Seismic Performance of Residential Buildings with Staggered Walls

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SUMMARY:
In this paper seismic performance of reinforced concrete staggered wall structures with middle corridor was evaluated. To this end, 6, 12, and 18-story structural models were designed and were analyzed to investigate the seismic load-resisting capacity. The response modification factors were computed link beams increased up to 100%, the overall overstrength increased by only about 25%. When the re-bar ratio of the link beams was increased by 50%, the overstrength increased by about 40%. The replacement of the RC link beams with steel box beams resulted in superior per based on the overstrength and the ductility capacities obtained from pushover curves. The effect of a few retrofit schemes on the enhancement of strength and ductility was also investigated. The pushover analysis results showed that the response modification factors ranged between about 4.0 and 6.0 with the average value around 5.0. When the bending rigidity of the formance of the structures with reduced beam depth. The displacement time histories of the model structures subjected to the earthquake ground motions scaled to the design seismic load showed that the maximum inter-story drifts were well below the limit state specified in the design code. Based on the analysis results it was concluded that the staggered wall systems with a middle corridor had enough capacity to resist the design seismic load.

Keywords: Staggered Wall Systems, Seismic Performance, Response Modification Factors

1. INTRODUCTION

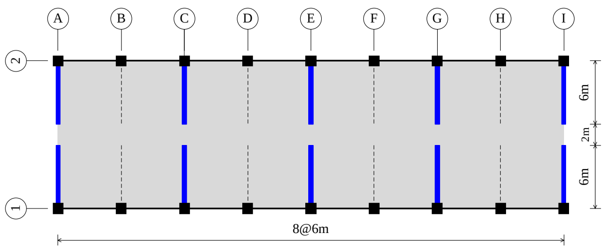
The staggered-wall system (SWS) structure is composed of a series of storey-high reinforced concrete (RC) walls spanning the total width between two rows of exterior columns and arranged in a staggered pattern on adjacent column lines. The staggered systems were originally developed in MIT as steel staggered-truss systems (Scalzi, 1971). The system is known to have advantages such as low floor-to-floor heights, large column-free spaces, increased design flexibility, fast erection time, and reduced weight of the superstructure and therefore reduced foundation cost. For seismic performance evaluation of staggered truss systems, Kim et al. (2007) carried out nonlinear analysis of staggered truss systems and found that plastic hinges formed at horizontal and vertical chords of vierendeel panels, which subsequently leaded to brittle collapse of the structure. Recently Kim and Jun (2011) investigated the seismic performance of an apartment building with partially staggered shear walls.

In this study the seismic performance of reinforced concrete staggered wall system (SWS) structures with a middle corridor was evaluated. To this end, 6, 12 and 18-story SWS structural models were designed and were analyzed by nonlinear static analyses to obtain their force-displacement relationship up to failure. The response modification factors were computed based on the overstrength and the ductility capacities obtained from the capacity envelopes. Finally the seismic responses of the model structures subjected to three earthquake ground motions was evaluated by nonlinear dynamic analysis.

2. DESIGN OF ANALYSIS MODEL STRUCTURES

2.1. Configuration of the analysis model structures

In the staggered wall systems the story-high RC walls that span the width of the building are located along the short direction in a staggered pattern. The floor system spans from the top of one staggered wall to the bottom of the adjacent wall serving as a diaphragm. The staggered walls with attached slabs resist the gravity as well as the lateral loads as H-shaped deep beams. The horizontal load is transferred to the staggered walls below through diaphragm action of floor slabs. In this study the staggered walls were designed as story-high deep beams. With RC walls located at alternate floors, flexibility in spatial planning can be achieved compared with conventional wall-type structures with vertically continuous shear walls. Fig. 1 shows the flow of horizontal shear force from the staggered walls above to the columns and staggered walls below through floor diaphragm. Fig. 1 shows the structural plan and 3-D configuration of the model structures. Columns and beams are located along the longitudinal perimeter of the structures providing a full width of column-free area within the structure. Along the longitudinal direction, the column-beam combination resists lateral load as a moment resisting frame.



(a) Plan shape

(b) 3-D view

Figure 1. Shape of the staggered wall system model structures

2.2 Structural design of analysis model structures

To evaluate the seismic performance of staggered wall system structures, 6, 12, and 18-story structural models were designed and were named as SWC06, SWC12, and SWC18, respectively. The model structures were designed per the ACI 318-06 (ACI 2005) using the seismic loads specified in the IBC 2009 (ICC 2009). For gravity loads, the dead and live loads of 7kN/m^2 and 2kN/m^2 were used, respectively. The design seismic load was computed based on the design spectral response acceleration parameters S_{DS}=0.31g and S_{D1}=0.13g. This corresponds to the design seismic load in LA area with site class B, which is a rock site. As the response modification factor for a staggered wall system is not specified in the current design codes, the response modification factor of 3.0 was used in the structural design of the staggered wall systems, which is generally used for the structures to be designed without consideration of seismic detailing. Along the longitudinal direction the structures were designed as ordinary moment resisting frames with *R*-factor of 3.0. The ultimate strength of concrete is 27 MPa and the tensile strength of re-bars is 400 MPa. The thickness of the staggered walls is 20 cm throughout the stories, and the connecting beams have the size of 200×600mm. The thickness of the floor slabs is 21 cm which is the minimum thickness required for wall-type apartment buildings in Korea to prevent transmission of excessive noise and vibration through the floors.

2.3 Modeling for analysis

The displacement-controlled pushover analyses were conducted using the nonlinear analysis/design program code MIDAS (MIDASIT, 2009) to obtain the nonlinear load-displacement relationships of the model structures. The lateral loading profiles for the pushover analysis were determined proportional to the fundamental mode shapes of the model structures. The staggered walls were modeled by the top and bottom rigid beam elements and the vertical line element composed of the nonlinear axial, flexural, and shear springs as shown in Fig. 2. The middle line element behaves like a 3D beam-column element, and the top and bottom rigid beams act as rigid bodies in the x-z plane. The moments about the z-axis represent the in-plane bending behavior, and the out of plane bending behavior was not considered in the wall element.

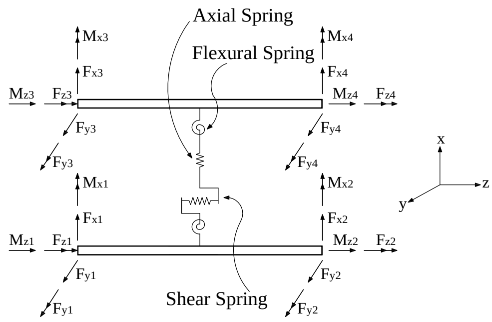
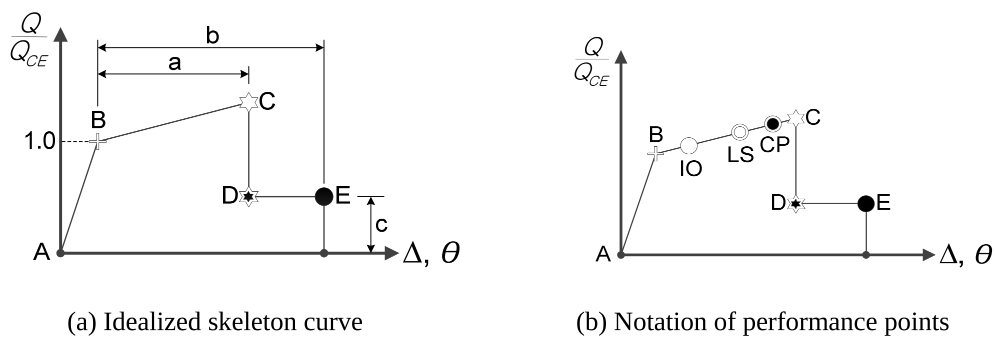


Figure 2. Modeling of staggered walls for pushover analysis

The nonlinear force-deformation relationship of the structural members recommended in the FEMA-356 (2000), which is shown in Fig. 3(a), was used in the pushover analysis. In the idealized skeleton curve, linear response is depicted between point *A* and an effective yield point *B*. The slope from *B* to *C* represents strain hardening. In this study the post-yield stiffness was set to be 2% of the initial stiffness. *C* has an ordinate that represents the maximum strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line *CD*). Beyond point *D*, the component responds with substantially reduced strength to point *E*. At deformations greater than point *E*, the component strength is essentially zero. The parameters *a*, *b*, *c*, *d*, and *e* that are required for modeling structural components can be obtained from the Table 6-7 and Table 6-8 of the FEMA-356. The performance points, such as Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP), and Collapse (C), which are indicated in Fig. 3(b), are also defined in the FEMA-356 report.



(a) Idealized skeleton curve (b) Notation of performance points

Figure 3. Generalized force-deformation relationship of a RC structural member defined in FEMA 356

3. MAXIMUM STRENGTH AND INTER-STORY DRIFT OF THE MODEL STRUCTURES

To evaluate the behavior factors of the model structures subjected to seismic load, pushover analyses were carried out along the transverse direction by applying incremental lateral load with its vertical profile proportional to the fundamental mode of vibration. The base shear *vs*. roof displacement relationship for each model structure is depicted in Fig. 4. Such information as the design base shear, the first yield points of the link beams and columns, the sudden strength drop, and the points where the inter-story drift reached 1.5% of the story height are also provided on the curves. It can be observed that the 18-story structure showed highest strength and lowest stiffness, and that the 6-story structure showed lowest strength but highest stiffness. In all structural models the maximum strengths were higher than twice the design base shears. The major strength drop occurred before the maximum inter-story drift reached 1.5% of the story height which is generally considered as the limit state for the Life Safety performance level. It was observed that the sudden drop of strength occurred due mainly to the formation of plastic hinges in the lower story link beams.

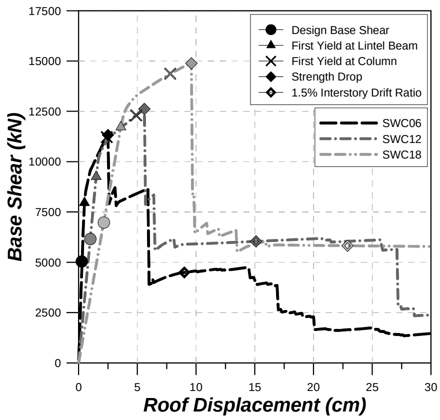
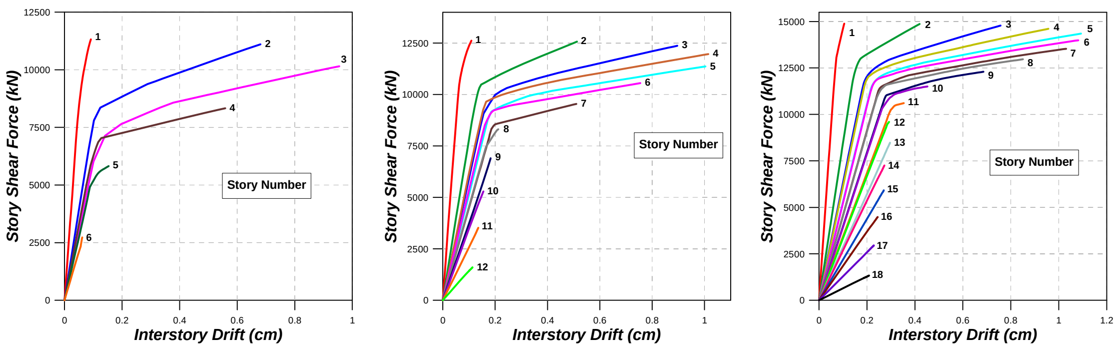


Figure 4. Nonlinear static pushover analysis

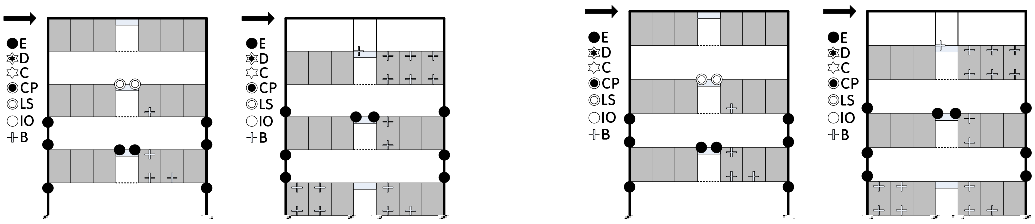
Fig. 5 shows the story shear-inter story drift curves of the model structures. It can be observed that in all model structures the story stiffness and strength are generally higher in lower stories. The ductility demands are higher in lower stories. Compared with the story shear *vs*. inter-story drift relationship curves of higher stories, the curves of lower stories generally show distinct yield points.



(a) 6 story (b) 12 story (c) 18 story

Figure 5. Relationship of story shear *vs*. inter-story drift

Fig. 6 depict the plastic hinge formation of the 6-story model structure when the strength dropped suddenly and when the maximum inter-story drift reaches 1.5% of the story height. It was observed that the link beams in the lower stories yielded first followed by yielding of the lower story columns. Only minor deformation occurred in the lower story staggered walls, and the walls located in the higher stories generally remained elastic.

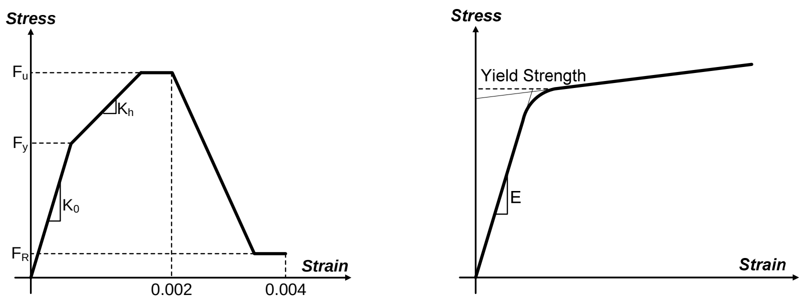


(a) Strength drop (b) 1.5% inter-story drift

Figure 6. Plastic hinge formation at the points of sudden strength drop and at the 1.5% inter-story drift

4. EVALUATION OF THE DYNAMIC RESPONSES

The seismic performance of the staggered wall system structures was evaluated by nonlinear dynamic analyses using the program code Perform 3D (Computers and Structures 2006) in which various types of nonlinear models including user defined models were implemented for simulation of inelastic behavior of structures. The staggered walls were divided into many fiber elements and the deformation such as fiber strains, hinge rotations, and shear deformations was monitored using the axial and rotational gage elements.



(a) Reinforced concrete (b) Reinforcing steel

Figure 7. Nonlinear stress-strain relationship of structural materials

The stress-strain relationship of concrete in the compression part was defined using the trilinear model proposed by Paulay and Priestley (1992), and the tensile strength was neglected (Fig. 7). As seismic detailing was not applied in the model structures, the confinement effect of re-bars was not considered in the modeling of concrete. The maximum compressive strength (F_u) was assumed to be 27MPa, and the yield and the residual strengths were assumed to be 60% and 20% of the maximum strength, respectively. The strain at the maximum strength and the ultimate strain were 0.002 and 0.004, respectively. As a wall element has no in-plane rotational stiffness at its nodes, a beam element was imbedded in the wall to model the moment connection between staggered walls and the connecting link beams. The behavior of re-bars was modeled by bilinear lines with the post-yield stiffness ratio of 0.02, and the behavior of the beams and the columns were modeled by the FEMA-356 type nonlinear models.

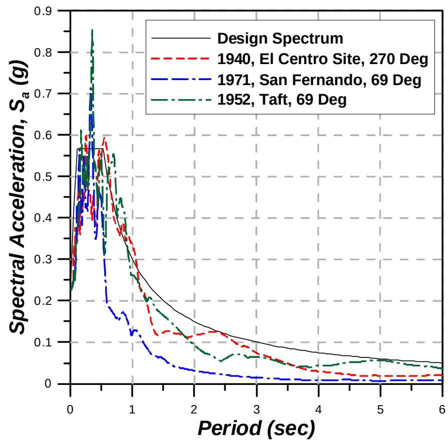
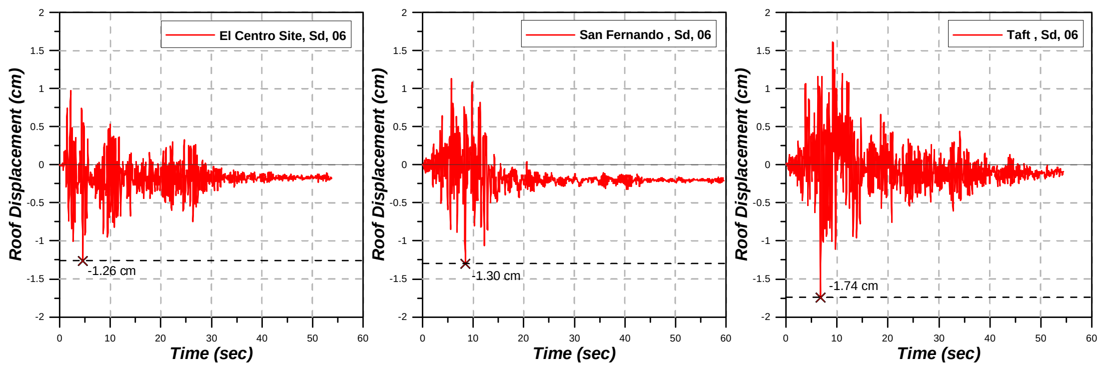
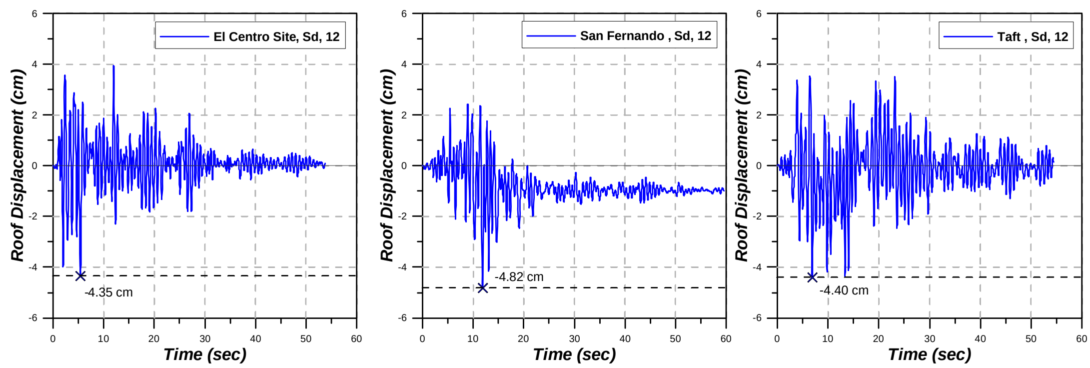


Figure 8. Response spectra of the scaled ground motions

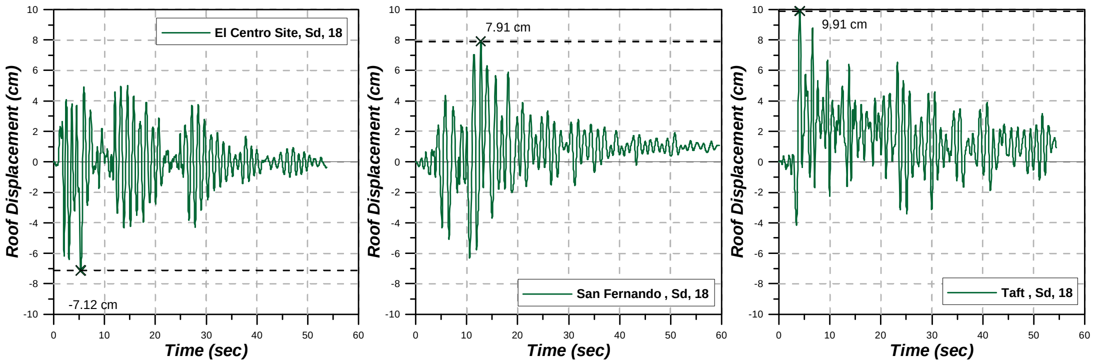
Dynamic time history analyses of the model structures were carried out to obtain the responses for the El Centro (NS), San Fernando (NS), and Taft (NS) earthquake ground motions. Fig. 8 shows the response spectra of the earthquake records and the IBC 2009-based design spectrum with the spectral response acceleration parameters S_{DS}=0.57g and S_{D1}=0.3g**.** The matching ground accelerations were scaled to have the effective peak accelerations (EPA) of 0.227g. This corresponds to the design seismic load in LA area with site class D. The displacement time histories of the model structure subjected to the three earthquake ground motions are presented Fig. 9. It can be noticed that at the end of the analysis slight permanent displacement occurred, which implies that the structure experienced plastic deformation. However the maximum inter-story drifts, shown in Fig. 10, turned out to be far less than the limit state of 0.015 specified in the design code. It was observed that a few plastic hinges formed at the lower story link beams, which is similar to the results obtained from the pushdown analysis. It was also observed that the magnitude of plastic rotation did not reach the Life Safety performance objective specified in the FEMA 356.



(a) Six-story structure

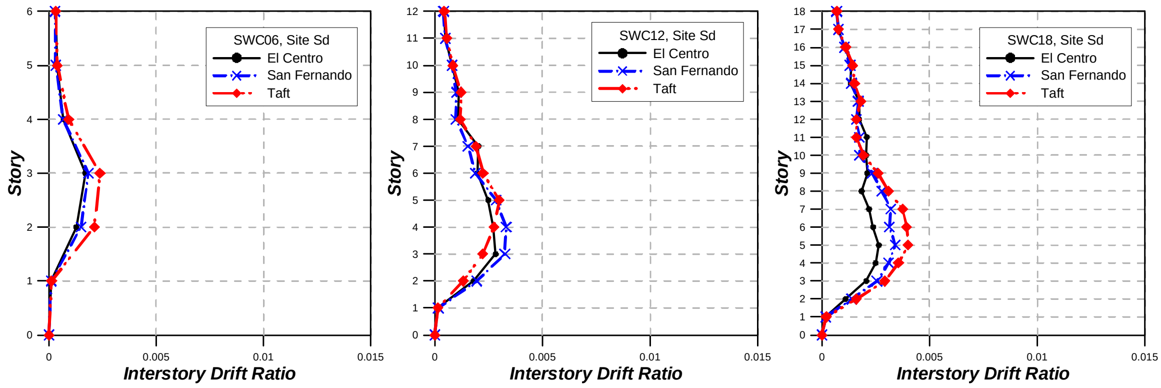


(b) Twelve-story structure



(c) Eighteen-story structure

Figure 9. Time-history of the roof story displacement



(a) Six-story (b) Twelve-story (c) Eighteen-story

Figure 10. Maximum inter-story displacement of the model structures

7. CONCLUSIONS

In this paper seismic performance of reinforced concrete staggered wall system structures with middle corridor was evaluated. To this end, 6, 12, and 18-story structural models were designed and were analyzed by pushover analysis to investigate the force-displacement relationship. Nonlinear dynamic time history analyses of the model structures were carried out to obtain the responses for the El Centro (NS), San Fernando (NS), and Taft (NS) earthquake ground motions. The analysis results showed that plastic hinges formed first at the link beams located between two staggered walls and the structures failed by formation of weak stories. The dynamic time history analysis results showed that for the three earthquake ground motions the maximum inter-story drifts turned out to be far less than the limit state of 0.015 specified in the design code. Based on these observations it was concluded that the staggered wall systems with a middle corridor had enough capacity to resist the design seismic load.

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