DESIGNING MULTISTORY BRACED STEEL FRAMES FOR EARTHQUAKES

by

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SUMMARY

A comparative study of seismic response of 7-story concentric and eccentric K braced frames alongwith that of an unbraced frame is presented. The main objective is to study the influence of different member proportions. It is noticed that strong bracing members can cause excessive inelastic activity in the girders or columns. Eccentric braced frames with slender bracing members are preferred since they seem to show the ductile behavior of open-moment resisting frames combined with the stiffness efficiency of the braced systems.

INTRODUCTION

The two most commonly used types of framing systems for earthquake resistant design of steel structures are: Open-moment resisting and the braced frames. The open-moment resisting frames are considered by most engineers to be more ductile than braced frames. The Uniform Building Code (1) records this judgement by granting a K factor of 0.67 for the open-moment resisting frames as opposed to 1(0.8x1.25) or 1.25(1.0x1.25) for the braced frames. Open-moment resisting frames tend to be quite flexible and the control of lateral drift can make their design rather uneconomical for tall buildings. Braced frames, on the other hand, are very efficient in providing strength and stiffness against lateral forces. However, excessive inelastic activity could occur in the members of these frames of certain bracing patterns and member proportions when subjected to severe earthquake motions.

Building Codes generally do not give guidelines for selection of desirable member proportions for structures. For a set of prescribed design forces, it is possible to design a frame with several different bracing arrangements and stiffness combinations. The strength or slenderness of bracing members relative to that of beams and columns can significantly affect the seismic response of a braced frame. Further, the seismic behavior of concentric braced frames can be significantly different from that of eccentric braced frames.

A study of seismic response of 7-story concentric and eccentric K braced frames is presented in this paper. The main objective was to study the influence of different member proportions with a view to develop some practical recommendations for their improved performance during a severe earthquake motion.

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SELECTION OF STRUCTURE

The structure selected for this study represents a 7-story, three bay, interior frame of a symmetrical building with concentric K or split K bracing in the middle bay. Typical plan and elevations are shown in Figure 1. A single bay model which eliminates the interaction of the unbraced outer bays is used to represent the frame. The dead and live loads per floor are 65 psf and 100 psf, respectively. The base shear V was computed by using the Uniform Building Code formula (1):

$$V = ZICSKW$$

where horizontal force factor K = 1.0, zone factor Z = 1, importance factor I = 1, soil factor CS = 0.08 and weight W = 13,000 kips. All members were designed for 1.25 times the required forces due to the lateral loads. The steel is A36.

The frames were designed so as to arrive at three different member proportions in each case. These are: weak girder-strong brace frame (DK-1, DSK-1), weak grider-intermediate brace frame (DK-2, DSK-2), and strong girder-weak brace frame (DK-3, DSK-3). The terms DK and DSK stand for concentric K and split K braced frames, respectively. A girder is called weak or strong depending upon the relative stiffness of the corresponding story columns. Bracing members of slenderness ratio of the order of 60 or less are called strong, between 60 to 120 are called intermediate and more than 120 are called weak. Allowable stresses were increased by 33% for the bracing members in frame DSK-1, DSK-2, DSK-3 and DK-2. This increase was not allowed in case of DK-1. The strong grider - weak brace proportion for frame DK-3 was obtained by using the horizontal force factor K = 0.8 and the "two-bit" frame philosophy according to the Uniform Building Code (1). The relative member stiffness of these six frames are given in Tables 1 and 2.

METHOD OF ANALYSIS

The method of analysis is based on the following assumptions:

a. The building is assumed to be laid out in a rectangular grid plan.
b. The floor diaphragms are rigid in their own plane and all out of plane flexural resistance is provided by the girders.
c. Each nodal point in the structure model has three degrees of freedom: a horizontal translation, a vertical translation and a rotation. Due to assumption b, the horizontal translation is the same for all nodes at a floor level.
d. The mass of the structure is lumped at the floor levels. Each floor mass consists of entire mass of the floor and half the mass of the columns in stories immediately above and below, and 25% of the live load on each floor. Rotatory inertia forces have been neglected.
e. Inelastic hysteresis behavior of the frame members is the only source of energy dissipation.
f. The effect of axial and flexural inelasticity in the columns and bracing members is represented by the end moment-buckling hysteresis model developed by Jain and Goel (2)(Figure 2) for use with the DRAIN-2D program by Kanaan and Powell (3).

g. A conventional elastic-plastic hysteresis behavior in flexure is assumed for the girders.

DISCUSSION OF RESULTS

The response of the six braced frames was computed due to first 10 seconds of the 1-5 times the N-S component of May, 1940 El Centro earthquake. Selected response parameters are shown in Figures 3 and 4 for concentric and split K braced frames, respectively.

Figure 3 for the DK frames shows that as the bracing members become more slender the horizontal floor displacements increase whereas the story shears and the column axial forces decrease. Stronger bracing members in frame DK-1 also cause somewhat larger ductilities in girders, columns as well as bracing members themselves. In their original report Jain and Goel (4) also found that a reduction of about 3.5% in the strength of the columns of frame DK-1 caused one of the columns in the bottom story to buckle leading to very large floor displacements.

Figure 4 for the DSK frames shows that the horizontal floor displacements for the DSK-1 and DSK-3 frames are about equal which are smaller than those in frame DSK-2. Stronger bracing members such as those in frames DSK-1 and DSK-2 result in quite large girder ductilities (up to about 15) as compared with those in frame DSK-3 with strong girder - weak brace proportions. It is believed that such large girder ductilities could cause excessive floor damage in the building. The reduction in girder ductility demand in frame DSK-3 is, however, achieved at the expense of somewhat increased inelastic activity in the columns and bracing members.

CONCLUDING REMARKS AND RECOMMENDATION

From the results presented in the preceding section it is noticed that bracing members of small and intermediate slenderness ratio caused excessive girder ductility in eccentric K braced frames whereas similar proportions caused large ductilities in girders, columns as well as the bracing members themselves in concentric K braced frames. Column buckling also occurred in one concentric K braced frame with stronger bracing members. On the other hand strong girder-weak brace proportions resulted in a more even distribution of inelastic activity among the members of eccentric as well as concentric braced frames, DSK-3 and DK-3.

A comparison of the characteristics of 7-story strong girder-weak brace frames DSK-3 and DK-3, as used in this study, is shown in Table 3. Also shown in this table are the corresponding features of a 10-story open-moment resisting frame by Anderson and Bertero (5). All these three frames were designed by using the allowable stress design procedure of the then current Uniform Building Code and resulted in generally strong girder proportions. As is evident from the fundamental period of these frames, the unbraced frame is the most flexible, frame DK-3 with concentric K bracing the most stiff and the stiffness of split K frame DSK-3 falls in between. It should be noted that the floor masses in the two braced frames are approximately five times of those in the unbraced frame.

Reviewing the response parameters of these frames due to 1.5 times the N-S component of the 1940 El Centro earthquake it is noticed that the unbraced frame shows the largest horizontal displacement whereas the frame DSK-3 with eccentric K bracing shows the smallest displacements. Similarly, the member ductilities also are the smallest in frame DSK-3 as compared with those in the unbraced or the concentrically braced frame DK-3.

From the study presented in this paper the following recommendations can be made with regard to the earthquake resistant design of braced steel frames:

1. Strong bracing members should be avoided as they could cause excessive inelastic activity in the girders or columns.
2. Eccentrically braced frames such as the split K braced frame DSK-3 with slender bracing members are preferred over similar frames with concentric bracing. The former seem to combine the ductile behavior of the open-moment resisting frames with the stiffness efficiency of the bracing systems. Also, it is convenient to provide large openings in the walls with eccentric systems.

ACKNOWLEDGEMENT

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TABLE 1 - MEMBER STIFFNESSES OF K BRACED FRAMES

Case	Frame	Column		Girder*	Brace
		I/L	$L/r_{max.}$	I/L	$L/r_{max.}$
Weak Girder Intermediate Brace	DK-1	4.5-9	35-60	1.5-2.5	70-100
Weak Girder Intermediate Brace-II	DK-2	4.5-9	35-60	1.5-2.5	80-105
Strong Girder-Weak Brace	DK-3	4.5-9	35-60	2.5-4	80-130

*Girder span L = 24 ft.

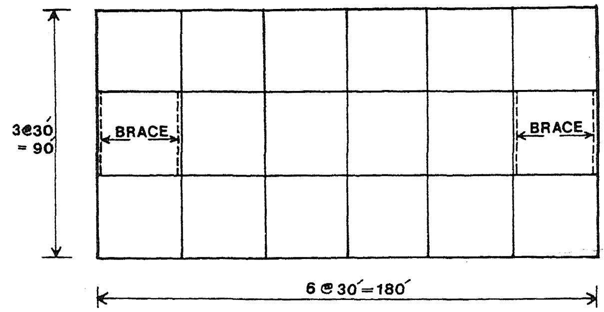
TABLE 2 - MEMBER STIFFNESSES OF SPLIT K FRAMES

Case	Frame	Column		Girder*	Brace	
		I/L	$L/r_{max.}$	I/L	I/L	$L/r_{max.}$
Weak Girder-Strong Brace	DSK-1	7-13	38	7-9	8-10	48
Weak Girder-Intermediate Brace	DSK-2	7-13	38	7-9	3	90
Strong Girder-Weak Brace	DSK-3	7-13	38	22-27	-	120

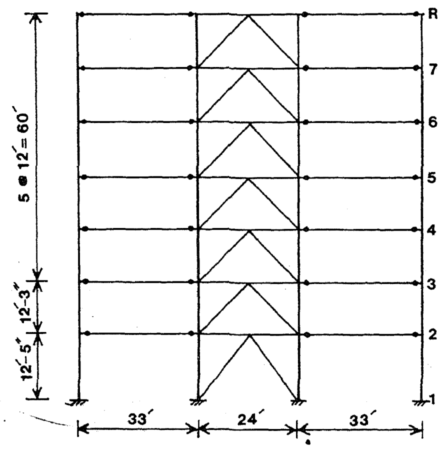
*Girder span L = 30 ft.

TABLE 3 - RESPONSE CHARACTERISTICS OF UNBRACED, ECCENTRIC AND CONCENTRIC FRAMES DUE TO 1.5 x EL CENTRO 1940 EARTHQUAKE

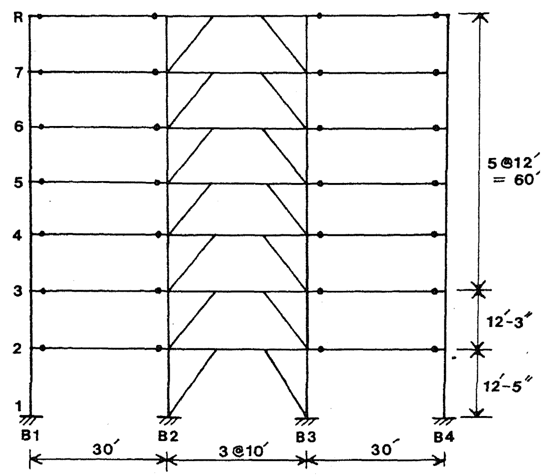
Parameter	10-Story Open Frame ASD/10/1.8/1.5/1.2 (Ref. 1)	7-Story Eccentric Split K Frame DSK-3	7-Story Concentric K Frame DK-3
1. Bay width	30 ft.	30 ft.	24 ft.
2. Girder span	30 ft.	10 ft.	12 ft.
3. Story height	12 ft.	12 ft.	12 ft.
4. Floor mass	0.24 kip-sec^2/inch	1.27 kip-sec^2/inch	1.27 kip-sec^2/inch
5. Design governing criterion	drift-0.3%	drift-0.5%	strength, drift was under 0.5%
6. Period, I mode	1.80 second	1.64 second	1.01 second
7. Horizontal force factor K	0.67	1.0	0.8
8. Upper 7-story sections: Columns Girders	14WF74-14WF167 18WF55-30WF99	W14x87-W14-167 W30x172-W30x210	W14x61-W14x136 W18x45-W18-70
9. Brace Slenderness Ratio	-	120	80-130
10. Max. base shear/ UBC value	4	2.75	2.75
11. Max. top floor displacement	14 inch	8.5 inch	12 inch
12. Max. story drift	1.5%	1.4%	2.2%
13. Column ductility	1-6	<1-4	<1-3.5
14. Girder ductility	3-7	<1-4	1-9.5
15. Brace ductility	-	1-8	1-17



(a) Framing Plan



(b) K Braced Frame



(c) Split K Braced Frame

Fig. 1 - The Structure

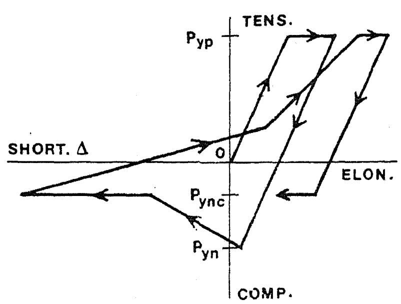


Fig. 2(a) - Buckling Component

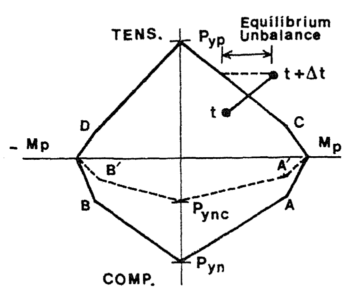


Fig. 2(b) - End Moment - Axial Force Interaction Component

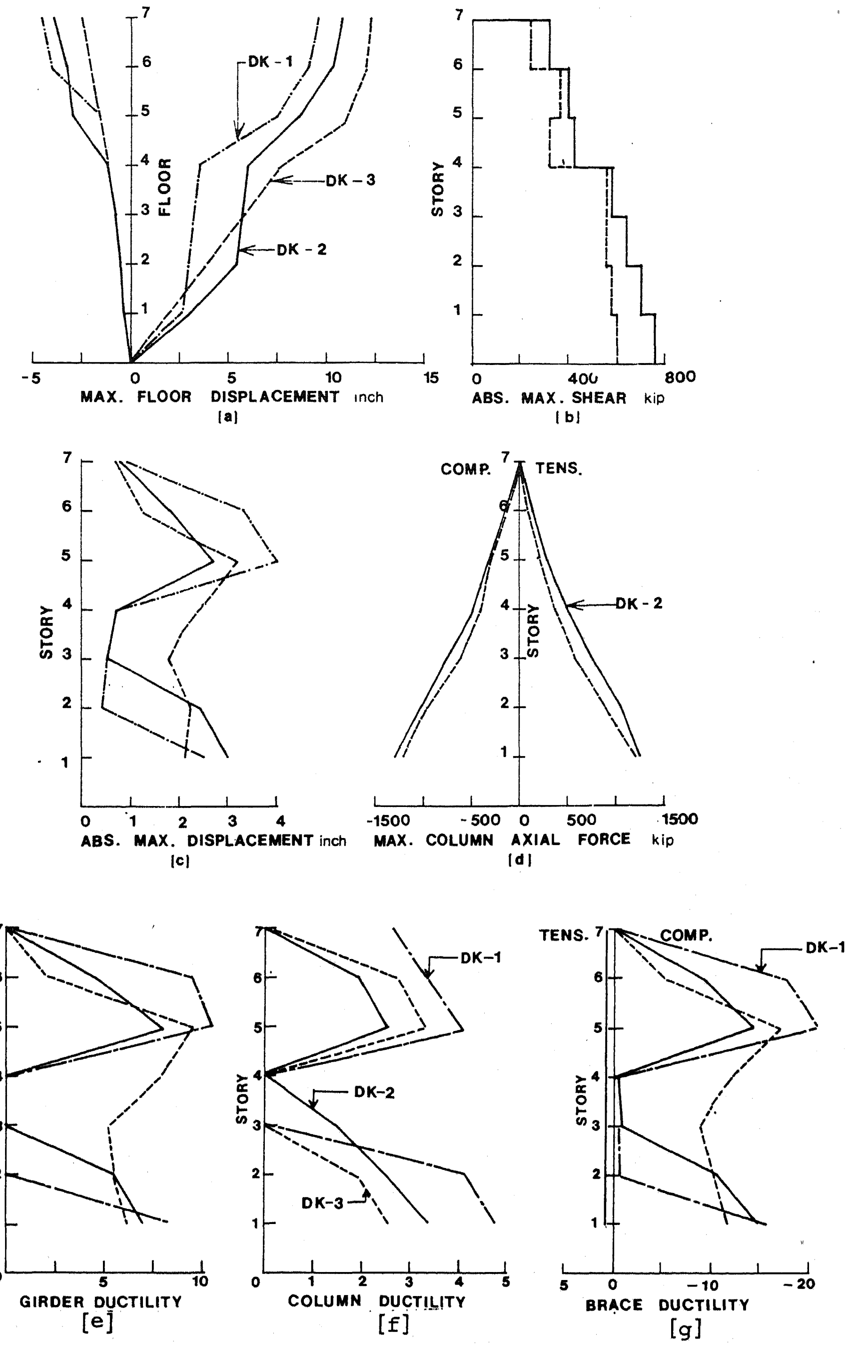


Fig. 3 - Influence of Different Members Proportions - K Frames

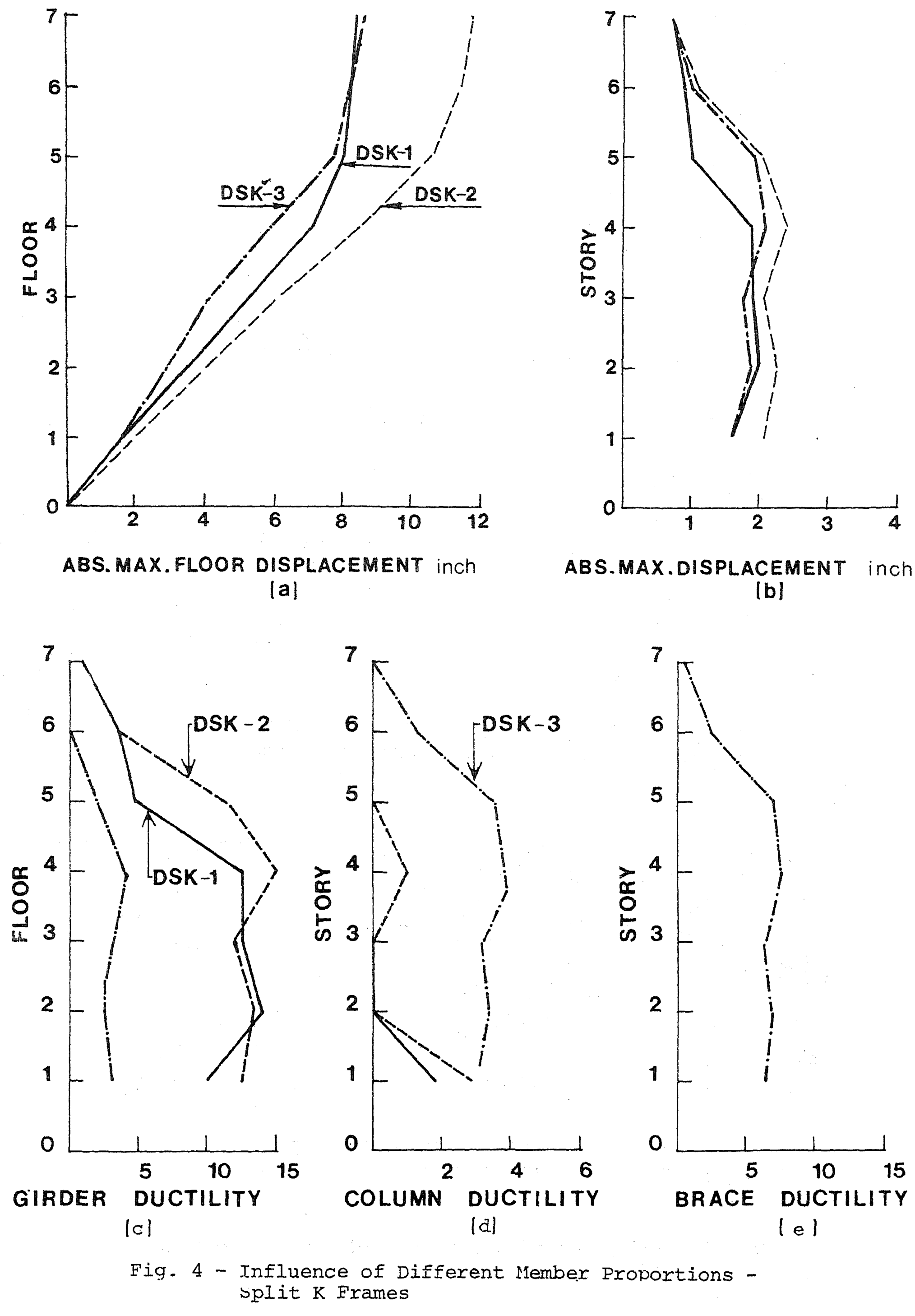


Fig. 4 - Influence of Different Member Proportions - Split K Frames