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Performance evaluation of seismic strengthened irregular RC-steel hybrid frames

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Abstract: New carbon-emission requirements have brought about high demand on retrofitting and strengthening of structures in power-generation plants to accommodate installation of pollution control devices. A typical form of the retrofitted structure is a hybrid reinforced concrete (RC)-steel frame with specific irregularity features. This paper presents the seismic performance evaluation of a representative irregular RC-steel hybrid frame formed by a vertical combination of newly added steel frame and a pre-existing RC frame. The prototype structure originates from a real retrofitting project, and the evaluation is carried out numerically by creating six variants of the hybrid frame retrofitted with different strengthening strategies on the RC frame using nonlinear finite element modelling. The seismic performance of hybrid frame structures is comparatively assessed in terms of roof displacement, story drift ratio, residual displacement and hysteretic energy ratio when subjected to a group of selected earthquake ground motions at different intensity levels. Results show that among the three main types of retrofitting schemes, strengthening the pre-existing RC frame with shear walls appears to be most effective in limiting the roof and inter-story drifts. Strengthening the pre-existing RC frames at the local member level tends to be least effective. Overall, retrofitting the hybrid structure with steel bracings is deemed to be a more robust solution, both in terms of controlling the displacements and enhancing the hysteretic energy dissipation capacity.

Author keywords: seismic performance evaluation; irregular hybrid frame; dynamic time-history; strengthening strategy

Introduction

New carbon-emission requirements have brought about high demand on implementing denitrification device in power-generation plants to reduce emission of sulphur dioxide and nitrogen oxide. In consideration of the factors such as economic cost and construction period, retrofitting and strengthening of the pre-existing structure to accommodate installation of pollution control devices tends to be a more attractive option than demolition and re-constructing a new structure. Hybrid form of structures, such as adding a steel frame on top of a pre-existing RC frame, is seemingly a suitable choice to meet the structural extension needs for power-generation plants. At the same time, such extension design also brings about a special type of vertically irregular frame structures, with particular characteristics such as greater variation in strength and stiffness between RC frame and steel frame, and mass concentration on a certain floor, which are not well covered by existing studies on irregular frames.

In the general context of irregularities of structures, many studies have been carried out on the vertical irregularity, and they provided valuable insights into the seismic performance of vertical irregular structures. Tremblay and Poncet (2005) particularly studied the influence of mass irregularity along the height of the structure on the seismic response, and indicated that the mass irregularity conditions had a limited negative impact on the seismic performance of the structures. Based upon inelastic dynamic time-history analysis, Sadashiva et al. (2012) and Zhou et al. (2015) carried out the seismic response of RC structures with vertical strength or stiffness irregularity, respectively, and they pointed out that the vertical strength irregularity and stiffness irregularity have a significant effect on the seismic performance of the structures. Chintanapakdee and Chopra (2004) studied the seismic demands of RC frame structures with strength, stiffness, and strength and stiffness irregularity, and indicated that the effects of combined strength and stiffness irregularity on seismic demands are larger than the two types of irregularity occurring separately. Then, Michalis et al. (2006)

and Nezhad et al. (2015) investigated the effects of four types of irregularity including stiffness, strength, mass, and combined stiffness and strength irregularities along the height of steel frames on the seismic response. These studies concluded that the effects depend not only upon the type and the location of irregularity, but also on the intensity of the earthquake.

Varadharajan et al. (2014a; 2014b) studied various types of vertical irregular RC frames with changes in location and magnitude of irregularity, and proposed equations to estimate the seismic response parameters of irregular RC frames. Lu (2002) pointed out that the distribution of the story shear overstrength is a stable indicator of the general inelastic behaviour of frames. Further, by combining the effects of story stiffness and strength, Lu et al. (2009) introduced a new story capacity factor to evaluate the regularity of frame structures and indicated that the inverse of the story capacity factor correlates well with the story drift distribution. Considering the characteristics of the seismic input, Zhou et al. (2013) carried out a nonlinear dynamic time-history analysis to investigate the effects of vertical irregularities under pulse-type ground motions, and provided a reference modification of ductility reduction factors accounting for vertical irregularities. Lately, Bohlouli and Poursha (2016) performed an eigenvalue analysis to evaluate the dynamic characteristics of steel moment resisting frames with setbacks, and indicated that the location and the degree of setback are two key factors affecting the seismic response of irregular structures.

Two types of irregularity locations have commonly been considered in previous studies, namely structures with setbacks in the upper stories (Athanassiadou 2008; Sarkar et al. 2010) and structures with irregularity located at a certain story, more generally at the first story (Soni and Mistry 2006; Mwafy and Khalifa 2017). Some complex structures with significant irregularities in geometry, mass, and stiffness distributions are investigated as well (Ceci et al 2011; Zhou et al 2014). However, very few studies have been dedicated to the irregularity induced by hybrid structures with a combination of two different frames in elevation. In this study,

an irregular hybrid structure with a combination of a newly added steel frame on top of a pre-existing RC frame is investigated. To accommodate the steel frame and withstand the increased loads, several strengthening strategies for retrofitting the RC frame are proposed. Thus, various profiles in terms of the vertical irregularity of the hybrid structures arise, as a result of (1) the difference of configuration and material properties between the steel frame and the RC frame; and (2) different degrees of enhancement in stiffness, strength, and distribution of mass caused by the structural strengthening on the RC frame. The seismic performances of such irregular hybrid frames are comparatively evaluated through the Dynamic History Analysis (DHA) in terms of several structural parameters, and a preferable strengthening strategy is proposed to minimize the negative effect induced by the vertical irregularities.

Representative retrofitted structure and retrofitting schemes

Representative retrofitted structure

For the present study an actual retrofitted structure is employed as a representation of the type of hybrid frame structures. In this case, for the purpose of reducing the atmospheric pollution caused by flue gas emission, a denitrification device named Selective Catalytic Reduction (SCR) reactor and flues needed to be installed to the original power plant RC structure. Because of the limited horizontal space of the surrounding area and the denitrification process requirements, the structure had to be extended upward to support the newly added SCR reactor and flues. Considering the site condition of the boiler and the layout of the denitrification devices, a new steel frame was constructed upon the pre-existing RC frame. Thus, a RC-steel hybrid frame structure with a vertical combination of a RC frame located in lower part and a steel frame located in upper part was generated. In fact, similar installation requirements and space constraints occur in many such power-generation facilities across China, see selected examples in Fig. 1. Therefore, the present case structure can be considered as rather representative.

Typical geometric characteristics of the retrofitted hybrid structure and the sizes of primary components are illustrated in Fig.2. The reinforcement details of RC beams and columns are listed in Table 1. The cube compressive strength of concrete is approximately 30 MPa, the yield strengths of longitudinal reinforcement and stirrups are taken as 335 MPa and 235 MPa, respectively. The yield strength of steel columns and beams in the newly-added steel frame is 335 MPa and that of steel bracings is 235MPa.

An important consideration in such a retrofitting scheme is that the load-carrying capacity of the pre-existing RC frame has decreased due to strength and stiffness degradation in the previous long period of service. Therefore, the RC frame should be strengthened first to provide necessary enhancement in the structural stiffness and strength before the addition of the upper steel frame. A suitable strengthening strategy should be considered to adequately cover local strengthening of components, strengthening of beam-column joints, and the global structural retrofitting. Therefore, a variety of possible strengthening and retrofitting schemes can arise. This prompts the needs to conduct a systematic evaluation of the seismic performance of hybrid frame structures with various strengthening arrangements, so that comparative results can be generated to assist in the retrofitting design for this category of structures under the framework of performance-based design.

Local strengthening on components

For the existing RC frame, steel plate jacketing was chosen as the local strengthening technique for the beam-column connections, as schematically shown in Fig.3(a). The beam-column joints were jacketed by steel plates of thickness of 12 mm. Four angle steels ($\angle 125 \times 80 \times 12$ mm) were bonded to the four corners of the RC beam, and steel batten plates (80×10 mm) spaced at 400 mm were welded to the angle steels to form a steel cage. Epoxy resin adhesive was injected into the gap between angle steels, batten plates and RC beams. An enlargement of the cross section was chosen for strengthening of the RC columns. A concrete layer with

higher strength was casted around the existing RC column. A new layer of concrete was reinforced with longitudinal reinforcement according to the strengthening design, and with additional stirrups which were welded to the original stirrups to ensure the integrity of the RC column. The dimensions of column section became 950×950 mm after the strengthening. The strength capacity curves of the RC beams (moment-curvature relationship) and columns (moment-axial force relationship) before and after strengthening are shown in Fig.4.

To understand the strengthening effect on the RC frame, especially the performance of the beam-column joint region, a detailed finite element model was created using the software ABAQUS. The FE model and main results are shown in Fig.5. The results confirmed that the load-carrying capacity of RC beams and columns were improved greatly, and no obvious damage was observed within the beam-column joints. Thus, in the main structural analysis using the structural dynamics analysis program PERFORM-3D, which will be described later, the beam-column joints can be treated as rigid, while the strengthened beam and column end regions are modelled as elastic segment with an equivalent elastic flexural rigidity of the strengthened sections.

Global retrofitting approach

For the RC frame part, in addition to the abovementioned member-level strengthening, global strengthening was also considered, and this was achieved by installing either X-pattern steel bracings or concrete shear walls in the two side spans, as shown in Fig.2(b). Steel plates with thickness of 16 mm were used as jackets for the beam-column joints, and steel gusset plates were used to connect the steel bracing and steel plate by welding and bolting, as shown in Fig.3(b). The I-shaped steel bracings were of dimensions 200×200×8×12 mm, while the thickness of the concrete shear walls was 200 mm and the ratio of wall reinforcement was 0.9% in the horizontal and vertical directions, respectively. The cube compressive strength

of concrete for the shear walls was approximately 40 MPa, and the yield strengths of longitudinal reinforcement and stirrups were 335 MPa and 300 MPa, respectively. The yield strength of steel bracings and plates was 235 MPa.

Variants of retrofitting scheme and analysis models

In such retrofitted hybrid frames, the newly added steel frame part in the upper story is designed in accordance with the current design requirements. Therefore, the vertical distribution of the story strength and stiffness of the overall hybrid frame is dependent mainly upon the strengthening schemes of the RC frame in the lower stories. To evaluate the regularity features of the strengthened hybrid frames and their effects in the seismic response, six variants of the retrofitted frame are created, as shown in Fig. 6 and explained as follows:

(1) Model A: A basic hybrid structure by simply adding the new steel frame to the top of the pre-existing RC frame.

(2) Model B: A hybrid structure with the pre-existing RC frame strengthened with X-pattern steel bracings in the two side spans.

(3) Model C: A hybrid structure with the pre-existing RC frame strengthened with concrete shear walls.

(4) Model D: A hybrid structure with the components of the pre-existing RC frame strengthened locally.

(5) Model E: A hybrid structure with the pre-existing RC frame strengthened with both X-pattern steel bracings and member strengthening.

(6) Model F: A hybrid structure with the pre-existing RC frame strengthened with both concrete shear walls and member strengthening.

It should be noted that for all the six cases, the newly added steel frame in the upper five stories remain the same. The horizontally braced 6th story within the steel frame was to accommodate the installation of the SCR reactor and the flues. The associated stiffening and mass concentration at this story is another feature

of the hybrid structures in terms of the vertical irregularity.

Characterisation of vertical irregularity of the hybrid frames

The maximum lateral deformation demands, namely the roof displacement and the maximum story drift ratio, are key response parameters in the seismic performance evaluation of structures. These response parameters relate closely to the vertically irregularity for a frame. Such correlation is reflected in many simplified analysis methods. For example, for multi-story buildings responding primarily in the fundamental mode, Miranda (1999) proposed the following formulas for the calculation of the roof displacement, Δ_{roof} , and the maximum story drift ratio, θ_{max} , respectively:

$$\Delta_{\text{roof}} = \beta_1 \beta_3 S_d \quad (1)$$

$$\theta_{\text{max}} = \beta_2 \beta_4 \frac{\Delta_{\text{roof}}}{H} \quad (2)$$

where S_d is the displacement spectrum ordinate corresponding to the fundamental period, H is the height of the building, β_1 is the approximate mode participation factor, β_2 is an amplification factor accounting for the concentrations of interstory drifts, β_3 is the nonlinear amplification factor defined as the ratio of the maximum inelastic displacement to maximum elastic displacement, and β_4 is a factor to represent the intensified concentration of interstory drift due to the nonlinear response. The effects of the vertical regularity are represented through the β_2 and β_4 factors.

To characterise the vertical irregularity quantitatively, Lu et al. (2009) proposed a so-called story capacity factor, which represents the combined effect of the story strength and stiffness, and it is defined as:

$$i_{\text{sc}} = i_{\text{os}} \times i_{\text{ns}} \quad (3)$$

where i_{os} and i_{ns} are story overstrength factor and story stiffness factor, respectively.

The story drift distribution is then related to the story capacity by an inversed proportion, i.e. $\beta_i = 1 / i_{\text{sc}}$.

Normalizing $\bar{\beta}_i = \beta_i / \beta_{i,\text{max}}$ such that its maximum story drift coefficient equals unity, the story drift can

then be calculated as:

$$\theta_i = \bar{\beta}_i \theta_{\max} \quad (4)$$

Herein the above method using the story capacity factor is adopted to characterise the vertical irregularity, and hence the distribution of story drift, for the six different designs of the hybrid structures. Consider that the design peak ground acceleration was 0.4g, the values of $\bar{\beta}_i$ coefficient for the six hybrid frames are calculated and plotted in Fig. 7.

It can be seen first of all that the bracing-stiffened 6th story manifests as a drastic reduction in the relative story drift, such that the corresponding $\bar{\beta}_i$ coefficient is considerably smaller than other stories.

Leaving aside the above outlier for the 6th story, the distributions of the $\bar{\beta}_i$ coefficient exhibit distinctive features of the hybrid frame structures retrofitted with different strengthening schemes. For the basic version of the hybrid structure (Model A) without any strengthening of the pre-existing RC frame, the $\bar{\beta}_i$ coefficients within the RC frame stories (story 1 to 3) are much larger than those of the steel frame in the upper stories. Similar is the hybrid frame Model D where the RC frame is strengthened only at the member level, although the drift concentration in the lower 3 stories is somewhat improved. In contrast, for the hybrid structures with the pre-existing RC frame being retrofitted with shear walls (Model C and F), the $\bar{\beta}_i$ coefficients within the RC frame stories are much smaller than those of the newly added steel frame. By comparison, the hybrid structures (Model B and E) with the pre-existing RC frame being retrofitted with steel bracings show a quite uniform drift distribution. The drift distributions from the $\bar{\beta}_i$ coefficients represent clearly the enhancement degree of stiffness and strength resulted from the structural retrofitting schemes.

In order to evaluate more accurately the effect of the different retrofitting schemes on the seismic response of the hybrid structures, and to assess the adequacy of using the story capacity factor in estimating the story drift distribution, dynamic time-history analyses are performed on the six hybrid frames. The detailed

dynamic analysis and the study of the time-history responses of the frames are presented in the following sections.

Seismic response analysis of hybrid frames: structural model and input ground motions

Structural modelling

The structural analysis program PERFORM-3D is used to carry out a nonlinear dynamic time-history response analysis of the six hybrid frame models. The chord rotation model with plastic hinges at the end of a member is selected to simulate both steel and RC beams and columns, as shown in Fig. 8(a), and nonlinear bar elements are used for modelling of steel bracings. The shear wall is simulated by a concrete fiber model with reinforcement details as specified from the design. According to the detailed finite element analysis for the strengthened RC beam-columns as mentioned in the section “Local strengthening on components”, the strengthened end regions of the RC beams and columns are modelled as elastic end zones with an equivalent sectional rigidity. The beam-column joints are treated as rigid, with the sizes of the rigid joint zones being obtained from the actual beam-column connections. In order to satisfy the requirement of the hinge connections at the beam end in the steel frame, a moment release is applied at the end of the steel members which are hinge connected to the RC frame.

A general elastic-plastic and strength degradation model is employed to represent the nonlinear behavior of the individual structural components. The details of the force-deformation relationship are illustrated in Fig.8(b), and it represents the moment-rotation relationship for beams and columns, the axial force-deformation relationship for bracings, respectively. The strength loss of structural components is considered to take into account the generated damages caused by tensile fracture, concrete crushing, concrete shear failure, and steel bracing buckling. The residual strength of RC and steel components is defined according to

the strength loss ratio of residual strength to ultimate strength according to FEMA 356 (FEMA 2000). The residual strength ratio of beams, columns and bracing members in compression is defined as 0.2, and that of bracing in tension is defined as 0.8. Furthermore, stiffness and strength degradation of structural components is also taken into account. For the current study, stiffness and strength degradation parameters are defined based on the degradation rule of reversed cyclic loading in Mander's model (Chen et al 2010); the stiffness degradation is defined according to the degradation factor of unloading stiffness as shown in Fig.8(c), while the degradation factor of unloading strength in all regions is taken as 1.0. Geometric nonlinearity, i.e. the $P-\Delta$ effect, is considered in elastic and inelastic dynamic time-history analyses, and a 4% constant modal damping (Papageorgiou and Gantes 2010, 2011) is defined in the seismic response analysis.

Input ground motions

Several procedures have been proposed in the literature to scale the ground motion records (Watson-Lamprey and Abrahamson 2006; Baker 2010; Sextos et al 2011; Kwong et al 2015). Scaling the records to fit a target spectral acceleration S_a with a given range of period T is adopted in the present study. Generally, scaling the records to the target spectrum at the fundamental period (T_1) has been found to be effective to predict the response of first-mode dominated buildings, while for long-period buildings, studies suggested that scaling the records to the target spectrum at a period range between $0.2T_1$ - $1.5T_1$ (ASCE 2010) or $0.2T_1$ - $2T_1$ (EC8 2004) was more reasonable.

Eight ground motion records as listed in Table 2, obtained from the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) strong motion database, are selected based on magnitude and soil classification. The magnitude of these records is in the range of 6.0 to 7.5, and the average shear-wave velocity in the upper 30 m (V_{s30}) is in the range of $V_{s30}=250$ to 500 m/s to represent the earthquake scenarios with stiff soil class II (classification in Chinese seismic design code).

Scaling procedure of the ground motion records to match the target design spectrum is completed using the software SeismoMatch, which is an application capable of adjusting earthquake record through adding wavelets to the acceleration time-history of the selected record to match the target design spectrum (Hancock et al 2006). Based on Eurocode 8 (EC8 2004), the target design spectra as shown in Fig.9 are determined according to primary characteristics such as Peak Ground Acceleration (PGA) of 0.4g, 0.8g, and 1.2g, damping ratio of 5%; and site class. The ground motion records are scaled to match the entire target design spectrum, as shown in Fig.10(a)-(c). In this way, 24 scaled input ground motions are finally employed in the dynamic time-history analyses.

Seismic response analysis of hybrid frames: results and discussion

Basic structural parameters, such as roof displacement, story drift ratio, residual drift ratio, deformation capacity and energy dissipation, are presented and discussed in this section. Eurocode 8 stipulates that the most unfavorable value of the seismic response should be selected if three different seismic records are used in the dynamic time-history analysis; otherwise, the response can be derived from the average value when seven or more different seismic records are used. Thus, the average value of seismic responses from the eight earthquake record analyses is used in this study.

Roof displacement

The roof displacement is a global measure that relates to the structural damage of the frames. The roof displacements for six hybrid structure models resulting from the time-history analysis with PGA=0.4g, PGA=0.8g and PGA=1.2g, respectively, are shown in Fig.11.

Compared with the basic hybrid frame Model A, Model B and C with the RC frame being retrofitted with steel bracings and concrete shear walls result in a decrease in roof displacements, and the reduction in Model C is more remarkable than that in Model B. The roof displacement decrease in Model D where the RC frame

is strengthened only at the member level is much smaller. These results suggest that steel bracings and concrete shear walls can efficiently reduce the maximum roof displacements of the hybrid frame structure, whereas member level strengthening is less effective. This may be explained by the fact that local strengthening techniques applied on components have limited effect on the improvement of the structural lateral stiffness and the base shear capacity. The same conclusion can be drawn from the comparison on roof displacements between the hybrid structures with pre-existing RC frame being retrofitted with both global and local strengthening techniques and the hybrid structures with pre-existing RC frame being retrofitted with only global strengthening.

The roof displacements, Δ_{roof} , are calculated using response spectrum method assuming the response is governed primarily by the fundamental mode. For this purpose, the fundamental period of the six structures are calculated and these are listed in Table 3. The fundamental periods of the hybrid structures with RC frame being retrofitted with either structural-level or member-level strengthening techniques decrease to varying degrees. Included in the table are also the elastic spectral accelerations and displacements for the six structures, and roof displacements calculated according to Eq. (1). Fig.12 compares the roof displacements calculated according to Eq. (1) and the roof displacements attained in the dynamic time-history analysis.

As can be seen, the roof displacements calculated by Eq. (1) are almost equal to those obtained from dynamic time-history analysis except for the hybrid frames with pre-existing RC frame being retrofitted with concrete shear wall. This may be explained by the fact that the concrete shear wall increases the degree of vertical irregularity of the hybrid frame; besides, the response of the hybrid frame is also affected by higher mode effects while the mode participation factor of the fundamental mode decreases remarkably.

Story drift ratio

Owing to the vertical irregularity, undesirable failure mechanisms of structures are usually governed by

weak stories. Therefore, the story drift ratio, δ , known as one of the key factors in seismic response of the structure, is analyzed to verify the location of weak stories and the damages along the structural height. The story drift ratios obtained from the dynamic time-history analysis with PGA=0.4g are shown in Fig.13. For a comparison, the story drifts calculated according to Eq. (4) are also shown in the figures.

For the basic version of the hybrid structure (Model A), the distributions of story drift ratios calculated by Eq. (4) and the corresponding $\bar{\beta}_i$ coefficients are almost equal to the story drift ratios obtained from dynamic time-history analysis, as shown in Fig.13. The same conclusion can be drawn for the hybrid structure with RC frame being strengthened only at the member level (Model D) and being retrofitted with steel bracings (Model B and E). For the hybrid structure Model C and F where RC frame is retrofitted with concrete shear wall, the story drift ratios calculated by Eq. (4) are smaller than those obtained from dynamic time-history analysis, but the distribution of corresponding $\bar{\beta}_i$ coefficients is consistent with the vertical distribution of stiffness and strength characteristics in the hybrid frames.

Story drift ratios, δ , for six hybrid structures in different seismic levels are illustrated in Figs.14-16. The story drift ratio of the 6th story is evidently smaller than other stories, and it is consistent with the corresponding $\bar{\beta}_i$ coefficients. For Model A and Model D, the maximum story drift ratios within the pre-existing RC frame on the lower part are much larger than those of the newly added steel frame in the upper part. Thus, the weak stories of Model A and Model D occur in the pre-existing RC frames under strong ground motions. For Model C and Model F, on the contrary, the maximum story drift ratios within the RC frame are smaller than those of the steel frame, which illustrates that the weak stories of Model C and Model F, although not remarkable, are located at the bottom story of steel frame. For Model B and Model E, the maximum story drift ratios are relatively uniform except for the strengthened story, and this suggests that no typical weak story exists in these two models.

Based on the above comparisons, the approach using the capacity factor and the corresponding $\bar{\beta}_i$ coefficient proves to yield a good indication of the distribution of irregularity along the frame height. It can be observed in Figs.15-16 that retrofitting the RC frame part with global strengthening techniques, including steel bracings and concrete shear walls, can efficiently reduce the story drift ratios and consequently avoid the occurrence of weak story in the pre-existing RC frame. Although concrete shear walls have more positive effect than steel bracings in reducing the story drift ratio, strengthening the RC frame with steel bracings is deemed to be a preferred approach considering the ease of construction. Furthermore, the structure retrofitted with steel bracings possesses a more uniform distribution of story drift ratios.

Comparatively, the local strengthening techniques only achieve a limited improvement in the lateral stiffness and base shear capacity of the global frame, and consequently results in a limited reduction of story drift ratios. It should be noted, however, that the improvement effect of this strengthening approach tends to increase with the increase of the seismic ground motion intensity. This is mainly due to the fact that after the structure enters into an advanced level of nonlinear response, the structural components that are improved by local strengthening techniques tend to play an increasingly important role.

Distribution of plastic hinges and failure mechanism

The plastic hinge patterns of the six hybrid structure models under one of the ground motions (Northridge, scaled to PGA=1.2g) are depicted in Fig.17. The plastic hinges in beams and columns are shown using circle marks, and the axial yielding of bracings are shown using oval marks.

It can be seen that the inelastic deformation is mainly concentrated in the lower three stories in the basic version of the hybrid structure (Model A) and the hybrid frame which is strengthened only at the member level (Model D), and there is almost no inelastic deformation in the newly added steel frame. Thus, the damage of these two hybrid structures is concentrated in the pre-existing RC frame, even with the member

level strengthening.

The concentration of inelastic deformation in the RC frame stories is effectively eliminated in the hybrid structures with the pre-existing RC frame being retrofitted with steel bracings or concrete shear walls. For the hybrid structures with RC frame being strengthened with shear walls (Model C and F), the inelastic deformation is mainly concentrated in the newly added steel frame, and the failure of the hybrid structure is possibly caused by the damage of steel bracings at the bottom story of the steel frame. For the hybrid structures with RC frame being retrofitted with steel bracings (Model B and E), the inelastic deformation is mainly developed in the steel bracings in both RC frame and steel frame stories, while the inelastic deformation in the RC components is avoided due to the member level strengthening.

Distribution of nonlinear hysteretic energy dissipation in primary components

For hybrid frames with vertical irregularity, the evaluation of the hysteretic energy dissipation in primary components provides further insight into the distribution of cumulative damage and possible failure mechanisms. For this purpose, the hysteretic energy dissipation participation factor γ is adopted, which is defined as a proportion of nonlinear hysteretic energy dissipation in a certain component to the total nonlinear hysteretic energy dissipation. Thus, the sum of the hysteretic energy dissipation participation factors of all components in a structure equals to 1. The hysteretic energy dissipation participation factor can be expressed as follows:

$$\gamma_i = \frac{EH_i}{EH} \quad (5)$$

where γ_i is the hysteretic energy dissipation participation factor of an individual member, for example a steel bracing, a beam or a column.

The average values of γ for each type of the primary components in the six models under the ground motions with PGA=1.2g are indicated in Fig.18, in which $\gamma_{\text{beam-c}}$, $\gamma_{\text{column-c}}$, and $\gamma_{\text{bracing-c}}$ are the

hysteretic energy dissipation participation factors of beams, columns, and bracings in the pre-existing RC frame, respectively. $\gamma_{\text{bracing-s}}$ is the hysteretic energy dissipation participation factor of bracings in the newly added steel frame.

It can be seen from Fig.18 that the energy dissipation is almost totally concentrated in the pre-existing RC frame for Model A and Model D, where the flexural yielding of beams and columns in the RC frame is the primary source of energy dissipation, with beams playing a dominant role. For Model F with the pre-existing RC frame being retrofitted with concrete shear walls, no inelastic deformation occurs in the RC frame, whereas the bracings in the steel frame dissipate almost all seismic energy. The concrete shear walls exhibit hardly any inelastic deformation when the structure is subjected to the strong seismic ground motion.

When the RC frame is retrofitted with steel bracings (Model B and E), the yielding and buckling of steel bracings becomes the main source of the energy dissipation, and at the same time the beams and columns in the RC frame also dissipate an appreciable amount of seismic energy, as can be observed in Fig.18(b) and Fig.18(e). These two cases exhibit a more favorable mechanism of hysteretic energy dissipation, which includes the first lateral-resisting system of steel bracings and the second lateral-resisting system of RC frame. Overall, the hybrid structure with the RC frame being retrofitted with steel bracings and local strengthening techniques achieves a generally uniform distribution of response characteristics along the frame height, and consequently provides a preferable hysteretic energy dissipation capacity in the structural seismic resistance.

Residual roof drift ratio

The residual displacement of a structure system following nonlinear deformation plays an important role in the performance evaluation of the structure, especially in terms of the system stability and repairability. The residual displacements are sensitive to the inelastic mechanism of the system, the hysteretic behavior of structural components, as well as the intensity of the input ground motion; therefore, uncertainty and

variability exists in the assessment of the residual displacements.

Herein the residual roof displacements of the various hybrid frame models under a strong ground motion are discussed. One of the aforementioned 24 scaled input ground motions (Northridge, PGA=1.2g) is selected to carry out a representative study on residual displacements of the six models, and the roof drift ratio is illustrated in Fig.19. In comparison with the basic version of the hybrid structure (Model A) without any strengthening of the pre-existing RC frame, the residual roof drift ratios of the structures with pre-existing RC frame being retrofitted with either global or local strengthening techniques decrease to some extent. A combination of local and global strengthening techniques on RC frame is more effective in reducing residual roof drift ratio.

Conclusions

This paper investigated a special type of irregular hybrid frames arising from retrofitting and extending a pre-existing RC frame with additional stories by a steel frame. The prototype of the hybrid frame was from an actual re-development project. To provide a systematic evaluation of the seismic performance of such hybrid structures, six different strengthening schemes have been considered, resulting in six different hybrid frame structures. These hybrid frames have been analyzed with regard to their irregularity characteristics and the distribution and magnitude of the seismic responses, and the comparative results are examined in order to identify more favorable strengthening schemes especially on the pre-existing RC frame. The following conclusions can be drawn:

- (1) The story capacity factor and the corresponding story drift distribution coefficients ($\bar{\beta}_i$) are observed to represent well the degree of stiffness and strength enhancement resulting from the structural retrofitting schemes, as well as the actual distribution of story drifts comparing with the time history analysis results.

(2) The hybrid structures with RC frame being retrofitted with either steel bracings or concrete shear walls prove to be most effective in reducing the maximum roof displacement. On the contrary, the hybrid structures where the RC frame is strengthened only at the member level appears to be least effective in reducing the roof displacement. This can be attributed to the fact that the local strengthening techniques applied on the RC components generally lead to only a limited enhancement on the overall lateral stiffness and the base shear capacity.

(3) Nevertheless, the hybrid frame structure with RC frame being retrofitted with only local strengthening techniques exhibits a favorable response mechanism with a strong-column and weak-beam characteristic. The hybrid structure where RC frame is strengthened with steel bracings exhibits a dual lateral-resisting system with a good energy dissipation capacity in the structural seismic resistance. On the other hand, the hybrid frame structure with the RC frame being strengthened by concrete shear walls does not perform well in terms of the energy dissipation in the strengthened hybrid frame structure.

(4) The residual roof displacement of the hybrid frame structure with pre-existing RC frame being retrofitted with either global or local strengthening techniques decreases in comparison with the basic version of the hybrid structure, while the combination of local and global retrofitting techniques is more effective in reducing the residual roof displacement.

In summary, while using concrete shear walls in retrofitting the pre-existing RC frame provides remarkable effect on reducing the roof displacement and the story drift ratios, installing steel bracings along the two side bays is deemed to provide a more robust solution in terms of a more uniform distribution of story drift ratios and avoidance of weak stories. Such a scheme also enables a dual lateral-resisting system with good hysteretic energy dissipation capacity in the structural seismic resistance.

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Table 1. Details of RC beams and columns

Component	Size(mm)	Longitudinal reinforcement	Stirrup
Beam-1	400×600	4 Φ 25+4 Φ 25	Φ 8@100/200
Beam-2	400×800	6 Φ 25+4 Φ 25	Φ 8@100/200
Column	650×650	4 Φ 28+16 Φ 25	Φ 10@100

Table 2. Eight ground motion records used in analysis

Number	Earthquake	Year	Station name	Magnitude	V_{s30} (m/s)	PGA (g)
1	Duzce	1999	Duzce	7.14	281.86	0.4043
2	Imperial Valley	1995	Shin-Osaka	6.90	256.00	0.2333
3	Kobe	1979	Coachella Canal #4	6.53	336.49	0.1280
4	Loma Prieta	1989	Gilroy Array #3	6.93	349.85	0.5399
5	Morgan Hill	1984	Gilroy Array #2	6.19	270.84	0.2129
6	Northridge	1994	Leona Valley #2	6.69	435.64	0.0633
7	Parkfield	1966	Cholema-Shandon Array #12	6.19	408.93	0.0597
8	Victoria	1980	SAHOP Casa Flores	6.33	338.60	0.0690

Table 3. Calculated parameters of response spectrum method

Models	T (s)	S_a (g)	S_d (mm)	Δ_{roof} (mm)
Model A	0.552	1.15	86.98	75.97
Model B	0.420	1.15	50.36	38.17
Model C	0.353	1.15	35.57	18.22
Model D	0.499	1.15	71.08	59.54
Model E	0.393	1.15	44.09	34.18
Model F	0.319	1.15	29.05	13.94

(a)

(b)

Fig 2

(a)

(b)

(c)

(d)

(e)

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