

Seismic Evaluation of RC Tall Shear Wall Building Using Nonlinear Dynamic Analysis

*Seung Yong Jeong¹⁾ and Thomas Kang²⁾

^{1), 2)} *Department of Architecture & Architectural Engineering, Seoul National University,
Seoul, Korea*

²⁾ tkang@snu.ac.kr

ABSTRACT

Recently, construction of tall buildings with reinforced concrete (RC) shear walls as a seismic-force-resisting system has been increased. According to ASCE 7-10, there is a strict height limitation of 49 m for special RC shear walls assigned to Seismic Design Category D or E. To overcome the limitation, seismic evaluation of designed tall buildings by nonlinear dynamic analysis is necessary to confirm their performance, which is also called as performance-based seismic design. In this study, a performance based seismic design is carried out in comparison with conventional analysis and design method that incorporates the response modification coefficient for special RC walls.

1. INTRODUCTION

As one of the lateral load resisting systems, shear wall system has been commonly used for residential buildings. Following the trend of construction of tall residential buildings, construction of tall RC shear wall building also increased. However, conventional design codes such as ASCE 7-10 (American Society of Civil Engineers 2010) do not specifically deal with the design of high-rise buildings. The design criteria using response spectrum analysis with the response modification coefficient, R , are mainly for low to mid-rise buildings. According to ASCE 7-10, there is a strict height limitation of 49 m for special RC shear walls assigned to the seismic design category D or E. However, the structural design should be possible that ensures satisfactory seismic performance, even if the height of a building exceeds the limitation, which is called performance based seismic design (PBSD). Because the PBSD is not based on prescriptive codes, the engineer should provide reliable bases. ASCE 41-13 (American Society of Civil Engineers 2014) is one of the commonly used guidelines for PBSD.

¹⁾ Graduate Student

²⁾ Associate Professor

Table 1 Properties of reinforcement

Material	Yield Strength (F_y)	Tensile Strength (F_u)	Modulus of Elasticity (E_s)	Expected Yield Strength (F_y)	Expected Tensile Strength (F_u)
Reinforcement	500 MPa	620 MPa	200,000 MPa	625 MPa	775 MPa

Table 2 Properties of concrete

Material	Compressive Strength (f_c')	Modulus of Elasticity (E_c)	Expected Strength (f_c')
Unconfined Concrete	27 MPa	24,422 MPa	40.5 MPa
Confined Concrete	34 MPa	24,422 MPa	47.4 MPa

2.3 Shear Wall Modeling

Shear walls are modeled as shell elements. In seismic design, cracking of concrete elements should be accounted for, so their effective stiffness should be used. Because shear walls of 1/6 to 1/8 of the total building height are generally assumed to be cracked, the stiffness of shear walls of 1st to 5th floors is reduced. The effective stiffness of $0.35I_g$ for cracked walls and $0.7I_g$ for uncracked walls suggested by ACI 318-14 are used for the linear analysis.

The fiber models for shear walls are used in the nonlinear analysis. Nonlinear behavior of the fiber model is determined by the stress-strain relationship of materials.

2.4 Coupling Beam Modeling

Coupling beams of 600 mm depth and 250 mm width are used at the core of the building. ACI 318-14 suggests effective stiffness of $0.35I_g$ for beams. However, this value is not appropriate for coupling beams according to the recent research of PEER/ATC 72-1 (PEER/ATC 2010), which suggests $0.15I_g$ for coupling beams. Thus, the value of $0.15I_g$ is adopted in this study.

2.5 Slab Modeling

Generally, some analysis techniques are used to simplify the analysis of slabs. First, a rigid diaphragm model excluding slabs from the lateral resisting system is used to transfer the lateral forces to the vertical resisting elements in direct proportion to the stiffness of vertical elements. When slabs are included in the lateral resisting system, the equivalent frame method or effective beam width method is used. These techniques can reduce the analysis time, but can lead to wrong estimation of the lateral stiffness of the building. Hence, membrane elements with out-of-plane bending stiffness, so called shell elements are used for slabs.

ACI 318-14 suggests the effective stiffness value of $0.25I_g$ for bending stiffness of slabs. Because ETABS 2015 does not provide nonlinear analysis of slab, so a conservative value of $0.2I_g$ is used for effective stiffness of slabs in both the linear and nonlinear analyses.

3. LINEAR RESPONSE SPECTRUM ANALYSIS (LRSA)

The LSRA is a design method confirming the safety by linear behavior of a structure against the response spectrum with consideration of response modification coefficient. The R factor reflects the ductility of the structure, and allows for the reduction of design earthquake load in LRSA. However, the R factor is determined based on the entire structural system, not each element and connection. ASCE 41-13 proposes the performance evaluation method for linear system using m-factor which takes into account the nonlinear behavior of each member. It can be directly compared with the result of nonlinear analysis by performance levels. In this study, this method is not used, as the paper focuses the comparison between conventional linear design and nonlinear dynamic design.

3.1 Design Response Spectrum

The design response spectrum is determined according to ASCE 7-10, as shown in Fig. 2. The used building system is the special shear wall system, the R factor of 5 is used, and the important factor of 1.25 is used. The input earthquake load is determined from the design spectrum that is multiplied by the important factor and divided by R factor.

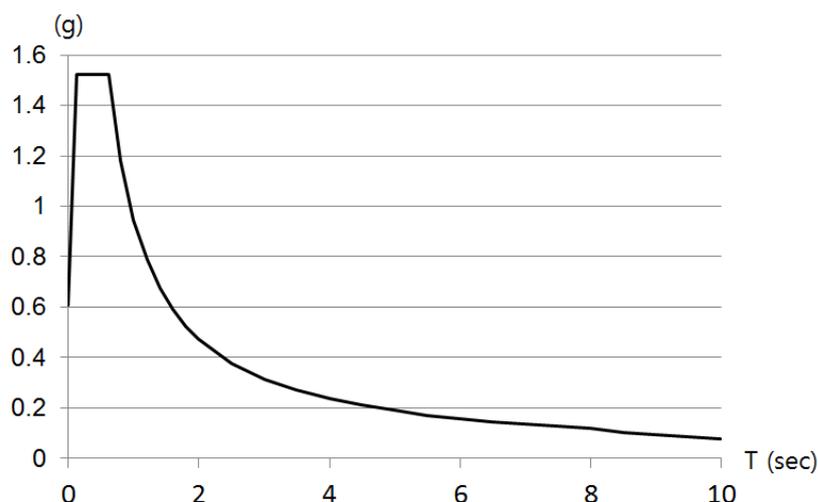


Fig. 2 Design response spectrum

3.2 Damping for LRSA

There is a decreasing tendency for the damping of high-rise buildings, and thus Tall Building Initiative (TBI) suggests 2~3% of damping ratio for high-rise buildings. The analysis model is not considered tall enough to match up to the high-rise building criteria of TBI; hence, the general value of 5% is used for viscous damping ratio.

3.3 Linear Static Analysis

Before LRSA, the linear static analysis is conducted to compare the base shear. If the base shear of LRSA is less than 85% of that of the linear static analysis, the modification factor should be applied. The base shear forces of the linear static analysis and LRSA, and the modification factors for X and Y directions are shown in Table 3.

Table 3 Base Shear and Modification Factor

	Linear Static (kN)	LRSA (kN)	Modification Factor
X-dir. Base Shear	9599.5	6611.7	1.23
Y-dir. Base Shear	9599.5	9097.3	Not Needed

3.4 Analysis Result

The purpose of this study is not to obtain the exact design, but for the comparison between the linear and nonlinear design. Thus, the safety check of whole members is not conducted. Demand capacity ratios (D/C ratio) for only five critical parts of shear walls with specified steel ratios are checked.

The D/C ratios from LRSA are calculated by using P-M diagrams in ETABS 2015 as shown in Fig. 3 (Computers & Structures, Inc. 2014). The point *L* indicates the demand of (P_u and M_u), and Point *C* denotes the point where the extension line of *OL* and P-M diagram meet. The D/C ratio is the length of *OL* divided by that of *OC*.

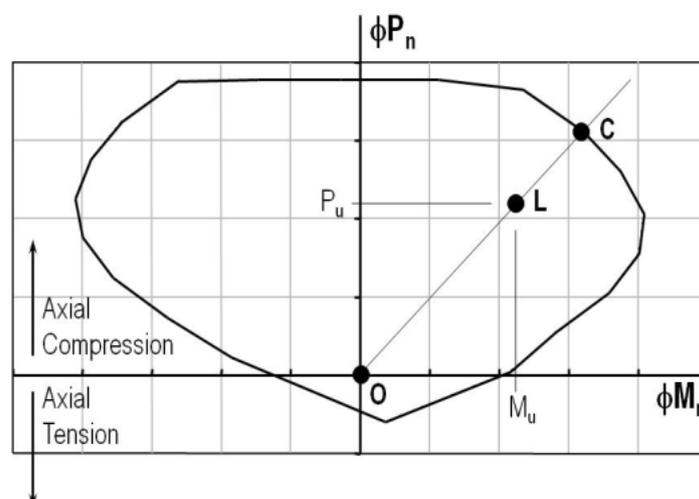


Fig. 3 D/C ratio calculation from P-M diagram

The cross sections of five critical shear walls are shown in Fig. 4. The exact confined wall area is the inner area between the layers of horizontal web reinforcement, but the area from surface to the horizontal reinforcement is assumed to be confined for convenience of the analysis. The D/C ratios are shown in Table 4 (see Fig.1 for shear wall numbers).

Table 4 D/C ratio of LRSA

Story	D/C Ratio				
	W33	W26	W40	W07	W39
1	1.421	1.307	1.282	1.193	1.095
2	1.153	1.077	1.012	0.973	0.958
3	0.948	0.938	0.837	0.826	0.855
4	0.786	0.792	0.684	0.71	0.753
5	0.67	0.664	0.553	0.643	0.658
6	0.596	0.59	0.474	0.582	0.587
7	0.537	0.529	0.441	0.528	0.51
8	0.493	0.481	0.411	0.481	0.443
9	0.46	0.446	0.383	0.44	0.403
10	0.435	0.42	0.355	0.406	0.367
11	0.414	0.399	0.329	0.375	0.335
12	0.395	0.381	0.304	0.348	0.306
13	0.375	0.366	0.281	0.322	0.279
14	0.356	0.365	0.263	0.297	0.254
15	0.343	0.359	0.246	0.274	0.23
16	0.326	0.348	0.228	0.258	0.206
17	0.305	0.333	0.207	0.24	0.184
18	0.28	0.313	0.186	0.221	0.162
19	0.254	0.29	0.164	0.207	0.145
20	0.228	0.266	0.142	0.196	0.133
21	0.202	0.241	0.122	0.184	0.121
22	0.178	0.215	0.104	0.171	0.119
23	0.153	0.19	0.09	0.159	0.12
24	0.126	0.164	0.078	0.146	0.12
25	0.104	0.145	0.053	0.131	0.113

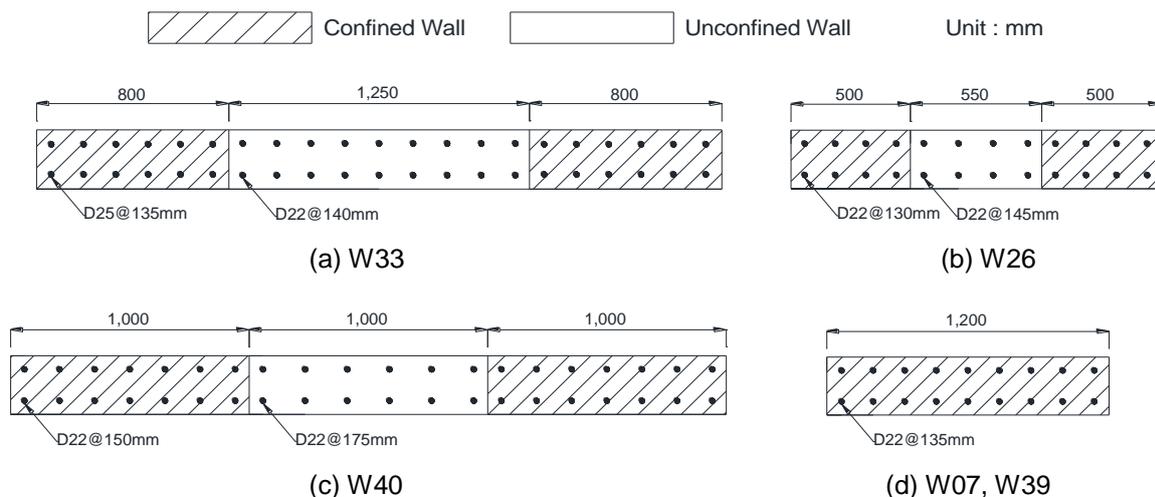


Fig. 4 Shear wall sections

4. NONLINEAR TIME HISTORY ANALYSIS (NTHA)

4.1 Nonlinear Material Model for Fiber Elements

For nonlinear time history analysis (NTHA), the same expected strengths in Table 1 and Table 2 are used. Mander's model implemented in ETABS 2015 is used for both unconfined and confined concrete. The stress-strain curves of concrete and steel are shown in Figs. 5 and 6, respectively.

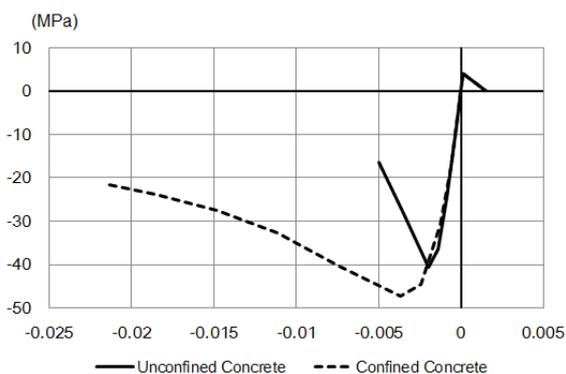


Fig. 5 Stress-strain curve of concrete

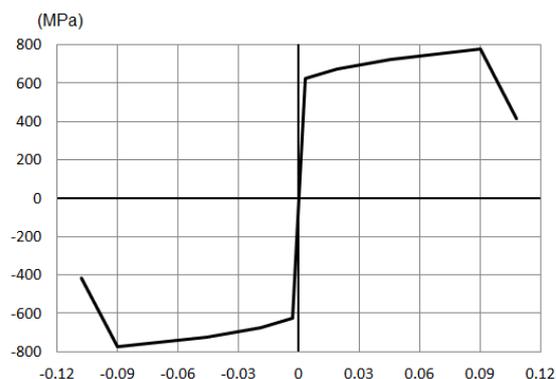


Fig. 6 Stress-strain curve of reinforcement

4.2 Nonlinear Behavior of Members

The nonlinear behavior of shear walls is generally governed by the stress-strain curves of materials, while the nonlinear behavior of coupling beams is affected mostly by plastic hinges at the each end of the beam. The properties of plastic hinges are

adopted from ASCE 41-13. For slabs, linear shell elements are used because ETABS 2015 does not offer nonlinear analysis of a slab.

4.3 Damping for NTHA

ASCE 41-13 suggests that target elastic effective damping ratio do not exceed 3% for NTHA. Thus, the target Rayleigh damping ratio of 2.5% is applied because NTHA also includes the damping caused by the energy dissipation from nonlinear hysteresis behavior.

4.4 Ground Motion

Three ground motion records are used as shown in shown in Table 5. Ground motions are scaled not to be less than the design spectrum for the period range of 0.2T to 1.5T. The 5% damping response spectrums are shown in Fig. 7.

Table 5 Ground motion record

Ground Motion Record	Magnitude	Time Step (sec)	Duration (sec)
Imperial Valley-02 (1940)	6.95	0.02	53.72
Kern County (1952)	7.36	0.02	54.35
Northridge-01 (1994)	6.69	0.02	60.00

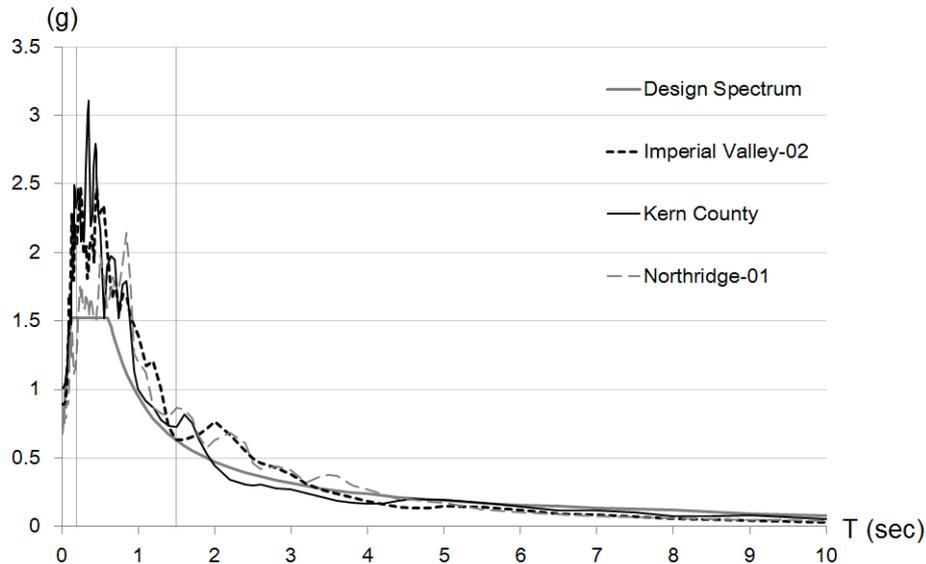


Fig. 7 Response spectrums of input ground motions

4.5 Seismic Performance Criteria

The target seismic performance is the life safety in ASCE 41-13, where the

seismic performance of a member is generally controlled using its plastic hinge rotation. For the shear walls modeled using fiber elements, the maximum compressive strain of concrete fibers as well as compressive and tensile strains of reinforcement fibers are used as performance criteria. The ASCE 41-13 suggests the maximum concrete strain limit of 0.002 for unconfined concrete, compression steel strain limit of 0.02, and tensile steel strain limit of 0.05. However, ASCE 41-13 does not provide a strain limit for confined concrete. In this study, Turkish seismic code (TSC-07) (Ministry of Public Works and Resettlement 2007) is adopted so that the conservative value of 0.0035 is used for the confined concrete strain limit.

3.7 Analysis Result

The D/C ratios from nonlinear analysis (strain values divided by strain limits) are summarized in Table 6. Because three ground motions are used, the maximum demands from the three ground motions are used to determine D/C ratios.

Table 6 D/C ratio of NTHA

Story	D/C Ratio				
	W33	W26	W40	W07	W39
1	1.022	0.88	0.603	0.555	0.413
2	0.867	0.783	0.499	0.416	0.378
3	0.721	0.687	0.44	0.338	0.344
4	0.593	0.588	0.42	0.3	0.316
5	0.503	0.496	0.4	0.262	0.291
6	0.425	0.417	0.378	0.225	0.267
7	0.376	0.356	0.351	0.191	0.242
8	0.372	0.313	0.318	0.164	0.215
9	0.371	0.3	0.283	0.142	0.185
10	0.37	0.295	0.252	0.132	0.155
11	0.358	0.295	0.248	0.127	0.128
12	0.355	0.282	0.235	0.127	0.12
13	0.334	0.281	0.218	0.125	0.111
14	0.303	0.26	0.197	0.122	0.099
15	0.265	0.232	0.173	0.119	0.087
16	0.222	0.199	0.146	0.105	0.077
17	0.187	0.175	0.118	0.099	0.069
18	0.16	0.151	0.097	0.091	0.059
19	0.128	0.124	0.079	0.085	0.048
20	0.099	0.097	0.059	0.079	0.039
21	0.078	0.073	0.046	0.073	0.039

22	0.073	0.058	0.044	0.066	0.038
23	0.064	0.045	0.042	0.059	0.038
24	0.047	0.031	0.03	0.051	0.037
25	0.02	0.019	0.012	0.034	0.025

Based on the NTHA, the most critical wall is W33, which is the same result as that from LRSA. The 2nd to 5th most critical walls from NTHA satisfied the life safety performance criteria, although the critical locations are different from LRSA. Indicated in Table 6 are the results of the walls of W33, W26, W40, W07, and W39, the most critical walls in the LRSA (not NTHA). These results are presented to be compared with LRSA.

4. CONCLUSIONS

Use of ASCE 7-10 is more suitable for low to mid-rise buildings, as it has a strict height limitation for special shear wall system. In this study, given the same shear wall conditions, the LRSA according to ASCE 7-10 is compared with the NTHA used for performance-based seismic design. Because the two different analysis results are difficult to be directly compared, D/C ratios for different criteria are compared.

The D/C ratios of the most five critical walls from LRSA exceeded 1, which means that the design may not be appropriate, whereas the D/C ratios of the same walls from NTHA did not exceed 1 except for W33 at the 1st floor. Although the evaluation methods are different, the results of NTHA almost satisfied the demand and give promising directions for the design of high-rise buildings. Also, it is indicated that the LRSA may lead to an overly conservative design and that the critical locations between the LRSA and NTHA can be different. The LRSA inherently includes many assumptions and may not reflect the actual nonlinear behavior under earthquakes. Therefore, it is confirmed that the strict height limitation in ASCE 7-10 can be overcome by incorporating the performance based seismic design using nonlinear dynamic analysis.

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