Design Criteria for Seam and Sheeting-to-framing Connections of Cold-formed Steel Shear Panels

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Abstract

In recent years important research activity was undertaken to determine the earthquake performance of light gauge steel house structures. Usually, studies approach the problem of earthquake performance trough experiments on wall panels conducted either under monotonic or cyclic loads. All studies underline the overwhelming importance that the behavior of the connections has in determining the overall performance of the wall panels. However, these studies focused on the global behavior of the panel and did not try to characterize the behavior and provide design criteria for connections, as components of the structural system. The present paper attempts to fulfill this gap.

Introduction

During recent years a growing number of experiments have been undertaken in Europe to study the behavior of light-gauge steel wall panels. One of the first extended experimental programs in Europe on light-gauge steel wall panels, aiming to characterize their cyclic response, has been undertaken at the "Politehnica" University of Timisoara. The experimental results and their interpretation have already been published (Fülöp & Dubina 2002, Fülöp & Dubina 2004) and are very briefly presented hereby.

The associated testing program on the component connections, especially the ones between the sheeting material and the light-gauge steel frame of the wall is presented in detail. Based on the two set of experimental results, the ones on panels and the ones on connections, performance criteria for connections, as

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main responsible for the global performance of panels is proposed. Finally, a methodology for finite element modeling is described.

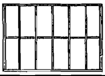
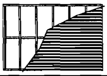
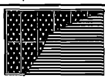
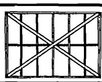
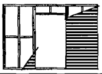
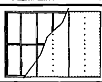
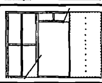
Summary of wall panel experiments

The experimental program on shear walls with common sheeting solutions has been carried out in order to clarify aspects related to hysteretic behavior of different wall typologies. Each specimen series consisted of identical wall panels (3600×2440mm), tested under lateral loads applied statically, both in monotonic and cyclic way (Table 1).). The skeleton of the wall panels was made of C shaped cold-formed steel elements, with studs at 600 mm intervals.

Fixing between profiles within the wall skeleton, and between the sheeting and skeleton was with self-drilling self-taping screws (d=4.8 mm). Studs at wall panel ends and in vicinity of openings were doubled. The main outputs of the experiments were curves of shear force vs. lateral displacement, measured at the top of the-panel. Representative load versus lateral displacement curves are presented in order to illustrate differences of monotonic to cyclic response (Fig.1). Wall-panels exhibited highly non-linear behavior. Elastic modulus, ultimate force and ductility, have been interpreted according to the procedures proposed by the ECCS (1985) and by Kawai (1997). The design strength of the panels was referred as the minimum of the force at story drift angle 1/300 (F_{300}) and 2/3 F_{max}.

As observed during the experiments, lateral deformation of the panel depends on: the shear deformation of the sheeting; the deformation due to corner uplift, and most significantly on the nonlinear deformation of the connections between the sheeting and the skeleton.

Table 1. Summary of tested wall panels

Series	Configuration	Cladding	Testing Method	No. Test
O		-	Monotonic	1
I		Cor. Sheet LTP20/0.5 (Ext)	Monotonic	1
			Cyclic	2
II		Cor. Sheet (Ext) Gyps Board (Int)	Monotonic	1
			Cyclic	2
III		Cross Bracing (Ext-Int)	Monotonic	1
			Cyclic	1
IV		Cor. Sheet LTP20/0.5 (Ext)	Monotonic	1
			Cyclic	2
OSB I		OSB Panels (Ext)	Monotonic	1
			Cyclic	1
OSB II		OSB Panels (Ext)	Monotonic	1
			Cyclic	1
Total number of specimens				15

In case of wall panels sheeted with corrugated sheeting placed horizontally (Fig.2.a.) most of the nonlinear deformation was due to the inelastic deformation of seam fasteners. Seam connectors will be the ones to deform excessively, later load being redistributed to the vertical screw lines connecting the sheeting to the skeleton. In case of the wall panels sheeted with OSB, as the skeleton deforms into a parallelogram the OSB panels have "rigid body" rotation (Fig.2.b.). As a consequence connections at the corners of OSB panels will be the ones which have to accommodate the largest slip and will be damaged.

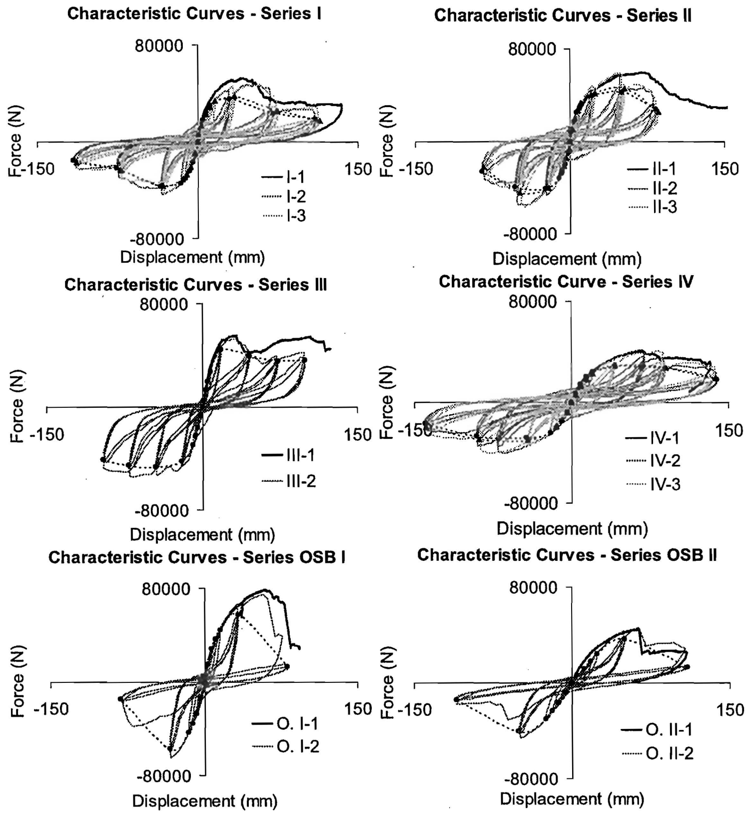


Fig.1. Experimental load versus displacement curves

As experimental curves are non-linear from the beginning, *elastic* design capacity of the panels can only be defined in a conventional way. Any assumption of *elastic* design limit is to be related to a tolerable deformation in the group of connectors that are subjected to the highest forces, and therefore be more based on serviceability than strength criteria at the level of the entire panel. Also, there will be an important strength reserve beyond any *elastic* design limit considered, due to the redundancy of the structural system and to the supplementary load bearing capacity of the remaining active connectors when the few, most loaded ones have excessive deformations or fail.

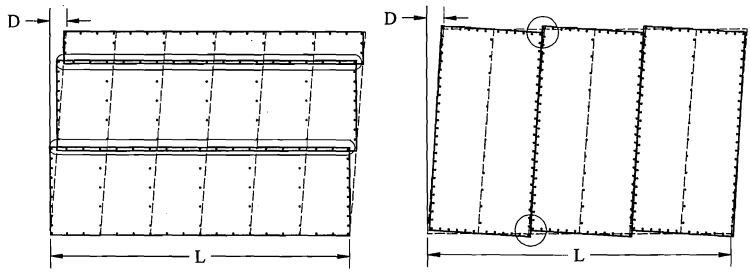


Fig.2. Typical deformation pattern of corrugated sheet (Series I, II) and OSB sheeted specimens (Series OSB I)

Observing comparative monotonic to cyclic behavior a reduction of strength of about 10-15% (Fig.1) can be identified in case of cyclic loading. Hence, if the design strengths of the panels are deduced based on monotonic experiments the performance will be overestimated. Important characteristic of the cyclic curves is the strong pinching (factor of 0.85-0.95), the source being the slip in the connectors at reverse loading, which implies reduced energy dissipation. Strength degradation in consecutive loading cycles is also present as result of accumulated damage in the panel during the loading history, leading to significant difference between stabilized and unstabilized envelope curves. At large deformations softening is present for all wall specimens. Finally, it is important to mention that the failure of the OSB sheeted panels was less ductile compared to the ones sheeted with corrugated sheeting.

It can be observed that the major contribution to the panel's load bearing capacity and deformability comes from the connections between the sheeting material and the skeleton, a particularity that has been many times emphasized. However, there were only a few attempts to try to relate the behavior of connections to the overall behavior of the entire wall panel. The subject has mostly been investigated in the case of wood skeleton wall panels sheeted with OSB (McCutcheon 1985, Salenikovich 2000).

Experiments on connections

After observing the paramount importance of the connection details on the behavior of the wall panels, an experimental program dealing with the component materials and the connections has been designed, two series of experiments being carried out on components of the wall panels. The first series had the aim of determining the properties of the base materials (ie. steel, OSB panels). These experiments revealed good correlation between nominal and measured mechanical properties of the base steel, and a big scatter in the measured properties of the OSB.

The second series of experiments was carried out to determine the mechanical properties of the connections that were identified to have a crucial effect on the behavior of the panels. From this point of view the self-drilling screws connecting the sheeting material to the steel skeleton and seam connections are important and were of the following typologies: (1) connection between the corrugated steel sheeting and steel profiles; (2) sheeting-to-sheeting seam connections and (3) connections between the OSB panels and steel profiles.

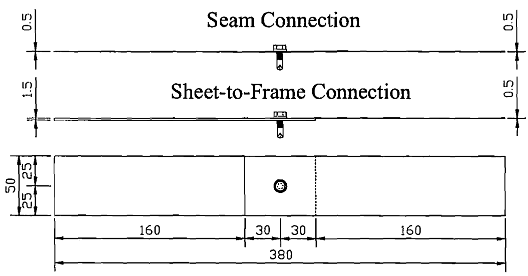


Fig.3. Steel-to-steel connections

Steel-to-steel connections using self-drilling screws are largely used in everyday practice, and both testing and calculation methods are well established. Specimens were chosen in a way that they represent (1) and (2) connection typologies used in the tested wall panels. The dimensions of the specimens were chosen according to the European standards (ECCS 1983), the dimensions of the connecting plates being 50×220 mm (Fig.3). Tests were conducted on two types of specimens: (1) connecting corrugated sheet to skeleton (0.417 mm to 1.42 mm sheet) using SD3-T15-4.8×22 (4.8 mm) screws and (2) corrugated sheet to corrugated sheet (0.417 mm to 0.417 mm sheet) using SL2-T-A14-4.8×20 (4.8 mm) screws. The edge distance of the sheets in the direction of the loading was 30 mm, in order to facilitate bearing failure of the thinner sheet the failure mode observed during the panel test.

Test connections were executed with the same materials and in similar conditions as the ones in the panels. In the case of the thin-to-thick sheet specimens, the thinner sheet was equipped with a supplementary plate to ensure centric transmission of the loading to the screw. The testing was undertaken in the laboratory of the Department of Steel Structures and Structural Mechanics (CEMSIG Research Centre – *cemsig.ceft.utt.ro*) at the "Politehnica" University of Timisoara, using a UTS Testwell universal testing machine. The minimum number of tests of the same typology was 3. If during tests technical problems appeared this number was supplemented to 5 or 7. Two loading velocities were applied, V_1=1mm/min for quasi static loading conditions and V_2=420mm/min for high velocity tests. The extensometer captured the elongation of an 80 mm portion from the middle part of the specimens.

Table 2. Summary of tested connections

Base thick. (mm)	**Connected thick. (mm)**	**Screw Diam. (mm)**	**Loading velocity**			
			Slow (1mm/min)		**Fast (420mm/min)**	
			Nr.	**Code**	**Nr.**	**Code**
1.42	0.417	4.8	4+2	I-TP-M-V1-1...4, 6, 7	4	I-TP-M-V2-1...4
0.417	0.417	4.2	4	I-TS-M-V1-1...4	3	I-TS-M-V2-1...3

Because the failure modes of the specimens were not always identical to the ones observed during the panel tests, the testing methodology had to be adapted to simulate the behavior of connections into the panel. This was the case of sheeting-to-framing connections where for the first four specimens (Series *I-TP-M-V1*), different failure modes were observed (Fig.4.a.). *I-TP-M-V1-2* and *I-TP-M-V1-3* had the desired mode of failure through tearing of the thinner sheet, while in the other two specimens the end of the thinner sheet distorted during the deformation (Fig.4.a.). This distortion cased net section rupture of the thinner sheet. This failure mode was not at all observed during panel tests, being impossible to develop due to the presence of the corrugations which play the role of out of plane stiffeners for the thin sheet.

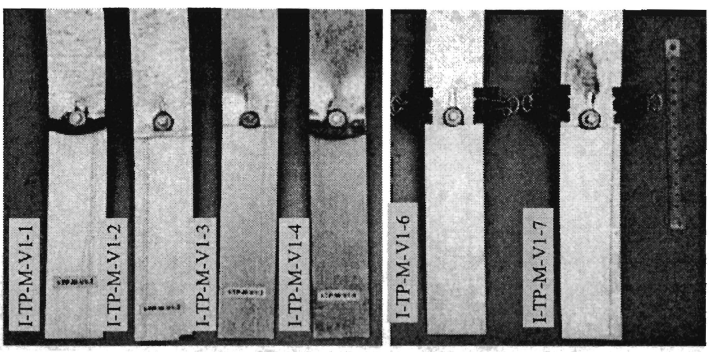


Fig.4. Failure mechanism for connection series *I-TP-M-V1*

In order to force the connection specimens to fail in the desired mode of failure (tearing of thinner sheet) additional clips were provided to prevent the distortion of the thinner sheets, as shown in Fig.4.b, for specimens *I-TP-M-V1-6* and *I-TP-M-V1-7*.

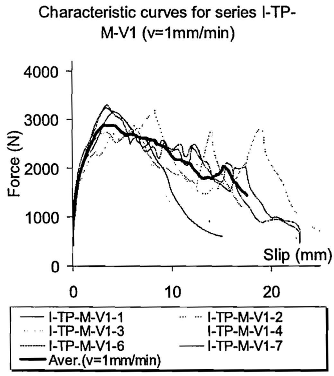


Fig.5. Exp. curves for *I-TP-M-V1*

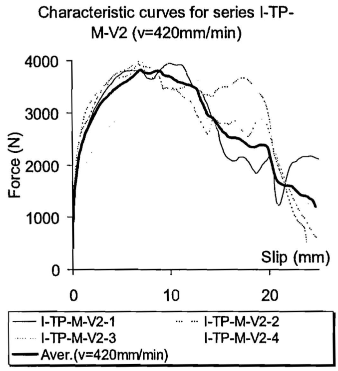


Fig.6. Exp. curves for *I-TP-M-V2*

These two specimens failed similarly with *I-TP-M-V1-2* and *I-TP-M-V1-3*, sustaining a much higher load bearing capacity at large displacements. This is desired since it influences the overall ductility of the wall panels and the average force-displacement curve in Fig.5 was obtained by eliminating the results for specimen *I-TP-M-V1-1* and *I-TP-M-V1-4*. For the rest of the experiments clips were used. The following set of experiments (*I-TP-M-V2*) was similar but the loading velocity was 420mm/min, experimental curves being presented in Fig.6.

In case of thin-to-thin sheet connections, modeling seam fasteners, the failure mode was due to tilting and pull-out of the screw (Fig.7).

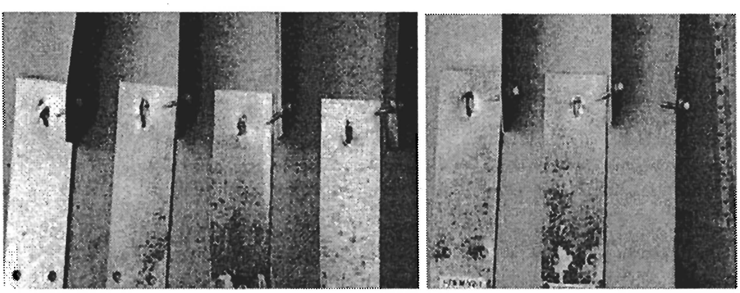


Fig.7. Failure for specimens in Series *I-TS-M-V1* and *I-TS-M-V2*

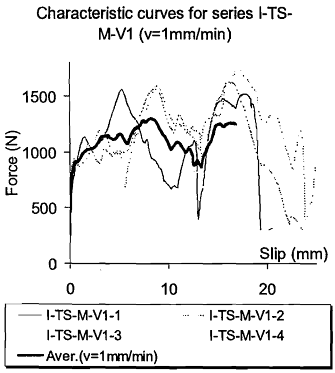


Fig.8. Exp. curves for *I-TS-M-V1*

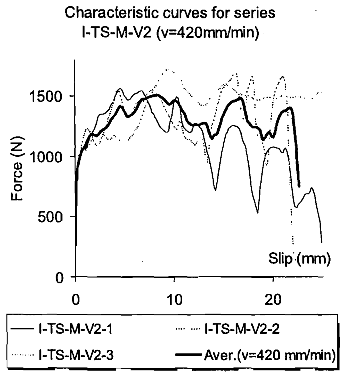


Fig.9. Exp. curves for *I-TS-M-V2*

The same failure mode was observed for seam connections during the wall panel experiments, with the observation that the two sheets could not depart perpendicularly to the loading direction due to the presence of the corrugation. Therefore clips were used in the case of these experiments, the loading velocities being the ones already mentioned. Comparative results are presented in Fig.8 and Fig.9.

The load bearing capacity (F_{nom}) and the rigidity (K_{serv}) of the connections can analytically be calculated using several methods, the subject of connections using self-drilling screws being well documented in the technical literature and in design standards. However, calculation methods usually refer to the calculation of the capacity and in some cases rigidity, the ductility of such connections being considered of less interest. In this case three alternative calculation methods were applied for the connection typologies under investigation. Using the design method of Eurocode 3 (EC3 2001) only the load bearing capacity of the connections can be calculated (Table 3).

Table 3. Comparative results of experiments and analytical methods

	v (mm/ min)	N & kN/mm	EC 3	Fan	Experiment	ECCS
0.417 –to– 1.42mm	1	F_{nom}	1961.9	2618.2	2206.4	1775.1
		K_{serv}	-	0.760	8.62 (4.31*)	2.86**
	420	F_{nom}	2037.3	2718.8	2200.3	1843.3
		K_{serv}	-	0.760	8.61 (4.31*)	2.86**
0.417 –to– 0.417 mm	1	F_{nom}	881.2	1074.3	1091.3	798.6
		K_{serv}	-	0.428	13.25 (6.62*)	4
	420	F_{nom}	915.0	1115.6	1247.7	829.2
		K_{serv}	-	0.428	9.53 (4.76*)	4

1) F_{nom} is the reference bearing capacity; the design strength results by dividing F_{nom} with the relevant safety factor. K_{serv} is the design rigidity considered in the serviceability limit state check.
2) * In the literature it is suggested that the rigidity of a connection with a single screw is twice as big as the rigidity (reduced to a screw) of a connection with multiple screws.
3) ** The closest value to be assimilated is for d=5.5mm screw with neoprene.

Using the ECCS (ECCS 1995) method the load bearing capacity can be calculated and the rigidity of the connection is included, for typical connection typologies, as empirical values. The proposal of Fan (Fan 1996, Fan 1997) is an improvement of the EC 3 method and allows for the calculation of more varied connection typologies and contains specific recommendations for the evaluation of rigidity. The three methods have been applied for the tested connections, comparative results being presented in Table 3.

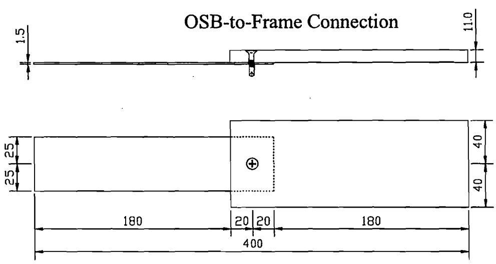


Fig.10. OSB to steel skeleton connections

A third connection typology used in the wall panel test was the one connecting OSB to the steel skeleton. In order to test this typology of connections, specimens as presented in Fig.10 were prepared. The testing of these specimens yielded very inhomogeneous results (Fig.11.a.) depending on the direction and density of fibers in the vicinity of the screw and between the screw and the margin of the OSB panel.

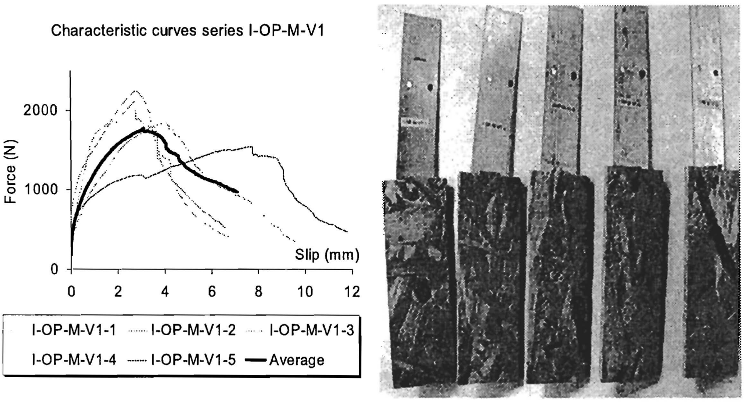


Fig.11. Experimental curves and failure modes of the OSB-to-steel connections

No generalizing conclusion can be drawn from these experiments, besides that OSB connections possess less ductility, the most likely reason for the less ductile failure of the wall panels sheeted with OSB.

Performance criteria

An important aspect of performance based seismic design philosophy is to define acceptable damage levels and relate them to the performance objectives. Performance objective proposals are based on three or four goals (FEMA-273 1997): (1) Serviceability under ordinary occupancy conditions; (2) Immediate occupancy following moderate earthquakes; (3) Life safety under design-basis events; (4) Collapse prevention under maximum considered event. Such vague goals can be translated into practice by relating performance objective to deformations, using for horizontal loads the inter-story drift (δ), as measure.

In case of the 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 connection level and, for different wall typologies, relate this to the overall deformation of the wall panel. To establish global performance criteria acceptable deformation levels in the seam fasteners are suggested.

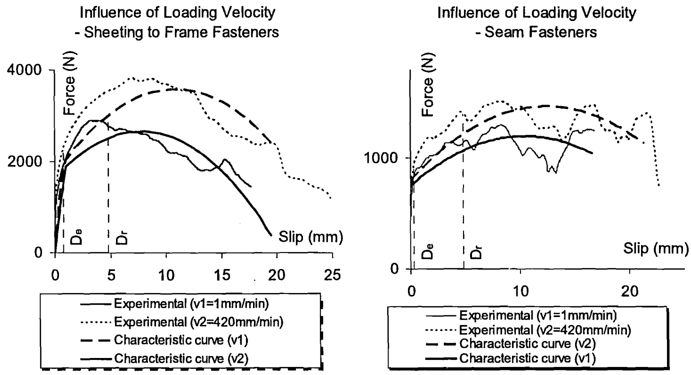


Fig.12. Performance criteria at steel-to-steel connection level

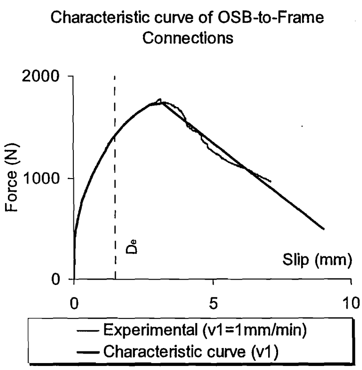


Fig.13. Performance criteria for OSB-to-steel connection

(1) If slip of the seams does not exceed the elastic limit (D_e, Fig.12), corresponding to $0.6F_{max}$, damage is limited and can be considered negligible. In this case the integrity of the cladding is fully preserved, no repairs are required; it correspods to serviceability conditions. *(2)* If slip is limited to the diameter of the screw (D_r=4.8mm, Fig.12) the cladding requires some repair. There is damage, but not excessive and by minor interventions, the structure can be repaired. This could correspond to immediate occupancy. *(3)* In case of life safety criteria any kind of damage is acceptable, without endangering the safety of the occupants. This corresponds to the attainment of the ultimate force (F_{ult}) and the starting of the downwards slope.

As it can be observed from the experiments the behaviour of the connections is non linear. Both initial rigidity and especially design capacity can only be assigned as conventional value in relationship with the curves, especially in case of thin-to-thick steel sheet connections. Any such assumed value of the design capacity will inevitably lead to some supplementary strength of the connection not considered. Furthermore, the load distribution in the connections of the panel is uneven. While some connections are loaded to failure others are loaded with much lower loads, and when some connections fail the load will be redistributed to other groups of connections.

Based on these assumptions at connection level, the following performance criteria 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 ($\delta<0.025$).

Table 4. Performance criteria

Spec.	**Connection Deform.** *(mm)*	**Force** *(N)*	**Panel Top Disp.** *(mm)*	**Drift (%)**
I-3	0.197	21423	6.71	0.274
	4.8	43885	29.22	1.197
IV-2	0.197	10106	7.96	0.326
	4.8	35613	44.13	1.808
IV-3	0.197	8849	8.11	0.332
	4.8	26332	42.22	1.730

The first performance level does not provide ductility, because shear panel work 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. In case of OSB-to-steel connections, which are characterized by a fragile behavior, the design has to be controlled by the *elastic* limit (D_e - Fig.12). In such a case multiple performance levels can not be applied.

Numerical modeling of the behavior of the panels

Based on the component characteristics determined experimentally, an attempt of Finite Element (FE) modeling (Fig.14) was made in ANSYS to reproduce the behavior of the entire wall panel. As first step the wall panels sheeted with corrugated sheeting were considered because component behavior was more homogeneous in this case.

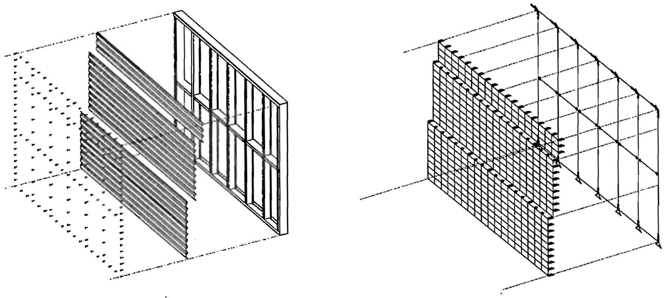


Fig.14. Elements to be taken into account for the FE modeling

The bars of the skeleton were modeled as elastic beam elements (*BEAM4*) taking into account that these elements were not heavily deformed in the post-elastic range. The corrugated sheet was modeled as an equivalent orthotropic plate (*SHELL43*) in order to take into account the fundamentally different mechanical properties of the corrugated sheet in the two principal directions and the distortion of the corrugated sheet when loaded in shear. The equivalent elastic modules (E_l – longitudinally and E_t - transversally) have been determined trough preliminary modeling of a single corrugation loaded in tension in the two principal directions. The equivalent shear modulus (G_{eff}) has been calculated taking into account the end distortion of the corrugated sheeting. Connections, both between the skeleton and the sheeting and seam connections were modeled using *COMBIN39* elements taking average deformation properties of the tested connection loaded at 1mm/min (Fig.5, Fig.8). Besides these components it is important to take into account the uplift deformation of the wall panel corners.

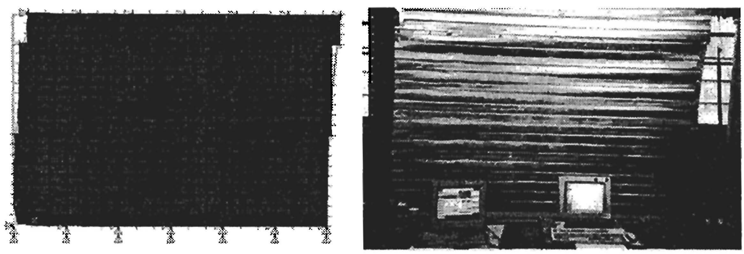


Fig.15. Comparison of deformed shape (exp. vs. FEM – Series I)

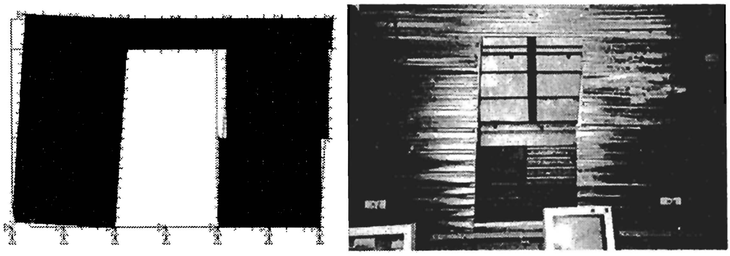


Fig.16. Comparison of deformed shape (exp. vs. FEM – Series IV)

The FE model was subjected to increasing horizontal loading at the upper part of the panel similarly to wall-panels during the full scale test. The deformation pattern and the non-linear behavior curve obtained with the FE model was compared with the monotonic curves obtained from the wall panel experiments (Fig.17).

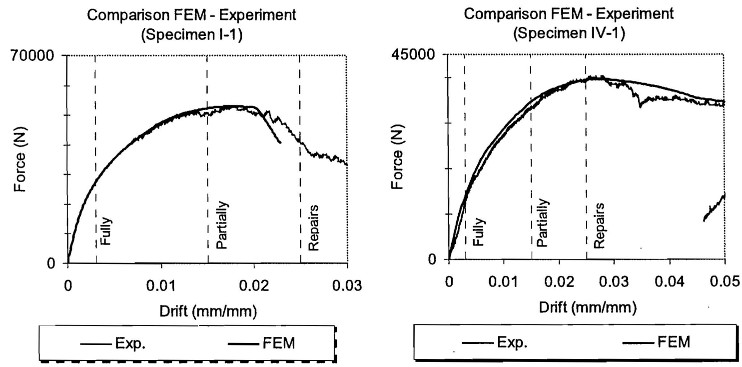


Fig.17. Characteristic curve (exp. vs. FEM – Series I, Series IV)

In Fig.15, Fig.16 and Fig.17 the remarkable similarities in terms of deformation pattern and non-linear behavior up to large displacements can be seen. Using the described FEM technique the performance of wall panels with different configurations, but using the same basic materials and connection typologies can be evaluated. The correspondence between the experimental and numerical drift values associated with the three performance levels defined in the previous section is quite perfect. So, the possibility of using such a FE modeling reduces considerably the amount of full scale tests, even if a few experiments are still necessary to calibrate the FE model.

Conclusions

Provided the failure of the bottom track in the region of the anchor bolts is prevented, the seam fasteners and the sheeting-to-skeleton fasteners are the components of the light-gauge steel framed wall panels most sensitive to damage. Most of the non-linear deformations are concentrated in these connections. The failure of the wall panels can also be attributed to the failure of the connections.

The most common way to evaluate the performance of the panels is trough testing. However, this method is time consuming and very costly the main reasons why alternatives have to be considered. As nonlinear deformation of the panel can be attributed mostly to the deformation of the connections between the sheeting and the skeleton, it is logical to seek to understand the relationship between these characteristics.

In this study the full scale wall panel test have been completed with tests on the component connection and a FE model was developed which is capable to take into account the main components of the deformation of the panels. The FE model was used for modeling wall panels sheeted with corrugated sheeting results being compared with experimental ones. The FE model proved to be able to replicate the characteristic curves and the deformation pattern of the panels, and can be used to partially replace experiments in evaluating the performance of the panels.

An attempt was made to establish performance criteria on the level of the wall panel based on the behavior characteristics of the seam connection. This procedure can be applied in case of the corrugated sheeting clad panels only.

An important conclusion of the tests on steel-to-steel connections is that at higher velocity loading connections gain load bearing capacity without loosing ductility. This observation justifies the use of quasi static loading for wall panel experiments as, at higher velocity load transmission rates (eg. near field pulse type earthquakes), the panels can be expected to have better performance than in case of low rate static loading. According to the author's knowledge, such types of results are firstly reported in the present paper.

No generalizing conclusion can be drawn regarding OSB sheeted wall panels due to the low homogeneity of the OSB-to-steel connection tests, besides that the lower ductility of the connection is responsible for the non-ductile failure of these wall panels.

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