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PERFORMANCE-BASED DESIGN FOR SEISMIC STRENGTHENING OF RC FRAMES USING STEEL CAGING AND ALUMINUM SHEAR YIELDING DAMPERS

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ABSTRACT :

Gravity-load designed RC buildings with open ground-story are generally considered as seismic-deficient in terms of lateral strength, stiffness, drift capacity, and energy dissipation potential. This paper evaluates effectiveness of two strengthening techniques involving external steel cage for strengthening of ground-story columns and aluminum shear panels for dissipation of seismic energy. A performance-based method is developed to design various elements of the proposed strengthening schemes, based on balancing the input energy due to earthquakes to the energy dissipated through plastic hinges and aluminum panels for a target yield mechanism. Nonlinear static and time-history analyses of strengthened frames verified excellent seismic performance of strengthened frame in terms of enhanced lateral strength, stiffness, and energy dissipation and reduced damages in RC frame members under selected ground motions.

KEYWORDS: Seismic strengthening, RC frame, Performance-based design, Steel caging, Aluminum Shear link, Open ground-story

1. INTRODUCTION

Non-ductile reinforced concrete (RC) buildings with open ground-story are more vulnerable to severe damages or complete collapse under earthquakes, primarily due to inadequate lateral strength and limited plastic rotation capacity of ground-story columns. To enhance their global and local behavior under seismic loading conditions, strengthening of such deficient columns using effective techniques is necessary. Steel caging has been used for strengthening rectangular/square RC columns in several countries (Rodriguez and Park 1991; Dritsos and Palikoutos 1994). Steel cage consisting of rolled steel angle sections with intermediate battens is placed around RC columns with or without using any binder material on the interface between column and steel cage. Passive confinement due to steel cage increases the compressive strength of column concrete, which enhances lateral strength, plastic rotational capacity, and energy dissipation potential of RC columns (Nagaprasad 2005). Moreover, energy dissipation devices are currently employed to protect the deficient structure from severe damage. Soft aluminum panels yielding in shear can be effectively used to dissipate the seismic energy through hysteresis (Rai and Wallace 1998). As shown in Figure 1, such devices have excellent energy dissipation potential without premature inelastic buckling up to 20% shear strains under cyclic shear loads (Jain et al. 2008). Matteis et al. (2007) showed that these low-yield panels remarkably improve the stiffness, strength, and energy dissipation potential of steel moment resisting frames.

This paper evaluates effectiveness of two simple strengthening techniques involving steel caging of ground-story columns and aluminum shear panels as energy dissipation system for open-ground story non-ductile RC frames. A performance–based design methodology based on energy balance concept and target yield mechanism is developed to proportion various elements of the proposed strengthening schemes. Further, seismic performance of both existing and strengthened frames is evaluated by nonlinear static and time-history analyses using a computer program SAP2000 (CSI 2006) to validate the proposed design methodology.

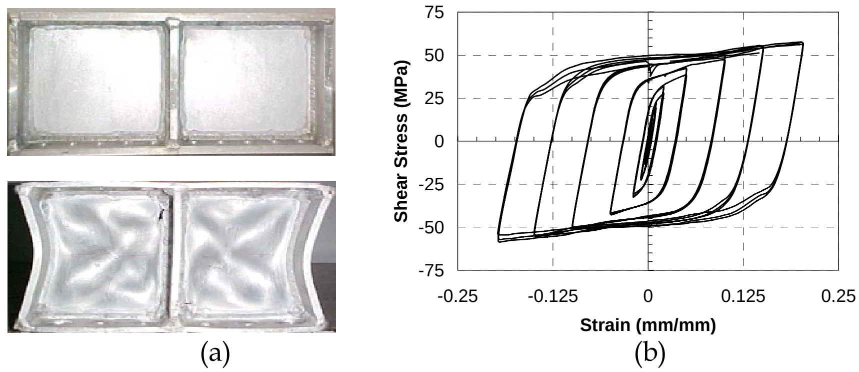


Figure 1 Typical configuration and behaviour aluminum shear panel (a) Original and buckled configuration (b) Hysteretic response under cyclic shear load (Jain et al. 2008)

2. DESIGN METHODOLOGY

Performance-based design procedure based on energy balance concept focuses on designing various elements of the structure at the ultimate state corresponding to a target yield mechanism. The base shear is determined based on target maximum deformation by equating the input energy of a design earthquake to internal energy dissipated through plastic hinges in the system at yield mechanism. This procedure has been earlier presented by Leelataviwat et al. (2002) for design of new building and is suitably modified for design of various strengthening elements for open-ground-story RC frames in this study. As per energy balance concept, energy needed to push a structure monotonically up to the maximum target deformation is equal to the maximum earthquake input energy of an equivalent elastic system (Newmark and Hall 1982). Total input plastic energy of the system, E_p due to earthquake can be expressed as (Leelataviwat et al. 2002):

$$E_p = \frac{WgT^2}{8\pi^2}\left[a^2 - \left(\frac{V_y}{W}\right)^2\right] \tag{2.1}$$

where, M = total system mass; a = normalized pseudo-acceleration with respect to acceleration due to gravity, g; W = weight of system; T = fundamental period. V_y = yield base shear. This input plastic energy must be equal to energy dissipated through the plastic hinges and supplemental devices in the structures. For the frame with open-ground story, the plastic hinges are generally concentrated in ground-story columns and hence, it is reasonable to assume the uniform distribution of inertia forces and displacement over the height of frame (Figure 2). Equating the internal work done to the work done by inertia forces, and noting that sum of inertia forces is equal to the total base shear, V_y, total plastic energy supplied by the frame can be expressed as follows:

$$E_p = \sum_{i=1}^{n_s} F_i h_1 \theta_p = V_y h_1 \theta_p \tag{2.2}$$

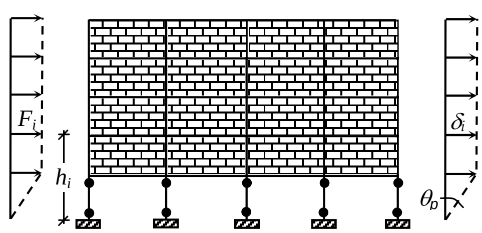


Figure 2 Yield mechanism and distribution of force and displacement for open-ground story RC frame

Comparing Eqns. 2.1 and 2.2, one can get

$$\frac{V_y}{W}\frac{8\pi^2\theta_p h_1}{T^2 g}=\left[a^2-\left(\frac{V_y}{W}\right)^2\right] \tag{2.3}$$

where, n_s=number of story; F_i=equivalent inertia force at level i, h_1=height of ground story, and θ_p=inelastic drift (approximately equal to the plastic rotation of the members). The admissible solution of the quadratic equation as given in Eqn. 2.3 can be given by

$$\frac{V_y}{W}=\frac{-\alpha+\sqrt{\alpha^2+4a^2}}{2} \tag{2.4}$$

where,
$$\alpha\cong\frac{8\theta_p h_1}{T^2} \tag{2.5}$$

Various strengthening elements should be designed such that energy dissipated by the whole structure should be equal to energy demand corresponding to yield base shear, V_y.

3. STRENGTHENING TECHNIQUES

The primary components of proposed strengthening techniques are external steel cage and aluminum shear yielding damper. RC frames with ground-story columns strengthened by external steel cage are termed as *partially-strengthened* (PS) frames, whereas frames strengthened with aluminum panels as energy dissipation devices in addition to steel caging of ground-story columns are termed as *fully-strengthened* (FS) frames. Various elements of each strengthening scheme for an open ground story RC frame are proportioned using the proposed performance-based design methodology as follows:

3.1 Design of PS frame

Since design of steel caging is not intended to enhance lateral strength and stiffness of ground-story of RC frame to be comparable with that of upper stories with masonry infill, it is assumed that all plastic hinges will be formed only in ground-story members in the mechanism state. Thus, total plastic energy dissipated through the plastic hinges, E_p can be expressed as:

$$E_p=\left[2n_b M_{pb}+(n_b+1)M_{pc}\right]\theta_p \tag{3.1}$$

where n_b=number of bays; M_{pb} and M_{pc} = plastic moment of beams and columns, respectively. Comparing Eqns. 2.2 and 3.1, the required moment capacity of the strengthened columns can be given by:

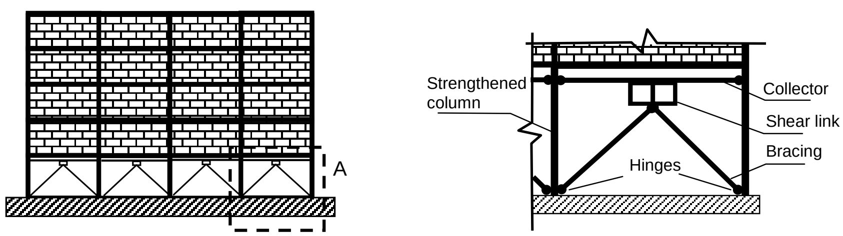
$$M_{pc}=\frac{V_y h_1-2n_b M_{pb}}{(n_b+1)} \tag{3.2}$$

Angle sections and battens of the steel cage can be designed based on a theoretical model developed by Nagaprasad (2005) considering the confinement effect of steel cage on column concrete. Strengthened columns should satisfy the requirements of moment capacity, M_{pc} and plastic rotational capacity of section, θ_p.

3.2 Design of FS frame

A schematic diagram of the FS frame with arrangements of various strengthening elements is shown in Figure 3. Aluminum shear link is supported by two steel braces at the bottom and a shear collector beam at the top. Because of inherent stiffness of aluminum panels and steel braces, lateral stiffness of the strengthened frame increases significantly reducing its fundamental period. Thus, yield base shear, V_y can be computed using Eqns. (2.4) and (2.5) for assumed values of fundamental period and inelastic target drift of the FS frame. Knowing the value of plastic moment capacity of beams and columns, energy needed to be dissipated though these devices, E_d may be computed from the following expression:

$$\left[2n_b M_{pb} + \left(n_b + 1\right) M_{pc}\right]\theta_p + E_d = V_y h_1 \theta_p \tag{3.3}$$



(a) FS frame (b) Details of 'A'

Figure 3 Arrangements of various strengthening elements in fully-strengthened (FS) frame

Force-displacement behavior of aluminum shear panels can be generally modeled as a bilinear elasto-plastic model based on four parameters, namely, characteristic strength, Q, initial stiffness, K_1, post-yield stiffness, K_2, and ultimate displacement, δ_{us}. The post-yield stiffness may be taken as 2.3% of the initial stiffness (Rai and Wallace 1998). Assuming that each bay of the FS frame will be strengthened by aluminum shear panels of same energy dissipation potential, the required characteristic (or yield) strength of the panels can expressed as follows:

$$E_d = 4Qn_b(\delta_{us} - \delta_{ys}) \tag{3.4}$$

where, δ_{ys} and δ_{us} are yield and ultimate displacement of shear panels and equal to 0.002 and 0.20 times the panel height, respectively. The required size of each panel can be obtained using average shear stress, τ_s as 2.4 times the tensile yield stress (may be taken as 25 MPa for annealed aluminum alloy 6063) (Jain et al. 2008). Brace sections can be determined such that (a) the horizontal component of buckling strength of braces must be greater than the ultimate shear strength of aluminum panels, and (b) the equivalent natural period of the strengthened frame must be equal to the natural period assumed for the design of aluminum panels.

4. APPLICATION OF DESIGN METHODOLOGY

A typical four-bay five-story RC frame with open-ground story as shown in Figure 4(a) located in the highest seismic zone-V as per Indian standard IS:1893 (2002) is considered as the study frame. The thickness of infill masonry walls was 230 mm. Elastic moduli of concrete and masonry are taken as 25,000 MPa and 4,400 MPa, respectively, and values of Poisson's ratio of both materials are assumed as 0.2. Unit weights of concrete and masonry are taken as 25 kN/m^3 and 20 kN/m^3, respectively, whereas characteristic compressive strengths of concrete and masonry are considered as 25 MPa and 8 MPa, respectively. Figure 5(b) and (c) show the details of frame members designed for combined gravity loads and wind load as per Indian standard IS: 456 (2000) provisions. Using Rayleigh quotient, the natural period of the RC frame was found to be 0.71 s. It is reasonable to assume the natural period of the PS frame equal to that of the RC frame since steel caging of columns do not cause significant enhancement in overall lateral stiffness of the frame (Sahoo 2008).

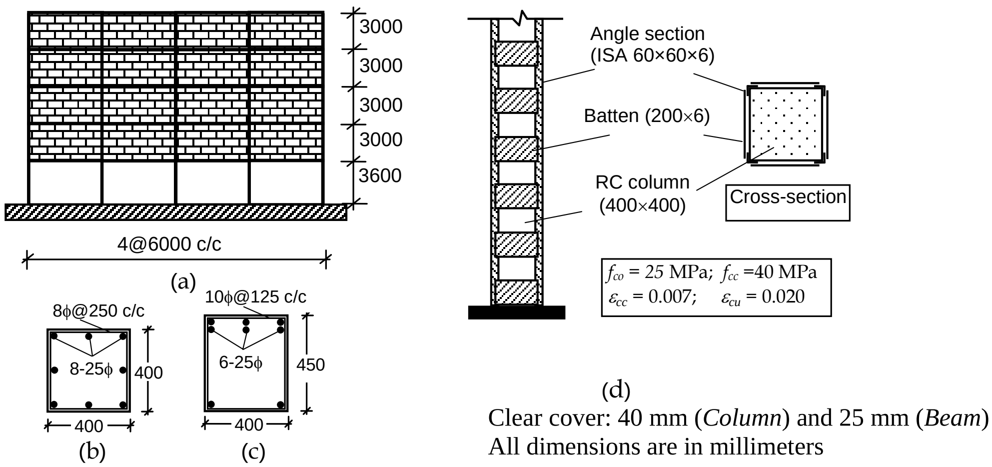


Figure 4 Geometric properties (a) RC frame (b) RC Column (c) RC Beam (d) Strengthened column

Considering moment capacities of existing RC beams and columns as 445 kNm and 235 kNm, respectively, the capacities of strengthened columns required at various plastic drift levels are summarized in Table 4.1. At 1.5% drift, the moment capacity of ground-story columns should be enhanced by 50% that of the RC column. Using the theoretical model proposed by Nagaprasad (2005), steel caging of strengthened columns consisted of four Indian Standard ISA 60×60×6@53.0 N/m angle sections and mild steel battens of 200 mm×6 mm as shown in Fig. 5(d). Table 4.1 also summarizes the length of shear panels of overall height of 300 mm and thickness of 16 mm required at different inelastic drift of the FS frame. In this study, the characteristic strength and length of shear panels were considered as 555.0 kN and 567 mm, respectively, which approximately corresponds to an inelastic drift of 1.9%. Similarly, initial stiffness and ultimate shear strength of panels were computed as 925 kN/mm and 1820.2 kN, respectively. Steel tube sections of size 200 mm×200 m×10 mm were used as braces and Indian Standard ISMB250@373N/m sections were used as collector beams. Both ends of braces and collector beams were pin-connected to strengthened columns.

Table 4.1 Design of strengthened column and energy dissipation device

	PS frame (T = 0.71 s)				FS frame (T = 0.4 s)				
θ_p	V_y/W	V_y (kN)	M_{pc} (kNm)	M_{pc}/M_{pec}*	V_y/W	V_y (kN)	E_d (kNm)	Q (kN)	Ls (mm)
0.005	0.385	2708.5	1019.4	4.4	0.556	3483.7	3884.5	4087.2	4183.1
0.008	0.336	2011.1	804.1	3.5	0.450	2818.4	2565.7	2966.6	2763.0
0.010	0.296	1556.5	624.2	2.7	0.373	2334.8	1724.3	1814.3	1856.9
0.015	0.235	1252.6	349.3	1.5	0.272	1706.7	837.3	881.0	901.7
0.020	0.193	1040.9	156.3	0.7	0.212	1330.6	448.9	472.3	483.4

* *Moment capacity of existing columns* (M_{pec})

5. ANALYTICAL VERIFICATION

Nonlinear static (pushover) and direct-integration time-history analyses were carried out to evaluate seismic behavior of the RC (bare) frame and strengthened frames using SAP-2000 (CSI 2000). Beams and columns were modeled as frame elements and masonry infill walls were modeled as diagonal single-strut elements of width equal to one-fourth of their diagonal width (Paulay and Priestley 1992). Five selected ground motions were considered for time-history analysis (Table 5.1). The equivalent damping was assumed as 5% of critical and soil-structure interaction effect was not included in the analyses.

Table 5.1 Earthquake data and site information of selected ground motions

No.	Name of the earthquake	Station	Richter	Distance (km)	PGA (g)	Soil type
1	El Centro (1940)	Array station#9	6.9	12.2	0.35	Alluvium
2	Chi-Chi (1999)	Taichung	7.6	8.3	0.31	Soft
3	Whittier (1987)	Obregon Park	6.1	14.2	0.43	Alluvium
4	Superstition Hills (1987)	Station#5051	6.6	1.0	0.38	Deep stiff
5	Chamoli (1999)	Gopeshwar	6.6	17.3	0.36	Rock

5.1 Natural period and Initial stiffness

The fundamental periods for the RC, PS and FS frames were observed as 0.84 s, 0.73 s, and 0.45 s, respectively, and the corresponding values of initial lateral stiffness were computed as 51.6 kN/mm, 68.3 kN/mm, 180.0 kN/mm. This indicates an increase in lateral stiffness of about 250% for the FS frame and 32% for the PS frame as compared to the RC frame. Thus, lateral stiffness of the RC frame can be significantly increased by addition of steel braces and aluminum shear panels.

5.2 Displacement-time history

As shown in Figure 6, the existing non-ductile RC frame did not survive strong motions of any selected earthquake and failed within 5 s of ground motion due to formation of plastic hinges in ground-story columns causing uncontrolled story drift. The PS frame also did not survive the entire strong motion of all selected earthquakes except the Whittier earthquake where significant permanent deformation was observed. In contrast, the FS frame survived the entire durations of all ground motions without formation of plastic hinges in any frame members and exhibited significant reduction in roof displacements as compared to both RC and PS frames. Maximum displacement of the first story of the PS frame was 56.1 mm for the El-Centro ground motion, which was reduced by 77.7% for the FS frame. Hence, strengthening of the non-ductile RC frame with steel caging and aluminum shear link significantly reduced the story drift and damage levels of RC members.

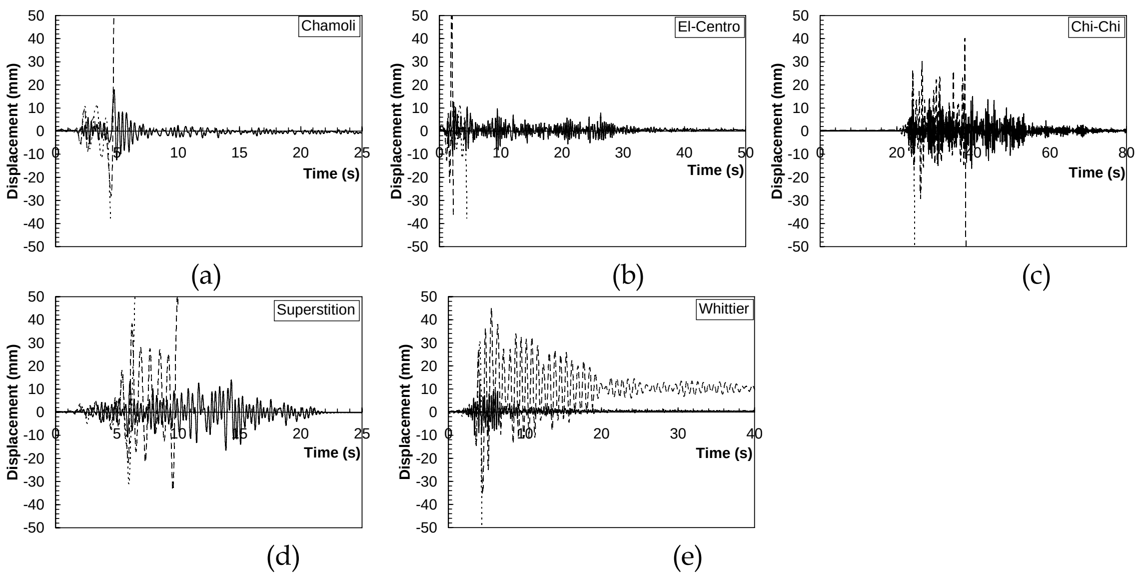
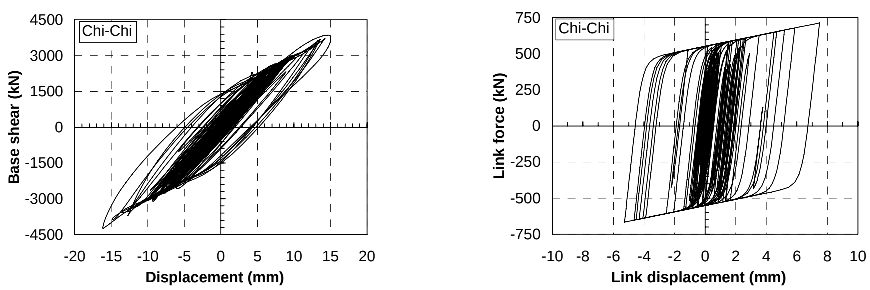


Figure 6 Comparison of roof displacement response of various frames

5.3 Hysteretic response

The FS frame showed full and stable hysteretic loops under each selected ground motions with a maximum base shear of 4327.0 kN as shown in Figure 7(a). The increase in base shear for the FS frame was 369.0% and 83.7%, respectively, as compared to the RC and PS frames. Moreover, both RC and PS frames did not show any notable hysteretic response due to their premature failure at very low drift levels. As shown in Figure 7(b), aluminum shear panels showed full and stable hysteresis loops with significant post-yield strain-hardening behavior. The maximum shear force and shear strain in aluminum panels were found as 722.7 kN and 2.7%, respectively, for the Chamoli earthquake. Because of significant contribution of aluminum panels in load sharing (about 66.8% of total base shear), the seismic demand on existing RC members was also largely reduced. Further, the incidence of inelastic buckling was not noted in shear panels for any selected ground motion as the maximum observed shear strain was well below the buckling strain of 20%.



(a) (b)

Figure 7 Hysteretic response (a) FS frame (b) Aluminum shear panel

5.5 Energy dissipation response

The energy dissipated by the RC, PS and FS frames is calculated as the area of hysteretic loops enclosed during total duration of the ground motion. As expected, energy dissipation potential is negligible for the RC frame. The FS frame dissipated significant amount of seismic energy as compared to the PS frame, which is as high as about 9.0 times for the Chi-Chi earthquake. Hence, energy dissipation potential of non-ductile open ground-story RC frame was significantly improved using aluminum shear panels as energy dissipation devices.

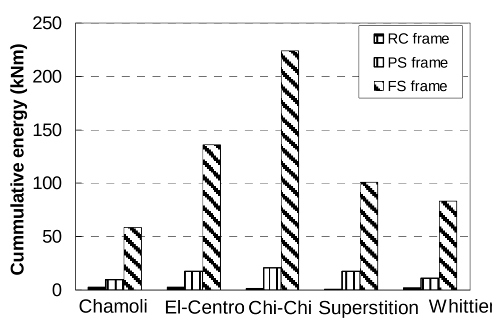


Figure 8 Comparison of cumulative energy dissipation capacities of all frames

6. SUMMARY & CONCLUSIONS

Seismic performance of gravity-load designed RC frames with open-ground story primarily depends on lateral strength, lateral stiffness, and energy dissipation capacity of ground-story members. Two strengthening techniques involving external steel caging for column strengthening and aluminum shear panels as energy dissipation device are designed using energy balance concept to improve the seismic performance of these deficient frames. The RC frame with steel cage strengthened ground-story columns designed for plastic drift of 1.5% showed much better performance in term of lateral strength, controlled drift, and energy dissipation capacity in nonlinear time-history analysis. However, the strengthened frame failed to survive most of the selected ground motions and collapsed due to uncontrolled story drift. In contrast, the RC frame with strengthened ground-story columns and aluminum shear panels as energy dissipation devices enhanced the lateral strength by 360% that of the RC frame and reduced lateral displacement by 77% that of the RC frame with strengthened columns only. Due to shear yielding of panels at very low stress and significant post-yield strain-hardening behavior, the energy dissipation capacity of the RC frame strengthened with aluminum shear panels was increased by about 8 times that of the RC frame with strengthened columns only. More importantly, the strengthened frame with shear panels survived the selected strong motions without any major damage to the RC frame members.

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