Title: **Application of Buckling Restrained Braces in a 50-Storey Building**

Authors: Jose A. Sy, SY2 + Associates, Inc.
Naveed Anwar, Asian Institute of Technology
Thaung Htut Aung, Asian Institute of Technology
Deepak Rayamajhi, Asian Institute of Technology

Subject: Seismic

Keywords: Code Compliance
Seismic

Publication Date: 2014

Original Publication: International Journal of High-Rise Buildings Volume 3 Number 1

Paper Type:

1. Book chapter/Part chapter
2. **Journal paper**
3. Conference proceeding
4. Unpublished conference paper
5. Magazine article
6. Unpublished

International Journal of High-Rise Buildings
March 2014, Vol 3, No 1, 81-87

International Journal of
High-Rise Buildings
www.ctbuh-korea.org/ijhrb/index.php

Application of Buckling Restrained Braces in a 50-Storey Building

Jose A. Sy[1†], Naveed Anwar[2], Thaung Htut Aung[2], and Deepak Rayamajhi[2]

[1]*Sy^2 + Associates, Inc., Philippines*
[2]*AIT-Consulting,Asian Institute of Technology, Thailand*

Abstract

The use of Buckling Restrained Braces (BRB) for enhancing the performance of the buildings is gaining wider acceptance. This paper presents the first application of these devices in a major high-rise building in the Philippines. A 50-storey residential reinforced concrete building tower, with ductile core wall, with BRB system is investigated. The detailed modeling and design procedure of buckling restrained brace system is presented for the optimal design against the two distinct levels of earthquake ground motions; serviceable behavior for frequent earthquakes and very low probability of collapse under extremely rare earthquakes. The stiffness and strength of the buckling restrained brace system are adjusted to optimize the performance of the structural system under different levels of earthquakes. Response spectrum analysis is conducted for Design Basis Earthquake level and Service level, while nonlinear time history analysis is performed for the most credible earthquake. The case study results show the effectiveness of buckling restrained braces.

Keywords: Buckling restrained braces buckling restrained braces, Ductility, Performance-based, Performance objectives, Non-linear time history, Finite element modeling

1. Introduction

Buckling restrained braces (BRBs) are widely used in seismic design and retrofitting of buildings, especially in the United States and Japan. The effective use of buckling restrained braces enhances the performance of the structural system under severe earthquakes (Dutta and Hamburger, 2011).

In this case study, BRB system is utilized as the first time for the design of a 50-storey tower, located in Makati City, Philippines. The case study building is 166.8-meter tall reinforced concrete residential building, standing on the one-level podium, with the tower plan area of 34.5 m × 26 m. Reinforced concrete bearing walls, gravity columns and post-tensioned (PT) flat slabs are used in gravity load resisting system. The typical plan and elevation view are shown in Figs. 1 and 2, respectively. The typical story height of the building is 3.1 m.

The lateral load resisting system consists of reinforced concrete bearing wall coupled with outrigger columns, connected by the BRBs (as shown in Fig. 3) along the weak axis direction (as shown in Fig. 1). Sixteen BRBs are used, in which each BRB is connected in between two floors. Eight BRBs are located in between 19th and 23rd floors and remaining eight BRBs are located in between 43rd and 47th floors. The BRBs are manufactured by Star Seismic Inc. The building has 3½-storey of below-grade parking, resting on the mat foundation. The tower consists mainly of residential units, and a terrace and amenity deck. The ground level contains retail and back of the house space. The total project area is approximately 79,000 gross square meters. This building was the first building with application of BRBs for lateral load resisting system in the Philippines.

In this paper, the overall procedure for the performance- based design of tall buildings is described. In the following section, the application of BRB is explained followed by seismic performance objective, design criteria, and acceptable limits for the project are described. Moreover, several modeling techniques are explained and finally results are presented from nonlinear time history analysis.

2. Application of BRB

Conventional earthquake resistant structural systems depend on the strength and ductility to control seismic responses. In this strategy, seismic energy is absorbed by the formation of plastic hinges in the specifically designed regions, such as plastic hinge at the base of the wall and at the ends of the coupling beams. These regions should be able to deform into an inelastic range and sustain reversible cycles of plastic deformation without degrading strength and stiffness to a level which could harm the stability and integrity of the structure. However, the structure may suffer structural and non-structural da-

†Corresponding author: Jose A. Sy
Tel: +63-896-9704; Fax: +63-896-9760
E-mail: boysy@sysqured.com

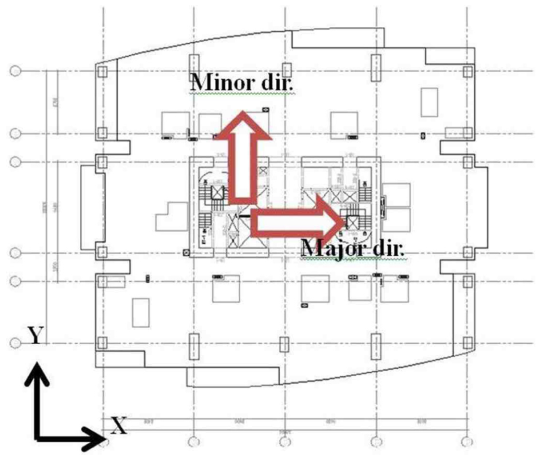


Figure 1. Typical Floor Plan of Tower.

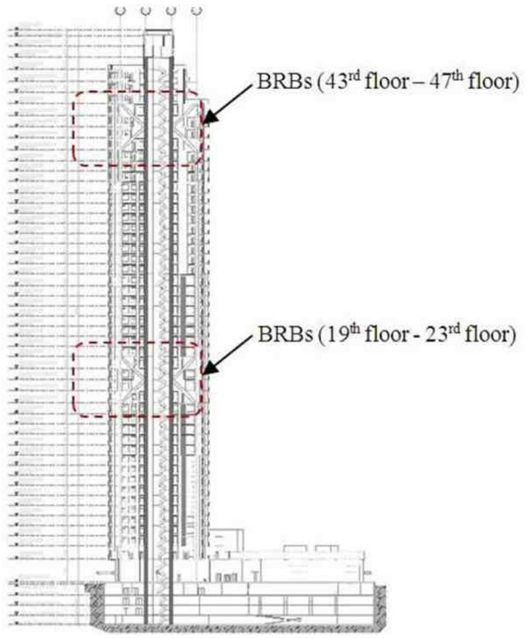


Figure 2. Sectional View.

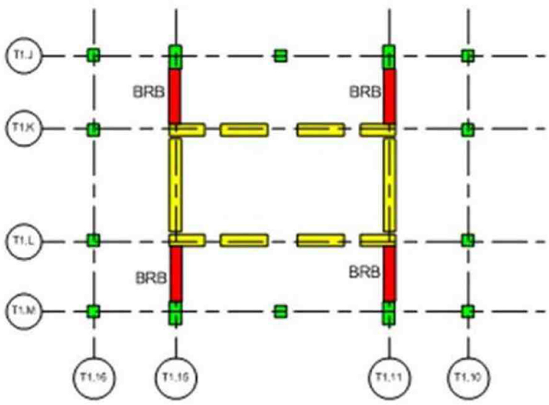


Figure 3. BRB Locations in Plan.

mage to an extent that it may not be economically repairable (Ahmed, 2011).

To avoid this, another strategy which incorporates energy dissipating devices in the structural system to reduce the inelastic energy dissipation demand on the framing system was developed. In this strategy, as the structural components may remain elastic during an earthquake, the structural and non-structural damage may be considerably reduced. BRBs are one of the promising options to use as the energy dissipating devices (Ahmed, 2011).

3. Seismic Performance Objectives

The specific seismic performance objectives are defined for the design of the case study building against three levels of earthquake hazards. These performance objectives are based on the Los Angeles Tall Buildings Structural Design Council (LATBSDC 2008) for the performance-based design of tall buildings (Fry et al., 2010)

3.1. Frequent/service level earthquakes (50% of probability of exceedance in 30 years with 43-year return period):

The structure should remain essentially elastic with minor damage to structural and non-structural elements, remaining serviceable after earthquakes.

3.2. Maximum considered earthquake level (2% of probability of exceedance in 50 years with 2475-year return period):

Substantial damage to the structure is allowed, potentially including significant degradation in the stiffness and strength of the lateral force resisting system. The building may be on the verge of partial or total collapse.

In addition, in order to maintain the design of the building to the level of conventional design code, the design of the building is also considered for *Design Basis Earthquake (DBE) Level (As defined by ASCE 7-05, Section 11.4):* Moderate structural damage is allowed in which extensive repairs may be required. However, it should be noted that this level of design is omitted from LATBSDC (2008) guidelines.

4. Overall Design Procedure

Three phases are involved in the overall design procedure to meet the performance objectives; 1) Preliminary Design, 2) Service Level Evaluation and 3) Collapse Prevention Level (MCE level) Evaluation.

4.1. Preliminary design

In this phase, elastic response spectrum analysis and design are performed in accordance with the code-based design approach by using appropriate load factors and strength reduction factors against the gravity loads, wind load and seismic load. Appropriate initial sizes of buckling restrained braces are used in the analysis. Site specific response spectrum for DBE level is used for the preliminary design phase. Structural components to be remained elastic are designed by applying the appropriate amplification factors.

4.2. Service level evaluation

Primary response characteristics such as story drift, coupling beam and shear wall demand to capacity ratios are checked against the demands resulting from the response spectrum analysis using site specific service level response spectrum with 43-year return period (50% of probability of exceedance in 30 years). The required capacities of buckling restrained braces are determined so that they remain elastic under the service level earthquakes.

4.3. Collapse prevention level evaluation

Design verification is performed by nonlinear response history analysis (NLRHA) against the MCE level earthquakes with 2475-year return period (2% of probability of exceedance in 50 years). Seven pairs of site specific ground motions are used to conduct the nonlinear response history analysis. Average of demands from seven ground motions approach is used for design evaluation at MCE level. The preliminary design is modified as required in order to meet the acceptance criteria.

Coupling beams, core wall flexural response and buckling restrained braces are checked in anticipation of nonlinear response while core wall shear, diaphragms, basement walls, foundations and columns are checked to remain essentially elastic during the nonlinear response history analysis.

5. Seismic Performance Criteria

5.1. Service level

The expected responses of building components to fulfill the performance objective at Service level earthquake are shown in the Table 1.

5.2. Maximum considered earthquake level

The expected responses of building components to fulfill the performance objective at MCE level earthquake are shown in Table 2.

6. Finite Element Modeling

A complete full three-dimensional finite element model is created, which includes the tower and the whole podium. For the evaluation of nonlinear response of the building under MCE level earthquakes, the flexural response of core wall, slender coupling beams and slab outrigger beam; the shear response of deep coupling beams and the axial load response of BRBs are modeled with nonlinear force-deformation behavior.

Table 1. Performance Criteria for Service Level Earthquake

Item	Value
Story drift	0.5 percent
Coupling beams (conventional shear reinforcement)	shear strength to remain essentially elastic
Core wall flexure	Remain essentially elastic
Core wall shear	Remain essentially elastic
BRBs	Remain elastic

Table 2. Performance Criteria for MCE Level Earthquake

Item	Acceptable value
Story drift	3 percent
Coupling beam rotation (diagonal shear reinforcement)	0.06 radian rotation limit
Coupling beam rotation (conventional longitudinal reinforcement)	0.025 radian rotation limit
Core wall reinforcement axial strain	Rebar strain = 0.05 in tension and 0.02 in compression
Core wall concrete strain	Concrete compression strain $= 0.004+0.1\rho(fy/f'c)$
Core wall shear and basement walls	Average shear demand times 1.3

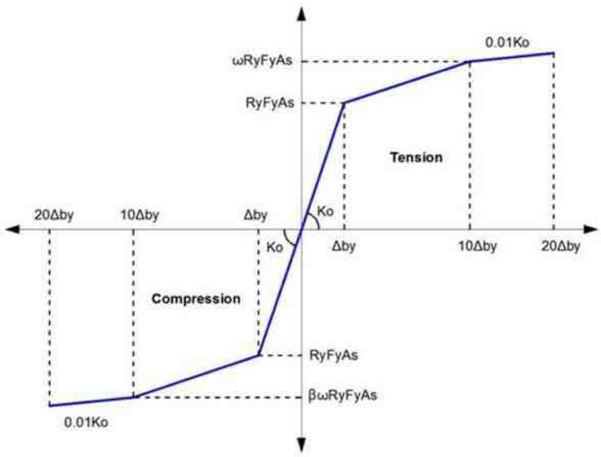


Figure 4. Assumed Backbone Curve for Buckling - Restrained Braces.

6.1. Analysis tools

The modeling and analysis of building for evaluation and design at Service Level earthquake and DBE level are carried out in ETABS 9.5 computational platform. For the MCE level performance evaluation, nonlinear three-dimensional model is created in PERFORM-3D (Version 4.0.4) computational platform.

6.2. Nonlinear modeling of buckling restrained braces

PERFORM-3D-"*BRB compound component*" is used to model the BRBs. The BRB compound component has three parts; 1) BRB basic component, which incorporate the nonlinear behavior (inelastic deformation) of BRB 2)

Elastic bar basic component, which corresponds to the BRB steel outside the main inelastic zone, and 3) Stiff end zone, with a specified length and a cross section area that is a multiple of the elastic bar area. The end zone accounts for the gusset plates at both ends of the member. The backbone curves used for the modeling of BRB is shown in Fig. 4.

The coefficients R_y, ω, and β are estimated based on the properties of the BRBs provided by Star Seismic Inc. The initial stiffness (Ko) of the BRB is estimated based on cross sectional properties and material properties by A_sE/L (where A_s cross sectional area of Steel, E is Young's modulus of Elasticity of steel, and L is the effective length of the brace in inelastic behavior i.e., approximately 70% of the pin to pin BRB length).

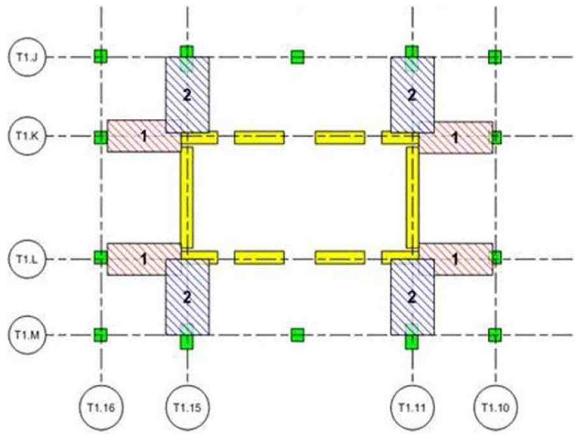


Figure 5. Slab Outrigger Beams in Plan.

6.3. Nonlinear modeling of ductile core wall

Orakcal and Wallace (2006) present several modeling techniques for ductile reinforced concrete wall. Fiber modeling technique is used to model the nonlinear flexural behavior of the core wall. For the given wall cross-section, quantity of longitudinal reinforcement, and transverse reinforcement, the modeling of wall involves: 1) diving the wall cross-section into concrete (unconfined and confined) fibers and reinforcement fibers; 2) selection of appropriate constitutive models for concrete and steel; and 3) providing appropriate boundary conditions(Orakcal and Wallace, 2006; Wallace, 2007).

PERFORM-3D shear wall element is used to model the nonlinear behavior of core wall. Two parallel fiber sections are used to model the shear wall section. The first fiber section consists of only uniformly distributed steel (steel only) and the second fiber section consists of both concrete and boundary zone steel reinforcement. For the uniformly distributed steel, auto-size fiber elements are used whereas for latter one, fixed size fiber elements are used. The height of fiber element is modeled as floor height at every floor level except at podium level, where wall is discretized into 5 elements along the longitudinal axis of the tower. Shear behavior in the wall is modeled with elastic material properties.

6.4. Nonlinear modeling of coupling beams

In this building, two types of coupling beams are present. First one is deep beam having span to depth ratio of 1.9 (span/depth < 4), and second one is slender beam having span to depth ratio of 4.3 (span/depth > 4). Since deep beams are dominated by shear behavior, they are modeled as shear deformation controlled behavior while the slender beams are modeled as flexural deformation controlled behavior.

The modeling of coupling beams were carried out by procedure described in Wallace (2007) and Wallace et al. (2009).The deep coupling beam is modeled with elastic frame section with a nonlinear shear hinge located at the mid span of the element. The capacity of the shear hinge is calculated based on the diagonal reinforcements. The elastic stiffness of the deep beams is reduced to $0.16EI_g$. The effective stiffness calculation is based on the dimension and required ductility (Wallace et al. 2009). The ultimate capacity is taken as the 1.33 times of the yielding capacity.

The slender coupling beam is modeled with two moment hinges, located at the ends of the beam. The capacity of the moment-curvature hinges are calculated based on the longitudinal reinforcements provided in the beams. The deformations capacities are used in accordance with ASCE 41-06 for the flexural coupling beams. The elastic stiffness of the slender beams is reduced to $0.5EI_g$ (ASCE 41-06).

6.5. Nonlinear modeling of slab outrigger beams

In the tower portion, the floor is modeled as rigid floor diaphragm. The slab is not modeled as area elements in the tower. However, equivalent "slab outrigger beams" are used in the model in order to determine the nonlinear response of post-tensioned slab, interaction with core wall and columns. Slab outrigger beams are modeled with nonlinear hinges at both ends of the beam (as shown in Fig. 5). Moment-curvature type hinge is used to model nonlinearity in the slab-beam. The moment capacity of the slab beam is calculated based on the reinforcement in the slab. The nonlinear properties of the moment hinges were matched to the study of Klemencic et al. (2006). The post-tensioning effect is considered in the calculation of the flexural yielding capacity of the slab. However, the performance of the moment hinges is not specifically reviewed.

At the podium and basements level, the slabs are modeled without rigid floor diaphragm. Slabs in the podium and basement are modeled using linear shell elements. The elastic flexural stiffness of the slabs and equivalent slab-beams are reduced to $0.5EI_g$.

6.6. Modeling of foundation

To simulate the soil-structure interaction effects, retaining walls in the basement were also modeled with elastic linear shell element and surrounding soil was modeled with nonlinear springs. The properties (force and deformation) of nonlinear springs are estimated from the geotechnical soil report provided geotechnical consultant. Though the lateral resistance of soil was considered, the damping of the soil on the sides of the basement wall was neglected in the study. In terms of boundary condition, the base of the core wall was modeled as pin connection (i.e., without rotational restrained), base of the columns are modeled as fixed and soil springs are fixed along their main axis. The modeled was subject to the 7 sets of ground motions at the base of the mat foundation and average response was computed. The ground motions are used from Geotechnical earthquake engineering site specific study report available for this project.

7. Detailed Design of BRB System

7.1. Design of buckling restrained braces

The typical BRBs system used in the building is shown in Fig. 6. Initial sizing of the BRB was carried out based on the wind and seismic loading, with the worse condition from seismic loading for DBE level earthquake. After nonlinear analysis for MCE level earthquake, the size of the BRBs was adjusted for the optimal performance and to control the story drift. The stiffness of the BRB was adjusted by varying cross-sectional area and effective length of steel core. The stiffness was adjusted in such a way that the stiffness of BRB at the floor is higher than the story stiffness in that floor. Relatively high stiffness of bracing leads to attract higher seismic demand on the BRB caused to early yielding than reinforced concrete core wall. Moreover, yielding of BRB serves as energy dissipation and provides the higher ductility to core wall system as well.

7.2. Detailed connection design

Detailed connection design of BRB system is carried out in accordance with AISC 360-05 and ACI 318-08.

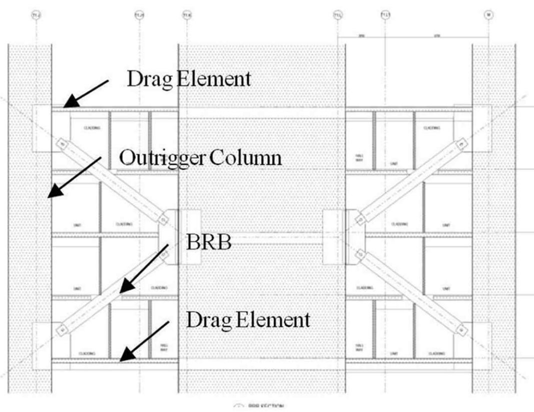


Figure 6. Buckling Restrained Brace System.

Gusset plate connection with BRBs is designed to satisfy the dimensional requirements for the pin-connected members in accordance with AISC 360-05. In gusset plate design, gusset plates are modeled separately and the maximum design yield force of BRB is applied.

Steel studs that restrained the gusset plate embedded in the concrete are designed against the shear demand in accordance with ACI 318-08.

Furthermore, the load path of net vertical force from the gusset plate to the steel column (embedded in the reinforced concrete outrigger column) and then from the steel column into the concrete column is checked based on the capacity of each element. The axial capacity of the drag element is also checked to transfer the horizontal component of the force from the BRB.

8. Analysis Results

8.1. Modal analysis

The natural periods of the building are 5.75 s and 4.86 s in principal directions with 0.40 and 0.42 modal participating mass ratios. The first three mode shapes of the building are shown in Fig. 7. The first and second mode are dominated in lateral deformation in Y and X, respectively, and third mode is in torsional deformation.

8.2. Base shear

The base shear is compared between DBE level response spectrum analysis and average of MCE level nonlinear response history analysis in Table 3. The base shear is calculated above the podium level and considered the tower portion only. The seismic weight of the tower above the podium level is 616,900 kN. The design base shear (shear calculated above the podium) is approximately 3.5% and 3.8% in each principal direction, which is higher than the minimum limit of 3%, set by the LATBSDC-2008 guidelines. Furthermore, the dynamic base shear calculated from the average of seven time histories is approximately two times higher than the design base shear, which is typical in high rise buildings.

8.3. Storey shear and storey moment

Storey shears and storey moment distributions are plot-

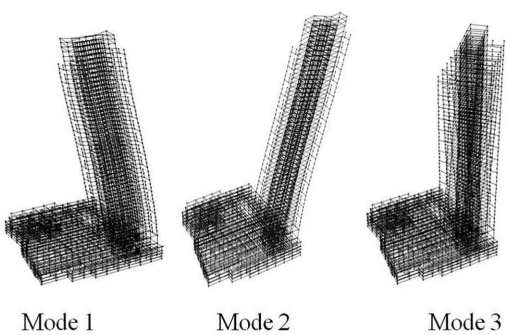
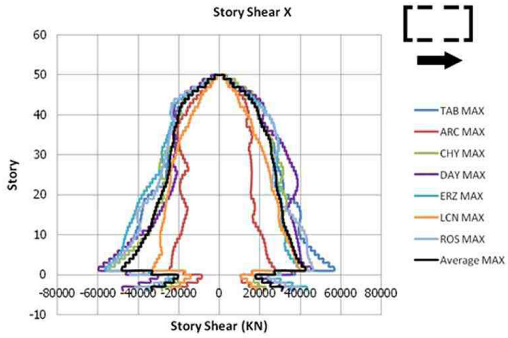


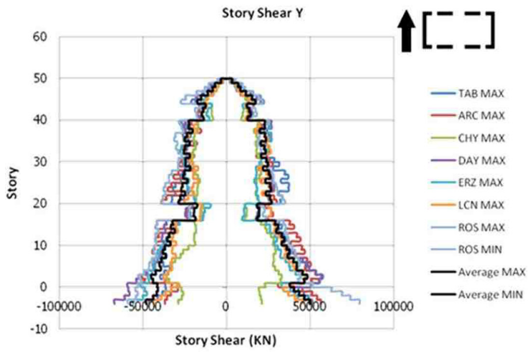
Figure 7. Mode Shapes.

Table 3. Base Shear Comparison

Load cases	Base shear (kN)	% of Seismic weight
DBE (major dir.)	21,012	3.56
DBE (minor dir.)	22,691	3.84
MCE (major dir.)	47,892	7.76
MCE (minor dir.)	46,462	7.53



(a) Along X-axis (Major dir.)



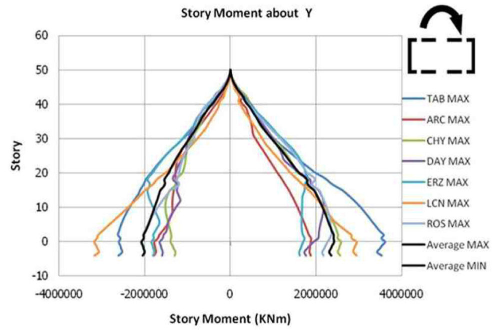
(b) Along Y-axis (Minor dir.)

Figure 8. Plot of Storey Shear (a) in X, (b) in Y.

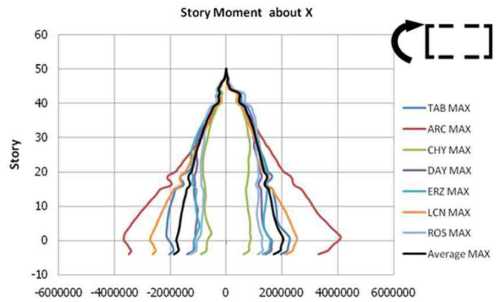
ted along the height of the building and shown in Figs. 8 and 9, respectively. The storey shear at the basement level is generally decreased in most of the time history except some time histories where the storey shear has increased. This may happen due to the irregular distributions of basement walls and supports i.e., soil springs at the back of the basement walls. Furthermore, abrupt change in shear demand at the mid-height of the tower in y-direction is due to the BRBs. BRB cause reduction in seismic shear demand; however, little effects in storey moment demand.

8.4. Storey drift

In the preliminary investigation, the storey drifts are checked at MCE level without using BRBs. Then, the BRBs are applied in the model and the storey drifts are rechecked. One advantage is that BRB system reduces the storey drift of the building in the principal minor direction. The maximum storey drifts envelope for both principal directions are less than 3% which is acceptable limit



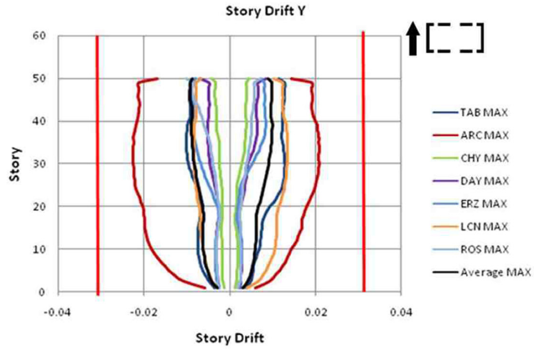
(a) About Y-axis (Minor dir.)



(b) About X-axis (Major dir.)

Figure 9. Plot of Storey Moment (a) aboutr Y, (b) about X.

(a) Along X-axis (Major dir.)



(b) Along Y-axis (Minor dir.)

Figure 10. Plots of Storey Drift (a) along X, (b) along Y.

against MCE level earthquakes as shown in Fig. 10.

8.5. Ductility of buckling restrained braces

In order to evaluate the response of BRBs and performance level, the ductility of buckling restrained braces is checked against the acceptable limit. Firstly, the strain for each BRB is extracted from each analysis and average ductility demand is calculated. It is found that all BRBs have average ductility demand less than 9, which is the maximum allowable ductility demand for primary braces components mentioned in ASCE 41-06.

8.6. Axial strain in core wall

The flexural capacity of shear wall is evaluated in terms of the yielding of vertical steel and crushing of concrete materials. The strain in steel fiber fibers and concrete fiber fibers are checked against the acceptable strain limits. The compression strain of MCE analysis is increased by 2 times and compared with the acceptable limit. From the results, all the strains are within the acceptable limit.

8.7. Effectiveness of BRB system

Since the buckling restrained braces are yielded significantly at MCE level earthquakes, the design base shear is reduced compared to the building without BRB system. Especially, moment and shear in the core wall is reduced remarkably due to the utilization of BRB system.

9. Conclusion

The buckling restrained braces system is used as first time in the primary lateral force system of the high-rise building in Philippines. The design and application of buckling restrained braces are initiated into local structural engineering practice. The buckling restrained braces combined with ductile core wall systems lead to a better performance for tall buildings for reducing base shear and controlling deformation.

Acknowledgements

The authors would like to thank Magnusson Klemencic Associates for their peer review, as well as their guidance in design of BRB system for the case study project. Furthermore, the authors acknowledge Star Seismic Inc. for the provided information about buckling restrained braces. Lastly, the authors extend sincere thanks to Ayala Land, Inc., the Client of the case study project.

References

ACI 318-08. (2008). Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318 R-08). American Concrete Institute: Farmington Hills, MI.

Ahmed, M. (2011). Optimal Reduction Inelastic Seismic Demand in High-rise RC Core Wall Buildings Using Energy Dissipating Devices. Dissertation, Structural Engineering Field of Study, Asian Institute of Technology.

American Society of Civil Engineers (2005). Minimum Design Loads for Building and Other Structures (ASCE 7-05). American Society of Civil Engineers.

American Society of Civil Engineers (2006). Seismic Rehabilitation of Existing Buildings (ASCE 41-06). American Society of Civil Engineers.

Computers and Structures, Inc. (2006). Perform 3D, Nonlinear Analysis and Performance Assessment for 3D Structures User Guide, Ver. 4. Computers and Structures, Inc.

Berkeley, C. A., Dutta, A., and Hamburger, R. O. (2011). Case Study of a 40-Storey Buckling-Restrained Braced Frame Building Located in Los Angeles. The Structural Design of Tall and Special Buildings, Wiley InterScience. 19, pp. 77~93.

Fry, J. A., Hooper, J. D., and Klemencic, R. (2010). Core Wall Case Study Design For Pacific Earthquake Engineering Research/California Seismic Safety Commission. The Structural Design of Tall and Special Buildings, Wiley InterScience. 19, 61-75.

Klemencic, R., Fry, J. A., Hurtado, G., and Moehle, J. (2006). Performance of Post-Tensioned Slab-Core Wall Connections. *PTI Journal*, 4(2), pp. 7~23.

Los Angeles Tall Buildings Structural Design Council. (2008). An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located Los Angeles Region. Los Angeles Tall Buildings Structural Design Council.

Orakcal, K. and Wallace, J. W. (2006). Flexural Modeling of Reinforced Concrete Walls - Model Calibration. *ACI Structural Journal*, 103(2), pp. 196~206.

Wallace, J. W. (2007). Modeling Issues for Tall Reinforced Concrete Core Wall Buildings, The Structural Design of Tall and Special Buildings, Wiley InterScience, 16, pp. 615~632.

Wallace, J., Naish, D., Fry, J. A., and Klemencic, R. (2009). Presentation, PEER Annual Meeting, Tall Buildings Initiative Session. October 15-16, San Francisco, CA.