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Numerical Modelling and Calibration of CFS Framed Shear Walls under Dynamic Loading

I. Shamim¹ and C.A. Rogers²

Abstract

This paper describes the numerical modeling using OpenSees of steel sheathed cold-formed steel framed shear wall test specimens under dynamic loading. Two modeling phases were carried out; the first phase comprised non-linear models calibrated using existing reversed cyclic shear wall test data, and the second phase involved more advanced models calibrated using data from dynamic shake table tests of single- and double-storey shear walls as well as other ancillary test programs. The second phase models incorporated the behaviour of the hold-downs, floor framing and blocked bare frame, in addition to the sheathing. The final calibrated models were able to accurately predict the displacement and force response time histories of the single- and double-storey shear wall specimens. These calibrated models will later be relied on for Incremental Dynamic Analyses (IDA) of representative building structures to evaluate seismic design provisions for cold-formed steel framed shear walls to be used in conjunction with the National Building Code of Canada (NBCC).

Introduction

Seismic design provisions for cold-formed steel (CFS) lateral framing systems, specifically strap braced walls and wood sheathed shear walls, have recently been included in the 2010 National Building Code of Canada (NBCC) (NRCC, 2010). The Building Code refers to the CSA S136 Standard (2007) for design and detailing information, which in turn references the AISI S213 lateral design standard (2007). In terms of available design information for CFS lateral

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framing configurations a missing component is the cold-formed steel sheathed / framed shear wall. The NBCC and the S213 Standard, at present, do not address the design of steel sheathed / CFS framed shear walls for use in Canada. Monotonic and reversed cyclic tests of these shear wall assemblies have been carried out in the US by Serrette et al. (1996, 1997), Yu et al. (2007) and Yu & Chen (2009), as well as in Canada by Ong-Tone & Rogers (2009), Balh & Rogers (2010) and DaBreo & Rogers (2012). A design method to be used in conjunction with the NBCC has been developed (Balh & Rogers, 2010; El-Saloussy & Rogers, 2010), however, to justify the inclusion in the Building Code and AISI S213 of the proposed Canadian seismic design approach derived from the results of static tests it was necessary to investigate the performance of representative shear walls by means of dynamic shake table tests and to develop nonlinear numerical models capable of capturing their response history under dynamic loading. Balh & Rogers (2010) conducted preliminary dynamic numerical analyses of representative CFS framed buildings using the Ruaumoko software (Carr, 2004) which accounted for the pinching behaviour as well as the stiffness degradation in the shear walls' force-displacement hysteretic response; however, these models did not include strength degradation. This paper presents a description of improved nonlinear numerical models developed in OpenSees (Mazzoni et al., 2009) and a brief description of the corresponding dynamic test program. A comparison of the modelling results and test measurements is provided to demonstrate the accuracy of the numerical models.

Dynamic CFS Shear Wall Test Program

Ten full-scale CFS sheathed shear wall specimens were dynamically tested on a shake table (Shamim et al., 2012). Five single-storey (1.22 m × 2.44 m) and five double-storey (1.22 m × 5.18 m) walls (Figure 1) were included in the test matrix. The double-storey platform framed walls consisted of a 2.44 m high first storey wall, on top of which a 0.3 m deep CFS framed floor structure was attached followed by another 2.44 m high second storey wall. CFS frames were assembled of ASTM A653 92.1 mm × 41.3 mm × 12.7 mm C-studs and 92.1 mm × 31.8 mm channel tracks with two or three rows of blocking, equivalent in size and shape to the track members. The steel sheathing panels were 1.22 m × 2.44 m in size with either 0.46 mm or 0.76 mm thickness. Sheathing screw fasteners were installed at 50 mm, 75 mm, 100 mm or 150 mm spacing for the various specimens. The floor structure comprised a rim joist supporting standard floor joist members. The walls at the floor and at their base were connected with hold-down devices and threaded rods. A 12.5 kN gravity load was also applied on top of the test specimen. A comprehensive description of this laboratory study can be found in the paper by Shamim et al. (2012).

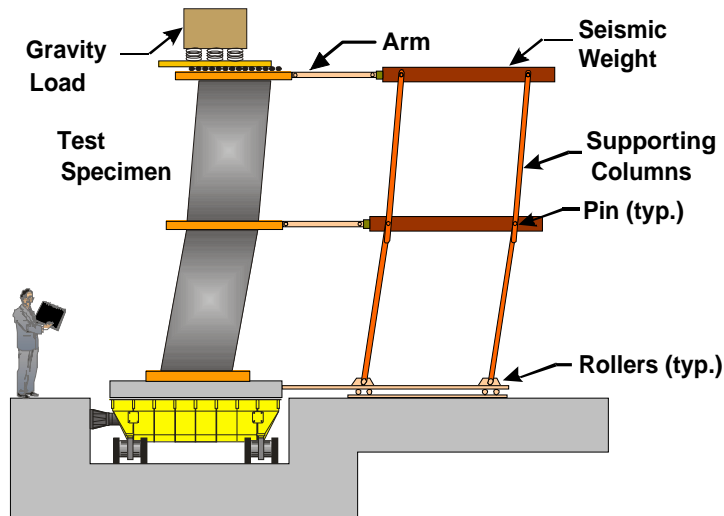


Figure 1: Test set-up of double-storey wall on shake table

Instrumentation was provided to measure lateral and vertical displacements, lateral inertia forces and accelerations: potentiometers were installed in a triangulated pattern such that both vertical and lateral displacements could be measured; accelerometers were placed at the mass locations as well as on the shake table; and load cells were connected to the hold-down anchor rods and to the arms linking the shear wall to the seismic weight. Once installed and instrumented each wall was subjected to a suite of excitations: impact test to measure the linear-viscous damping ratio; harmonic excitation to estimate the natural period of vibration of the wall specimens, and ground motions at elastic and enhanced levels. Two synthetic records provided by Atkinson (2009) were closely matched to the design response spectrum for Quebec and Vancouver (Canada) for site class C to represent strong ground motions in eastern (Quebec) and western (Vancouver) Canada. These records were then scaled to obtain elastic behaviour or inelastic behaviour in the test walls.

Numerical Models

The Open System for Earthquake Engineering Simulation (OpenSees) software (Mazzoni et al., 2009) was utilized for all modeling in this study. OpenSees is an open source framework for simulating the response of structural (and geotechnical) systems subjected to earthquakes. The modeling was conducted in two phases: the initial models (Figure 2a) were developed prior to the shake table tests in order to predict the hysteretic response of the shear wall specimens

and to establish the level of scaling to use for the ground motions. The CFS frame, including the chord studs and the tracks as well as the floor (if applicable), was modelled as rigid truss elements; the shear wall including steel sheathing and screw connections was modeled with truss elements using the Pinching04 material property (Lowes et al., 2004). Pinching04 is a nonlinear material which represents a pinched force-deformation hysteretic response and exhibits un-loading, re-loading, and strength degradation under reversed cyclic loading. Figure 3 shows the parameters required to define the Pinching04 uniaxial material in OpenSees, which includes a backbone trend line, degradation factors, as well as other force and displacement related parameters.

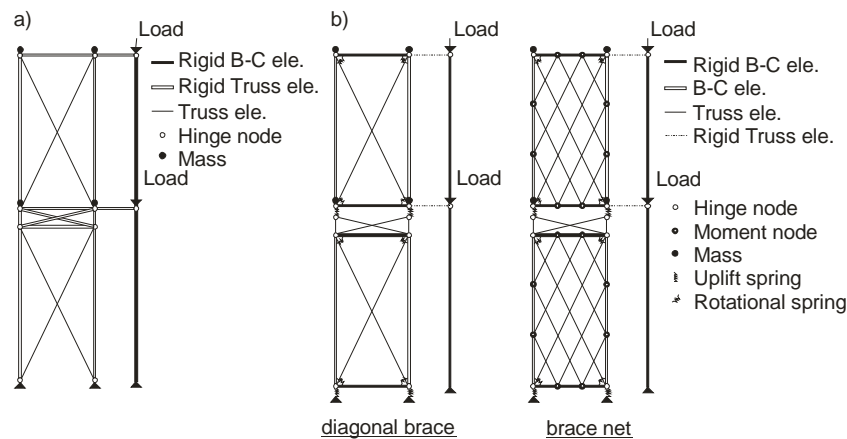


Figure 2: Numerical models in OpenSees: a) initial model (prior to dynamic tests), and b) developed model (after dynamic tests)

The seismic mass represented the seismic weight and supporting columns (Figure 1a) which was lumped at each storey level. A value of 3502 kg was used for the single-storey walls, whereas 3663 kg and 3502 kg were required for the double-storey walls at the first and second storey, respectively. The P-delta apparatus was made of a fictitious frame comprising a rigid beam-column element with Co-rotational Coordinate Transformation capability and a rigid truss element linking the fictitious column to the CFS frame, with the gravity and seismic mass load exerted on the column at each storey level.

More advanced numerical models were developed (Figure 2b) after running the dynamic tests and additional ancillary component testing. The data obtained from these tests were used to calibrate the second phase models. The chord studs were modeled with elastic beam-column elements whose properties were representative of the members used for the tested walls; elastic truss elements

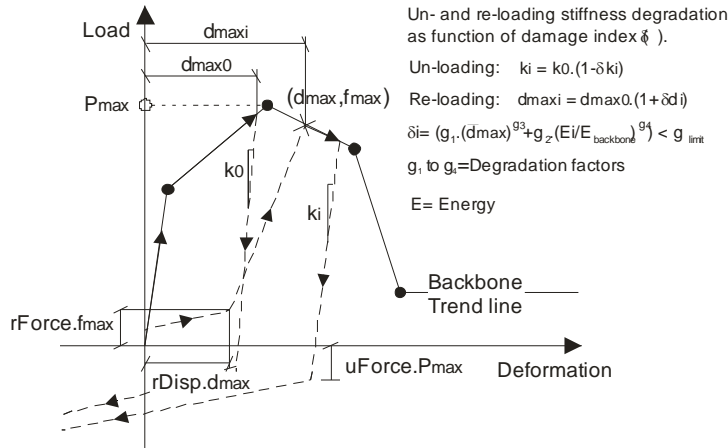


Figure 3: Definition of Pinching04 uniaxial material model

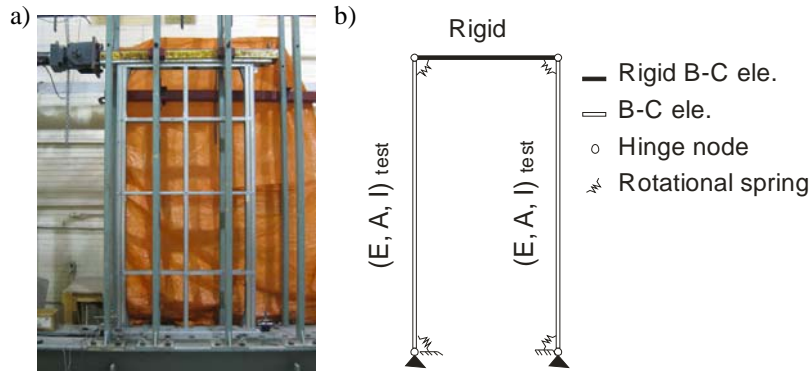


Figure 4: Bare frame test and corresponding model: a) test set up, b) OpenSees model

were used to model the floor having an in-plane lateral stiffness equal to that measured during dynamic testing ranging from 3.5 kN/mm to 4.5 kN/mm; linear uplift springs were installed at the bottom of the floors to represent the hold-down anchor rods in order to capture the wall lateral displacement due to anchor rod elongation. The uplift spring stiffness, obtained from the dynamic tests, ranged from 17.6 to 30.4 kN/mm and from 8 to 17.6 kN/mm for the 1st and 2nd floors, respectively. The stiffness value was first estimated from the shear strength-uplift lateral displacement hysteresis of the tested walls and then determined such that the test shear strength-measured shear displacement and the shear strength-computed shear displacement provided a reasonable match,

assuming the anchor rod behaviour was linear. Rotational springs were placed at the CFS frame joints to represent the lateral stiffness provided by the gravity frame and horizontal blocking.

To determine the stiffness of the rotational springs, four 1.22 m × 2.44 m CFS bare frames made of 92.1 × 41.3 × 12.7 double chord studs, 92.1 × 31.8 tracks and three rows of blocking were assembled and tested under monotonic lateral load (Figure 4a). The CFS bare frames differed in nominal thickness; 1.09 mm, 1.37 mm and 1.73 mm for studs and tracks, and 1.09 mm and 1.37 mm for blocking. The bare frames were then modeled as a simple beam-column frame with a rotational spring at each corner joint (Figure 4b). The rotational spring stiffness was then calibrated so that the test and the model lateral stiffness matched. An average rotational spring stiffness of 114,965 Nm was used in all the advanced models since the stiffness values of the bare frame test specimens were fairly close.

The shear wall was modeled with Pinching04 truss elements. The backbone trend line was drawn for shear strength-shear displacement response hysteresis of each dynamic test and directly used to define the Pinching04 backbone trend line in the models (Figure 6). The Pinching04 material could be assigned two separate backbone trend lines each representing the positive or negative response excursions. Since the hysteretic response of the tested walls was almost symmetric (Shamim et al., 2012), the same trend line was used for both excursions in the model. The degradation factors were first approximated from the force-displacement hysteresis results of the dynamic tests and then systematically changed along with the rDisp, rForce, and uForce factors, which were pre-measured from the test response hysteresis, until a reasonable fit between the test and the numerical model strength as well as displacement response histories was observed.

Comparison of Numerical Models and Test Results

Initial Models

Prior to the dynamic shake table testing the wall specimens' response histories and strength-displacement hystereses were predicted using the simplified numerical models. Since the anchor rods were not included, the models did not account for the rigid rotation of wall caused by anchor rod elongation. In this fashion, the displacement of the wall in the initial models was that resulting from wall shear deformation alone. Diagonal truss elements with Pinching04 material property were used to represent the combined sheathing and screw connection behaviour since the Pinching04 material was deemed to provide an appropriate representation of the steel sheathed shear walls' shear deformation.

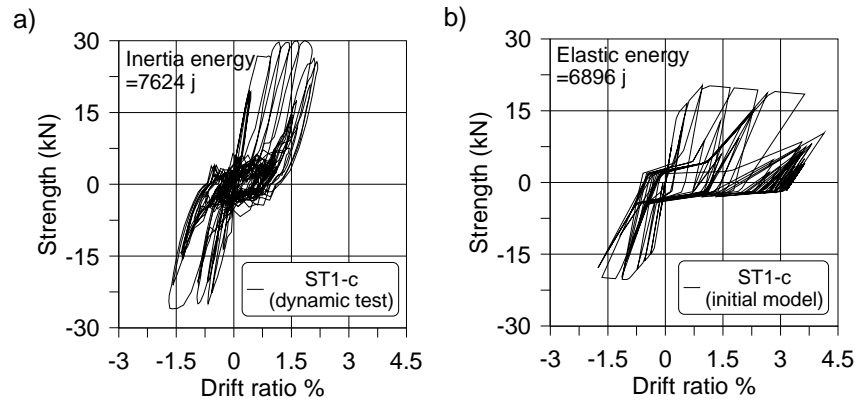


Figure 5: Comparison of strength-drift hysteresis of test ST1-c with the initial numerical model: a) dynamic test, b) initial model

The results of previous reversed cyclic tests performed by Ong-Tone & Rogers (2009) and Balh & Rogers (2010) were used to calibrate the Pinching04 material. Once the dynamic tests had been carried out and the data compared with the model's prediction it was concluded that the initial models were not accurate in capturing the strength and displacement response under dynamic load (Figure 5).

Advanced Models

After conducting the shake table tests, the models were improved in a variety of ways: the rigid truss-made frame was replaced with elastic beam-column elements as the chord stud; the anchor rods were included using linear uplift springs to include uplift displacement due to anchor rod elongation; the rotational springs were installed at the frame joints to provide lateral stiffness produced by the blocking; the rigid floor was replaced with a flexible floor; and the Pinching04 material was calibrated based on the dynamic test data. The Pinching04 material properties were extracted from the shear strength-shear displacement hysteresis of the dynamically tested walls and used to calibrate the sheathing elements. To perform calibration from the test data, first the shear displacement (lateral displacement due to wall shear deformation) and uplift displacement (lateral displacement due to anchor rod elongation) were separated from each other. The shear strength-shear displacement hysteresis was then drawn for the dynamically tested walls, from which the Pinching04 backbone trend line was obtained (Figure 6).

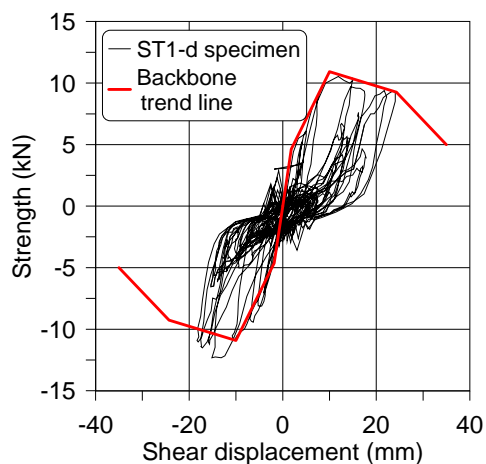


Figure 6: Extracting the backbone trend line from shear strength-shear displacement hysteresis for specimen ST1-d

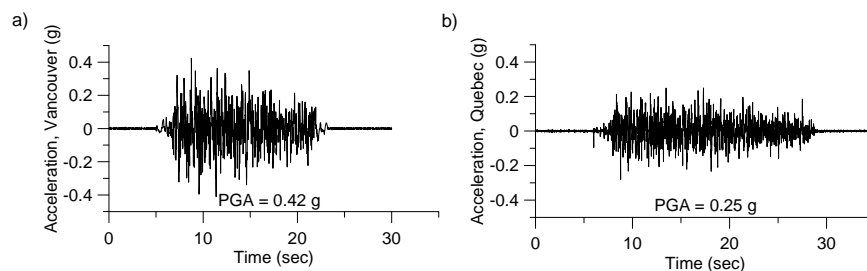


Figure 7: Synthetic ground motion records closely-matched to UHS for a) Vancouver and b) Quebec (site class C)

The pre-measured Pinching04 degradation factors were systematically changed along with the $rDisp$, $rForce$, and $uForce$ factors (Figure 3) as well as the Rayleigh linear viscous damping ratio until an adequate match between the test and model was obtained. The displacement and strength response histories were determined using the advanced numerical models with the ground motion records illustrated in Figure 7 and were compared with the test measurements. A good agreement between the tests and numerical models were observed for both the single and double-storey walls (Figures 8-10). The test and model response time histories were in phase, and the numerical model was able to capture the strength as well as displacement response peak points especially well before the wall failure point. Inclusion of the linear uplift springs and defining the backbone trend line from the dynamic test data significantly improved the

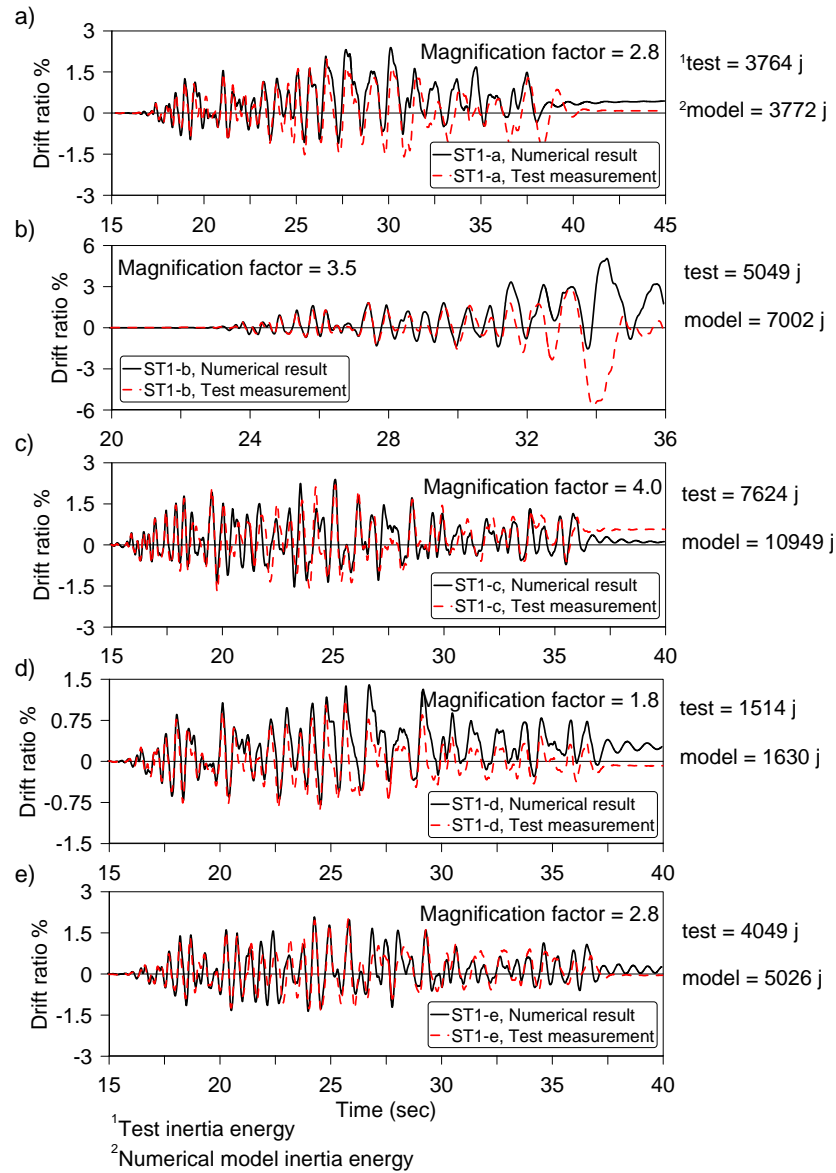


Figure 8: Comparison of the drift time history response of the enhanced dynamic models with test results for specimens; a) ST1-a, b) ST1-b, c) ST1-c, d) ST1-d and e) ST1-e

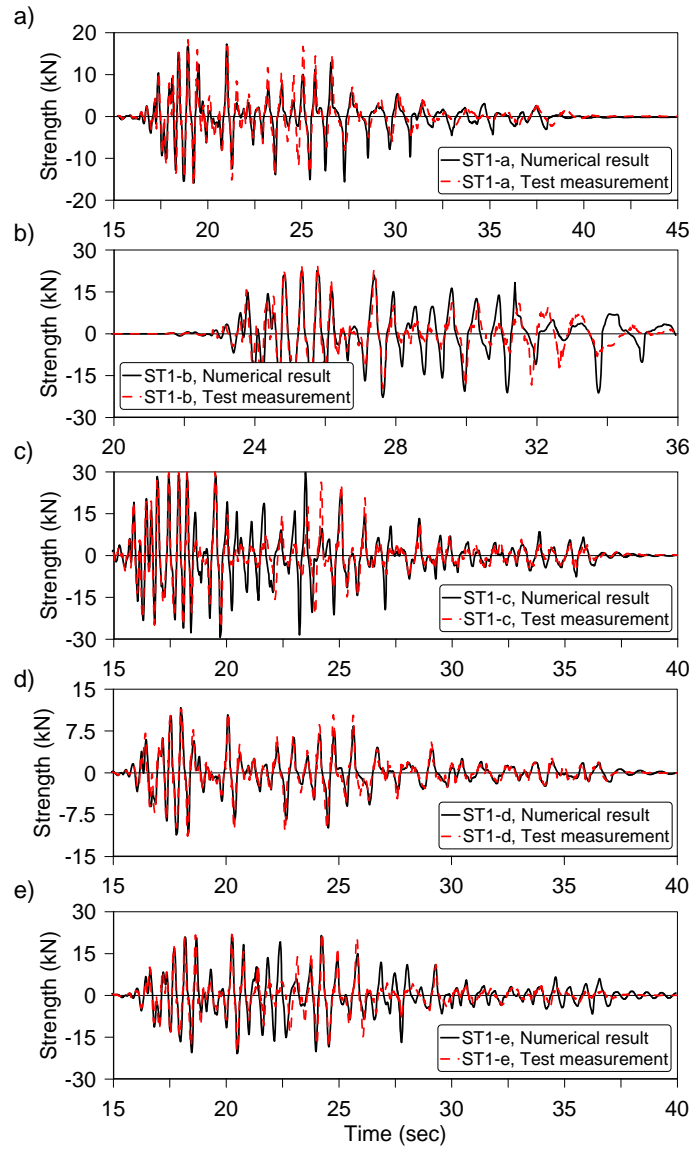


Figure 9: Comparison of shear strength time history response of the enhanced dynamic models with test results for specimens; a) ST1-a, b) ST1-b, c) ST1-c, d) ST1-d and e) ST1-e

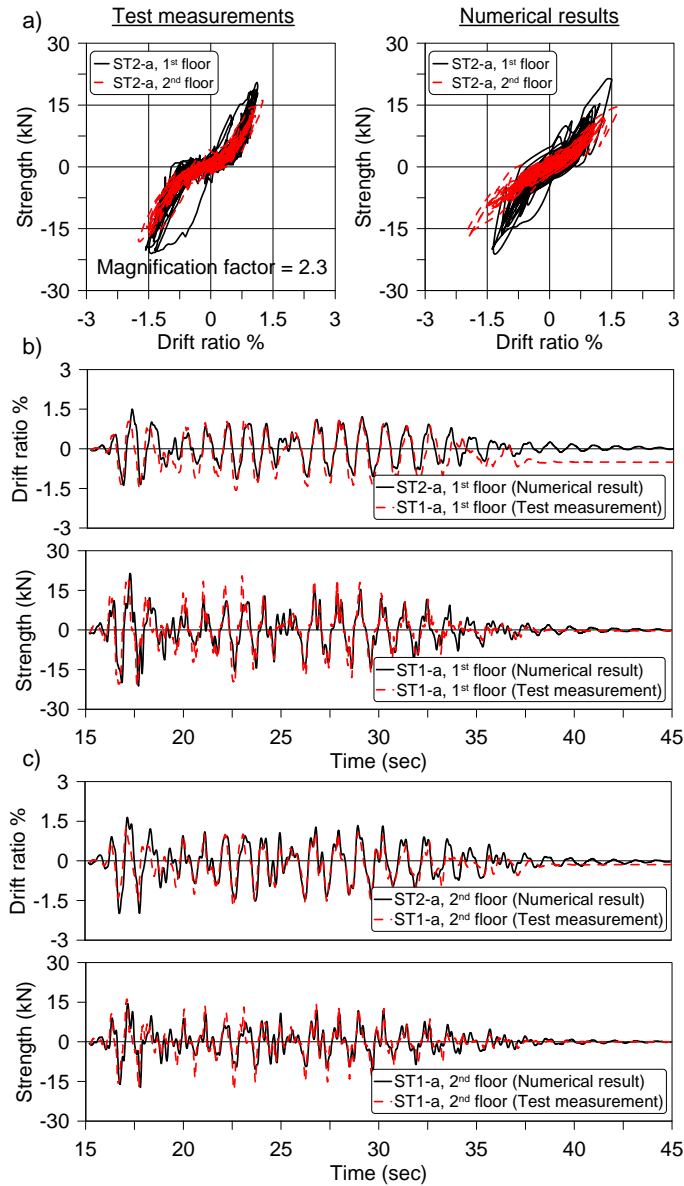


Figure 10: Comparison of the results from testing and the advanced numerical model for specimen ST2-a; a) strength-drift hysteresis of the 1st and the 2nd floor, b) time history of the 1st floor, and c) time history of the 2nd floor

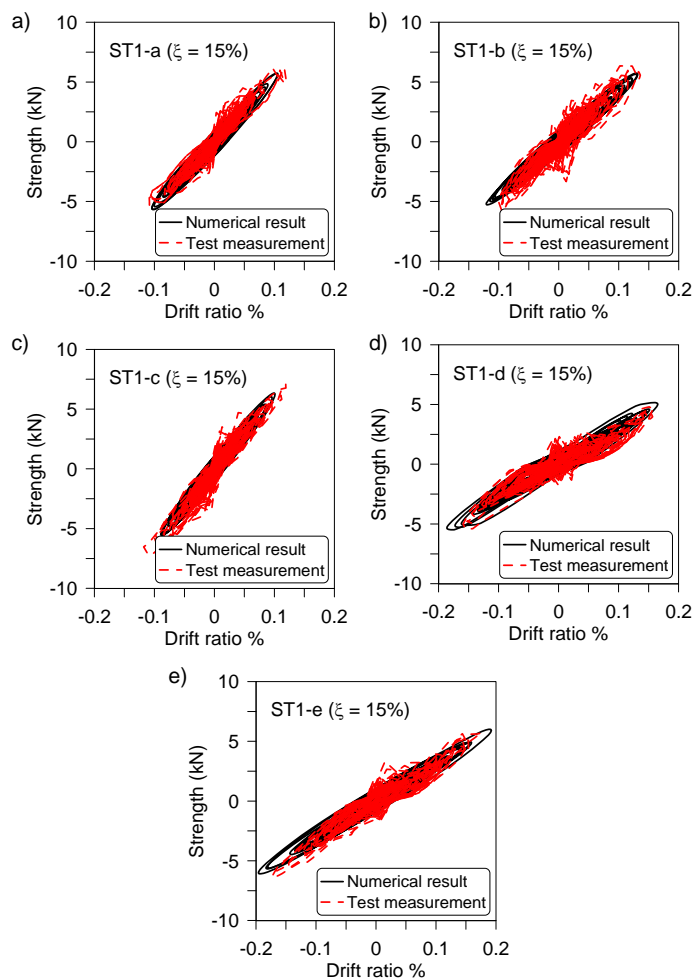


Figure 11: Strength-drift hysteresis of elastic dynamic test and corresponding numerical model for specimens; a) ST1-a, b) ST1-b, c) ST1-c, d) ST1-d and e) ST1-e

capability of the numerical model to capture the dynamic response of the tested shear walls. The ratio of the test to the model inertia energy was obtained equal to 1.18 on average, ranging from 0.81 to 1.47.

In addition, the wall test specimens were modelled at the elastic level of ground motion to match the shake table loading protocol. A Rayleigh damping ratio of

the tested walls at the elastic level was determined by matching the elastic dynamic tests with the numerical model results. The Rayleigh damping ratios obtained from the calibrated numerical models of the elastic dynamic tests ($\xi_{\text{average}}=14\%$) were higher compared with those obtained from numerical modelling of the dynamic tests ($\xi_{\text{average}}=5\%$), as was expected. The average Rayleigh damping ratio measured from the shear wall tests was 7.6%, excluding the second mode damping ratio (double-storey walls) in the averaging. The strength-drift hysteresis of the single-storey tested walls is shown in Figure 11. The hysteretic shape illustrates that a damping force is involved in the strength of the tested wall specimens even at elastic levels of loading.

Conclusions

Numerical models of steel sheathed / CFS framed shear walls were developed and calibrated using measurements of displacement, force and acceleration of dynamic shear wall tests and other ancillary component tests. The advanced numerical OpenSees models were able to adequately simulate the steel sheathed shear wall behaviour under dynamic load. The models were found to be accurate in capturing the response strength and displacement time histories as well as strength-displacement hysteresis of the single- and double-storey tested walls. Based on the accuracy of the advanced numerical models they are considered adequate to be used in the incremental dynamic analyses of representative buildings implementing FEMA P695 methodology (2009), the results of which will be used to evaluate the adequacy of the steel sheathed shear wall design methods developed in previous studies.

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