SEISMIC ASSESSMENT OF CONTROLLED ROCKING STEEL BRACED FRAME

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ABSTRACT

Enhancement of seismic resilience of structuresunder severe earthquake is attainable with various system such as controlled rocking self centering concentric braced frame, SC-WCBF-CR. This paper investigates the rocking effect in the SC-WCBF-CR system in comparition with conventional steel braced frame,WCBF_FBD time-history nonlinear dynamic analysis of 9-story SC-WCBF-CR and WCBF_FB seismic systems is conducted using two horizontal components ofrecords of far-fault scaled under DBE and MCE hazard level. The results of time history analyses of these systems are compared to each other. The results of seismic analysis show enhancement in performance of SC-WCBF-CR system in significant decrease in permanent drift and nonlinear deformations, and creation of damge concentration in fuse element in comparison with WCBF_FB system.

INTRODUCTION

Eatherton and Hajjar (2010) developed Controlled Self-centering concentric braced frame with PT cables and shear panels fuses system and conducted at Illinois University in Urbana-Champaign quasi- cyclic static test half-scale frame. The controlled rocking system is designed to rock upon its foundation during an earthquake, vertical post-tensioning strands that anchor the top of the frame down to the foundation, which brings the frame back to center and provide overturning resistance. Hall et al (2010)studied numerically the effective parameters on the behavior of the controlled rocking steel frame system. Ma, (2011) at Stanford University analyzed and examined shear steel yielding fuse dampers of the SC-WSCB-CR system. Ma, et al (2010)performed experiments shaking table test, 0.68 scale single self-centering frame at Japan's DefenseCenter. Etheron et al (2014) developed limites state design concepts of the system. The self-centering controlled rocking system consists of a steel braced frame, post-tensioned cable, and replaceable structural fuses to dissipate earthquake energy.

The mechanics of the system response are shown in Figure 1 .The flag shape response is characteristic of a self-centering system which is intuitive in that the displacement returns to near zero as the force is removed. The response of the combined system is defined by uplift of the frames, yield of the fuse, Arbitrary point of load reversal, fuse is at zero force and begins to load in the opposite direction , fuse yields in the opposite direction, frames set back down.

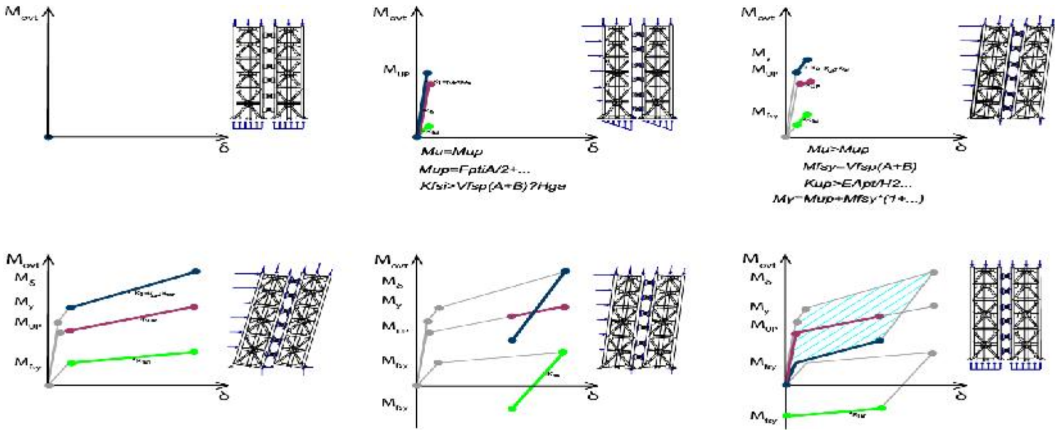

Figure 1.Response of Self centering frames

Following the introduction and reviewing the literature related to the system and its performance, archetype set are introduced. Upon describing the 2D finite-element simulation method used for archetype's details, the nonlinear dynamic response history analysis is performed on the models.Finaly, The numerically results of some engineering response parameters are evaluated from various aspects.

ARCHETYPE

The Prototype Building is based on a similar prototype building with the steel framing configuration used in the SAC Joint Venture Project and referenced in Gupta and Krawinkler (1999). It is a four bays × six bay building with 9 meters width of each bay. Figure 2 shows plan and elevation of the office building, assumed to be located near Los Angeles, California. Seismic resistance is provided by the SC-WCBF-CR system for the controlled rocking and WCBF_FB for the fixed base archetypes, while the rest of the structure carries the gravity load only. Double braced frame with space B of each other and width A is located in the seismic frame span (W = 2A + B). The ratio of spans (A/B) is considered to 2.5. Dead and live loads and seismic mass of floors is 9459 kN, 1974V kN and 1033 kN.sec2 / m respectively.

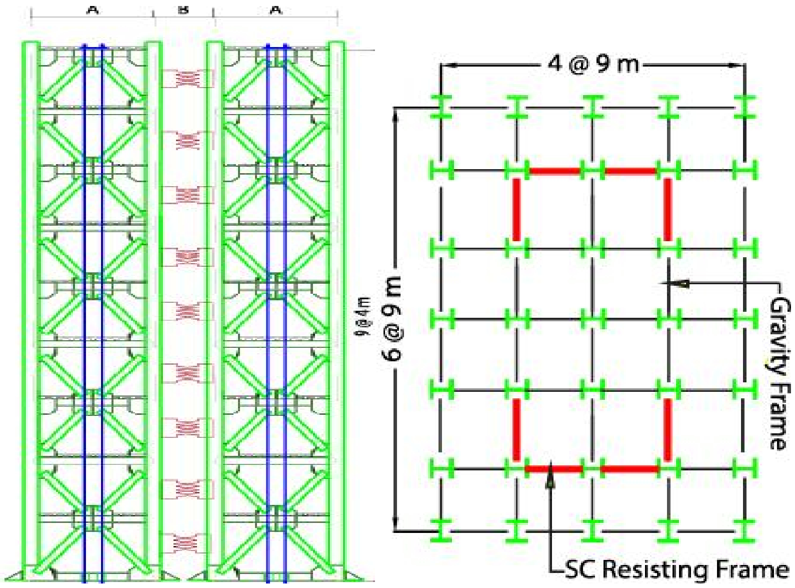

Figure 2.9-story archetype

NONLINEAR SIMULATION OF ARCHETYPE

The two-dimensional finite element simulation was created using the OpenSees softwareto perform nonlinear analysis and assese performance of archetypes model. A 2D computational fixed based is similarto rocking frame differs from in boundary conditions and fuse and PT components.To simulatie nonlinear archetypes, primary seismic elemenst and gravity frames (leaning column), constraints, the seismic mass and loads are mode led, and for rocking mode led added cables and fuses, respectively. Beams and coloumns of the seismic braced frames are constructed with nonlinear materials and fiberdispBeamColumn elements. Two leaning columns on each side of the seismic frame that link with rigid truss elements are modeled with elasticBeamColumn element and zero-stiffness spring to simulate the interactions and P-Δ effect due to gravity frame. Geometric nonlinearity effect due to large displacements and unusual boundary conditions were cause for concern by geomTransfPdelta command. The Concentric braces and gusset plate to provide accurate simulation of global and local (buckling-yielding) behaviors of are achieved by using several nonlinear Force-based BeamColumn elements along the length of the braces. To capture global in-plane braces buckling, braces are modeled in parabolic shape using 10 equal length fiberelement with initial geometrical imperfection equal to 0.1% of the length of a brace. To initiate out-of-plane buckling behavior, FB fiber elements are used at the gusset plates to capture the rotational restraints and out-of-plane buckling when the brace buckles due to its in-plane initial geometrical imperfection and bending. To allow rocking and to prevent sliding The vertical and horizontal restraint points are modeled with zero-length gap element using elastic-perfectly plastic material (EPP) with a no-tension elastic constitutive relationship. In SC-WCBF-CR archetype, The post-tensioning is modeled using corotational truss elements and the fuse was created with fiber section elements to simulate the flexural, axial, and lateral- torsional buckling behavior of the fuse links.

FAR-FAULT GROUND MOTIONSSET

A subset of a two- horizontal component of 22 far-field record presented in the project Fema -P695 was selected to to perform dynamic analysis and evaluation designed archetypes. Some of records specification is given in table 1. Two target spectra was considered for the DBE and MCE hazard levels. Normalizing and Scaling of the recordes wasconducted to DBE and MCE target hazard spectra (10% probability of exceedance in 50 years and 2% in 50 years) using the method recommended in FEMA P695(2009). Recordes were firstnormalized with factors caluculated based on the geometric mean of the two horizontal components of each ground motion. Then, the median spectral acceleration was found by fitting alognormal distribution to the normalized spectral accelerations. The scales factor were calculated based on the ratio of the design spectralacceleration to this median spectral acceleration associated with the fundemental period of the structure and the entire set of ground motions was scaledusing the resulting scale factor. Figure 3 shows the spectral acceleration response and also normalized scale median response of the target seismic hazard spectrum.

Table 1. subset of far feildrecordes

R	Normaliz. Factor	PGA	Fault type	Station	Name	Year	Mw	#
13.30	0.65	0.52	Blindthrust	Beverly Hills Mulhol	Northridge	1994	6.7	1
26.5	0.83	0.48		CanyonW Lost Cany	Northridge	1994	6.7	2
41.3	0.63	0.82	Strike-slip	Bolu	Duzce, Turkey	1999	7.1	3
26.5	1.09	0.34		Hector	Hector Mine	1999	7.1	4
33.7	1.31	0.35		Delta	Imperial Valley	1979	6.5	5
29.4	1.01	0.38		El Centro Array#11	Imperial Valley	1979	6.5	6
8.7	1.03	0.51		Nishi-Akashi	Kobe, Japan	1995	6.9	7
46	1.10	0.24		Shin-Osaka	Kobe, Japan	1995	6.9	8
98.2	0.69	0.36		Duzce	Kocaeli, Turkey	1999	7.5	9
53.7	1.36	0.22		Arcelik	Kocaeli, Turkey	1999	7.5	10
86	0.99	0.24		Yermo Fire Station	Landers	1992	7.3	11
82.1	1.15	0.42		Coolwater	Landers	1992	7.3	12
9.8	1.09	0.53		Capitola	Loma Prieta	1989	6.9	13
31.4	0.88	0.56		Gilroy Array #3	Loma Prieta	1989	6.9	14
40.4	0.79	0.51		Abbar	Manjil, Iran	1990	7.4	15
35.8	0.87	0.36		El Centro Imp. Cent	Superstition Hills	1987	6.5	16
11.2	1.17	0.45		Poe Road (temp)	Superstition Hills	1987	6.5	17
22.7	0.82	0.55	Thrust	Rio Dell Overpass FF	Cape Mendocino	1992	7.0	18
32	0.41	0.44		CHY101	Chi-Chi, Taiwan	1999	7.6	19
77.5	0.96	0.51		TCU045	Chi-Chi, Taiwan	1999	7.6	20
39.5	2.10	0.21		LA - Hollywood Stor	San Fernando	1971	6.6	21
20.2	1.44	0.35		Tolmezzo	Friuli, Italy	1976	6.5	22

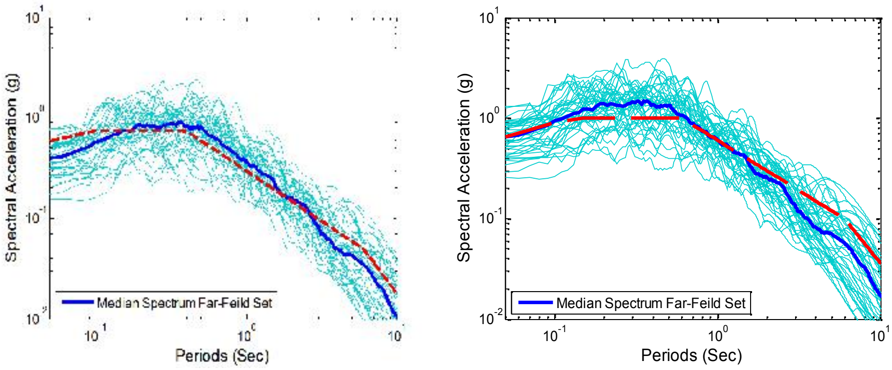

Figure 3.a) 9-storey building (DBE), b) 9-storey building (MCE)

Dynamic analysisresults

Figure 4 shows the comparition of RDR (horizontal displacement to height of roof) of 9-stories SC-WCBF-CR (CR9) to WCBF_FB (FB9) systems subjected to second component of scaled Northridge record to MCE hazard level spectrum.The CR9 and FB9 archetypes reached RDRmax2.5% and 3.4%, that are occurred at 9.02 and 9.24 second, respectively. The residual RDRmax of CR9 and FB9 archetypes are caculated0.7% and 0.006%, respectively. These results show that CR archetypeis experienced more maximum roof drift ratio, RDRmax, than FB.

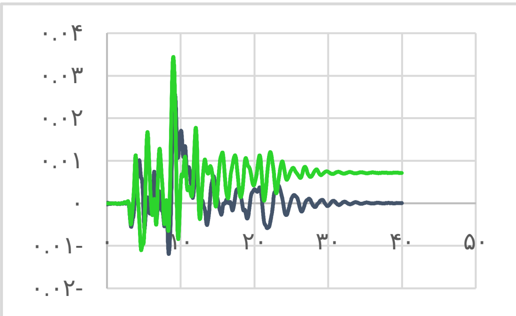

Figure 4.response history comparison of the of the RDR archetype of CR9 and FB9

As Figure 5 shows, hysterticoverturning moment to roof drift ratio response, Mu- RDR, of CR archetype also demonstrated the ability of self centering and reducing permanent drift in CR archetypes at the end of earthquake. Idealize predicted cycling curve is also ploted with hysteric response of CR9 in Northridge earthquake.

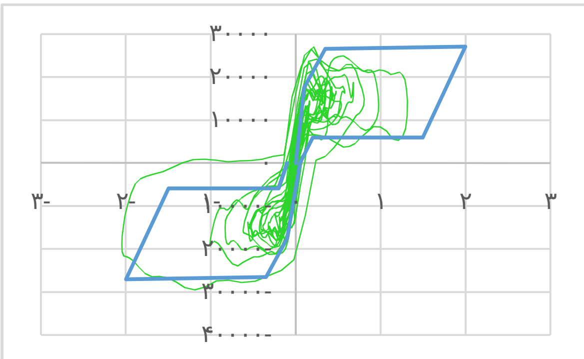

Figure 5.hystertic overturning moment to roof drift ratio response CR9archetype

Median of maximum response (MEDPmax)values of roof drift ratio, RDR, Interstory drift ratio, IDR and residual roof displacement, RRD, of CR9 and FB9 archetypesresulted from time history anlysissubjected to 44 records scaled to the DBE and MCE level are summarized in Table 2. The MEDPmaxof RDR and IDR values of CR9 archetype are estimated higher than FB9 incomparision.This shows that the uniformIDR distributed along the height of the CR9 archetype. On the other hands, RRD values in CR9are decreased respect to FB9. This shows that the thecosts of structural damagewould be reduce under the severe earthquakes.

Table 2. mean of maximum response of Engineering demand parameters

	EQ levle	base	MEDPmax 9st
IDR, %	DBE	CR	1.36
		FB	1.08
		K	1.26
	MCE	CR	2.17
		FB	1.53
		K	1.42
RDR , %	DBE	CR	1.31
		FB	1.06
		K	1.24
	MCE	CR	2.09
		FB	1.43
		K	1.46
RRD , mm	DBE	CR	2.43
		FB	8.38
		K	0.29
	MCE	CR	5.22
		FB	46.05
		K	0.11

CONCLUSIONS

This research investigates numerically nonlinear seismic response of 9-storey archetype of self-centering controlled rocking, and its efficiencies are compared to the similar steel concentrically braced frames. This system consists of a conventional frame with the ability to rock off their bases. Two-dimensional nonlinear dynamic time-history analysis of both types systems are done by applying 22 pair of far-field motions scaled to the DBE and MCE. The compared results of the analysis demonstrate the significant enhancement of performance in rocking system by decreasing the residual deformations with comparison to conventional system.

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