SEISMIC BEHAVIOR OF MULTISTORY BRACED STEEL FRAMES

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SYNOPSIS

A step-by-step numerical procedure is used to compute the inelastic response of single-bay braced and unbraced steel frame structures. The dynamic response is computed by assuming an elasto-plastic type hysteresis behavior in tension only for the diagonal bracing members, in bending for the girders, and in bending with axial force effect on the plastic moment and the P-Δ effect included for the columns. The N-S component of the 1940 El Centro earthquake with the accelerations multiplied by 1.5 is used. The results of the analyses are presented to study the influence of the method of design and varying arrangements of the bracing members on the seismic response of these structures.

INTRODUCTION

Braced frame buildings are constructed in seismically active regions throughout the world. However, studies concerning their elastic or inelastic response during strong earthquakes have been few and recent. Clough and Jenschke (1) studied the elastic behavior of a specific braced structure and Housner's (2) limit analysis was restricted to supporting structures of elevated water tanks or storage bins. Hanson and Fan (3) studied the inelastic earthquake response of a number of multistory frames with minimum cross bracing by assuming the columns to behave elastically. Unfortunately, an error was found in the formulation of the P-Δ effect for response computations which makes their reported results of questionable accuracy. Nevertheless, the results did show that even a light cross bracing may cause large increases in the column axial forces and questioned using the assumption of elastic column behavior in braced frames. Workman (4) followed the work of Hanson and Fan (3) and re-formulated the procedure of response computation by assuming an elasto-plastic type hysteresis behavior for all frame members.

The earthquake response of several multistory frames having different layouts of diagonal bracing is studied herein using the procedure developed by Workman (4). The structures are designed by two different design philosophies. Those of the first series are proportioned by the allowable strength design procedure currently used in the United States and those of the second series are a class of period-controlled structures.

METHOD OF ANALYSIS

For details of the procedure reference should be made to the report by Workman (4). However, the most significant assumptions employed in the mathematical modelling of the structures are given in the following:

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1. The actual structure is represented by a single-bay, multi-story, rigid-jointed frame with the mass of the structure (equivalent to the dead weight of the finished building) concentrated at the floor levels.

2. The braces are tension members having an elasto-plastic type force-deformation relationship with no compression strength (Figure 1). The girders and columns follow an elasto-plastic moment-curvature behavior (Figures 2 and 3) with the axial force effects included in the axial deformation and the plastic moment capacity for the columns. The hysteresis behavior for the columns is non-linear in the plastic range because of the modification of the plastic moment by the axial force, which is constantly varying during the motion of the structure.

3. In addition to the lateral displacement of the structure, two joint rotations and two vertical displacements are included at each floor. Thus, the number of degrees of freedom is equal to five times the number of stories. Only the lateral inertia force for the lumped floor masses is considered - the rotatory and vertical inertia effects are assumed negligible.

4. The P-Δ effect from both the gravity and the dynamic axial loads in the columns is included.

5. Viscous damping equivalent of 5 percent of critical in each elastic mode is also included in some structural models.

Once the mathematical model of the structure is formulated the governing differential equations of motion are integrated by a numerical technique using a step-by-step procedure in which the structure is assumed to respond linearly during each short time interval. A fourth order Runge-Kutta method is used in the procedure.

PROGRAM OF INVESTIGATION

Structures Considered

The structures considered in this study are 10 stories high with a uniform story height of 12 ft, bay width of 20 ft and fixed at the base. Diagonal bracing is used with four different layouts - fully braced, bottom story open, alternate stories open and completely unbraced. These patterns of bracing arrangement are shown in Figure 4 and designated as F, B, A and U frames, respectively.

Earthquake Accelerogram

The accelerogram used to compute the response of these structures is the N-S component of the May 18, 1940 El Centro earthquake with accelerations multiplied by a factor of 1.5 and subsequently referred to as the modified El Centro accelerogram. This accelerogram is considered to be a representative of severe earthquakes. The first seven seconds of this accelerogram which contain the peak acceleration impulses in the ground motion are used in the response computation.

Response Parameters

The parameters selected to study the earthquake response of the structures are: the maximum absolute horizontal displacement of floors relative to the base, the maximum absolute value of the relative horizontal displacement between floor levels, the maximum absolute value of the horizontal acceleration at floor levels, the maximum column axial force ratios, the maximum ductility ratios and the energy dissipated by the various frame members.

The column axial force ratio is defined as the maximum axial force in a column divided by its yield force which is the product of the column area and the yield stress.

The ductility ratio for the bracing members is defined as the maximum elongation divided by the yield elongation. The ductility ratio for girders and columns in the elastic range is defined as the absolute value of the end moment to the plastic moment. In the plastic range the ductility ratio is defined as one plus the ratio of the maximum plastic hinge rotation to the yield rotation for the member with equal end rotations and no joint translation. The modified plastic moment due to axial force is used in the calculation of yield rotation for columns.

Series One

Four 10-story structures of Series One (designated as frames F1, B1, A1 and U1) were designed using typical dead and live loads for multistory office buildings and lateral earthquake forces similar to those specified by the Uniform Building Code 1970. Two design criteria, member stress and a lateral deflection limit of 0.35% of the height at design earthquake forces were used. The lateral earthquake forces were based on a dead load per floor of 44 kips, C = 0.05, K = 1.0 and J = 1.0. This is somewhat crude but conservative estimate of the code lateral forces. The frames F1 and B1 were considered to carry three bays of lateral force and one bay of vertical loads. The moment resisting frame consisting of beams and columns for these two structures was designed for 25% of the total lateral load and the complete system for the full loads. Frames A1 and U1 carried one bay of lateral and vertical loads. The live load was taken as 32 kips per floor with no reduction. In the design of beams and columns under combined gravity and earthquake forces allowable stresses were increased by 33%. The diagonal braces were designed as tension members using an allowable stress of 22 ksi without utilizing a 33% increase. A minimum area of 2.88 sq. in. was used for these members which corresponds to a slenderness ratio of about 300. A36 steel was assumed for these frames. The structural properties of these frames are given in Tables I-IV.

In the structures F1, B1 and A1 the bracing was designed to provide strength under full lateral design forces. The lateral deflections of these structures at these design forces were found to be within permissible limits. But the unbraced frame U1 showed excessive lateral drift at design forces. Another structure was obtained by adding minimum

cross bracing (area = 2.88 sq. in.) in each story of the structure U1 for the purpose of controlling drift at design loads. This is quite often done in practice. This frame is designated U1X and its structural properties are given in Table V.

The response of these five structures was computed for the modified El Centro accelerogram. The results are presented in Figure 5(a)-(i) and discussed below.

The maximum floor displacements are shown in Figure 5(a). It is noticed that the frame U1 has the largest displacements of all the five frames in this series, including F1 and B1 which carried three times as much floor mass. Addition of minimum cross bracing to the frame U1 for drift control proves very effective as can be noticed from much reduced floor displacements in the frame U1X. Similar observations can be made about the relative story displacements which are shown in Figure 5(b). The presence of diagonal bracing in these structures shows remarkable control over lateral displacements.

The floor accelerations are plotted in Figure 5(c). The presence of cross bracing results in increased accelerations. The frame U1 shows least floor accelerations and the addition of minimum cross bracing to it resulted in a 3 to 4 times increase in accelerations. It is also interesting to note that the accelerations in frames F1 and B1 are smaller than those in U1X or A1. A logical explanation for this may be that the frames F1 and B1 carry three times the floor mass compared with the other three frames of this series and since the bracing members yield at fairly small amplitude level this renders F1 and B1 relatively more flexible structures for most part of their dynamic response. The diagonal bracing also results in considerable increase in the column axial forces, Figure 5(d).

The inelastic activity in the various members is shown in Figure 5(e)-(i). The frames F1, B1 and A1 show considerable inelastic activity in the columns whereas those in U1 and U1X generally remain elastic. Comparing the girder ductility ratios in the frames U1 and U1X, it can be seen that the addition of minimum cross bracing to the unbraced frame U1 causes substantial reduction in the inelastic activity in the girders. The girder ductility ratios for frames F1 and B1 are about the same as in U1 even though the former have three times as much floor mass. The diagonal braces undergo quite large deformations beyond the yield point - the ductility ratio being as large as 7 in frames B1 and F1. The diagonal bracing also accounts for about half of the total energy dissipated in these structures, Figure 5(h) and (i).

Series Two

The braced structures in this series were obtained by adding minimum cross bracing (slenderness ratio about 300) to an unbraced structure U2 having a fundamental period of 1.25 sec. The beams and columns were identical in all the frames in this series. To counteract the stiffening effect of the bracing members in frames F2, B2 and A2 the floor masses were adjusted to obtain a fundamental period of about 1.25 sec. This may

be considered as different amounts of floor area contributing their inertia forces to one braced bay. Thus, this series represents a class of period controlled structures which are stiffer than those of Series One except for the special case U1X. The structural properties are given in Tables VI and VII.

Some of the response parameters of these structures due to the modified El Centro accelerogram are presented in Figure 6(a)-(e). A striking feature of these results is that there is not as much variation in the response of these frames as was observed in the case of Series One. Also, the response is relatively more controlled in terms of displacements and the inelastic activity in the members. The columns generally remain elastic, the ductility ratio of the girders ranges between 1 and 2, and that for the bracing members between 1 and 4.

Type of Analysis

The nine structures of Series One and Series Two were also subjected to the following four different types of analytical formulations for the response computation:

1. Inelastic without viscous damping (IU)
2. Inelastic with 5% viscous damping (ID)
3. Elastic with 5% viscous damping (ED)
4. Elastic without viscous damping (EU)

In the current design practice an elastic analysis with some percentage of critical viscous damping in each mode (such as 5%) is generally performed to predict the anticipated response of a structure due to a given ground motion. A comparison of the results from the ED analysis with those of IU or ID analysis would be of particular interest to check the validity of such a procedure.

The response of the structure B1 due to the modified El Centro accelerogram as computed from the above four analyses is compared in Figure 7(a)-(e). The floor displacements and relative story displacements as computed from the IU analysis are about the largest whereas those from the ED analysis form the lower bound. Accentuation of displacements in the upper stories (the so-called "whip-lash" effect) is much pronounced in the results of the EU analysis. Except for a few stories near the top the member ductility ratios are very much underestimated by the elastic analysis in both undamped as well as damped cases.

CONCLUSIONS

This study has provided some insight into the seismic behavior of multistory steel structures with light diagonal bracing members. Some of the significant aspects of the results are summaried in the following:

1. The structures of Series One showed rather large lateral displacements, column axial forces and inelastic activity in the members for the 1.5 times the N-S component of the May 1940 El Centro accelerogram. The largest ductility ratios for the bracing members and the columns were about 7 and 5, respectively.

2. The frames of Series Two, which were stronger and stiffer than those of Series One, showed smaller lateral displacements and inelastic activity in the members.

3. The braced frames generally showed reduced lateral displacements and inelastic activity in the columns and girders as compared with the corresponding unbraced frames. This was accompanied by substantially increased axial forces in the columns and large floor accelerations.

4. The elastic analysis with or without viscous damping does not represent the behavior of these structures when considerable yielding occurs in most of the structural members.

ACKNOWLEDGMENTS

This paper summarizes the results obtained from a research project entitled, "Earthquake Resistance of Braced Steel Frame Structures," conducted at the University of Michigan, sponsored by the American Iron and Steel Institute and reported in reference (5).

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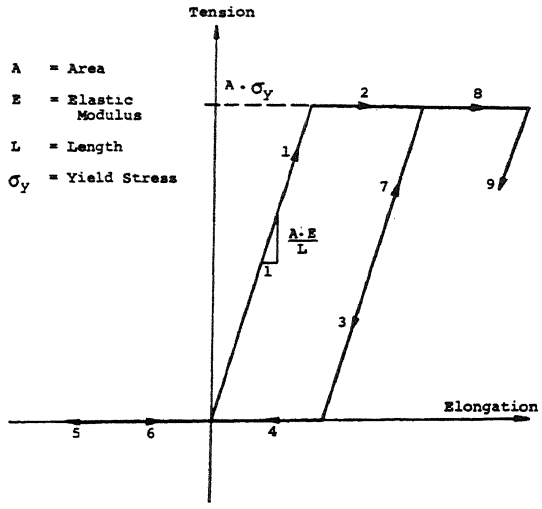

Figure 1. Force-deformation relation of diagonal cross bracing.

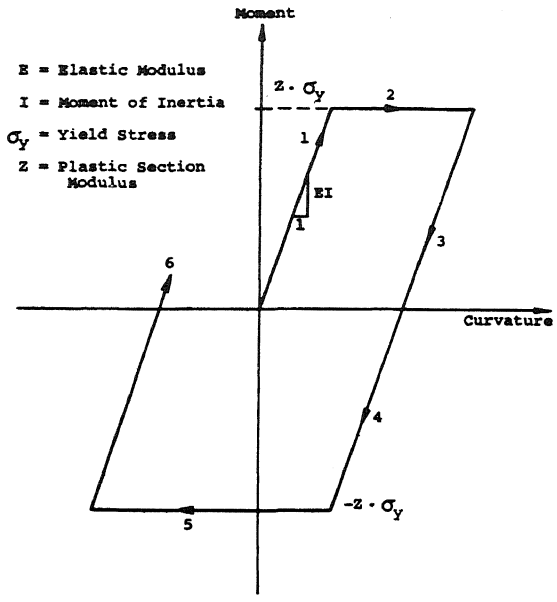

Figure 2. Girder elasto-plastic moment-curvature relation.

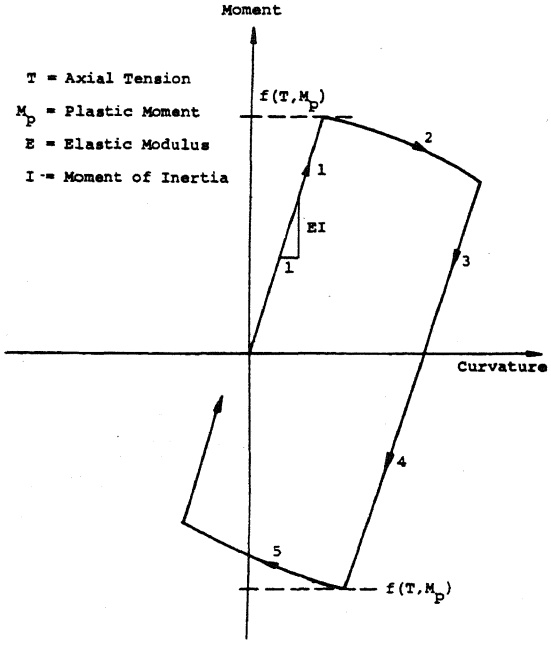

Figure 3. Column modified elasto-plastic moment-curvature relation.

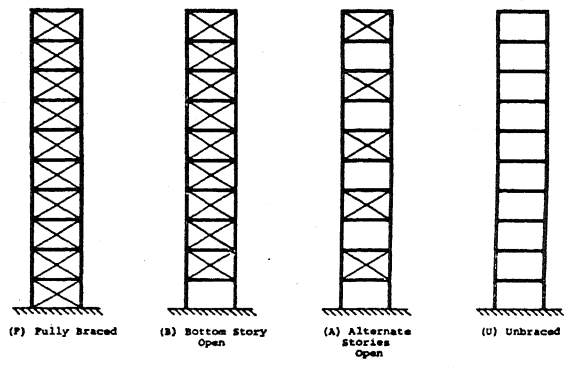

Figure 4. Types of Structures.

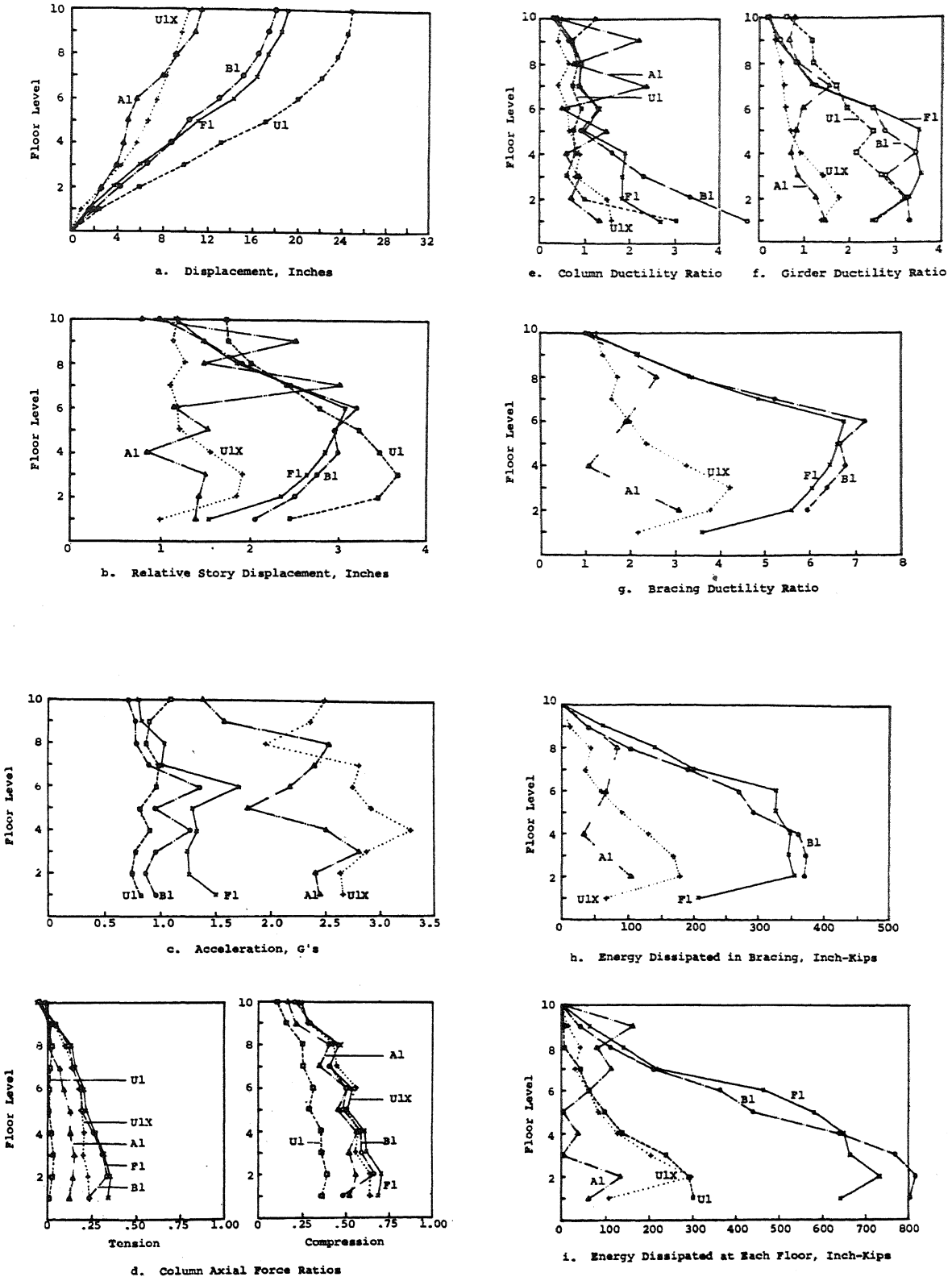

Figure 5. Series One - inelastic undamped analysis.

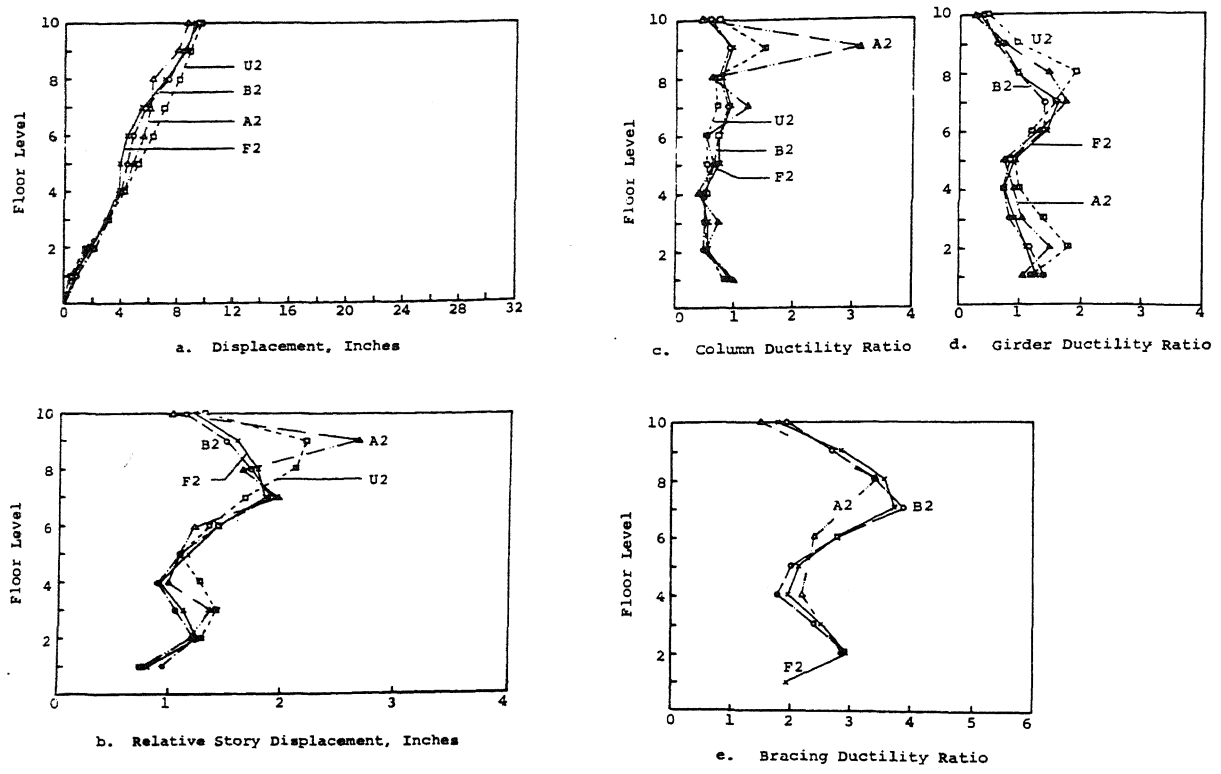

Figure 6. Series Two - inelastic undamped analysis.

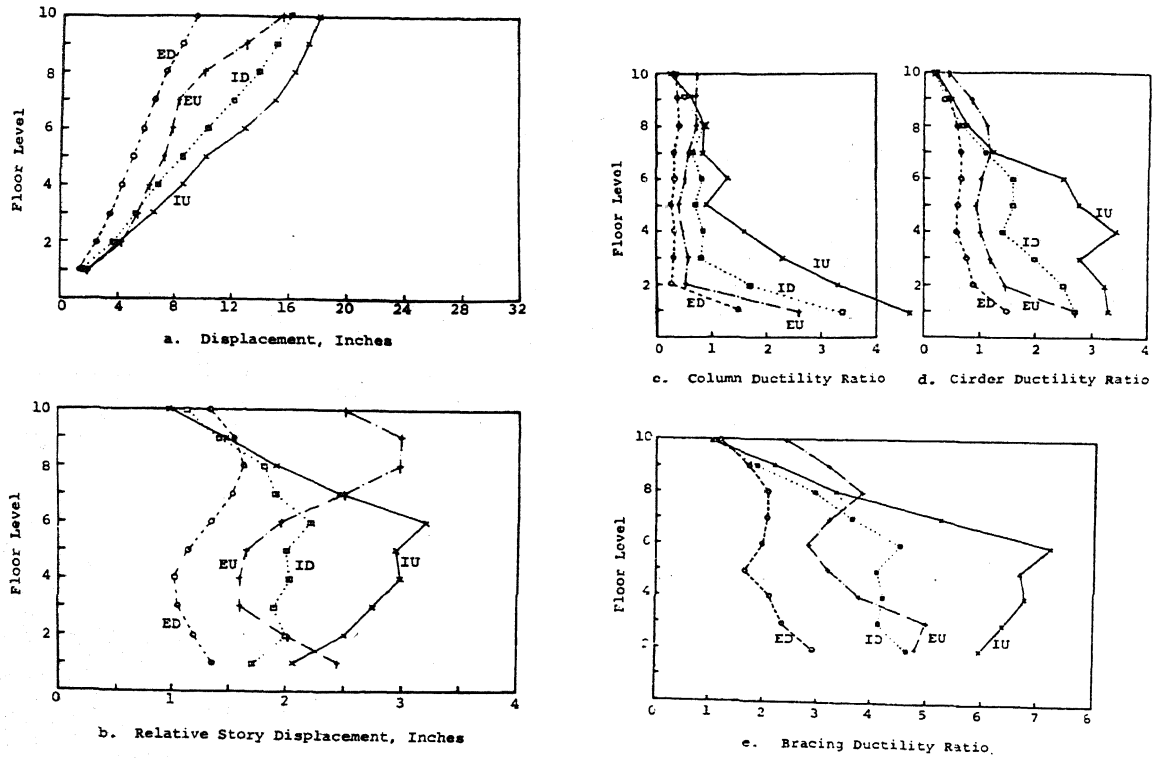

Figure 7. Structure B1, IU, ID, ED and EU analyses.

Table I. Structural Properties of Frame F1

Floor Level	Section		Bracing Area sq. in.
	Girders	Columns	
10	W18x50	W14x34	2.88
9	W18x50	W14x53	2.88
8	W18x50	W14x53	2.88
7	W18x50	W14x78	2.88
6	W18x50	W14x78	2.88
5	W18x60	W14x103	2.88
4	W18x60	W14x103	2.94
3	W18x60	W14x119	2.94
2	W18x60	W14x119	3.38
1	W18x60	W14x136	3.38

Fundamental period of vibration = 1.58 sec.

Table II. Structural Properties of Frame B1

Floor Level	Section		Bracing Area sq. in.
	Girders	Columns	
10	W18x50	W14x34	2.88
9	W18x50	W14x53	2.88
8	W18x50	W14x53	2.88
7	W18x50	W14x78	2.88
6	W18x50	W14x78	2.88
5	W18x60	W14x103	2.88
4	W18x60	W14x103	2.94
3	W18x60	W14x119	2.94
2	W18x60	W14x119	3.38
1	W18x77	W14x184	0.0

Fundamental period of vibration = 1.84 sec.

Table III. Structural Properties of Frame A1

Floor Level	Section		Bracing Area sq. in.
	Girders	Columns	
10	W18x45	W14x34	2.88
9	W18x45	W14x43	0.0
8	W18x45	W14x43	2.88
7	W18x45	W14x61	0.0
6	W18x45	W14x61	2.88
5	W18x50	W14x74	0.0
4	W18x50	W14x74	2.88
3	W18x50	W14x87	0.0
2	W18x50	W14x87	2.88
1	W18x50	W14x103	0.0

Fundamental period of vibration = 1.50 sec.

Table IV. Structural Properties of Frame U1

Floor Level	Section		Bracing Area sq. in.
	Girders	Columns	
10	W18x50	W14x34	0.0
9	W18x50	W14x53	0.0
8	W18x50	W14x53	0.0
7	W18x50	W14x68	0.0
6	W18x50	W14x68	0.0
5	W18x60	W14x87	0.0
4	W18x60	W14x87	0.0
3	W18x60	W14x103	0.0
2	W18x60	W14x103	0.0
1	W18x60	W14x119	0.0

Fundamental period of vibration = 2.03 sec.

Table V. Structural Properties of Frame U1X

Floor Level	Section		Bracing Area sq. in.
	Girders	Columns	
10	W18x50	W14x34	2.88
9	W18x50	W14x53	2.88
8	W18x50	W14x53	2.88
7	W18x50	W14x68	2.88
6	W18x50	W14x68	2.88
5	W18x60	W14x87	2.88
4	W18x60	W14x87	2.88
3	W18x60	W14x103	2.88
2	W18x60	W14x103	2.88
1	W18x60	W14x119	2.88

Fundamental period of vibration = 1.10 sec.

Table VI. Structural Properties of Frame U2

Floor Level	Section		Floor Weight kips
	Girders	Columns	
10	W18x96	W14x78	45.45
9	W18x96	W14x78	45.45
8	W18x96	W14x127	45.45
7	W18x96	W14x127	45.45
6	W18x96	W14x176	45.45
5	W21x127	W14x176	45.45
4	W21x127	W14x219	45.45
3	W21x127	W14x219	45.45
2	W21x127	W14x264	45.45
1	W21x127	W14x264	45.45

Fundamental period of vibration = 1.25 sec.

Table VII. Additional Structural Properties of Series Two Braced Frames F2, B2, A2

Floor Level	Frame F2		Frame B2		Frame A2	
	Bracing Area sq. in.	Floor Weight kips	Bracing Area sq. in.	Floor Weight kips	Bracing Area sq. in.	Floor Weight kips
10	2.88	93.82	2.88	90.00	2.88	67.40
9	2.88	93.82	2.88	90.00	0.0	67.40
8	2.88	93.82	2.88	90.00	2.88	67.40
7	2.88	93.82	2.88	90.00	0.0	67.40
6	2.88	93.82	2.88	90.00	2.88	67.40
5	2.88	93.82	2.88	90.00	0.0	67.40
4	2.88	93.82	2.88	90.00	2.88	67.40
3	2.88	93.82	2.88	90.00	0.0	67.40
2	2.88	93.82	2.88	90.00	2.88	67.40
1	2.88	93.82	0.0	90.00	0.0	67.40

Fundamental period of vibration = 1.25 sec.