13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3351

BRACING SYSTEMS FOR SEISMIC RETROFITTING OF STEEL FRAMES

Luigi DI SARNO[1] and Amr S. ELNASHAI[2]

SUMMARY

The present study assesses the seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems. Three brace configurations were utilized: special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). A 9-storey steel perimeter MRF was designed with lateral stiffness insufficient to satisfy code drift limitations in zones with high seismic hazards. The frame was then retrofitted with SCBFs, BRBFs and MBFs. Inelastic time-history analyses were carried out to assess the structural performance under earthquake ground motions. Local (member rotations) and global (inter-storey and roof drifts) deformations were employed to compare the inelastic response of the retrofitted frames. It is shown that MBFs are the most cost-effective bracing systems. Maximum storey drifts of MBFs are 70% lower than MRFs and about 50% lower than SCBFs. Configurations with buckling-restrained mega-braces possess seismic performance marginally superior to MBFs despite their greater weight. The amount of steel for structural elements and their connections in configurations with mega-braces is 20% lower than in SCBFs. This reduces the cost of construction and renders MBFs attractive for seismic retrofitting applications.

INTRODUCTION

Bracing is a very effective global upgrading strategy to enhance the global stiffness and strength of steel and composite frames [1]. It can increase the energy absorption of structures and/or decrease the demand imposed by earthquake loads. Structures with augmented energy dissipation may safely resist forces and deformations caused by strong ground motions. Generally, global modifications to the structural system are conceived such that the design demands, often denoted by target displacement, on the existing structural and non-structural components, are less than their capacities (Figure 1). Lower demands may reduce the risk of brittle failures in the structure and/or avoid the interruption of its functionality. The attainment of global structural ductility is achieved within the design capacity by forcing inelasticity to

[1] Department of Structural Analysis and Design, University of Naples, Federico II, Italy, disarno@unina.it
[2] Department of Civil and Environmental Engineering, University of Illinois, Urbana, IL 61801, USA.

occur within dissipative zones and ensuring that all other members and connections behave linearly.

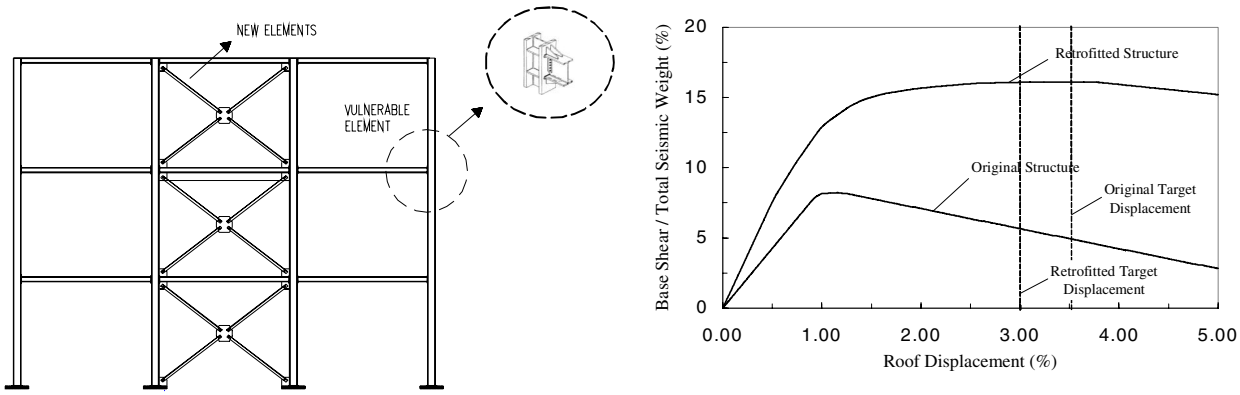

Figure 1. - Characteristics of global intervention approaches in seismic retrofitting.

Bracing may be inefficient if the braces are not adequately capacity-designed [2]. Braces can be aesthetically unpleasant where they change the original architectural features of the building [3, 4]. In addition, braces transmit very high actions to connections and foundations and these frequently need to be strengthened.

Several configurations of braced frames may be used for seismic rehabilitation. The most common are concentric braced frames (CBFs), eccentric braced frames (EBFs) and the novel knee-brace frames (KBFs), recently proposed for earthquake loads [5, 6]. The existence of tension/compression braces in CBFs results in a lateral stiffness well above that of MRFs. However, due to buckling of the compression members (*struts*) and material softening due to the Bauschinger effect, the hysteretic behaviour of CBFs is unreliable. It follows that the key to improving seismic behaviour depends on the scrupulous design of bracing members [7]. Common configurations for CBFs include V and inverted-V bracings, K, X and diagonal bracings. However, V bracings are not advised for retrofitting because of the likelihood of damage in the beam mid-span. Under horizontal forces the compressed braces may buckle, thus reducing their load bearing capacity abruptly. Conversely, the force in the tension braces increases monotonically reaching yield strength and eventually strain-hardening. The net result is an unbalanced force concentrated at the brace-to-beam connection. The effects in the beam, e.g., additional bending and shear, should be added to those due to gravity loads [8]. Alternatively, the unbalanced force in the beams may be eliminated through ad hoc bracing configurations such as macro-bracings, e.g., two, three storey X-bracings or V-bracings with a zipper column [9].

Macro-bracings can be utilised for strengthening and stiffening of steel and composite steel-concrete buildings. They are often employed to form MBFs, which exhibit high stiffness and enhanced ductility. Brace configurations with MBFs have been utilised in the present analytical work to retrofit a medium-rise steel MRF with inadequate lateral stiffness. Alternative systems, such as traditional SCBFs and frames with unbounded braces, have also been assessed. Unbounded braces are becoming very popular in Japan and the US for seismic retrofitting [10]. These braces are based on the same metallic yielding principle of added-damping-added-stiffness (ADAS) devices, i.e., tension/compression yielding brace. They consist of a core steel plate encased in a concrete filled steel tube. Yielding of the interior component under reversal

axial loads provides stable energy dissipation; the exterior concrete filled steel tube prevents local and member buckling. In this study a light-weight concrete with γ=1800 kg/m^3 has been assumed as filler material. A special coating is applied to reduce friction between steel and concrete. Since lateral and local buckling are prevented in unbounded braces, high energy dissipation is attainable. Several experimental tests have been carried out on this type of braces in the last decade [4, 11, 12]. They show compressive strength which is about 10-15% greater than tensile. These braces can reach cumulative cyclic inelastic deformations exceeding 300 times the initial yield deformation of the brace before failure. The latter depends on several factors, including material properties, local detailing, loading conditions and history. Unbounded braces are often fabricated with low yield steels, e.g., LYP100 and LYP235 with yield strengths (f_y) of 100 and 235 MPa, respectively. Configurations with low yield unbounded braces have also been considered in this study for comparisons; both SCBFs and BRBFs employ LYP235 braces.

The present work assesses the seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems. These include special concentrically braced frames (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). The inelastic seismic response has been quantified in terms of both local (member rotations) and global (inter-storey and roof drifts) deformation parameters derived by means of nonlinear time history analyses.

ANALYSED FRAMES

A medium-rise steel MRF was designed with inadequate lateral stiffness to fulfill code drift limitations for Los Angeles, California. This frame has 9-storeys and five 9.15 m bays. The height of the first storey is 5.49 m while all other storeys are 3.96 m high. At the first and second floors, beam span loads are equal to 14.88 kN/m and joint vertical loads are equal to 158 kN and 107 kN at interior and perimeter joints, respectively. Beam loads of 12.65 kN/m and joint vertical loads of 140 kN (interior joints) and 92 kN (exterior joints) were used at the roof. The slab employs composite metal deck (76.2mm thick), with 63.5mm of normal weight concrete. This system ensures rigid floor action. The total seismic load of the frame is W_{tot} = 45070 kN. Nominal yield strength equal to 345 MPa (50 ksi) was used for columns while girders have strength equal to 248 MPa (36 ksi). The MRF is considered to be a typical perimeter frame of multi-storey residential buildings. The seismic base shear (V_B) estimated through provisions in [13] is equal to V_B =5803 kN, corresponding to response modification factor $R=R_d \cdot R_0=8.50$. The period used to evaluate V_B is 1.28 seconds; it was computed as a function of the frame height (H=37.185m). The period derived by eigenvalue analysis is 2.05 seconds, about 60% higher than the approximated period. The reliability/redundancy factor (ρ) was assumed equal to 1.25 to account for the perimeter configuration of the MRF. The maximum displacement (d) of the frame was found at the first storey; the estimated maximum storey drift (d/h) is 3.96%. The latter exceeds the recommended drift provided by [13] for limit state of 'near collapse', i.e. 3.8%.

The MRF was retrofitted with bracing systems. The design target was the reduction of the drift d/h at first storey. Different configurations and brace types were utilised. Two layouts were selected to stiffen the MRF; these are SCBFs and MBFs as shown in Figure 2. Both braced configurations allow a reduction of 77% in the first storey to be achieved. Their fundamental periods are very similar: 1.08 (SCBF) *vs*. 1.01 (MBF). The period of the MRF is thus halved.

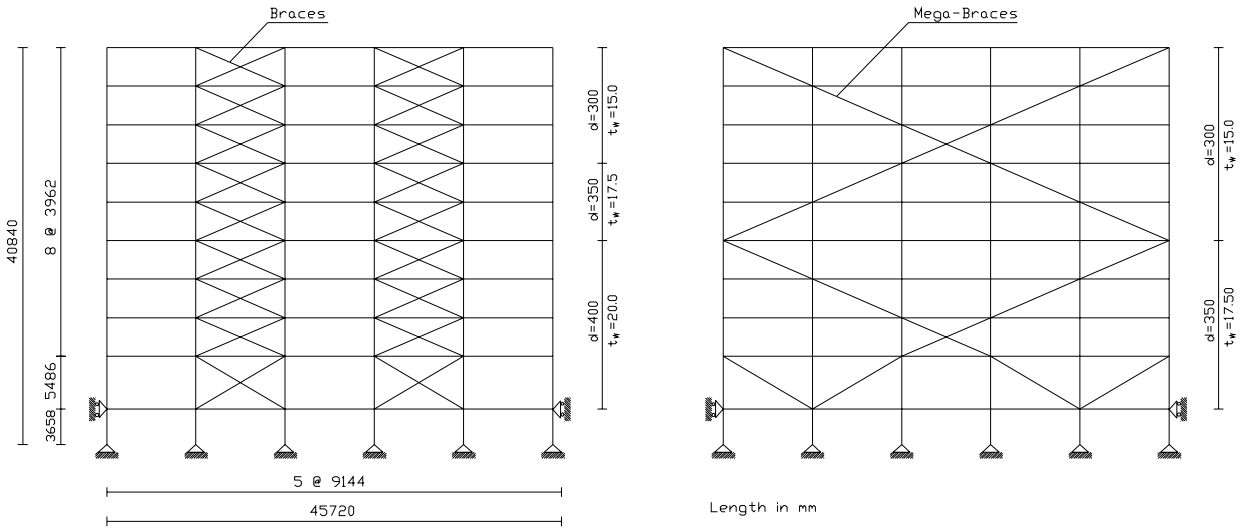

Figure 2. – Layout of braced frames: concentrically- (*left*) and mega-braced (*right*) frames.

In SCBFs, X-braces were placed in two central bays, while MBFs employ four-storey braces. Circular hollow sections were used in both configurations for the braces; their strength is equal to 248 MPa (36 ksi). In SCBFs, sections with diameters (d) of 400mm and wall thickness (t_w) of 20mm were used between the first and fourth floors, while hollow sections with d=350mm and t_w = 17.5mm were used at the fifth, sixth and seventh floors. Braces at the eighth and ninth floors have d=300mm and t_w= 15.0mm. Two sections were utilised for diagonals in the MBF: at the first four storeys, the hollow sections have d=350mm and t_w =17.5mm, while in the remaining d=300mm and t_w = 15.0mm. The design of the braces was carried out in compliance with the provisions in [14]. Local slenderness ratios d/t_w are 20; this value is nearly half of the limiting width-to-thickness ratio λ_p=35 recommended for A36. Bracing global slenderness ratios (kl/r) vary between 80 (lower floors) and 106 (upper floors); intermediate storeys have kl/r=85. Consequently, braces have intermediate slenderness. Comparisons between the above ratios and the limitations in the European standards [15, 16] show that the employed braces also satisfy strength and stiffness design requirements in these codes. For example, the maximum non-dimensionalised slenderness $\bar{\lambda}$ is 1.16, about 25% lower than the allowable value $\bar{\lambda}$=1.50.

Unbounded braces were also used to retrofit the sub-standard MRF; these braces were employed for both SCBFs and MBFs. They are made of two steel grades: A36 (f_y=248 MPa) and LYS235 (f_y=235 MPa). The sample frames assessed in this study are summarised in Table 1.

The combination of the different typologies (SCBFs and MBFs) and braces (ordinary and unbounded) gives rise to 8 different configurations.

Table 1. - Frames assessed in the present study.

FRAME LABEL	BRACE CONFIGURATION	BRACE TYPE
SCBF-N	Concentric	Ordinary
BRBF-CN	Concentric	Unbounded
SCBF-L	Concentric	Ordinary
BRBF-CL	Concentric	Unbounded
MBF-N	Mega	Ordinary
BRBF-MN	Mega	Unbounded
MBF-L	Mega	Ordinary
BRBF-ML	Mega	Unbounded

Inelastic response history analyses were carried out using DRAIN-2DX [17]. Bare frames were modelled by means of inelastic beam-column elements with lumped plasticity. A linearised bi-axial plastic domain was utilised to account for bending-axial interaction. Bilinear elasto-plastic behaviour with strain hardening of 1% was adopted to model plastic hinges. Inelastic truss elements were employed for diagonal braces. These were assumed to buckle elastically in conventional SCBFs while buckling is restrained for unbounded braces in BRBFs. Geometric non-linearities were included in all performed analyses.

GROUND MOTION RECORDS

Response-history analyses were carried out employing suites of ground motions developed for the FEMA-SAC steel project in the USA [18]. These earthquakes include horizontal records matching the 1997 NERHP design spectrum [19]. The selected ground accelerations correspond to the 1997 USGS hazard level for downtown Los Angeles. The seismological properties of the records used for this study are summarised in Table 2. Three levels of seismic hazard were employed: 50%, 10% and 2% probability of exceedence in a 50-year period.

Table 2. - Characteristics of ground motions.

RECORD	PR. OF EXC. (% in 50 yrs)	MAGNITUDE (M_w)	SOURCE DISTANCE (km)	PGA (g)	PGV (m/s)	PGD (cm)	ARIAS INTENSITY (m/s)	DURATION (s) UNIFORM	BRACKETED	SIGNIFICANT
Morgan Hill	50	6.2	15	0.32	0.32	6.14	1.71	23.70	39.44	22.64
Whittier	50	7.3	17	0.77	0.92	11.32	5.42	10.38	37.10	8.70
Loma Prieta	10	7.0	12.4	0.66	0.70	18.41	4.24	13.00	35.22	11.30
Landers	10	7.3	36	0.42	0.36	16.08	2.10	22.98	47.90	22.28
Northridge	2	6.7	7.5	0.43	0.65	12.21	2.03	11.18	14.82	7.80
Kobe	2	6.9	3.4	1.28	1.46	30.31	14.61	9.38	16.46	6.86

The distances from the sources for the records used to carry the inelastic analyses range between 3.4 km (Kobe, in Japan) and 36 km (Landers, in California). Therefore, the above suite of strong motions covers a range of design scenarios (near and far-field). In this study near- and far-field records were chosen to compare seismic performance during earthquakes with different frequency content as per Figure 3.

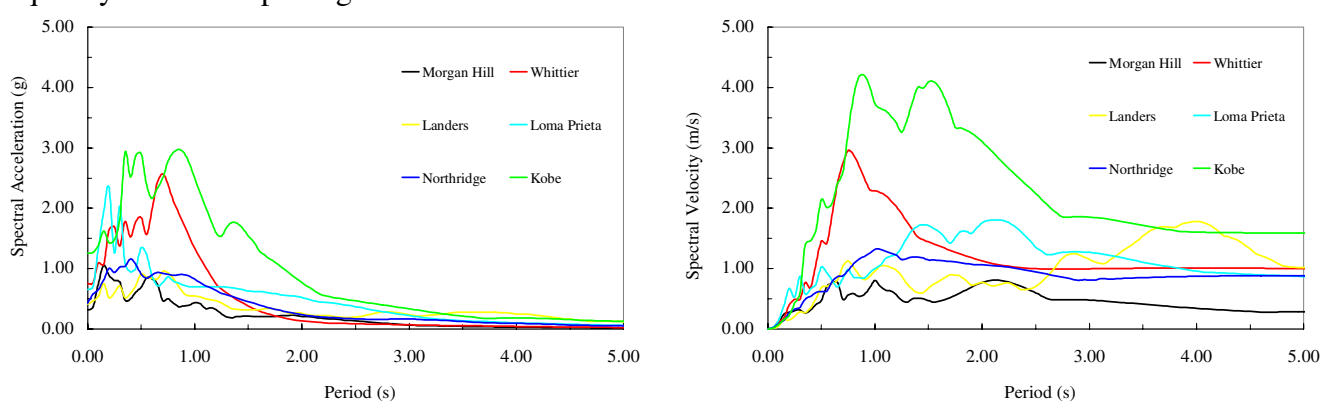

Figure 3. – Spectral accelerations (*left*) and spectral velocity (*right*) for the earthquake ground motions used (damping = 5%).

The values of the duration and energy content, expressed as ARIAS intensity, of the records summarized in Table 2 show that Kobe is the shortest record but the most demanding in terms of input energy.

INELASTIC PERFORMANCE

Inelastic response history analyses were carried out to assess the seismic performance of the configurations used to retrofit the MRF with insufficient lateral stiffness. The results of nonlinear analyses are summarised in the next sections in terms of local (member rotations) and global (storey and roof drifts) deformation parameters.

LOCAL DEFORMATIONS

The maximum plastic rotations experienced by the MRF under the sample earthquakes exceed the target plastic rotation (TPR) of 3.0% recommended by [20]. Values of 3.18% were found in beam elements when the frame was hit by Northridge (Figure 4). Retrofitting the MRF with mega-braces was found generally effective in reducing the maximum plastic beam plastic rotations. MFBs show lower rotations than SCBFs independently of the type of braces (either ordinary or buckling-restrained) and steel grade (either A36 or LYS235).

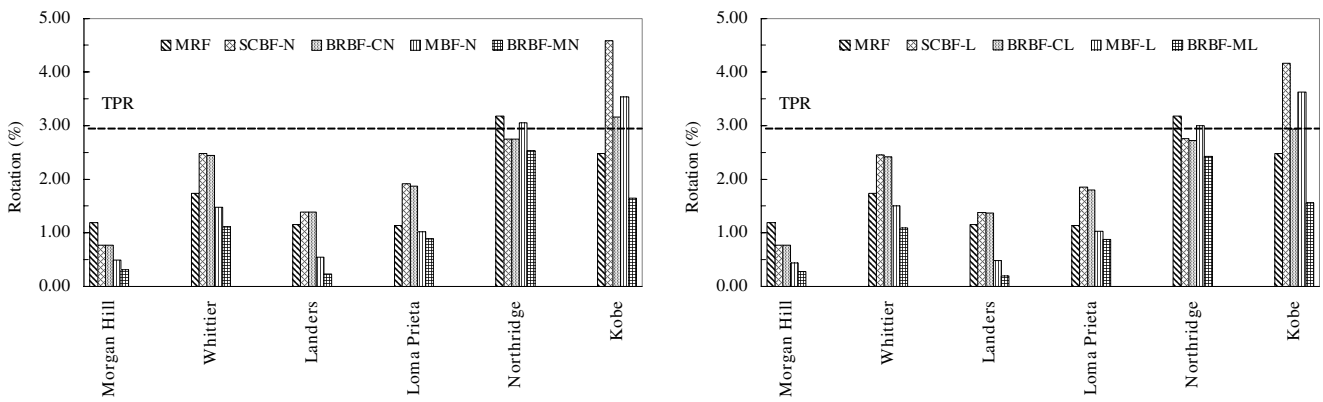

Figure 4. - Beam plastic rotations: grade A36 (*left*) and LYS235 (*right*).
Keys: TPR = Target plastic rotation.

Maximum beam rotations in MRF are significantly reduced in MBFs loaded by earthquakes with 2% and 10% probability of exceedence in 50 years. For Morgan Hill and Landers, reductions of more than 300% were computed; for Whittier and Loma Prieta the reductions were 20%. The use of buckling restrained diagonals leads to further reductions of 15-20%. Similar results were found when LYS235 was employed. The effects of lower yield steels are, however, less beneficial than the restraining of local and global buckling. Unbounded braces and MBFs are very effective in reducing the inelastic demands in columns when the Northridge earthquake is considered.

Elevated axial loads in columns and resonance at high frequencies cause column plastic rotations greater than 3.0% when Kobe earthquake is considered. Using unbounded braces is a viable means of reducing such rotations, provided that the structural configuration is with mega-braces (BRBFs).

GLOBAL DEFORMATIONS

The maximum inter-storey drift (d/h) computed for the MRF was found for the Northridge earthquake. Bracing is a viable solution to reduce this large drift, which exceeds the limit of 'near collapse' provided by [13], i.e. 3.8%. Maximum drifts for both SCBFs and MBFs are well below the 'life safety' limit (2.5%). The effect of low yield steel is negligible when compared to grade A36. The use of buckling restrained braces renders the drift d/h more uniform heightwise.

It is worth noting that MBFs exhibit lower drifts than SCBFs; the use of unbounded braces further reduces lateral displacements. In several cases, especially for earthquakes with probability of exceedence of 2% and 10%, it was found that inter-storey drifts in configurations with unbounded braces are 10% smaller than in MBFs. For Northridge and Kobe, however, this type of braces is proved to be much more effective; the computed drifts d/h are below 2.0% in all cases. Figures 5 and 6 provides the time-history response of the roof and first storey lateral drifts for the MRF and retrofitted systems under Morgan Hill and Loma Prieta earthquakes. Similarly, Figure 7 provides the results for the Kobe ground motion.

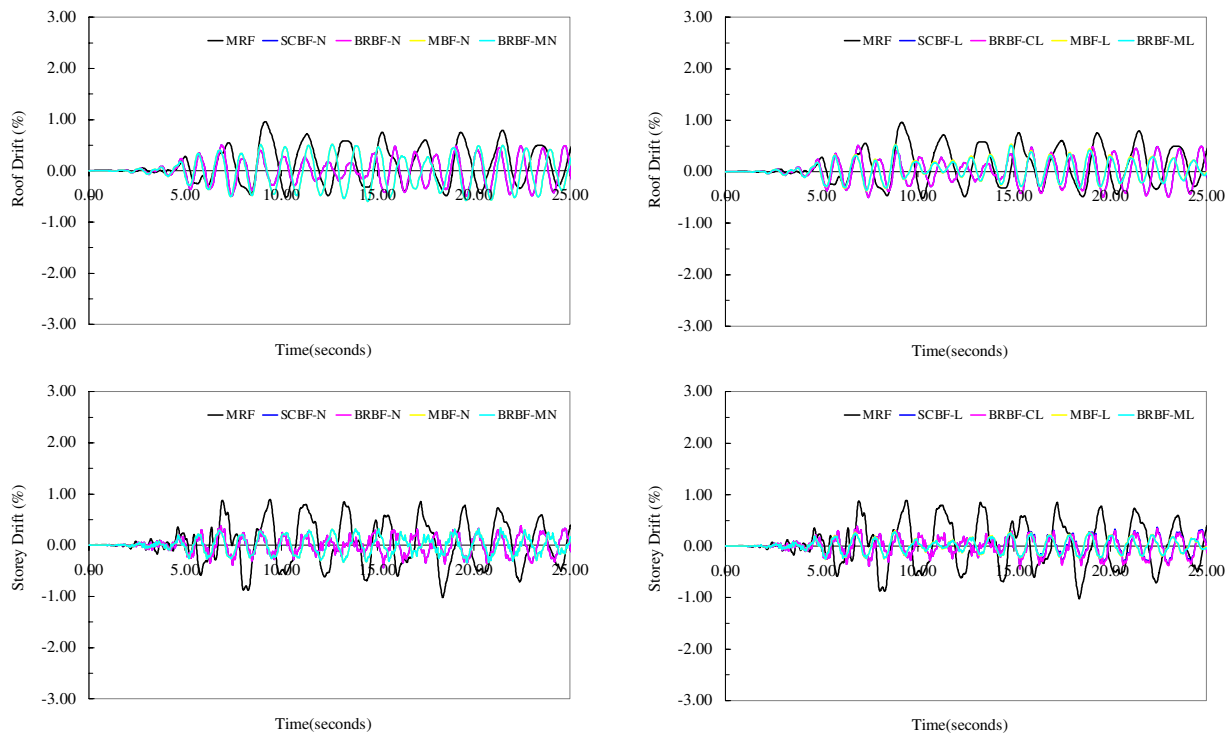

Figure 5. – Global deformations for Morgan Hill: roof drifts (*top*) and first storey drifts (*bottom*).

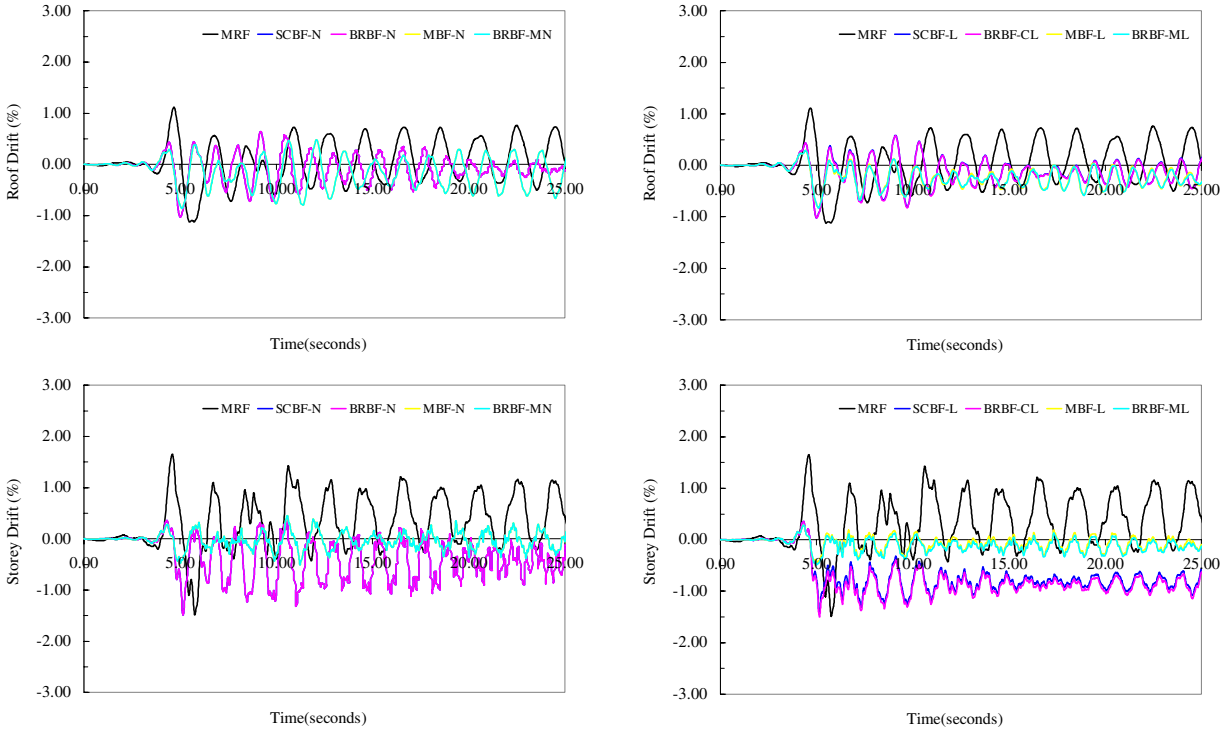

Figure 6. – Global deformations for Loma Prieta: roof drifts (*top*) and first storey drifts (*bottom*).

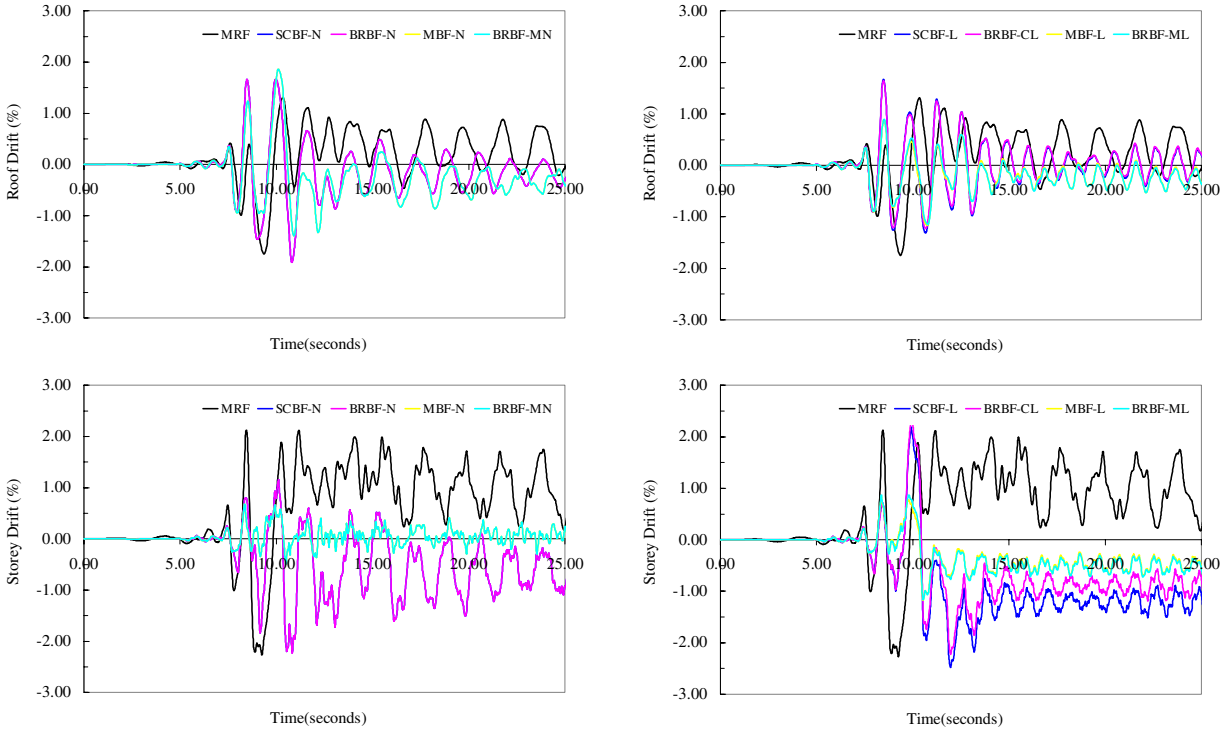

Figure 7. – Global deformations for Kobe: roof drifts (*top*) and first storey drifts (*bottom*).

Variations of maximum inter-storey drifts in the assessed frames are plotted in Figure 9 as a function of the weight of each configuration. The plotted values are mean values and were

computed using MRF as a benchmark. Configurations with mega-braces are the most cost-effective. They result in weight increase of 13.50% (MBF-N and MBF-L) and 18.45% (BRBF-MN and BRBF-ML). The corresponding reduction of inter-storey drifts with respect to the original MRF is on average equal to 70% especially for far-field earthquakes. This is also shown in Figure 9 where the variations of the maximum inter-storey drifts relative to Morgan Hill and Loma Prieta are provided; similar results were found for Landers and Whittier.

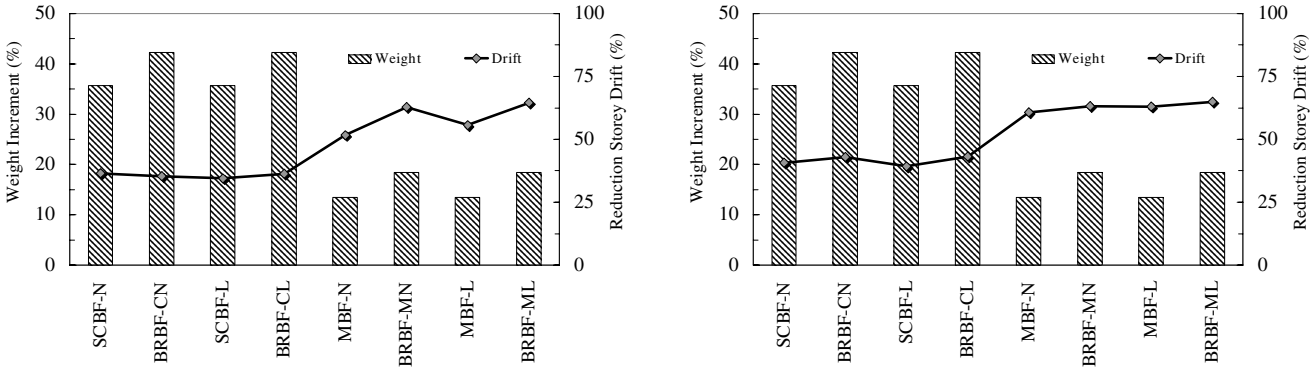

Figure 8. - Variation of maximum storey drifts: mean values with (*left*) and without (*right*) near-field records.
Keys: Values of MRF assumed as benchmark.

In addition, lateral drifts of MBFs are about 50% smaller than those relative to SCBFs as shown in Figures 8 and 9.

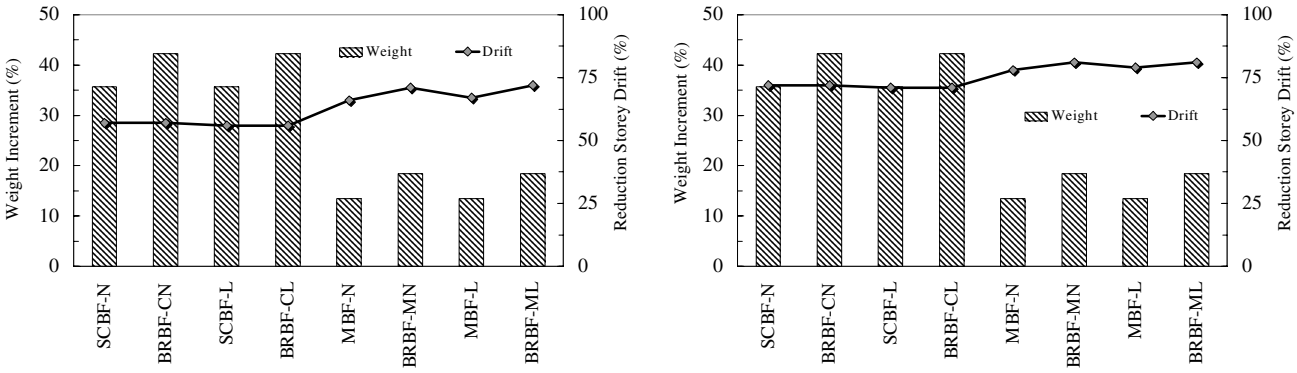

Figure 9. – Variation of maximum storey drifts: Morgan Hill (*left*) and Loma Prieta (*right*).
Keys: Values of MRF assumed as benchmark.

CONCLUSIONS

The present analytical work showed that moment resisting frames (MRFs) with insufficient lateral stiffness can be retrofitted with diagonal braces. The latter are a viable solution to augment both global lateral stiffness and strength of MRFs. In this study three configurations of bracing systems were assessed: special concentrically braced frames (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). The results of the performed inelastic analyses demonstrate that MBFs are the most cost-effective. The reduction of inter-storey drifts with respect to the original MRF is on average equal to 70%. Maximum lateral drifts in MBFs are 45-55% lower than SCBFs; the reductions in global deformations depend, however, on the

characteristics of earthquake ground motions, especially frequency content. For near-field records the benefits in using MBFs are generally lower than for far-field records. Systems retrofitted with BRBFs are only marginally superior to MBFs despite their greater weight (18.45 *vs*. 13.50). The total amount of structural steel in configurations with mega-braces is 20% lower than in SCBFs thus reducing the cost of construction. Finally, mega-braces can be installed without business interruption within the building.

ACKNOWLEDGEMENTS

This work was supported in part by the Earthquake Engineering Research Centers Program of the National Science Foundation under NSF Award Number EEC 97-01785 through the Mid-America Earthquake Center (University of Illinois at Urbana-Champaign, USA). Any opinions, findings and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the National Science Foundation.

REFERENCES

[1] Di Sarno, L. and Elnashai, A.S. (2002). Seismic retrofitting of steel and composite building structures. Mid-America Earthquake Center Report, *CD Release 02-01*, University of Illinois at Urbana-Champaign, IL, USA.

[2] Bruneau, M., Uang, C.M. and Whittaker, A. (1998). Ductile design of steel structures. McGraw-Hill, New York, USA.

[3] Tena-Colunga, A. and Vergara, A. (1997). Comparative study on the seismic retrofit of a mid-rise steel building: steel bracing vs. energy dissipation. *Earthquake Engineering and Structural Dynamics*, 26(6), 637-655.

[4] Iwata, M., Kato, T., Wada, A. (2000). Buckling-restrained braces as hysteretic dampers. *Proceedings of the 3rd International Conference on Behavior of Steel Structures in Seismic Areas* (STESSA 2000), Montreal, Canada, 33-38.

[5] Sam, M.T., Balendra, T. and Liaw, C.Y. (1995). Earthquake resistant steel frames with energy dissipating knee. *Engineering Structures*, 17(5), 334-343.

[6] Balendra, T., Yu, C.H. and Lee, F.L. (2001). An economical structural system for wind and earthquake loads. *Engineering Structures*, 23(5), 491-501.

[7] Goel, S.C. (1992). Cyclic Post-Buckling Behavior of Steel Bracing Members. In '*Stability and Ductility of Steel Structures Under Cyclic Loading*', Edited by Fukumoto, Y. and Lee, G.C., CRC Press, Boca Raton, 75-84.

[8] Goel, S.C. (1992). Earthquake Resistant Design of Ductile Braced Steel Structures. In '*Stability and Ductility of Steel Structures Under Cyclic Loading*', Edited by Fukumoto, Y. and Lee, G.C., CRC Press, Boca Raton, 297-308.

[9] Khatib, I., Mahin, S.A. and Pister, K.S. (1988). Seismic Behavior of Concentrically Braced Steel Frames. Earthquake Engineering Research Center, *Report No.UCB/EERC-8-01*, Berkeley, CA, USA.

[10] Housner, G.W., Bergman, L.A., Caughey, T.K., Chassiakos, A.G., Claus, R.O., Masri, S.F., Skelton, R.E., Soong, T.T., Spenser, B.F., Jr. and Yao, T.P. (1997). Structural control: past, present and future. *Journal of Engineering Mechanics*, ASCE, 123(9), 897-971.

[11] Watanabe, A., Hitomoi, Y., Saeki, E., Wada, A. and Fujimoto, M. (1988). Properties of braced encased in buckling-restraining concrete and steel tube. *Proceedings of the 9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan, Vol.IV, 719-724.

[12] Kasai, K., Fu, Y. and Watanabe, A. (1998). Passive control systems for seismic damage mitigation. *Journal of Structural Engineering*, ASCE, 124(5), 501-512.

[13] Seismology Committee Structural Engineers Association of California (1999). Recommended Lateral Force Requirements and Commentary. Seventh Edition. California.

[14] American Institute of Steel Construction (1997). Seismic Provisions for Structural Steel Buildings. Chicago, IL, USA.

[15] Eurocode 3. (1998). Design of steel structures. Part 1.1: General rules and rules for buildings. European Communities for Standardisation, Brussels, Belgium.

[16] Eurocode 8. (2002). Design provisions for earthquake resistance of structures. Part 1.3: General rules. Specific rules for various materials and elements. European Communities for Standardisation, Brussels, Belgium.
[17] Prakash, V., Powell, G.H. and Campbell,S. (1993). DRAIN-2DX. Base Program Description and User Guide. Version 1.10, University of California at Berkeley.
[18] Somerville, P., Smith, N., Punyamurthula, S. and Sun, J. (1997). Development of ground motion time histories for Phase-2 of the FEMA/SAC Steel Project. *Report No.SAC/BD-97-04*, Sacramento, CA, USA.
[19] Federal Emergency Management Agency (1997). NERPH Recommended Provisions for Seismic Regulations for New Buildings, *FEMA Report No.302*, Washington, D.C.
[20] Federal Emergency Management Agency (2000). State of Art Report on past performance of steel moment frame buildings in earthquakes. *Report No. FEMA 355E,* Washington, D.C., USA.