# Post elastic modeling techniques and performance analysis of cold formed steel structures subjected to earthquake loadings

N.Bourahla, M. Berediafe-Bourahla, B. El Djouzi & H. Meddah University Saad Dahlab Blida, Algeria

**N. Allal** *R&D unit, ALRIM, Algeria* 



### SUMMARY:

A simplified FE modeling technique for shear wall panel made of cold formed steel is proposed. The approach is based on idealizing the whole panel by a nonlinear shear link element connected to rigid body elements which transmit the forces to the end elements (studs) that resist the tension and the compression. A procedure to evaluate the elastic and post-elastic properties of the multi-linear hysteresis model is elaborated using analytical and empirical methods. Series of ambient vibration testing on a recently constructed five storey building are carried out to validate the initial elastic stiffness of the wall panels. The numerical model of the structure was subjected to several ground acceleration time histories to evaluate the seismic performance of the structure in terms of lateral displacements and energy dissipation capacity. In spite of the simplified modeling technique, the results can easily portray the key features of the elastic and post-elastic response of this type of structures.

Keywords: Cold formed steel, Shear wall panel, ambient vibration, modeling technique

# **1. INTRODUCTION**

The use of cold formed steel (CFS) structures in seismic prone area requires that the buildings have to be designed to withstand the lateral loads prescribed by the seismic code. In practice, the engineers use the equivalent static method to determine the seismic forces on the shear wall panels. At the design level the use of finite elements analysis is not yet practical because of the complexity of the interaction between the different elements (studs, racks, sheathing and the screws) which constitute the load bearing wall or the lateral resisting element called shear wall panel (SWP). Moreover, cold formed steel members are susceptible to a wide variety of buckling modes. The FE models based on elementary components require numerous parameters that may not be directly extractable from experimental data. Hence, these models are very sensitive to the parameter calibration that affects closely the accuracy and reliability of the results. With such models it is not possible to treat structures of buildings for design purposes. Thus, these models are cumbersome due to the high analytical skills required for their numerical implementation and they are restricted only to practitioners with a high level of knowledge [Bourahla 2010]. In the few last decades, technical advances have been made in seismic resisting cold formed steel buildings and particularly, the development of design procedures and analytical methods to allow for the design of walls carrying horizontal and vertical loads (Fiorino 2009, Dubina 2008, Gad 2006 and McDonald 2008).

The overall seismic behavior of cold formed structures depends on the wall panels, which are governed essentially by the performance of the connectors e.g.: sheeting-to-sheeting connectors, and sheeting-to-framing connectors. To predict this behavior and particularly the failure mechanisms and possibility of progressive collapse, finite element (FE) models were developed and proposed in recent years (Bae 2008). These models are cumbersome due to the high analytical skills required for their numerical implementation and they are restricted only to practitioners with a high level of knowledge. Hence, this paper present a simple modeling technique where the shear behavior of the wall panel is idealized by an equivalent shear link connected to rigid elements which transmit the traction and

compression to the end studs of the panel. A series of ambient vibration testing are used to validate the model of multistoried building. The performance of the building is evaluated using several spectrum compatible acceleration time histories.

## 2. MODELING OF THE SHEAR WALL PANEL

A cold formed steel structure is composed of shear or vertical load bearing walls. These are made of a frame (studs and tracks) and sheets (metal profiled or plan sheets, wood-based panels or gypsumbased panels). The sheathings are connected to the CFS frame by self-piercing screws. The seismic performance of the shear walls is strongly dependent on their components characteristics particularly the sheathing-frame connections. Therefore, the step towards capturing the structure response is to find suitable modeling technique for a shear wall, which is easily integrated into a global model. This can be achieved by an acceptable evaluation of the initial rigidity and the elastic load bearing capacity of the wall. Post elastic characteristics can be obtained from full scale experimental results which are available for different type of sheathing configurations. For the purpose of this study, an equivalent simple nonlinear shear link connected to a rigid triangular shell element is introduced to account for the overall lateral stiffness and strength of a wall (Fig. 2.1).



Figure 2.1. Illustration of the shear wall model

#### 2.1. Elastic and Post-elastic characteristics of the shear wall model

The elastic rigidities of shear walls are determined using either an empirical or analytical method. The former can be calculated per AISI standard for cold formed steel framing:

$$K = \frac{R_n \times length_{wall}}{\Delta_{total}}$$
(2.1)

 $R_n$  is the nominal ultimate shear values to resist seismic forces. It depends on the type, the thickness of the panel and the fixing screws spacing. The nominal resistance can be obtained from the table C2.1 of the AISI S213 (AISI 2007).  $\Delta_{total}$  is the total deflection computed per AISI equation C2.1-2 (AISI 2007).

The lateral strength and the associated displacement can be also evaluated using the analytical method proposed by (Martinez 2007) which takes into account a broad range of factors that affect the behavior and strength of SWP, namely: material property, thickness and geometry of sheathing and studs, spacing of studs, and construction details such as size and spacing of the sheathing-to-stud fasteners.

A multi-linear model is derived using the equivalent energy elastic plastic (EEEP) method to match the post-elastic experimental curves of shear walls. Fig.2.2 shows a typical model together with the elastic and post-elastic limits.



Figure 2.2. Multi-linear model for shear wall force-displacement curve

- **R**<sub>u</sub> : Nominal shear strength;
- $\Delta_{u}$  : Ultimate displacement corresponding to  $R_{u}$ ;
- $\mathbf{R}_{0.4u}$  : Displacement corresponding to 40% of the nominal strength  $R_u$ ;
- $\mathbf{R}_{0.8u}$  : displacement corresponding to 80% of the nominal strength  $R_u$ ;
- $\mathbf{R}_{y}$  : Displacement corresponding to 85% of the nominal strength  $R_{u}$ ;
- $\Delta_{0.4u}$  : Elastic displacement limit;
- $\Delta_{0.8u}$  : Displacement at failure point corresponding to 80%  $R_u$  or a lateral displacement equal to 2.5% of the height of the wall.

Experimental results, such those published by Balh et al (2010) shows that the displacement ratio  $\alpha$  of the ultimate  $\Delta_u$  to the elastic  $\Delta_{0.4u}$  values vary from 3.28 to 2.51, with an average of 2.87. The ratio  $\beta$  of the failure displacement limit  $\Delta_{0.8u}$  to the ultimate displacement  $\Delta_u$  varies from 1.0 to 1.63 with an average value of 1.40.

# 2.2 Calibration of the hysteresis loop

The multi-linear plastic-pivot hysteresis model of the FEA software package, SAP2000 (CSI 2004) was used to account for the nonlinear behavior of the cold formed steel panel. The hysteretic model incorporates stiffness degradation, strength deterioration and non-symmetric response. Two rules are necessary to capture the hysteresis behavior of the shear hinge of a panel.

First, the parameters of the strength envelop are specified using those defined above for the multilinear curve.

Second, for each loading direction two factors are specified:

- $-\alpha$ , by which the yield strength in one direction is multiplied to define the position of the corresponding primary pivot point.
- $-\beta$ , by which the yield strength in one direction is multiplied to define the position of the pinching pivot point.

For calibration purposes, several specimens from the literature were modeled using the shear link element and subjected to CUREE test protocol similar to the experimental loading. The computed hysteresis loops are matched to those obtained experimentally in order calibrate the unloading slopes. Values of  $\alpha$  and  $\beta$  in the interval [0.1 - 0.2] and [8 - 12] respectively lead to a reasonable similar slopes. For a value of  $\alpha = 10$  and  $\beta = 0.15$ , the overall shape is portrayed by the numerical curves with some differences in the inner loops where the experimental loops show more pronounced degradation at subsequent cycles. Fig. 2.3 shows typical calculated and measured shear-deformation curve of the test specimen 11 (Bah N., Rogers C.A. 2010).



Figure 2.3. Typical experimental loop (Bah N. and Rogers C.A. 2010) captured by a multilinear plastic pivot hysteresis model (CSI 2004)

# **3. GENERAL DESCRIPTION OF THE CFS STRUCTURE**

A typical five story housing building made of cold formed steel located in a moderate seismic area was considered in this study. The layout of the ground floor is 23.75 m long and 12.50 m large. The structural system is made of load bearing walls. The lateral resisting system in both directions is composed of shear walls located in the peripheral and internal walls as shown in Fig. 3.1.



Figure 3.1. Building plan layout and shear walls locations

The frames of the panels are made of cold formed profiles with different sections, placed at 600 mm intervals. The studs and tracks are stiffened using 10 mm thick magnesium board for interior walls and corrugated steel sheets fixed to one or both sides for shear walls. Hot rolled steel elements at the ends of the shear walls are used to resist the induced axial forces. Material properties of the cold-formed steel used in this structure are for members of 1.14 mm thickness and lighter having minimum yield strength of 228MPa. All members 1.4 mm thickness and heavier were formed from steel with minimum yield strength of 345MPa. This structure is designed to resist the dead load, live load, wind load and seismic load for Seismic Zone II (RPA99v2003).



Shear walls

Figure 3.2. Global views of the building during construction and when completed

# 4. AMBIENT VIBRATION TESTS

Two series of ambient vibration testing were carried out on a bare structure and a fully completed building. For this particular case, four measurements points were performed on the top and the third floor to capture the lateral and torsional fundamental frequencies. The tests were performed using three degrees of freedom seismometer type Lennartz electronic (Le3Dlite) and a data acquisition system type City Shark II. The measured signals were processed using the GEOPSY program (Wathelet 2005) capable to perform most of the signal processing operations for the analysis of ambient vibration data. The sensors were located at the centre and the corner of the floors. The recording time for each sequence was set to 4 mn and found to be largely sufficient to obtain smooth FRF curves. The natural frequencies of the building were identified using a "peak cursor" on the frequency response functions. The curves in Fig. 4.1 shows the FRF of the transverse and longitudinal vibrations measured on the corner of the top floor. The clearly distinct two first peaks at 4.81 Hz and 7.44 Hz correspond to the fundamental lateral mode on the transverse direction and the torsional mode. Along the perpendicular direction the peak at 3.90 Hz correspond to longitudinal frequency. The second pair of FRF curves (Fig. 4.2) is obtained from a measurement on the completed building and shows that the frequencies have increased to reach respectively 5.64 Hz, 8.24 Hz and 4.78 Hz.



Figure 4.1. The measured FRF curves of the bare shear wall structure



Figure 4.2. The measured FRF curves of the completed building

As shown in table 4.1, the experimental lateral frequencies are very close (less than 5%) to those estimated by the analytical model using the procedure described in section 2.1 to compute the shear wall rigidities. It should be noted that usually the measured frequencies at very low ambient vibration amplitudes are higher because they represent the initial rigidities of the shear walls.

The second series of vibration testing on the completed building generate higher frequencies because of the contribution of infill panels to the overall rigidity of the building. The lateral frequencies increased about 20% and the torsional frequency about 10% as most of the infill panels are in the interior walls. The damping ratio has increased for the completed building and the obtained values are within the range of values obtained for low vibration amplitudes.

		Bare shear wall building			Fully finished building	
Mode	Direction	Analytical Frequency (Hz)	Experimental Frequency (Hz)	Damping (%)	Experimental Frequency (Hz)	Damping (%)
1	Longitudinal	3.91	3.90	0.26	4.76	0.35
2	Transversal	4.60	4.78	0.22	5.64	0.86
3	Torsional	7.30	7.44	0.40	8.24	1.10

**Table 4.1.** Natural frequencies and corresponding damping ratios

# 5. SEISMIC PERFORMANCE OF THE CFS 5 STOREY BUILDING

# 5.1 Equivalent static method

One of the objectives in developing this modeling technique is to provide a simple procedure to model CFS structures for design analysis. In this section, the results in terms of shear forces in the SWP and the story drift computed by the finite element model under static seismic loads are compared to those obtained by the conventional method based on the distribution of the seismic forces proportional to the overall SWP stiffness'. Moreover a modal spectral analysis is also carried out and the results are correlated to those of the conventional method. The results are summarized in Table 5.1. It can be easily noticed that the shear forces computed by the model are close to those of the conventional method. The differences can be attributed to the fact that in the conventional method the total seismic forces are totally resisted by the SWPs.

# 5.2 Nonlinear dynamic analysis

# 5.2.1 Seismic input

For the purpose of the present analysis the model described previously is subjected to five ground accelerations. In addition to the ground acceleration of El-Centro and Kobe earthquake, three other real earthquake ground motions, which have been recorded during Boumerdes (Algeria) earthquake in 2003 at different stations: Dar-El-Beida, Hussein-Dey and El-Affroun distant of 50km, 65km and 110km respectively from the epicenter, have been used. A ground acceleration time history artificially generated has been added to the ensemble. These earthquake accelerograms were adjusted to fit the design response spectrum of the site in the period range 0.02 to 2.0s using the SeismoMatch program (seismosoft 2009). SeismoMatch gives users the opportunity to simultaneously match a number of accelerograms, and then obtain a mean matched spectrum whose maximum misfit respects a predefined tolerance (Fig. 5.1). The complete list of these earthquakes with the main characteristics appears in Table 5.1.

Table 5.1. List of the earthquake ground acceleration time histories

Station	Dar-el- Beida 2003	Hussein-dey 2003	El Affroun 2003	Kobe 1995	El Centro 1940	Artificial
<b>Duration</b> (s)	25	20	30	30	30	15
PGA (g)	0.540	0.232	0.164	0.332	0.313	0.436



Figure 5.1. Matched accelerograms and target spectrum

### 5.2.2 Nonlinear dynamic response

The inelastic behavior of the CFS structure, which is subjected to the abovementioned earthquake ground motions, is investigated in this section. This study focuses on the following basic design parameters: maximum horizontal floor displacements and inter-storey drift ratios, SWP hysteresis loops and the energy dissipation capacity. Furthermore, the incremental dynamic analysis (IDA) technique is used to investigate the overall post-elastic behavior of the structure and to determine the failure mechanism.

The inter-storey drift ratios for all earthquake ground motions and all storeys are relatively uniform and lay between 0.3% and 0.7% of the storey height. The maximum drift ratio increases almost linearly with the ground acceleration scale factor in the range [1.0 - 2.0] and reaches the code limit 1%h for an amplification factor of 1.75 (Fig. 5.2).



Figure 5.2. Inter-storey drift ratios variation with height and ground acceleration scale

Statistically, most of the shear wall pannels of the structure attain the elastic limit under the different earthquake ground motions. When the ground acceleration is increased, more shear wall pannels enter the post-elastic range. About 9% of the SWP reached their ultimate displacement (failure status) for a ground acceleration scale factor sf = 1.75. and 20% when the scale factor is equal to 2. At this stage only 6% of the SWP remained in the elastic range (Fig. 5.3). This behaviour indicates that the majority (more than 70%) of the SWP undergo post-elastic deformation at an early stage when subjected to an earthquake ground motion. The failure of some SWP appeared at high level of ground acceleration.



Figure 5.3. Statistics of the SWP status for increasing ground acceleration scale

A typical shear force time history in a SWP at the first storey under a full scale earthquake acceleration is dominated by the fundamental frequency and became altered by a wave elongation due to the non linear behavior of the SWP when the seismic input is twice the original (Fig. 5.4).



Figure 5.4. Shear force in a SWP of the first storey

At relatively low level of yielding of the SWP, the shear-deformation curves are characterized by pinched loops with poor energy dissipation capacity (Fig. 5.5). For increasing level of yielding, the hysteresis energy capacity improves slightly but it remains below 25% of the total dissipated energy (Fig 5.6). The hysteresis loops for lower stories becomes larger during the strongest part of the ground acceleration time history. About 20% of the SWP reached their ultimate deformation (failure status) when the ground acceleration scale factor increased to two times which corresponds to a PGA equal to 0.54g. The analysis revealed that the first elements to fail were the SWP of the first storey but they did not lead to permanent deformations or a premature failure mechanism because of a sufficient restoring force provided by the remaining SWP.



Figure 5.5. Hysteresis loops of the lowest SWP under scaled earthquake ground accelerations



Figure 5.6. Energy time histories for scaled earthquake ground accelerations

### 6. CONCLUSION

Computational modeling of shear wall panels made of cold-formed steel requires sophisticated mechanics which are often beyond the level of engineering practice. This paper presents a simple and efficient modeling technique for the analysis of cold formed steel structures. For this purpose, a multilinear model for shear wall force-displacement curve is developed. Using a single SWP element, the hysteresis loops are validated against available experimental data. At a structure level, a case study of an existing multi-storey building is used to compare the analytical frequencies and those obtained by ambient vibration testing. The performance of this modeling technique was evaluated in the linear domain using the equivalent static force. The results of the FE model and those obtained by the conventional method are in good agreement. The nonlinear performance has been also investigated by subjecting the model to several earthquake ground motions. The post-elastic response of the structure remains dominated by the elastic behavior with almost insignificant permanent deformation. The hysteresis energy dissipated by SWP contributes moderately to damp out the response of the structure. However, when increasing the level of the ground acceleration, the shear-deformation curves of the SWP of the low storey exhibit larger loops and more energy is dissipated. A damage survey indicates that only few SWPs reach their ultimate deformations at this ground acceleration level. From the findings of the comparison study it was ascertained that the modeling technique is very appropriate for design analysis of cold formed steel structures.

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