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Response Modification Factor and Deformability for Structural Walls Designed with Different Details

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ABSTRACT

This study investigates the seismic performance of bearing walls with rectangular sectional shape and specific details of reinforcements developed for 10 to 20-story apartment buildings in Korea. To investigate seismic behavior of structural walls, several specimens were experimented by author and laboratory test results by other researchers were collected and analysed. Structural behaviors of walls were evaluated by means of ductility, deformation, and strength capacities. For this purpose, thirty six specimens having different properties such as aspect ratios and details were considered. Based on the results of this study, deformability of the walls with specific details is discussed. Also this study compares the response modification factor(R) for the bearing wall systems in seismic design provisions between Korea and United States.

1. INTRODUCTION

Structural walls have been commonly used for the lateral forces resisting system against winds and earthquakes. Many low to mid-rise RC buildings have either interior or exterior walls. If the walls are designed to resist lateral and gravity forces, these walls are classified as bearing wall system. This system has been most commonly used for constructing mid-rise(10-15 stories) apartment buildings in Korea, which is classified as a low and moderate seismic zone according to the Korean Seismic Design Provisions(2000). Since this system is used for residence buildings, a rectangular sectional shape is preferred for providing better interior space. Also, to secure the seismic resistance of walls in mid-rise apartment buildings, special reinforcement details have been provided.

This study investigates the deformability of walls with a different cross-sectional shape, aspect ratio and reinforcement details, etc. For this purpose, test results for thirty six wall specimens were collected and analyzed. The deformability of these specimens is compared with the drift limit

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details in Korean residence building construction is discussed.

Also, this study compares the R factor of bearing wall systems in three different seismic design provisions such as UBC(1994), ATC 3-06(1978), and Korean Seismic Design Provisions(KSDP, 2000). KSDP has been developed based on UBC and ATC 3-06. Thus, in calculation of design base shear according to KSDP, R factor is included in the formula for calculating design base shear. The major role of R factor is to reduce the elastic design base shear whereby structures can behave in the inelastic range during design level earthquake ground motions(mean return period of 475 yr.). R factors are assigned according to material and structural systems. Based on the comparison of R factors in 3 different provisions and the investigation of deformability of the tested walls, the R factor for the walls with specific details is discussed.

2. COMPARISON OF SEISMIC DESIGN BASE SHEAR IN DIFFERENT PROVISIONS

The design base shear formula has been developed based on either a working stress or ultimate strength basis. For example, the design base shear in UBC(1994) is on a working stress basis, but both NEHRP Provisions(BSSC, 1994) and ATC 3-06(1978) have an design base shear for ultimate strength.

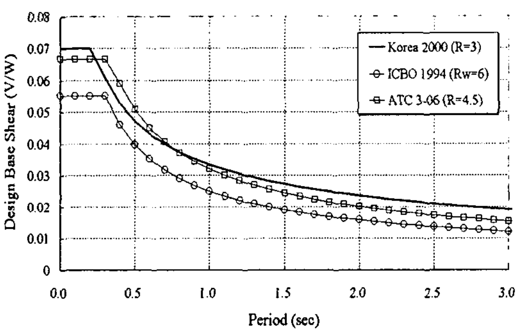

Fig. 1 Design Base Shears for Bearing Walls

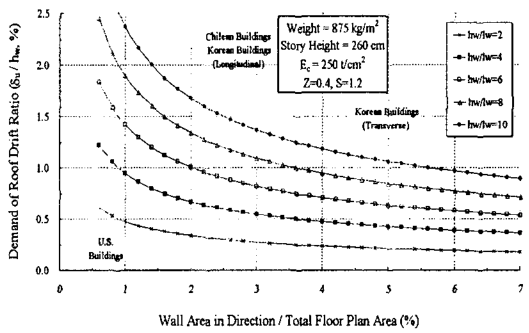

Fig. 2 Displacement demand for Bearing Walls

KSDP was established in 1988 and revised in 2000. The design base shear in this provision is working stress level. Table 1 shows the design base shear formulas in UBC, ATC 3-06, and KSDP. Assigned values for R factor in these provisions are also shown in Table 2.

Figure 1 shows the comparison of design base shears in ATC 3-06, UBC, and KSDP. The R factor in this plot is the value for bearing wall system with reinforced concrete shear walls. For this comparison, the zone factor, importance factor, and soil factor are set to be 0.12(A=0.12, Z=0.12, Aa=Av=0.12), 1.0, and 1.0, respectively. A zone factor of 0.12 is the assigned value for the Seoul area in Korea. According to this figure, design base shear in KSPD is larger than that in UBC(1994) throughout the whole period range. Also, the design base shear in KSDP exceeds that of ATC 3-06 when the fundamental period becomes either less than 0.2 second or larger than 0.7 second. By simply comparing design base shear for the bearing wall system, it is concluded that the design based shear used in KSDP is the highest. If it is assumed that the values for design base shear in ATC 3-06 and UBC are reasonable, the R factor in KSPD needs to be calibrated to reduce the design base shear. This study assumes that R factors provided in ATC 3-06 and UBC are resonable. Thus, R factor is calibrated to make the design base shear in KSDP similar to that in UBC(1994). However, in calibrating R factor, both structural details and structural performance are important since R factor is related to those.

Table 1 Comparison of Design Base Shear Formulas

Korea (2000)	UBC (1994)	ATC 3-06 (1978)
$V/W=\frac{AIC}{R}$ $C=\frac{S}{1.2\sqrt{T}}\angle 1.75$ A(zone factor) I (importance factor) S (soil factor)	$V/W=\frac{ZIC}{R_W}$ $C=\frac{1.25S}{T^{2/3}}\angle 2.75$ Z(zone factor) I (importance factor) S (soil factor)	$V=C_S W$ $C_S=\frac{1.2A_v S}{R\,T^{2/3}}<\frac{2.5A_a}{R}$ Av, Aa (zone factor) S(soil factor)
working stress design level	working stress design level	strength design level

Table 2 Comparison of Response Modification Factor for Structural Walls

Structural Systems	Lateral Force Resisting Systems	R (ATC,1978)	R_w (ICBO,1994)	R (Korea,1988)	R (Korea,2000)
Bearing Wall System	Reinforced Concrete Shear Walls	4.5	6	3	3
	Reinforced Masonry Shear Walls	3.5	6		
	Unreinforced Masonry Shear Walls, Partially reinforced Masonry Shear Walls	1.25	-		
	Reinforced Concrete Shear Walls having Boundary Elements like Tied Columns	-	-	3.5	
Frame System	Reinforced Concrete Shear Walls	5.5	8	-	4

The mean drift ratios for various aspect ratios of flexural walls can be plotted as a function of aspect ratio and wall area to floor plan area like as Fig. 2. The displacement demand is sensitive to the amount of wall area; the sensitivity of displacement demand to wall areas increases with lower amounts of wall areas to floor area, and drift is nearly independent of wall area for higher amounts of wall area to floor plan area. For U.S. buildings, the ratio of wall to floor area is typically on the order of 1%. Meanwhile, typical Chilean and Korean residence buildings rely almost exclusively on structural bearing walls for lateral load resistance where ratios of wall to floor area of 2-4 % are common, resulting in relatively stiff buildings. According to, Fig. 2, the displacement demand for typical U.S. construction exceeded 1% of drift ratio for all walls with aspect ratios greater than 2. And, the displacement demand for Chilean and Korean buildings was less than 1.2% of the drift ratio for walls with aspect ratios of up to 6. The maximum drift demand in a structural wall building subjected to severe earthquakes may range from 1~1.5% of drift ratio in Chilean and Korean buildings to 2% of drift ratio in U.S. buildings. Therefore, it is necessary that the structural walls should be designed to attain such level of deformability.

3. DIFFERENT PRACTICES IN STRUCTURAL WALL DETAILS

Requirements for the design of structural walls are introduced in chapter 11, chapter 14, and chapter 21 in ACI 318. The design code of Korean Concrete Institute(2000, referred to as KCI hereafter) has been basically developed based on ACI 318.

According to ACI 318-99, structural walls are classified as ordinary and special reinforcement concrete structural walls. Ordinary RC structural walls must satisfy the requirements from chapter 1 to 18 in ACI 318-99 and special RC structural walls must satisfy the requirements (boundary element or details) of chapter 21 in ACI 318-99 in addition to the requirements for ordinary RC structural walls.

Details of structural walls commonly used for bearing wall systems in Korea are quite different from those used in the Unites States. Figure 3 shows a wall details to apply in Korean construction practice for mid-rise residence buildings. The sectional shape is rectangular rather than barbell shape with boundary elements. A rectangular shape provides more useable interior space. Flexural reinforcement is concentrated at the wall boundary (the end region with 10% of wall length, *lw*) as shown in Figure 3.

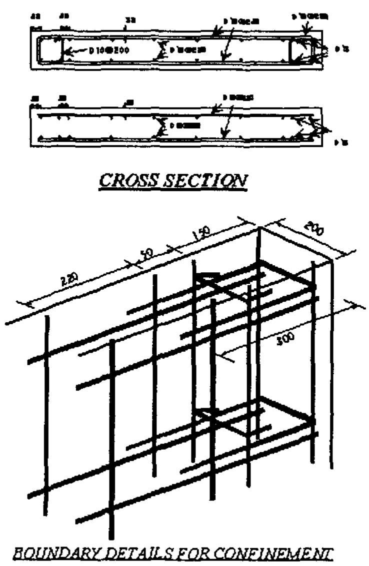

Fig. 3 Detail for Wall Boundaries

U-type transverse reinforcements and tie bars are placed. The spacing of U-type transverse reinforcements and tie bars is determined from the code requirement for column in KCI and ACI 318. Tie spacing in columns should not determine more than the minimum value among (1) 16 longitudinal bar diameters, (2) 48 tie diameters, and (3) least dimension of a column. In case of walls considered in this study, the minimum dimension requirement governs. U-type transverse reinforcements are extended into the wall web with the length of $20d_b$ (d_b: diameter of reinforcement). This is also determined based on the development length in KCI. The ends of ties are anchored by a 90 or 135 bend around a bar.

4. DEFORMABILITY IN THE PREVIOUS STUDIES ON STRUCTURAL WALLS

Experimental tests by PCA researchers(W.G. Corley, A.E. Fiorato and Oesterle, 1981) were carried out for walls having various section-shapes(rectangular, barbell, flanged) and different failure modes. Test results showed that all specimens have displacement ductilities larger than 3.0 and have drift ratios larger than 1.5 %.

Wallace and Moehle(1992) investigated the level of damaged buildings in the city of Vina del Mar due to Chile earthquake(M=7.8) occurring in March 1985. They reported that in the city of Vina del Mar there were about 400 modern reinforced concrete buildings, which contained numerous shear walls and had been designed for lateral forces comparable to those used in regions of high seismicity in the Unites States. Seismic design provisions in Chile do not require boundary element like in the Unites States. Also, reinforcement details, according to their paper, are less stringent than those commonly used in the Unites States. However, they reported that these walls performed well with little or no apparent damage in the majority of buildings during the earthquake.

Figures 4 and 5 show drift and ductility capacities vs. maximum observed shear stress of various walls tested by many researchers. Some information for each specimen in this figure is in Table 3. The test parameters of these structural walls were sectional shapes (rectangular, barbell, flange shape), details of reinforcement distribution, shear span ratio, existence of boundary element, ratio of axial load, etc.

It is considered that deformation and ductility capacities of walls depend on the level of maximum shear stress and/or failure mode because the level of maximum shear stress is related to the failure mode of structural walls. Figure 4 shows that all specimens have a drift capacity of over 1.5 percent except for one specimen governed by shear. A drift ratio of 1.5 percent is the allowable limit value against a design earthquake in seismic provisions (NEHRP Provisions, BSSC 1994). Thus, it is judged that most structural walls have satisfactory deformation capacities irrespective of the test variables.

When maximum shear stress is lower than 0.1 MPa, all specimens have a ductility capacity larger than 3.0 (see Figure 5). It is prescribed in the UBC-94 provisions that the R factor for a shear wall system is 8.0 (see Table 2). Expected maximum displacements according to the UBC-94 can be calculated by multiplying the design displacement by $3/8R_w$. This implicitly indicates that the displacement ductility capacity of a wall should be larger than 3.0. Thus drift capacity of 1.5% and displacement ductility ratio of 3 can be treated as the limit values of deformation and ductility capacities. According to Figures 4 and 5, most walls have satisfactory capacities in ductility and deformation.

In Figure 4, the scatterness of drift capacities of structural walls is large with respect to maximum shear stress. It is worthwhile noting that there is a relationship between maximum shear stress and drift capacity. Ductility capacity decreases as maximum shear stress increases. As maximum shear stress increases, structural walls become more likely to be shear-critical members. But, this is limited since it considers only isolated walls rather than an entire structural system.

Table 3 Specimens for evaluating the deformability

실험체	형상비	$\frac{P}{A_g f'_{ck}}$ (%)	변위 연성비	최대변위각(%)	가력 방법	파괴 모드	연구기관
R1	2.4	0.4	7.7	2.3	점증	휨	PCA
R2	2.4	0.4	6.2	2.9	점증	휨	PCA
R4	2.4	7.5	3.4	1.7	점증	휨	Northwestern
B1	2.4	0.3	7.4	2.9	점증	휨	PCA
B3	2.4	0.3	10.1	3.9	점증	휨	PCA
B4	2.4	0.3	15.6	6.9	단조	휨	PCA
F3	2.4	5.9	4.6	2.2	점증	휨	Northwestern
CI-1	2.88	1.0	3.5	2.3	변동	휨	PCA
US-J	2.78	4.9	5.8	1.5	점증	휨	PCA
W1	2.9	8.0	3.9	2.9	점증	휨	Michigan
RW2	3.13	7.0	3.7	2.2	점증	휨	Clarkson
W2-0	2	10.0	4.6	2.2	점증	휨	Hanyang
W2-20	2	10.0	6.5	2.7	점증	휨	Hanyang
W2-10	2	10.0	9.2	2.9	점증	휨	Hanyang
W3-20	3	10.0	8.7	2.8	점증	휨	Hanyang
R3	2.4	7.0	2.2	1.7	변동	전단	PCA
B2	2.4	0.3	4.1	2.3	점증	전단	PCA
B5	2.4	0.3	4.5	2.8	점증	전단	PCA
B6	2.4	14.1	2.4	1.7	점증	전단	PCA
B7	2.4	7.9	3.8	2.9	점증	전단	PCA
B8	2.4	9.3	4.2	2.9	점증	전단	PCA
B9	2.4	8.9	4.0	3.0	변동	전단	PCA
B10	2.4	8.6	4.3	2.8	변동	전단	PCA
B11	2.4	0.3	4.4	2.8	변동	전단	PCA
B12	2.4	0.4	3.5	2.2	변동	전단	PCA
F1	2.4	0.4	3.0	1.1	점증	전단	PCA
F2	2.4	7.6	4.8	2.2	점증	전단	PCA
SW1	1.28	7.9	6.0	3.5	변동	전단	Berkeley
SW2	1.28	7.6	2.9	1.7	점증	전단	Berkeley
SW3	1.28	7.8	8.7	5.7	단조	전단	Berkeley
SW4	1.28	7.5	3.6	2.3	점증	전단	Berkeley
SW5	1.26	7.3	5.0	2.4	단조	전단	Berkeley
SW6	1.26	7.0	4.4	2.3	점증	전단	Berkeley
W3	2.9	8.0	2.0	1.5	점증	전단	Michigan
RW3O	3.13	10.0	2.9	2.2	점증	전단	Clarkson
W1-20	1	10.0	3.9	2.0	점증	전단	Hanyang

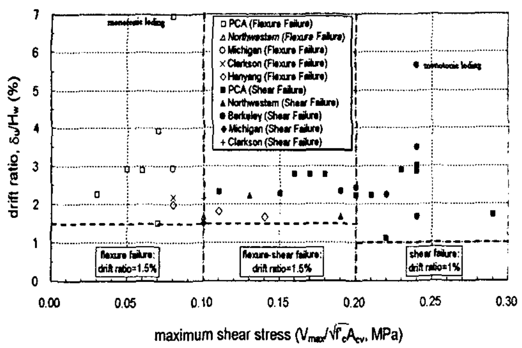

Fig. 4 Drift ratio vs. maximum shear stress

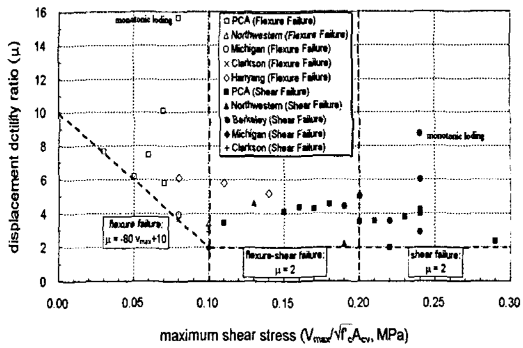

Fig. 5 Ductility vs. maximum shear stress

5. CONCLUSIONS

This study investigates the deformability and response modification factor of structural walls used in Korean residence buildings. Following conclusions are obtained from this study.

1) Most specimens have ductility and deformation capacities greater than 3.0 and 1.5% of height, respectively. Thus, the walls considered in this study have satisfactory deformation and ductility capacities.
2) The design base shear for bearing walls in KSDP is higher than that of ATC 3-06 in the period range shorter than 0.2 second and longer than 0.7 second. Also it is higher than UBC-94 in the whole range of period. It is noted that design base shear in Korean Seismic Design Provisions (KSDP) and UBC are working stress level whereas that in ATC 3-06 is strength level.
3) Since the elastic design base shear forces in UBC and KSDP are almost identical, it is concluded that KSDP assigned lower value of R factor for bearing wall systems, which causes higher value of design base shear. Considering the deformability of the test walls, it is conservative to assign a lower value of the R factor in KSDP. If it is assumed that the value assigned for R factor in UBC is appropriate, the R factor used in KSDP needs to be calibrated.

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