SEISMIC DESIGN AND EVALUATION OF BUCKLING-RESTRAINED KNEE-BRACED FRAME WITH SINGLE PLATE SHEAR CONNECTIONS

[1]PISETH DOUNG, [2]SUTAT LEELATAVIWAT

[1,2] Department of Civil Engineering,King Mongkut's University of Technology Thonburi, Thailand
E-mail: [1]piseth.dng@mail.kmutt.ac.th, [2]sutat.lee@kmutt.ac.th

Abstract— The design and evaluation of Buckling-Restrained Knee-Braced Frame(BRKBF) with Single Plate Shear Connections (SPSCs)are presented in this study. The objective of this research is to apply a design methodology called Performance Based-Plastic Design (PBPD) to design BRKBF system and to estimate the collapse probability of the framing system under earthquake excitations. The PBPD approach utilizes yield mechanism and pre-selected target drift as the main design parameters to control structural deformation.This method was applied to the design of an example BRKBF structure. The collapse probability of the example BRKBF was then investigated under 44 different ground motions based on FEMA P695 framework. Performance assessmentresults based on nonlinear static and time history analyses are presented and discussed.

Keywords— Seismic Design, Seismic Evaluation, BRKBF, Single Plate Shear Connections, PBPD, FEMA P695

I. INTRODUCTION

Knee-Braced Frame (KBF)systemcombinesa steel momentresisting frame and knee bracing elements near beam-column connections to enhance seismic resistance and control the inelastic behavior of the frame. To date, analytical and experimental studies **[1, 2, 3]** have been conducted to investigate the effectiveness of KBF system.

In this research, the behavior and design of a KBF with Buckling-Restrained Braces (BRBs) and single plate shear connections (SPSCs) is investigated. Thesystem is called Buckling-Restrained Knee-Braced Frame (BRKBF)and is illustrated in**Fig. 1**. BRBs are expected to enhance the ductility of the structure due to their stable hysteresis behaviorboth in tension and compression. SPSCsallow for ease of construction. They can also be easily repaired after an earthquake.

This paper first describes a design methodology called Performance Based-Plastic Design (PBPD) **[4, 5]**for the design of BRKBFs. An example frame was selected as a case study. Nonlinear static and dynamic analyses and collapse evaluation procedure were carried out to verify the system behavior and robustness.

II. DESIGN OF BRKBF WITH SPSCs

2.1. Design Base Shear

The design base shear for a BRKBFcan be computed using the PBPD procedure. This design approach utilizes pre-selected yield mechanism and target drift as key performance parameters **[4, 5]**. This approach is based on the use of energy balance conceptto obtain the design base shear. The selection of the yield mechanism **[7]** is the keystep in the design procedure to ensure desirable inelastic performance of the structure. For a BRKBF system where SPSCs are used, the selected mechanism consists of frame swaying with plastic hinging at column bases and yielding in the BRBs as shown in **Fig. 2**. Based on the energy balance concept, the design base shear of the system corresponding to a selected target drift can be calculated by**[4, 5]**:

$$V/W = \left(-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}\right)/2 \qquad (1)$$

where$W$is the total seismic weight of the structure, γ is the energy modification factorwhich is a function of ductility ($\mu = \Theta_u/\Theta_y$),C_e is the design spectral acceleration based on design code (e.g.,ASCE or IBC specification), and α is a dimensionless parameter depending on mass distribution, target plastic drift rotation (Θ_p) and fundamental period (T):

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

Once the design base shear has been computed, the lateral force at level$i$ can be written as:

$$F_i = \lambda_i V \qquad (3)$$

whereλ_i is the lateral force distribution factor at story level i and can be found in various relevant studies in literature**[4, 5]**.

2.2. Design of Structural Members

For the design of a BRKBF with SPSCs,adequate frame strength as required by **Eq. (1)**is needed to ensure that the deformation remains within the target value. Since SPSCswere used, the frame strength can be considered as provided solely by the BRBs and the column bases. The connections were assumed essentially as pins. The required axial strengths of BRBs can be determined by the principle of virtual work. By assuming that the compression force and tension force associated with BRBs in the same floor

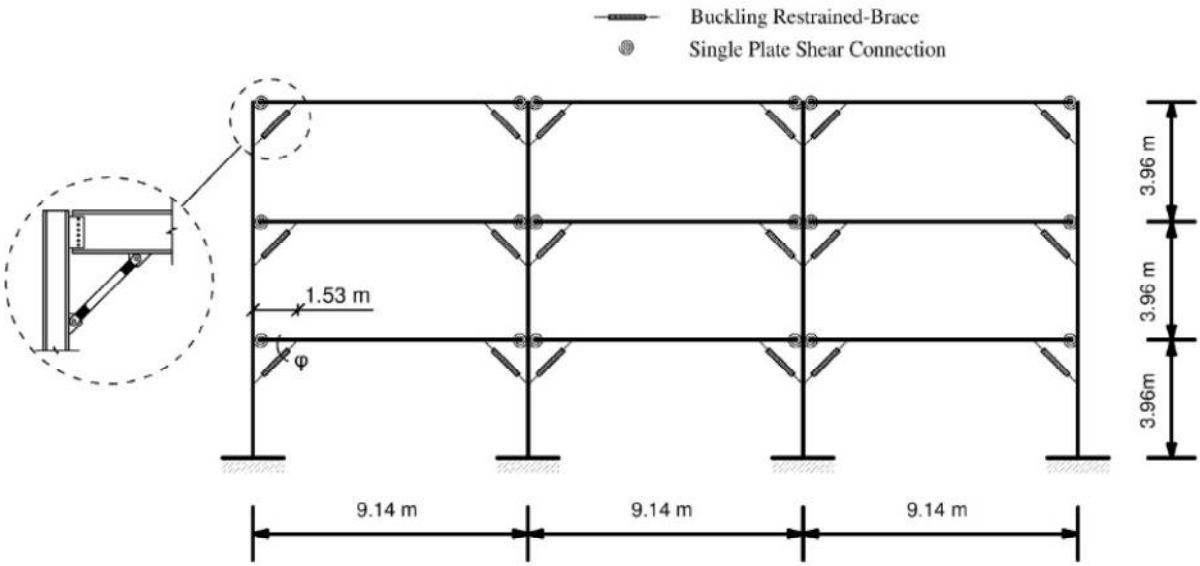


Fig.1. BRKBF with SPSCs Configuration

level are approximately equal,work equation for the 3-bay frame can be written as:

$$\sum_{i=1}^{n}\left(F_i h_i \theta_p\right)=\sum_{i=1}^{n}\left(3\times 2\beta_i P_{BRB}\delta_p\right)+\sum_{j=1}^{m}\left(M_{pc,j}\theta_p\right) \quad (4)$$

where F_i is the lateral force at story level I,θ_p is the plastic target drift, P_{BRB} is required axial strength of BRB at the roof level, $\delta_p$is the plastic axial deformation of BRBs, β_i is ratio of the strength of the BRB at level i to that of the roof level, and M_{pc} is plastic moment capacity of the first-story column. In order to solve the above equation for the required BRB strength, M_{pc}, β_i, and δ_p must be assigned. M_{pc} and $\beta_i$can be assigned based on the recommendations from previous studies **[4, 5]**.The plastic axial deformation of the BRB, δ_p, is a function of the frame drift. Based on the kinematics of the frame, δ_p, and can be expressed as:

$$\delta_p=\left(l_k\theta_p \sin 2\varphi\right)/2 \quad (5)$$

where$l_k$is the length of the BRB, and φ is the angle between BRB and a horizontal line. Once the target drift is selected, **Eq.(4)** can be solved to find the required BRB strength. In addition, the strain in the BRB should be kept less than the fracture strain. The strain in the core of the BRB at the design target drift level can be estimated using δ_p and by assuming that the yielding length of the BRB is approximately 70% of the length:

$$\varepsilon_p=\delta_p/\left(0.7l_k\right)=\left(\theta_p \sin 2\varphi\right)/1.4 \quad (6)$$

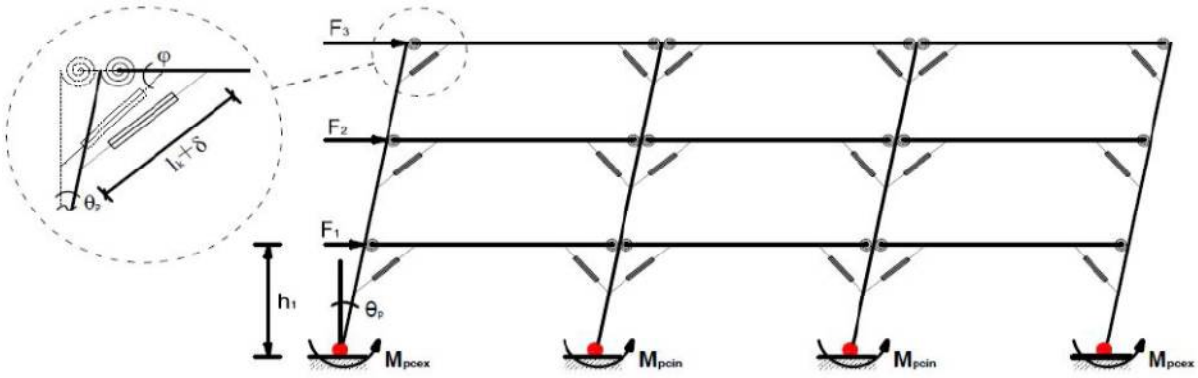
Using the required BRB strength, the core sizecan be calculated based on AISC specification **[8]**. The core area of the BRB can be calculated by:

$$\phi P_{ysc}=0.9F_{ysc}A_{sc}=\beta_i P_{BRB} \quad (7)$$

whereF_{ysc} is the nominal yield stress of steel core and A_{sc} is the cross-sectional area of steel core. The factor 0.9 is the strength reduction factor. The beams and columns were designed to resist the forces from the fully yielded and strain-hardened BRBs in tension and compression. The adjusted strengths of a BRB in tension and compression are given respectively by **[8]**:

$$P_{ut}=\omega R_y P_{ysc} \quad (8)$$

Fig. 2. Yield mechanism of BRKBF with SPSCs



$$P_{uc} = \omega\beta_0 R_y P_{ysc} \quad (9)$$

whereω is an adjustment factor to account for strain hardening, β_0 is an adjustment factor for overstrength in compression, and R_y is material overstrength factor. The values of ω,and $\beta_0$can be obtained based on BRB test results. A series of testsperformed by Merritt, Uang, and Benzoni**[10]**indicated that the approximate values of $\omega$and $\beta_0$are 1.4 and 1.1 respectively. R_y can be taken as 1.0 if the actual yield strength is used. For columns, they must be designed to remain elastic except at the bases where plastic hinges are allowed to form. The design of columns is based on the equilibrium of all the forces associated with each column **(Fig. 3)**. The lateral forces must be compatible with the fully-yielded and strain-hardened condition. The design of beams and columns are based on AISC specification **[9]**.

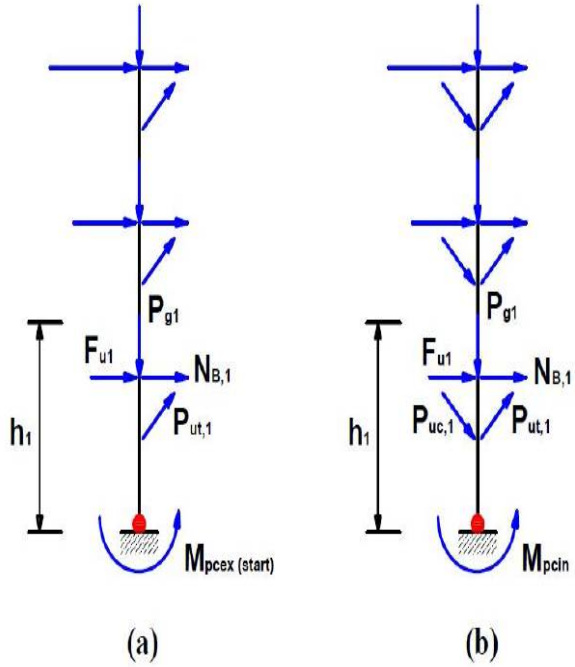


Fig.3. Column tree free-body diagram, (a) exterior column, (b) interior column

2.3. Example Frame

An example of BRKBF with SPSCs, 3-bays wide and 3-story high**(Fig. 1)** was designed per PBPD procedure as described above. The structural system was assumed to bein Design Category D with S_1= 0.6g and S_s= 1.5g **[11]**. The design base shear of the study frame was evaluated at two hazard levels, DBE and MCE. The design spectral acceleration values were determined to be respectively 0.764g at DBE level and 1.146g at MCE level. The assumed target drift was 2% at DBE level and 3% at MCE level. The yield drift of the system was assumed to be 0.7%.The governing base shearout of these two hazard levels was used for the design of structural members.**Table 1** summarizes the design parameters and **Table 2** summarizes the member sizes.

Table 1. Design parameters for BRKBF with SPSCs

Parameters	DBE	MCE
Spectral Acceleration (g), C_e	0.764	1.146
Fundamental period (s), T	0.786	0.786
Target drift ratio (%), θ_u	2	3
Yield drift ratio (%), θ_y	0.7	0.7
Plastic drift ratio (%), θ_p	1.3	2.3
Energy modification factor, γ	0.578	0.412
Dimensionless parameter, α	1.616	2.858
Design base shear factor, V/W	0.188	0.179

III. PERFORMANCE EVALUATION

The purpose of the performance evaluation is to assess the seismic behavior and the collapse probability of BRKBF with SPSCs. To do so, Incremental Dynamic Analysis (IDA) **[12]** was conducted based on FEMA P695 Methodology **[5]**.
In this paper, the analyses were conducted usingthe nonlinear software called Perform-3D **[13]**. The behavior of each component was controlled by its deformation capacity which was estimated using existing test results available in literature.

Table 2. Summary of member sizes

BRB		
Floor level	Capacity (kN)	Size (cm^2)
Roof	423	18.8
3rd	658	29.5
2nd	767	34.4
Beam		
Floor level	All bay	
Roof	W24x76	
3rd	W27x94	
2nd	W27x102	
Column		
Story	Exterior	Interior
3	W14x176	W14x211
2	W14x176	W14x211
1	W14x176	W14x211

3.1. Structural Modeling

Beam and column modeling were based on ASCE 41-06 **[14]**recommendationswith some modifications. A study by Newell and Uang**[15]** suggested that a typical wide flange shape should be able to

accommodate lateral drifts between 7 to 10% before failure. This drift limit was used in the modeling of the columns.

For theBRBs, a tri-linear force-deformation relationship was assumed in modeling. The model was constructed based on the test results by Merritt **[10]** which indicated that the strain capacity was in the order of 2.5% with the core strain of up to 2.8%.

A single plate shear connection (SPSC)is normally considered as a flexible connection and its flexural resistance is normally disregarded. However, in reality, a small amount of moment capacity can be expected. For this reason, the modeling of the connections was based on models from previous studies proposed by Liu and Astaneh-Als **[16]**, Wen **[17]**,and Crocker and Chambers **[18]**. The test results from Liu and Astaneh-Als were used to estimate the moment-rotation capacity of the connections. Some modificationswere applied to improve the rotation capacity and the ultimate flexural strength of the connections.The moment capacity depends on the capacity of the single plate**[17].** The flexural capacity of the plate is given by:

At the onset of yielding:

$$M_y = t_p d^2 F_y / 6 \quad (10)$$

At plastic limit:

$$M_p = t_p d^2 F_y / 4 \quad (11)$$

wheret_p and $d$are respectively the thickness and the depth of the steel plate, and F_y is the yielding stress of the steel plate.The yield and plastic rotations are empirically selected as 0.42% and 5% **[16]**. Once the yield moment capacity is computed, the initial elastic stiffness of the connection can be found by$K_s = M_y/\theta_y$. The ultimate rotation capacity of SPSC depends on the depth of bolt group and was assumed to be equal to 7%. **Fig. 4** shows the moment-rotation model for the connection.Based on themodeling techniques described above Nonlinear Static Pushover (NSP) and Dynamic Time History (NDTH) Analyses were conducted. The results are as follows.

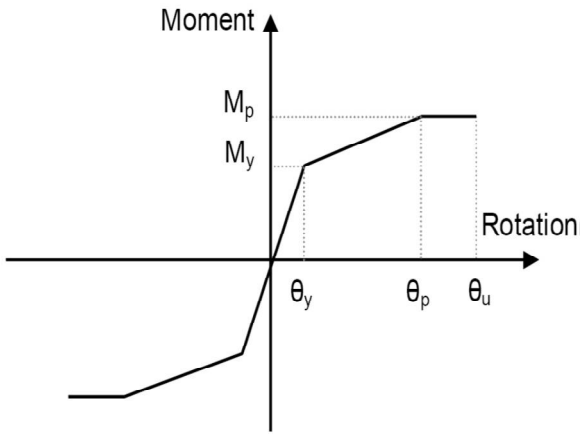


Fig.4. Moment-rotation model for single plate shear connections [14, 15]

3.2. Nonlinear Static (Pushover) Analysis

Nonlinear static analysis was performed to assess the overall global behavior including yielding sequence, overstrength factor, and structural ductility (μ_T) of the proposed system. The pushover result is shown in **Fig. 5**.Some of the key pushover response parameters are:

$$\Omega = V_{\max} / V \quad (12)$$

$$\mu_T = \delta_u / \delta_{y,eff} \quad (13)$$

where$\Omega$is the structural overstrength, μ_T is the structural ductility,V_{max} is the maximum base shear capacity from pushover curve **(Fig. 5)**, V is the design base shear from PBPD, and δ_u and $\delta_{y,eff}$are respectively the roof displacement at failure and effective yield roof displacement. From the response, BRB members started to yield at 0.42% roof drift. The full mechanism started to format the roof driftbetween 0.96% and 1.1%.BRB started failing at 3.5% roof drift. Finally, allBRB membersfailed at 4.2% roof drift. The strain and rotation demands of the BRBs and SPSCs at the target roof drift of 2% are shown in **Table 3**. As can be seen, at the target drift of 2%, the deformation demands in the key elements were still far below theircapacity. The system overstrength (Ω) and system ductility (μ_T) were 1.6 and 6 respectively.

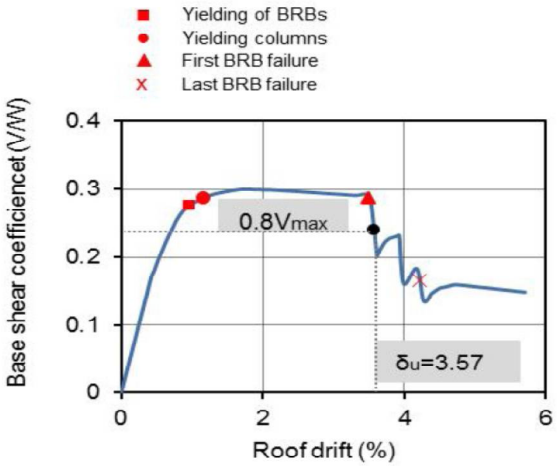


Fig.5. Plot of pushover curve

Table 3. Max. BRB strain and SPSC rotation demands at 2% target roof drift

Component	Floor	Demand (%)		Capacity (%)
		Left	Right	
BRB	Roof	0.475	1.06	2.8
	3	0.765	1.168	2.8
	2	0.845	1.209	2.8
SPSC	Roof	0.544	1.9	7
	3	1.055	1.645	7
	2	1.28	1.77	7

3.3. Nonlinear Dynamic (Time History) Analysis

A series of nonlinear dynamic (time history) analyses (NDTHA) were conducted to assess the deformation capacity, the median collapse intensity (S_{CT}), and the collapse margin ratio (*CMR*). The median collapse intensity (S_{CT}) is defined as the intensity where collapse occurredin half of all the ground motions used. The collapse margin ratio (*CMR*) is asafety criterionthat quantifies the robustness of the structure in preventing collapse. CMR is defined as the ratio of median collapse intensity to the intensity at MCE level (S_{MT}) **[5].** The analyses were performed using 44 far-field ground motion records recommended by FEMA P695.

The maximum interstory drifts at MCE level are shown in **Fig. 6.** Inelastic activities of the frame under a selected ground motion which produced the response nearest to the median values are depicted in **Fig. 7.** The results in terms of maximum interstory drifts at various ground motion intensity levels are shown in **Fig. 8**. From the figures, interstory drifts were below the desirable target drifts (2% for DBE level, 3% for MCE level) as intended. The rotation and strain demands were still less than the capacities of the corresponding members. The results indicate that the PBPD method presented earlier is an effective design procedure that can be applied to the design of BRKBF.

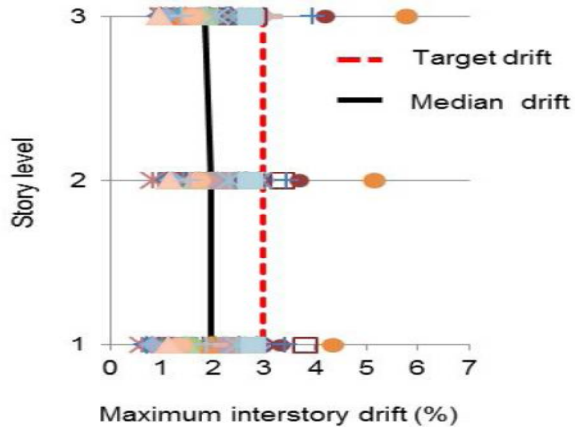


Fig.6. Interstory drift at MCE level

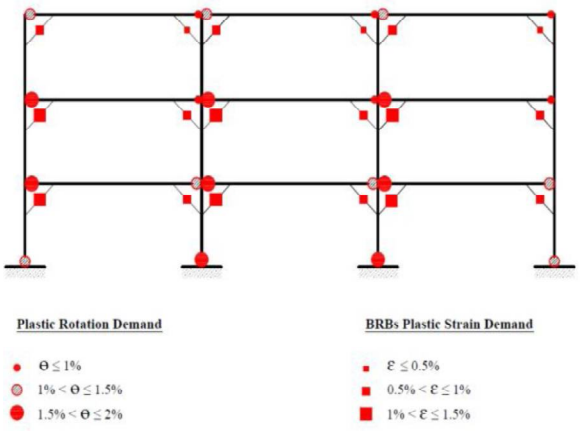


Fig.7.Plastic rotation demands and BRB strain under a selected ground motion (KOCAELI/DZC270) at MCE level

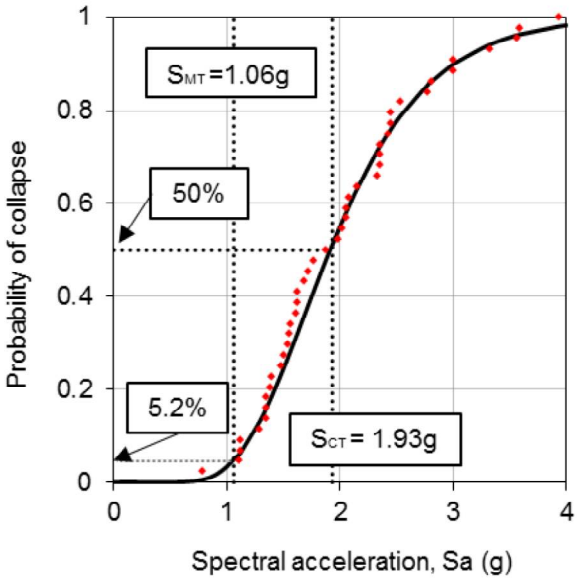


Fig. 9.Fragility curve

Based on the results shown in **Fig. 6**, a fragility curve showing the collapse probability was constructed as shown in **Fig. 9**. The median collapse intensity (S_{CT}) was determined to be 1.93*g*. At MCE level, the probability of collapse was equal to 5.2% with S_{MT} equal to 1.06*g*leading to a collapse margin ratio ($CMR = S_{CT}/S_{MT}$) of 1.82. This collapse margin ratio must be adjusted by a spectral shape factor (*SSF*)because of the influence of the frequency content. In FEMA P695, SSF can be determined as a function of the fundamental period (*T*), the ductility(μ_T), and the seismic design category (SDC). The adjusted collapse margin ratio (ACMR) is given as:

$$ACMR_i = SFF_i \times CMR_i \qquad (14)$$

To evaluate the structural collapse, the required collapse margin ratio at 20% probability ($ACMR_{20\%}$) must be calculated based on the total system collapse uncertainty. The results showed that for this example individual structure, the*ACMR*and the required value for *ACMR* ($ACMR_{20\%}$)were found to be 2.48 and 2.22 respectively. As be seen, the *ACMR*valueis greater than the required $ACMR_{20\%}$ which means that the study frame provides sufficient robustness under the selected ground motions.

CONCLUSIONS

This paper investigated the seismic performance of BRBKF with SPSCs. An example frame designed by the PBPD was presented and discussed. The nonlinear static and dynamic analysesresults indicated that the structure performed as intended. The plastic rotation demands of key elements and strain demands of the BRBs were well below the deformation and strain capacities at both DBE and MCE levels. Under 44 ground motions, the probability of collapse at MCE level was

approximately 5.2% which is below the generally accepted limit of 10%.The proposed system also satisfied the FEMA P9695 criteria in term of collapse margin ration. Based on the results in this study, BRBKF appears to be a viable alternative to conventional seismic load resisting structural systems.

ACKNOWLEDGEMENT

The authors wish to thankthe financial support from Thailand On-Place Scholarship under Asea-Uninet program at King Mongkut's University of Technology Thonburi.

REFERENCES

[1] S. Leelataviwat, B. Suksan, J. Srechai, and P. Wanitchai, "Seismic Behavior and Design of Knee-Braced Moment Frames", J. Struct. Eng, ASCE, Vol. 137, No. 5, pp. 579-588, 2011.

[2] T.Y. Yang, Y. Li, and S. Leelataviwat, "Performance-Based Design and Optimization of Buckling Restrained Knee Braced Truss Moment Frame", ASCE, Journal of Performance of Constructed Facilities, 0887-3828/A4014007(11), 2014.

[3] B. Farokhi, and A. Bazvand, "Seismic Performance Evaluation of Steel Knee-Braced Moment Frames", Academia Arena 2014, vol.6, pp. 1-8, 2014.

[4] S.C. Goel, S-H. Chao, S. Leelataviwat, and S-S, Lee, "Performance Based Plastic Design Method for Earthquake-Resistant Structures", The 14th World Conference on Earthquake Engineering, Beijing, 2008.

[5] S. Leelataviwat, "Drift and Yield Mechanism Based Seismic Design and Upgrading of Steel Moment Frames", Ph.D Dissertation, The University of Michigan at Ann Arbor, USA, 1998

[6] FEMA, "Recommended Methodology for Quantification of Building System Performance and Response Parameters", Emergency Management Agency, FEMA P695, Washington, D.C., 2009.

[7] S.C. Goel, and S. Furukawa, "Role of Yield Mechanism Selection on Seismic Behavior of Steel Moment Frames Designed by Performance-Based Plastic Method", Structures Congress, ASCE, 2009.

[8] AISCa, "Seismic Provision for Structural Steel Buildings", American Institute of Steel Construction, ANSI/AISC 341-10, Chicago, 2010.

[9] AISCb, "Specification for Structural Steel Buildings", American Institute of Steel Construction, ANSI/AISC 360-10, Chicago, 2010.

[10] S. Merritt, C-M. Uang, and G. Benzoni, "Subassemblage Testing of Corebrace Buckling-Restrained Brace",Report No. TR-2003/01, University of California, San Diego, California, 2003.

[11] ASCE, "Minimum Design Loads for Buildings and Other Structures", American Society of Civil Engineers, ASCE/SEI 7-10, Reston, Verginia, 2010.

[12] Vamvatsikos, D., and Cornell, C-A., "Incremental dynamic analysis", Journal of Earthquake EngngStruct. Dyn., Vol 31, pp. 491–514, 2002.

[13] CSI, "PERFORM 3D User's Manual", Ver. 4.0, Computers and Structures Incorporate, Berkeley, California, 2007.

[14] ASCE, "Seismic Rehabilitation of Existing Buildings", American Society of Civil Engineers, ASCE/SEI 41-06, Reston, Virginia, 2007.

[15] J. Newell, and C-M. Uang, "Cyclic Behavior of Steel Columns with combined High Axial Load and Drift Demand", Technical report, No. SSRP-06/22, University of California, San Diego, California, 2006.

[16] J. Liu, and A. Astaneh-Als, "Moment–Rotation Parameters for Composite Shear Tab Connections", J. Struct. Eng, Vol. 130, pp. 1371-1380, 2004.

[17] R.Wen, B. Akbas, and J. Shen "Practical moment–rotation relations of steel shear tab connections", Journal of Constructional Steel Research, Vol. 88, pp. 296-308, 2013.

[18] J. P. Crocker, and J. J. Chambers, "Single Plate Shear Connection Response to Rotation Demands Imposed by Frames Undergoing Cyclic Lateral Displacements", J. Struct. Eng, Vol. 130, pp. 934-941, 2004.

★★★