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STUDY OF MOMENT RESISTANT KNEE BRACED FRAMES AS AN ALTERNATIVE FOR MOMENT RESISTING FRAMES WITH RIGID CONNECTIONS

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ABSTRACT

It is important to recognise that whether or not knee braced connections could be an appropriate alternative for rigid beam-column connections in moment resisting steel structures, and if they are used , what are the advantages and disadvantages of these type of structures and what should be done to overcome the shortcomings , at the first stage because of wider plastic hinge regions provided for moment resistant knee braced frames , it seems that they show better ductility performance. Moreover, because of lesser horizontal deformations they are less susceptible to p- effects. In this paper several aspects of knee braced moment resisting frames which have been studied, are:

1. Study of linear behaviour of moment resistant knee braced frames (KBF) compared with ordinary moment rigid frames (OMRF) within serviceability regions ,
2. Comparison of stability characteristics of KBF and OMRF structures,
3. Ductility performance and collapse mechanism of two structural systems are compared ,
4. Reparability and rehabilitation of existing structures and damaged structures using knee braced connections are studied.

For the above mentioned study, a parametric analysis and design have been carried out on different steel structures ranging from one story up to 10 stories with variable span lengths. The results of this parametric study are summarised as:

1. The internal stresses of KBF structures due to gravity loads and earthquake forces in most of beams and columns points are about 25 to 30 percent less than corresponding values in OMRF structures. It must be noted that in some parts of beams and columns, the shear stresses in KBF structures are more than OMRF structures. However in moment resisting frames , shear stresses are not as important as normal stresses.
2. In evaluation of stability characteristics , it is shown that KBF structures have larger load buckling multiplier and shorter effective lengths.
3. KBF structures have better ductility performance and better collapse mechanism patterns.
4. In number of case studies , it is shown that non earthquake resistant existing structures , and also damaged structures could be rehabilitated and repaired with very simple detailing using knee braced connections.

KEYWORDS

knee braced frames, ordinary moment resisting frames,ductility, collapse mechanism repairability, rehabilitation.

INTRODUCTION

In moment - resisting frames, the connections need be designed for vertical loads and/or for the combinations of vertical and prescribed seismic loads, with one third increase on the allowable stresses. So the most important weakness of these type of structures may be their connections which have to be carefully designed and properly constructed.

In this paper, knee braced frames (KBF), as illustrated in the Figure, are introduced as an appropriate alternative for ordinary moment resisting frames (OMRF). The effect of different parameters on behaviour of knee braced moment connection structures are studied and a convenient criteria for comparison of these frames with ordinary moment resisting frames are presented.

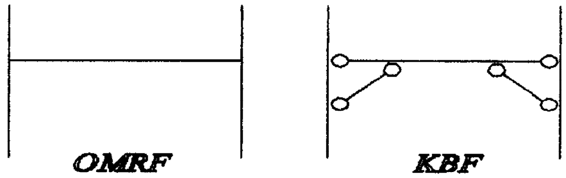


Fig. 1.

The advantages of KBF are:
1) Providing more open space in architectural design,
2) Transfer of forces and moments at the beam to column connections in wider areas,
3) Use of simple connections with less cost and simpler detailing, instead of rigid connections,
4) Reducing the effective length of columns,
5) Providing more ductility's at the connection regions,
6) Rehabilitation of existing low lateral load resisting structures are provided.
The disadvantages of KBF are:
1) Increasing shear and normal forces at beams in distances between knee bracing's and column faces.
2) The probable formation of plastic hinges in columns at the column - knee bracing connections.
In this paper, with detailed study of the above advantages and disadvantages, the feasibility of knee braced frames are evaluated.

ELASTIC BEHAVIOUR OF KBF COMPARED WITH OMRF

Designing a structure to resist the expected loading is generally aimed at satisfying established or prescribed safety and serviceability criteria. The important aspects of a good design is that the structure has to have sufficient strength in order to withstand external loads and its own weight, without collapse. Besides, the structure should have such a stiffness not to have excessive deformations under imposed vertical and horizontal loads. So in this section, strength and stiffness of knee braced frames (KBF) and ordinary moment resisting frames (OMRF) are evaluated and compared.
In this study a steel frame with rigid beam to column connection is considered and it is assumed that the main objective is to provide a structure with a complete ideal moment resisting connections. As an alternative, it is tried to introduce a tentative connection as shown in the Figure 2, in order to model the behaviour of rigid connections.

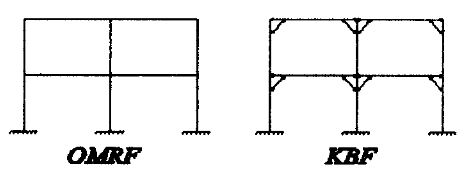


Fig. 2.

In order to evaluate the advantages and disadvantages of the modelled connection, a parametric study with the following assumptions has to be carried out:
1) The structures under consideration are plane frames in which the number of bays varies from 1 to 5 and number of stories from 1 to 10,
2) The frames have equivalent span lengths which varies from 3 to 6 meters with 0.5 meter intervals and equivalent story height of 3.3 meter,
3) The distances from knee bracing ends to beam ends and column ends varies from 0.1 to 0.15 of length of beam or column respectively,with 0.01 interval,
4) Dead loads and live loads on frame beams are uniformly distributed with an intensities of 3.1 ton/m, and 1 ton/m respectively,

5) seismic loads on the frames are considered according to Iranian 2800 standard ,
6) The following load combinations are considered

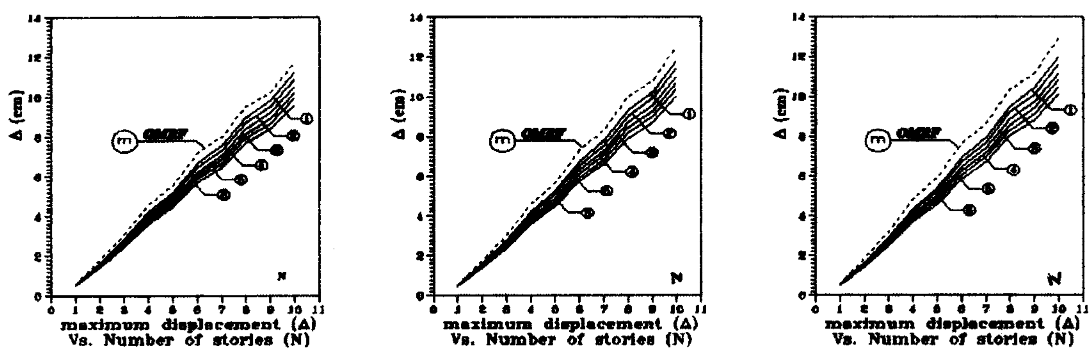
c1= dead load + live load
c2=0.75 (dead load + live load + seismic load)
c3= 0.75 (dead load + live load - seismic load)

Considering the above physical assumptions for the parametric study, a pre-processor, and a post processor, compatible with the SAP90 for systematic data inputting are prepared. First, the ordinary m oment resisting frames are analysed and designed according to AISC specification. In order to compare the differences between KBF and OMRF, the member properties found in OMRF, are used for analysis of KBF. Furthermore the knee bracing's are designed under their axial forces.
To compare the behaviour of KBF and OMRF , the following factors are evaluated.

a) Maximum lateral displacement of frames under seismic load,
b) Internal forces induced in beams , columns, and knee bracing members.

a) Comparison of maximum lateral displacement of KBF and OMRF

Maximum lateral displacement of KBF and OMRF for all the above cases have been calculated and the results have been compered. Typical results for lateral displacement are illustrated in following Figures. In the figures, $k_1 . l$ and $k_2 . h$ are distances from knee bracing ends to beam ends and column ends respectively in which (l) is the beam length and (h) is the column height.



No. of bays = 3, span lengths = 4 meters | No. of bays = 2, span lengths = 5 meter | No. of bays = 4, span length = 5.5 meter

m= OMRF
KBF(1:K1=K2=0.1/2:K1=K2=0.11/3:K1=K2=0.12/4:K1=K2=0.13)
(5:K1=K2=0.14/6:K1=K2=0.15)
Fig.3.

b) Comparison of internal forces of KBF and OMRF

b-1) Bending moment in beams: maximum bending moment have been computed for all the loading cases and are typically tabulated in following table;

Table 1. Comparison of maximum bending moment in beams in KBF and OMRF,
(two bay story frame with l=5m)

At story	M in (OMRF)	M in (KBF)	Extra moment capacity in (KBF)
1	15.45 t.m	8.38 t.m	46 %
2	16.23 t.m	8.88 t.m	45 %
3	14.90 t.m	8.04 t.m	46 %
4	13.35 t.m	6.86 t.m	49 %
5	10.82 t.m	5.23 t.m	52 %
6	9.84 t.m	4.71 t.m	51 %

b-2) Axial forces in beams: Results obtained from the pararametric analysis show that maximum values of axial forces in beams of KBF are increased slightly. This is due to truss action of the connection in KBF. Although rigidity of floor diaphragm would contribute to the transfer of in plane forces induced in floor levels.

b-3) Shear forces in beams: As it was predicted , the shear forces in beam of KBF have been dramatically reduced in service gravity loading condition. This is because, the bracing members, as shown in Figure 4, act as elastic support for beams. However, in condition of gravity and seismic loading combination, considerable differences have been observed as shown in Figure 5. the direction of shear forces in beam ends could be upward which implies that the most convenient shear connection may be a simple framed connection at beam webs.

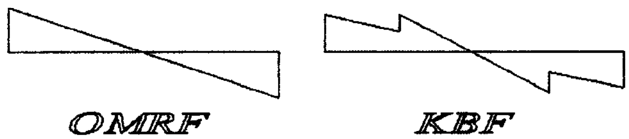


Fig. 4.

The values of shear forces in beams of KBF between bracing member conjunction and beam end have increased about 10-25%.

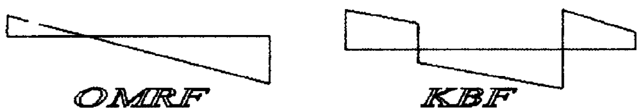


Fig. 5.

However, because the beams are designed due to bending moments, increase of up to 25% in shear forces usually is not the main weakness, although they have to investigated.

b-4) Axial forces in columns: Results have shown that axial forces in column of KBF are reduced up to 15%. Besides the deflected shape of columns show more convenient performance of columns with respect to effective length.

b-5) Shear forces in columns : As in beams , the values of shear forces in columns of KBF between bracing member conjunction and column end have increased considered considerably almost in all loading combinations. However because of large shear capacity of columns, this increase is not a problem for columns at all.

COMPARISON OF STRUCTURAL STABILITY OF KBF AND OMRF

In this section, the lateral stability of two structural system, i.e. KBF and OMRF, are compared in terms of effective length factor of their columns.

Since the lateral displacements of KBF in all cares, are always less than their corresponding OMRF , it seems that first buckling loads of KBF are greater than OMRF.

To assure that, consider an ideal column without geometrical or material imperfections such as member AB with elastic supports R1, R2 , and R5 as shown in Figure 6.

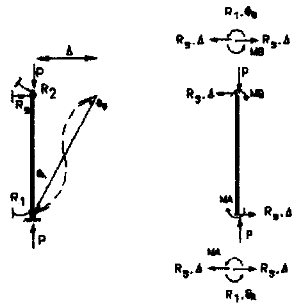


Fig. 6.

Equilibrium equations of the end forces are:

$MA + R_1 . \theta_A = 0$

$MB + R_2 . \theta_B = 0$

$MA + MB + P . \Delta - R_S \Delta l = 0$

Now if we rewrite the above equations in a matrix form, we obtain

$$\begin{Bmatrix} MA \\ MB \\ Hl \end{Bmatrix} = \frac{EI}{l} \begin{bmatrix} s + R_1' & cs & -s(1+c) \\ cs & s + R_2' & -s(1+c) \\ -s(1+c) & -s(1+c) & 2s(1+c) + R_s' - \alpha^2 \end{bmatrix} \begin{Bmatrix} \theta_A \\ \theta_B \\ \Delta / L \end{Bmatrix}$$

in which

$$R_1' = \frac{R_1 . l}{EI} \qquad R_2' = \frac{R_2 l}{EI} \qquad \alpha = l\sqrt{\frac{P}{EL}} \qquad R_s' = \frac{R_s l^3}{EI}$$

$$S = \frac{\alpha^2 \cosh\alpha - \alpha \sinh\alpha}{2 - 2\cosh\alpha + \alpha\sinh\alpha} \qquad C = \frac{\alpha - \sinh\alpha}{\sinh\alpha - \alpha\cosh\alpha}$$

Structural instability occurs when determinant of coefficient matrix become zero, or

$$R_s' = \frac{[R_1' + R_2' + 2S(1-C)](1+C)^2 S^2}{(R_1' + S)(R_2' + S) - S^2 C^2} - 2S(1+C) + \alpha^2$$

With algorithm explained above number of examples both for KBF and OMRF have been performed. Results have shown that buckling load multiplier for KBF are about 20 to 25 percent greater than corresponding values in OMRF. Study of local instability analysis have also indicated that KBF are in better than OMRF. Following example is a typical stability analysis of two structural systems.

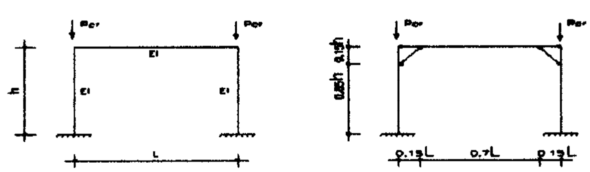


Fig.7.

Beams and columns	EI = constant
Beams	EA= ∞
Columns	EA = 3000 EI / l^2
Bracing members :	EI $= \frac{1}{3}$ EI (BEAMS)
	EA =700 EI / l^2
	h = 0.66 l

Results obtained for above structural systems are:

Ordinary moment resisting frame: $Pcr = 6.578 \frac{EI}{h^2} = \frac{\pi^2 EI}{(k_m h)^2} \Rightarrow k_m = 1.225$

Knee bracing frame: $Pcr = 8.32\frac{EI}{h^2} = \frac{\pi^2 EI}{(k_z h)^2} \Rightarrow k_z = 1.089$

In which K and K are effective length coefficient of OMRF and KBF respectively:

DUCTILITY PERFORMANCE OF KBF COMPARED WITH OMRF

For most structural systems, particularly those consisting of rigidly connected frame members and other multiply redundant structures, economy is achieved by allowing yielding to take place in some critically stressed elements under relatively strong earthquakes. This means designing a structure for force levels significantly lower than would be required to ensure a linearly elastic response. Analysis and experience have shown that structures having adequate structural redundancy can be designed safely to with stand strong earthquakes even if yielding take place in same part of structure. In order to inelastic deformation take place in structures which have been designed to such reduced force levels, an additional requirement has to be imposed for those structures, i.e., they much posses sufficient ductility.

Because of high redundancy of knee braced structures compared with corresponding ordinary moment resisting frame structure, it seems that KBF are in better condition than OMRF as far as the ductility is concerned. To assure that, using an algorithm based on "unit shape factor analysis" we compare ductility performance of two structural systems. In this procedure, it is assumed that inelastic deformations take place at plastic hinges and other region of structure remain elastic. Under this assumption we try to draw the lateral force versus lateral displacement of two structural system until the mechanism takes place. In Iranian seismic code, the base shear, V, resulted from seismic is defined as,

$$V = \frac{ABI}{R}W$$

in which

A = Seismic zone factor

B = Coefficient which is dependent on soil characteristics of the site and period of the structure

I = Importance factor

R = Coefficient the measure of ductility of structural systems.

In order to find the R factor for two structural system, the following assumptions are set:

1) Materials have similar behaviour in tension and compression,

2) The strains vary linearly through transverse section,

3) In elastic deformations take place at plastic hinges,

4) The structure has linear behaviour between two successive plastic hinges,

5) Interaction between compression force and bending moment of columns follows the following relation.

$$\frac{M}{M_P} + 0.604\frac{P}{P_y} + 0.439\left(\frac{P}{P_y}\right)^2 = 1$$

in which

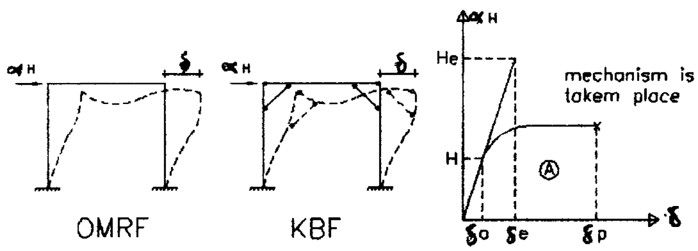
P = axial compression force of column

M = bending moment of column

P = Yield force of column

M = Plastic moment of column

By increasing the amount of lateral force on structure until plastic hinges take place in beams or column or yielding of knee bracing is achieved and finally the mechanism of structure is taken place. Drawing the added lateral forces versus lateral displacement as shown in Fig. 8., we have



$$R = \sqrt{\frac{2A}{\delta_0 H}}$$

Fig. 8.

Using the above procedure, here are some typical examples which shows that the knee braced frame structures have significantly performance than similar ordinary moment resisting frame structures.

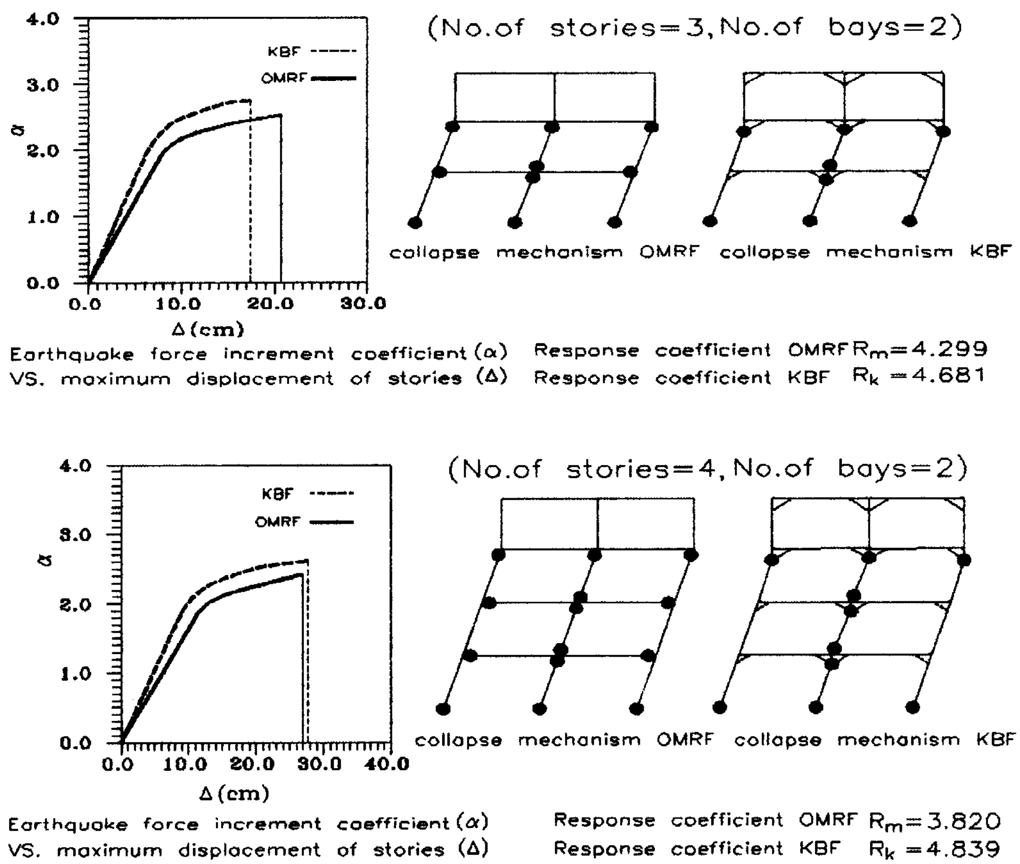
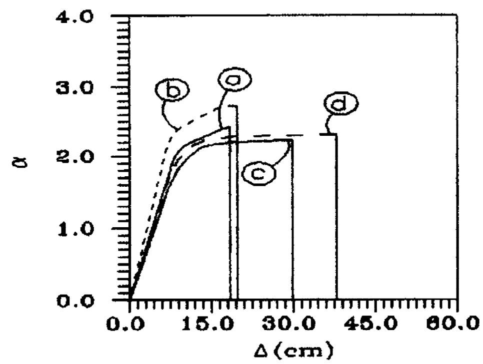


Fig. 9.

As discussed earlier the above comparison has been made by the fact that the member properties (beams and columns) of knee braced frames similar to their corresponding ordinary moment resisting frames. Figures 9. clearly show that, first mechanism pattern of KBF are reasonably the same as those of OMRF, which means that the two structural systems have similar behaviour. Secondly R factors of KBF structures are always greater than OMRF structures.

If the member properties of KBF (beams and columns) are selected based on the strength and stiffness criteria, the R factors obtained from incrementally linear analysis (as discussed above) have little changes but still greater than OMRF. Fig. 10. shows a typical result of these changes.



Earthquake force increment coefficient (α)
VS. maximum displacement of stories (Δ)

a) OMRF R = 3.86
b) KBF with member properties (beams and columns) similar to OMRF R=5.03
c) KBF with member properties (beams and columns) designed based on their own internal forces. R=4.80
d) KBF with columns properties similar to OMRF but beams properties designed based on their own internal forces R=4.41

Fig. 10. Variation increased seismic force versus maximum lateral displacement for a three bay - three story frame.

REHABILITAION OF EXISTING STRUCTURES WITH KNEE BRACING

Another feature of these structures are for strengthening existing structures which have either no lateral resisting element or those lateral resisting structures with poor technical detailing and they have been damaged during the earthquake.

As explained in the preceding sections, using knee bracing members at the corner of beams and columns, the beam - column joints become ductile moment resisting elements and may be capable of energy absorbing during the subsequent earthquake, Fig. 11. shows a typical detailing of strengthening a existing structures.

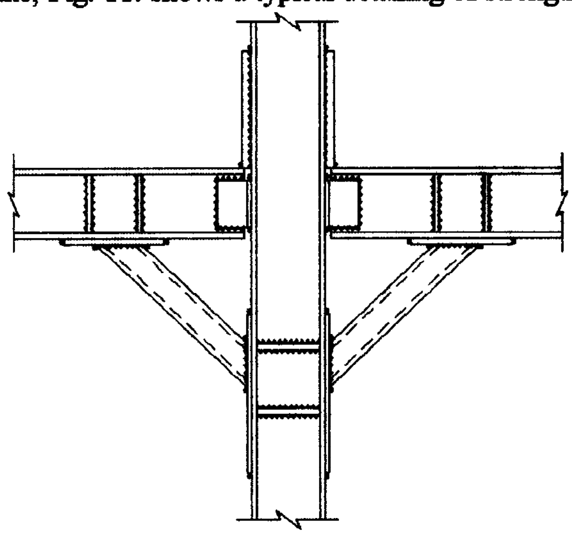


Fig. 11. Typical detailing for strengthening a existing structure.

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