17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

PERFORMANCE-BASED PLASTIC DESIGN OF MID-RISE SOFT-STOREY RC FRAMES WITH SRC COLUMNS

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Abstract

Force-based elastic approach adopted in the current codes for seismic design of structures rely on certain modification factors to account for the inelastic behaviour during earthquakes. However, the actual response of structures involves significant plastic deformation of its members during severe earthquakes. This deficiency has given rise to the development of a methodical and reliable design philosophy based on a targeted collapse behaviour to predict the seismic response. The critical factor controlling the seismic behaviour of multi-storied framed structures is the capacity of the members to dissipate the seismic energy imparted on them. Several residential RC framed buildings have open ground stories to facilitate parking and these structures generally tend to perform poorly in seismic prone regions around the world. The sudden reduction in lateral stiffness in the ground story due to the absence of infills tend to induce seismic deficiency in the structure in terms of lateral strength, lateral deformation capacity and energy dissipation.

Steel-reinforced concrete (SRC) structural members are being increasingly preferred over their RC counterparts owing to their enhanced lateral strength, displacement ductility and energy dissipation. These characteristics may assist in eliminating the seismic deficiencies and improving the overall seismic performance of the frame. This paper presents a systematic performance-based plastic design (PBPD) methodology of a mid-rise RC frame structure with encased columns in the open ground story to achieve pre-selected target drift based on performance limits. Energy-work balance principle was used to carry out the plastic design of the structural members employing the capacity-based design philosophy. The modified energy equation was used in this study which accounts for damping in the structure and pinched hysteresis response typical of RC framed structures. This method was applied to a single degree of freedom (SDOF) RC framed structure with encased columns in the open ground storey. Time-history analyses were performed on the both the original and modified frames by subjecting them to several near-field and far-field ground motion records to verify their seismic performances. The inter-story drift demands, ductility of the frame, yield and failure mechanism, propagation of hinge mechanism and dissipation of seismic energies were some of the key parameters evaluated in the study.

Keywords: Soft storey, SRC columns, Energy-balance, PBPD, Time-history analysis.

1. Introduction

Many existing non-ductile reinforced concrete (RC) framed buildings have shown to be highly vulnerable to complete collapse under moderate to high earthquakes. Irregularities in strength or stiffness along the storey heights further aggravates this issue owing to a concentration of forces and moments on specific members which may lead to early collapse of the structure owing to inadequacy in their lateral strength and deformation capacities. Open ground storey (OGS) RC structures are often preferred in urban residential constructions to facilitate parking or storage. The absence of infills in the open storey creates a stiffness deficiency in comparison to the upper stories leading to concentration of seismic forces/displacements demand in columns of the ground storey leading to the complete collapse of buildings.

Steel Reinforced Concrete (SRC) columns have shown higher lateral strength, lateral deformation capacity, and energy dissipation potential when compared to conventional RC columns of the same cross-sectional dimensions [1, 2, 3, 4]. The composite action of structural steel and reinforced concrete acts in tandem to improve the axial and seismic performances of these encased column components. Shear capacity of SRC columns surpasses its flexural strength for most of the cross-sections, thereby preventing a premature catastrophic brittle shear failure phenomenon noted for a typical soft-storied RC framed building.

In this paper, the seismic performance of multi-storied OGS framed structures having SRC columns in the soft storey as a viable alternative constructional feature is validated by performing non-linear time history analyses on numerical structural models. The required axial and moment capacities of the ground storey columns were determined from a performance-based plastic design methodology based on energy balance concept for a pre-determined yield mechanism. The results were compared with the original non-ductile RC framed structure to assess the improvement in performance using the proposed scheme. The modelling and analyses of both the multi-storied framed structures have been conducted using an open-source software *OpenSees* (2000) [5].

2. Design Methodology

Many design codes suggest a force-based methodology for determining the seismic base shear and design of RC structures. While this method proves adequate for regular structures, this generalized design approach fails to reproduce similar results for irregular structural configurations. Performance-based design principles have evolved as an alternative design philosophy by limiting the deformations in the structure. Energy-balance principle proposed by Housner [6] and later expanded to design of multi-storey framed structures by Leelataviwat and Goel [7] is one such displacement based design strategy where the yield base shear is determined by equating the seismic demand in the structure to the energy dissipated by the plastic hinges formed in the structure. Several researchers [8, 9, 10, 11] have successfully utilized this design philosophy for the design of suitable strengthening strategies for seismically deficient multi-storied framed structures.

Energy-balance concept can be mathematically represented as [7]:

$$E = E_e + E_p \tag{1}$$

Where, E, E_e and E_p are the total, elastic and plastic energies of the system, respectively. The total plastic energy in the system can be derived as

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

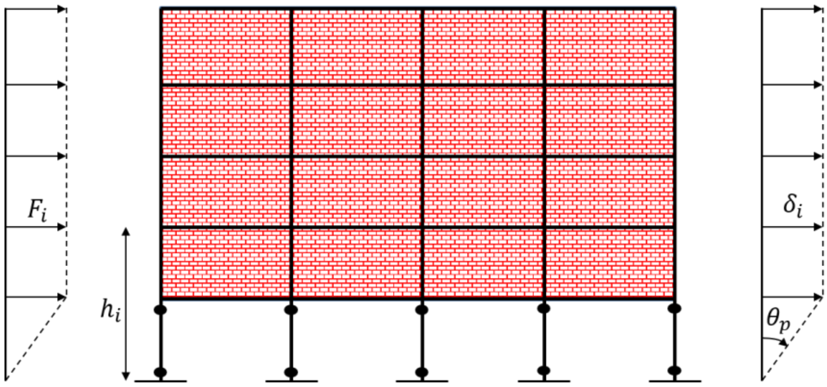


Fig. 1 – Yield mechanism and distribution of forces and plastic displacements for soft storey frame

where, W is the total weight of the system, S_a is the normalized pseudo-acceleration of the demand ground motion, T is the fundamental time period and V_y is the yield base shear. The plastic hinges formed in the framed system must be capable of dissipating this plastic energy. In the adopted case of an open ground storey structure, the plastic hinges are typically accumulated only in the ground storey columns. Therefore, the inertial forces and displacements of the upper stories are generally assumed to be uniformly distributed along the height (Fig. 1). The internal work done by the inertia forces gives the plastic energy capacity of the OGS frame in terms of the yield base shear of the structure:

$$E_p = \left(\sum_{i=1}^{n} F_i h_1\right)\theta_p = V_y h_1 \theta_p \tag{3}$$

By equating Eq. (3) and Eq. (2), the yield base shear of a structure can be obtained as follows:

$$V_y/W = \frac{-\alpha + \sqrt{\alpha^2 + 4(S_a)^2}}{2} \tag{4}$$

Where the non-dimensional parameter α is expressed as:

$$\alpha \cong \frac{8\theta_p \pi^2}{T^2 g} \tag{5}$$

2.1 Time-period Modification Factor

The energy balance relationship defined in Eq. (1) is only applicable for systems having time periods in the acceleration sensitive region of the spectrum. Hence a modification factor γ is applied for the energy-balance expression to be applicable for all time periods. The modified energy-balance equation is given by:

$$\gamma E = E_e + E_p \tag{6}$$

Where,

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{7}$$

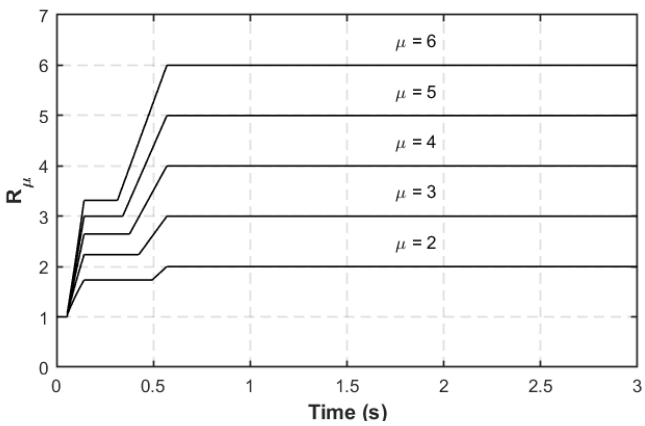


Fig. 2 – Relationship between R_μ, μ_s and T

where, μ_s is the ductility of the system, $\mu_s = \Delta_u/\Delta_y$ and R_μ is a factor which depends on T and μ_s. Fig. 2 illustrates the relationship between R_μ, μ_s and T. By incorporating this time period modification factor into the energy balance equation, Eq. (4) now becomes:

$$V_y/W = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma(S_a)^2}}{2} \tag{8}$$

2.2 Energy Reduction Factor

For RC structures, the hysteresis curves are not as uniform and stable as observed for steel framed structures but rather will have a pinched hysteresis response due to opening and closing and widening of the incurred cracks in concrete. A factor η_p is considered to modify the energy-balance expression considering this pinched hysteretic behaviour. Fig. 3 illustrates the consideration of energy reduction factor from pinched hysteresis curves. The modified energy balance equation is given in Eq. (9):

$$\gamma E = E_e + \eta_p E_p \tag{9}$$

Where,

$$\eta_p = \frac{C_{LT}(1 + r\mu_s - r)}{2(1-r)} \tag{10}$$

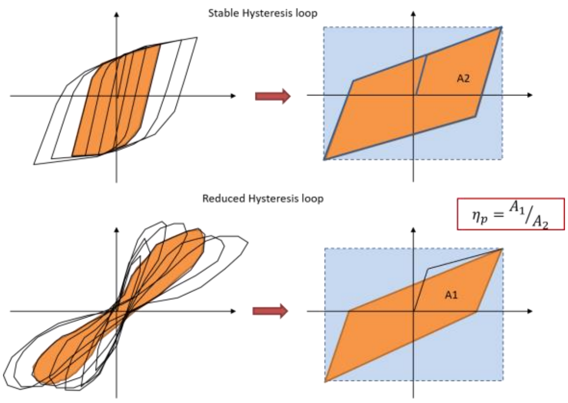


Fig. 3 – Smooth and Pinched hysteresis models

where, r is the post yield stiffness and C_{LT} is a constant depending on T [12]. By considering the energy reduction factor into the energy balance equation, the expression for yield base shear becomes:

$$V_y/_W = \frac{-\alpha + \sqrt{\alpha^2 + 4\frac{\gamma}{\eta_p}(S_a)^2}}{2} \tag{11}$$

3. Design of Ground Storey Columns

The plastic energy in the system is assumed to be dissipated only through the plastic hinges formed in the ground storey following a predefined yield mechanism. This can be expressed as follows:

$$E_p = \left(2mM_{pb} + 2(m+1)M_{pc}\right)\theta_p \tag{12}$$

Where, m is the number of bays and M_{pb} is the plastic moment capacity of a reinforced concrete beam. By comparing Eq. (12) with plastic energy due to inertial forces in the frame system (Eq. (3)) an expression for the moment capacity of the ground storey columns can be obtained as follows:

$$M_{pc} = \frac{V_y h_1 - 2mM_{pb}}{m+1} \tag{13}$$

Axial demand on columns at a height h, can be estimated using the following expression [13]:

$$P_c(h) = \sum_{i=1}^{n} \delta_i \left(\frac{2M_{pb}}{L}\right) + P_{cg}(h) \tag{14}$$

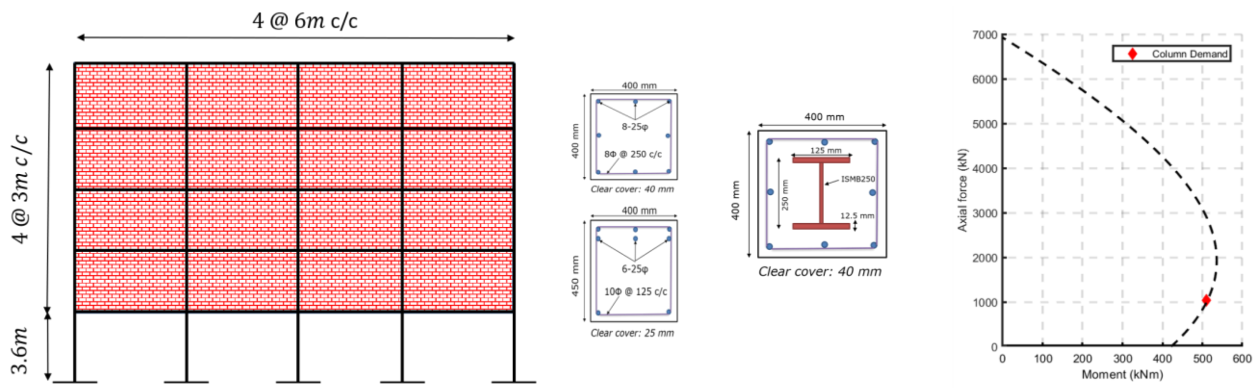
Where,

$$\delta_i = \begin{cases} 1, & h \leq h_i \\ 0, & h > h_i \end{cases} \tag{15}$$

And $P_{cg}(h)$ is the axial force due to dead and live loads in the structure. Once the required moment and axial capacities of the ground storey columns have been determined, the required SRC cross-sections are selected by comparing with the design interaction curves for a selected encased column cross-section. The selected SRC columns should satisfy the calculated axial, moment and plastic rotational demands.

4. Building Models

A five-storey RC framed structure having open ground storey in all its four bays is shown in Fig.4(a). This chosen frame, typical of residential buildings found in India, is located in seismic zone V of the IS-1893 (2016) [14]. The upper stories have infill walls of 230 mm thickness in its bays. Modulus of elasticity of concrete and masonry used in the structure were 25,000 MPa and 4,400 MPa, respectively, with a Poisson's ratio of 0.2. Characteristic compressive strengths of concrete and masonry were taken as 25 MPa and 8 MPa, respectively. Unit weights of concrete and masonry were assumed as 25 kN/m^3 and 20 kN/m^3, respectively. Modal analysis of the soft-storied RC frame provided the fundamental time period of the structure as 0.76 sec., while the modified structure had a time period of 0.65 sec. owing to the increase in stiffness due to the encased columns.



(a) (b)

Fig. 4 – (a) Dimensions of study frame (b) cross-section and interaction curve of SRC column

Fig. 4(a) illustrates the dimensions and structural details of the adopted beam and column cross-sections. Table 1 summarizes the adopted and calculated parameters of the study frame. The moment capacity of the RC beams was calculated as 453 kNm and the required axial and moment capacity of the ground storey columns is given in Table 1. Suitable SRC cross-sections whose interaction curves calculated according to Euro Code 4 (2004) [15] were adopted in a way that the required axial and moment demands lie within the calculated interaction curve of the cross-section. Rolled steel Indian standard steel sections ISMB250 (SP:6(1) 1964) [16] having a yield strength of 250 MPa was found satisfactory to be used in the cross-section. Fig. 4(b) depicts the adopted cross-section of the encased column and its corresponding axial force-moment interaction curve.

Table 1 – Summary of the adopted and calculated parameters of study frame

Design Parameters	Study Frame	Remarks
Grade of concrete	M25	---
Grade of reinforcing steel	Fe415	---
Width of frame	24.0 m	---
Height of frame	15.6 m	---
Fundamental Period, $\boldsymbol{T}$	0.655 sec.	Modal analysis
Coefficient of horizontal acceleration, $\boldsymbol{S_a/g}$	2.077	IS 1893 (2016)
Zone factor, $\boldsymbol{Z}$	0.36	IS 1893 (2016)
Importance factor, $\boldsymbol{I}$	1.0	IS 1893 (2016)
Normalized acceleration, $\boldsymbol{S_a}$	0.75	Calculated
Target drift, $\boldsymbol{\theta_t}$	0.02	Assumed
Yield drift, $\boldsymbol{\theta_y}$	0.003	Assumed
Plastic drift, $\boldsymbol{\theta_p}$	0.0017	Calculated
Ductility factor, $\boldsymbol{\mu_s}$	6.667	$\boldsymbol{\mu_s = \theta_t/\theta_y}$
Ductility reduction factor, $\boldsymbol{R_\mu}$	6.667	Calculated
Energy reduction factor, $\boldsymbol{\eta_p}$	0.525	Dwairi et al. (2002)
Dimensionless parameter, $\boldsymbol{\alpha}$	1.15	Calculated
Total seismic weight of the structure, $\boldsymbol{W}$	7504 kN	Calculated
Base shear ratio, $\boldsymbol{V/W}$	0.216	Calculated
Moment capacity of beams, $\boldsymbol{M_{pb}}$	383 kNm	Calculated
Moment capacity of RC columns, $\boldsymbol{M_{pc}}$	243 kNm	Calculated
Shear capacity of RC columns, $\boldsymbol{V_{pc}}$	178.3 kN	Calculated
Required moment capacity at the ground storey columns, $\boldsymbol{(M_{pc})_{req}}$	520 kNm	From Energy balance
Required axial capacity at the ground storey columns, $\boldsymbol{(P_c)_{req}}$	1034 kN	Calculated

5. Analytical Results

Non-linear static cyclic pushover analysis and time-history analyses were performed on the study frames to verify their seismic performances. These study frames were numerically modelled in *OpenSees* (2000) [5] environment. *Concrete02* material was adopted to model the confined and unconfined concrete, whereas *Pinching4* material was used to model the hysteretic behaviour of reinforcement steel. The material model adopted for structural steel section was *Steel02*. Masonry infills were modelled as diagonal struts having equivalent width and a non-linear hysteretic shear spring in the mid-height of the storey [17]. Transient analyses were performed on the study frames by subjecting them to five selected ground motion records which are detailed in Table 2. Equivalent damping in the structure was assumed to be 5%.

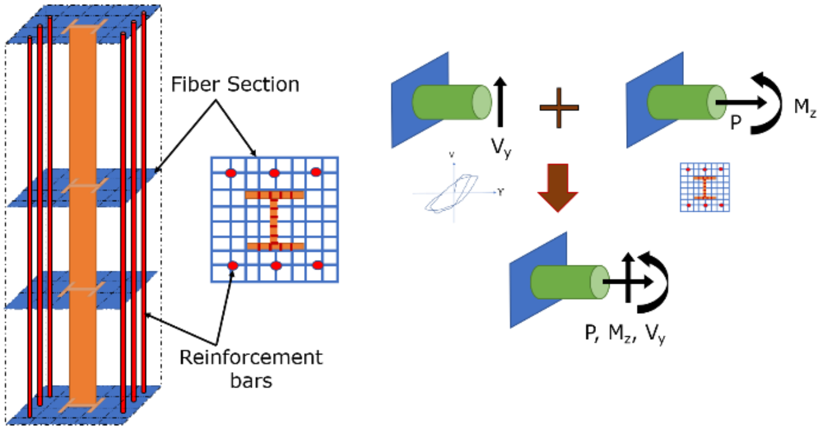


Fig. 5 – cross-section of the SRC column fiber sections with shear hinge modelling

Table 2 – Earthquake data of selected ground motions.

Earthquake	Magnitude	PGA (g)
El-Centro, CA (1940)	6.9	0.35
Chi-Chi, Taiwan (1999)	8.3	0.32
Imperial Valley, CA (1979)	6.53	0.24
Kobe, Japan (1995)	6.9	0.34
Kocaeli, Turkey (1999)	7.51	0.21

Fiber sections were used to model the RC and SRC beam-column elements wherein the interaction between axial forces and moments was taken care of inherently. This sectional model however fails to indicate a shear failure in the member for which shear hinges were defined at the critical ends of the column member. These shear hinges were modelled as zero-length elements having the corresponding rotational shear limit curve. Fig. 5 illustrates the modelling of the fiber sections and the shear hinges used in the study frame. The behaviour of columns follows a purely flexural behaviour until the shear failure is initiated after which the general trend follows the shear degradation curve defined for the member [18]. Shear capacity of RC and SRC columns were determined from analytical expressions available in literature [19].

5.1 Cyclic performance

The study frames having RC and SRC columns designated as R-1 and S-1, respectively, were subjected to a gradually increasing fully reversed-cyclic displacement history to assess their hysteresis performance. The loading protocol adopted in this study was taken from ACI 374.1 [20] for moment frames. Shear failure was initiated at a drift of 0.3% for study frame R-1 and the frame quickly started degrading in the subsequent cycles. The SRC frame S-1 had a higher initial stiffness and lateral strength compared to R-1 and the behaviour was flexural owing to the higher shear capacities of the SRC columns. Fig. 6 shows the hysteresis curves for R-1 and S-1 study frames.

Yield hinges were formed at both ends of all the ground storey columns in frame R-1 with the subsequent formation of shear hinges at the base indicating a soft storey behaviour. In frame S-1, the beams of the first storey yielded first followed by flexural yielding at the base of the ground storey columns. No shear hinges were observed in frame S-1, which frame showed the favourable pattern of hinge progression for the seismic energy dissipation.

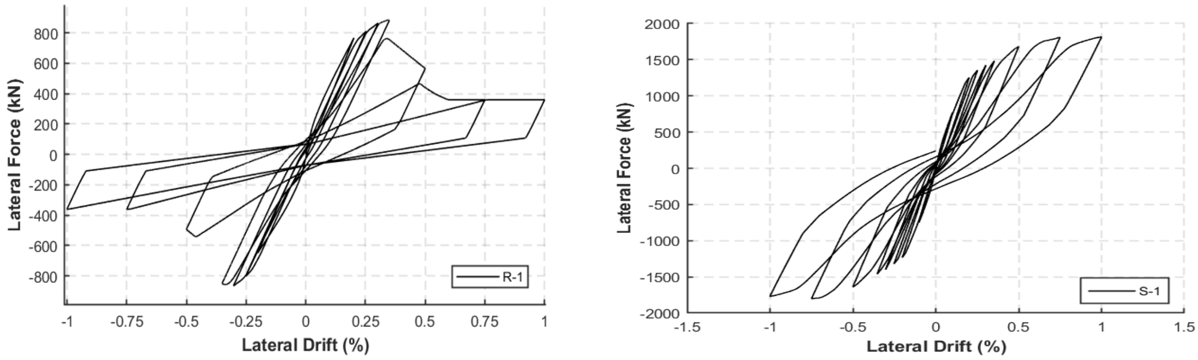


Fig. 6 – Hysteresis response of R-1 and S-1 frames

5.2 Displacement Time-Histories

Fig. 7 depicts the peak roof displacement vs. time history response of both the frame models R-1 and S-1 when subjected to the selected ground motion histories. It was inferred that the non-ductile open ground storied frame R-1 failed prematurely due to soft storey effects in the open storey causing shear induced damage in the ground storey. Table 3 presents the time at which the non-ductile frame failed in the respective earthquake ground motion and the mode of failure. On the other hand, study frame S-1 was able to withstand all five ground motions owing to the increased shear capacity of SRC columns in the open storey surpassing its flexural strength and the ductile behaviour of encased columns. The formation of ductile plastic hinges in the frame ensured a ductile behaviour preventing a soft storey mechanism in the ground storey. It could be inferred from these observations that by providing SRC columns in the deficient soft storey of a building, the premature shear failure due to soft storey effect could be eliminated and the higher ductility offered by these column components help in enduring strong ground motion records without significant damage to structural elements.

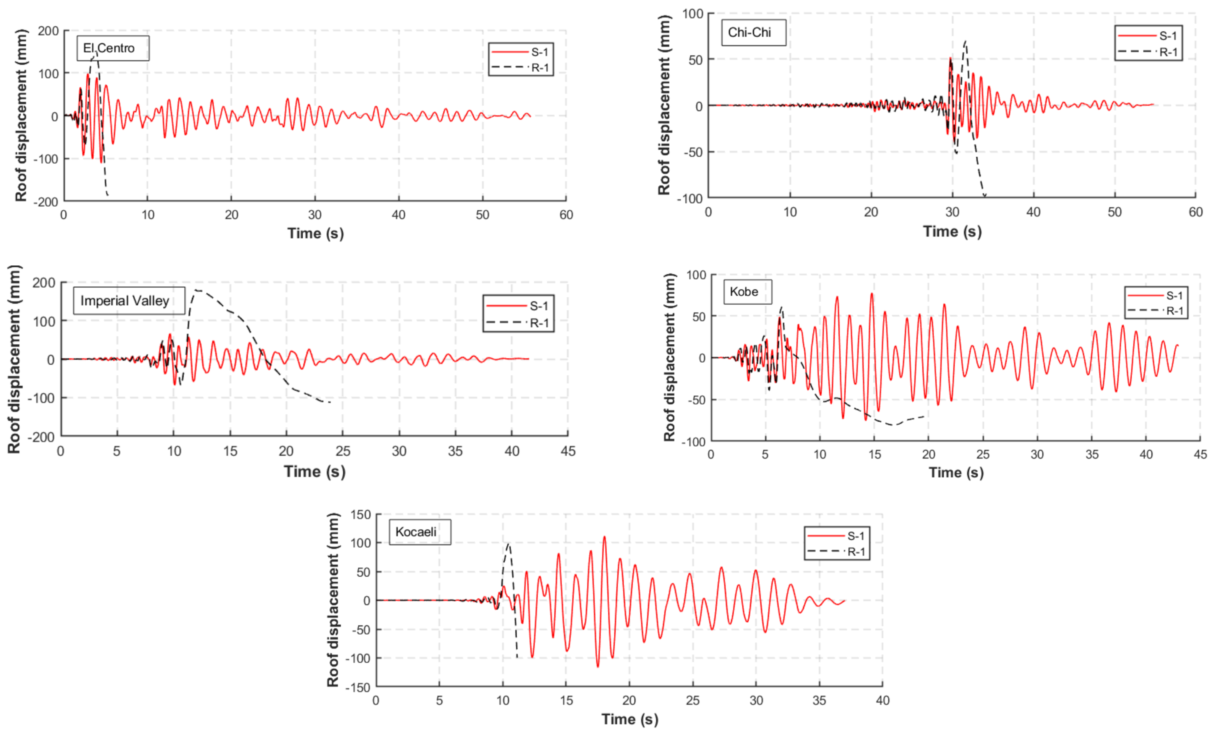


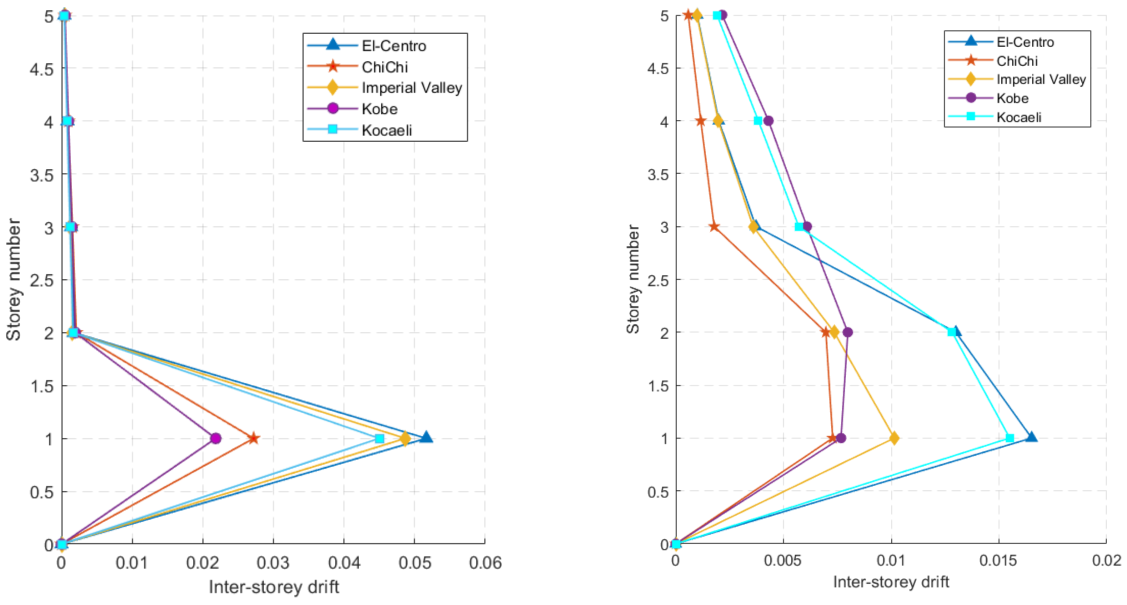
Fig.7 – Displacement time history response of study frames

Table 3 – Time of failure of frame R-1 for the ground motions

Earthquake	Time at failure (s)	Mode of failure
El-Centro	5.284	Shear
Chi-Chi	34.16	Shear
Imperial Valley	14.38	Shear
Kobe	7.936	Shear
Kocaeli	11.13	Shear

5.3 Inter-storey drift ratios

For all five considered ground motions, the corresponding peak inter-storey drift (ISD) ratios were plotted against the storey height for both study frames in Fig. 8. For the non-ductile frame S-1, it could be observed that the maximum drift was concentrated at the level of the soft storey and it exceeded the target drift level of 2% for all five ground motions considered. For three of the ground motions namely, El-Centro, Imperial Valley and Kocaeli, the peak ISD ratio exceeded the collapse prevention limit state of 4% drift indicating the poor performance of the non-ductile framed system. In contrast to this, the ductile frame S-1 had a more gradual drop in ISD across the floors for all the considered ground motions with the peak value being under the target drift of 2% for all the earthquakes. Except the El-Centro and Kocaeli ground motions, for the rest of the considered earthquakes the frame S-1 were within the Immediate Occupancy performance level of 1% according to FEMA-356 (2000) [21] indicating satisfactory seismic performance of the frame. Hence, it could be concluded that by having, SRC columns in the open ground storey of a non-ductile frame designed using energy balance concept, the soft storey effect can be prevented while simultaneously improving the seismic performance of the framed structure.



(a) (b)

Fig.8 – Comparison of peak ISD for the study frames (a) R-1 and (b) S-1

6. Conclusions

Open ground storey RC framed structures are seismically deficient owing to a stark contrast in stiffness in the ground storey leading to a concentration of forces in that storey. This soft storey effect causes the frame to collapse prematurely accelerated by the shear damage in the ground storey columns. From the modelling and analysis results discussed in this paper, it is evident that SRC column when used in place of the conventional RC columns are efficient in not just eliminating the soft storey mechanism but also improves the lateral stiffness and strength of the deficient frame. The following are some of the major observations from this study:

- An improvement in lateral drift capacity was observed from the cyclic pushover and dynamic analyses of frame S-1 when compared with frame R-1 which deteriorated in shear at an earlier drift.
- A comparison with the inter-storey drift ratios of the considered frames showed a significant improvement in frame S-1 for all the considered earthquakes with the damage state being in the Immediate Occupancy or Life Safety limit states whereas the non-ductile RC frame had collapsed for all the ground motions.
- The displacement-time history response of the considered frames shows that the ductile frame S-1 sustained the ground motions without collapse whereas frame R-1, failed in shear for all the considered ground motions.
- Frame S-1 showed favourable hinge progression pattern with the plastic hinges being formed at the beam ends and base of the columns of the ground storey whereas in frame R-1, the plastic hinges were concentrated at both the ends of the ground storey columns resulting in soft-storey mechanism.

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