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ON DESIGN OF EBFS FOR SEISMIC RETROFIT OF REINFORCED CONCRETE FRAMES

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ABSTRACT

This paper presents selected results of an analytical study that investigated the use of steel eccentrically braced frames (EBFs) for seismic retrofit of existing reinforced concrete buildings. The paper focuses on a low rise reinforced concrete building, representative of the 1960s US west coast construction. The response of the building to strong motion earthquake records is investigated both before and after being retrofitted with steel EBFs. Three retrofit schemes are presented. This research shows that EBFs constitute an effective retrofit measure that can improve safety and control drift. Preliminary Design recommendations for EBF retrofit schemes are presented.

KEYWORDS

seismic retrofit, dynamic analysis, eccentrically braced frames, reinforced concrete building

INTRODUCTION

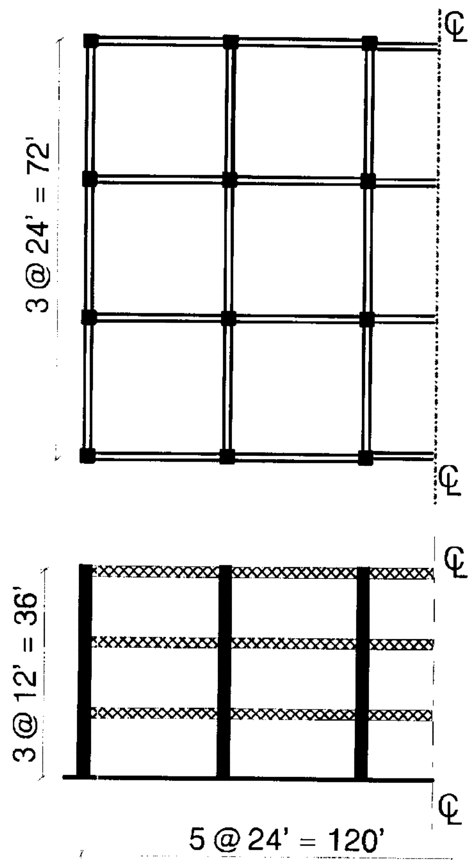
Older reinforced concrete buildings, which are characterized by limited strength and stiffness and by low ductility, are often found inadequate to resist seismic loading by current standards. A number of seismic retrofit techniques for such structures have been developed and implemented in recent years. Steel eccentrically braced frames (EBFs), although a well recognized lateral load resisting system for new construction, have received less attention in retrofit application. This paper presents selected results of an analytical study that investigated the use of steel eccentrically braced frames (EBFs) for seismic retrofit of an existing reinforced concrete frame. Design implications are presented.

DESCRIPTION OF THE EXISTING BUILDING

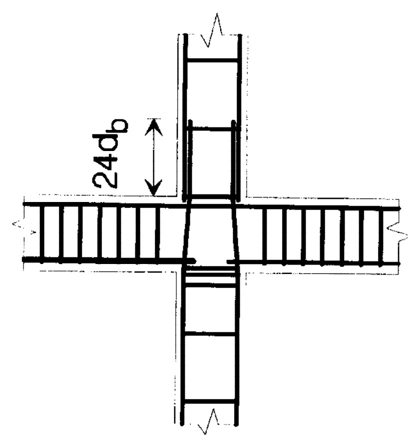
The building presented in this paper is a three-story reinforced concrete building, with 24 ft long bays and 12 ft story heights (Fig. 1a). The building was designed according to the lateral load levels of the 1964 Uniform Building Code (UBC). The member design and detailing followed the recommendations of the 1963 edition of the ACI 318 code. The existing building exhibits several detailing deficiencies (Fig. 1b) which include:

- **Column splice**: The column longitudinal reinforcement was provided with a short lap splice of 24 bar diameters only. The short splice limits the flexural capacity of the columns

- **Confining reinforcement**: The transverse reinforcement in the potential hinging area of the column was widely spaced, resulting in poor concrete confinement. This limits the rotational capacity of the columns and their ability to sustain large deformations or repeated cyclic loads.
- **Anchorage of Bottom Beam reinforcement**: The bottom reinforcing bars in the beams were embedded only six inches into the joint. Consequently the beams have limited positive moment capacity.



(a). Plan and Elevation View



(b). Beam-Column Joint

FIG. 1 Description of the Existing Three-Story Building

DESCRIPTION OF THE RETROFIT SCHEMES

Several steel EBF retrofit schemes were investigated to strengthen the original reinforced concrete building. The behavior of three schemes, referred to in the original study as EBF1-.75, EBF1-1.5, and EBF1-1S, is presented here to illustrate some of the study's main findings. Figure 2 shows the configuration of these schemes and Table 1 summarizes the member sizes.

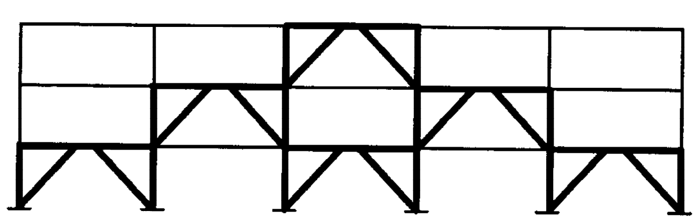


FIG. 2 Configuration of the Retrofit Schemes

Table 1 Member properties for EBF retrofit schemes

	Beams A36	Braces A 500 Gr. B	Links A36		
			Size	Length, e	$\frac{e}{(M_p / V_p)}$
EBF1-.75	W24x55	TS7x7x1/2	W24x55	18 in.	0.75
EBF1-1.5	W24x55	TS7x7x1/2	W24x55	40 in.	1.50
EBF1-S	W14x38	TS7x7x5/16	W14x38	24 in.	1.00

(1) M_p: Plastic moment capacity of the link
V_p: Shear plastic capacity of the link

To limit disruption to the building, the steel EBFs were added to the outer bays only. The strength of the EBFs was reduced with height, by reducing the number of braced bays, in an attempt to match the seismic demands. Such strength distribution with height was found to improve the performance of the structure by promoting a more uniform distribution of yielding among links and by avoiding a concentration of inelastic distribution at the lower stories (Bouadi, 1994).

In order to obtain the maximum stiffness, strength and ductility from the EBFs, short shear yielding links were used. Links of length less than 1.6 M_p/V_p yield primarily in shear (Kasai and Popov, 1986). M_p is the plastic moment capacity and V_p is the plastic shear capacity of the link. The link length of EBF1-.75 and EBF1-1.5 were taken equal of 18 in. and 40 in., representing approximately 0.75 M_p/V_p and 1.5 M_p/V_p respectively. EBF1-1S was designed in an attempt to provide the smallest member sizes capable of resisting the selected ground motions without exceeding the link rotational capacity. This scheme had a link length of 24 in., representing approximately M_p/V_p.

DESCRIPTION OF THE ANALYTICAL PROCEDURE

The DRAIN2D computer program (Kanaan and Powell, 1975), which allows for nonlinear analysis of two dimensional frames under lateral load was used in this study. The structures were analyzed under static incremental loading and under dynamic earthquake loading. A stiffness degrading element was used to model the behavior of reinforced concrete members. Strength and stiffness degradation after splice failure of the columns and beams was also modeled. The steel braces were modeled assuming elastic perfectly plastic behavior in tension and a stiffness and strength reduction after first buckling in compression. The link element used in this study was developed by Tang and Goel (1988) to model short shear yielding links of EBFs. The inelastic behavior of the link was modeled by considering a bi-linear relation between the shear and the relative end displacement of the link.

STATIC ANALYSIS

A static inelastic analysis was performed to investigate the strength and stiffness of the original structure and retrofitted structures. Figure 3 shows the relationship between the base shear and the maximum interstory drift for the original and the retrofit schemes.

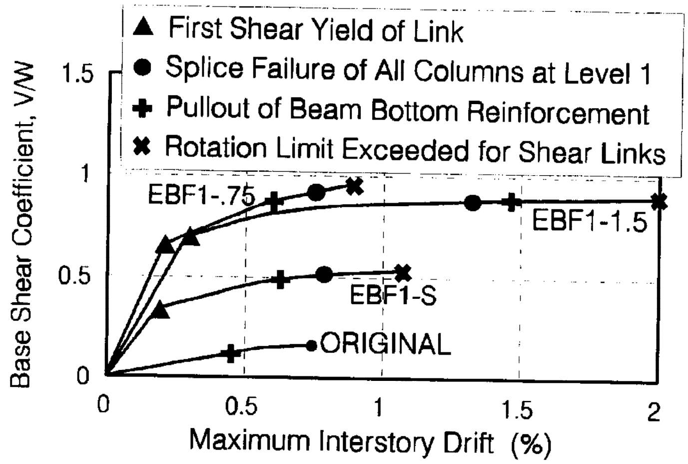


FIG.3 Maximum Interstory Drifts of the Original and Retrofit Buildings under Static Loads

The behavior of the existing building was essentially elastic until pull-out of the beam bottom reinforcement. Continued loading caused a splice failure in some columns. At 0.75% interstory drift, corresponding to a load of 0.16 W (with W being the building weight) all columns of the first floor experienced a splice failure resulting in a rapid degradation in strength and stiffness. This event was considered as defining the maximum deformation capacity of the building.

The static analysis showed that the three-story building was very flexible and had low strength and ductility. As indicated by Fig. 3, the retrofit schemes provided an increase in stiffness, strength, and in inelastic deformation capacity. Inelastic action in EBF1-.75 started at a drift of 0.22% upon yielding of the first level links. This drift corresponded to a base shear of about 0.7 W. The maximum strength of the EBF1-.75 was reached at a drift of 0.9% and was equal to 0.97W. EBF1-1.5 behaved elastically until yielding of the first and second level links at a drift of 0.31% corresponding to a lateral load of 0.7W. The links failed at an interstory drift ratio of 2% and the ultimate lateral capacity of EBF1-1.5 was estimated equal to 0.91W. For EBF1-1S, which had smaller member sizes, first yielding of the links was predicted at a drift level of 0.18% corresponding to a lateral load of 0.34W. At a drift of 1.1% the rotational capacity of the links was reached. The maximum lateral strength of the EBF1-1S scheme was found to be equal to 0.53W.

A comparison between EBF1-.75 and EBF1-1.5 shows that increasing the link length results in a stiffness decrease and in a substantial gain in ductility for the frame. Also, the deformation capacity of the structure is shown to increase with the link length. Note that larger link lengths correspond to larger e/L ratios, e, being the link length and L being the bay width. The relationship between link plastic rotation (γ) and the interstory drift (θ) for the EBF schemes shown here, can be estimated as: $\gamma = (L/e)\theta$ or $\theta = (e/L)\gamma$. This relation is based on the kinematics of a rigid-plastic mechanism. The links are considered to have uniform deformation throughout the height and all floors are assumed to have the same drift ratio. Thus, when the link rotation limit is achieved, a larger plastic frame drift is possible with a larger e/L ratio.

DYNAMIC ANALYSIS

The dynamic analysis was conducted using a set of firm and soft soil records. The present paper is limited to two strong records on firm soils: the N00E component of the 1940 EL Centro record scaled to a maximum ground acceleration of 0.5g and the N00E component of the Corralitos record from the 1989 Loam Prieta earthquake which had a peak ground acceleration of 0.64g.

Figure 4 shows the maximum interstory drifts for the original building subjected to the scaled El Centro record. Maximum interstory drifts occurred in the first level and was in the vicinity of 2.5%. Pullout of the bottom bar reinforcement in the first level beams and splice failure of all first level columns was predicted. Severe damage can be expected and the ability of this structure to sustain these large drifts without collapse appears doubtful.

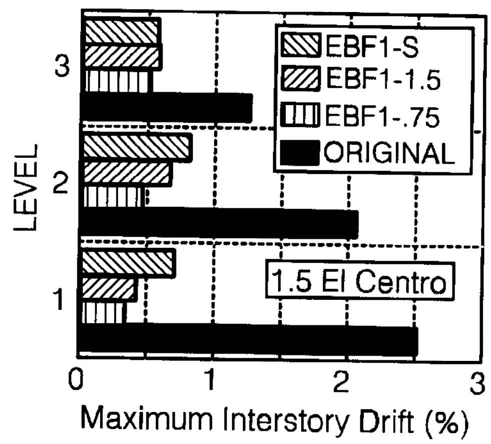


FIG. 4 Maximum Interstory Drifts Under the Scaled El Centro Earthquake Record

As indicated in Fig. 4, the retrofit schemes provided an increase in strength, stiffness, and in inelastic deformation capacity. The maximum interstory drift ratios for EBF1-.75 and EBF1-1.5 were kept below 0.6% and 0.7% respectively. Larger drifts were predicted for EBF1-1S. However, these drift levels remained within acceptable limits. Maximum interstory drifts were higher for EBF1-1.5 than for EBF1-.75. This is in agreement with the static analysis that showed that an increase in the link length resulted in a reduction in stiffness.

Figure 5 shows maximum link plastic rotations of the EBF retrofit schemes under the Corralitos earthquake record. Link rotation provides a good indication of the ductility demand of EBFs since inelastic activity is limited primarily to the links. Experimental research (Kasai and Popov, 1986) has shown that well detailed links can sustain a maximum plastic rotation of 0.10 rad. under cyclic load. Beyond 0.10 rad. of plastic rotation, link strength degrades rapidly due to buckling and tearing of the link web.

The links at all levels of EBF1-.75 and EBF1-1.5 experienced relatively large rotations. The maximum rotation occurred at the second level and was equal to 0.058 rad and 0.047 rad. respectively. The link deformations of EBF1-.75 and EBF1-1.5 suggest that the maximum link plastic rotation increased with a reduction in the link length, despite the fact that the frame stiffness increases with a reduction in link length. This can be attributed to the relationship between story drift and link rotation. As discussed earlier, the shorter links result in larger L/e ratios, and consequently result in larger link rotations for the same drift.

The maximum link rotation for EBF1-S under the Corralitos record measured 0.10 rad, and was thus equal to the link rotation capacity. Schemes with smaller link sizes were analyzed and were found inadequate to resist the strong motion earthquake records used in the study, since they exceeded link rotation limits.

In summary, the dynamic analyses indicated that the retrofit schemes provided a substantial improvement in the seismic behavior of the building. EBF1-1S was shown to be the most economical scheme in preventing collapse under the dynamic loading due to the selected records. This scheme required a minimum amount of material and kept the interstory drifts and the link rotations within acceptable limits. EBF1-1.5 and EBF1-.75 provided greater strength and stiffness, maintained lower interstory drifts, and provided a higher degree of damage control.

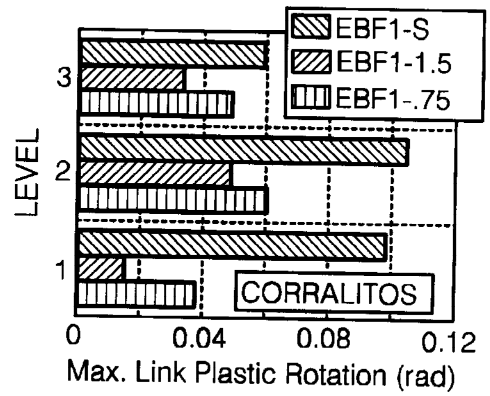


FIG. 5 Maximum Plastic Link Rotation Under the Corralitos Earthquake Record

DISCUSSION AND DESIGN IMPLICATIONS

Based on the analytical results obtained in this study, preliminary recommendations for the design of EBF retrofit schemes are made in this section. These recommendations are considered preliminary, since they are based on a limited number of analyses. The design recommendations are intended for use with the procedures of the 1991 UBC.

Estimation of Required Strength.

The dynamic analysis indicated that EBF1-S provided the minimum strength needed to survive the selected ground motion records. The design of EBF1-S was obtained by trial and error procedure, with the trial designs evaluated with inelastic dynamic analysis.

EBF1-S would result from a design based on UBC 91 using an acceleration factor, Z of 0.4, a soil coefficient, S of one, a period of vibration, T from an elastic dynamic analysis, and a force reduction factor, R_w of 4.5. The UBC forces obtained from this procedure should be applied to the added EBF only when designing the steel members of the EBF retrofit scheme. The soil coefficient was taken equal to one to reflect the firm soil conditions. The period of vibration was based on the dynamic analysis of the retrofitted system (original frame and added EBF), to better reflect the vibration characteristic of the retrofitted structure.

Estimation of Ultimate Drift and Maximum Link Rotation

In design practice, the actual maximum drift expected under an earthquake is often estimated by multiplying the drift obtained under code design forces by a displacement amplification factor, DAF. The UBC 91 code specifies an amplification factor of 3/8 R_w. Figure 6a shows the maximum interstory drift for EBF1-S under three strong ground motions and the maximum interstory drift calculated using the UBC procedure with a DAF of 3/8 R_w. In general, UBC DAF significantly underestimates the maximum drift predicted by the inelastic dynamic analysis. The ultimate drift ratio predicted by the UBC DAF for the EBF1-S configuration was equal to 0.22%. However the dynamic analyses indicated that maximum interstory drift ratio was close to 1%.

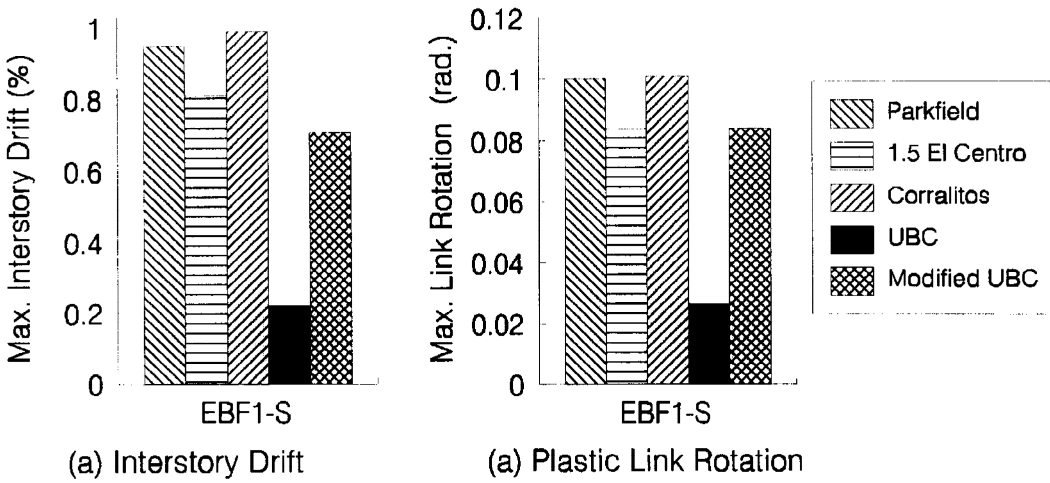


Fig. 6 Maximum Interstory Drift and Plastic Link Rotation for EBF1-S Configuration

In an attempt to make a more accurate prediction of interstory drift, the elastic drift under UBC forces was amplified by a displacement factor equal to 1.2 Rw. The drifts, computed with this higher DAF are also shown in Fig. 6a and are referred to as "modified UBC". With this DAF, a better correlation was obtained with the computed drift under dynamic loading. Drift estimations were still somewhat unconservative but were closer to the computed values than the drifts given by the UBC DAF.

Links can be considered the most critical member in an EBF structure. Energy dissipation under seismic loading is limited to the links and the capacity of an EBF is controlled by the link's strength and deformation capacity. For design purposes, and as explained earlier, ultimate link rotation is, γ, is typically taken equal to $(L/e)\theta$, where θ is the interstory drift, L is the bay length, and e is the link length.

Figure 6b shows the computed plastic link rotations of EBF1-S based on inelastic dynamic analysis. These are compared, with the link rotations estimated using the above equation, with the ultimate interstory drift computed using the UBC DAF. These simplified estimates of the link rotation greatly understimated the computed link rotations. This is primarily due to UBC's underestimation of maximum story drift. To improve predictions of the link rotations, calculations were made using a displacement amplification factor of 1.2 Rw. The corresponding results are also shown in Fig. 6b and are referred to as "Modified UBC". Overall, this procedure resulted in a satisfactory prediction for link rotations. The predicted link rotation was within 15% of the measured rotation.

In summary, the current UBC displacement amplification factor of 3/8 Rw significantly understimated the ultimate drift for the building presented herein. An amplification factor of 1.2 Rw was found to provide more accurate estimates of the ultimate drift under the severe earthquakes considered in this study.

CONCLUSIONS

EBFs constitute an effective retrofit scheme that can improve safety and control drift. For the case study, the added EBF can be designed to resist severe ground motions using the UBC procedures but taking a force reduction factor, R_w, of 4.5 and a period of vibration, T, from a dynamic analysis. The design lateral load should be applied to the added steel EBF only. The displacement amplification factor (DAF) of 3/8 R_w used in the current UBC was found to be too low. A higher value for the DAF is needed in order to obtain a satisfactory estimate of the interstory drift and plastic link deformation under earthquake loads. A value of 1.2 Rw is proposed here.

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