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Performance Based Seismic Design State of Practice, 2012 Manila, Philippines

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Abstract

The purpose of this paper is to present the state of practice being used in the Philippines for the performance-based seismic design of reinforced concrete tall buildings. Initially, the overall methodology follows "An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region, 2008", which was developed by Los Angeles Tall Buildings Structural Design Council. After 2010, the design procedure follows "Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, 2010" developed by Pacific Earthquake Engineering Research Center (PEER). After the completion of preliminary design in accordance with code-based design procedures, the performance of the building is checked for serviceable behaviour for frequent earthquakes (50% probability of exceedance in 30 years, i.e,, with 43-year return period) and very low probability of collapse under extremely rare earthquakes (2% of probability of exceedance in 50 years, i.e., 2475-year return period). In the analysis, finite element models with various complexity and refinements are used in different types of analyses using, linear-static, multi-mode pushover, and nonlinear-dynamic analyses, as appropriate. Site-specific seismic input ground motions are used to check the level of performance under the potential hazard, which is likely to be experienced. Sample project conducted using performance-based seismic design procedures is also briefly presented.

Keywords: Maximum considered earthquake, Design basis earthquake, Service/Frequent earthquake, Performance-based seismic design

1. Introduction

Performance-based design is a state-of-the-art design tool in the seismic design, which has been widely used for seismic evaluation of existing buildings and seismic design of number of new tall buildings. The conventional seismic design codes apply the global response modification factors (R factors) as the important role in the determination of seismic design forces. The R factor accounts for reduction of seismic forces to predict the inelastic response of the building, resulting from the simplified elastic analysis methods. The shortcoming is that R factor does not account for the structural performance of component level, as well as the seismic ground motion characteristics. The elastic analysis procedures do not consider the redistribution of seismic demand in the various components of the building at the state of inelastic behaviour under strong seismic events.

In contrast to prescriptive design approaches, performance-based design provides a systematic methodology for assessing the performance capability of a building, system or component. The performance-based design explicitly evaluates the response of the building under the potential seismic hazard, considering the probable site-specific seismic demands as well as the uncertainties in the post-yielding response and behaviour of the building under seismic events.

Since late-2000s, performance-based design procedures have been utilized in the design of most of the tall buildings in the Philippines. Most of the buildings are 40- to 70-storey tall, reinforced concrete residential buildings. The structural systems are dual system (special moment resisting frame with shear walls) and bearing wall system.

2. Overall Methodology

Initially, the overall methodology follows "An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region, 2008", which was developed by Los Angeles Tall Buildings Structural Design Council. After 2010, the design procedure follows "Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, 2010" developed by the Pacific Earthquake Engineering Research Center.

As a beginning step, a schematic design is carried out to achieve the good performance and cost effectiveness of the structural system. The most appropriate gravity and lateral load resisting system is selected to provide a regular dynamic response, redundant and continuous load

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Table 1. Seismic Performance Objectives

Level of Earthquake	Seismic Performance Objective
Frequent/Service: 50% probability of exceedance in 30 years (43-year return period), 2.5% of structural damping	Serviceability: Limited structural damage, should not affect the ability of the structure to survive future Maximum Considered Earthquake shaking even if not repaired.
Maximum Considered Earthquake (MCE): 2% probability of exceedance in 50 years (2475-year return period), 2 to 3% of structural damping	Collapse Prevention: Building may be on the verge of partial or total collapse, extensive structural damage; repairs are required and may not be economically feasible.

path under gravity and lateral loadings, in close collaboration with the project architects. Normally, either the beam and slab system or post-tensioned flat slabs are used in the gravity load resisting system.

Following the schematic design, the preliminary design is carried out in accordance with the National Structural Code of the Philippines to determine the size of the members and reinforcements. The overall response of the building, including modal response parameters, base shear, inter-storey drifts, lateral displacements, storey shear, storey moments and distribution of shear between shear wall and moment resisting frames are checked to ensure that the vertical and lateral stiffnesses of the structural system are adequate.

After substantial completion of code-based design, performance-based evaluation is carried out to check the performance at two levels of earthquakes: serviceable behaviour for Service/Frequent Earthquakes (50% probability of exceedance in 30 years with 43-year return period) and Maximum Considered Earthquakes (MCE) (2% of probability of exceedance in 50 years, 2475-year return period). Then, the design is revised as appropriate, based on the performance-based evaluation results and findings in order to meet the seismic performance objectives and acceptance criteria set for the project.

3. Seismic Performance Objectives

The specific performance objectives for the design of the building at two levels of earthquake hazards are shown in Table 1.

4. Acceptance Criteria

The following global acceptance criteria and component level acceptance criteria are used to check the performance level of the building at specified levels of earthquakes.

4.1. Frequent/Service level earthquake

Response spectrum analysis is conducted using the site-specific service level response spectrum. Story drift is limited to 0.5% of storey height in any storey to prevent the damage of non-structural components and minimize the permanent lateral displacement of the building. For component level assessment, demand to capacity ratios of the primary structural members are limited to 1.5, in which the capacity is computed by nominal strength multiplied by the corresponding strength reduction factors defined in ACI 318-08.

4.2. Maximum considered earthquake

Nonlinear time history analysis is conducted using the site-specific ground motion records. Two types of storey drifts; peak transient drift and residual drift are checked in the global acceptance criteria. The mean peak transient drift is limited to 3% in any storey and maximum transient drift resulting from any pair of ground motion is limited to 4.5%. The mean value of residual drift is limited to 1% and maximum residual drift from any analysis is limited to 1.5.

In component level acceptance criteria, two types of actions, either force-controlled actions or deformation-controlled actions are evaluated in each component based on their response.

For fore controlled actions, the failure of brittle elements which could result in structural collapse, the capacity is calculated by using expected material strength and code-specified strength reduction factors. The force-controlled actions are checked against 1.5 times the mean in the case of computed demand is not limited by well-defined yielding mechanism. Otherwise, for well defined yielding mechanism, the mean plus 1.3 times the standard deviation obtained from individual time history analysis, but not less than 1.2 times the mean.

The deformation capacity of the components to withstand the imposed deformation demands is evaluated using the expected material properties and strength reduction factor of 1. The maximum considered earthquake level performance acceptance criteria is shown in Table 2.

5. Seismic Input

The Philippines is located on one of the most seismically hazardous regions of the world. Currently, the structural design of the buildings in the Philippines is governed by the “National Structural Code of the Philippines” (NSCP), developed by the Association of Structural Engineers of the Philippines. The most recent version of the code was released in 2010 incorporating most of the development to date. However, many important provisions of the code, in particular, those related to the seismic design and safety, could not be made in line with the international standards because of the lack of

Table 2. Maximum Considered Earthquake Level Performance Acceptance Criteria

Element	Action Type	Classification	Expected Behavior	Acceptance Limit
Girders	Flexure hinge rotation	Ductile	Nonlinear	Hinge rotation ≤ ASCE41-06 limit
	Shear	Brittle	Linear	D/C < 1
Columns	Axial-Flexural interaction	Ductile	Nonlinear	Hinge rotation ≤ ASCE41-06 limit
	Shear	Brittle	Linear	D/C < 1
Shear Walls	Axial-Flexural interaction	Ductile	Nonlinear	Tensile strain in rebar ≤ 0.05 Compressive strain in rebar ≤ 0.02 Compressive strain in concrete ≤ 0.005
	Shear	Brittle		D/C < 1
Coupling Beam (Diagonal Reinforcement)	Shear hinge rotation	Ductile	Nonlinear	0.03 radian
Coupling Beam (Conventional Reinforcement)	Flexure hinge rotation	Ductile	Nonlinear	Hinge rotation ≤ ASCE41-06 limit
	Shear	Brittle	Linear	D/C < 1
BRB	Axial	Ductile	Nonlinear	Ductility demand ≤ ASCE41-06 limit

proper definition of seismic hazards and mapping. The seismic parameters and maps currently available were developed to support the "Uniform Building Code" (1997) and the corresponding NSCP.

This disparity is leading to the design of structures to a much older and arguably less safe provisions. The recent international codes, practices and design software and tools do not support the outdated data and methodologies, causing difficulty for the structural engineer, the client, and building officials, to ensure public safety.

In the meantime, the probabilistic seismic hazard assessment has been carried out for Metro Manila, which quantifies the hazard at a site from all earthquakes of all possible magnitudes, at all significant distances from the site of interest, as a probability by taking into account their frequency of occurrence. Seismic hazard maps for the earthquakes with different return periods (43-year, 475-year and 2475-year) are developed, which are consistent with the latest development in design methodologies being used around the world.

For performance based-design, conditional mean spectra (CMS) are developed for the key natural periods of the building at the site of interest. The earthquake scenarios of each CMS at selected sites are obtained from the results of geographical deaggregation which include earthquake magnitude, source-to-site distance, epsilon, azimuth, and soil condition. The selected ground motion records are matched to the suite of conditional mean spectra, each matched to the key natural periods. For sites located closer than 5 km from causative fault, the chosen orientation of scaled and selected strong ground motions are in fault normal (FN) and fault parallel (FP) directions. For sites located beyond 5 km from causative fault, the orientation of selected and scaled strong ground motion would be mixed between as-recorded and fault normal orientation.

6. Modeling Procedures

Three-dimensional finite element models of the building are created with varying complexity and refinement, suitable for developing understanding of the response. Elastic models are created in ETABS computational platform for the preliminary design at DBE level earthquakes and evaluation at Service level earthquakes. For MCE level evaluation, nonlinear finite element models are created in SAP2000 or PERFORM 3D software, based on the geometric configuration of the building.

6.1. Columns and girders

In the nonlinear model, columns and girders are modeled as frame elements. To model the post-yielding behaviour of the girders and columns, plastic hinges are applied in the frame elements in the finite element model. Uncoupled moment hinges are assigned to both ends of the girders and coupled P-M2-M3 hinges are assigned to the ends of the columns at the foundation or podium level. The deformation capacities of the hinges are taken from ASCE 41-06. Nonlinear response is not considered in secondary beams since they are not part of the primary lateral load resisting system.

6.2. Shear walls

In modelling of shear walls, effect of confinements is taken into account for the compressive strength and ductility of concrete. Mander's (1994) confinement model is used to determine the confinement effect. In PERFORM 3D, fibre modelling technique is used to model the flexural behaviour of the shear walls. PERFORM-3D shear wall element is used to model the nonlinear behaviour of shear wall. In SAP2000, nonlinear layered shell elements are used to model the flexural behaviour of shear walls. The entire cross section of shear wall is discretized into separate layers; concrete and vertical reinforcements. These layers are located by a specific distance from the reference surface and with the specified thickness. The material properties of each layer are specified by the properties of concrete and steel. Each layer is assigned as shell, membrane or plate element depending upon the requirement. The hysteretic response of the wall section is simulated by assigning the hysteretic behaviour in the property of concrete and steel materials explicitly.

6.3. Coupling beams

Deep coupling beams with the diagonal reinforcement, having span to depth ratio less than 4, are modelled for shear deformation controlled actions. Nonlinear shear hinge is assigned at the mid span of the element, in which the yielding capacity is calculated based on diagonal reinforcement. The slender coupling beams with conventional longitudinal reinforcement, having span to depth ratio greater than 4, is modelled for flexural deformation controlled actions. Flexural hinges are assigned at the ends of the beam. Yielding capacity of the flexural hinges is calculated based on the longitudinal reinforcements provided in the beams. The deformation capacities of the hinges are taken from ASCE 41-06. In addition, the slender beams are checked for shear capacity to meet the acceptance criteria for brittle elements.

6.4. Floor slab

In the bearing wall system with ductile core wall, equivalent "slab outrigger beams" are used in the tower portion to model the response of slab between core wall and column. Slab outrigger beams are modelled with nonlinear flexural hinges at both the ends. The moment capacity of the slab beam is calculated based on the reinforcement in the slab. However, the performance of the moment hinges is not specifically reviewed. Rigid diaphragm behaviour is assumed for the in-plane behaviour of the slab.

The slabs in the dual systems, at the podium and basement levels of the bearing wall systems, the slabs are modelled as shell elements without considering the rigid floor diaphragm assumption.

6.5. Buckling Restrained Braces (BRBs)

In PERFORM 3D, "BRB compound component" is used to model the BRBs while "Multi-linear Plastic Link element" with kinematic hysteresis behaviour is used in SAP2000. The backbone curves used for the modelling of BRB is shown in the following Fig. 1. The coefficients Ry, ω, and β are estimated based on the properties of the BRBs provided by the BRB supplier. The initial stiffness (Ko) of the BRB is estimated based on cross sectional properties and material properties by As E/L (where As cross sectional area of Steel, E is Young's modulus of Elasticity of steel, and L is the effective length of the

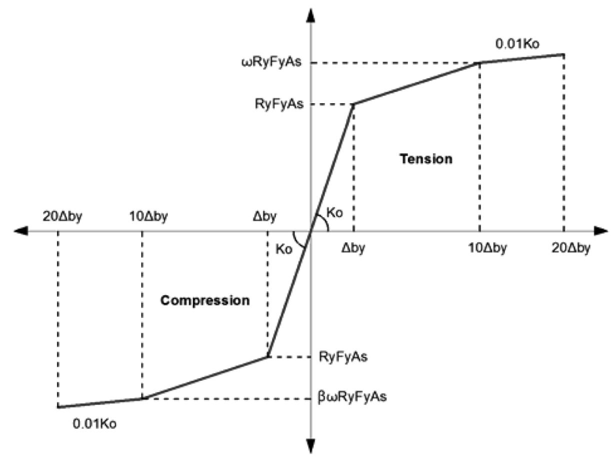


Figure 1. Assumed backbone curve for buckling-restrained braces.

brace in inelastic behaviour i.e. approximately 70% of the pin to pin BRB length).

6.6. Foundation

Soil-structure interaction is considered in the modelling of foundation. Equivalent vertical and lateral spring stiffness values are calculated based on the information from the soil report. Area springs are used in basement walls and mat foundations while line springs are used for pile foundations.

6.7. Damping

In the preliminary design at DBE level, 5% damping is used in the analysis. In service level evaluation, 2.5% damping is considered while 2% to 3% of damping is considered in the analytical model for MCE level evaluation.

7. Peer Review and Approval Process

Since the performance-based designed approach is a relatively new concept and falls outside of the prescriptive building code, the independent peer reviews have been a general practice or requirement as described in PEER (2010) methodology. The ultimate objectives are to design economical, technically engineered and robust structure, and ensuring the public safety. The scope of the reviewer's team is established at the beginning of the project and generally limited to seismic design; however, scope may include the wind design and non-structural components under the building official discretion. The peer review process has significant contribution to produce improved building design, though there may be several disagreements between the designer and the reviewer in the design process. The review process can be iterative (issues resolved with additional analyses or study) to finally come to the agreements between the owner, the designer, and the reviewers. After incorporating all the comments/suggestions of peer reviews and coming to the consensus, the reviewer team submits a written report to the owner/building official indicating the reviewer's scope, comments and opinions regarding the general conformance of the design to the requirement of the design criteria and specified performance objectives.

8. Sample Project

The Park Terraces Tower Project is located in Arnaiz St., Makati City near the Glorieta Commercial Center, consisting of three high-rise residential towers resting on a one-level common podium (Fig. 2(a)). Two residential towers are identical and 50-storey tall (166.8 m above ground level). The remaining one is a 62-storey high-rise building (about 200 m above ground level). The typical storey height of the building is 3.1 m. The towers consist mainly of residential units, and a terrace and amenity deck. The ground level contains retail and back of the house space. The towers have 3½-storey of below grade parking, resting on the mat foundation. The total project area is approximately 79,000 m^2. Reinforced concrete bearing walls, gravity columns and post-tensioned (PT) flat slabs are used in the gravity load resisting system. Amongst the three towers, Tower 1 is presented as sample project in this section. The lateral load resisting system consists of reinforced concrete bearing wall coupled with outrigger columns, connected by the BRBs as shown in Fig. 2(b) for the Tower 1. The typical floor plan of the lateral resisting system is shown in Fig. 3, which consists of shear core wall for lateral load and columns for gravity loading. In order to limit the lateral displacement of the tower, sixteen Buckling Restrained Braces were provided at the locations shown in Fig. 2(b). The main energy dissipating elements in this tower are designed, from yielding of coupling beams, yielding of BRBs, and yielding of main core shear wall at the ground level.

From the modal analysis, the natural periods of first 3 modes were found to be 5.75s, 4.86s, and 3.77s respectively. Nonlinear modelling and analysis procedures follow the steps as described earlier. In this project, the building was first designed using DBE earthquake spectra by considering response modification factor (R) equal to 5.0. The selection of R values is made in agreement with the PEER reviewer's team. The selection of R value is mainly governed by the level of shear demands in the core-wall and an appropriate value is selected to reflect the actual behaviour of the building designed.

In Service level evaluation, the storey drifts and lateral displacements are found to be within the acceptable limits of 0.5% and H/200 (H = height of building) respectively. The capacities of the primary structural members were

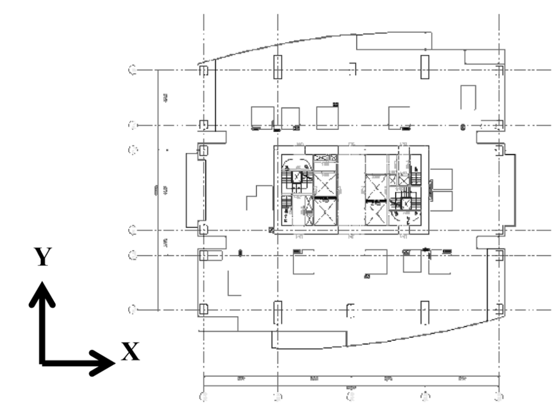
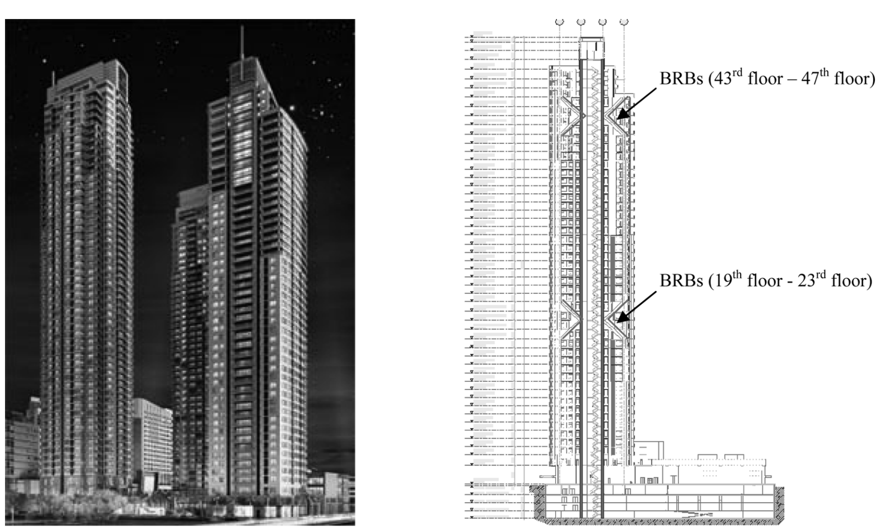


Figure 3. Typical floor plan for lateral resisting system in Park Terraces Tower 1.



3D-View of Park Terraces Towers

Lateral resisting system elevation of park Terraces Tower 1

Figure 2. Sample project a) 3D view b) Lateral resisting system elevation view.

checked under Service level seismic demand so that the members remain elastic under Service level earthquakes.

Table 3 shows the comparison of base shear obtained from DBE response spectrum analysis and nonlinear time history analysis at MCE level. It is observed that the nonlinear demand at MCE level on the building is much higher (approximately 2 times) than the DBE response spectrum analysis. These higher shear demands led to the revision of the reinforcing detailing of the shear wall and designed for more ductility in the building. Therefore, it was a great concern to select the appropriate R values to ensure proper stiffness and strength with the consequences of possible increased in shear demands.

The profile plot of storey shear and storey moments at MCE level are shown in Figs. 4 and 5. In the design of primary structural members against the brittle failure mode at MCE level, the demands in the primary members are increased by 30% to account for the additional uncertainty in the variation of shear forces under dynamic loadings and this methodology is well discussed with the peer reviewer's team for approval.

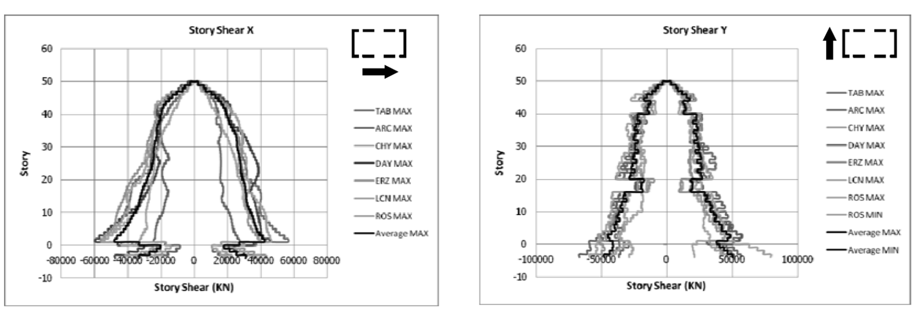
Table 3. Base Shear Comparison

Load Cases	Base Shear (KN)	% of Seismic Weight
DBE -X	21,012	3.56
DBE -Y	22,691	3.84
MCE -X	47,892	7.76
MCE -Y	46,462	7.53

In terms of global response, the storey drifts at MCE level are found to be under acceptance criteria (i.e. 3%) as shown in Fig. 6. The BRBs were quite effective in reducing the deflection of building and all BRB's were yielding at MCE level. 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. In addition, deep coupling beams are found to be meeting the acceptance criteria for shear deformation at MCE level. The slender coupling beams also satisfied the rotation limit at the end; however, some of the coupling beams were found to be failed in shear at MCE level. Revisions were made for the members with inadequate capacity to resist the demand.

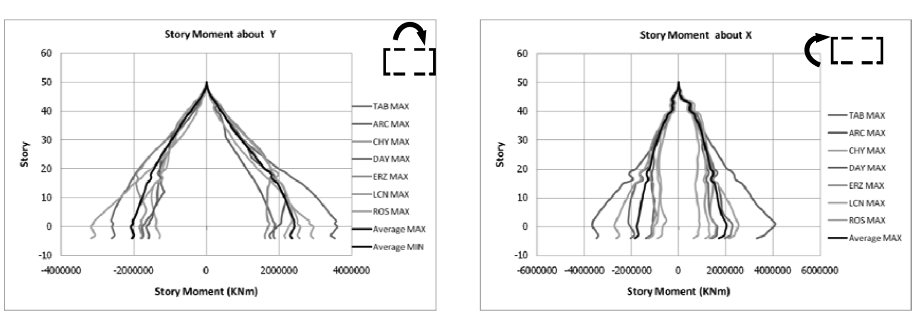
9. Conclusion

In conclusion, performance-based seismic design is common in local structural engineering practice, as well as in the awareness of the real estate developers in the Philippines. Most of the high-rise buildings are designed using performance-based seismic design procedures in accordance with the most recent guidelines and standards. In spite of the absence of proper definition of seismic hazards and mapping in National Structural Code of the Philippines, probabilistic seismic hazard assessment is conducted for Metro Manila and site-specific seismic



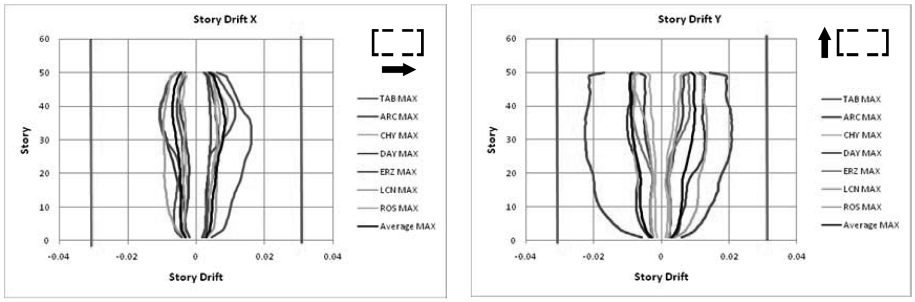
(a) Story Shear Along X (b) Story Shear Along Y

Figure 4. Story Shear Plots from Nonlinear Analysis at MCE level.



(a) Story Moment About Y (b) Story Moment About X

Figure 5. Story Moment Plots from Nonlinear Analysis at MCE level.



(a) Story Drift Along X (b) Story Drift Along Y

Figure 6. Story Drift Plot from Nonlinear Analysis at MCE level.

hazard information is used in the performance-based design of tall buildings located in the Manila area.

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