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Application of Performance-Based Seismic Design to Tall Ordinary Shear Wall Buildings in Low to Moderate Seismicity Region

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Abstract

The objective of this study is to apply performance-based seismic design to high-rise apartment buildings in Korea considering collapse prevention level. The possible issues during its application were studied and the suggestions were made based on the findings from the performance-based seismic design of buildings with typical residential multi-unit layout. The lateral-force-resisting system of the buildings is ordinary shear wall system with a code exception of height limit. In order to allow the exception, the serviceability and the stability of the ordinary shear wall structure need to be evaluated to confirm that it has the equivalent performance as the one designed under the Korean Building Code 2009. The structure was evaluated whether it satisfied its performance objectives to withstand Service Level Earthquake and Maximum Considered Earthquake.

Keywords: Performance-Based Design; Service Level Evaluation; MCE Level Evaluation; Nonlinear Response History Analysis; Ordinary Shear Wall

1. Introduction

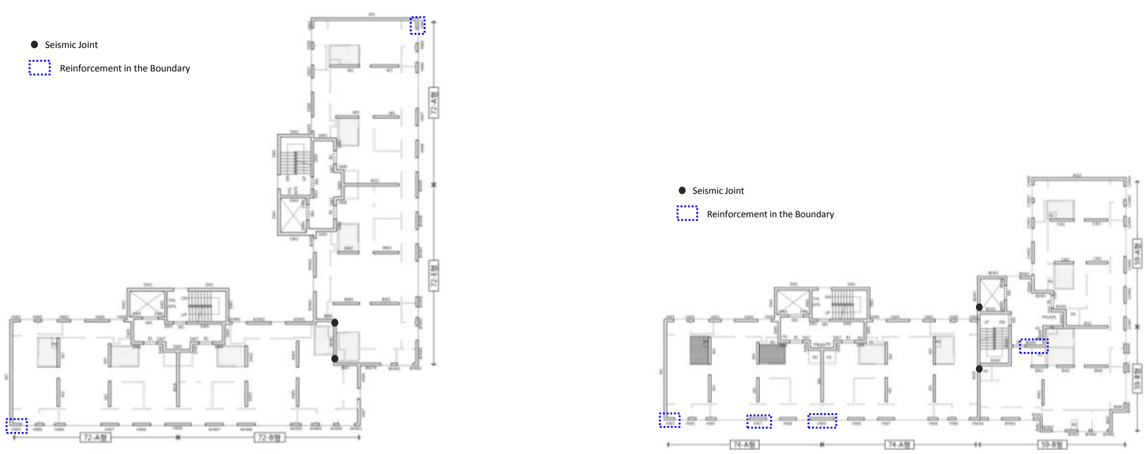
For a couple of decades, the significant progress based on findings from recent earthquakes and considerable research has given more explicit approach and procedure for performance-based seismic designs. The recent development by PEER (TBI) [1] covers detailed aspects involved in conceptual framework of performance-based seismic design and its application. PEER developed the guidelines as recommended alternative to the prescriptive procedure for seismic design of buildings contained in ASCE 7. The guidelines were developed considering the seismic response characteristics of tall buildings. In US, performance-based design has been mainly used when structural system was not covered by the current code and height limit specified by the current code for selected seismic-force-resisting system was exceeded. The Korean building industry embraced the same approach to apply ordinary shear wall system buildings, which has similar height limitation of 60 m at soil profile of Sd. Per KBC 2009 [2] the ordinary shear wall system is not allowed at soil profile S_d, which triggers more restrictive seismic design criteria. TBI recommends that a tall building should be designed for serviceability for service level effects and stability for ultimate limit states which are commonly used in other loadings to provide compliance of the current standards as the philosophy of Load-Resistant-Factored Design. While the main goal of TBI is to provide the equivalent performance in current seismic design codes with analyses of two discrete performance objectives as aforementioned, the performance objective of the Korean *guideline* [3] is limited to merely the Life Safety at so-called the 'Design Earthquake' (DE) as 2/3 of Maximum Considered Earthquake (MCE) which is identical to the prescriptive provisions. With this limitation of the guideline it is difficult to verify performance equivalence of performance-based designs conforming to the capability to withstand MCE shaking with low probability (on the order of 10%) of either total or partial collapse as the minimum performance objectives.

The objective of this study is to provide feasibility of the performance-based seismic design of tall ordinary shear wall buildings in low to moderate seismicity region and suggest its procedure and applications. To achieve this goal, the performance-based seismic design of 2 buildings with ordinary load bearing shear walls was conducted per TBI procedure. If some guidelines are not specified in TBI or needed to compare with Korean industrial practices in lieu of limited-ductile system, Korean *guideline* is referenced. The height of the buildings is around 82m and 74m. The buildings were first designed as prescribed by KBC 2009 with a code exception of height limit. In order to apply this exception, according to TBI, it is evaluated for the serviceability and stability of the structure to verify equivalent performance that intended for a similar building designed in conformance with the requirements of KBC 2009. The building and its components were then checked to meet the service–level acceptance criteria using a linear response spectrum analysis and collapse prevention criteria under MCE using 7- nonlinear response history analyses.

2. Building Descriptions

The tall residential buildings designed per TBI will be built in Incheon, Korea and has L-shaped residential floor plan as shown in Fig.1. The height of Building A is around 82 m and the building has 29 floors and 2 underground floors. Building B has 26 floors and 1 underground floors and its height is 74m. The typical story height at all floor above ground level, 1st basement floor, and 2nd basement floor is 2.8 m, 4.7 m, and 3.5 m, respectively.

The thickness of the slab is 210 mm, which is the minimum thickness required for wall-type apartment buildings in Korea to prevent transmission of excessive noise and vibration through the floor. The depth of the coupling beams in the cores is 60 mm. The thickness of interior walls, core and exterior walls, and basement walls are as thin as 200 mm, 220 mm, and 250 mm, respectively, and the ratio of the total wall area to the floor footprint is approximately 3% in each direction of the building. These slender wall configurations are very common in Korean construction industry and widely used in developing countries due to economic advantages. The similar type of shear wall system is easily found in typical buildings in Chile which were severely damaged during the 2010 Chile Earthquake (Mw 8.8). For Chilean RC wall buildings, the ratio of the total wall area to the floor plan area is on average roughly 3% in each principal direction of a building and wall thickness of 150 mm to 200 mm is common [4]. Based on these observations and recommendations, it is required that the acceptance criteria is needed to be more stringent to assure the stability of the structure with consideration of global buckling and rebar buckling, instead of increase of wall thickness due to architectural limitation.



(a) Building A (b) Building B

Fig. 1 Typical Floor Plan

3. Performance-Based Seismic Design

3.1 Establishment of Performance Objectives

To verify that the capacity of the ordinary shear wall structure is equivalent to that intended for a similar building designed in conformance with the requirements of KBC2009, the building was evaluated for the following two performance levels;

(a) Remain in essentially elastic range under Service Level Earthquake

(b) Withstand Maximum Considered Earthquake with very low probability of collapse

3.2 Design Process

Procedures of performance based design can be classified in three steps of the preliminary design, Service Level evaluation using linear analysis and MCE Level evaluation using non-linear response history analysis. The preliminary design step is aimed to design the member for performance-based design and proceed with the preliminary sizing and reinforcement arrangement of walls, coupling beams and slabs through linear response spectrum analysis. Service Level evaluation is intended to demonstrate that the structure will be capable of withstanding relatively frequent, moderate-intensity shaking with limited structural damage. Finally, in MCE evaluation step, the structure is evaluated to have acceptable response prevention when subjected to 2475 year return period earthquake. The structure is assessed as the mean response obtained from nonlinear response history analysis using MCE level ground motions that reflects the characteristics of the soil conditions. If performance is satisfied all of the target performance, design is complete but if it does not satisfy target performance, it should be re-perform the non-linear analysis with the redesigned structural members.

4. Preliminary Design

The preliminary design of building was performed per KBC 2009 with a code exception of height limit under DE level. And KCI 2012 [5] is used for the referenced material standard for concrete. Structural design and modal analysis were conducted using Midas ADS, which is a general-purpose finite-element-based structural analysis and design program. A 3-D linear dynamic model is composed of the shear walls, coupling beams and slab elements. The concrete strength of shear walls, coupling beams and slabs varies from 21 to 27 MPa. The concrete strength of all elements from 2nd basement to 8th floor, from 8th floor to 16th floor, and from 16th floor to roof level is 27 MPa, 24 MPa, and 21 MPa, respectively. The yield strength (f_y) of rebar is 500 MPa, but when the diameter of the rebar is larger than 16 mm, high strength rebar with 600 MPa is used. The specified strength of the elements are used in DE level using the elastic model and the expected material properties are used in the models for Service Level and MCE Level evaluations based on AIK(2015) (Table1). The different stiffness modifiers are selected per each model. A summary of stiffness modifiers is shown in Table 2. The effective stiffness used for Service Level and MCE Level evaluation followed TBI and Korean *guideline*, respectively. The fundamental period used in the design is listed in Table 3. The parameters and resulting forces used for wind and seismic design loads per KBC2009 are listed in Table 4 and Table 5. A linear response spectrum analysis was conducted for the seismic design.

Table 1. Expected Strength

Material	Nominal Strength(f_{ck} or f_y)	Factor
Concrete	Below 21MPa	1.2
	21MPa ~ 40MPa	1.1
Reinforcing Steel	500MPa ~ 600MPa	1.05
	Above 600MPa	1.0

Table2. Stiffness Assumption

Concrete Element	Code-Level Analysis	Serviceability Analysis	MCE-Level Nonlinear Models
Specified vs. Expected Concrete Strength	Specified concrete strength	Expected concrete strength	Expected concrete strength
Shear Walls	Flexural-1.0Ig Shear-1.0A	Flexural-0.75Ig Shear-1.0A	Flexural-1.0Ig* Shear-1.0A
Slab	Flexural-0.15Ig Shear-1.0A	Flexural-0.5Ig Shear-1.0A	Flexural-1.0Ig Shear-1.0A

*Because shear walls were modeled using fiber elements, the effective stiffness EI_{eff} is no assigned explicitly; the EI_{eff} decreased as the strains on the fiber elements increased.

Table3. Fundamental Period

Vibration mode		Period(s)	Dominant direction	Mass participation (%)
Bldg. A	1	4.00	Translational mode on X,Y direction	X-dir. : 26.5 / Y-dir. :19.0
	2	3.19	Translational mode on X,Y direction	X-dir. : 27.5 / Y-dir. :37.2
	3	2.84	Rotational mode	43.0
Bldg. B	1	2.81	Translational mode on X direction	59.32
	2	2.64	Translational mode on Y direction	27.27
	3	1.86	Translational mode on Y direction	33.39

Table 5. Wind Design Criteria

Parameter	Value
Basic Wind Speed	30 m/sec
Exposure Category	B
Importance Factor	1
Mean Roof Height	81.65 m
Gust Factor	1.795(Bldg. A)/ 1.819(Bldg. B)

Table 6. Seismic Design Criteria

Parameter	Value
Seismic Zone Factor	0.176
Importance Factor(I)	1.2
Site Class	S_D
Site Class Coefficients	F_a=1.448, F_v=2.096
Spectral Response Coefficients	S_{Ds}=0.425, S_{D1}=0.246
Seismic Design Category	D
Lateral System	Ordinary Shear Wall
Response Modification Coefficient(R)	4

5. Service Level Evaluation

To achieve the serviceability evaluation in performance-based seismic design, the Service Level earthquake is defined on the basis of a probability of exceedance of 50% in a service life of the structure. Here the service life of 50 years for new residential buildings is considered per a statistic data from Korean building industry. The seismicity of 72 years of return period is reasonable as serviceability level earthquake in Korea. Note that the serviceability limit state in high seismicity region such as West Coast in US is set for much frequent earthquake as 43 years of return period. [6] SLE spectrum shown in Fig. 2 is used for evaluation.

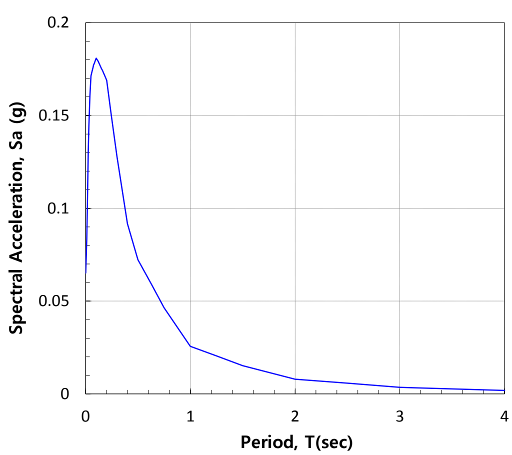


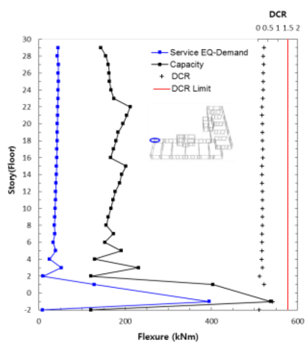
Fig.2 SLE Spectrum

The serviceability of these structures were evaluated using linear response spectrum analyses. The analyses included sufficient modes to include participation of at least 90 percent of the building mass for each principal horizontal direction of response. Modal responses are combined using Complete Quadratic Combination (CQC) method. The linear-elastic models used for design were revised with the stiffness assumption. The stiffness of elements is based on expected material properties per Table 1 and 2. Accidental eccentricity was not considered for serviceability evaluation per section "7.5.3" of TBI, and the structure were evaluated for the following load combinations (section "7.6.1" of TBI):

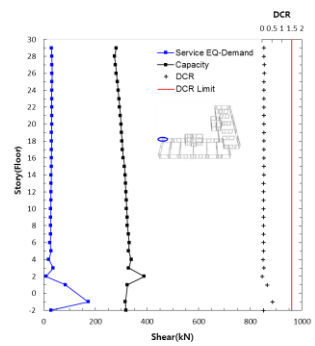
$$1.0D+0.25L\pm1.0E_x \pm0.3E_y \tag{1}$$

$$1.0D+0.25L\pm0.3E_x\pm1.0E_y \tag{2}$$

where D and L are the dead and the unreduced live Loads, respectively and response modification factors wasn't applied to serviceability evaluation. According to section 7.7 of TBI, the resulting element forces and deformation were evaluated with the acceptance criteria. Demand to capacity ratio (DCR) of the shear walls with flexure and shear action is limited to 1.5, where demand is computed form Equation (1) and (2) and capacity is calculated using specified material properties and code specified phi-factor. Fig. 3 shows the DCR of a single wall, which is located in the left corner of the plane, over the whole building and Fig. 4 plots the maximum DCR of each wall. As shown in the figures, the flexure and shear demand of all walls is below the provided capacity and the maximum value of the DCR among the all walls is 0.73 and 0.63 (for Bldg. A), 1.04 and 0.64(for Bldg. B). In addition, per TBI, story drift should not exceed 0.5% of story height in any story. According to the analysis results, the maximum values of the story drift at the corner of the plane are 0.02% (in X and Y-dir.) for Bldg. A and 0.03% (X-dir.) and 0.04% (Y-dir.) for Bldg. B. From these results, therefore, it was founded that these structures remained elastic and had sufficient performance under the Service Level Earthquake.

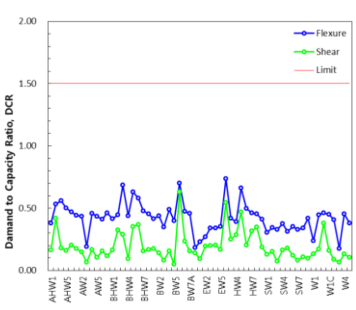


(a) Flexure

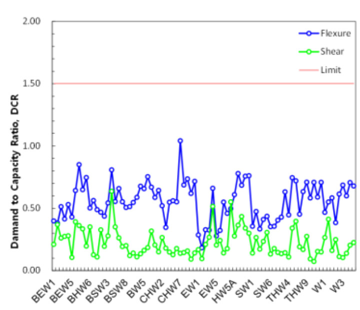


(b) Shear

Fig. 3. DCR of a Shear Wall (in left corner of plane in Bldg. A ; HW1)



(a) Building A



(b) Building B

Fig. 4. Maximum DCR of All Walls

5. MCE Level Evaluation

5.1 Ground Motion Records Selection

To enable precise analysis, seismic environment of the site in which the building will be built were considered and 7 pairs of the ground motion data corresponding to KBC2009 were selected. The depth of the weathered rock in the site is from 1.6 m to 28 m. The average shear wave velocity in the upper 30 m of the site measured from the borehole seismic test is 334 m/s, so the site is classified in S_d according to the standard.

The 7 pairs of ground motion records that are compatible with the earthquake magnitude and site-source distance found from deaggregation were selected. Keeping the frequency characteristic of the selected records similar to the originals, the acceleration response spectrums were corrected to match the MCE response spectrum. [7] The used program is OPTIME [8] and this program has an algorithm to adjust the time history to match the target spectrum using wavelet. After calculating the Square Root of the Sum of the Square (SRSS) of 5%-damped acceleration response spectrum for the two components of each record using OPTIME, the average spectrum of these were calculated and the results were compared with MCE response spectrum. The average SRSS spectrum was then adjusted to be larger than 1.3 times MCE spectrum over period range (0.2T to 1.5T, where T is the fundamental period) (Fig.5).

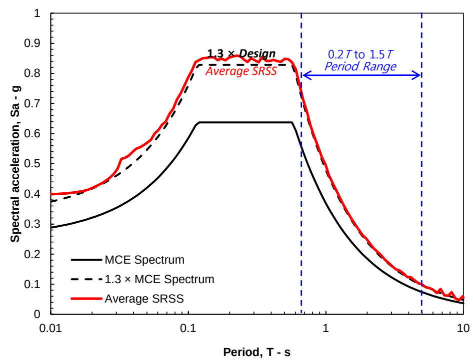
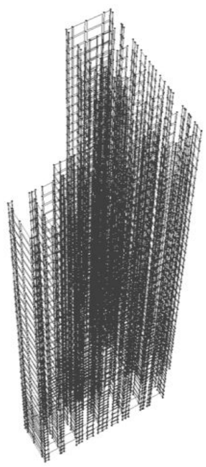


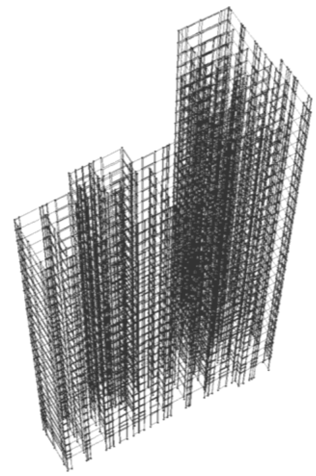
Fig. 5. Modification of Selected Ground Motion Records

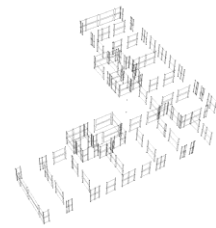
5.2 Nonlinear Model

To evaluate the response of the structures to 7 pairs of ground motions that represent MCE, nonlinear response history analyses were performed using Perform 3D (Fig.6). The models includes inelastic member properties for elements that are anticipated to be loaded beyond their elastic limit, which are the walls. The walls were modeled with fiber elements. Fiber elements of walls were defined from the stress-strain relationship of rebar and concrete considering expected strengths (Fig. 7). The tensile region of the concrete was ignored. Based on Korean *guideline*, stress-strain curve of concrete was plotted referring to the curve of unconfined concrete elements of modified Kent-park model and Mander model, and strain corresponding to maximum compressive strength was set to be 0.002. Elastic modulus of concrete was calculated according to KCI 2012. Also, by considering expected strength coefficient, tri-linear curve was used for material properties of rebar. Fracture strain was set to be 0.1 according to section "4.3.3" of Korean *guideline* and allowable strain was set to be 0.05 according to the ASCE/SEI 41-13[9]. For wall modeling, elastic material model was applied in the shear direction and 1/4 of in-plane stiffness was applied to the out-of-plane stiffness. The inelastic behavior of thin walls is usually concentrated on the very lower level from previous findings and the shorter plastic hinge length is recommended per several researches [4, 10, 11]. To evaluate these characteristics of slender walls more mesh dividing in vertical direction is adequate. So, plastic hinge area of walls was assigned to 1/2 of the story height at the lower floors. (Fig. 6)

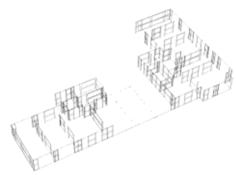
Mass was applied with lumped mass at the location of the calculated center of mass and was taken as 1.0DL. The effective stiffness summarized was Table 2 is applied. Generally, the gravity loads applied in nonlinear analysis differ from the linear response spectrum analysis for design. According to section“8.3.2” of TBI, the load combination of 1.0DL + 0.25LL was applied. Initial damping ratio of structure was set to 2.5% according to Korean *guideline*.





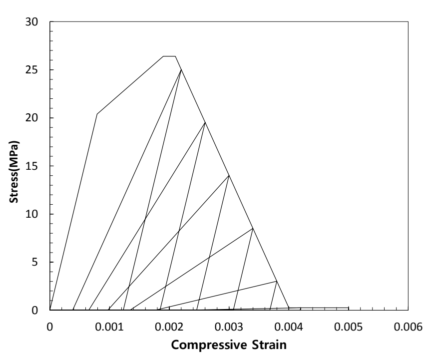


(c) 1^{st} Floor (Building A)

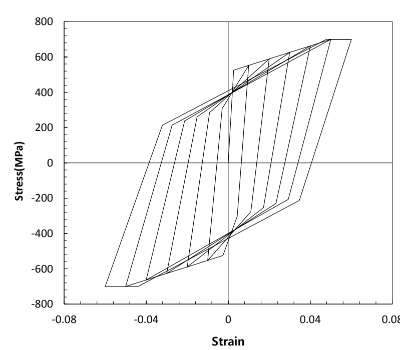


(a)Building A (b) Building B (d) 1^{st} Floor(Building B)

Fig.6 3-D Model



(a) Concrete(f_{ck}=24MPa)

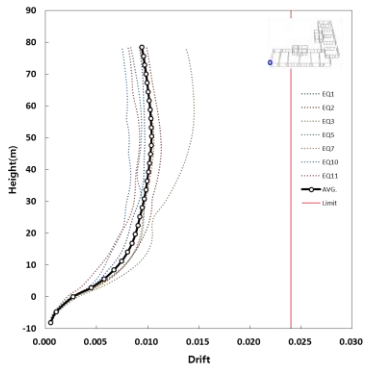


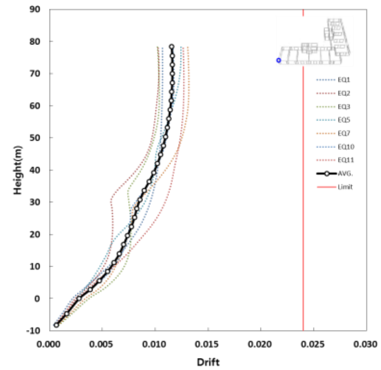
(b) Rebar(f_y=500MPa)

Fig. 7 Material Models

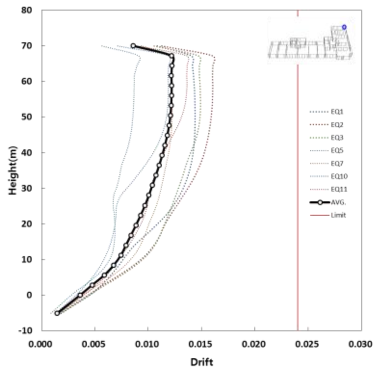
5.3 Evaluation Result

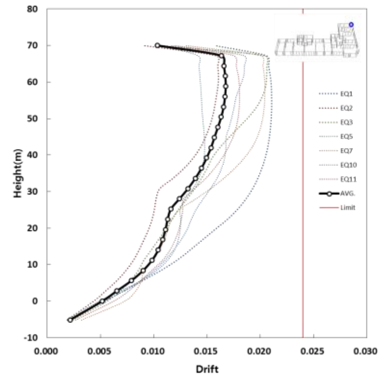
Acceptance criteria for the evaluation of the collapse-preventing performance level of the structure about MCE earthquake was followed by chapter 8.6 and 8.7 of TBI. Global acceptance criteria include the story drift and the limit is reduced to 80% (1/I=1/1.2) in order to ensure the equivalent performance as the one designed under the KBC 2009. Which means, the average value of the seven pairs of the maximum seismic response must not exceed 2.4% (= 3.0% × 0.8) and maximum seismic individual response must not exceed 3.6% (= 4.5% × 0.8). The drifts of the structure were reviewed in both flat corners. After reviewing each directional drift, Building A and B were confirmed that they meet the acceptance criteria (Fig. 8). For Building A, the maximum drift was 1.45% in X direction in the bottom left corner of 22nd floor and 1.32% in Y direction in the bottom left corner of 25th floor. For Building B, the maximum drift was 1.6% in X direction in the bottom left corner of 24th floor and 2.11% in Y direction in the bottom left corner of 20th floor.





(a)X-dir. Story Drift of Bldg. A at left corner (b) Y-dir. Story Drift of Bldg. A at left corner

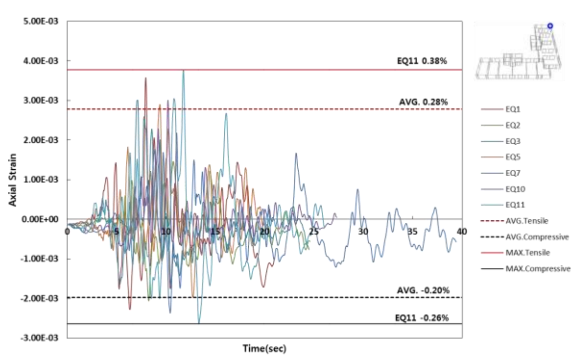


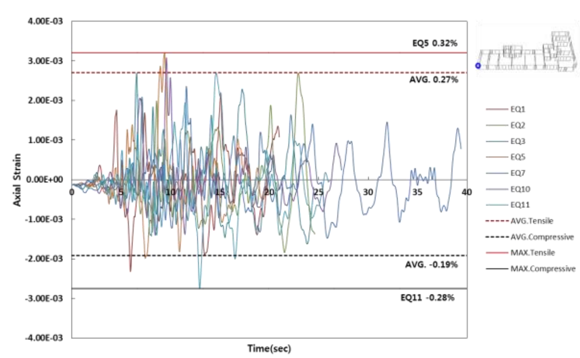


(c) X-dir. Story Drift of Bldg. B at right corner (d) Y-dir. Story Drift of Bldg. B at right corner

Fig. 8. Story Drift

Acceptance criteria for member level include review of the compression strain, and the wall member forces. Evaluation for the compressive strain at the outermost end of walls is a critical review. In this study, allowable compressive strain of concrete was set to 0.002 referring to 4.3.2 provision of Korean *guideline*. It was checked if the average compressive strain exceeds 0.002 and the maximum compressive strain from the individual ground motion exceeds 0.0025. As results, some walls in 1st floor do not meet the acceptance criteria for both Building A and B (Fig.9). Fig.1 showed the members that exceed the limit. Accordingly, extra retrofits are required for the wall elements.





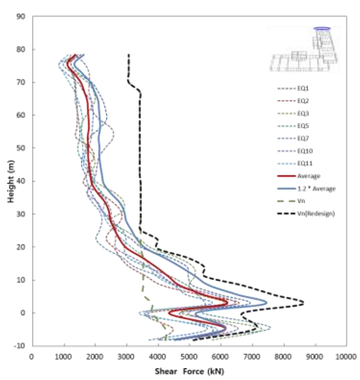
(a) Building A (b) Building B

Fig. 9. Wall Axial Strain History

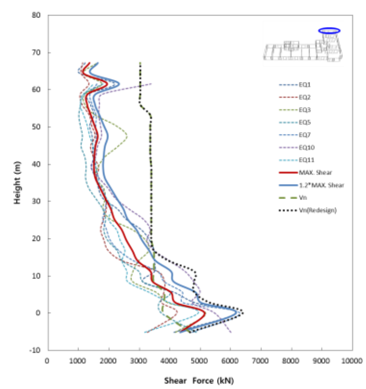
Finally, shear in the walls which is classified into force-controlled actions was evaluated using following acceptance criteria.

$$\lambda F_u \leq \Phi \ F_{n,e} \tag{3}$$

where, λ is 1.2 and F_u is the demand obtained from evaluation of nonlinear response history analyses and $F_{n,e}$ is nominal strength as computed from the material code but based on expected material properties and Φ is strength reduction factor obtained from material standards. As results, it was founded that shear force capacity in some walls was significantly insufficient about MCE load. And both buildings were assessed that additional horizontal rebar, and changes in concrete strength and wall thickness were required (Fig.10). Also, it was found that excessive shear forces were concentrated in some shear walls in both buildings. According to the architectural plan, Z-shaped wall that connects the three walls in the center of the plane is present in the Building A. Since the Z-shaped wall is connected with the entire surrounding outer walls, it has very large stiffness, and shear forces are expected to be concentrated around the wall. For Building B, wall is not continuous and transfers to the floor because of the opening. So the shear force in the above that floor is overestimated. Throughout the analyses, it was proved that the shear force distribution effect can be obtained by installing the seismic joint in the proper position of the shear walls (Fig 1).



(a) Building A



(b) Building B

Fig. 10. Wall Shear

6. Conclusion

This study examined the application of performance based design to tall residential buildings in Korea. The performance-based seismic design of buildings with typical ordinary load bearing shear walls in Korea was conducted per TBI. The structure was evaluated whether it satisfied its performance objectives to withstand Service Level and Maximum Considered Earthquake. The findings are as follows.

(1) The thickness of the walls in these buildings is 200 mm. The ratio of the total wall section to the floor plan is 3% in each direction of the buildings. These configurations are similar to the typical buildings in Chile which were severely damaged during the 2010 Chile Earthquake. Based on this observation, it is concluded that the acceptance criteria shall be more restrictive and conservative to assure the stability of the structure. Also, the consideration for global buckling and rebar buckling is recommended such as the limitation of the ratio of floor height to wall thickness and reduced strain limitation of reinforcement.

(2) To conduct the serviceability evaluation in performance-based seismic design, the return period of the service level earthquake is defined as of 72 years (50% probability exceedance in 50 years). The structure was evaluated to have more than 50% stability margin than the limitation under the Service Level Earthquake. As the building shows a sufficient structural performance under the Service Level Earthquake, additional case studies and discussions are needed to check whether the Service Level evaluation is the mandatory process for Korean tall residential building.

(3) To evaluate the response of structures under MCE, 7 - nonlinear response history analyses were performed. The results showed that some walls didn't satisfy the requirement for axial strain and shear. Therefore, the walls that exceed the limit of axial strain needed to have boundary reinforcement and the walls that have sufficient resistance for shear were needed to redesign the horizontal reinforcement, concrete strength, and wall thickness. In addition, as excessive shear force was concentrated in some walls in both buildings, seismic joints were installed in the proper position of the walls. As results, the shear force distribution effect can be obtained.

(4) The slender shear wall system in low to moderate seismicity region is not easy to apply the capacity design concept due to its limitation of wall thickness and congestion of reinforcement. So there is not safe guard to avoid brittle failure. Therefore, the design professional should understand the limitation of current performance-based design approach in limited ductile system and at the early phase of design more stringent acceptance criteria shall be established with local authority and peer review panel.

Acknowledgements

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