Seismic performance evaluation of steel knee-braced moment frames

Bahman Farokhi[1], Alireza Bazvand[2]

[1]Student, ph.d of civil engineering, university of malayer , iran (corresponding Author)

Abstract: One of the earthquake lateral-force-resisting systems are steel moment frames which depending on their design, have good behavior in terms of ductility but limitations in terms of stiffness and displacements control. Recently a system is introduced by adding knee members to the conventional moment frame system in front of the flexural joints which acts as structural fuses and improves seismic behavior of the moment frame that is known as knee-braced moment frame. In this study, the seismic behavior of a conventional steel moment frame with knee braces is investigated by using dynamic time history analysis and nonlinear static analysis for three models of 3, 6 and 10 stories with different bay numbers. Results show that the overall overstrength factors are different from what the regulations have prescribed for moment frames. By comparing the behavior factor, stiffness and strength of the conventional moment frames with knee-braced moment frames and according to their improvements it is clear that this system can be a good alternative for the intermediate moment frames.

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Keyword: Steel moment frame, knee brace, time history analysis, over strength factors, nonlinear static analysis, behavior factor

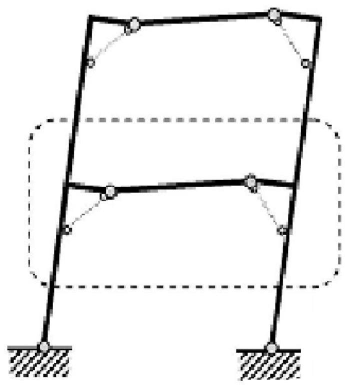
1. Introduction

In order to resist the lateral forces due to earthquake various lateral systems, including braced frames and moment frames are used, each of which in turn have their advantages and disadvantages. For example, moment frames do not create any difficulty for architectural considerations and have a stable hysteresis behavior, on the other hand, they are stiff and have little strength, and also their performance depends on execution and connections. In contrast, concentrically braced frames have great stiffness and strength but in terms of architectural considerations they have limitations on creating openings, also their ductility and energy dissipation depends on the post-buckling behavior of the brace members [1-2]. Knee braced frames was invented by Aristizabal [3] and was developed by analytical and experimental works of other researchers including Balendra et al [4], HUANG Zhen [5] and Mahmoud Miri et al [6]. Recently, a system called knee-braced moment frame (KBMF) was introduced by Leelataviwat [7] which has the main features of moment frames and braced frames. Design of the proposed system is based on capacity which results in the ductile design of the system. This system is designed so that under seismic loads the knee braces yield and buckle. This mechanism continues with the formation of plastic hinges in the beams, at their end where knee braces are attached to the beam. In this system, all beam-to-column connections and columns remain elastic. After earthquake, in the event of damage or destruction, these braces can be replaced and in terms of architectural considerations it is less restrictive than braced frames. In this study, 6 short-and-mid-level models with different bay numbers are designed according to the design method of KBMFs. Then in order to assess the seismic parameters of these models and using dynamic time history analysis and nonlinear static analysis methods, their seismic parameters including overall overstrength factor, behavior factor, stiffness and elastic strength are calculated.

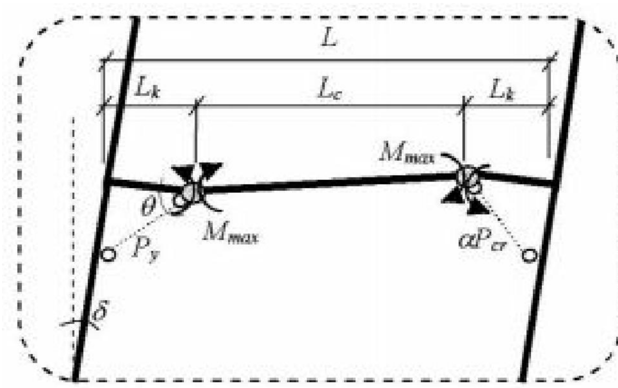
2. Design and modeling of the KBMF

2.1. Concept of knee-braced frame

The first step in designing a KBMF, is to design the base moment frame using a valid design regulation, after the determination of the beams and columns dimensions, the knee braces dimensions are determined so that the system after design and loading reaches the desired mechanism. Mechanism is the yielding of the knee braces and the formation of the plastic hinges in the intermediate beams and columns bases.



a) Desired mechanism



b) Details of the forces generated in the plastic hinges

Fig.1: Overview of the KBMF [7]

According to the design concept of this system which is the columns and the beam-to-column connections must remain elastic, the maximum moment in the beam-to-column connection should be less than a fraction of the beam plastic capacity. According to figure 1, the maximum moment generated in connection is calculated from equation 1.

(1) $$M_C = M_{max} + V_{max} L_k - \alpha P_{cr} L_k Sin(\theta)$$

Where $M_{max}$is the maximum moment in plastic hinge, $V_{max}$is the maximum shear in the plastic hinge and $\alpha P_{cr}$is the post-buckling strength of the knee brace. Other parameters are set according to the frame geometry. Due to the capacity of the beams, $M_{max}$and V_{max}values are obtained from equations 2 and 3, respectively.

(2) $M_{max}=\zeta M_P$

In this equation $M_p$is the beam plastic moment based on the expected yield stress and ζ is a coefficient greater than one.

(3) $$V_{max} = 2M_{max} / L_c$$

According to the above, equation 4 establishes the condition of elastic beam-to-column connection.

(4) $$Mc<\gamma M_p$$

In this equation γ is a numerical coefficient less than one. By substituting equations 1 to 3 in equation 4 and simplifying the result, we reach to the buckling load main equation for the knee brace, according to equation 5.

(5)
$$\frac{\alpha P_{cr}}{\frac{M_P}{L}} \geq \frac{(\zeta - \gamma)}{\left(\frac{L_K}{L}\right)\sin(\theta)} + \frac{2\zeta}{\left(1 - \frac{2L_K}{L}\right)\sin(\theta)}$$

Using equation 5, and considering appropriate values for dimensionless parameters of ζ, α and γ for each beam section, the design axial force of the knee member corresponding to that beam is obtained.

2.2. Design of the base moment frames

In this study, frames with 3, 6 and 10 stories with the bay length of 4m and the height of 3m in each direction are investigated. The design of all base moment frames under consideration is performed by using the commercial software ETABS version 9.7.4 [8]. All beams are IPE sections, all columns IPB sections and the box sections are used for knee element. The common steel material ST-37 with the yield stress of 2400Kg/cm^2 is used for the design. Floor dead load is 600Kg/m^2 and its live load is 200Kg/m^2. Construction site is considered to be Los Angeles California with SD_S=1.11 and SD_1=0.6, Design spectral acceleration parameters of SD_S and SD_1 are in periods of 1 and 0.2 seconds, respectively. The reason behind choosing this site is due to the recent major earthquakes like lomaprieta, Northridge and accurate information about their properties from several seismic stations. Thus, structural design is performed using regulations ASCE7-10 [9] and AISC360-2005 [10]. In this study, in order to design all the base moment frames, the conventional steel moment frame with Ru=3.5, Ω=3.0, seismic category C and ground type C in accordance with regulation ASCE7-10 has been used.

2.3. Design of knee members

After the design of the base moment frames and determination of beams and columns sections, knee members buckling load is calculated by using equation 5 and taking into account the parameters needed in Table 1 with the assumption that members both ends are hinges and using the method of designing the columns under net axial load, the corresponding knee members of each beam are designed by considering box sections.

Table 1: Needed parameters for the design of knee element

value	parameter
1.25	ζ
0.8	γ
1	α
4(m)	L
0.8 (m)	L_k

Table 2: dimensions of knee braces sections corresponding to each beam

Knee element cross section(mm)		Knee element buckling load(ton)	Beam section
	120*120*8	43.75	IPE33
	120*120*6	34.23	IPE30
	100*100*6	26.29	IPE27
	90*90*4	19.94	IPE24
	80*80*4	15.52	IPE22
	70*70*4	12.01	IPE20
	70*70*3	9.04	IPE18

2.4. Modeling with software

After the determination of sections in base moment frames and knee members, by using the finite element software SeismoStruct [11] which has the capability to consider the geometric and material nonlinearity and also performs a variety of linear and nonlinear, static and dynamic analysis; two-dimensional models of KBMFs are built and static and dynamic analysis are performed. Before any modeling, it is necessary that the model built with this software is verified, therefore, a valid experimental model [7] is simulated in this software and after loading similar to the experimental model, the results are compared. Figure 2 shows the comparison of hysteresis diagram for the experimental model and the model generated by the software; as can be seen the results are well matched, so this software can be used for modeling the desired frames with high precision. After modeling the KBMFs, the columns should be controlled for the additional force due to yielding of the knee braces. For this purpose by using the method of elastic analysis of columns, presented in reference [7], axial strength of the columns is calculated using AISC-360 and after the addition of knee elements the applied axial force is controlled so that the columns remain elastic during loading.

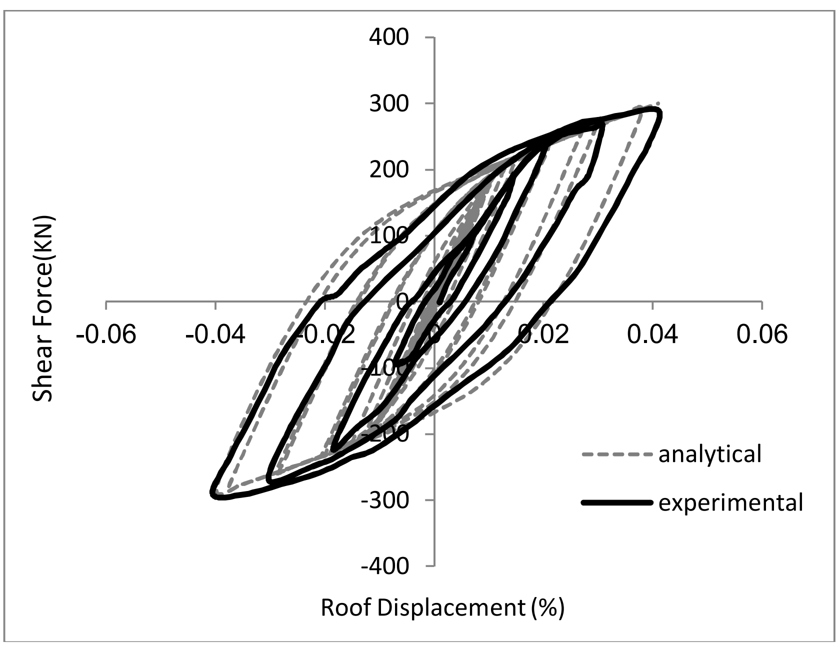


Fig.2: Comparison of hysteresis diagram for experimental and software model

2.5. Accelerograms used in the dynamic analysis

In order to perform dynamic time history analysis on the models under consideration, seven acceleration records from the events in California have been chosen and used. Accelerograms are scaled separately for the 3, 6 and 10 stories structures according to the method presented in ASCE-7 so that in the range of 0.2T to 1.5T the average accelerations spectrums be larger than the design spectrum of the considered site. In Table 3 name of the records and scale coefficients for each structure is shown separately.

Table 3: Name of the earthquake records and their scale coefficients

Record	PGA	Scale factor		
		3-story	6-story	10-story
lomaprieta-bran	0.45	1.42	0.98	0.79
lomaprieta-hall	0.2	3	2.9	2.1
lomaprieta-gillroy4	0.21	2.9	2.33	1.87
Northridge-hollywood	0.25	3	2.16	2.13
Northridge-sunvalley	0.45	2.1	1.66	1.72
Northridge-new hall	0.56	1.36	1.63	2.15
Northridge-la dam	0.5	1.95	1.58	1.34

For example, the modified and unmodified spectrums of accelerograms for 3-story structure, with average spectrum and design spectrum of ASCE-7 are shown in Figures 3 and 4, respectively.

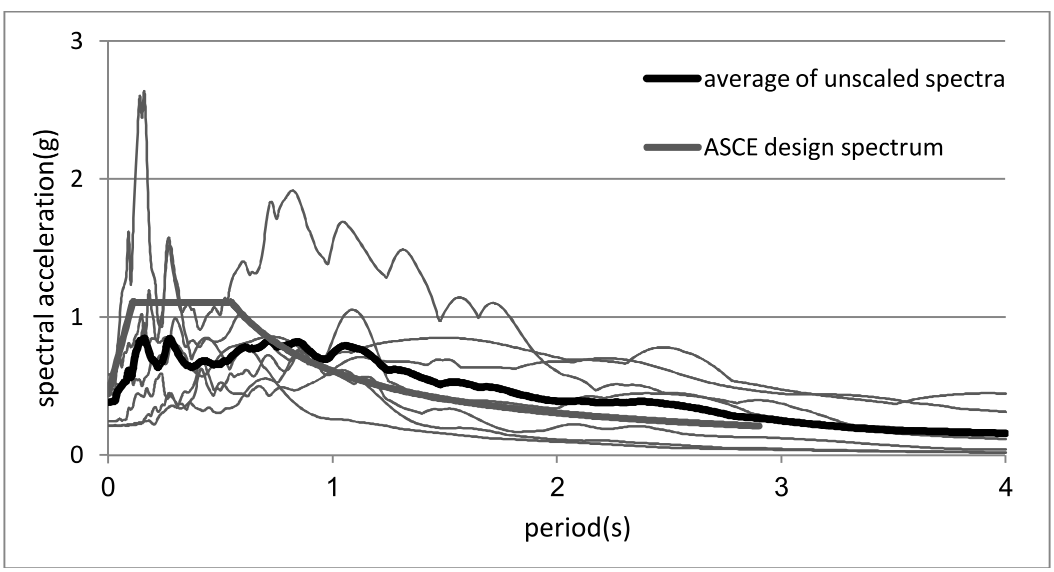


Fig.3: Unmodified spectrums and design spectrum of ASCE

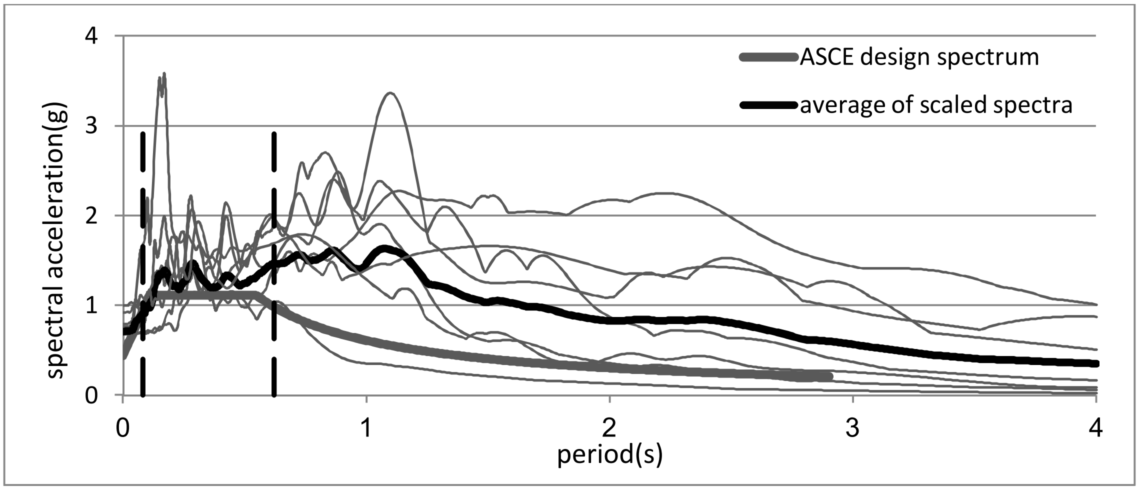


Fig.4: Modified spectrums for 3-story structure and design spectrum of ASCE

For example, in Figure 5, the 3-story, 3-bay plan of the structure is shown and in Figure 6, the details of the base moment frame and the corresponding software model of KBMF is illustrated.

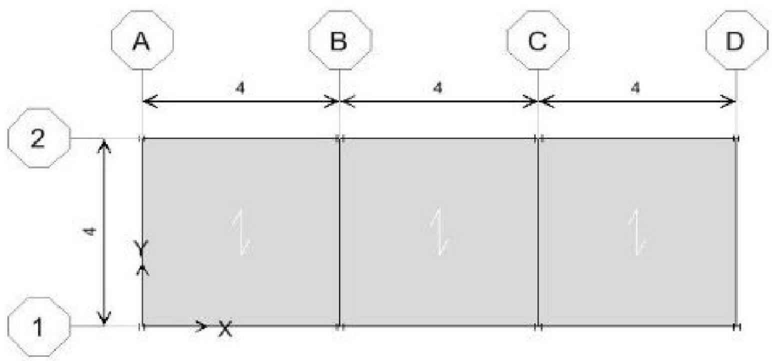


Fig.5: Plan of the 3-bay structure with bay length of 4m

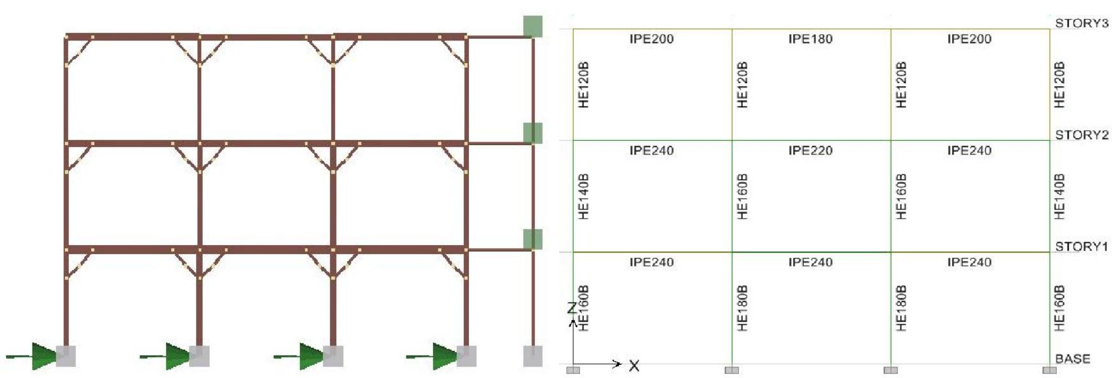


Fig.6: Base moment framed and KBMF for 3-story, 3-bay model

3. The results of models analysis

3.1. Overall overstrength factors

After performing nonlinear dynamic time history analysis for models built, based on the analysis results for each model the frames overall overstrength factors are calculated. The average base shear of the earthquake records is considered to be the base shear due to the dynamic analysis is compared with the resulted base shear of the equivalent static analysis and their ratio which is the overall overstrength factor of the frames is presented in Figure 7. The overall overstrngth factors are considered important due to their direct involvement in calculating the behavior factors of the frames and since the dynamic time history analysis results are highly accurate, to calculate the behavior factor of models according to 2.3, its overstrength is used. It is noteworthy that in this Figure the models names is shown with numbers so that from left to right, the first number shows the number of stories and the second one shows the bay numbers; for example: 6-3 means model with 6-story, 3-bay.

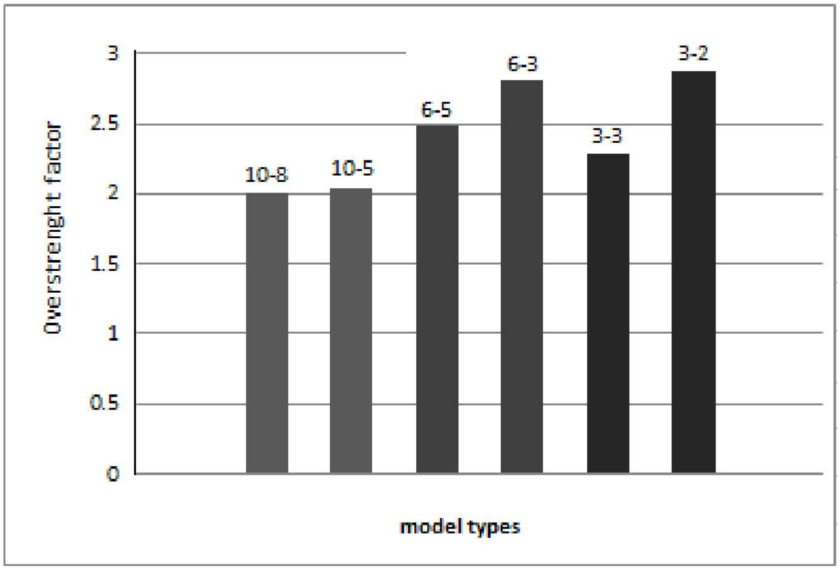


Fig.7: Overall overstrength values of models

According to that the overall design overstrngth factor for conventional moment frames is 3, comparison of the values of KBMFs in Figure 7 shows that by adding knee members a lower overstrength factor compared to the value proposed by the regulation is obtained. In the case of 3-story frames, the average overstrength value is obtained as 2.58 and its value decreases by increasing the bay numbers. In the case of 6-story frames the average overstrength value is obtained as 2.62 and this value also decreases by increasing the bay numbers. In 10-story models the average overstrength value is shown as 2.01 which by increasing the bay numbers almost remains constant.

3.2. Nonlinear static analysis

From the most important results that can be extracted from nonlinear static analysis (pushover) one can refer to elastic stiffness, secondary stiffness, strength (capacity), ductility factor, overstrength factor and behavior factor of structures. In this study, in order to calculate the seismic parameters of KBMF models simple pushover analysis is performed by using SeismoStruct [11]. According to the regulation FEMA-356 [12] all the models are pushed to the point of collapse prevention limit (CP) andfor calculating ductility factor, lateral safety limit (LS) is considered for maximum deformation. In order to bilinearize the capacity curve of structures a software is used which satisfies the condition of equality of areas under the actual curve and the bilinearized one. For example, the capacity curve ofbase moment frame, KBMF and performance points of LS and CP for model 3-3 are shown in Figure 8.

3.2.1. Calculation of behavior factor (Ru)

In the force-based seismic design methods, behavior factor reduces the elastic base shear of the structure to the design base shear. In fact by using the inelastic capacity of the structures, the base shear is reduced, but proper methods and detailing must provide the desired behavior factor. In this study, in order to assess the behavior factor of KBMFs and to compare their hysteresis behavior with the conventional base moment frames which were designed with the behavior factor of 3.5, after bilinearizing the capacity curves of the models and calculating the ductility factor according to equation 6, the behavior factor of knee models is calculated.

$$R_u = \Omega_o * R_\mu \quad (6)$$

In equation 6, Rμ is the force reduction factor due to ductility which is derived according to the proposed method of Miranda [13] and Ω0 is the overall overstrength factor of the frames which is calculated according to the results of the dynamic time history analysis. In Figure 9, the behavior factor for different models is shown. As it can be seen the obtained values of behavior factors are larger than those for the conventional and intermediate moment frames, thus with the addition of knee members to the conventional moment frame the behavior factor is improved and became larger than that of the intermediate moment frame. Another set of parameters that can be obtained using nonlinear static analysis are stiffness and elastic strength which can be a criterion for evaluating the models under consideration. By using the capacity curves of the models and comparing their stiffness and elastic strength values, as shown in Table 4, by adding knee members to the base moment frame it can be seen that the stiffness and strength of the frames are remarkably increased; so that the increase in the average stiffness of the models is more than 90% and in the average strength of the models is about 50%.

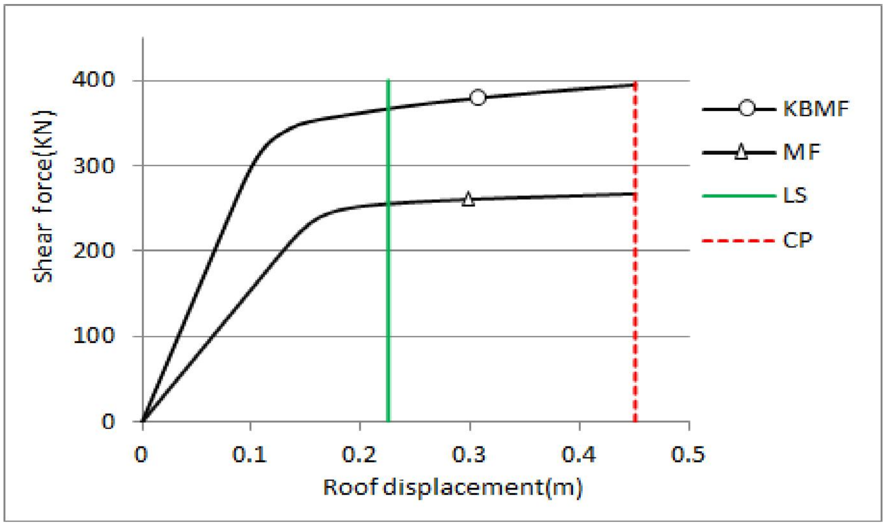


Fig.8. Capacity curve comparison for base moment frame and KBMF for model 3-3

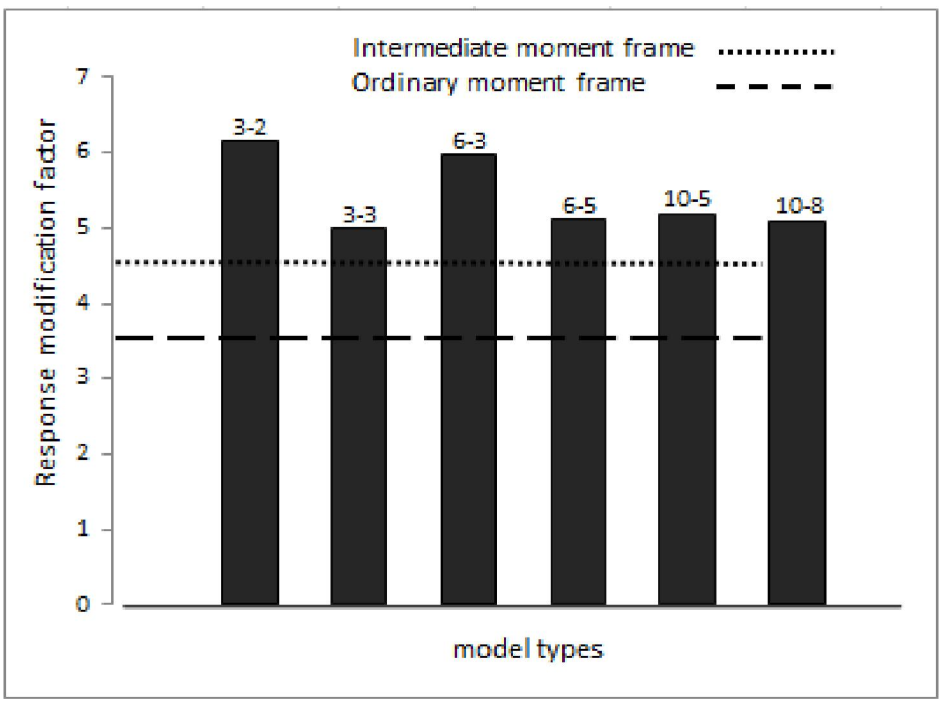


Fig. 9: Comparison of behavior factor for KBMFs and conventional and intermediate moment frames

Table 4: Comparison of stiffness and strength of the moment frame and KBMF models

Increase in		KBMF		M-F		Model name
Strength(%)	stiffness(%)	Strength(KN)	stiffness(KN/m)	Strength (KN)	stiffness (KN/m)	
37.9	92.7	251	2250	182	1186	3-2
39	94.8	349	3034	251	1557	3-3
48.4	93	536	2401	361	1244	6-3
48.3	94.4	847	3708	571	1907	6-5
56.9	89.9	1360	3794	852	1997	10-5
55	90	2110	5982	1361	3143	10-8

4. Conclusions

1. The overall overstrength factor for KBMFs is obtained between 2 to 3 and is reduced by increasing the number of stories; so that this factor for 10-story frames is equal to 2. While the proposed value in the regulation is an estimation of theoverstrength in frames and is equal to 3.
2. Comparison of the design behavior factors of base moment frames with KBMFs indicate that this system by using ductile knee elements can provide larger behavior factors than intermediate moment frames (about 5) and can be a good alternative for the intermediate moment frame.
3. On average, the strength of the KBMFs models compared to the base moment frames is about 50% and their stiffness is more than 90%. Due to the significantincrease in stiffness, knee elements can be used as a retrofitting method in steel frames in order to control the drift.

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