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EQUIVALENT SHEAR LINK MODELING AND PERFORMANCE ANALYSIS OF COLD FORMED STEEL STRUCTURES UNDER EARTHQUAKE LOADING

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ABSTRACT

Cold-formed steel members and components have been produced and widely used as structural elements for buildings. However, the design analysis of such structures is often complex due to the slenderness of members, walls and cross-sections. This paper introduces a simple model based on a nonlinear equivalent shear link connected to a rigid triangular shell element to account for the overall lateral stiffness and strength of the wall panels. Series of ambient vibration testing on a five story building is carried out to validate the initial elastic stiffness of the wall panels. A mutilinear plastic-pivot hysteresis model integrating stiffness degradation, strength deterioration and non-symmetric response is then used to account for the nonlinear behavior of the cold formed steel panels. The numerical model of the structure was subjected to a ground acceleration time history recorded during a recent earthquake in the region. The performance of the structure, in terms of lateral displacement, energy dissipation capacity and structural damage, is investigated using nonlinear time history analysis.

The results elucidate some aspects of the stiffening and energy dissipating capacity role of the shear wall panels, and highlight inadequacies in the FE modeling for a better modeling practice.

Introduction

The growth in structural use and understanding of the behavior of structures made of cold formed steel started more than half a century ago. Modern design specifications have taken substantial steps in providing design analysis methodology, but these are becoming more complex, and can nowadays involve greater labour than rigorous analysis using numerical methods implemented in dedicated computer packages.

The major factor, which arises in the design of cold formed steel members, is the susceptibility of these members to a wide variety of buckling modes. The thin walls of such members are often

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liable to suffer local buckling in compression, and this must be taken into account in the design of almost any cold formed steel structural member. Local buckling is stable in the elastic range, and locally buckled members may have substantial post-buckling strength. However, local buckling does modify the behavior of a member, and the effects of local buckling on member behavior, and its interaction with other buckling modes, must be considered in the design (McDonald 2008 and Schafer 2006).

The complications induced by such effects occurred more often under severe earthquake loadings. This brings the requirement to ensure that the resistance and stability of the lateral resisting elements are precisely modeled. In this context, in the few last decades, technical advances have been made in seismic resisting cold formed steel buildings and particularly in the dynamic behavior of the shear walls. These lateral resisting elements were extensively investigated experimentally and analytically in order to establish design tables containing the ultimate capacities for static and seismic stress for different wall assemblies (Serrette 1996, Serrette 1997, Moghimi 2009 and Gad 1999). Design procedures and analytical methods were also developed to allow for the design of walls carrying horizontal and vertical loads (Lange 2006, Al-Khrat 2007 and Serrette 2006).

The overall seismic behavior of cold formed structures has been also investigated in post-elastic domain. The performance of the building, as a whole, depends on the wall panels, which is governed 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). Along with the lack of the implementation data of the guidelines, the FE models also needed to be investigated in terms of their accuracy and efficiency. As refined 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. In this context, this paper proposed a simple modeling technique based on equivalent shear link element to idealize the wall panels. The performance of a multistoried building is evaluated in terms of lateral displacement, energy dissipation capacity, and structural damage.

Description of the 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. 1.

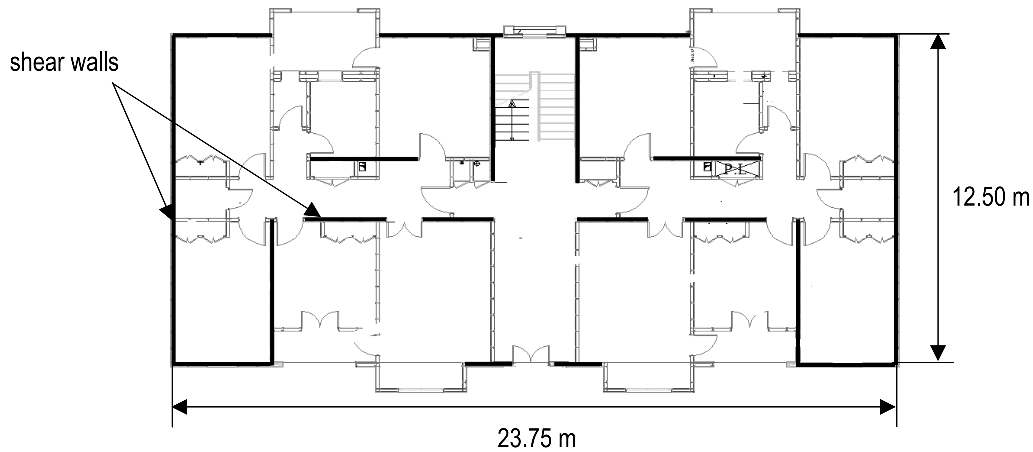


Figure 1. Building plan layout and shear walls locations

The frames of the panels are made of cold formed profiles with different sections, placed at 650 mm intervals. The studs and tracks are stiffened using 12 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 (Fig. 2). 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).

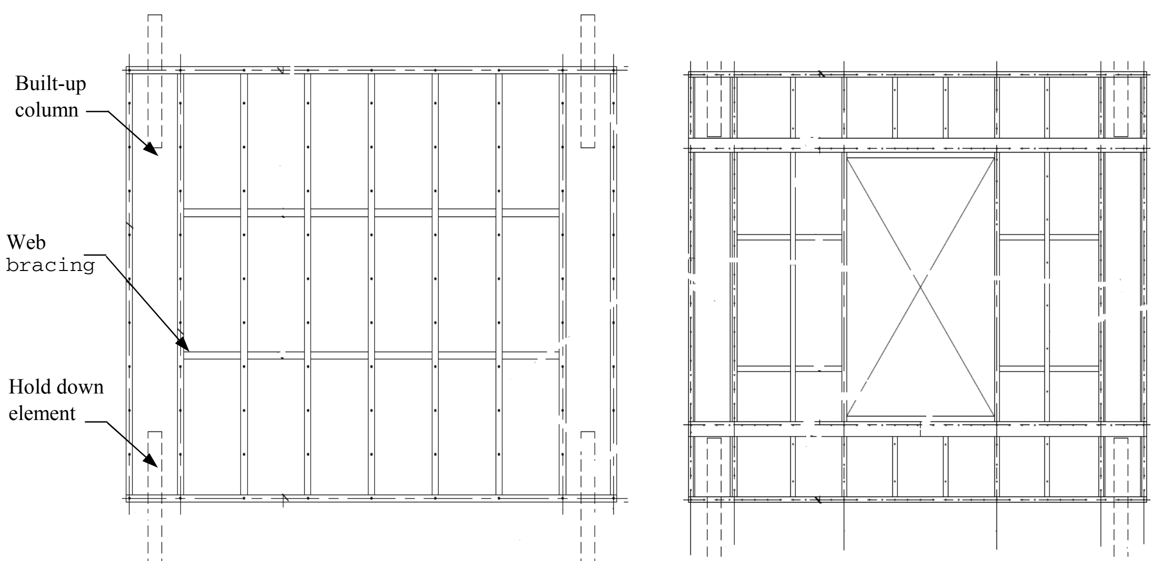


Figure 2. Typical shear wall panels with and without opening

a- completed building

b- building with shear walls only

Figure 3. Global views of the building

Numerical Modeling

Undoubtedly, one of the most important parameters to consider for a structure is the actual behavior characteristics of its structural components. As stated earlier, in the case of cold formed structures the basic structural elements such as shear or vertical load bearing walls are made of different components (studs, tracks, sheets) fastened together to form a single member. Therefore, the step towards capturing the structure response is to find suitable modeling technique for a panel, 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 panels. Post elastic characteristics can be obtained from full scale experimental results which are available for a limited number of sheeting 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 panel (Fig. 4). The elastic rigidities of shear walls are calculated using AISI standard for cold formed steel framing:

$$K = \frac{R_n \times length_{wall}}{\Delta_{total}} \tag{1}$$

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. Δ_{total} is the total deflection computed per AISI equation C2.1-1 (AISI 2007).

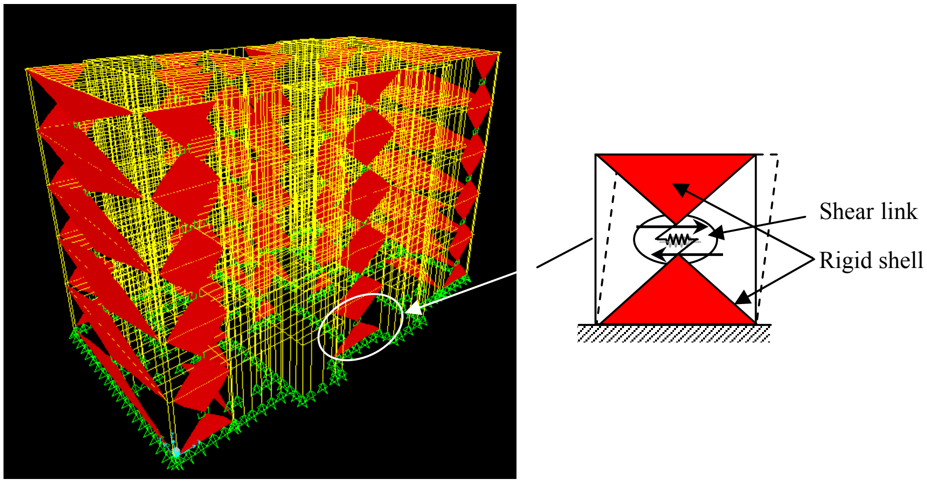


Figure 4. The 3-D model with an illustration of the shear link model of a panel

The mutilinear 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.

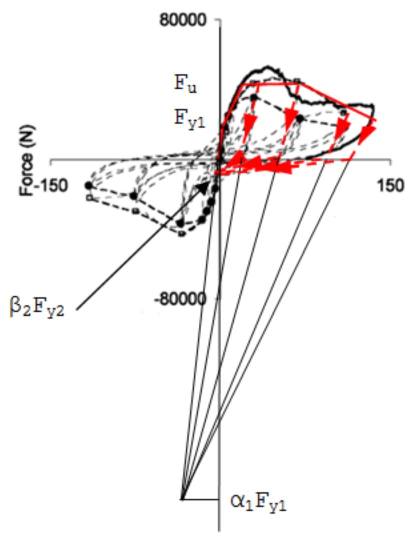


Figure 5. Typical experimental loops capture by a multilinear plastic pivot hysteresis model (CSI 2004) and (Fulop 2004)

Three rules are necessary to capture the hysteresis behavior of the shear hinge. For each loading direction two factors are specified:

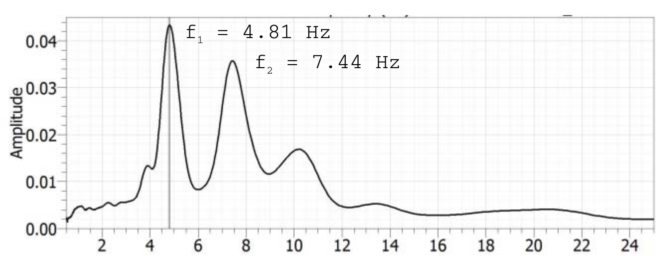
- α, by which the idealized yield strength in one direction is multiplied to define the position of the corresponding primary pivot point.
- β, by which the idealized yield strength in one direction is multiplied to define the position of the pinching pivot point.

In addition, the strength envelope is calibrated graphically as a quadric-linear curve using the experimental curve of a cyclic loading of corrugated sheet specimens (Fulop 2004) as shown on Fig. 5. The envelope is positioned proportionally to the nominal ultimate shear value and the elastic stiffness of a wall determined using AISI S213-07 (AISI 2007).

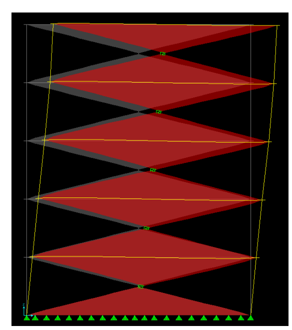
Ambient Vibration Testing

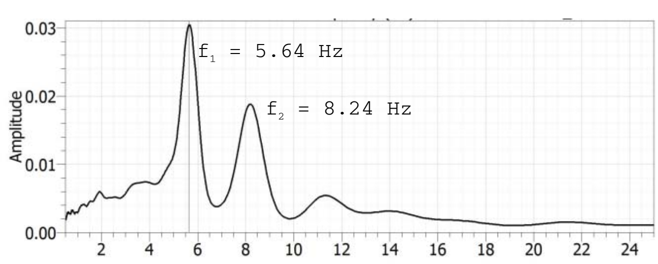
The elastic dynamic properties, particularly the natural frequencies and the corresponding mode shapes are a combined measure of the structural characteristics of the construction. These model characteristics can be successfully estimated, especially in elastic range, using the well known ambient vibration testing (Bourahla 2005). In this paper we present briefly the main issues pertaining to this particular modal testing; frequency response function (FRF) measurement techniques, testing procedure, and modal parameter estimation method. For this particular case, four measurements points were performed at 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. In order to estimate the contribution of the non structural magnesium board sheeting to the overall dynamic characteristics of the building, series of ambient vibration testing were carried out on the building with the shear walls only. The natural frequencies of the building were first identified using a "peak cursor" on the frequency response functions.

The first curve in Fig. 6 shows the FRF of the transverse 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. The second FRF curve is obtained from a measurement on the completed building and shows that both the transverse fundamental lateral frequency and the torsional frequency have increased to reach respectively 5.64 Hz and 8.24 Hz.

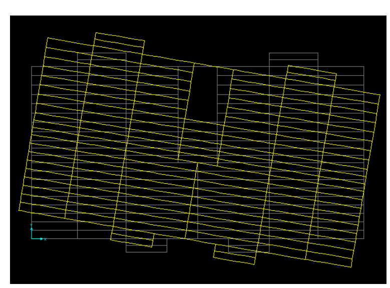


a-bare shear wall building





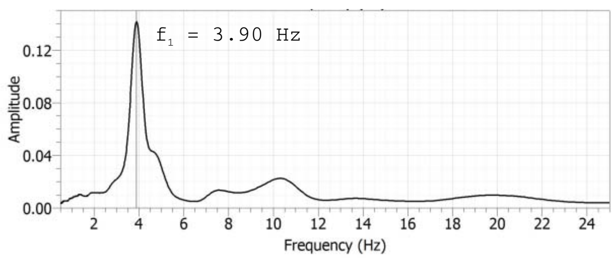
b- completed building



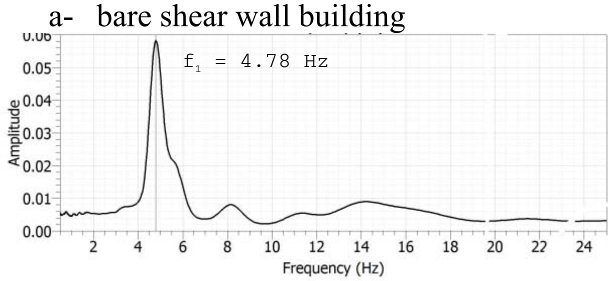
Torsional mode

Figure 6. The measured FRF curves and the numerical transverse and torsional mode shapes

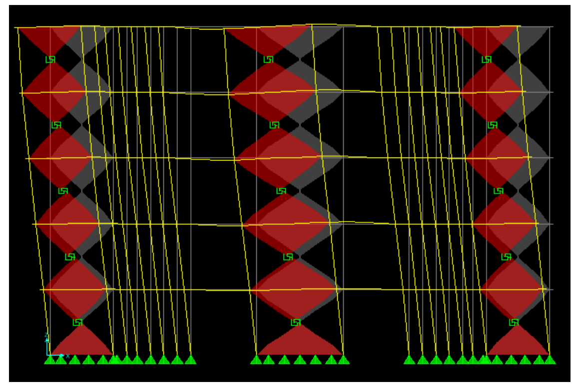
Along the longitudinal direction the FRF curves (Fig. 7) are characterised by a single peak indicating a fundamental frequency equal to 3.90 Hz for the bare shear wall building and 4.78 Hz for the completed building.



a- bare shear wall building



b- completed building



Longitudinal lateral mode

Figure 7. The measured FRF curves and the corresponding numerical longitudinal mode shape

As shown in table 1, the experimental lateral frequencies are about 10% higher than those estimated by the analytical model using Eq. 1 to compute the shear wall rigidities. This is reasonably acceptable because the measured frequencies at very low ambient vibration amplitudes represent the initial rigidities of the shear walls whereas the computed rigidities correspond to the ultimate deformation. The discrepancies in the torsional frequencies is attributed to low estimate of the rigidities of the shear walls at the ends of the building which contribute most to the torsional rigidity of the building. It should be noted that these two shear walls have an aspect ratio which exceeds 1:4 (12.5 m long and 3.06 m height). The actual rigidities of such wide shear walls are higher than those evaluated by the AISI equation C2.1-1 (AISI 2007).

Table 1. Natural frequencies and corresponding damping ratios

		Bare shear wall building			Fully finished building	
Mode	Direction	Analytical Frequency (Hz)	Experimental Frequency (Hz)	Damping (%)	Experimental Frequency (Hz)	Damping (%)
1	Longitudinal	3.55	3.90	0.26	4.76	0.35
2	Transversal	4.50	4.78	0.22	5.64	0.86
3	Torsional	6.13	7.44	0.40	8.24	1.10

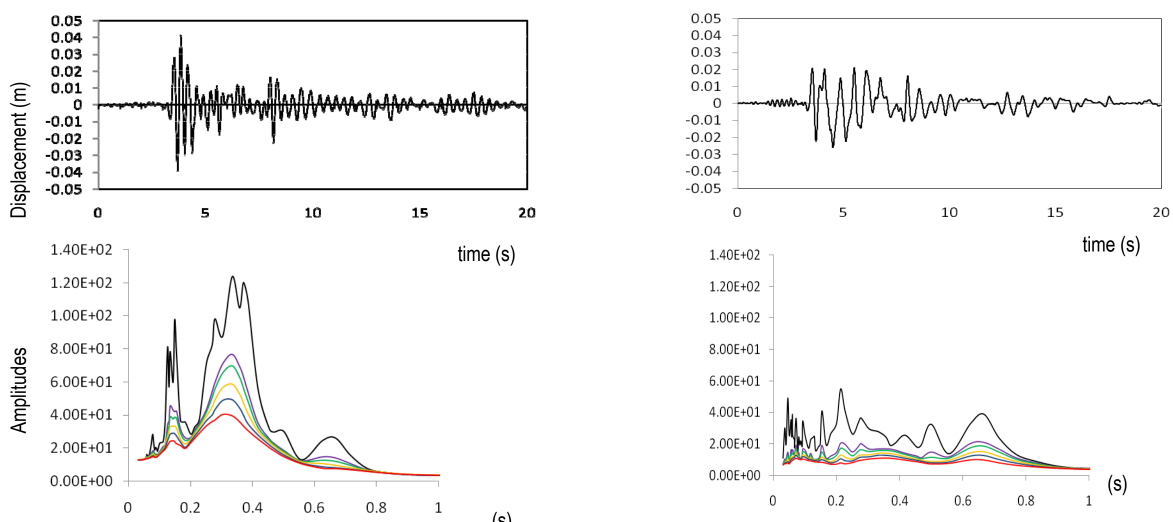
The second series of vibration testing on the completed building generate higher frequencies because of the contribution of infill panels to the overall rigidity of the building. The lateral frequencies increased about 20% and the torsional frequency about 10% as most of the infill panels are in the 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.

Nonlinear Seismic Performance

Nonlinear dynamic analyses are carried out to investigate the overall behavior of the building under earthquake ground motion. For the purpose of the present analysis the model described previously is subjected to a ground acceleration recorded during Boumerdes (Algeria) earthquake in 2003. The duration of the strong motion used for the analysis is 20 seconds. The response analysis is carried out using the E-W component with a PGA equal to 0.52g. Modal damping has been included in the model and this was set to 5% for all modes as is usual practice for steel structures. The step-by-step numerical integration is carried out at time interval 0.005 second.

For a comparison reason, a linear dynamic analysis is performed assuming that all elements remain in the elastic range. The global response measured in terms of the top floor displacement indicates that the overall behavior is dominated by the fundamental frequency as shown by the peak of the response spectra of the displacement time history (Fig. 8).

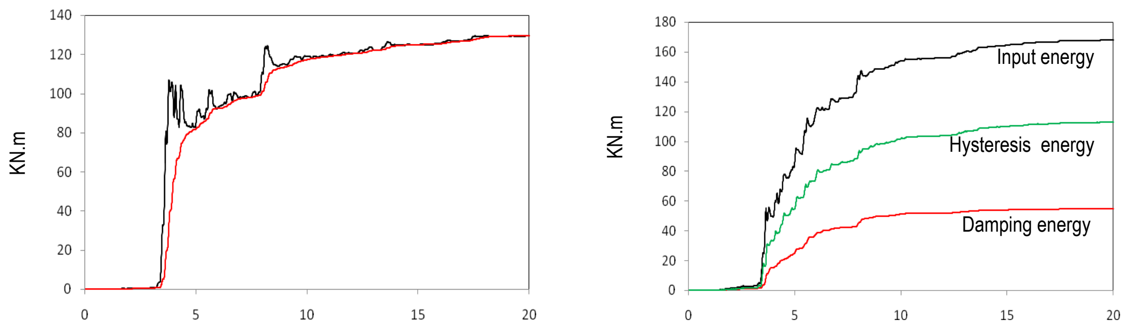
Under the same earthquake ground motion conditions to which the linear model has been subjected, the structure with plastic pivot multi-linear characteristics exhibit different behaviour. At an early stage several panels undergo plastic deformations at the lower stories and propagate to the upper stories during the severe phase of the ground acceleration. The building showed evidence of a considerable variation of the overall stiffness which become visible on the response time history characterised by an elongated waveform (Fig. 9). The maximum displacement of the top floor is more than 30% lower than the linear model and more importantly, the vibration are damped out after the main peak acceleration with almost no permanent deformation at the end of the response.



a- Linear response b- nonlinear response

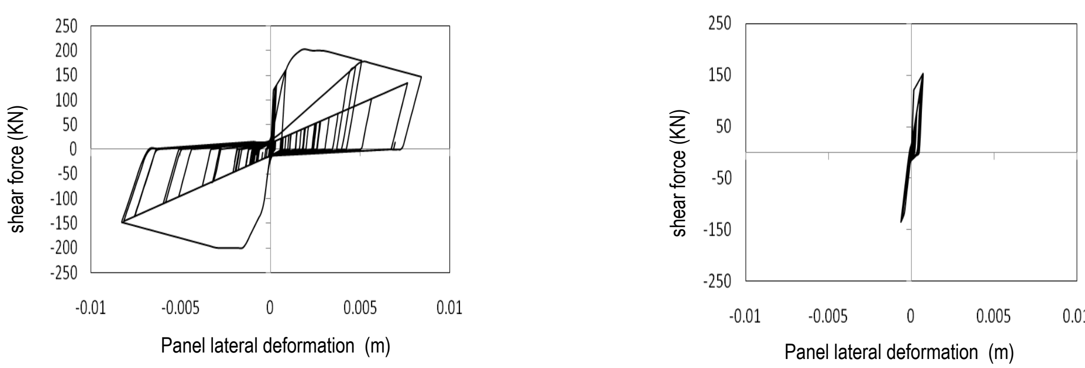
Figure 8. Top floor displacement time history and spectra responses

The relatively low yield strength of the panels provide the building with an energy dissipation capacity at an early stage of the response which help damp out intense vibration and detune the structure from possible resonance. The energy dissipated by hysteresis effect of the shear wall panels is twice the modal damping energy of the structure. The curve of the input energy is characterized by small fluctuations which reveal that the energy imparted by the ground motion to the structure is dissipated almost instantaneously which indicate an efficient dissipation capacity (Fig. 9). It should be noted however that the yielding is concentrated in the lower stories. The panel shear–deformation curves of the lower stories are characterized by pinched inner loops with a large hysteresis loop corresponding to the strongest part of the ground acceleration time history. Smaller loops, however, characterize the hysteresis curves of the upper story (Fig. 10).



a- Linear energy time history b- Nonlinear energy time history

Figure 9. Energy time histories



a- Lower story b- upper story

Figure 10. Shear-deformation curves of the shear walls

Conclusions

Shear wall panels made of formed cold steel have a complex behavior because of the slenderness of their components and the mode of fastening. This paper presented a simple and efficient modeling technique for the analysis of building with shear wall panels.

The model is based on the multi-linear plastic pivot hysteresis curve, but any suitable relationship could be used. This makes it particularly attractive for non-linear response history analysis, but could be of great help for engineers to perform non-linear static analysis (push-over) both at the design stage or the seismic evaluation of existing structures.

The ambient vibration testing used at different construction stages is practical in extracting the rigidity contribution of different structural and non structural components of the building. The results showed that the relative rigidities of shear walls estimated by the AISI standard for CFS framing for lateral design are in good concordance with the measured values except for wide shear walls with aspect ratio in the vicinity of 4:1.

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