigma \leq \sigma_{pl}$, the unloading path is fully reversible. For this reason, the point $\sigma = \sigma_{pl}$ is also called the "reversibility" point;
5. In the range $\sigma > \sigma_{pl}$, the plate rapidly loses its stiffness and reaches the ultimate strength, σ_u.

When both geometrical and material imperfections are present, a continuous deformation process starts from the beginning (Fig. 1); the more the initial deflection w_0

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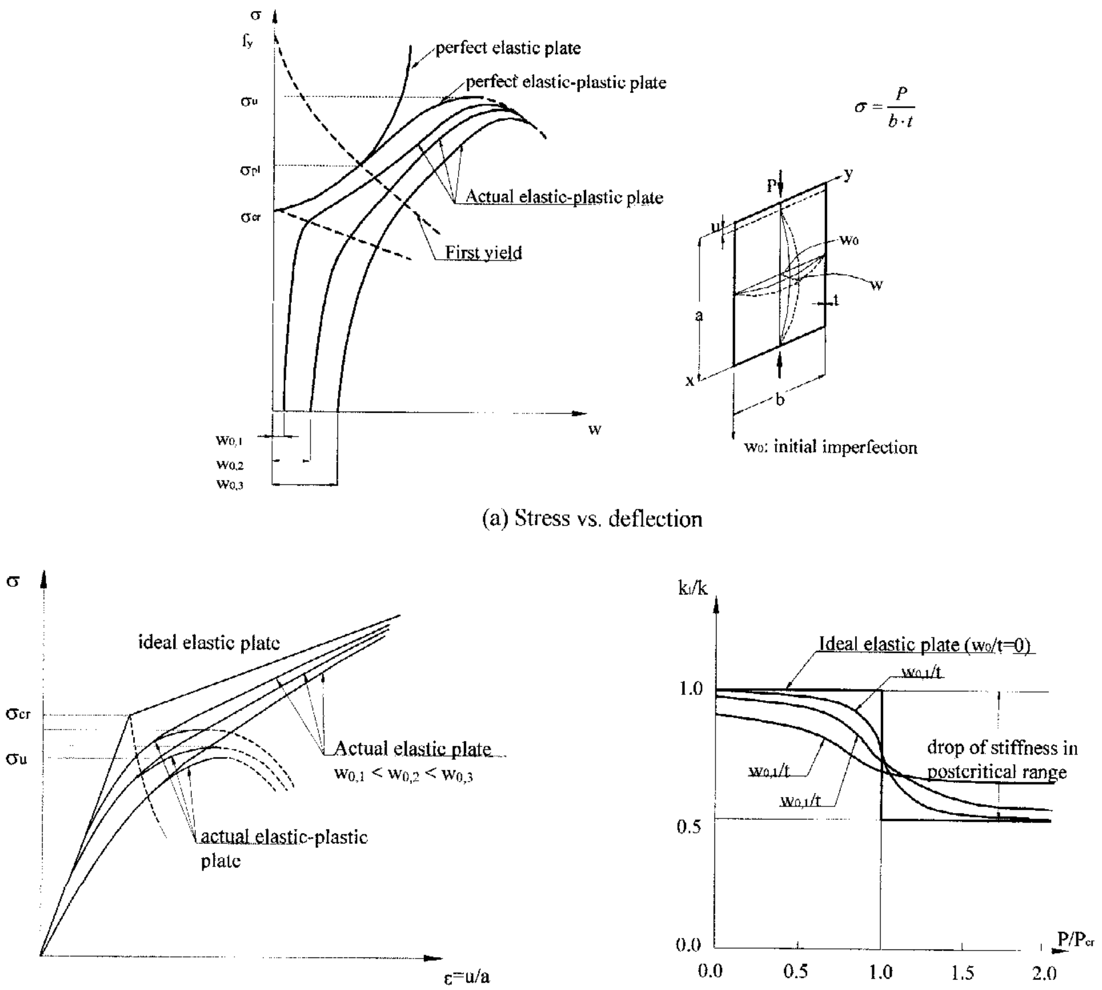
Table 1. Behavior factor "q" (per EN 1998-1)

Design concept	Behavior factor "q" EC8	Required ductility class
Highly dissipative structures	q≥ 4.0	H (high)
Medium dissipative structures	2.0 ≤ q ≤ 4.0	M (medium)
Structures with limited dissipation	q = 1.5 ~ 2	L (low)



(a) Stress vs. deflection

(b) Stress vs. axial strain

(c) Axial stress vs. stiffness

Figure 1. Behavior of ideal and actual simply-supported plate in uniaxial stress.

increases, the smoother the σ-w curve becomes. It is difficult to capture the points σ_{cr} and σ_{pl}, and often during tests, σ_u is taken as σ_{cr} and vice versa. Therefore, for the actual thin plate, the σ_{cr}-σ_u range is significantly reduced.

In the case of thin-walled bars, the sectional buckling (e.g., local or distortional buckling) occurs prior to the initiation of plastification. Sectional buckling is characterized by the stable post-critical path and the bar does not fail when it occurs, but significantly loses its stiffness. The yielding starts at the corners of the cross-section a few times before the failure of the bar, then the sectional buckling changes into a local plastic mechanism quasi-simultaneously with the occurrence of global buckling.

2.2. Localization of buckling patterns and the local buckling mechanism

For stub columns, multiple local buckling modes may occur simultaneously under the same critical load. For a slender member, multiple local buckling modes, e.g., $m-1$, m, $m+1$, characterized by corresponding L_{m-1}, L_m and L_{m+1} half wave lengths, may interact with each other and create an unstable post-critical behavior called "localization of the buckling pattern."

The localized buckling mode is in fact an interactive or

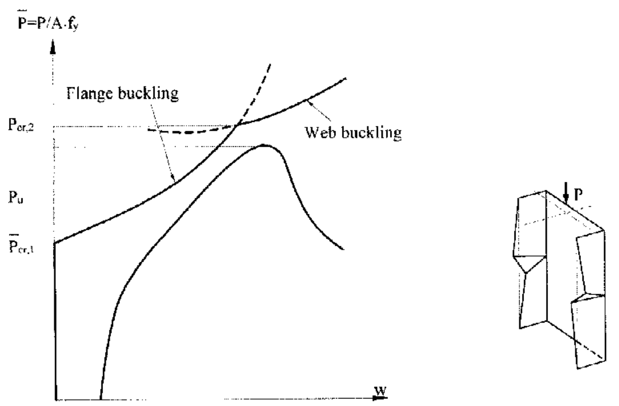


Figure 2. Local plastic mechanism failure of a plain channel stub column.

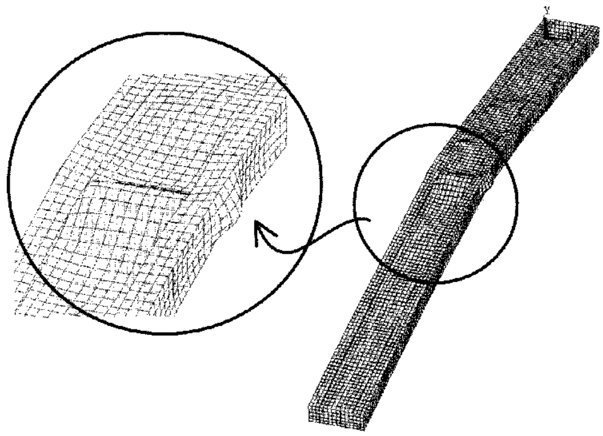


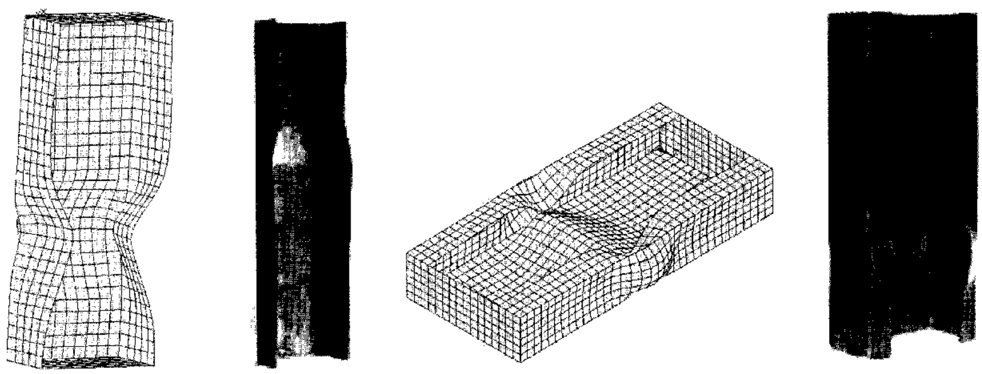
Figure 4. FEM simulation of plastic-elastic interaction between the local plastic mechanism (roof type) and flexural buckling for a lipped channel section.

coupled mode. This "first" interaction may occur prior to the overall buckling of the member. The "second" interaction, between the localized mode and the overall one is very dangerous because it is accompanied by a strong erosion of the critical bifurcation load. When localization of the buckling patterns occurs, the member post-buckling behavior is characterized by large local displacements in the inelastic range, which produce the plastic folding of walls and the member falls into a plastic mechanism (Fig. 2). This kind of behavior is specific to cold-formed steel sections and is confirmed by both tests and numerical simulations (Fig. 3), and could be the source of some ductility.

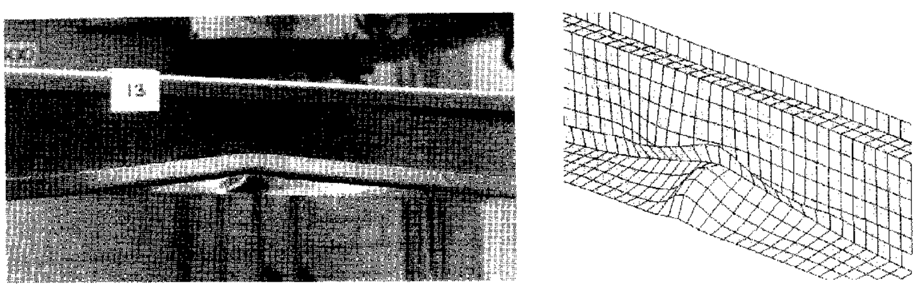
For slender bars, when local buckling first appears, it always changes into local plastic mechanism when the member fails (see Fig. 4).

2.3. Plastic rotation capacity of thin-walled sections

Ungureanu and Dubina (2004) used the local plastic mechanism theory developed by Murray and Khoo



(a) members in compression



(b) members in bending

Figure 3. Numerical and experimental evidence of plastic mechanism failure.

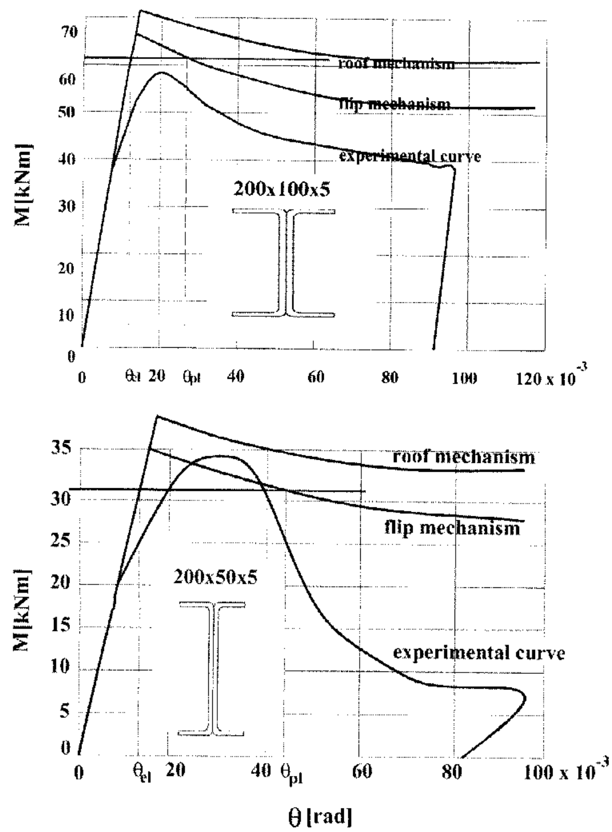


Figure 5. M-θ curves for built-up sections.

(1981) to characterize the local and interactive buckling of cold-formed steel sections. Moldovan *et al.* (1999) used the same theory to evaluate the plastic rotation capacity (e.g., ductility) of built-up U and C cold-formed steel beams. They developed the DUCTROT-TWM computer code, calibrated via test results obtained at the University of Naples (DeMartino *et al.*, 1992) and found an available cross-sectional ductility, μ, equal to 1.7:

$$\mu = \chi_u/\chi_y \tag{1}$$

where χ_y is the beam curvature corresponding to the initiation of plastic deformations, while χ_u is the ultimate curvature.

Fig. 5 shows the moment-rotation curves for two sections studied by Moldovan *et al.* (1999). One can see for these particular sections that the "flip" type local mechanism better approximates the ultimate bending moment. Based on these results, the authors suggested using a behavior factor of q = 1.7 when designing cold-formed steel structures.

2.4. Post-elastic behavior and fatigue

The North American Specification (AISI, 2001) in Section G "Design of cold-formed structural members and connections for cycling loadings," and the revised version of the Australian and New Zealand Standard (Revision of AS/NZS 4600:1996) in Section 6 "Fatigue," as previously mentioned, are the only modern cold-formed steel design codes to address the problem of fatigue behavior (e.g., cycling loading effect). However, the relevant provisions in these codes refer mainly to connecting details and not to the member strength under cyclic loading, for either high or low fatigue circumstances.

Both codes consider:

- the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration of fatigue design;
- an evaluation of fatigue resistance is not required if the number of cycles of application of live load is less than 20,000;
- calculated stresses shall be based upon elastic analysis.

For the purpose of this paper, it is useful to emphasize that in recent years, the low fatigue approach, accounting for cumulative plastic deformations of steel members or connections under repeated and reversal loadings, is considered a better way to evaluate the seismic response of steel structures.

Related to the particular problem of fatigue behavior of thin-walled, cold-formed steel sections, Lindner and Gläßer (2004) show that, in such a case, the fatigue damage is a plastic problem rather than an elastic one. In fact, the tests carried out by the authors clearly demonstrated that failure under cycling loads occurred only if plastic strains were present. When the ultimate load in the test was reached, cracks occurred along the plastic hinges of a local plastic mechanism, as shown in Fig. 6.

Practically no damage occurred in the tested specimens, even after 2 million elastic cycles. This means that thin-walled cold-formed steel members are not sensitive to the effects of cyclic loading, providing they remain in the elastic range.

Calderoni *et al.* (2004) tested cyclically built-up double plain channel beams (200 x 50 x 3) and applied the "low cycle fatigue" approach to interprete the results. The conclusions show that even with significant strength degradation during cycles due to local buckling, these sections provide a plastic rotation capacity around 0.05 rad and corresponding ductility.

3. Behavior Factors. Ductility, Overstrength and Redundancy

Earthquake force reduction factors are widely used in design codes to reduce elastic spectral demand to design ones. Structural design to earthquake forces lower than those necessary for an elastic response are derived from the observation that most structures are able to survive a major earthquake due to dissipation of energy by plastic excursions and overstrength.

In the traditional "capacity design" procedure, a single

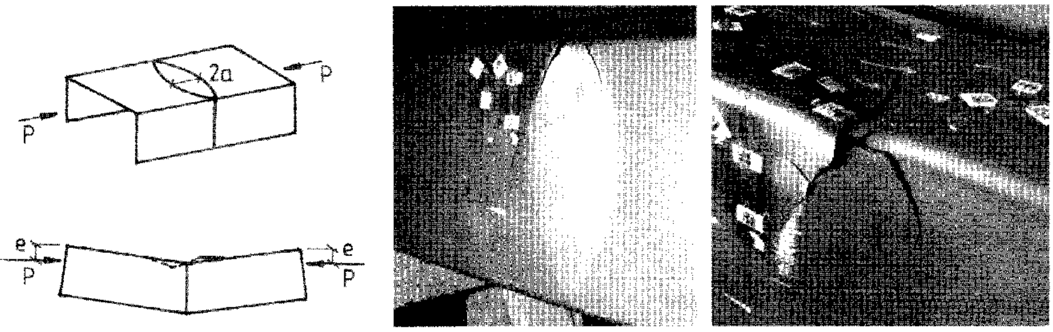


Figure 6. (a) *flip disc* in web; (b) *flip disc* in test specimen X; (c) Crack start in web hinges (Lindner and Gläβer, 2004).

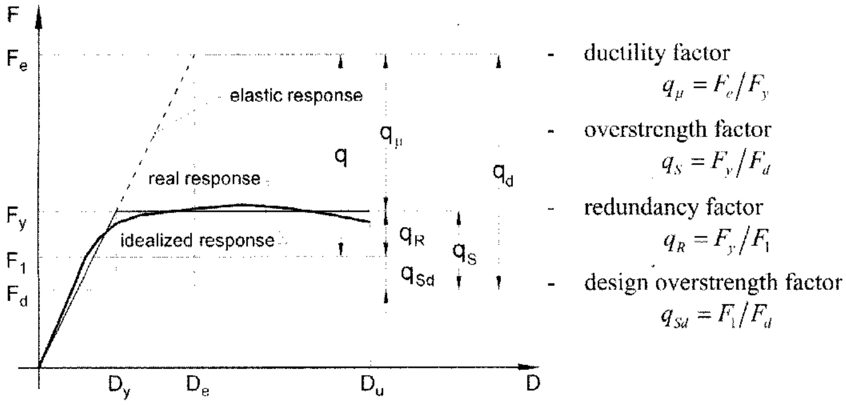


Figure 7. Definition of force reduction factors (Fischinger and Fajfar, 1994).

reduction factor is generally used. However, distinction and quantification of different components of the force reduction factors are useful for a better understanding of the seismic response of structures. Fig. 7 presents a typical relationship between base shear and top displacement of a structure. Based on the bilinear idealization of the real response, the ductility may be defined as:

$$\mu = D_u/D_y \tag{2}$$

where D_u is the ultimate top displacement, and D_y is the top displacement at global yield. Other terms used in the figure are: F_e-elastic base shear; F_y-yield base shear; F_1-base shear at the first plastic hinge; F_d-design base shear.

The total reduction factor used in design is:

$$q_d = q_\mu \cdot q_S = q_\mu \cdot q_{Sd} \cdot q_R \tag{3}$$

The redundancy factor q_R used herein represents the plastic redistribution capacity of the structure (the α_u/α_1 ratio of Eurocode 8). It is well known that thin-walled steel sections do not possess a significant post-elastic strength. Therefore, assuming local plastic mechanisms instead of plastic hinges, the available redundancy of structures made by these types of sections is based only on their hyperstatic characteristics.

For slender structures, using an equivalent static elastic-plastic analysis, q factor can be evaluated with the following formula (Mazzolani and Piluso, 1996):

$$q = \frac{\alpha_u}{\alpha_1}[(1-\beta')\alpha_{cr}+\beta'] \tag{4}$$

where $\beta' = 1 - T$; $\beta' \geq 0.5$; T- is the fundamental period of structure; and α_{cr}- is the critical load multiplier of gravitational loads, V (e.g. $\alpha_{cr} = V_{cr}/V$).

4. Cold-formed Steel Wall Stud Shear Panels

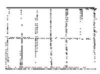
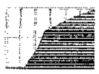
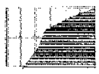
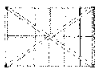
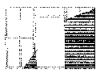
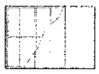
Light gauge steel buildings, both for residential and non-residential purposes, are usually made by cold-formed sections framing, designed to carry gravitational loading, and with shear walls to resist horizontal forces from wind and earthquake loading. For seismic response, the performance of shear walls is crucial. Significant research in the field and important findings have been previously obtained in the US (Serette and Ogunfunmi, 1996; Serette, 1998, Salenicovich, 2000), Japan (Kawai *et al.*, 1999), Australia (Gad *et al.*, 1999, 2000) and Europe (de Matteis, 1998). In addition, these findings have been reviewed by Fulop and Dubina (2004a).

In the following sections, the research program on characterizing the seismic performance of wall-stud shear panels (Fulop and Dubina, 2004a,b) carried out at P.U. Timisoara is summarized.

4.1. Experimental program at P.U. Timisoara

Six series of full-scale wall panels (3600 mm x 2440

Table 2. Description of wall specimens

Ser.		Open.	Brac.	Exterior Cladding	Interior Cladding	Testing Method	Load Vel. (cm/min)	No. Test
O		-	-	-	-	Monotonic	1	1
I		-	-	Corr. Sheet LTP20/0.5	-	Monotonic	1	1
						Cyclic	6;3	2
II		-	-	Corr. Sheet LTP20/0.5	Gypsum Board	Monotonic	1	1
						Cyclic	6;3	2
III		-	Yes	-	-	Monotonic	1	1
						Cyclic	3	1
IV		Door	-	Corr. Sheet LTP20/0.5	-	Monotonic	1	1
						Cyclic	6 ;3	2
OSB I		-	-	10 mm OSB	-	Monotonic	1	1
						Cyclic	3	1
OSB II		Door	-	10 mm OSB	-	Monotonic	1	1
						Cyclic	3	1
				Total Number of Specimens				15

mm), made by cold-formed wall-stud skeleton and different cladding arrangements commonly used for residential buildings, have been tested in the Laboratory of Steel Structures of the Politehnica University of Timisoara, Romania.

Table 2 describes the specimens. Details about materials and fabrication technology of the specimens are presented in Fulop and Dubina (2004a). Cyclic loading was introduced according to ECCS Recommendations (1985).

Figure 8. summarizes the main results of cyclic tests and presents a comparison with monotonic curves of tested specimens from the six series.

4.2. Seismic performance

Test results have been used to calibrate a FEM model by applying Incremental Dynamic Analysis to obtain the values of reduction factors q_{μ}, q_{Sd}, and q, respectively. The main results from this research are summarized below.

Shear-resistance of wall panels is significant in terms of both rigidity and load bearing capacity, and can be effective against lateral load. The hysteretic behavior is characterized by very significant pinching, and therefore reduced energy dissipation.

The seam fasteners represent the most sensitive part of the corrugated sheeting specimens; damage is gradually increased in seam fasteners, until their failure causes the overall failure of the panel. Much of the post-elastic deformation of the panel is in the region of seam fasteners, therefore increasing the load capacity and ductility of the seams will improve the behavior of the panels.

Figure 9 (a and b) show the failure modes for panels with corrugated sheeting and OSB, respectively.

An additional testing program on connection specimens has been carried out in order to determine design criteria for fasteners. Tests on sheeting-to-frame fasteners, sheeting-to-sheeting fasteners (seam) and OSB-to-frame fasteners have been performed using two different loading velocities, i.e.

v_1 = 1 mm/min (quasi-static)
v_2 = 420 mm/min (high seismic strain rate)

Figs. 10 and 11 summarize these tests.

For wall panels with corrugated sheeting, the main damage was concentrated in the seam fasteners. It is important to establish an acceptable level of deformation at the connection level and, for different wall typologies, relate this to the overall deformation of the wall panel. To establish global performance criteria, the following acceptable deformations in the seam fasteners are suggested:

-If slip of the seams does not exceed the elastic limit (D_e, Fig. 10), corresponding to $0.6F_{max}$ of the seam connection, damage is limited and can be considered negligible. In this case, the integrity of the cladding is fully preserved and no repairs are required; it

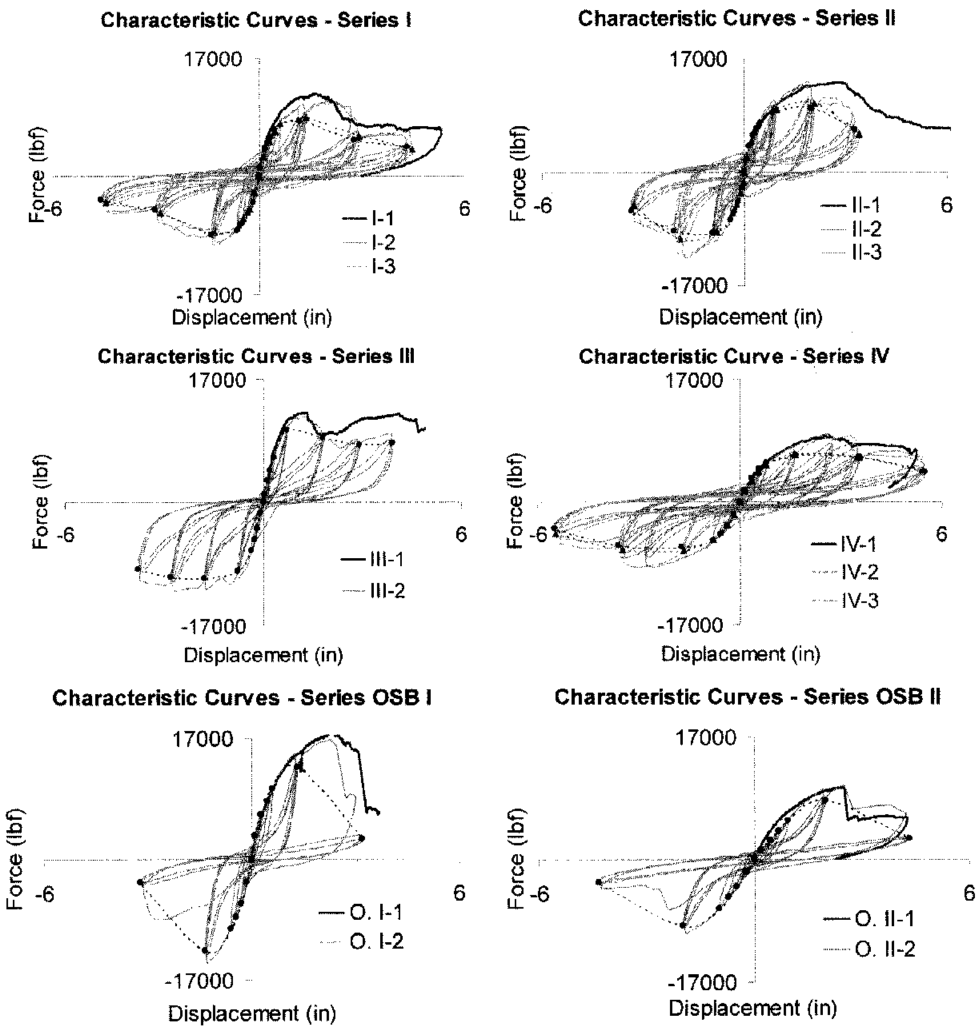


Figure 8. Experimental curves for all specimens.

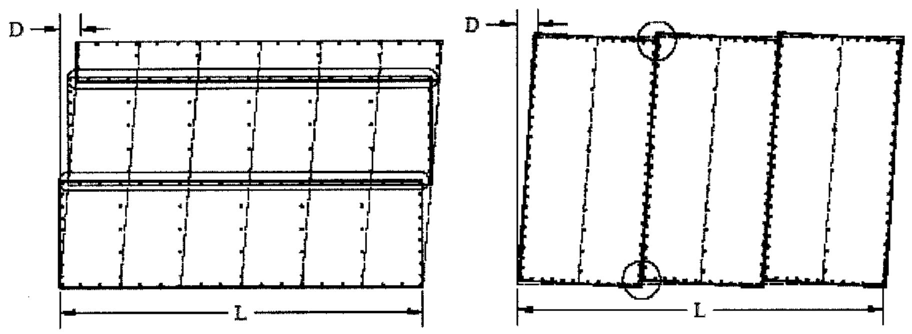


Figure 9. Typical deformation pattern of corrugated sheet (Serie I, II) and OSB sheeted specimens (Series OSB I).

corresponds to serviceability conditions.

-If slip is limited to the diameter of the screw ($D_r = 4.8$ mm, Fig. 10) the cladding requires some repair. There is damage, but it is not excessive and the structure can be repaired by minor interventions, like replacing screws with larger diameter ones. This could correspond to immediate occupancy.

-For life safety criteria, any kind of damage is acceptable as long as it does not endanger the safety of the occupants. This criterion, D_u, corresponds to the attainment of the ultimate force (F_{ult}) and the starting of the downwards slope.

Based on these assumptions at the connection level, the following performance criteria, in terms of panel drift, δ, are suggested for wall panels clad with corrugated sheet: (1) fully operational ($\delta < 0.003$); (2) partially operational ($\delta < 0.015$); (3) safe but extensive repairs required ($\delta < 0.025$).

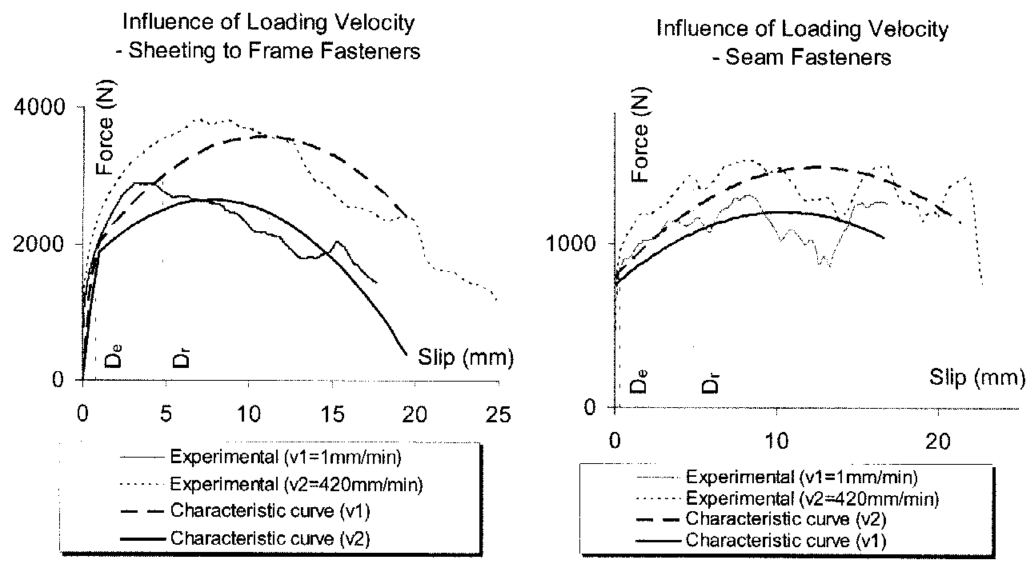


Figure 10. Performance criteria at steel-to-steel connection level.

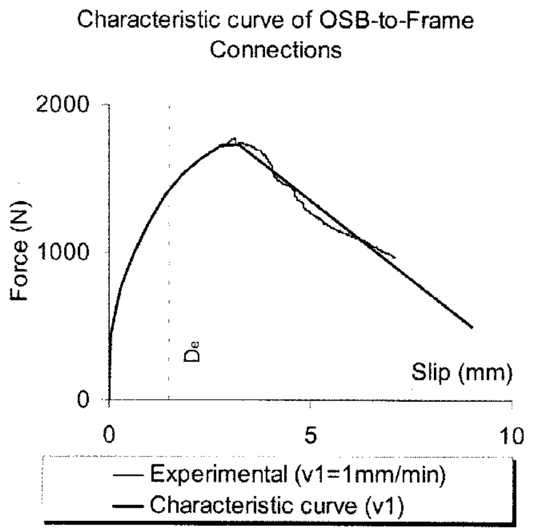


Figure 11. Performance criteria for OSB-to-steel connections.

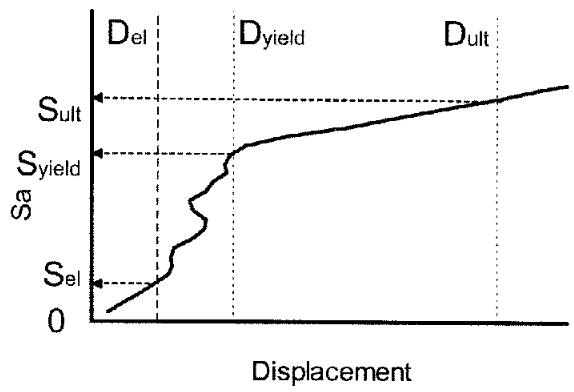


Figure 12. IDA behavior curve for shear wall panels.

The first performance level does not provide ductility, because the shear panel is elastic. This could be the design criteria for frequent, but low intensity earthquakes. In case of rare but severe earthquakes, the last two design criteria can be used and some ductility will be available.

For OSB-to-steel connections, which are characterized by fragile behavior, the design has to be controlled by the *elastic* limit only (D_e - Fig. 11). In such a case, multiple performance levels cannot be applied.

It is very important to underline that for all tested specimens, the wall stud system provided very good redundancy. Even when damage was significant, no collapse occurred. This is of real importance for buildings located in seismic areas. For corrugated sheet specimens, and similarly for others, relevant performance design criteria can be suggested.

The effect of overstrength (design overstrength of connections and redundancy of skeleton) was identified to be important in the post-elastic behavior of panels and the main source of a possible earthquake design force reduction. According to IDA procedures, spectral accelerations and displacements have been used to calculate q_μ and q_S factors instead of forces and displacements (Fig. 12):

$$q_S = S_{yield}/S_{el} \quad (5)$$

$$q_\mu = S_{ult}/S_{yield} \quad (6)$$

where S_{el}, S_{yield} and S_{ult} are the spectral acceleration corresponding to the equivalent elastic displacement (initiation of pseudo-inelastic behavior), to the attainment of equivalent plastic capacity and to the ultimate capacity, respectively.

The average resulting factor q_S of 2.2-2.6 harmonizes reasonably with the values 1.5-5 suggested by Gad *et al.* (1999). The possibility of design force reduction due to ductility and energy dissipation q_μ, seems to be more limited (e.g., q_μ = 1.4-1.6), probably due to low energy dissipation capacity of the hysteretic loops. This value is

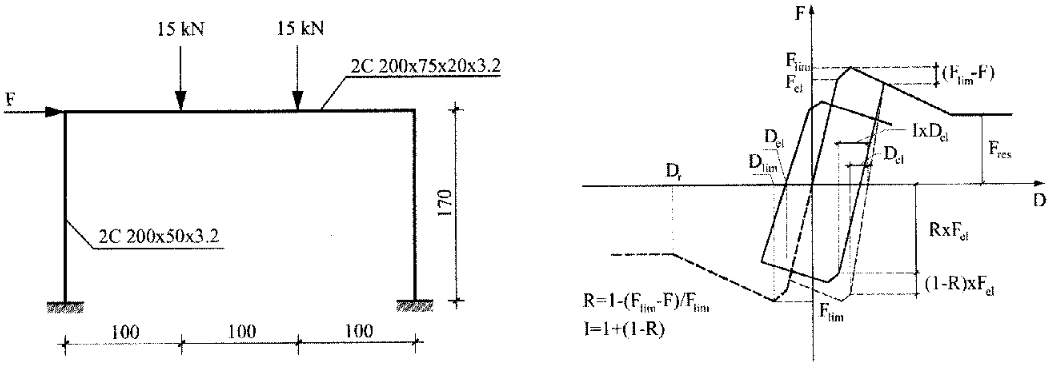


Figure 13. Analyzed frame and analytical cyclic behavior.

also in agreement with the findings of Gad *et al.* (1999). One can observe that the resulting q-factor value is at least 3.0!

5. Seismic Performance of Cold-formed Portal Frames

Extended research on this subject was performed by Calderoni *et al.* (1994). The authors started their research from the experimental findings of Ono and Suzuki (1986), who proved significant post-elastic strength and ductility of some cold-formed steel frames through testing. Figure 13 shows the frame tested by the Japanese researchers, and the corresponding numerical model proposed and calibrated by Calderoni *et al.* (1994) in order to study the behavior of these structures. By using this kind of cyclic load-displacement law, a lot of numerical step-by-step dynamic analyses were performed with reference to some built-up channel section portal frames. Geometrical and mechanical properties of frames were selected to provide monotonic F-D curves characterized by elastic stiffness, slope of the softening branch, and residual strength.

The dynamic response of the analyzed frames (performed by also taking into account the geometrical degradation due to second order effects) was obtained by using, as load conditions, thirty different real accelerograms recorded during some Italian earthquakes. They were selected in such a way that the corresponding average elastic response spectrum (50% of probability to be exceeded) is similar to that provided by EN 1998-1 for soil type A and PGA equal to 0.15 g.

The results of this wide numerical investigation (about 1000 analyses) showed that the seismic behavior of thin-walled portal frames was not so different with respect to the corresponding ideal elasto-plastic structure, provided that the slope of the softening branch of the monotonic F-D curve was reasonable (e.g., around 30°). In these cases, it seemed that a q factor greater than 1, varying in the range 2 to 3, could be used in low-seismicity zones, if the available ductility exhibited by the frame is about equal to 3 (Calderoni *et al.*, 1994). Nevertheless, it clearly appeared that the shape of the F-D curve, e.g., the lateral elastic stiffness, affects the structural response and consequently, the judgement on the possibility of using light gauge structures in seismic zones. If the traditional capacity method is applied to design these structures, Eq. (4) is recommended to calculate the q factor. However, the lowest limit of q factor, suggested by the authors of this study is 1.8 (the q value equal to 3, for the frame of Fig. 13, is really to big to be used in practice, since the sections are unusually thick). This value approaches the previous one proposed by Moldovan *et al.* (1999), and both are practically of the same order of magnitude with the value given in EN 1998-1 for non-dissipative structures, i.e., q = 1.5-2.0.

However, the performance of portal frames is mainly dependent upon the performance of the joints. In order to characterize the behavior of cold-formed steel bolted joints, an extensive experimental research was performed at the "Politehnica" University of Timisoara, Romania (Dubina *et al.*, 2004).

Realistic specimens have been designed, starting with a real pitched-roof portal frame with the following configuration: span 12 m, bay 5 m, height 5 m and roof angle 10°. The frame was analyzed and designed according to the current EN 1993-1-3 (2001) rules, for an approximately 10 kN/m uniformly distributed load. The size of the knee and the ridge specimens, and the testing setup, were chosen to obtain a similar bending moment in the connected members as observed in the structure.

From the design, the elements of the portal frame resulted in back-to-back built up sections made by Lindab Ltd. C350/3.5 profiles (f_y = 350 N/mm^2 - SUB350). In accordance to these cross sectional dimensions, three alternative joint configurations using welded connecting gusset elements (S235 - f_y = 235 N/mm^2) were designed (see Fig. 14 and Fig. 15).

The connecting bolts were subjected to shear and their design was carried out assuming the rotation of the joint around the center of the bolt group and a linear distribution of the arising forces in each bolt, depending on the distance from the rotational center. In the design of the joints, the bending moment reduced in the rotation center of the joint was used, not the theoretical one at the corner of the frame.

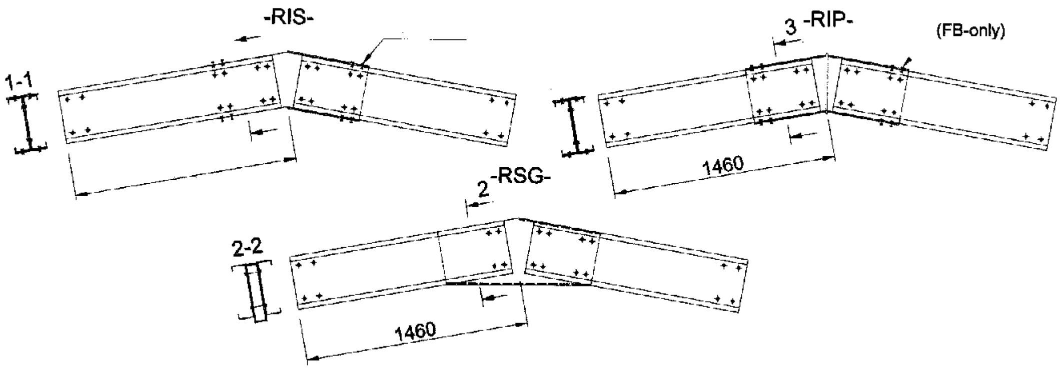


Figure 14. Main dimensions of ridge connections.

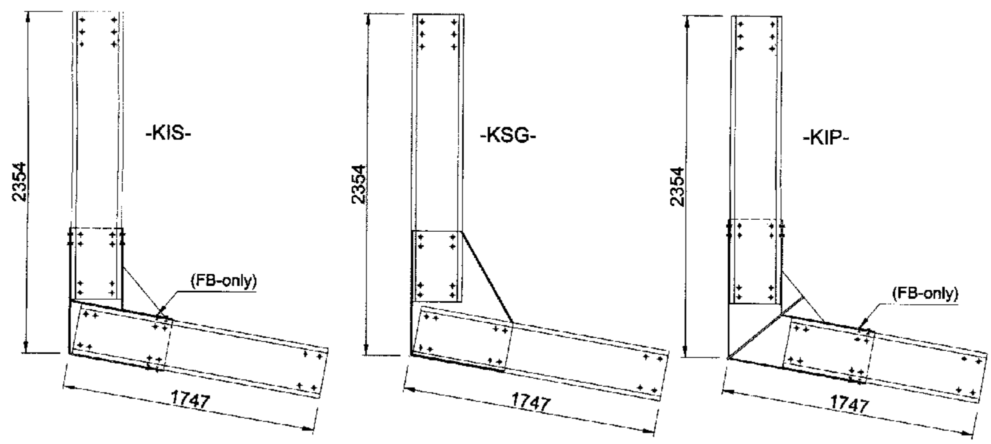
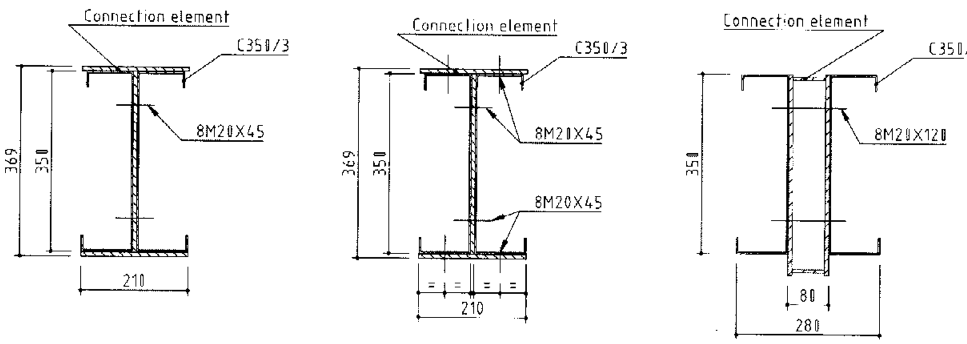


Figure 15. Main dimensions of knee connections.



(a) (b) (c)

Figure 16. Bolt configuration in the cross section.

One group of specimens (KSG and RSG) used spaced gussets (Fig. 16c). In this case, bolts were provided only on the web of the C350 profile. In the other cases, where two different details were used for the connecting bracket - i.e., welded I sections only (KIS and RIS), and welded I section with plate bisector (KIP and RIP), respectively - bolts were provided on the web only (Fig. 16a), or both on the web and the flanges (Fig. 16b). The case where bolts were also on the flanges had the distinctive FB in their name (see Table 3).

Monotonic and cyclic experiments were made for each specimen typology. For monotonically loaded specimens, the loading velocity was approximately 3.33mm/min, and the yield displacement was determined according to the ECCS (1985) procedure (see Fig. 17 and Table 4 and Fig. 18 and Table 5, respectively).

Samples of results are shown in Tables 4 and 5, both for monotonic and cyclic tests, while Figs. 19 and 20 display some failure modes of the tested specimens.

The conclusions of this research can be summarized as follows:

• The calculation model for the connection, based on the linear distribution of the force on each bolt, is not correct. The force distribution is unequal due to the flexibility of the connected member. In fact, the force is an order of magnitude bigger in the outer bolt rows

Table 3. Tested specimens

Element type	Code	Loading type
RIS (Ridge connection with I Simple profile)	RIS-FB-M	Monotonic
	RIS-FB-C1*	Cyclic - Modified ECCS procedure
	RIS-FB-C2*	Cyclic - Low cycle fatigue
RSG (Ridge connection with Spaced Gusset)	RSG-M	Monotonic
	RSG-C1	Cyclic - ECCS procedure
	RSG-C2	Cyclic - Modified ECCS procedure
RIP (Ridge connection with I profile and end Plate)	RIP-M	Monotonic
	RIP-M	Monotonic
	RIP-C1	Cyclic - ECCS procedure
KSG (Knee connection with Spaced Gusset)	KSG-M	Monotonic
	KSG-C1	Cyclic - Modified ECCS procedure
	KSG-C2	Cyclic - Low cycle fatigue
KIS (Knee connection with I Simple profile)	KIS-M	Monotonic
	KIS-FB-M*	Monotonic
	KIS-FB-C*	Cyclic - Modified ECCS procedure
KIP (Knee connection with I profile and end Plate)	KIP-M	Monotonic
	KIP-FB-M*	Monotonic
	KIP-FB-C*	Cyclic - Modified ECCS procedure

*FB Specimens (RIS, RIP, KIS, KIP) with supplementary bolts on the flange

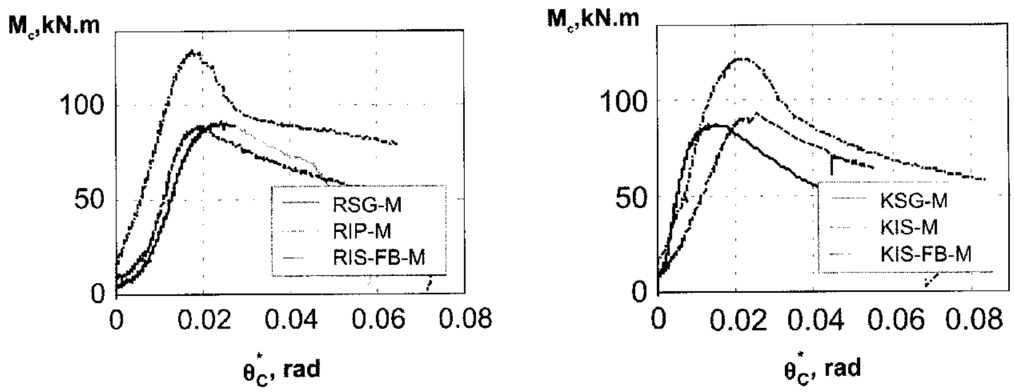


Figure 17. Moment-rotation curves for monotonic tests.

Table 4. Results for monotonic specimens

Monotonic specimens	$\mu = \theta_u/\theta_y$	$\theta_{pl} = \theta_u - \theta_y$ [rad]
RSG, RIP, KIS, KIP, KSG	1.5-2.5	0.01-0.014
RIS-FB, KIS-FB, KIP-FB	1.5-1.8	0.007-0.013

compared to the most inner one. There are two main components, namely the bearing of the bolts and the local buckling of the connected profile, which interact and determine both the rigidity and the load bearing capacity of the joint. A correct model for the behavior must include both these components.

• A connection with bolts only on the web of the profiles is always partial strength. If the load bearing capacity of the connected beam is to be matched by the connection strength, bolts on the flanges become necessary.

• The ductility of the connection is limited under both monotonic and cyclic loads, and the design, including the design for earthquake loads, should take into account only the conventional elastic capacity corrected with safety factors. Because there is no significant post-elastic strength, there are no significant differences in ductility and capacity of cyclically tested specimens compared with the monotonic ones. However, if the joints are loaded under the limit of their maximum capacity, even

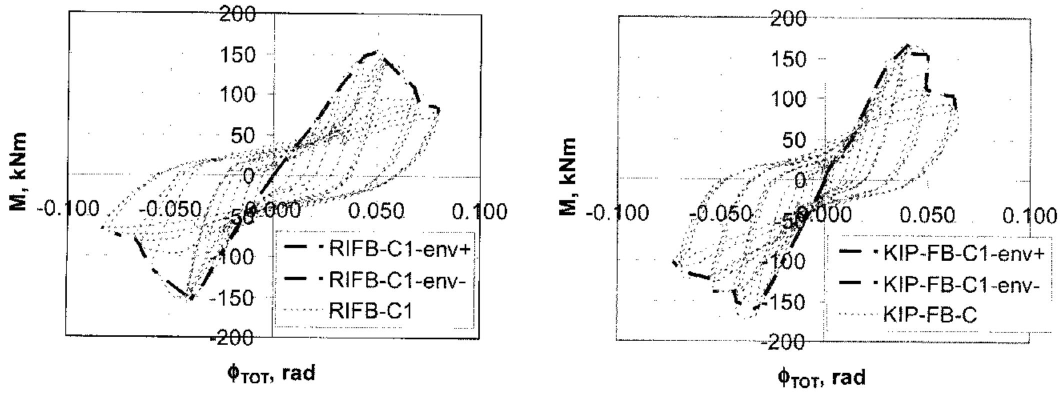
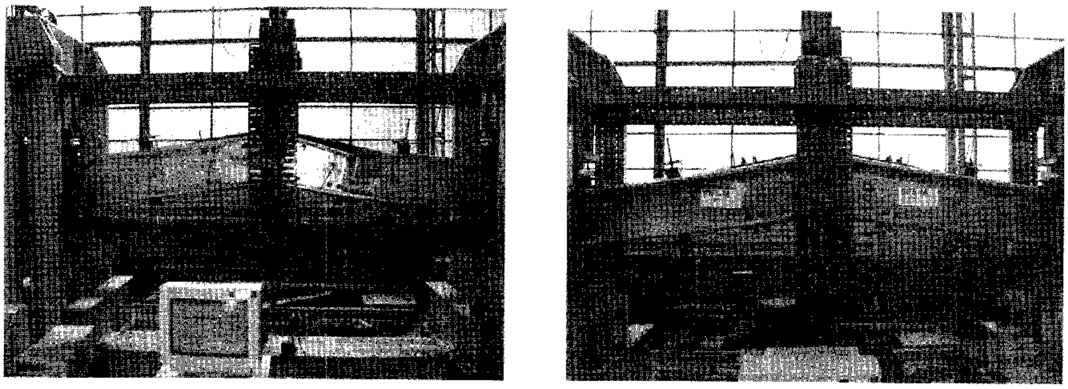


Figure 18. Moment-rotation curves for cyclic tests.

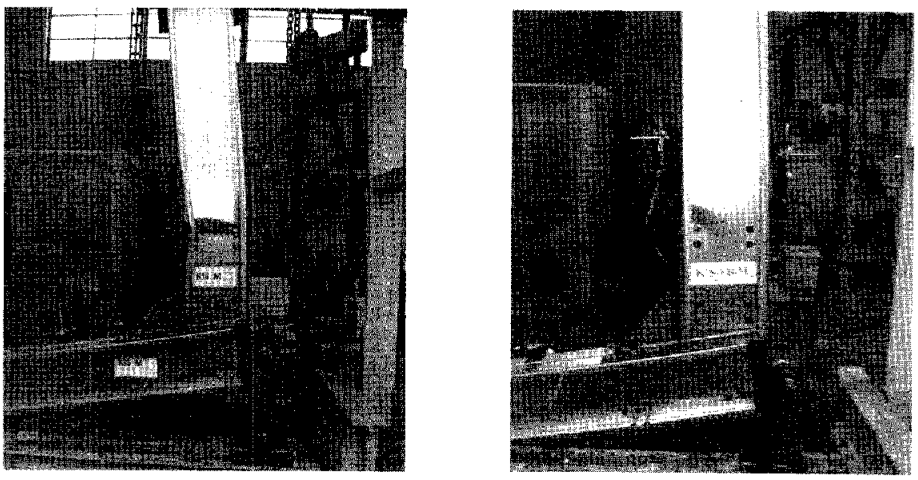
Table 5. Results for cyclic specimens

Cyclic specimens	$\mu = \theta_u/\theta_y$	$\theta_{pl} = \theta_u - \theta_y$ [rad]
RSG, KSG	1.55-1.88	0.003-0.012
RIS-FB, KIS-FB, KIP-FB	1.48-2.13	0.007-0.013



(a) (b)

Figure 19. Failure of ridge specimens RIS-M (a) and RIP-FB-M (b).



(a) (b)

Figure 20. Failure of knee specimens (a) KIS-M and (b) KIS-FB-M.

cyclically, their strength is not greatly affected. Consequently, if the joint detailing and connection components sizing provides at least 20% overstrength, the cold-formed steel pitched-roof frames could be

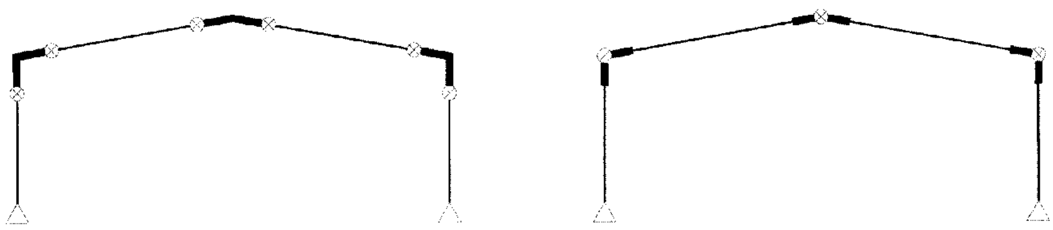


Figure 21. Frame models accounting for connection flexibility: detailed (a) and simplified (b).

classified as class L ductility (low) according to EN 1998-1, 2003.

• Due to the semi-rigid and partially-resistant character of apex and eaves connections in steel cold-formed frames, moment-rotation characteristics have to be considered explicitly in design. Two models are possible: a more refined one that considers each bracket-member connection separately (Fig. 21a), and a simplified one, that considers the total joint characteristics (Fig. 21b). Experimental moment-rotation relationships were derived for each of the two models, but future research should be conducted to analyze the differences in frame response considering the two possible approaches.

Concluding Remarks

Light gauge steel structures made by class 3 or class 4 sections fabricated by cold-forming can be effectively used in seismic resistant structures mainly due to their reduced weight/strength ratios.

Traditional capacity design based on equivalent elastic static analysis with reduction factors q of values $1 < q \leq 2$ can be used, provided the overstrength of joints and structural redundancy are available.

Seismic response of light-gauge steel framing can be significantly improved if shear walls are used to resist horizontal forces.

Both experimental and numerical results sustain classifying light-gauge steel structures as low-dissipative. Practically, an elastic design has to be conducted, but the seismic force can be evaluated by applying a reduction factor q of 1.5-2, corresponding to "L" ductility class, as specified in EN 1998-1. This is, in fact, a "pseudo ductility," because it is mostly based on overstrength and structural redundancy rather than on the post-elastic strength reserve of members and connections.

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