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Seismic evaluation and upgrading of chevron braced frames

D.C. Rai [a], S.C. Goel [b,*]

[a] *Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur 208 016, India*
[b] *Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI 48109, USA*

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Abstract

Many Chevron type "ordinary" steel concentric braced frame (OCBF) structures have suffered extensive damage in recent earthquakes which raises concerns about their performance in future earthquakes. A building in the North Hollywood area, which suffered major damage in the 1994 Northridge earthquake, was selected for detailed study. Response spectrum, nonlinear static (pushover), and nonlinear dynamic (time history) analyses for a ground motion recorded at a nearby site compared well with the observed damage. The state-of-health of the damaged structure was assessed to determine the need and extent of repair. The seismic performance of non-ductile CBFs can be improved by delaying the fracture of braces, e.g., in the case of the tubular braces by filling with plain concrete. Changing the bracing configuration from chevron to 2-story X configuration can avoid the instability and plastic hinging of floor beams. Further improvement can be achieved by redesigning the brace and floor beams to a weak brace and strong beam system, as in Special CBFs. This full upgrading to SCBFs results in excellent hysteretic response and, with inelastic actions confined to ductile braces, exhibits reasonable distribution of damage over the height of the building.

Keywords: Earthquake-resistant; Concentric braced frame; Bracing; Seismic evaluation; Seismic upgrading; Retrofitting

* Corresponding author. Tel.: 734-764-8419; fax: 734-764-4292.
E-mail address: subhash@engin.umich.edu (S.C. Goel).

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Nomenclature

b	gross width of a tubular section;
C_s	numerical constant for fracture life of tubular braces;
d	gross depth of a tubular section;
F_y	specified minimum yield stress of the type of steel being used;
g	acceleration due to gravity;
k	effective length factor for a brace;
l	unbraced length of a brace member;
L	length of the simply supported floor beam;
M_{un}	girder moment due to P_{un};
N_f	fracture life in terms of standard cycles;
P_{br}	elastic buckling capacity of a brace;
P_{un}	unbalanced vertical load at the intersection of braces in chevron pattern when compression brace is at its post-buckling capacity;
P_y	tensile yield capacity of a brace;
R_w	structural response modification factor;
r	minimum radius of gyration of a brace member; and
θ	angle which a brace makes with the horizontal.

1. Deficiencies of CBF structures and cyclic behavior of bracing members

Concentric Braced Frames (CBF) are among the most efficient structural systems in steel construction for resisting lateral forces due to wind and earthquakes because they provide complete truss action. However, this structural framing system has not been considered ductile by building codes and past design practices. In past earthquakes, including the recent 1994 Northridge and 1995 Kobe events, a significant number of CBF structures suffered extensive damage, requiring extensive repair and upgrading work. While the total and catastrophic failures of steel structures and the resulting loss of life have not been too common in the past, recent experiences suggest the need for evaluating the damage potential of structures for possible strengthening or upgrading. Upgrading deficient structures before a major earthquake will generally minimize the economic loss resulting from the closure or demolition of such facilities after the earthquake.

Under seismic loading, the bracing members undergo large deformations in the post-buckling range, causing large reversed cyclic rotation at the plastic hinges formed in the brace members and in the connections at either end. In a severe earthquake, these post-buckling axial deformations can be as large as 10–20 times their yield deformation. This amount of ductility was not ensured by design practice prior to Uniform Building Code 94 [1], which, instead of requiring more ductility from the bracing members and their connections, emphasizes the strength. This design philosophy results in rather poor performance of CBFs under severe ground motions [2,3].

Chevron type bracing (i.e., inverted V-bracing) is a very popular pattern of arranging braces for CBFs. Elastic analysis and design methods used in past practice did not require checking the strength of the floor beams for the unbalanced force that is induced due to tension in one brace and smaller compression force in the other after buckling. The lack of adequate strength and lateral support of these floor beams can seriously undermine the lateral resistance of the bracing system [4]. Comprehensive design provisions for new construction were adopted for the first time in the 1994 edition of the Uniform Building Code [1] as Special Concentric Braced Frame (SCBF) structural system. However, there exists a large inventory of non-ductile CBFs designed according to past code provisions in highly seismic regions.

2. Objectives of the study

The focus of this paper is on the seismic behavior of steel buildings in which CBFs are used as a primary lateral load system. One such building, which suffered major damage in the 1994 Northridge earthquake, is central to this study [5,6]. The objective of the paper is to describe and illustrate:

1. the seismic evaluation analyses that can be used to identify the deficiencies of CBF structures, which includes nonlinear dynamic analyses, the results of which are compared with those of a post-earthquake field investigation;
2. the analytical projections of the seismic demand, extent of failure/damage, and the resulting consequences in the event of the future earthquakes of greater severity; and finally
3. a few upgrading schemes for deficient CBFs with various levels of expected performance and comparison of their relative merits regarding the effectiveness and the feasibility of implementation.

3. Description of the study building

The study building is a four-story steel braced frame structure (Fig. 1(a)), constructed in 1986 in accordance with the 1980 edition of the Los Angeles Building Code [7]. The study building is located in the North Hollywood area of Los Angeles, CA, about 16.9 km east-southeast of the Northridge earthquake epicenter as shown in Fig. 1(b). This building was chosen as the study building because its characteristics are representative of a number of existing CBF structures. These CBF structures have the potential for damage to tubular bracing members, floor beams, and connections.

The typical framing plan and building shape on the first floor is shown in Fig. 2. The building has an additional basement level in the southern one-third portion, and the first floor plan is slightly different from the identical upper floors. Floor diaphragms consist of concrete fill on metal decking, spanning in the N–S direction, and can be considered as rigid diaphragms for the purpose of transferring the horizontal shear to vertical frames.

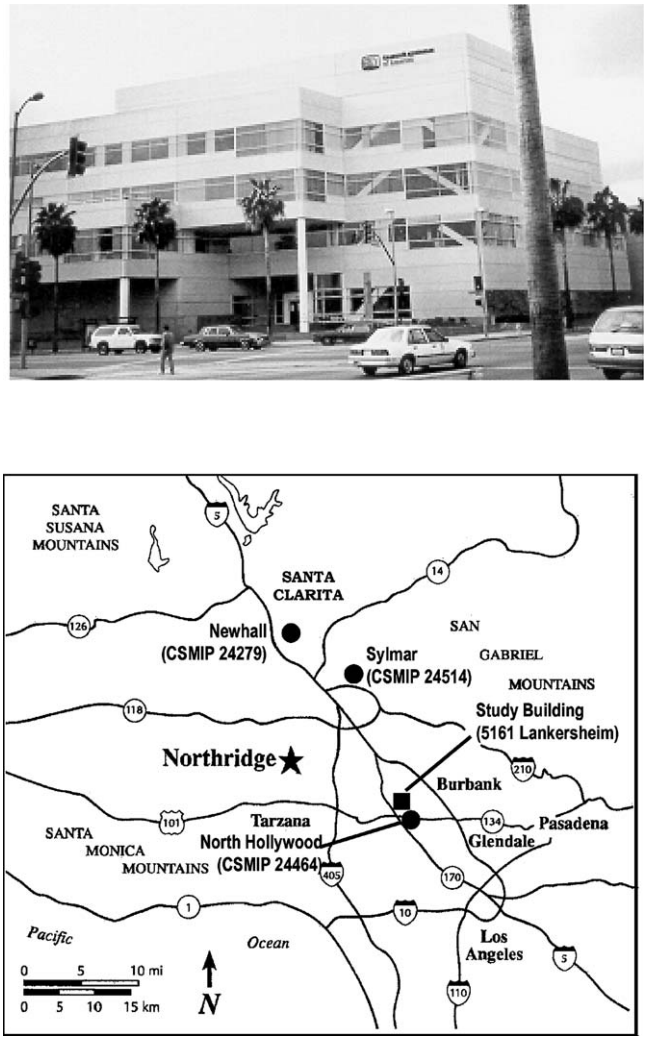

Fig. 1. (a) North-East elevation of the study building after the Northridge earthquake (b) Location of the study building, the epicenter of the Northridge earthquake and relevant CSMIP ground motion recording stations ([14]).

The lateral load resisting system consists of six braced bays oriented along the two principal directions of the building, as shown in Fig. 2. However, two frames J.5 and D.5 are at an angle with the principal N–S direction. The elevation and member sizes of a typical lateral braced frame are shown in Fig. 3. The foundation consists of both spread footings and drilled piers, where the latter type is used for all columns resisting seismic overturning. Soil investigations indicated that the site is located on a medium to very dense alluvial cohesionless soil, which can qualify as UBC S_2 type soil.

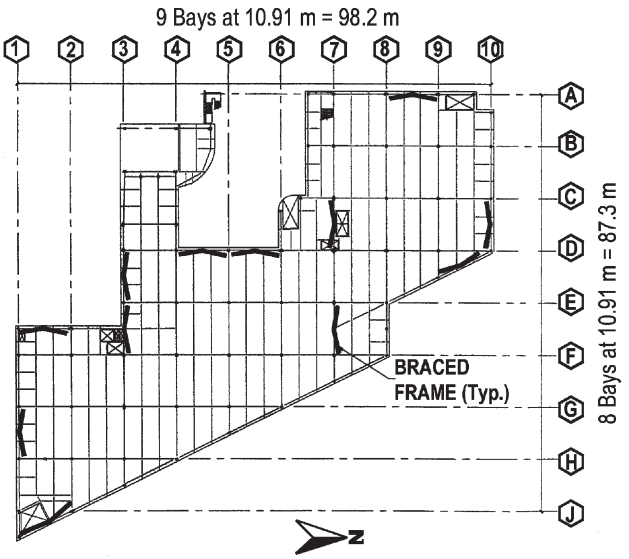

Fig. 2. Framing plan of the first floor of the study building [8].

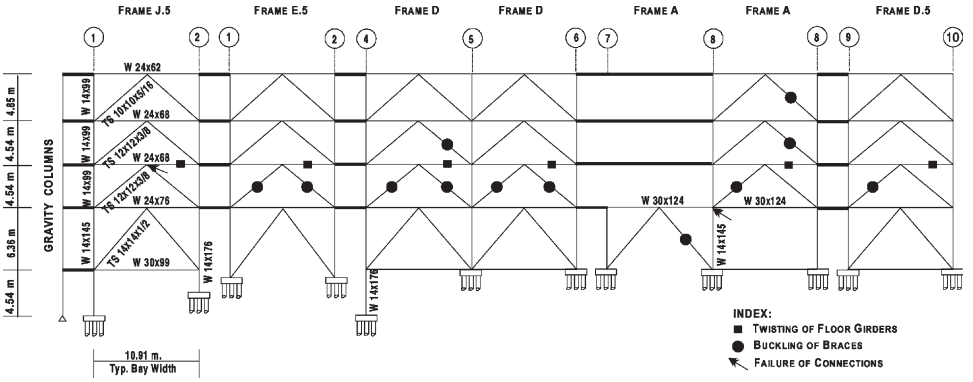

Fig. 3. Elevation of all six lateral frames of the study building in the N–S direction and location of observed damage during the Northridge earthquake.

4. Observed damage during Northridge Earthquake (1994)

The main findings of the post-earthquake damage investigation can be summarized in the following [8]:

1. Structural damage was observed in the lateral frames oriented along the N–S direction, consistent with earthquake damage observed in other buildings in the vicinity.
2. In damaged lateral frames, the failure was concentrated in the *second* story braces. Considerably less damage was observed in the other stories as shown in Fig. 3.

3. The main types of structural failure in the braced frames were: buckling, and fracture of tubular bracing members and some failure of brace connections. Typical failure patterns are shown in Fig. 4. Large width-thickness ratio of the tubular braces initiated the local buckling and instability following the global buckling of the braces, which led to fractures under several cyclic reversals of plastic deformations. The brace connection failures occurred due to high rotational ductility demand on the gusset plates when the braces were buckling out-of-plane. Floor beams were also laterally displaced above these buckled braces as seen in Fig. 4(c), causing considerable ceiling damage.
4. Damage to non-structural components was limited primarily to ceilings and mechanical systems in the penthouse. Ceiling damage was extensive in the upper stories above the buckled braces of the second story. However, little damage occurred in the exterior windowpanes, and the building in general was in plumb and posted with a yellow tag (i.e., limited entry but not continued occupancy) immediately after the earthquake.

5. Seismic evaluation and prediction of earthquake damage

5.1. Snap-2DX structural modeling

SNAP-2DX [9] program formulation is based on a member-to-member modeling approach, i.e., one-to-one correspondence exists between the elements of the model and members of the structure. The element force-deformation properties are specified in the form of hysteretic response expressed in terms of a force-deformation pair controlling the behavior of the element. For example, moment and rotation are the force and deformation quantities, respectively, used to describe the hysteretic behavior of the beam element.

The structural frame was discretized into two element groups: beam-column and bracing elements. Nominal yield moment capacities were used to describe the yield

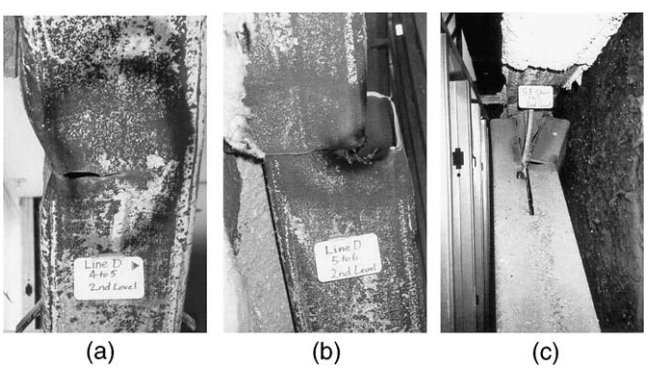

Fig. 4. Typical examples of observed damage: (a) Buckled brace, (b) Fractured brace, and (c) Failed brace to girder connection and lateral displacement of girder [8].

surface of the beam-column elements. The buckling element uses a hysteretic model for the post-buckling behavior, which allows for pinching of hysteretic loops and a reduced post-buckling capacity along with a criterion for fracture life of the braces [10,11]. An element is removed from the model when its fracture life is exceeded. The fracture life is expressed in terms of standard cycles N_f, of axial deformations and for tubular members is given below [11]:

$$N_f = \begin{cases} C_s\left(\dfrac{(b/d)(kl/r)}{\{(b-2t)/t\}^2}\right) & \dfrac{kl}{r} > 60 \\ C_s\left(\dfrac{(b/d)(60)}{\{(b-2t)/t\}^2}\right) & \dfrac{kl}{r} \leq 60 \end{cases} \tag{1}$$

where C_s is a numerical constant equal to 262 obtained from experiments, d is gross depth, b is gross width ($b \geq d$), and t is wall thickness of the section, and kl/r is the global slenderness ratio of the bracing member.

Some simplifying assumptions were made to obtain an idealized mathematical model for elastic/inelastic analyses. Despite some skewness of frames on the column line J.5 and D.5 with respect to the principal N–S axis, it is assumed that all six braced frames resist the lateral loads in the N–S direction. Rigid diaphragm action is assumed, which allows the braced bays to be connected by rigid links such that each story deflects laterally as a single unit. All gravity columns are lumped into a "super" column, which represents the stiffness and flexural capacities of gravity columns (also referred as "secondary" columns for lateral resistance) and is connected to braced frames by rigid links. The stiffness and yield properties of the secondary columns are summarized in Table 1. Masses were lumped at nodes at each floor level for dynamic analyses. The definition of gravity loads in Ref. [1] was used for the calculation of seismic loads and the corresponding reactive masses were computed as 2720, 2472, 2363, and 2677 Mg for the first, second, third floors and roof, respectively.

5.2. Elastic analyses

5.2.1. Response spectrum analysis: estimate of seismic demand

An estimate of the seismic demand on the structure in the event of a design level earthquake was obtained by a response spectrum analysis (RSA) of the analytical

Table 1
Properties of secondary columns

Story	No. of columns	ΣA (cm^2)	ΣI (Cm^4)	ΣM_P (kN m)	ΣP_Y (MN)
First	54	12,277	3,093,973	56,240	368
Second	54	10,155	2,502,591	38,460	254
Third	45	8232	2,019,014	30,830	204
Fourth	43	7890	1,935,851	29,555	196

Table 2
Modal story shears and demand/capacity ratio

Floor level	Factored story shear from RSA[a] (kN)	Estimated story capacity (kN)	Demand/capacity ratio (DCR)
Roof	6582	15,086	0.44
Third	10,840	23,140	0.47
Second	13,911	23,140	0.60
First	15,753	30,216	0.52

[a] A load factor of 1.5 is used over UBC base shear.

model of the building according to the UBC 94 [1]. The demand capacity ratio (DCR) is then obtained by dividing the ultimate modal story shear by the estimated story shear capacity of the frame as shown in Table 2. Only the contribution of bracing members was considered in calculating the shear capacity of a story using tensile yield strength (P_y) for the brace in tension, and the average post-buckling strength (0.55 P_{br}) for the brace in compression [10].

It is clear that the DCR is highest for the second story where the majority of brace damage was observed. However, for all levels, the DCR is less than unity and one can assume a safe behavior for the structure in design level earthquakes. The vulnerability of this structure as evidenced in the Northridge (1994) earthquake is due primarily to the usage of non-ductile bracing members, which underscores the need for enhancing the ductility of bracing members rather than their strength.

5.2.2. Comparison of 2-D and 3-D models

The results of the above analysis of the 2-D model can be compared to those from a 3-D model, which used the ETABS computer analysis program [12]. The fundamental period of the 2-D model (0.72 s) falls between periods of 0.84 and 0.63 s of similar translational modes in the 3-D model. The second mode in the 3-D model is the first torsional mode in which all floors rotate in the same direction. Obviously, this effect can't be simulated in the 2-D model. The fourth and fifth modes in the 3-D model are similar to the second mode in the 2-D model, with periods 0.29 and 0.22 s comparing well to 0.27 s of the 2-D model. The story shear values from the 2-D and the 3-D model are very similar to those from the UBC 94 (Table 3), which

Table 3
Modal story shears from 2D and 3D-models

Floor level	Story shear (kN) 2D-Model	3D-Model	ELF (UBC)
Roof	4388	4441	4223
Third	7227	7031	7062
Second	9274	8998	9154
First	10,502	10,502	10,502

indicates the dominance of the fundamental mode in both models. These comparisons suggest that the 2-D model adequately captures the dynamic characteristics of the building, especially the fundamental mode behavior, which is the most dominant mode for the building. It should be noted that the discussion is limited only to the behavior of models in the elastic regime. However, it can be projected that the 2-D model would be adequate to study the inelastic behavior of lateral frames.

5.3. Inelastic analyses

5.3.1. Pushover analysis

The objective of the static pushover analysis was to determine the lateral capacity of the structural frame, the failure mechanism, and the sequence of inelastic response events leading to near collapse. The 2-D model of the lateral frames in the N–S direction was subjected to the inverted triangular distribution of lateral loads of Ref. [1], as given in Table 3.

The base shear-roof displacement response of the 2-D model is shown in Fig. 5(a). The response is identified by five major events, each representing the occurrence of certain inelastic action in the frame and is illustrated in Fig. 5(b). Inelastic action began with the buckling of braces in the second story with significant loss of strength and stiffness. The next drop in strength and stiffness is noted when braces in the upper stories start buckling. The lateral strength of the frame increases with further redistribution of loads but drops again at the occurrence of plastic hinging of floor beams where chevron braces intersect. In a given story, with the compression braces buckled and the load carrying capacity reduced, the unbalanced vertical loads cause additional flexural demands on the floor beams. This unbalanced force is large enough to cause plastic hinging in the floor beams of lateral frames. Some column hinging in the second story is also observed as resistance demands shift to the columns.

The distribution of floor displacement, story drift, and story shear over the height of the frame is presented in Fig. 5(c) as they change through various stages of the incremental lateral loading. As can be seen, concentration of damage first began in the second story and then progressed to the first story followed by the third story. Deflection responses increased rather rapidly between stages 3 and 4 when plastic hinges in the floor beams were developing.

The shear forces resisted by the columns of lateral frame and gravity frames at various stages of loading are shown in Fig. 5(d). The contribution of columns to total shear is negligible when the structure is elastic, but it increases as the braces buckle, especially for the second and third story columns. The columns carry about 30% of the total shear following the brace buckling in a given story. Lateral and gravity columns as “non-considered” and redundant elements in CBF system can participate in a significant manner in resisting lateral loads following the strength loss due to brace buckling. This inherent characteristic of CBF system should be recognized in the design process.

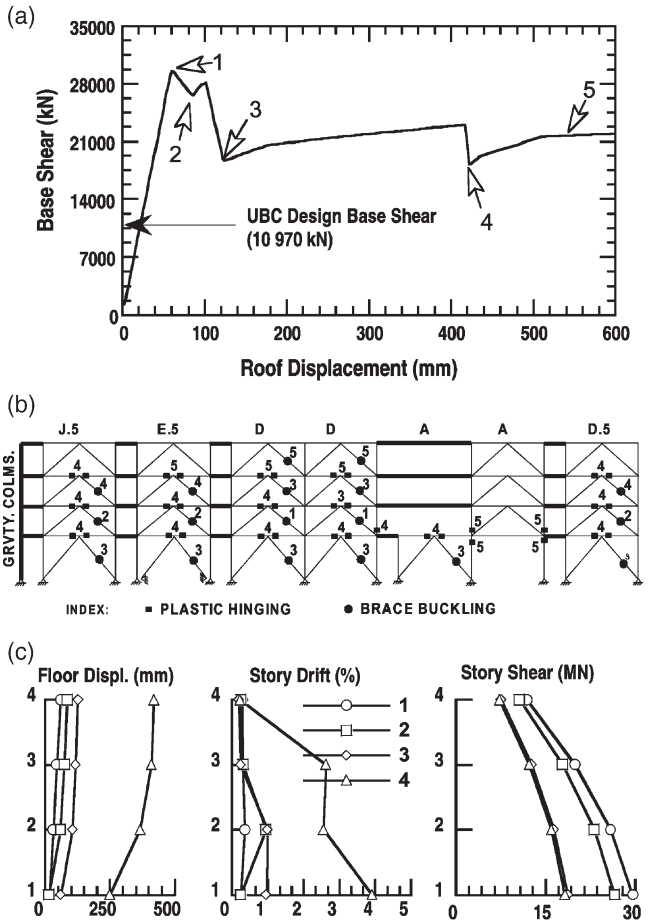

Fig. 5. Pushover analysis of 2-D model (a) Load-displacement curve (b) Force and deformation quantities at various stages of incremental loading (c) Location and sequence of inelastic activities and (d) Variation of story shears resisted by columns at various stages.

5.3.2. Time history analyses

The acceleration time record, used as base excitation in the time-history analysis came from the basement of a 20 story building (CSMIP Station C083), about 1.2 km from the site of the study building [13]. This response spectrum of the motion is completely enveloped by the UBC design spectrum for the site soil profile S_2 as shown in Fig. 6(a). The spectral energy peaks at period of 0.72 s, which is close to the fundamental period (0.75 s) of the 2-D model.

A reasonable agreement between the predicted inelastic activities (Fig. 6(b)) and the observed damage (Fig. 3) was obtained. In the northern part of the building, the first story columns of the braced frame A were subjected to large shear forces in the absence of braces and developed plastic hinges. All frames throughout the model

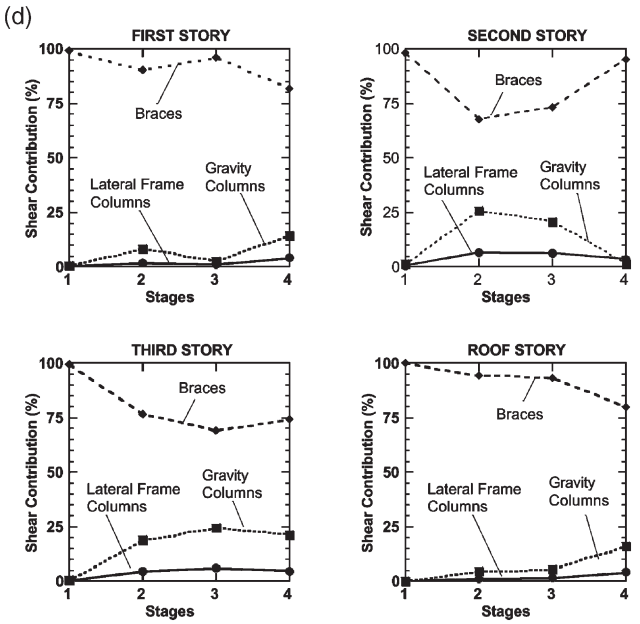

Fig. 5. *Continued*

suffered buckling and fracture of braces in the second story, as observed. The model also predicts damage in the second floor beams, which agrees with the observed twisting of beams, and extensive ceiling damage. The model also predicts buckling of braces in the first and the upper stories, however, such damage was only observed in the frame A. The analysis clearly shows the vulnerability of the upper stories.

The hysteretic responses, in terms of story shear and drifts, for all stories are shown in Fig. 6(c). It is clear that the first story behaved nearly elastically despite the buckling of braces indicating that inelastic deformations were small, whereas all upper stories experienced large inelastic excursions and suffered permanent lateral displacement. Significant reduction in the stiffness of the second story is obvious following the brace buckling and beam hinging. The story drift is concentrated in all stories above the first story, which varies from 1.43% for the second story to 0.67% for the top story. The base shear reached a maximum value of 31 897 kN, which is about three times the UBC design base shear and is close to the peak lateral resistance of 29 601 kN as obtained by the pushover analysis.

6. Post-earthquake capacity and behavior

The 2-D analytical model was subjected to two events of the North Hollywood accelerogram: two consecutive records of 0.31*g* North Hollywood, 30 s in length, and separated by 10 s of no excitation (i.e., free vibration) were used. This use of two records permits study of the system under a second similar ground motion,

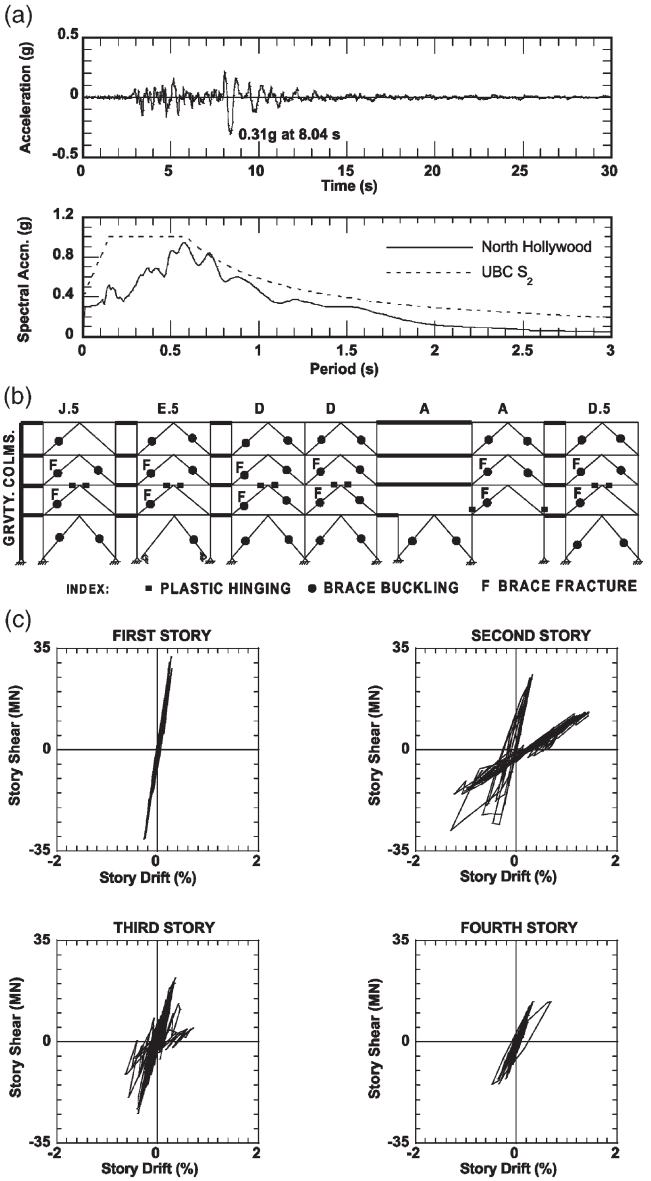

Fig. 6. (a) North Hollywood accelerogram (PGA = 0.31 *g*) and its acceleration spectrum compared with the UBC 94 S2 spectrum and (b) Predicted inelastic activities and (c) Hysteretic behavior of the N–S lateral frames.

assuming the building was left unrepaired after the first earthquake. The analysis results indicate the weakness of the structure in a repeat occurrence of similar ground motion and needed immediate repair. The structure, already weakened from the first event particularly in the second story, continued to experience further damage to its upper stories. The floor beams in the second and third stories developed plastic hinges in addition to fracture of braces. A significant 2.4 times increase in the story drift of the third story during the repeat record was observed. However, the second and third stories were prevented from developing a story mechanism by the columns (including the gravity columns), which maintained the integrity of the frame.

7. Seismic behavior under catastrophic ground motions

The North Hollywood accelerogram used for analyses has spectral acceleration characteristics similar to the UBC design spectrum for the periods of the structure. However, stronger ground motions can occur as evidenced during the Northridge earthquake. Predicting ground motion of maximum credible earthquake for a building site is fraught with uncertainty. Ground conditions, topography, basin edge effects and site location from the epicenter etc., for example, affect the earthquake record, duration and directionality in complex and even unpredictable ways. Two strong ground motions recorded during the Northridge earthquake in Newhall (CSMIP Station 24279: Los Angeles County fire station) and Sylmar (CSMIP Station 24514: 6-story County hospital) are representative of possible severe ground motion which the structure could be subjected to if it were located in those areas. The location of these sites and the epicenter of the Northridge earthquake are shown in Fig. 1(b) [14].

The N–S component of ground motions, their acceleration and response spectra are shown in Fig. 7(a). The peak ground accelerations are 0.59 and 0.84 g for Newhall and Sylmar, respectively. The peak horizontal ground velocities are significantly large; 1.30 m/s for Sylmar and 0.94 m/s for Newhall. The spectral acceleration of these motions is well above the UBC design spectrum for all the periods. The spectral power of the Newhall motion peaks at a period of 0.69 s, which is very close to the fundamental period (0.72 s) of the undamaged structure.

For the Newhall motion, story mechanisms formed in the first and second stories. Most of the braces in these stories buckled and fractured, floor beams formed plastic hinges where the chevron braces meet, and columns formed plastic hinges throughout the second story, resulting in a complete collapse mechanism. The damage is more widespread for the Sylmar motion in which complete story mechanisms developed for all four stories and the whole structure formed a collapse mechanism as shown in Fig. 7(b).

7.1. Comparison with North Hollywood motion

The Newhall and Sylmar ground motions have significantly larger damage potential than the North Hollywood ground motion. In the North Hollywood motion damage was concentrated primarily in the second story braces, and the columns prevented

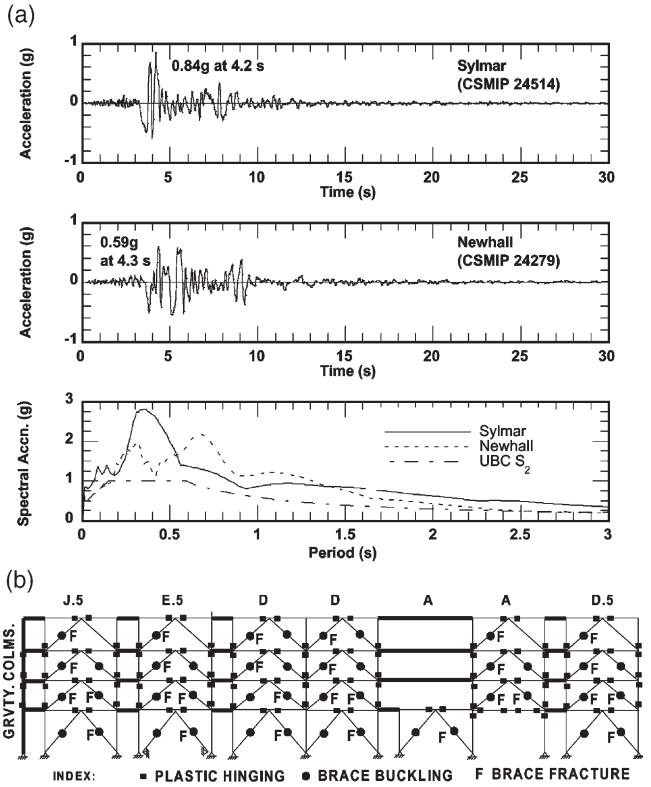

Fig. 7. (a) Characteristics of Newhall (CSMIP Station 24279) & Sylmar accelerograms (CSMIP Station 24514) and (b) Predicted inelastic activities in N-S lateral system for the Sylmar accelerogram.

the formation of a story mechanism. Under two consecutive applications of 0.32 *g* North Hollywood motion, damage spread in the upper stories but formation of story mechanism was avoided because of the columns. In the stronger Newhall motion, the frame was damaged very quickly during cycles of about 0.6 *g* acceleration, and a story mechanism developed in the first and second stories. For the strongest Sylmar motion, the structure was severely damaged during the first cycle of 0.8 *g* and then developed a complete collapse mechanism shortly thereafter.

8. Upgrading schemes and their effectiveness

Three upgrading schemes, which also involve different costs and levels of disruption to usage of the structure during the upgrading process, were studied for various levels of performance, from partial to "full" upgrading of the study building and other similar CBFs. The success and effectiveness of these schemes are evaluated with respect to the performance of the existing structure for North Hollywood and the more severe Sylmar ground motions.

8.1. Scheme 1: Increasing ductility of braces

The seismic performance of the lateral frames can be significantly improved if early fracture of brace members can be avoided. The tubular brace sections used in the study building were not compact enough and were susceptible to local buckling. The width-thickness ratio of bracing members varied from 26 to 30, which is about twice the recommended value of 14 (i.e., $250/\sqrt{F_y}$for A 500 Grade B steel material) according to UBC 1994 provisions for Special Concentric Braced Frames [1].

8.1.1. Concrete filled tubular bracing members

One inexpensive and simple way to reduce the effect of the width-thickness ratio of hollow tubular braces is to fill in plain concrete to increase their fracture life (ductility). The equivalent width-thickness ratio of such a concrete filled tube section is given in Ref. [15]:

$$\left(\frac{(b-2t)}{t}\right)_{equiv} = \begin{cases} \left\{\frac{(b-2t)}{t}\right\}(0.0082kl/r + 0.264) & 35 \le \frac{kl}{r} \le 90 \\ \left\{\frac{(b-2t)}{t}\right\} & \frac{kl}{r} > 90 \end{cases} \tag{2}$$

The fracture life of the concrete filled tubular members is calculated by replacing the width-thickness ratio in Eq. (1) with the equivalent web-thickness ratio given by Eq. (2). The fracture life of the braces of the study building increased about 1.32–1.76 times that of the hollow brace members.

8.2. Scheme 2: Changing brace configuration to 2-story x pattern

The seismic behavior of braced frames is influenced primarily by the following three factors: brace design and detailing, brace-to-beam or to-column connection, and brace configuration. While the first two factors are responsible for ductility at the member level, the last item is directly associated with the ductility and stability at the structure level. It is essential to utilize the concept of ductile braces in the design of all braced frames to improve the energy dissipation and fracture life of bracing members. However, the post-buckling response of such frames can be affected considerably by how the braces are arranged in a bay.

In chevron bracing, the post-buckling strength and stiffness are highly dependent on the stiffness and flexural capacity of the floor beam intersected by the braces [4]. The unbalanced vertical component of the force, due to significant reduction in the compression brace force after buckling, increases the flexural demand on the floor beam significantly. The lateral resistance is severely undermined if the plastic hinge forms in the beam. A 2-story X configuration minimizes (or even eliminates) the problem associated with floor beams in a chevron system as lateral loads are transferred to other braces after a brace buckles.

8.3. Scheme 3: Upgrading to SCBF

In the SCBF system, floor beams are designed for the unbalanced vertical force of braces after buckling. The resulting frame is a *strong beam–weak brace* system, which retains the advantages of the chevron configuration. For such a system, a higher value of response modification factor R_w can be used in calculating the UBC design base shear. A value of 10 is used which means a reduction of 25% in design forces of the existing OCBF design. The calculations for the design of braces and beams are tabulated in Table 4. For the design of floor beams, the flexural demand to resist the moment due to unbalanced vertical loads was added to the amount required for gravity load moments. The unbalanced vertical force P_{un} and the resulting moment M_{un} applied to the floor beam at the intersection of braces are obtained from the following relations:

$$\begin{aligned} P_{un} &= (P_y - 0.55P_{br})\sin\theta \\ M_{un} &= P_{un}\, L/4 \end{aligned} \tag{3}$$

where P_y and P_{br} are tensile yield and buckling capacity, respectively, of the brace which makes an angle θ with the horizontal, and L is the length of the simply supported floor beam. The term 0.55 P_{br} is used to represent an average value of the post-buckling capacity of tubular braces, Ref. [10].

Table 4
Design of braces and floor beams for upgrading to SCBF

Design of braces					
Story	Design story shear (kN)	Design brace force (kN)	Section size	Tensile yield capacity, P_y (kN)	Brace buckling capacity, P_{br} (kN)
First	1400	1081	TS 10 × 10 × 5/8	4584	2826
Second	1221	797	TS 8 × 8 × 5/8	2946	1722
Third	942	614	TS 7 × 7 × 1/2	2537	1210
Roof	564	374	TS 7 × 7 × 1/2	2537	1148
Design of floor beams					
Story	Plastic modulus for gravity loads (cm^3)	Unbalanced vertical load, P_{vn} (kN)	Moment due to P_{un} (kN m)	Plastic modulus required for unbalanced load (cm^3)	Section size
First	3277	2301	6309	25,400	W 40 × 397
Second	2901	1282	3515	14,158	W 36 × 256
Third	2901	1197	3285	13,224	W 36 × 245
Roof	2507	1264	3472	13,978	W 36 × 245

8.4. Comparison of pushover analyses of upgrading schemes

The inelastic (static) pushover analysis of the two upgrading schemes, i.e., changing to a 2-story X configuration and redesigning as SCBF, was conducted. The concrete filling of tubular braces enhances the fracture life measured in terms of cyclic inelastic excursions and its effect on load-deformation behavior can't be seen in a pushover analysis. The base shear versus roof displacement curves are shown in Fig. 8(a) and sequence of inelastic activities in frame members are shown in Fig. 8(b), for 2-Story X and SCBF upgraded frames. The 2-story X frame had a slightly lower stiffness than the original chevron frame due to the distribution of overturning forces in the columns. The first inelastic activity was noted in the second story of Frame D. The frame reached a maximum capacity of 32 929 kN at a roof displacement of 81.3 mm despite an early occurrence of inelastic actions. However, at this stage, the first major change in the strength and stiffness of the frame occurred when most of the braces in the second story reached their buckling and tension yield capacities

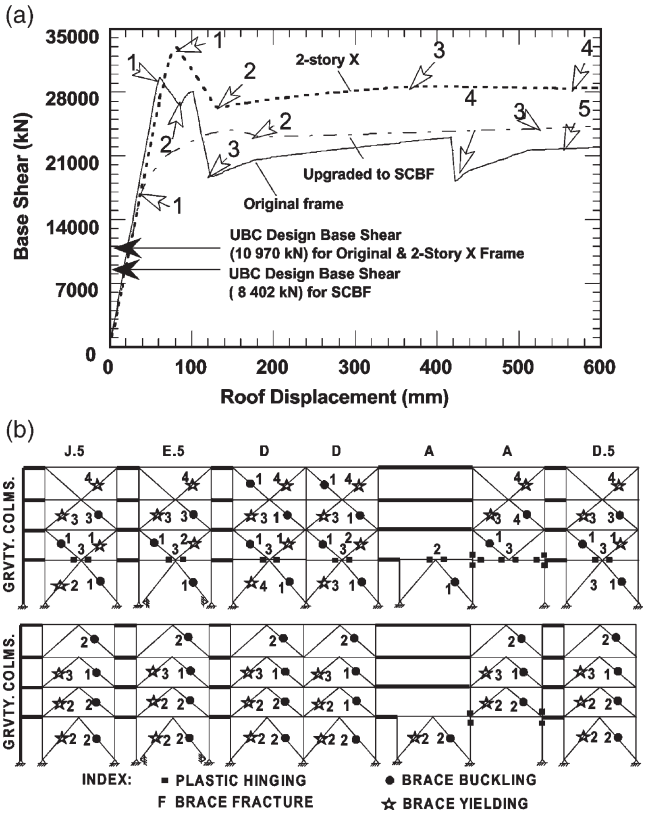

Fig. 8. Pushover analyses of upgrading schemes (a) Comparison of the base shear vs roof deflection curves (b) Sequence of inelastic activities in the lateral frames with braces arranged in the 2-Story X pattern and upgraded to SCBF.

and brace buckling occurred in the first and third stories. With further displacement, the lateral strength continued to decrease as the compression force of buckled braces decreased and reached the lowest value of 26 366 kN at a roof displacement of 132.1 mm. After that, redistribution of forces to the upper stories led to a gradual increase in the base shear, which reached a near constant value of 28 747 kN following the hinging of the second floor beams and yielding and buckling of the third story braces.

As seen in Fig. 8(b), for the special chevron frame, the inelastic action began with buckling of braces in the first story at a roof displacement of 38.1 mm and base shear of 16 752 kN. The frame continued to resist more lateral load, but with significantly reduced stiffness between events 1 and 2 as seen in Fig. 8(a). At the second event, the braces in the bottom three stories buckled and the first story braces reached their tension yield capacity. The load-displacement curve is nearly flat as all the braces have buckled and yielding of the upper story braces continued. At this stage, the frame can be considered to have reached its ultimate capacity of 23 550 kN, which is about 2.8 times its design base shear.

The 2-story X configuration reached a higher capacity than the original chevron frame due to additional strength derived from load redistribution after the first yielding. The plastic hinging of the second floor beams could not be prevented in this case because of unbalanced vertical forces after the buckling of dissimilar braces in the first and second stories. The special chevron behaved as a *weak brace–strong beam* system and inelastic activities were confined to the braces only.

8.5. Comparison of inelastic time history analyses of upgrading schemes

The fracture of braces was completely eliminated with concrete filled tubular braces for the North Hollywood accelerogram, which is in contrast to the fracture of several hollow braces in the second and third stories of frames as shown in Fig. 3. The overall spread of inelastic activity, in general, did not change and the second story received the maximum amount of damage. However, there was a general reduction in the ductility demands. For the more severe Sylmar ground motion, only one-third of all braces exceeded their fracture life and developed story mechanism only in the first story, unlike the hollow braces, which developed story mechanism in all stories. Plastic hinging of the first and second story floor beams and first story columns was also observed.

Significant improvements in the hysteretic behavior of frames were noted with concrete filled braces. For the North Hollywood motion, the absence of brace fractures clearly improved the hysteretic response of the bottom three stories, especially the second and third stories which suffered extensive brace fracturing with hollow braces, as can be seen by comparing Fig. 6(c) with Fig. 9(a). The response of the top story is very much unchanged as the brace ductility demands were not too high in this story. Similar trends of improved hysteretic response were observed for the upper stories for Sylmar (Fig. 9(b)) and Newhall ground motions. However, the first two stories developed story mechanisms and this upgrading scheme did not change the overall response in any significant manner for the severe ground motions.

In the second upgrading scheme, the braces were arranged in a 2-story X (or split

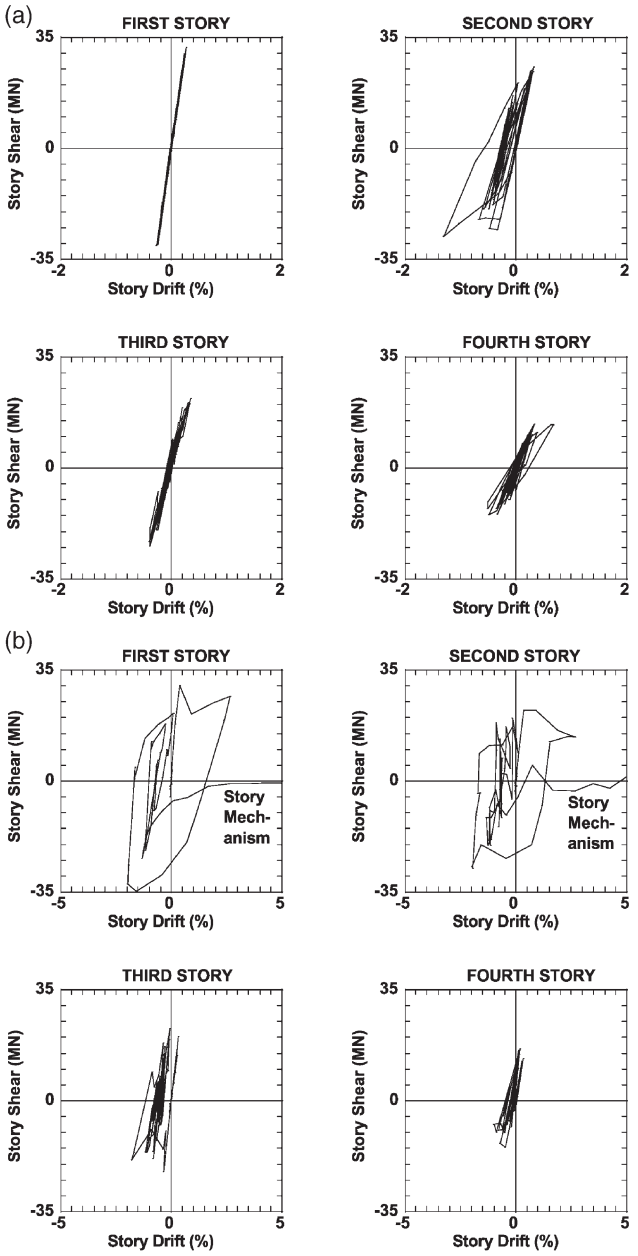

Fig. 9. Hysteretic behavior of the lateral frames with existing tubular braces filled with concrete for the (a) North Hollywood and (b) Sylmar accelerograms.

X) configuration. The same brace section sizes were used as in the study building; however, they were assumed to be filled with plain concrete for enhanced fracture lives. For the North Hollywood ground motion, the inelastic actions were concentrated primarily in the second story. The number of fractured braces was reduced to only two members in the second story. Some brace buckling was observed in the upper stories of Frames D and E.5, as well as in the first story of Frames J.5, A, and D.5. Plastic hinging of floor beams and columns did not occur. However, this could not be achieved for the Sylmar ground motion, as plastic hinges were observed in the first story floor beams and columns. The ductility demands of these plastic hinges were smaller, in general, than those with the chevron configuration.

Hysteretic behavior of the 2-story X lateral frames showed significant improvement over the chevron configuration, as shown in Fig. 10(a). For the Sylmar accelerogram, a stable hysteretic response was observed for the lower stories, and the upper stories behaved almost elastically except for buckling of a few braces. The superiority of the 2-story X configuration is obvious when comparing Fig. 10(a) with the hysteretic response of the chevron configuration shown in Fig. 9(b).

As expected with the lateral frames upgraded with a *strong beam–weak brace system* (SCBF), the inelastic activity was confined mainly to the braces, even for the Sylmar motion. A few plastic hinges were observed in the columns of the soft story of Frame A. The hysteretic behavior of SCBFs is shown in Fig. 10(b). The distribution of inelastic deformation and energy dissipation is consistent with those of properly designed regular frames with a dominant fundamental mode, i.e., maximum seismic demands for the first story, smaller demands for the upper stories, and minimum for the top story.

8.6. Relative merits and feasibility of various upgrading schemes

The first scheme focused on improving the fracture life (or ductility) of the braces, thereby, delaying (or even eliminating) the seismic demands imposed on the floor beams and columns due to early brace fractures. In the case of tubular braces, a popular choice for bracing members, this can be achieved by filling them with plain concrete. This relatively inexpensive and least disruptive of all upgrading schemes significantly improves the response to more frequent earthquakes of intensity comparable to that of the North Hollywood accelerogram. Methods to enhance the ductility and fracture life of bracing members should be employed with all upgrading schemes.

The second upgrading scheme utilizes a different configuration of bracing members to eliminate or minimize the formation of plastic hinges in the floor beams. Braces arranged in the 2-story X are considerably superior to the chevron system, as long as braces are ductile enough to prevent fracture. This scheme performed satisfactorily even for a severe ground motion such as the Sylmar. This upgrading scheme, along with methods to enhance the brace ductility, is very effective for ensuring an acceptable response to most seismic events. Although this scheme is somewhat disruptive to the occupants of the building during upgrading work, only half of all stories will be seriously affected.

The third upgrading scheme focused on changing the yield mechanism of an ordi-

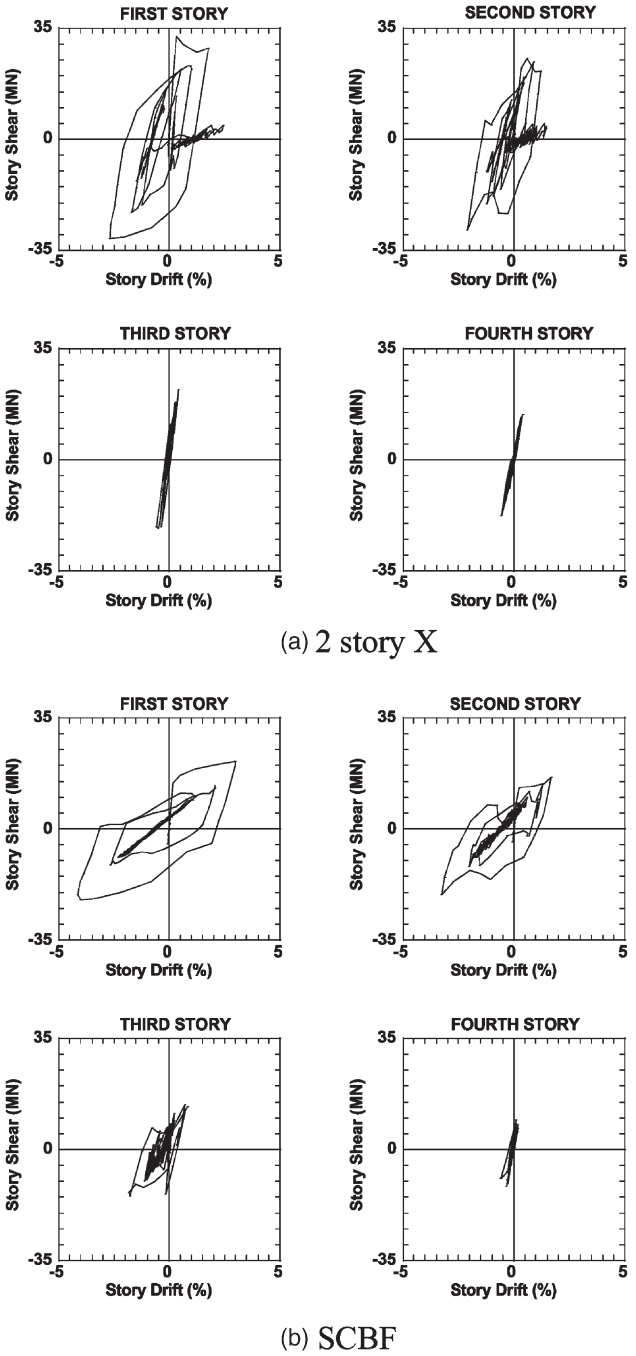

Fig. 10. Hysteretic behavior of the lateral frames (a) with braces arranged in the 2-story X pattern and (b) upgraded to SCBF for the Sylmar accelerogram.

nary chevron braced structure, which can be characterized as a strong brace and weak beam system. The special chevron system is designed as a weak brace and strong beam system. The performance of such a system to both the North Hollywood and Sylmar ground motions was extremely good. The inelastic demands were limited mostly to ductile braces, which dissipated most of the seismic energy. The story drifts and shears were distributed over the height of the frame as expected. Modifications to brace sizes as well as strengthening of the floor beams in braced frames are required for this upgrading scheme. The moment capacities of existing floor beams can also be increased by attaching cover plates at the top and/or bottom flanges of wide flange sections.

9. Conclusions

The paper presented an analytical study of evaluation of seismic behavior and upgrading of existing "ordinary" steel concentric braced frame (OCBF) structures for survival during future earthquakes. As a representative of this class of structures, a building in the North hollywood area which suffered major damage in the 1994 Northridge earthquake was the subject of this study. Several conclusions can be inferred from this analytical study:

1. The observed performance of the damaged study building can be adequately reproduced by the 2-D analyses, using the computer program SNAP-2DX and its element library. The program can be used effectively to assess the seismic demands and the performance of braced frame structures.
2. The study building, as well as other similar existing ordinary CBFs, have problems associated with their strong brace and weak beam design. However, CBF system has inherent redundancies in the form of the lateral and gravity columns, which can be substantial.
3. Inelastic dynamic analyses with the North Hollywood accelerogram showed that the study building could survive another seismic event of similar intensity without collapse, largely due to the participation of the columns. However, seismic demands and damage to the structure can be considerable, if left unrepaired. Therefore, the damaged study building should need immediate repair and upgrade after the earthquake.
4. Inelastic dynamic analyses for more severe strong ground motions indicated the inadequacy of the strong brace and weak beam design of existing OCBFs, which are likely to experience severe damage or even possible collapse.
5. The seismic performance of CBFs can be improved by delaying the fracture of braces. This is achieved by preventing the local buckling of brace members. In the case of the more popular tubular braces, fracture life can be increased, by filling with plain concrete. This should be the minimum level of upgrading for OCBFs with tubular bracing members.
6. The instability and plastic hinging of floor beams can be avoided by changing the popular chevron bracing configuration to 2-story X. If the fracture of braces

is prevented with the use of ductile braces, the of 2-story X configuration is a great improvement over the chevron system. Similar improvements can also be achieved by using a "Zipper Column" with chevron configuration [16].

7. Further improvement in the overall performance of OCBFs can be achieved by redesigning the brace and floor beams to a weak brace and strong beam system, as in SCBFs. This full upgrading to SCBFs results in excellent hysteretic response and, with inelastic actions confined to ductile braces, exhibits reasonable distribution of damage over the height of the building. This level of structural intervention is relatively intense, as it requires replacement of old braces as well as modification of floor beams.

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