Performance-Based Seismic Design and Evaluation of Buckling Restrained Knee Braced Truss Moment Frames

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SUMMARY:
Performance evaluation of an innovative structural steel framing systems called Buckling-Restrained Knee Braced Truss Moment Frame (BRKB-TMF) was carried out. This structural system harnesses the advantages of open-web steel truss girders and Buckling Restrained Braces (BRBs). Key advantages of open-web trusses include light weight, simple connections, and open passages for mechanical ductwork and pipes. In this system, the open-web trusses are designed to be elastic while BRBs are strategically placed and designed to dissipate seismic energy. The combined features of the open-web trusses and BRBs lead to a system with enhanced performance, safety, and economy. A performance based design procedure has been developed for the proposed system. A four story building structure was selected as a study case. The structure designed by the presented procedure was subjected to nonlinear static (pushover) and dynamic analyses. The pushover analysis was done to determine the overall response, the sequence of inelastic activity leading to collapse, and the failure mechanism. In the nonlinear dynamic analyses, the study frame was subjected to a suite of selected earthquake records scaled to represent various levels of earthquake ground motion intensity. Incremental dynamic analysis approach was applied to examine the behavior of the structure at different levels of ground motion intensity all the way up to the collapse level. The analyses provided very promising results in terms of the effectiveness and robustness of the system. The example structure showed low probability of collapse under the maximum considered earthquake (MCE) ground motions. The key design parameters were found to be the target drift and deformation capacity of the BRBs.

Keywords: Truss Moment Frames, Buckling Restrained Braces, Performance-Based Plastic Design, Incremental Dynamic Analysis, Collapse Evaluation

1. INTRODUCTION

Open-web steel truss moment frames are very economical and are commonly used in building frames especially in long-span and industrial structures. Key advantages of open-web trusses include light weight, simple connections, and open passages for mechanical ductwork and pipes. However, under extreme load events or accidental overloading, conventional truss girders may lack proper ductility which can lead to sudden and catastrophic failures. This shortcoming is caused mainly by buckling of the diagonal elements due to compressive forces (Goel and Itani 1994a). As a result of experimental and analytical research carried out in the 1990s, Special Truss Moment Frame (STMF) system was developed in order to enhance the inelastic deformation capacity of truss girder frames (Goel and Itani 1994b). This system is currently recognized as a seismic resistant system in the AISC seismic provisions (AISC 2010). The system uses ductile special segments designed to dissipate seismic energy.

In this study, the performance of truss moment frame system is further enhanced by using buckling restrained braces (BRBs). An innovative structural system called Buckling-Restrained Knee Braced Truss Moment Frame (BRKB-TMF), illustrated in Figure 1, is proposed. The BRKB-TMF system combines features of truss moment frame and BRBs. In this system, the open-web trusses and columns are designed to remain elastic while BRBs are strategically placed and designed to dissipate seismic energy. The design of BRKB-TMF in this research is based on a design procedure called Performance-Based Plastic Design (PBPD) approach. The PBPD method uses pre-selected target drift and yield mechanism as key performance limit states. The PBPD method accounts for structural inelastic behavior directly and minimizes the need for any assessment or iteration after an initial design. The PBPD method has been developed and validated for many conventional structural systems such as moment frames, eccentrically and concentrically braced frames, and special truss moment frames (Goel and Chao 2008). The combined features of PBPD method and BRKB-TMF concept lead to a structural system with enhanced performance, safety, and economy.

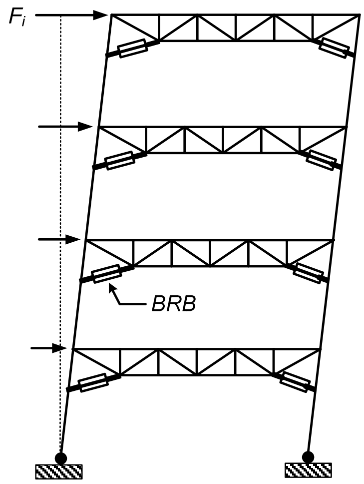


Figure 1. Buckling-Restrained Knee Braced Truss Moment Frame system.

In this paper, the PBPD design concept of the BRKB-TMF system is first introduced. To verify the performance of the proposed BRKB-TMF system, a four story frame structure was selected as a study case. The design of the structure was carried out and nonlinear static and dynamic analyses were performed to determine the overall response, the sequence of inelastic activity leading to collapse, and the failure mechanism. In the nonlinear dynamic analyses, the study frame was subjected to a suite of selected earthquake records scaled to represent various levels of earthquake ground motion intensity. Incremental dynamic analysis approach was applied to examine the behavior of the structure at different levels of ground motion intensity all the way up to the collapse level. The analysis results are presented and discussed.

2. PERFORMANCE BASED PLASTIC DESIGN OF BRKB-TMF

2.1 Target Drift and Design Base Shear

The design of BRKB-TMFs in this study is based on the PBPD approach. This method directly accounts for inelastic behaviour and considers the internal force distribution at ultimate limit state (Goel and Chao 2008).The design concept uses pre-selected target drift and yield mechanism as key performance limit states. The required design base shear is derived corresponding to a target drift level and a selected yield mechanism using the energy balance concept (Lee and Goel 2001). Plastic (limit) design is then used to design the structure to achieve the selected mechanism.

The PBPD method begins by selecting a target yield mechanism with a set of designated yielding members (DYMs). For BRKB-TMF systems, the selected mechanism consists of yielding of the BRBs and the plastic hinging at the base of the columns as illustrated in Figure 1. A target drift corresponding to a chosen hazard level is then selected. The target drift depends on the performance objective. It is selected mainly to limit system and element ductility demands to desired limits. To ensure satisfactory behavior, the inelastic deformation expected to occur in the BRBs in a severe earthquake should not exceed the inelastic deformation capacity of the BRBs. This can be done by choosing an appropriate value for the target drift to limit the deformation demands of the BRBs.

Inelastic deformation demand for a BRB can be calculated approximately from the target drift by assuming that the system deforms in a rigid-plastic manner after the mechanism has formed. Neglecting elastic deformations in the frame members, the plastic deformation of a BRB can be computed based on the truss configuration along with the law of cosines (Figure 2). For a special case where the depth of the truss at the center of the column is chosen to be twice the depth of the truss at mid-span ($D = 2D_o$), the plastic strain in the BRB, ε, simply becomes

$$\varepsilon = \delta / l_o = \theta_p D \sin(\varphi) / l_o \tag{2.1}$$

in which θ_p is the target plastic drift of the frame, l_o is the undeformed length of the BRB, φ is the angle the first diagonal member makes with the column, and D is the depth of the truss at the face of the column.

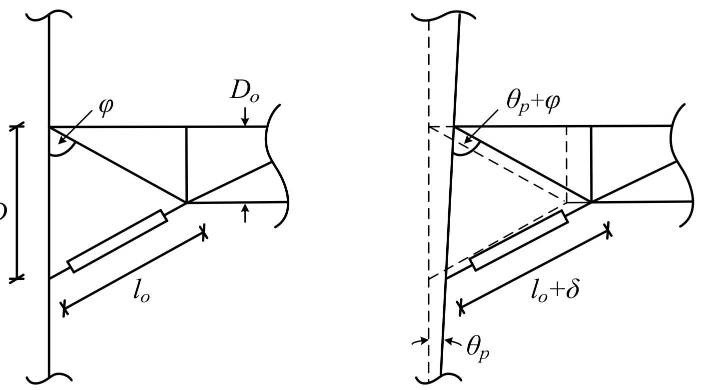


Figure 2. Plastic deformation of BRB

Numerous uniaxial and subassemblage BRB tests have been performed in recent years (Lopez and Sabelli 2004). Based on several of these test results, the deformation capacity of a BRB in terms of maximum brace strain appears to be in the range of 2% to 3% depending on the length and configuration of the BRB. Knowing the deformation limit of the BRBs, Equation 2.1 can be used to select an appropriate value of the target drift as well as the truss configuration. Once these are determined, the required strength of the system, or design base shear, for a selected hazard level is calculated using energy balance concept, i.e., by equating the work needed to push the structure monotonically up to the target drift to that required by an equivalent elastic-plastic single degree of freedom system to achieve the same state (Lee and Goel 2001). It can be shown that the required base shear, V, is given by

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2} \tag{2.2}$$

where W is the weight of the structure, C_e is normalized design pseudo acceleration (S_a/g), γ is the energy modification factor defined as the ratio between the work needed to push the structure up to the target drift and elastic input energy, and α is a parameter given by

$$\alpha = \left(\sum_{j=1}^{n} \lambda_i h_i \right) \frac{\theta_p 8\pi^2}{T^2 g} \tag{2.3}$$

in which T is the period, and h_i is the height from the ground to floor level i, and λ_i is the lateral force distribution factor such that

$$F_i = \lambda_i V \tag{2.4}$$

In general, the lateral force distribution should closely represent that which occurs during inelastic response under earthquake ground motions. In this study, a distribution proposed by Choa and Goel (2007) for steel moment frames is used and is given by:

$$\lambda_i = (\beta_i - \beta_{i+1})\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} \tag{2.5}$$

where w_n is seismic weight at the top level n, h_n is the height from ground to the top level, and β_i is ratio of the story shear at level i to that at the top story (level n). For $i = n$, β_{i+1} =0. An empirical equation for β_i is given by

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \tag{2.6}$$

Once the design base shear and lateral forces have been determined, the required strength of the BRBs and the truss members can be calculated.

2.2 Member Design

Principle of virtual work on the yield mechanism is used to determine the required strength of the BRBs (designated yielding elements). The relative strength of the BRBs at each floor level is initially assigned based on the ratio of the story shear, β_i, given by Equation 2.6. Using the plastic mechanism in Figure 1 and assuming that the tension and compression forces generated by the BRBs at each floor level are equal, the virtual work equation (for one bay) can be written as

$$\sum_{i=1}^{n} F_i^{'} h_i \theta_p = 2M_{pc}\theta_p + \sum_{i=1}^{n} 2(\beta_i N_{BRB})\delta \tag{2.7}$$

where $F^{'}_i$ is the lateral force per bay at level i, N_{BRB} is the axial strength of the BRB at the roof level, and M_{pc} is the plastic moment of the columns at the base. The above equation applies to BRKB-TMF with one bay, however, it can be easily extended to cover a multi-bay structure. By assigning the values for the plastic moment of columns in the first story, the required strength of BRBs at each level ($\beta_i N_{BRB}$) can be calculated. One possible approach is to assign the value of the plastic moment of the columns to prevent soft-story mechanism, that is

$$M_{pc} = \frac{1.1V^{'} h_{c1}}{4} \tag{2.8}$$

where $V^{'}$ is the required base shear per bay, h_{c1} is the clear height of the first story. The factor 1.1 is used to account for the possible strain-hardening in the plastic hinges. The above approach has been found to provide adequate column strength leading to acceptable seismic performance for many structural systems designed by the PBPD method (Goel and Chao 2008). The required BRB strength at each level is given by (AISC 2010)

$$\phi P_{ysc} = \beta_i N_{BRB} \tag{2.9}$$

After the sizes of the BRBs have been determined, the trusses are designed to remain elastic under the largest forces generated by the BRBs. The adjusted strengths for a BRB (AISC 2010) accounting for material overstrength, compression overstrength, and strain-hardening are given by

$$P_{pr}^{+} = \omega R_y P_{ysc} \tag{2.10}$$

for tension, and

$$P_{pr}^{-} = \omega \beta_o R_y P_{ysc} \tag{2.11}$$

for compression. In Equations 2.10 and 2.11, ω, β_o, and R_y are factors accounting for strain hardening, compression overstrength, and material overstregth respectively. R_y has a value of 1 if the yield stress is determined based on coupon test. The values for ω and β are generally best assigned based on test results for BRBs with similar length, configuration, and restraining mechanisms to those that will be used in the structure. Using the backbone curve from a test, one can extract the strength adjustment factors ω and β_o comparable to the level of deformation demands expected to occur in the BRBs. It is important to emphasize that, because the BRBs in BRKB-TMF system are generally short, the deformation demands experienced by the BRBs will generally be larger than those expected for BRBs in conventional braced frames. Therefore, the values for ω and β_o for BRKB-TMF system will be larger than those used for the design of conventional BRB frames.

Once the sizes of the BRBs have been determined, the truss at each level is designed to remain elastic mainly under the gravity loads and the adjusted BRB forces at that level. In addition, because the truss is normally connected to the column by welded gusset plates, the moment generated by the fixity of the top chord connection should also be taken into account. These end moments create additional flexural forces in the chords and axial forces in the vertical members. The truss is thus subjected to the forces as shown in Figure 3. In the figure, $M_{p\text{-}ch}$ is the plastic moment of the top chord and the factor 1.2 is used to account for possible strain-hardening. It should be noted that these plastic moments are neglected in the virtual work equation described earlier in Equation 2.7 because the energy dissipated by these plastic hinges is significantly smaller than that by the BRBs. However, it can locally affect the truss member forces and has to be included in the truss analysis. The analysis of the truss under the given loads can be easily carried out by hand or by computer.

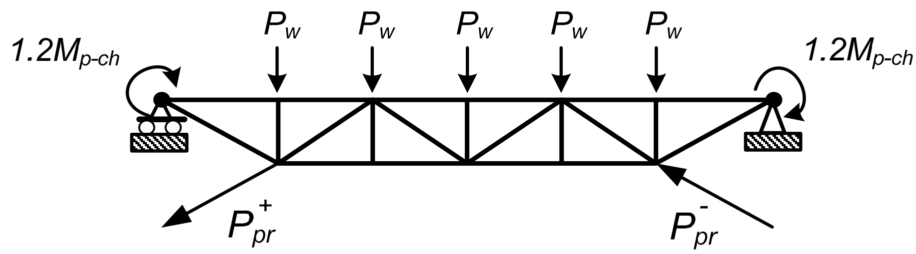


Figure 3. Truss design concept

The columns in BRKB-TMF systems are also designed to remain elastic except at the bases where plastic hinges are required to complete the yield mechanism. To do so, the columns are designed to resist the adjusted BRB forces given by Equations 2.10 and 2.11 and the forces generated by the truss members connected to the columns. Based on the PBPD approach, a capacity design method that considers the equilibrium of the entire column subjected to forces generated by the BRBs and the trusses can be used to design the columns. Alternatively, a pushover analysis can be carried out up to the expected displacement demand level assuming elastic columns. The forces obtained from such analysis can then be used in the design of the columns.

3. ARCHETYPE STRUCTURE

An example BRKB-TMF structure was selected to study the seismic performance of the system. This archetype structure is selected in such a way that it broadly represents a typical application and characteristics of the proposed structural system. The chosen structure was a four-story frame originally designed by Goel and Chao (2008) as a moment resisting frame. Important factors used to calculate the design forces were S_1 = 0.6g and S_s = 1.5g, Seismic Use Group I, Soil type D, and an estimated period of 0.94 sec. The frame was redesigned as BRKB-TMF using the PBPD approach presented earlier. For the PBPD method, multiple hazard levels can be considered, each with a different performance target drift. For this example frame, two levels of ground motion intensity were considered, the maximum consider earthquake (MCE) level and 2/3MCE level. The governing design base shear from the two hazard levels was then used to design the frame to ensure that the performance will be satisfactory in both hazard levels. For this frame, the maximum target drift was selected as 3.0% for MCE level and 2% for the 2/3MCE level. With an assumed yield drift of 0.75%, this results in target plastic drifts, θ_p, of 2.25% for MCE level and 1.25% for 2/3MCE level. The governing design base shear coefficient (V/W) calculated by Equation 2.2 was 0.154. The elevation view of the BRKB-TMF is shown in Figure 4 along with the floor masses. The design lateral forces for the entire building, F_i, and the distribution factors, β_i, for each floor level are also shown in Figure 4. The member sizes are summarized in Table 1.

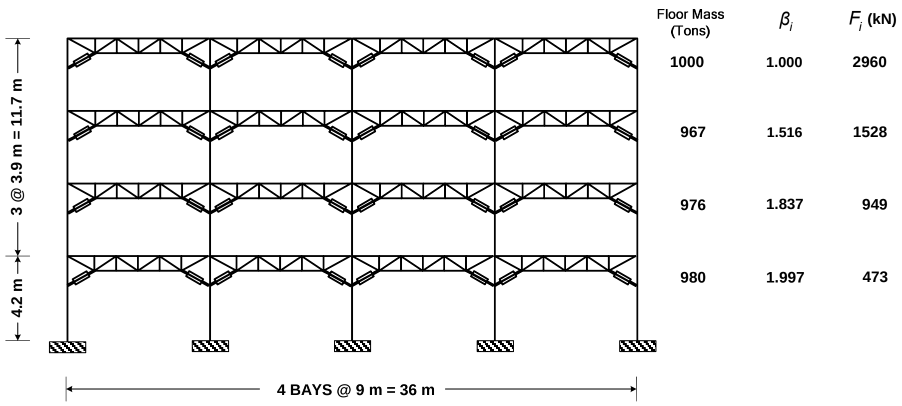


Figure 4. Example structure

Table 1. Summary of member sizes

Floor Level	Truss		BRB Capacity (kN)	Story	Column	
	Chord	Diagonal			Exterior	Interior
Roof	2MC100x20.5	2C150x15.6	460	4	W610x174	W610x285
4	2MC150x24.3	2MC150x17.9	700	3	W610x174	W610x285
3	2MC180x28.4	2C180x22.0	850	2	W610x262	W610x341
4	2MC200x31.8	2MC150x22.5	920	1	W610x262	W610x341

Note: All vertical members are L89x89x7.9 except the outermost vertical members are 2L L89x89x7.9

4. PEFORMANCE ASSESMENT

Performance assessment was carried out using inelastic static (pushover) analysis and incremental dynamic analysis (IDA). The pushover analysis was done to determine the overall response, the sequence of inelastic activity leading to collapse, and the failure mechanism. The IDA approach was used to examine the behavior of the structure at different levels of ground motion intensity all the way up to the collapse level. The procedure is a relatively new analytical tool utilizing a large number of nonlinear dynamic analyses under varying levels of ground motion intensity to systematically investigate the response of the structure (Vamvatsikosa and Cornell 2004). In this study, the ground motions and IDA were applied according to FEMA P695 methodology (FEMA 2009). A total of 44 ground motions were used. Their spectra with the median spectral acceleration value at the fundamental period of the frame scaled to match the design value at the MCE level are shown in Figure 5. Statistical analyses were performed on the IDA results to obtain the probability of collapse and the fragility curves for the structure.

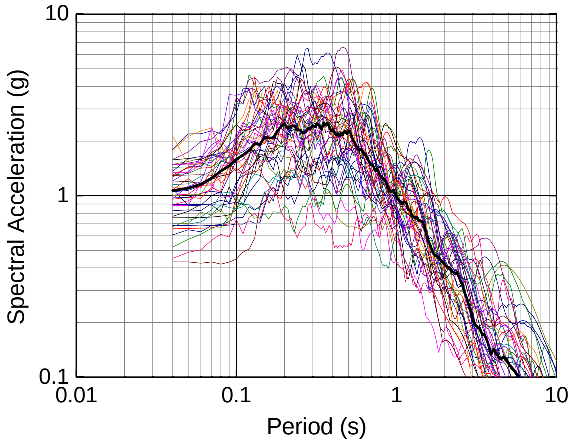


Figure 5. Response spectra of ground motions used in this study (FEMA 2009)

A 2-D analytical model was created to represent the frame. The model included P-Δ effect and "gravity" columns. The force-deformation characteristics of the columns and truss members followed the ASCE 41-6 (2006) recommendations. For the columns, a collapse criterion corresponding to a plastic rotation limit of 7% was assigned. This rotation limit was used as an indication for the onset of collapse. The force-deformation characteristics of the BRBs were calibrated based on existing test results (Merritt et al. 2003). A core strain limit of 4% was assigned for the BRBs. The fracture of the BRBs was modelled by a sudden strength drop with only a minimal residual strength. It is important to note that the loss of one or a small number of BRBs does not necessarily mean that the loss of gravity load carrying capability or that collapse would occur. However, it does lead to a significant increase in the deformation of the columns. In these analyses, the collapse was deemed to have occurred mainly when the plastic rotation of a column reached the limit. All of the analyses were carried out using PERFORM 3D computer program (CSI 2007).

The pushover curve is shown in Figure 6 with the sequence of inelastic activities indicated on the plot. The response of the frame was elastic up to a drift level 0.5% when the first set of BRBs became inelastic. The peak strength occurred at a story drift of 1.9%. P-Δ effect became apparent beyond this drift level as can be seen from the gradual strength reduction. At 3.6% drift, a set of BRBs fractured and the frame experienced severe strength drop. Beyond this drift, the frame had only a modest lateral load resistance. As the loading continued, the plastic rotations of columns at the bases and the plastic rotations of the truss top chords finally reached the rotation limit. It is apparent from the pushover results that the fracture of the BRBs signifies the impending collapse of the frame. It is therefore crucial to select the target drift that is compatible with deformation capacity of the BRBs. Overall, it can be seen that the presented PBPD procedure results in the frame that had all the inelastic activities confined to only the designated elements.

The results from the IDA are shown in Figure 7. One of the goals of FEMA P695 methodology is to assess the collapse capacity of the frame. The collapse capacity is expressed in terms of the collapse margin ratio (CMR) which is defined as the ratio between the median spectral acceleration of the collapse level ground motions (S_{CT}) and the spectral acceleration of the MCE ground motions S_{MT}. From the IDA results, the CMR for the example frame was found to be 1.56. The adjusted collapse margin (ACMR) ratio taking into account the spectral shape (FEMA 2009) was found to be 2.19. The fragility curves computed from the IDA results are shown in Figure 8. As can be seen, the probability of collapse for the MCE ground motions falls below the generally acceptable value of 10%. The results indicated that the story drifts reached between 6%-7% before the collapse occurred. The failure pattern was typically the fracture of a set of BRBs quickly followed by excessive rotation of the plastic hinges in the columns. For the columns, the critical plastic hinges were mainly located at the bases except in a few ground motions where they were located elsewhere. Although only one structure was investigated in this study, the low probability of collapse strongly shows the robustness of the proposed system.

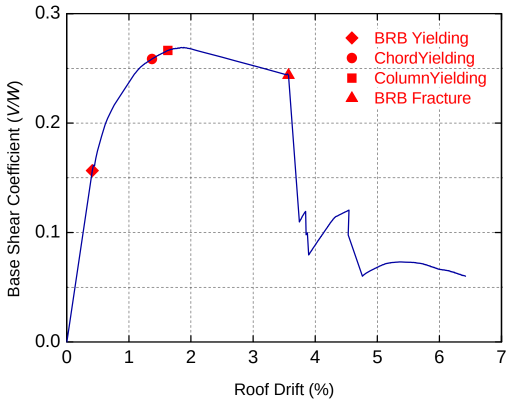


Figure 6. Base shear versus roof drift plot from Pushover Analysis.

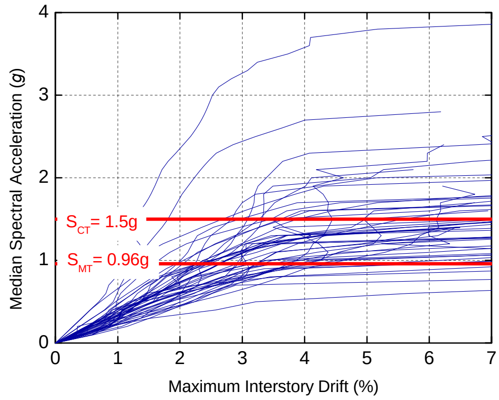
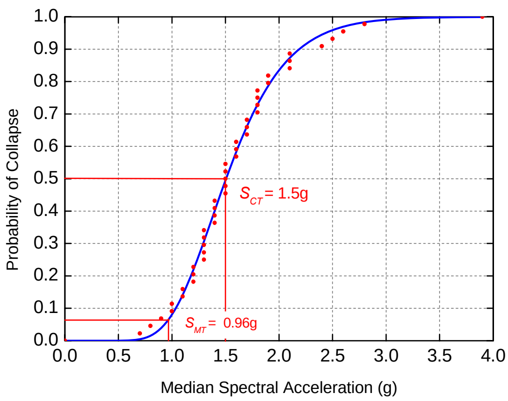
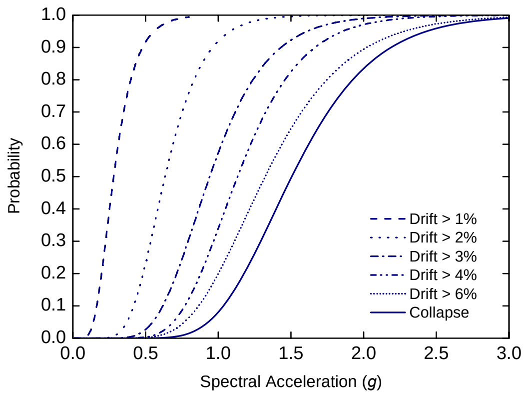


Figure 7. IDA curves



(a) Probability of collapse



(b) Probability of exceeding specified drifts

Figure 8. Fragility curves for the example BRKB-TMF

5. SUMMARY AND CONCLUSIONS

A new structural system called Buckling-Restrained Knee Brace Truss Moment Frame is investigated. The system harnesses the salient features of open-web trusses and buckling restrained braces. A performance-based design procedure for the system was developed and presented. A four story building structure was used as an example. The structure designed by the developed procedure was subjected to nonlinear static (pushover) and dynamic analyses. Collapse evaluation was also carried out. The main findings include:

1. The PBPD procedure presented in this paper can be used to design BRKB-TMFs. For the example structure, the PBPD procedure results in the frame with excellent response with all the inelastic activities confined to only the designated elements.
2. Both static and dynamic analysis results indicated that when the story drifts reached approximately 6%-7%, the collapse occurred. The failure pattern was typically the fracture of a set of BRBs leading to excessive rotation of the plastic hinges in the columns. Therefore, it is important to prevent early failure of the BRBs. This can be done by selecting the target drift

and the configuration of the frame corresponding to the deformation capacity of the BRBs.

3. The results of the collapse evaluation indicated that the probability of collapse for the MCE ground motions was less than 10%.

Although further investigations are required before this system can be fully validated, this study strongly demonstrates the potential of the proposed system. Currently, large-scale subassemblange tests as well as further detailed analytical studies are being planned as part of an international collaborative research project. The findings from these studies will provide full-fledged validation for this framing system.

AKCNOWLEDGEMENTS
Financial support for this research to the first author was provided by the Office of the Higher Education Commission, Thailand, under the Higher Education Research Promotion and National Research University Project. This work is part of an ongoing collaborative research project. The authors would like to acknowledge valuable input from Dr. T.Y. Yang, Dr. D.C. Rai, Dr. S.H. Chao, Dr. B. Stojadinovic, Dr. M. R. Bayat, and M. Baradaran Shoraka.

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