SEISMIC PERFORMANCE EVALAUATION OF TYPICAL NEW TALL BUILDINGS CONSTRUCTED IN VANCOUVER CANADA

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ABSTRACT

With the growing population and limited space in major cities worldwide, high-rise construction is becoming the only solution in solving the rapid urbanization issue. In area such as Vancouver, British Columbia, these high rise buildings are been constantly threatened by the potential that a large earthquake could rupture in the region. To address this concern, a detailed inventory of tall buildings in Vancouver has been created. A representative prototype building was designed according to the 2010 National Building Code of Canada and CSA A23.3-04. The prototype building was used to assessment the safety and performance of typical tall buildings in Vancouver. The evaluation uses the state-of-the-art performance assessment which accounts for the detailed seismic hazard, modeling analysis, fragility data and loss analysis to quantify the performance of the structure under the design level earthquake. The results can be used immediately by the building owners and other stake holders to make informed risk management decisions.

Keywords: Vancouver Tall Building Structure, Seismic Performance, Nonlinear Dynamic Analysis, Reinforced Concrete Shear Wall.

1. INTRODUCTION

Vancouver is third largest city in Canada and the second largest city in Pacific Northwest. The Vancouver metropolitan area is experiencing an upsurge of high-rise construction. Majority of the buildings are designed according to the National Building Code of Canada (NBCC) (NRC, 2010a). The current design code provides minimum requirements to achieve "life safety" performance in the event of a major earthquake. Life safety is achieved by ensuring a low probability of collapse, but does not necessarily prevent extensive structural and nonstructural damages which are not economical to repair after a major earthquake. UBC initiated a tall building research initiative to better understand the seismic performance of the recently constructed tall buildings in Vancouver designed to current codes, and provide design guidelines for improved seismic performance. The initial phase of this initiative is the identification, development and design of a typical tall building in Vancouver, followed by an in-depth seismic performance assessment which is presented in this paper.

2. CHARACTERISTICS OF TALL BUILDINGS IN VANCOUVER

A prototype building which reflects the state of design practice is developed to serve as the subject of the performance assessment. Detailed review of multiple recently constructed tall buildings in Vancouver was carried out to identify common characteristics. Based on the observation of the several typical tall buildings in Vancouver, a prototype building is developed.

2.1. Building Survey

At the time of survey, only a few buildings in the Metro Vancouver region has been constructed taller than 100 meters which are outside the Downtown Vancouver area. Hence, this building survey was limited to tall buildings in Downtown Vancouver. A total of 50 buildings taller than 100 meters have been identified. Figure 1 shows the location and year of construction completion for these buildings.

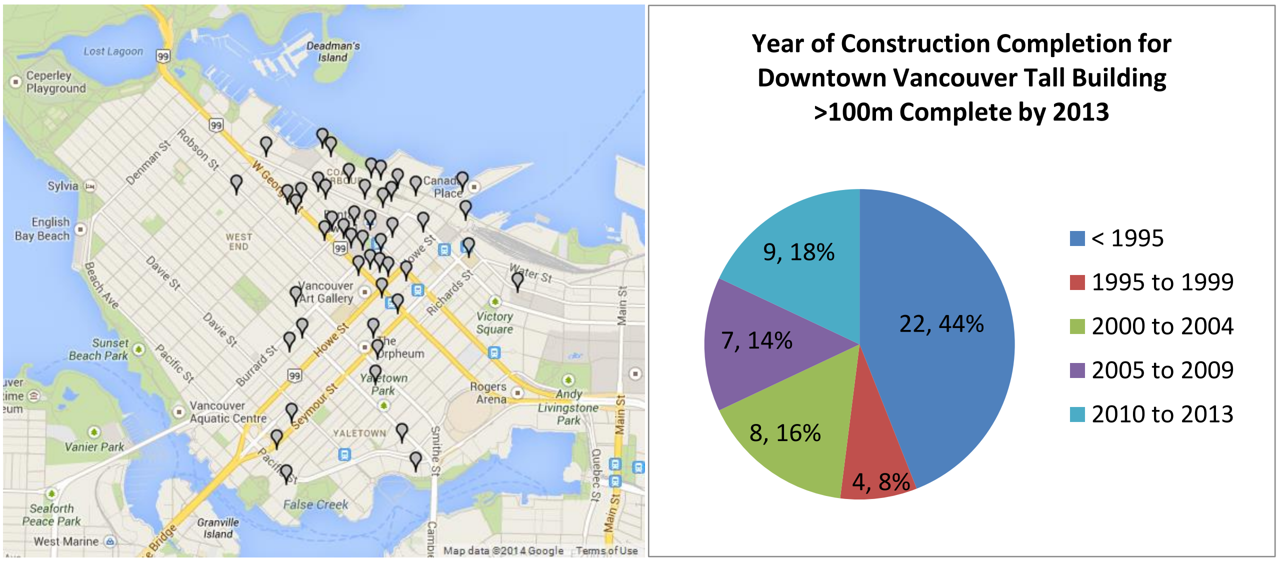

Figure 1: Year of Construction Completion and Geographical Distribution of Tall Buildings in Downtown Vancouver

To limit the scope of this study, the building inventory is then further limited to buildings constructed after 2005. This is because there was a major revision in the seismic provisions in the 2005 version of the National Building Code of Canada (Mitchell et al., 2010). A total of 16 buildings have been selected. One out of the 16 buildings is an office building, while the remaining 15 buildings are mixed residential, hotel and commercial usage. Most of the buildings feature an above-ground podium that extends beyond the footprint of the tower. Detailed review of the common characteristics of the inventoried buildings was carried out in order to produce a typical prototype building. The prototype building represents the typical newly constructed tall building in Vancouver, Canada. All inventoried buildings use reinforced concrete shear walls (coupled and non-coupled) with flat slabs (either post-tensioned or conventionally reinforced). Most of these buildings are between 100 meters to 150 meters tall. Gravity columns with high aspect ratios are commonly employed because these columns minimize the interference for the views. In addition, these columns can be fit into partition walls between units.

2.2. Prototype Building Description

Figure 2 shows the 3D and typical floor layout views of the prototype building. The prototype building is a 40 story, 126.5m tall, mixed-use residential and commercial reinforced concrete building with 4 parking basement levels and 3 commercial podium levels. The typical tower floor has a 22.5m x 33m floor plate with a shear wall core foot print of 9.6m x 9.75m. The podium structure has a foot print of 40m x 58.35m. The podium has multiple 400 mm thick perimeter shear walls extending from the top of the podium to the foundation. In addition, 300mm thick perimeter retaining walls are placed below grade in the region where the shear walls are not present.

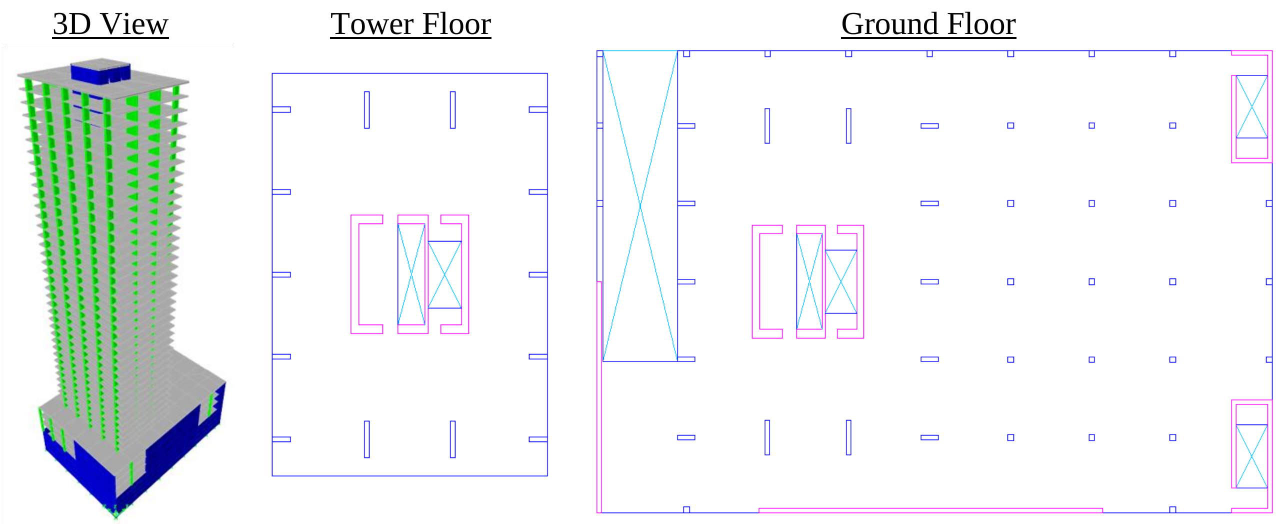

Figure 2: 3D View and Typical Floor Layout of Prototype Building

The design of the prototype building is carried out in accordance with the 2010 NBCC (NRC, 2010a) and CSA A23.3-04 Design of Concrete Structures with 2009 updates (CSA, 2009). The seismic and wind demand are calculated in accordance with the linear dynamic seismic procedure and dynamic wind procedure specified in the 2010 National Building Code of Canada (NRC, 2010b), respectively. Ductile detailing and capacity design principles are applied, as required by CSA A23.3-04 to provide sufficient ductility and ensure that yielding will develop in favorable ductile failure modes. Linear analysis are performed with ETABS (CSI, 2011a) to reflect common design practice. The buildings has a fundamental period of 6.7 seconds for seismic analysis with cracked effective stiffness, and a fundamental period of 5.5 seconds for wind analysis with modified cracked effective stiffness.

35MPa concrete is employed for all the columns and walls of the podium and basement. Tower columns, core walls and coupling beams employ concrete strength of 55MPa, 45MPa and 35MPa for the lower levels (P4-9F), mid levels (10F-24F) and upper levels (25F-Roof), respectively. 35MPa concrete is employed for slabs and roofs at all elevations.

The LFRS utilizes a ductile coupled shear wall in one direction, and a ductile shear wall in the other direction. The shear wall thicknesses are constant up the height of the building. The shear walls are 750 mm thick in the ductile coupled shear wall direction, designed with seismic ductility force modification factor (R_d) of 4.0 and overstrength force modification factor (R_o) of 1.7. The shear walls are either 600 mm or 300 mm thick in the ductile shear wall direction, designed with R_d of 3.5 and R_o of 1.6. The shear walls have three lifts of decreasing reinforcement ratio up the height of the building. The distributed vertical reinforcement ratio in the shear walls varies between 1.1% and 0.3%, while the vertical reinforcement ratio in the end zone varies between 2.08% and 0.35%. The coupling beams are 750mm wide by 700mm deep with span-to-depth ratio (l_u/l_d) ranging from 1.5 to 1.7. All coupling beams are diagonally reinforced with configurations ranging from 4-10M to 8-35M bars in each diagonal. The coupled shear walls have a 93% degree of coupling, thus, satisfying the 66% minimum degree of coupling requirement for ductile coupled shear walls.

The gravity system consists of reinforced concrete flat slabs supported by perimeter blade columns (wall-like columns with high aspect ratio). The flat reinforced concrete slab system without drop panels or column capitals has integrity steel and shear stud rails at every column-slab connection. Tower floor slabs are 200mm thick. Podium and parking floor slabs are 250mm thick. Roof slabs are 350mm thick. The dimensions of the tower columns are 3000mm x 400mm and 1500mm x 400mm, which is constant up the height of the building. The columns are spaced at 6.75m to 7m along the perimeter of the tower. The tower columns have decreasing vertical reinforcement ratio varying from 3.7% to 1.0% up the height of the building. The podium gravity columns with dimension of 500mm x 500mm and vertical reinforcement ratio of 2.4% are spaced at 6.75m to 7m.

3. SEISMIC PERFORMANCE ASSESSMENT METHODOLOGY

Seismic performance of the prototype building under the 2% probability of exceedance in 50 year was carried out using the state-of-the-art performance-based earthquake engineering (PBEE) framework published by ATC-58 (ATC, 2012). The framework applies the total probability theorem to combine the four key components of PBEE, including seismic hazard analysis, response analysis, damage analysis and loss analysis, to quantify the seismic performance of the structure. A Monte Carlo simulation was used to carry out the large array of damage and cost analysis. Detailed implementation procedure is explained in depth by Yang et al. (2009).

3.1. Seismic Hazard Analysis

The 2010 NBCC's uniform hazard design spectrum for site class B was used as the target spectrum. Strong historical crustal earthquake ground motions were selected from the PEER Strong Motion Database (PEER, 2011) and amplitude scaled to match the target design spectrum. Only ground motions recorded with soil site class B (V_{s30}: 760-1500 m/s) were considered. To represent the seismic hazard at the site (Halchuk et al., 2007), only records recorded within 0 to 100km from the rupture epicenter and moment magnitude between 6 and 8 were included in the study. As recommended by

ASCE7-10 (ASCE, 2010), the ground motions were amplitude scaled to match the target spectrum between 0.2 to 1.5 times the fundamental period of the structure (the fundamental period of the structure is calculated to be 4.7 sec). The selected ground motions along with the corresponding scale factors and other key parameters are summarized in Table 1.

Table 1: Selected Input Ground Motions

NGA#	Mean Square Error	Scale Factor	Event	Year	Mag.	R_{JB} [km]	R_{rup} [km]	V_{s30} [m/s]	Mechanism
1165	0.023	0.93	Kocaeli- Turkey	1999	7.51	3.6	7.2	811	Strike-Slip
143	0.033	0.28	Tabas- Iran	1978	7.35	1.8	2.0	767	Reverse
804	0.035	3.95	Loma Prieta	1989	6.93	63.0	63.1	1021	Reverse-Oblique
297	0.036	1.79	Irpinia- Italy-02	1980	6.20	14.7	14.7	1000	Normal
2107	0.052	3.53	Denali- Alaska	2002	7.90	49.9	50.9	964	Strike-Slip
1587	0.053	4.41	Chi-Chi- Taiwan	1999	7.62	62.1	65.2	845	Reverse-Oblique
946	0.068	7.00	Northridge-01	1994	6.69	46.6	46.9	822	Reverse

The geometric means of the horizontal components of individual earthquake records are plotted against the target spectrum in Figure 3.

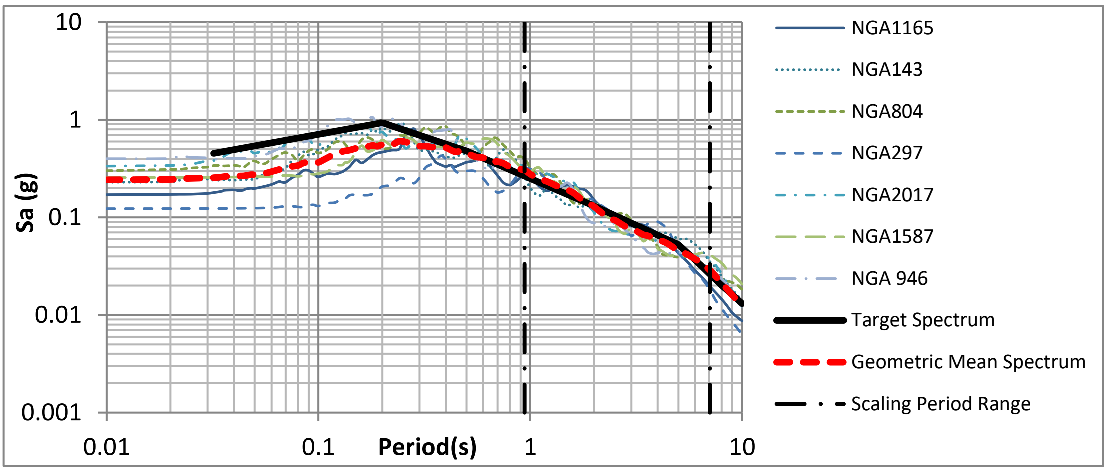

Figure 3: Geometric Mean Spectrum of Horizontal Component of Selected Ground Motions

3.2. Response Analysis

Nonlinear Dynamic Analysis was carried out using PERFORM-3D Version 5.0 (CSI, 2011b). The seismic weight was assigned as nodal weights according to tributary area. The weight was calculated using the combination 1.0 Dead load + 0.25 Snow load, which is approximately equal to 430 MN at the base of the structure. Gravity loads were also assigned as nodal loads using the combination 1.0 Dead load + 0.5 Live load + 0.25 Snow load. Soil-structure interaction (SSI) effects were not considered in the analysis. The ground motion records were applied at the base (foundation) of the structure without consideration of vertical component. Lateral earth pressures were also ignored. Rayleigh damping of 2.5% was assumed, and assigned at the first modal period (T_1) and 20% of the first modal period ($0.2T_1$). Guidelines for Performance-Based Seismic Design of Tall Buildings of Pacific Earthquake Engineering Research Center (PEER, 2010) and Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings of Applied Technology Council (ATC, 2010) and Task 12 Report for the Tall Buildings Initiative of PEER (Moehle et al., 2011), were adopted for the nonlinear modeling and analysis of the prototype building.

The median and individual ground motion responses, of the core wall response are shown in Figure 4. From the seven individual responses, statistical analysis was used to generate a large array of synthetic EDPs which have the same statistical distribution as the results presented in Figure 4.

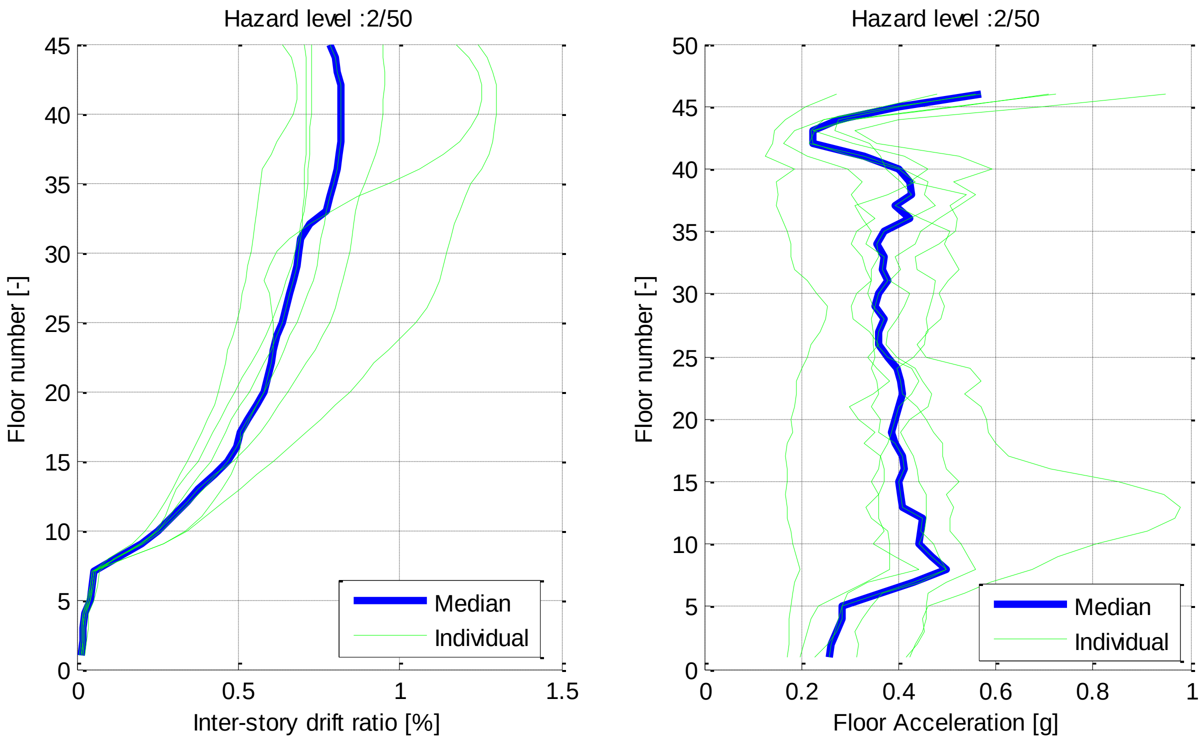

Figure 4: Peak Drift and Acceleration EDPs over the Height of the Building

3.3. Damage Analysis

Damage analysis was performed using fragility functions, which relate the potential damage states of PGs to EDPs from the Response Analysis. For the present study, ten PGs were selected: Gravity Columns (GC), Curtain Walls (CW), Interior Partitions (IP), Acceleration-sensitive building services and architectural components (INTA), Contents (CONT), Reinforced Concrete Slabs (SLAB), Core wall shear (SHEAR), Core wall reinforcement yielding (YIELD), Core wall crushing (CRUSH), and Coupling Beams (CB). Fragility functions were derived from ATC-58 and experimental tests, similar to those presented by Yang et al. (2012). Component quantities were estimated based on the intended use of the building.

3.4. Loss Analysis

The repair actions and associated costs were estimated based on data from the ATC-58 project (ATC, 2012) and engineering judgment. The loss analysis results for the ten PGs are shown below. The core wall is clearly the largest contributor to the overall repair cost for the building, predominantly from concrete crushing and reinforcement yielding due to flexure. No shear failure occurs in the wall, indicating that the capacity design principles have led to the desired limit state. Significant costs also resulted from the acceleration-sensitive building services and contents. Drift-sensitive partition walls, curtain walls, and concrete slabs also contributed, though to a lesser extent.

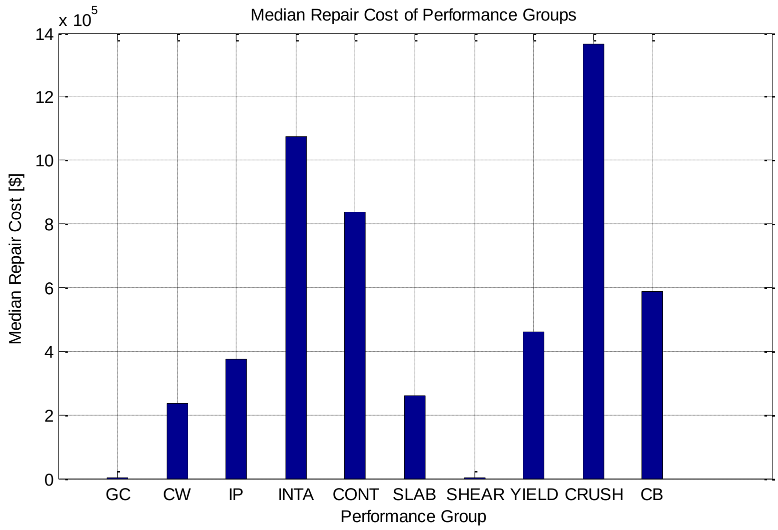

Figure 5: Median Repair Cost Estimate of Performance Groups

4. CONCLUSION AND FUTURE WORK

A representative newly constructed tall building is developed and design based on the observations from a detailed inventory study of recently tall buildings constructed in Vancouver, Canada. The performance of the building is investigated with nonlinear dynamic analysis and performance-based earthquake engineering assessment procedure outlined in ATC-58.

There are several ongoing and future research initiatives related to seismic performance of tall buildings. Some of these researches include the outrigger effect of slabs and gravity columns, effect of vertical ground motions, effects of damping assumptions, higher mode effects, backstay effect at podium level, and soil structure interaction.

The next major step is to collaborate with local practicing engineers in the development of practical seismic design guidelines to tackle seismic design challenges faced by practicing engineers.

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