Knee brace가 설치된 모멘트저항골조의 내진성능

Seismic Performance of a Knee-Braced Moment Resisting Frame

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Abstract

In this study the seismic performance of a three-story knee-braced moment-resisting frame (KBMRF), which is typically employed to support pipelines for oil or gas, was investigated. Nonlinear static pushover analyses were performed first to observe the force-displacement relationship of KBMRF under increasing seismic load. The results show that, when the maximum inter-story drift reached 1.5 % of the story height, the main structural members, such as beams and columns, still remained elastic. Then nonlinear dynamic time-history analyses were carried out using eight earthquake ground motion time-histories scaled to fit the design spectrum of UBC-97. It turned out that the maximum inter-story drift was smaller than the drift limit of 1.5 % of the structure height, and that the columns remained elastic. Based on these analytical results, it can be concluded that the seismic performance of the structure satisfies all the requirements regulated in the seismic code.

요　지

본 연구에서는 기름이나 가스의 송유관을 지지하기 위하여 일반적으로 사용되는 3층의 knee brace가 설치된 모멘트저항골조(KBMRF)의 내진성능을 평가하였다. KBMRF의 하중-변위 관계를 관찰하기 위하여 비선형 정적 pushover 해석을 수행하였다. 최대층간변위가 층높이의 1.5%에 도달하였을 때 보와 기둥과 같은 주요 구조부재는 탄성상태를 유지하는 것으로 나타났다. UBC-97의 설계스펙트럼에 부합되도록 조정한 8개의 지진기록을 이용하여 비선형 동적시간이력해석을 수행한 결과에 따르면, 최대층간변위는 구조물 높이의 1.5% 변위한계보다 작았고 기둥은 탄성적으로 거동하였다. 따라서 본 연구에서 고려한 KBMRF 구조물의 내진성능은 내진설계기준에서 규정한 모든 요구사항을 만족하는 것으로 나타났다.

Keywords : Knee-braced moment resisting frame, Seismic performance, Inelastic behavior
핵심 용어 : Knee brace가 설치된 모멘트저항골조, 내진성능, 비탄성 거동

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1. Introduction

Generally concentric braced frames, except those designed as the special concentric braced frames, are not considered desirable in high seismic region because the buckling of compressive braces soon lead to sharp drop in lateral stiffness. Especially K-type bracing is prohibited in most design codes because the large unbalanced force due to the difference between the forces of the yielded brace and the buckled brace may cause column failure. Although no code provision that prevents the use of knee-braced moment resisting frame (KBMRF) system exists, there is a disagreement on the safety of KBMRF in high-seismic region, because of the similarity in shape with the K-braced system.

KBMRF is different from the K-brace system due to the following reasons:

1) Compared with K-braces, the size of knee braces is very small; therefore the forces transferred from braces to columns are generally not large enough to cause plastic deformation in columns.

2) In K-braced frames, the braces are designed to resist most of the lateral force. Beam-column joints are often pinned. Therefore the failure of some braces may lead to total collapse of the structure. In KBMRF, however, the moment frame with rigid beam-column joints may resist significant portion of lateral force even after all the braces failed.

According to previous research on KBMRF with various shapes carried out by Hsu and Jean[1], the addition of knee braces resulted in significant enhancement of strength. In this study the seismic performance of a three-story knee-braced moment-resisting frame(KBMRF), typically employed in Middle East to support pipelines for oil or gas, was evaluated. The lateral load-resisting system of the structure was designed in accordance with the seismic provision of UBC-97[2]. Nonlinear static pushover analyses were performed first to observe the force-displacement relationship under increasing seismic load. Based on the nonlinear static parametric study, it was concluded that the performance of KBMRF was superior to that of typical moment frames.

In this study more detailed study on the seismic performance of the typical KBMRF was carried out. Both nonlinear static and dynamic analyses were employed to investigate the load-carrying capacity for monotonically increasing lateral load and to check the stability of the structure under design-level earthquake ground excitations. $P-\Delta$ effect was considered throughout the study.

2. Analysis Model

The analytical model structure is the 2-D knee-braced moment-resisting frame (KBMRF) shown in Fig. 1 which is originally a part of a long structure composed of identical 2-D structures.

The lateral load-resisting system was designed to meet the seismic provision of UBC-97[2], and all the structural members are made of A36 steel. The weights of the 1st, 2nd, and 3rd stories are 25.77, 32.52, and 28.40 tonf, respectively, and the first two natural periods are 0.52 and 0.13 seconds.

The gravity loads acting on each story are described in Fig. 2.

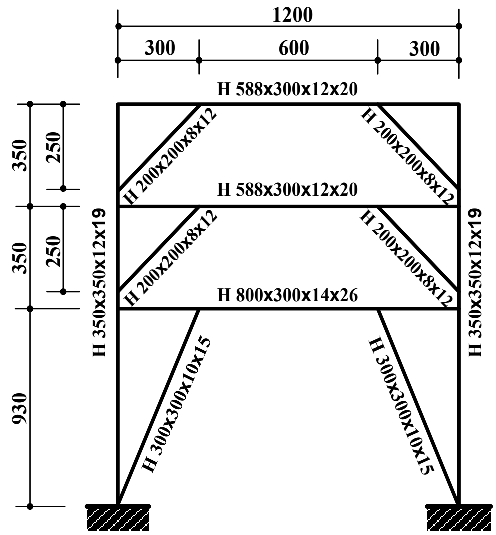
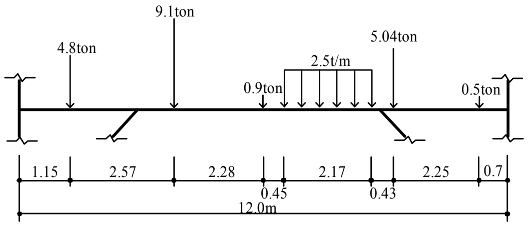
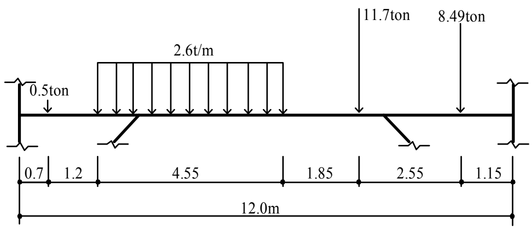


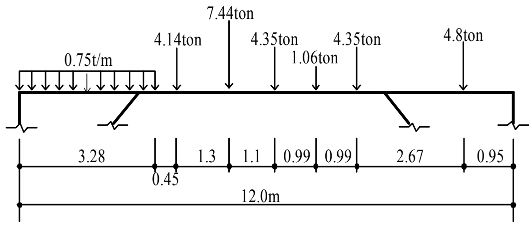
Fig. 1 Knee-Braced Moment-Resisting Frame



(a) 2nd story



(b) 3rd story



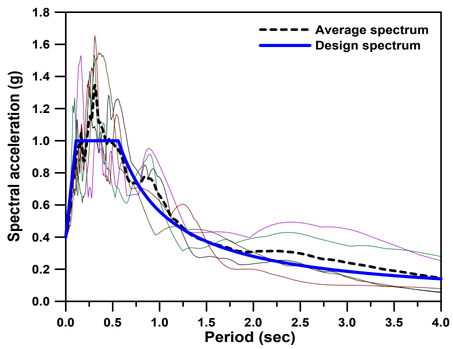
(c) Roof story

Fig. 2 Gravity loads in each story

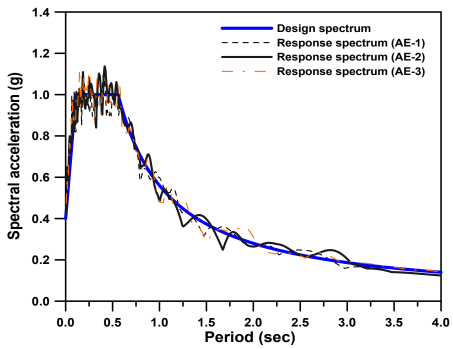
3. Earthquake ground motions for dynamic analyses

The earthquake records used in the dynamic time-history analyses are composed of three artificial records generated using the program code SIMQKE[3] and of five ground motions recorded near Los Angeles, USA[4]. All the records are scaled to fit the UBC design spectrum with the seismic coefficients $C_a = 0.4$ and $C_v = 0.56$. The soil type is S_c.

Fig. 3 compares the response spectra of the earthquake records used in the analyses and the UBC-97 design spectrum. Fig. 4 and 5 illustrate the time-histories of the records, which show that earthquake records with various characteristics were used in the analyses.

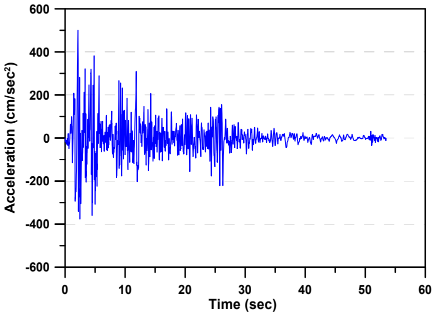


(a) Recorded earthquake

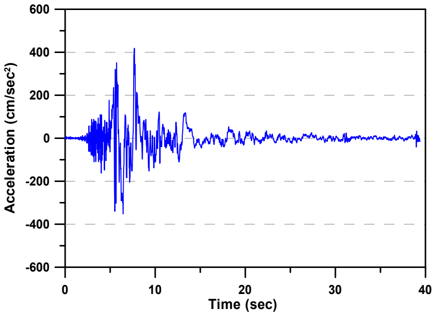


(b) Artificial records

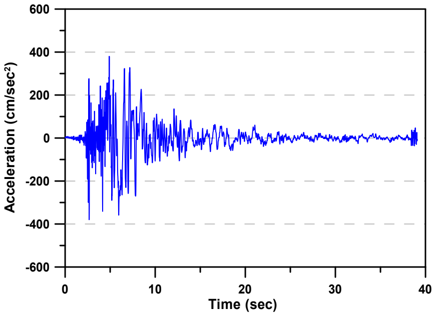
Fig. 3 Comparison of response spectra and design spectrum



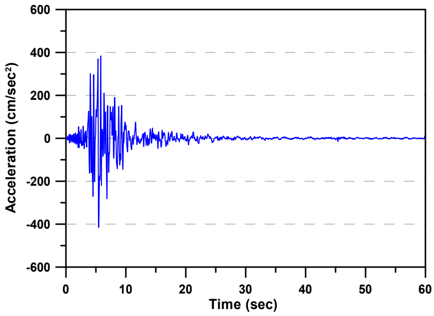
(a) EQ-1



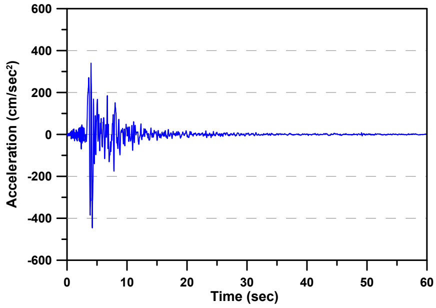
(b) EQ-2



(c) EQ-3

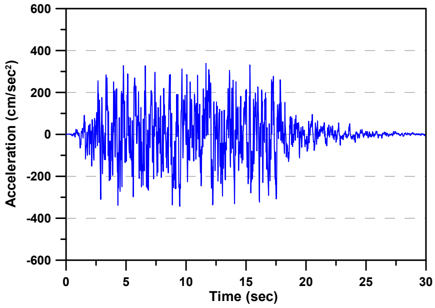


(d) EQ-4

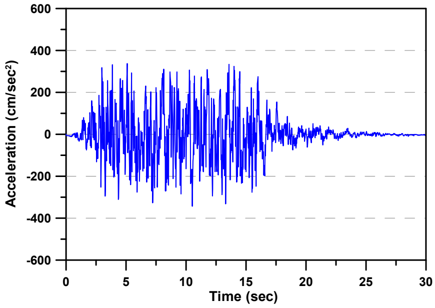


(e) EQ-5

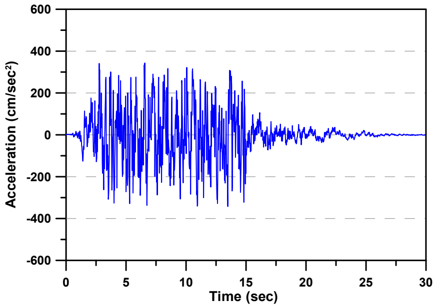
Fig. 4 Time-histories of recorded earthquakes



(a) AE-1



(b) AE-2



(c) AE-3

Fig. 5 Time-histories of artificial earthquake records

Table 1 Earthquake records used in the time-history analyses

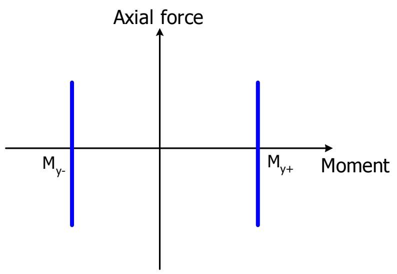
Name	Record	Distance (km)	Duration (sec)	PGA (cm/sec^2)	Scaled PGA (cm/sec^2)
EQ-1	Imperial Valley, 1940, El Centro	10	39.38	662.88	499.99
EQ-2	Imperial Valley, 1979, Array #05	4.1	39.38	386.04	418.45
EQ-3	Imperial Valley, 1979, Array #06	1.2	39.08	230.88	380.1
EQ-4	Northridge, 1994, Newhall	6.7	59.98	664.93	415.39
EQ-5	Northridge, 1994, Sylmar	6.4	59.98	801.44	446.19

4. Seismic performance evaluation

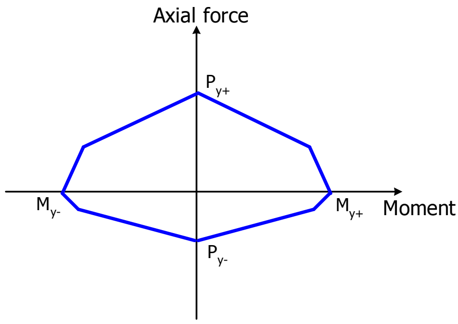
For evaluation of seismic performance of the model structure, both nonlinear static pushover analyses and dynamic time-history analyses were employed. The pushover analyses were carried out using the program code Drain-2DX[5], and the time-history analyses were performed using the SNAP-2DX developed in University of Michigan[6].

4.1 Modeling of inelastic behavior

The beams and columns were modeled to have point plastic hinges with bi-linear load-displacement relationship. The post-yield stiffness was assumed to be 2 % of the elastic stiffness. The yield characteristics of the plastic hinges in beams and columns are described in Fig. 6. The inelastic load-displacement relationships of the knee-braces,

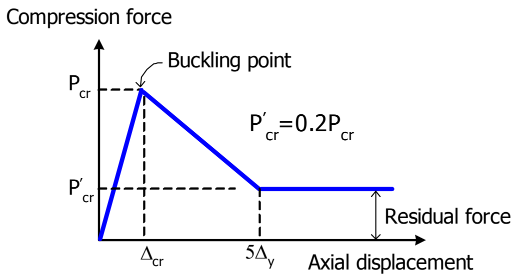


(a) Beams

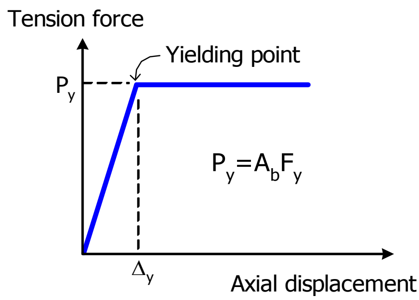


(b) Columns

Fig. 6 Yield characteristics of beams and columns



(a) Braces under compression



(b) Braces under tension

Fig. 7 Load-displacement relationship of knee braces both in tension and compression, are modeled as shown in Fig. 7, which are recommended in FEMA-274 report[7]. The post-yield stiffness of the braces was assumed to be zero. Both in static and dynamic analyses, $P-\Delta$ effect was included.

4.2 Nonlinear static pushover analyses

Pushover analyses were carried out to identify the performance of the structure in each loading stage. The gradually increasing lateral seismic loads proportional to the fundamental mode shape were enforced, and the load-displacement relationship was plotted in Fig. 8. The maximum inter-story drifts of 1/150 and 1.5% of the story height were also shown in the figure. The points that the stiffness of the

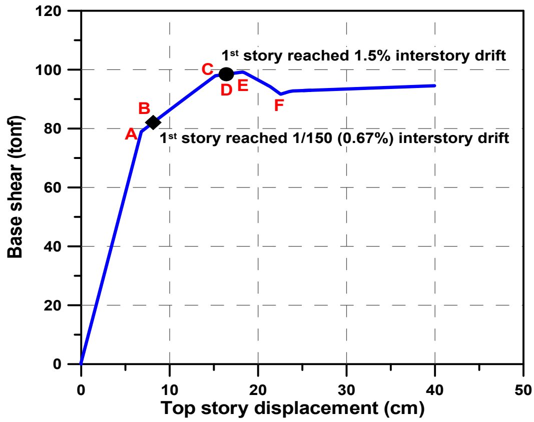
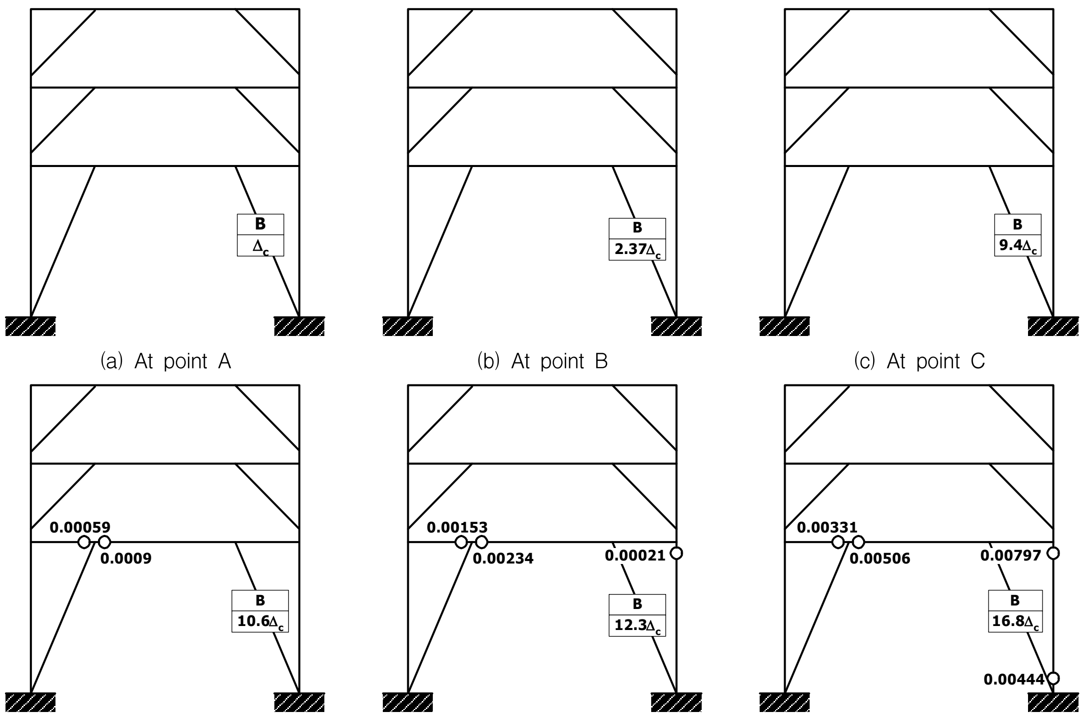


Fig. 8 Pushover curve of the model KBMRF structure

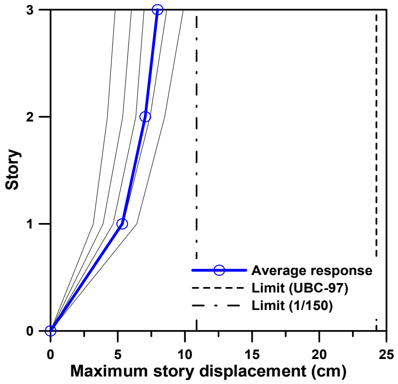
structure changes are marked on the curve as A~F.

Fig. 9 presents the inelastic deformation of structural members at points A~F. The letter B on

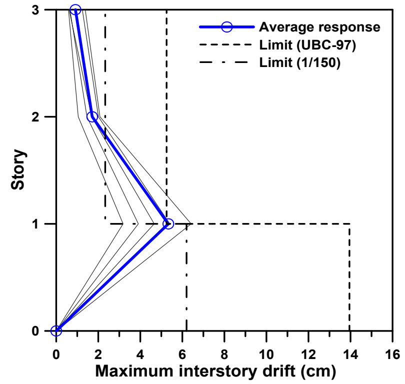


(a) At point A
(b) At point B
(c) At point C
(d) At point D
(e) At point E
(f) At point F

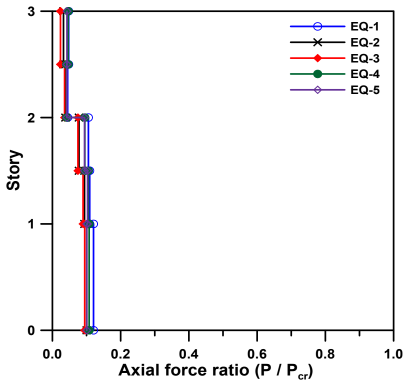
Fig. 9 Plastic hinge formation at various stages



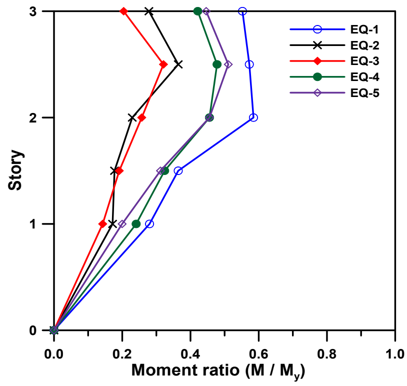
(a) Maximum story displacements



(b) Maximum inter-story drifts



(c) Maximum axial force in columns



(d) Maximum moment in columns

Fig. 10 Responses of the structure subjected to the recorded earthquakes

the brace denotes that the brace was buckled, and $\hat{}_c$ represents the axial deformation at buckling. The size of plastic hinges formed in the beams and columns is denoted in radian. The point A corresponds to the buckling of the first-story brace. It can be observed that in this stage the beams and columns remain elastic. At 1.5 % of the inter-story drift (point D), which is the code-specified limit state for lateral displacement, plastic hinges form at the floor beams in the second floor. However the size of the plastic hinges is too small to cause global instability of the structure. It also can be noticed that the locations of the beam plastic hinges are shifted to the brace-beam connection, which prevents the possibility of brittle joint failure. This observation matches with the results obtained by Hsu and Jean[1].

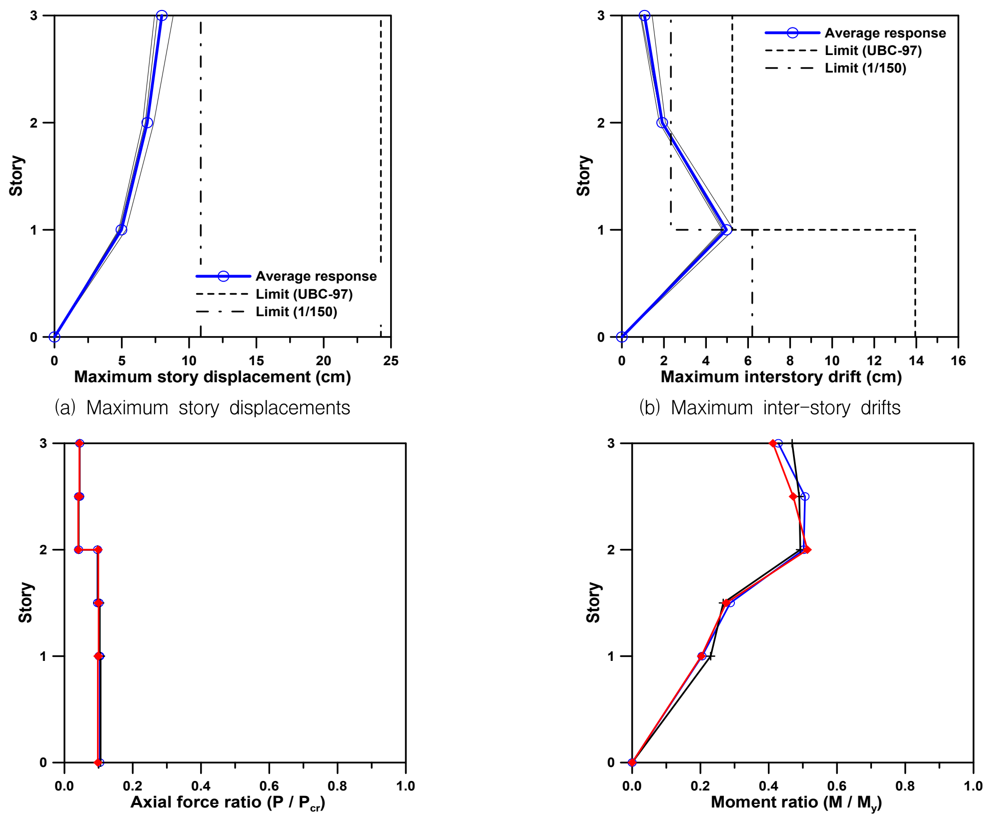
The results of the pushover analyses prove that the structure retains enough strength to remain stable in large displacement far exceeding the limiting state.

4.3 Time-history analyses

Nonlinear time-history analyses were carried out using the recorded and the simulated earthquake records, and the results were shown in Fig. 10 and 11. The analysis results show that the maximum inter-story drift and the roof-story displacement are significantly smaller than the limit state of 1.5 % of the height specified in UBC-97. Also the maximum axial forces in the columns are less than 10 % of the critical load, and the maximum bending moments range 15%~58% of the yield moment. Therefore columns possess large residual strength for the design level earthquakes. Fig. 12 plots the buckled braces and the maximum axial deformation of the braces. It can be observed that no plastic hinges formed in beams and columns, while most of the knee braces buckled. The axial deformation of braces is less than the FEMA-356[8] specified failure state of 5.0 times the buckling deformation, Even though all the braces failed, the structure would remain stable because the rigidly connected beams and columns are in elastic state.

5. Conclusions

In this study seismic performance of a knee-braced moment resisting frame was investigated using nonlinear static and dynamic analyses. According to the results most of the damages associated with buckling or plastic deformation were concentrated in knee-braces, and the beams and columns remained elastic under the design load. These



(a) Maximum story displacements

(b) Maximum inter-story drifts

(c) Maximum axial force in columns

(d) Maximum moment in columns

Fig. 11 Responses of the structure subjected to the artificial earthquakes

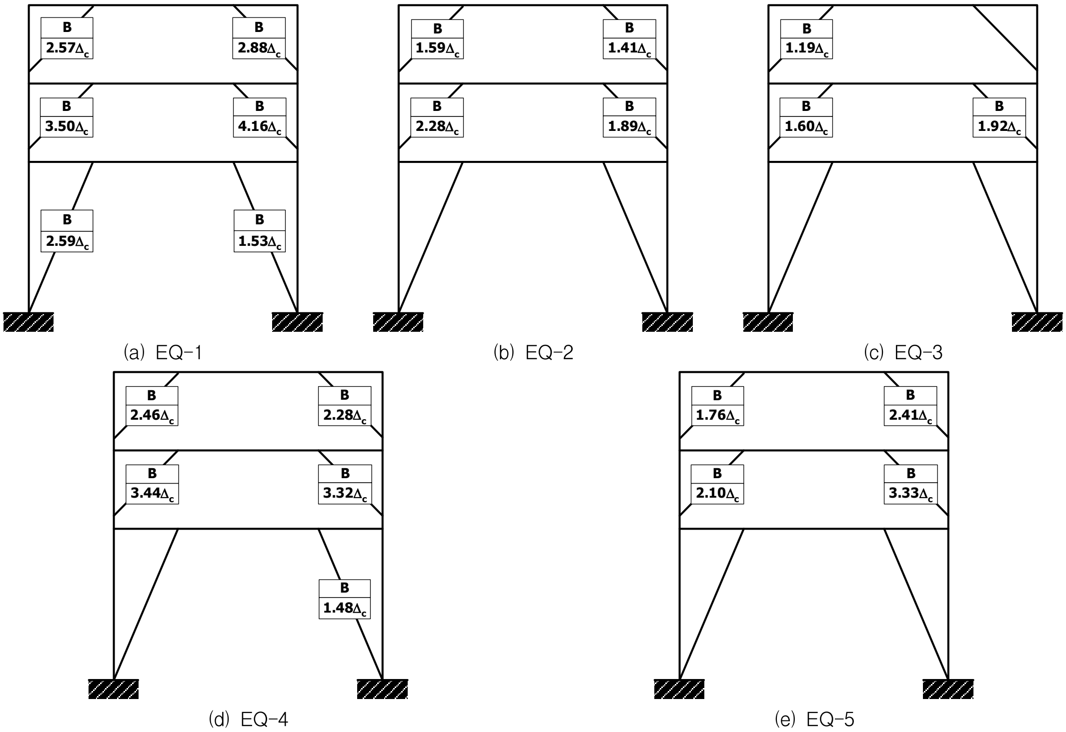


Fig. 12 Buckled braces and their maximum axial deformation for recorded earthquakes

observations are quite different from those observed in K-braced frames in which the buckling of the compression braces generally leads to sudden failure of columns due to large unbalanced force. Therefore it can be concluded that the structure will be stable with large residual strength when it is subjected to the design level earthquake. The KBMRF turned out to be quite effective in resisting earthquake load in the sense that mostly knee braces are damaged while the main structural members, the beams and columns, remain elastic. Also the possible brittle failure of moment frames, which were frequently observed in Northridge and Kobe earthquakes, can be prevented by employing knee-braces because the beam plastic hinges occur in brace-beam connections, not in beam-column joints.

Acknowledgement

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