11 WCEE
Paper No. 506. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC STRENGTHENING OF AN RC SLAB-COLUMN FRAMES WITH DUCTILE STEEL BRACING

SUBHASH. C. GOEL and ADNAN C. MASRI

Department of Civil and Environmental Engineering, The University of Michigan,
Ann Arbor, MI 48109-2125, U.S.A.

ABSTRACT

This study is concerned with developing a rational design procedure for use of ductile steel bracing for strengthening existing seismically weak RC slab-column building structures. A one-third scale, two-bay, two-story RC slab-column frame model was selected to represent existing seismically inadequate structures of its type. The design procedure, construction and test results of the steel bracing system for strengthening the RC frame are presented in this paper. The strengthened frame was subjected to a combination of gravity and cyclic lateral loads up to 2% overall frame drifts. The behavior of the strengthened frame improved dramatically over that of the bare RC frame. A maximum 2.75% drift in the first story was reached which is highly probable during severe earthquake motions.

KEYWORDS

Seismic strengthening; reinforced concrete; slab-column frame; steel bracing.

INTRODUCTION

Steel bracing systems are very effective and attractive for strengthening of seismically "weak" structures. Currently, there is a lack of rational provisions for analyzing, designing and detailing these systems. The main unknown is how the new bracing system will interact and behave when attached to an existing structure. Following an earlier study by Jones and Jirsa (1986), Lee and Goel (1990) conducted an experimental research on a two-thirds scale, one-bay, two-story reinforced concrete (RC) beam-column frame model strengthened by a chevron bracing system. One of their main findings was flexural contribution of the added vertical and horizontal steel elements to the overall lateral strength of the frame. This contribution is neither recognized in current design practice nor was it considered in the above study.

A two-story, two-bay RC slab-column frame at one-third of full scale was constructed for the present study. The RC frame represents older seismically inadequate structures. It was designed according to the early 1960's code requirements. An elevation view of the RC frame model is shown in Fig. 1. The slabs of this frame were loaded by sand bags to simulate gravity loads which produced punching shear stress of approximately 50 psi around the critical perimeter of an interior slab-to-column joint. The frame was subjected to a series of cyclic lateral displacements increasing in magnitude up to a maximum average building drift of 2%. The lateral load capacity of the RC frame was determined experimentally to be 20 kips. Details of the frame design and the test are described in the paper by Wight *et al.*, (1994).

The testing of the strengthened RC frame with ductile steel bracing system was carried out in two phases. In the first phase, a chevron steel bracing system was designed and used to strengthen the exterior bay of the RC frame. The braced frame was tested under a combination of cyclic lateral loads and gravity loads to investigate its behavior. In the second phase, a similar bracing system was used to strengthen the interior bay of the RC frame after removing the braces used in the exterior bay. This braced frame was tested under load conditions similar to those used in the previous test.

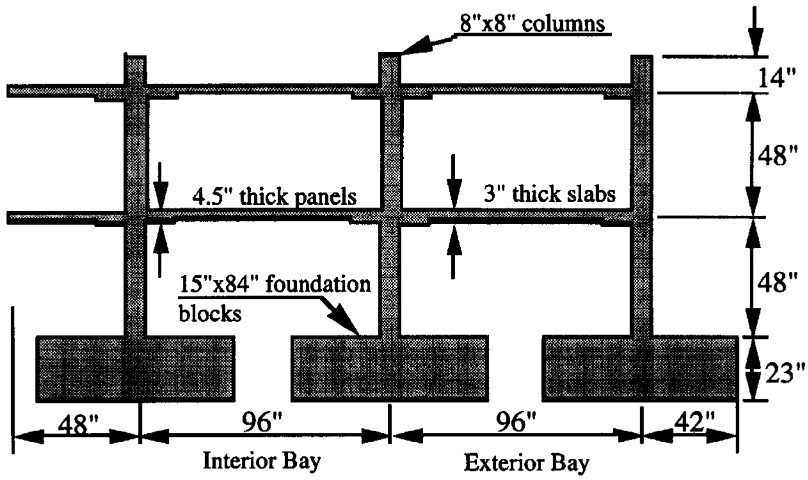

Fig. 1. Elevation view of the RC frame model.

ANALYSIS AND DESIGN OF THE FIRST STEEL BRACING SYSTEM

Preliminary Analysis

The approach used to analyze the first bracing system is described in this section. The strengthened frame is decomposed into two structural systems as shown in Fig. 2: the RC frame and the steel bracing system. The yield mechanism and the lateral load capacity, Q_{bf}, of the RC frame were computed analytically, where the frame mechanism governed and the value of Q_{bf} was 15.4 kips. The steel bracing system was designed for a lateral load capacity, Q_{bs}, equal to 70 kips and for the same yield mechanism as the RC frame (frame mechanism). The lateral loads for the braced frame were distributed in a ratio of 1 to 0.6 between the top and the bottom floors, respectively, in order to approximately simulate the first mode of vibration.

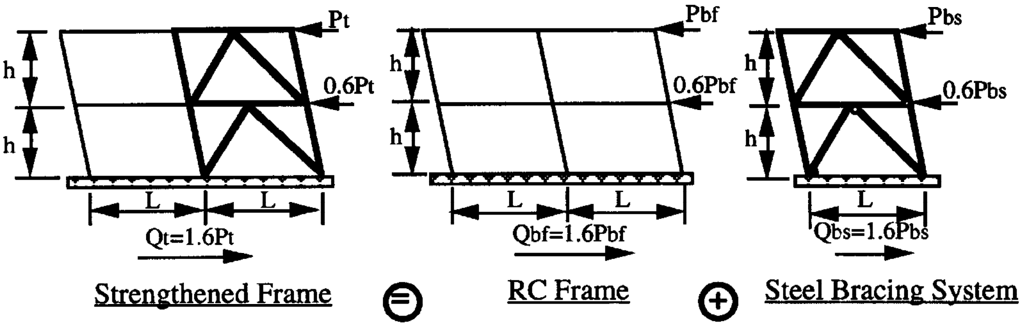

Fig. 2. Components of the strengthened frame.

The strength of the braces was limited by the capacity of the actuators, 100 kips each. A500 Grade B tube sections (2"x2"x1/8") were selected for the braces. The clear width-thickness ratio was 14 for ductile behavior as recommended by Lee and Goel (1987). The effective slenderness ratio of the braces was 61 with in-plane single gusset plates ($K_y = 1.0$). The tension yield load T and the buckling load C of the braces were determined using AISC-LRFD (1986) formulas without the Φ factors to be 41.5 kips and 33 kips, respectively. The post buckling load C_{pb} of the braces was taken as 0.33C.

Knowing the desired lateral load capacity of the steel bracing system Q_{bs}, the yield load of the braces T and the post buckling load of the braces C_{pb}, the yield mechanism is applied on the bracing system as shown in Fig. 3. M_{ph} and M_{pv} are the preliminary moment capacities of the horizontal and vertical steel elements, respectively.

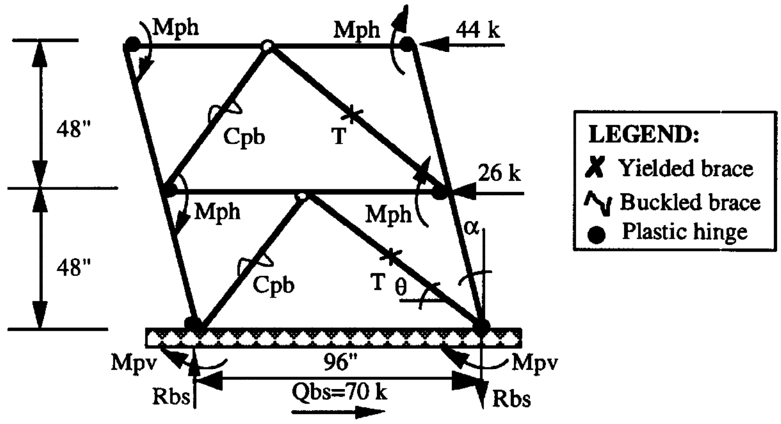

Fig. 3. The yield mechanism of the steel bracing system

Equating the external work done by the lateral loads to the internal work stored by the braces and the plastic hinges, the following relationship between M_{ph} and M_{pv} is obtained.

$$2M_{ph} + M_{pv} = 954 \qquad \text{(kip-inches)} \tag{1}$$

Moment equilibrium of the frame requires:

$$R_{bs} = 57 - M_{pv}/48 \qquad \text{(kips)} \tag{2}$$

In order to have plastic hinges are formed in the vertical elements at the foundation level only,

$$M_{pv} \geq M_{ph} \tag{3}$$

The values of M_{ph} and M_{pv} were determined from (1) to be 310 kip-in and 340 kip-in, respectively, assuming the ratio between M_{pv} and M_{ph} to be 1.1. Also, the value of R_{bs} was determined from (2) to be 50 kips. The ultimate forces acting on the steel bracing system are shown in Fig. 4. The steel bracing system was assumed to be a combination of two structural actions: a steel truss action formed by the braces and the vertical and horizontal steel elements, and a steel frame action formed by the vertical and horizontal steel elements with moment resisting connections. Subtracting the ultimate forces in the truss action from those of the bracing system, it is possible to determine the lateral forces of the steel frame action.

Design of Vertical Steel Elements

Angle sections were selected for vertical elements to wrap around each column and to tie them together by batten plates. The four angles were assumed to act as one unit section around the RC columns. The flexural capacity of this section is equal to the product of the yield force of two angles and the moment arm between the centroids of the two opposite angles. Assuming that the RC column would help resist the compression force, the vertical angles are designed for the ultimate bending moment (M_{pv}= 340 kip-in) and axial tension (F_{tv}= 20.65 kips) force due to the truss and frame actions. Four 1.5"x1.5"x3/16" angles were selected for vertical steel elements with the nominal moment capacity M_{nv} of the unit section formed by them being 255 kip-in.

Batten plates were used to tie the four angles around the corners of the RC columns and were designed so that the steel section formed by the four angles develops its yield capacity. The design forces of the battens are shown in Fig. 5. These forces are at the end sections of the column when the vertical angles develop their nominal yield capacity. An over-strength factor equal to 1.25 was used because no yielding was desired in the battens. Thus, each batten was designed for a shear force H'_{ba} and a moment M'_{ba} equal to:

$$H'_{ba} = 1.25(2M_{nv}/b)/N' \quad \text{and} \quad M'_{ba} = (H'_{ba}/b_{cl})/2 \tag{4}$$

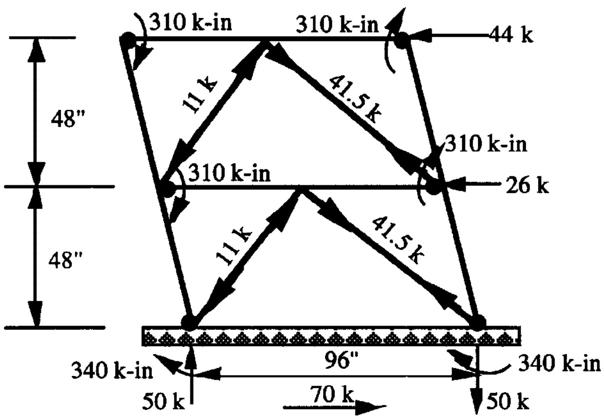

Fig. 4. Ultimate forces acting on the bracing system

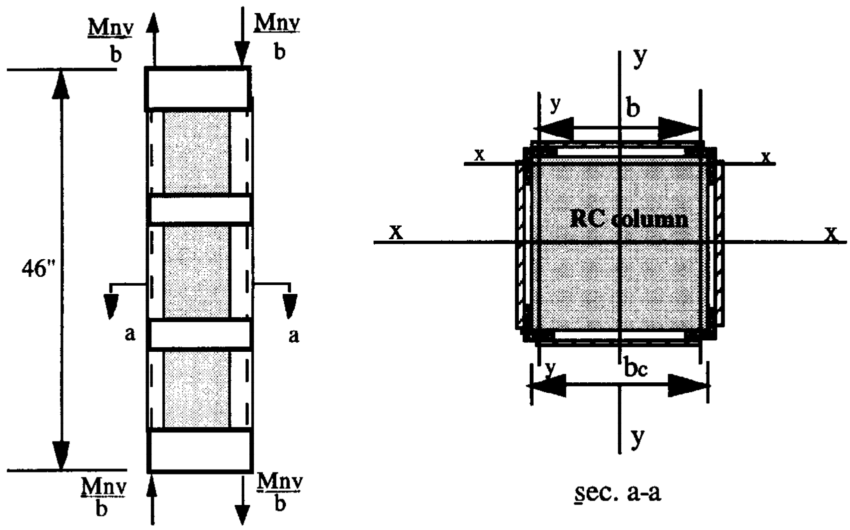

Fig. 5. Design forces of the battens.

where N' is the total number of battens on both sides of the column and b_{cl} is the clear distance between the two angles. The bending moment M'_{ba} governed the design of the battens and four 3"x5/16" batten plates were selected. Also, similar size plates were used on the other two faces of each column.

Design of Horizontal Steel Elements

Horizontal steel elements were selected to strengthen the top and bottom surfaces of the RC slabs. They were designed for the ultimate forces and moments at two critical sections: the support and the mid-span. . The vertical component of the unbalanced brace forces P_v, after one brace buckles, was included in the ultimate bending moment acting on the horizontal steel elements.

The top steel element was selected to have the same cross-section at all sections. Because of the drop panels, the bottom steel element was designed to have two different cross-sections. The first was designed for the forces at the support section and used to strengthen the bottom surface of the drop panel. The design of this bottom steel element was similar to that of the top steel element. Thus, the preliminary cross-section of this collector was selected to be a compact WT 3x4.5. The second section was designed for the forces at the mid-span section and it was used to strengthen the bottom surface of the slab between the drop panels. The ultimate axial tension force at the mid-span section was determined by:

$$F_{tm} = (P_v L)/(4t_s) + (P_h/2) \tag{5}$$

where t_s is the slab thickness at mid-span. Two Ls 4"x3"x1/2" were selected back-to-back with a 1/2" clear distance. The section was checked for compactness.

Threaded bolts as shear connectors were used to connect the top and the bottom horizontal elements together and to the RC slab. They were designed to allow the steel elements to develop their yield capacity. Figure 6 shows the forces that would be transferred by the bolts between the two support sections when the yield mechanism of the bracing system developed. The connectors were conservatively designed to resist the horizontal components of the brace forces at the slab-to-column connections in addition to the axial forces resulting from the moments. Thus, the bolts were designed for a total shear force H_{bo} given by:

$$H_{bo} = 1.25\,(2M_{nh}/h_c) + (T+C)\cos\theta \tag{6}$$

where h_c is the distance between the centroids of the top and the bottom horizontal steel elements at the column-face section. Twenty two, A329, 1/2" bolts were required, and were distributed in two rows. Six bolts were provided closely at mid-span section to resist the vertical component P_v of the braces unbalanced forces. The schematic of the first braced frame is shown in Fig. 7.

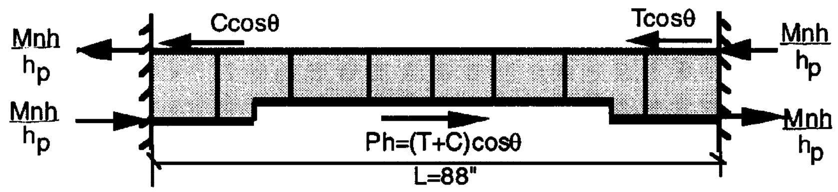

Fig. 6. Design forces of the shear connectors

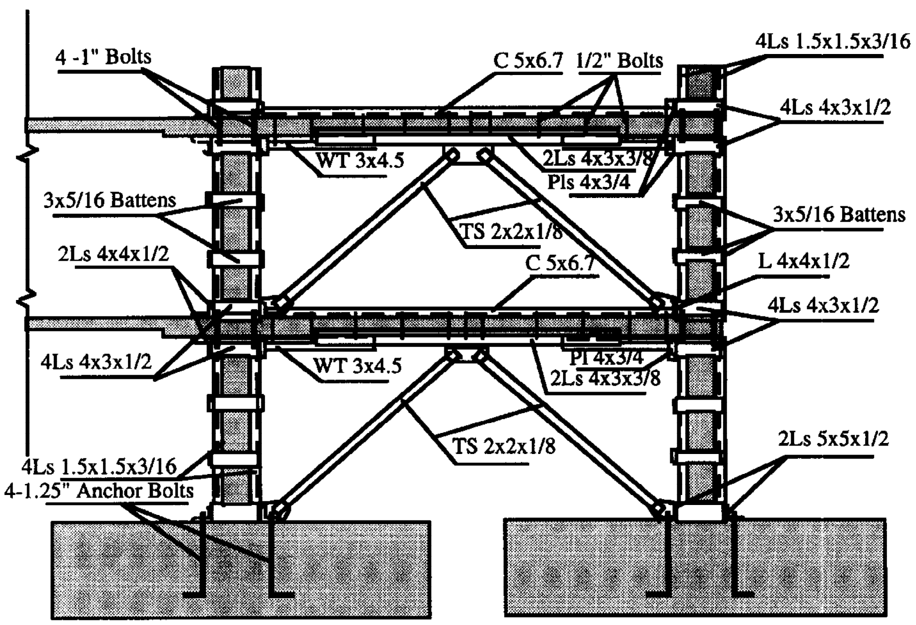

Fig. 7. Layout of the first braced frame

CONSTRUCTION OF THE FIRST STEEL BRACING SYSTEM

The first test on the bare RC frame damaged the RC slabs in regions around the exterior column. Also, fine flexural cracks were observed in other regions of the slabs and columns. The damage at the exterior connections was first repaired by epoxy grout to a limited extent. Then upper and lower horizontal steel elements (channels) were then attached to RC slabs by 1/2" bolts through already drilled holes in the slab. The holes were then sealed with epoxy grout. The vertical steel angles were wrapped around the two RC columns in each story and then the battens were welded to the angles at the four sides of each column. The continuity of vertical steel elements is provided by 1" diameter bolts through the slab at the corner of column connecting the collar angles placed at the top and underside of the slab. Next, 1/2" gusset plates and four 2"x2"x1/8" tube braces were welded to the steel elements, and strain gages were attached to the bracing system at various sections. To verify the steel frame action described earlier, the braced frame without the braces (partially strengthened frame) was subjected to same combination of gravity and lateral loads as the one used in the evaluation phase for the bare RC frame. The roof actuator was operated in a displacement control mode and the actuator at the first floor was slaved to 60% of the roof actuator force in order to simulate the first mode of deformation. Details of the tests are described in the original report by Masri and Goel, (1994).

TESTING OF THE SECOND BRACED FRAME

After testing the first braced frame, the tube braces were removed from the exterior bay while other steel elements were left in place. In the second bracing system the interior bay of the frame was strengthened in the same fashion as the exterior bay. However, the braces were made from two A36 1.75"x1.75"x1/8" angles welded together to form a box section which was then filled with plain concrete to further enhance their ductility.

The battened steel columns used in the previous strengthening schemes assumed that four angles with the battens around the RC columns behave as a unit section and the battens were designed for the axial yield capacity of the angles. However, it was observed that the number and spacing of battens had considerable influence on the behavior of column because of the significantly different stiffnesses of the battened angles from that of the unbattened ones along the column height. As a result, yielding in the vertical angles at the battened-angle connections was observed. Based on this observation, the new spacing of the battens was computed such that plastic hinges formed at the ends of each vertical angle between the battens.

Once again, four A36 1.5"x1.5"x3/16" angles were found adequate to strengthen the interior RC column. Five 2.5"x1/4" intermediate battens were provided at center-to-center spacing of 7". To improve the strength and stiffness of the strengthened middle column, three 2"x1/8" additional intermediate battens were added at each face of the column in each story.

Overall Behavior

In the first cycle, at a drift of 0.25% the unrepaired flexural cracks in the first-story RC columns and slabs reopened, however, the steel frame remained elastic. At a drift of 0.5% the first story east brace buckled out of plane. Yield lines appeared on the first story west brace also.

At a drift of 0.55% in cycle 5 the first story west brace buckled out of plane when the load was applied westward. Yield strains appeared in the vertical steel angles of the most interior column at the foundation level section. The curvature of the exterior strengthened column was obvious where the steel angles bent with the RC column. When the drift was increased above 0.75% in cycle 7, the second story east brace buckled out of plane when the load was applied eastward. Diagonal (shear) cracks appeared in the upper part of the first story most interior column. At the peak drifts (1%) of cycle 8, the measured base shear was 77 kips westward and 75 kips eastward.

In cycle 9, the second story west brace buckled out of plane at a drift of 1.2% when the load was applied westward. More yielding occurred in the vertical angles of the interior columns at the foundation level sections. Yielding also appeared on the stems of the bottom horizontal (WT) collectors near the connections. At the peak drift 1.5% (westward) of cycle 10, a crack was found adjacent to the weld between the vertical angle and the base angle at the foundation level of the most interior column. However, the applied load did not drop because of this local failure. But the test was stopped at the end of this cycle to repair the crack by new welding. The maximum measured base shear was 78.5 kips westward and 82 kips eastward.

Testing was resumed and a total drift of 2% was applied in cycles 11 and 12. Significant axial deformations occurred in the braces of both stories. More yielding occurred in the steel elements at several critical sections. The cracks in the RC columns and slabs became more severe, especially at the first story exterior slab-column joint. The cracked weld which was repaired at the end of cycle 10 reopened again, and another similar crack was found adjacent to the weld of the vertical angle-base angle connection of the first interior column. The applied load did not drop however, and the maximum measured base shear was 86 kips westward and 92 kips eastward.

Three additional cycles were applied. In the first cycle, a 1% drift was applied in both directions. The measured base shear at the peak drifts was 50 kips westward and 48 kips eastward. In the last two cycles, a 2% drift was applied in both directions. The maximum measured base shear was 80 kips eastward and 85 kips eastward. The hysteretic loops of these cycles were stable and full, and the stiffness deterioration in the 2% cycles was very slight since the measured loads compared well with the loads measured in cycle 12.

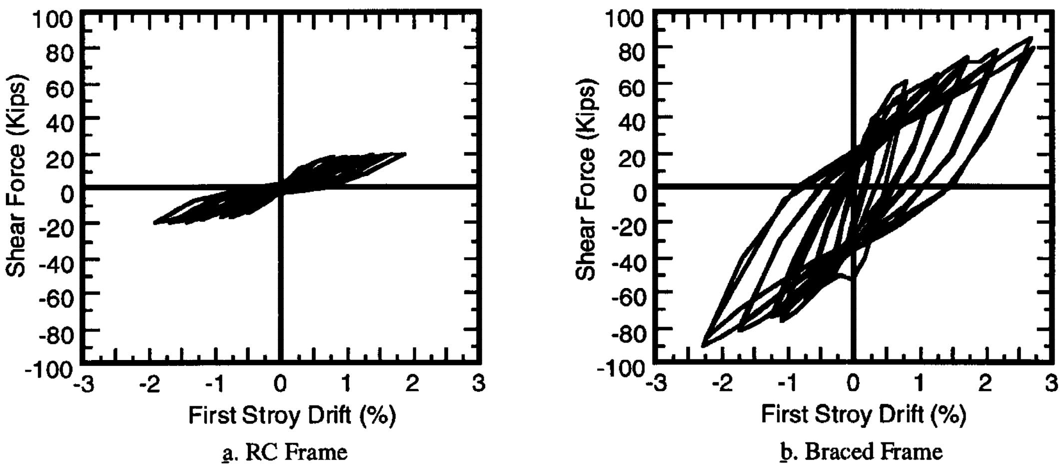

Fig. 8. Hysteretic loops of the RC and second braced frame.

Hysteretic Loops: The load-displacement hysteretic loops in the first story of the reinforced concrete frame from the first bare frame test and those of the braced frame are shown in Fig. 8. It is clear that the hysteretic loops of the braced frame are very full and stable with dramatic increases in strength, stiffness and energy dissipation over those of the original RC frame. A 2.75% drift in the first story of the second braced frame was reached. Story drifts in the order of 2-3% are generally considered acceptable for building structures during severe earthquakes.

Behavior of Braces: The concrete-filled braces behaved in a very ductile manner through the fifteen cycles with no failures either in members or in the end connections. Consequently, the braces provided expected additional lateral strength, stiffness as well as energy dissipation.

Behavior of Steel Elements: The horizontal and vertical steel elements also behaved satisfactorily in their dual role of truss members as well as frame members acting compositely with RC members, as intended in the design. Connection of horizontal steel to RC slabs is required in order to transfer the lateral inertia forces from the floors. However, it is convenient not having to connect the vertical steel with the RC columns. This test showed excellent "semi-composite" action that developed between the vertical steel and the core RC columns. The curvature of the RC columns forced the vertical angles to bend simultaneously and to provide the desired flexural strength. The lateral strength provided by the braces and the steel elements is shown in Fig. 9. It should be mentioned here that the maximum base shear calculated by mechanism analysis for this frame was 96 kips, while the maximum strength from the test was 92 kips.

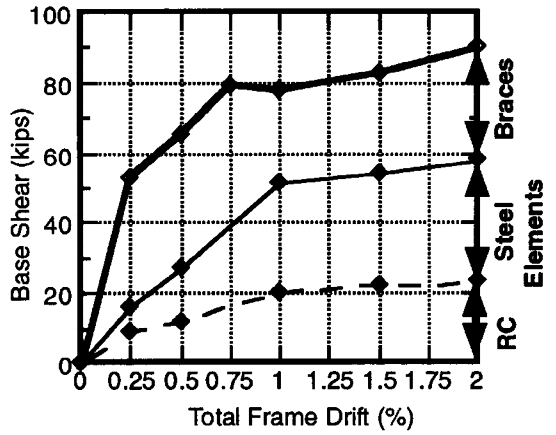

Fig. 9. Components of lateral strength.

CONCLUSIONS

Use of ductile steel bracing system proved very effective for strengthening of the seismically weak and damaged RC frame. The hysteretic loops were very "full" and stable with dramatic increases in strength, stiffness and energy dissipation. The design philosophy used for the steel bracing system proved satisfactory. Also, the construction process worked well and appears practical to use in real life projects. The detail and connections used in this study to maintain the continuity of the vertical angles of the steel bracing system through the slabs of the RC frame proved successful, and it is greatly encouraged because it further enhances the ductility of these joints by increasing their punching shear strength. The behavior of the vertical steel elements, used for external jacketing of RC columns, as a unit steel section depends greatly on the strength and spacing of the batten plates which tie them together. The method developed to design the strength and spacing of the batten plates for the vertical steel to develop full flexural strength proved very satisfactory.

ACKNOWLEDGMENTS

This study was supported by the U.S. National Science Foundation under Grant No. BCS-9016863. The conclusions and opinions expressed in this paper are solely those of the writers and do not necessarily represent the views of the sponsor.

REFERENCES

American Institute of Steel Construction (1986). Load and Resistance Factor Design. Manual of Steel Construction, 1st edition, Chicago, IL.

Jones, E.A., and Jirsa, J.O. (1986). Seismic strengthening of a reinforced concrete frame using structural steel bracing. *PMFSEL Report No. 86-5*, Dept. of Civil Engineering, The University of Texas at Austin, May.

Lee, H., and Goel, S.C. (1990). Seismic behavior of steel built-up box-shaped bracing members and their use in strengthening RC frames. *Report No. UMCE 90-7*, Dept. of Civil Engineering, The University of Michigan, Ann Arbor, MI, May.

Lee, S., and Goel, S.C. (1987). Seismic behavior of hollow and concrete-filled square tubular bracing members. *Report No. UMCE 87-11*, Dept. of Civil Engineering, The University of Michigan, Ann Arbor, MI, Dec.

Masri, A.C., and Goel, S.C. (1994). Steel bracing systems for seismic strengthening of reinforced concrete slab-column structures. *Report No. UMCEE 94-07*, Dept. of Civil and Environmental Engineering, The University of Michigan, Ann Arbor, MI, Apr.

Wight, J.K., Chien, S.H., and Dovich, L. (1994). Evaluation of R/C slab-column systems for seismic strengthening. *Proc. Fifth U.S. National Conf. on Earthquake Engineering*, Chicago, IL, July 10-14.