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Abstract

Cold-formed steel framed shear wall sheathed with corrugated steel sheets is a promising shear wall system for low- and mid-rise constructions at high wind and seismic zones due to its advantages of non-combustibility, high shear strength, and high shear stiffness. A lot of work has been done on this subject. However, all the previous work is focused on the wall panel levels and more research work is needed on the entire building systems. The objective of this paper is to investigate the response of a cold-formed steel framed building with corrugated sheet sheathing subjected to earthquake excitation primarily through nonlinear time history analysis employing the incremental dynamic analysis (IDA) framework. High fidelity models were simulated in OpenSees program. The detailed modeling information and system assessment are presented in this paper.

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Introduction

The cold-formed steel (CFS) corrugated sheet is widely used as the floor decking and roofing materials in both residential and commercial buildings. Only recently, CFS corrugated sheets have been used as sheathing material in shear walls. Researchers have been focusing on the performance of this new type of shear wall as the primary lateral resistance system. Fülöp and Dubina (2004) developed a testing program to investigate the structural characteristics of 2.44 m high × 3.66 m wide CFS shear walls with different sheathing materials including LTB20/0.5 corrugated steel sheet, gypsum boards, and OSB. A total of 7 monotonic tests and 8 cyclic tests were conducted. The test results indicated that the CFS walls were rigid and could effectively resist lateral loads. The failure of the seam fastener was the failure mechanism for the corrugated sheet specimens.

Stojadinovic and Tipping (2007) conducted a series of 44 cyclic shear wall tests on 2.49 m high × 1.22 m or 0.61 m wide CFS shear walls with corrugated sheet steel sheathing on one side or both sides. The shear walls specimens differed in gauge of the sheet steel, gauge of the cold-formed steel framing, size and spacing of the fasteners. Stojadinovic and Tipping reported that in all the tests, the failure mode was the eventual pulling out of the screws due to warping in the corrugated steel sheet.

A series of full scale shear wall tests were conducted at University of North Texas (UNT) in recent years (Yu el al. 2009, Yu 2013). The test program used typical framing configurations and the approved test method by International Code Council. The test results indicated that the CFS framed shear walls using corrugated steel sheathings demonstrated higher strength, greater initial stiffness and a similar ductility in comparison to CFS walls using conventional sheathing materials (flat steel sheets, plywood panels, OSB boards).

In order to investigate the influence of gravity/vertical loads and to assist in the fragility analysis recommended by FEMA P695 (2009), another test program on CFS shear wall with corrugated steel sheet sheathing was conducted recently at UNT under combined gravity/vertical and lateral loading (Zhang et al., 2016). The tests involved 4 shear wall specimens and 4 bearing wall specimens. The results indicated that moderate gravity loading led to an increase of shear strength and initial stiffness. Also, the bearing walls contribute almost 34.2% of shear strength and 35.5% of dissipated energy in comparison to shear walls. It's observed that both shear walls and bearing walls were able to carry the full weight during entire loading process without collapse. As a result, 7% drift was recommended by the authors as collapse drift limit in seismic fragility analysis.

The objective of this paper is to investigate the response of cold-formed steel framed buildings with corrugated sheet sheathing under earthquake excitation and to produce appropriate seismic performance factors for design usage. High fidelity models of one 2-story and one 5-story office building were simulated in OpenSees program (McKenna, 2015). The detailed modeling information and relevant system assessment are presented in this paper.

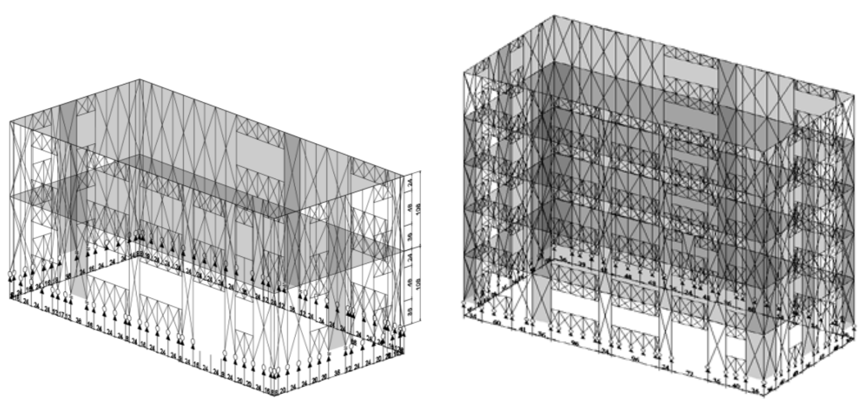
Finite Element Modeling

Building Prototype

The building archetype used in the NEES-CFS project (Madsen, Nakata, Schafer, 2011) was adopted as a reference in this research. The NEES building was redesigned by the authors to employ the CFS shear walls with corrugated steel sheathing. The hypothetical office buildings were assumed to be located in Orange County, California which has a total plan layout of 49.75ft×23ft (15.2m ×7m). Site Class D was chosen as is typical for sites in the vicinity of this project. For the office occupancy chosen, IE = 1.0 was used. The seismic force modification factors were based on wood light-frame shear wall systems with wood structural panel (ASCE/SCE 7-10), and were set at R = 6.5 and Ω= 3.0.

OpenSees Building Modeling

The nonlinear dynamic analysis software OpenSees (McKenna, 2015) was used in the FE analysis. The length and distribution of shear wall were re-assigned based on the test data since the sheathing material has changed from OSB to corrugated steel sheet. Figure1 illustrates the schematic drawings of FE models used in OpenSees (McKenna, 2015).



1a - 2-story building 1b - 5-story building

Figure 1 - OpenSees models

Modeling of Shear Walls

The shear walls were simulated in OpenSees (McKenna, 2015) as two diagonal truss elements and elastic frame boundary elements as illustrated in Figure 2. Rigid connection method was used since linear static analysis results showed that the diagonal bracing stiffness greatly exceeded the small moment stiffness of the stud-to-track connection. In order to achieve the pinching effect, the strength degradation as well as the stiffness degradation of the shear wall, pinching4 uniaxial hysteretic material was used for the diagonal truss elements. To obtain the backbone curve of pinching4 material, the horizontal load V vs. deflection Δ was first converted to stress-strain relationship according some derivation of basic equilibrium and geometry:

The axial force in the diagonal bracing F can be expressed as:

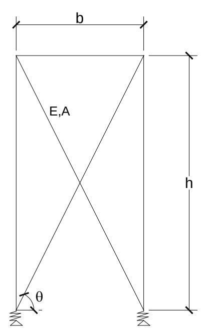
$$F = V / (2\cos\theta)$$

The stress and strain in the diagonal bracing can be obtained as:

$$\sigma = F / A = V / (2A\cos\theta)$$

$$\varepsilon = d / l = (\Delta\cos\theta) / l$$

Where $\cos\theta = b / \sqrt{b^2 + h^2}$, $l = \sqrt{b^2 + h^2}$. Herein b, h is the width and height of the shear wall respectively.



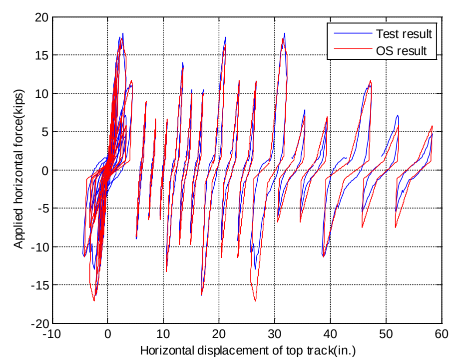


Figure 2 - Shear wall Modeling   Figure 3 - Simulation of shear wall

The OpenSees result was compared to the test result in Figure 3. It can be seen that the model has a good agreement with the test result and the model was able to simulate the post-peak behavior of the shear wall.

Modeling of Bearing Walls

In the building model, the bearing walls were designed to have the same sheathing material as the shear walls. Shear resistance of the bearing walls was considered in the FE analysis. The modeling technique of bearing walls was same as the shear walls.

The backbone curve and perimeters of pinching4 material of shear walls and bearing walls were according to the test results in Zhang et al. (2016). Aspect ratio adjustment recommended in AISI S213 (2012) was performed when the width of the wall in the building was different from the width of test specimen. As for the small bearing walls at the opening positions (windows and doors), ABAQUS model was first created for each height of wall and then aspect ratio adjustment was performed. The ABAQUS modeling technique was according to Mahdavian et al. (2016).

Modeling of Diaphragm

Rigid diaphragm was used in the model by a built-in element in OpenSees (McKenna, 2015). The rigid diaphragm element requires a master-slave relationship of nodes in the same plane. Lateral displacement in two directions and rotation about the vertical axis is defined at the master node.

Seismic Mass and Gravity load

Total seismic mass was set to the value from the design narrative (Madsen, Nakata, Schafer, 2011) and the mass of each story was divided equally and lumped to the four corners. Gravity load of the building should be added separately since seismic mass is only related to the mass matrix in the FE formulation. The weight applied herein was the product of seismic mass and the acceleration of gravity g. P-delta effect was included since large displacement might arise.

Static Pushover Analysis

Pushover analysis is performed in order to obtain the ductility parameter and system over-strength factor. The displacement ductility factor is defined as $\mu_T = \frac{\delta_u}{\delta_y}$, where δ_u is the displacement at peak load and δ_y is the displacement at yield. Over-strength factor is defined as $\Omega_0 = \frac{V_{max}}{V_{design}}$, where V_{max} is the maximum base shear in actual behavior and V_{design} is base shear at design level. The displacement ductility factor and over-strength factor are listed in Table 1.

Table 1 - Pushover results

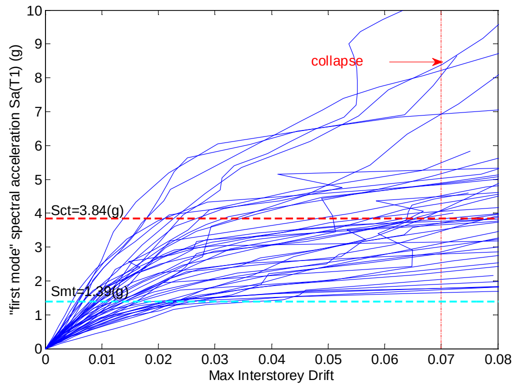
	Ω_0	μ_T
2-story building	8.69	2.07
5-story building	3.84	1.92

Incremental Dynamic Analysis

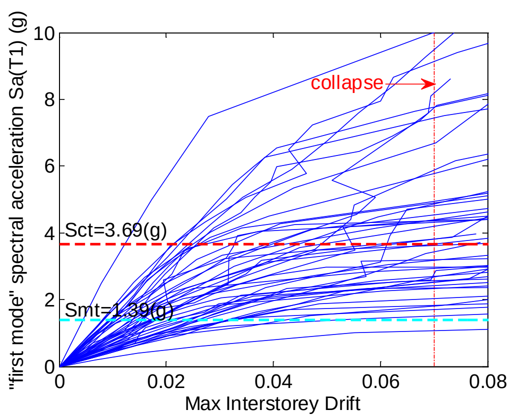
Nonlinear time history analysis lies in the core of the Incremental Dynamic Analysis method (IDA), where the structure is subjected to a suite of ground motion records. Every record is scaled to multiple levels of intensity until a designated DM limit for collapse is reached, producing the structure's capacity curve in terms of structure damage measure (DM) versus an intensity measure (IM). Story drift is a typical DM and the spectral acceleration of the first natural period of the structure is a typical IM.

To avoid bias, a specified set of ground motion records should be utilized as excitations. FEMA P695 (2009) recommends two sets of ground motion records for collapse assessment using nonlinear dynamic analysis: Far-Field record and Near-Field record set. The Far-Field record set includes twenty-two component pairs of horizontal ground motions from sites located greater than or equal to 10 km from fault rupture. The record sets do not include the vertical component of ground motion since this direction of earthquake shaking is not considered of primary importance for collapse evaluation, and is not required by the Methodology for nonlinear dynamic analysis. The Near-Field record set is only for supplemental information and is used in special studies to evaluate potential differences in the CMR for SDC E structures. As a result, the Far-Field record set was chosen and horizontal components of ground motion were used.

Figure 4 indicates the IDA curves and Figure 5 indicates collapse fragility curves of the two building models. The median collapse intensity, SCT, is defined as the spectral acceleration causing 50% collapse probability. The ratio between the median collapse intensity (SCT) and the Maximum Considered Earthquake (MCE) intensity (SMT) is the collapse margin ratio (CMR). CMR is the primary parameter used to evaluate the collapse safety of the building design.

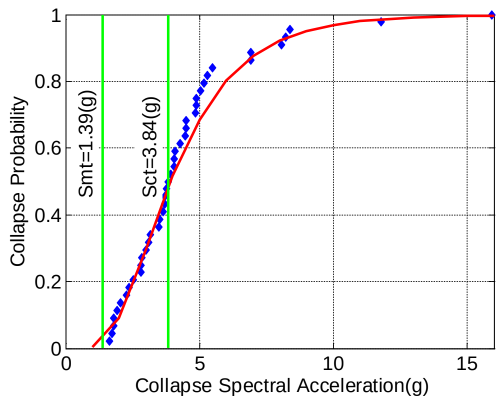


4a - 2-story building

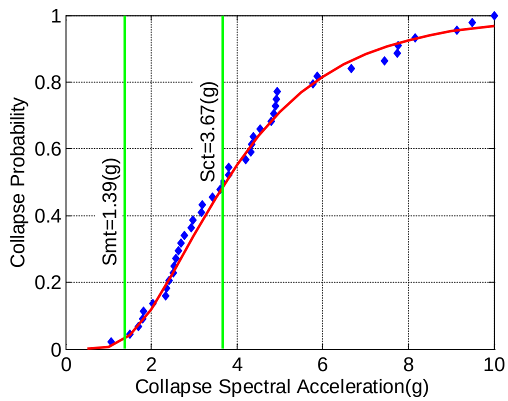


4b - 5-story building

Figure 4 - IDA curves



5a - 2-story building



5b - 5-story building

Figure 5: Collapse fragility curves

Building Performance Evaluation

According to FEMA P695 (2009), the collapse capacity is influenced by different sources of uncertainty. The sources of uncertainty include: uncertainty due to record-to-record variation, β_{RTR}; uncertainty due to design requirements, β_{DR}; uncertainty related the test data, β_{TD}; uncertainty related to modeling of the structure, β_{MDL}. FEMA P695 (2009) quantifies each of these uncertainties based on the following scale: (A) superior, β = 0.10; (B) good, β = 0.20; (C) fair, β = 0.35; and (D) poor, β = 0.50. The total system collapse uncertainty, β_{TOT}, is calculated based on these four uncertainties: $\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}$ .To account for the effects of the frequency content (spectral shape) of the applied earthquake record set, the CMR was adjusted using the spectral shape factor, SSF. For each archetype building, the adjusted collapse margin ratio, ACMR was calculated by multiplying the CMR by SSF (spectral shape factor).

Table 2 summarizes the aforementioned data, specifically, the median collapse intensity, S_{CT}, the collapse margin ratio, CMR, the adjusted collapse margin ratio, ACMR, and is compared with the reference value given in FEMA P695 (2009).The Record-to-record collapse uncertainty is calculated based on $0.2 \leq \beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.4$ ($\mu_T \leq 3$). The design requirements-related uncertainty, the test data related uncertainty and modeling of structure related uncertainty were taken as good. Results in Table 2 show that the collapse probability well passed the FEMA requirements, which improved that the design method, including the seismic force modification factors of R=6.5 and Ωo = 3.0, is appropriate for shear wall systems with corrugated steel sheet sheathings.

Table 2 - IDA results

	SCT	CMR	SSF	ACMR	$\boldsymbol{\beta_{TOT}}$	$ACMR_{20\%}$
2-story building	3.84	2.76	1.134	3.130	0.463	1.476
5-story building	3.69	2.65	1.125	2.981	0.453	1.464

Conclusion

Seismic fragility analysis was performed on one 2-story and one 5-story office building using CFS framed walls sheathed by corrugated steel sheathing. The finite element analysis program OpenSees was used and IDA was adopted for the nonlinear time history analysis. The results show that the current seismic performance factors for light wood framed structures seem appropriate for the new shear wall type. The modeling techniques described in this paper is appropriate for future more comprehensive seismic analysis on CFS buildings.

Acknowledgement

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