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BEHAVIOR OF RC WALLS WITH DIFFERENT STEEL GRADES AND ASPECT RATIO OF 2.0

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ABSTRACT

In the construction of nuclear power plants using massive walls, the use of high-strength reinforcing bars for shear design is necessary, to enhance the constructability and economy. In this study, walls (aspect ratio =2.0) with different steel grades were tested, to investigate the shear capacity and deformation capacity under cyclic loading. The specimens failed in mixed mode of shear and flexure, regardless of the grade of shear reinforcement. The lateral drift ductility was 2.27 to 2.77. Yielding occurred in all specimens despite different yield strains used in each test specimen. The average diagonal crack width tended to increase as the yield strength of web shear bars used in walls increases.

1. INTRODUCTION

In the construction of nuclear power plants, a number of large diameter reinforcing bars are used in massive reinforced concrete (RC) walls, which significantly affects the constructability and economy. Thus, to enhance the constructability and economy, the use of high strength 550 MPa bars needs to be considered. However, in ACI 349, the nuclear power plant related, the use of maximum yield strength of shear reinforcing bars is limited to Grade 420 MPa. Generally, the yield strength of shear bars is limited to ensure 1) to induce yielding of shear reinforcement by limiting the yield, and 2) to control the crack width of potential diagonal shear cracking. In particular, for the use of 550 MPa bars in nuclear power plants walls, which are large portion of nuclear power plant structures, a series of shear wall testing using 550 MPa shear reinforcement is necessary to investigate shear capacity and ductility capacity under cyclic loading.

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In the present study, cyclic lateral loading tests were performed for walls (aspect ratio=2.0) with different grades of bars and high shear reinforcement ratios, to investigate the behavior of heavily reinforced concrete walls after flexural yielding. The major test parameter was the grade of the shear reinforcement (i.e. web horizontal and vertical bar). The test results were compared with the failure mode, ductility, strains in bars, and average diagonal crack widths. On the basis of the results, the effect of Grade 550 MPa bars on the structural performance of walls (aspect ratio=2.0) was discussed.

2. EXPERIMENTAL PROGRAM

2.1 Maximum shear reinforcement ratio

In nuclear power plant walls, the shear reinforcement ratio is close to the permissible maximum shear reinforcement ratio specified by current design codes. Thus, the specimens tested in the present study were designed with the permissible maximum shear reinforcement ratio, focusing on the behavior of walls with high shear reinforcement ratio. In ACI 349-13 (ACI 349 Committee 2014), the shear strength of walls is defined as the sum of the contributions of concrete V_c, and shear reinforcement V_s:

$$V_n = V_c + V_s \leq 5/6\sqrt{f_c^{'}}hd \quad (1)$$

$$V_c = 1/6\sqrt{f_c^{'}}hd \quad (2)$$

$$V_s = A_v f_{yh} d / s \quad (3)$$

where $f_c^{'}$ = cylinder strength of concrete, MPa, h= thickness of wall, d= distance from the extreme compression fiber to the centroid of longitudinal tension reinforcement (=$0.8l_w$ in ACI 349), and A_v= area of shear reinforcement within spacing s.
Thus, the permissible maximum shear strength of web horizontal bars is defined as follows.

$$V_{smax} = \frac{5}{6}\sqrt{f_c^{'}}hd - V_c = \frac{2}{3}\sqrt{f_c^{'}}hd \quad (4)$$

For different yield strength of reinforcing steel f_{yh}, shear reinforcement A_v and spacing s can be determined corresponding to permissible maximum shear bar ratio by equating Eq. (3) and (4).

2.2 Major test parameters and specimens

Major test parameters were the grade of web shear reinforcement (i.e. horizontal and vertical bars) in walls. Four different types of shear reinforcement were placed in test specimens (Fig. 1(a) and (b)): 1) Grade 420 MPa, the yield strength of conventional reinforcement steel (Specimen NF2), 2) Grade 550 MPa, the target strength in this study (Specimen HF2), 3) Grade 500 MPa (Specimen HF2 500), and 4) Grade 600 (Specimen HF2 600). In Table 1, the actual strength of shear reinforcement in each specimen was presented. In all specimens, Grade 550 MPa bars were used at wall edges for flexural bars (Table 1). The flexural strength predicted by sectional analysis by ACI 349 V_f was designed to be equal to the shear strength prediction V_n, so that flexural yielding occurs before shear failure.

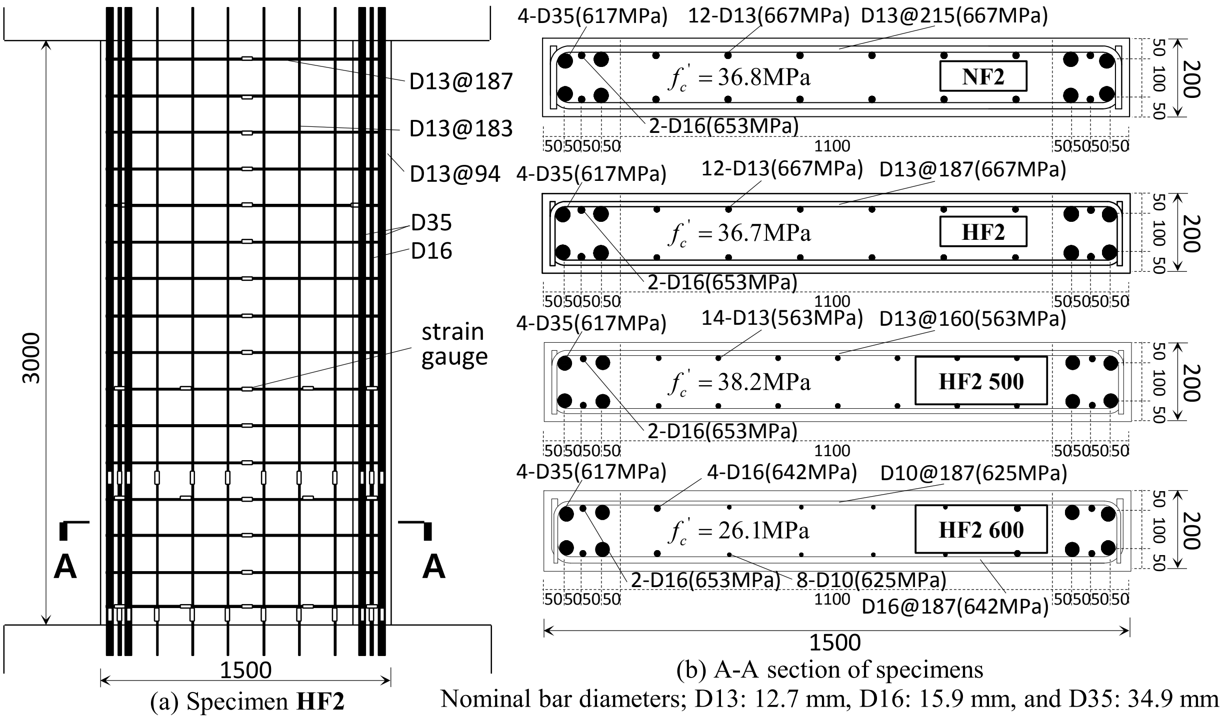


Fig. 1 Dimensions and reinforcement details of specimens

Table. 1 Design parameters of test specimens

Specimens	Design failuremode	Concrete strength f_c' MPa	Web region						Boundary region			expected $\frac{V_f}{V_n}$
			Horizontal bar				Vertical bar		Horizontal boundary hoop spacing	Vertical bar		
			$\frac{V_s}{V_{s\,max}}$	f_{yh} MPa	ρ_h (%)	$\rho_h f_{yh}$ MPa	f_{yv} MPa	ρ_v (%)		f_{yf} MPa	ρ_f (%)	
NF2	flexure	38.1	1.05	470	0.99	4.67	470	1.10	-	617	9.57	0.96
HF2		36.7	1.12	667	0.68	4.50	667	0.70	-	617	9.57	1.05
HF2 500		38.2	1.08	563	0.80	4.51	563	0.81	-	617	9.57	1.01
HF2 600		26.1	1.35	637	0.72	4.58	637	0.62	-	617	9.57	1.02

2.3 Test Procedure and Instrumentation

Axial compressive loading and lateral cyclic loading were applied using the test set-up shown in Fig. 2. An axial load of approximately $0.07A_c f_c'$ (800 kN for 38.1 MPa

concrete) was applied at the top of the wall by two displacement-controlled actuators. The level of the axial compressive force was maintained during cyclic lateral loading, by manually controlling the vertical displacement.
The lateral loading protocol followed the "Acceptance Criteria for Special Structural Walls" (Hawkins et al. 2003). Fig. 2 shows the LVDTs (Linear Variable Differential Transformer s) for the measurement of lateral displacements, sliding at the base, and shear deformations and the location of the strain gauges, to measure the strains of the flexural bars, web vertical bars, and horizontal bars.

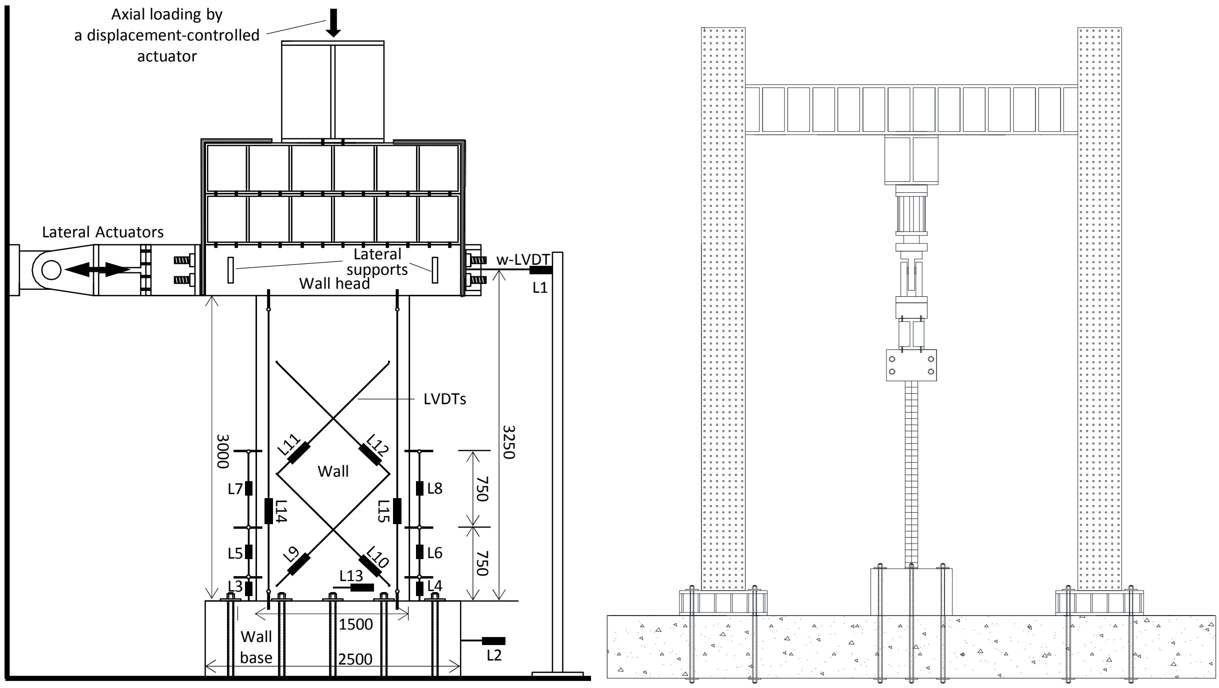
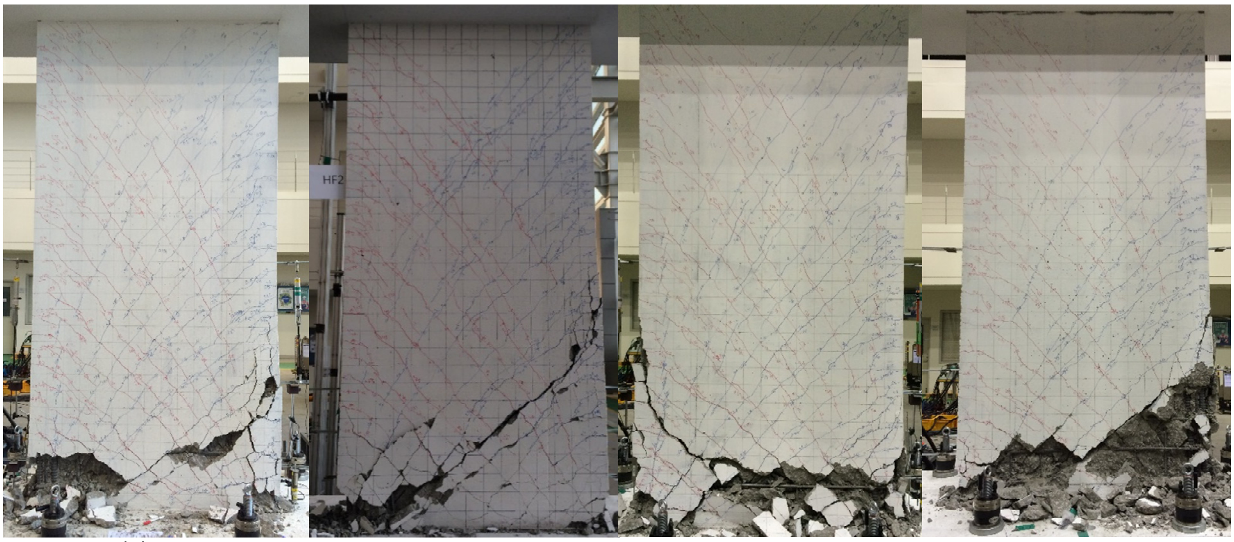


Fig. 2 Test set-up

3. TEST RESULTS

3.1 Failure modes

The specimens failed in mixed mode of shear and flexure: crushing in the compression end and diagonal tension cracking of the wall bottom, regardless of the grade of shear reinforcement (Fig. 3). In NF2 and HF2 with Grade 420 and 550, respectively, fracture of horizontal web bars occurred. On the other hand, in HF2 500 and HF2 600, bar-fraction did not occur. The results indicate that the bar-fraction is related to the bar material problem rather than the bar grade.



(a) **NF2** (b) **HF2** (c) **HF2 500** (d) **HF2 600**

Fig. 3 Failure modes of specimens

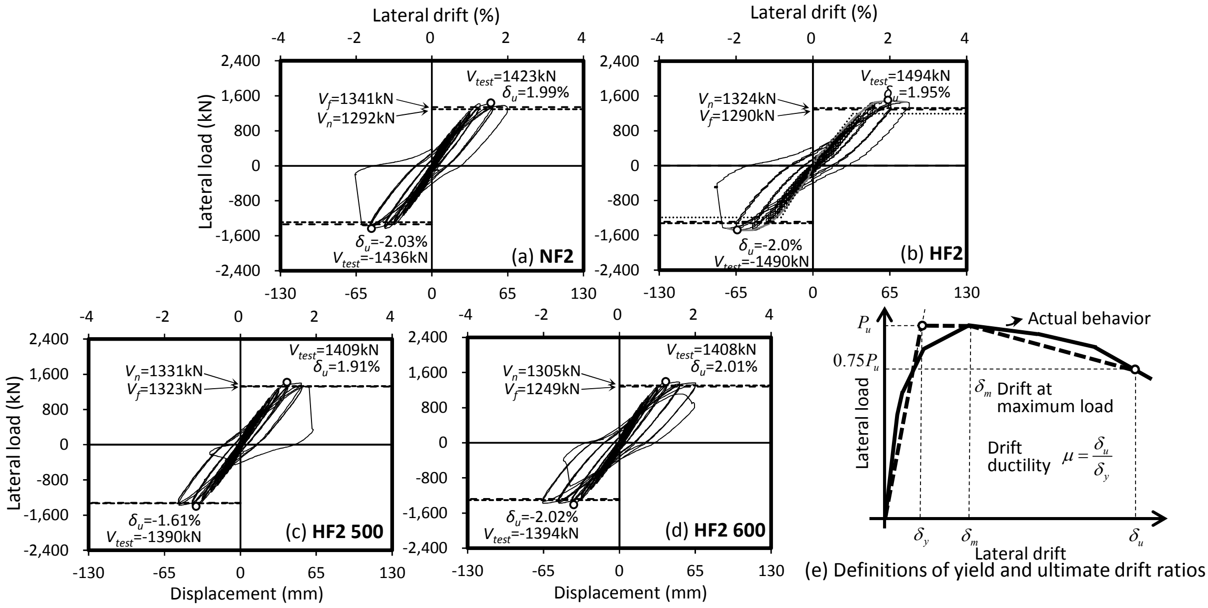


Fig. 4 Lateral load-displacement relationships of specimens

3.2 Lateral load-displacement relationships

In NF2 (Fig. 4(a)) with Grade 420 MPa web bars (f_{yh} =470 MPa, ρ_h =0.99 %, ρ_v=1.10 %), the peak strength was V_{test} =+1423, and -1436 kN at the drift ratios of +1.6, and -1.6 %, respectively. The lateral yield drift Δ_y, ultimate drift Δ_u, and ductility Δ_u / Δ_y were summarized in Table 2. The yield drift ratio Δ_y were +0.82, and -0.83 %, respectively. The ultimate drift ratio Δ_u were +2.0, and -2.0 %, respectively. The definitions of the yield drift Δ_y and the ultimate drift Δ_u are shown in Fig. 4 (e). The yield drift Δ_y was defined using the secant stiffness corresponding to 75% of the peak strength. The ultimate drift Δ_u was defined as the post-peak drift corresponding to 75% of the peak strength (Park 1988). The lateral drift ductility was estimated as 2.42 (Δ_u / Δ_y =2.0/0.82) in the positive direction, and 2.43 (Δ_u / Δ_y =2.0/0.83) in the negative direction. In HF2 (Fig. 4(b)) using Grade 550 MPa (f_{yh} =667 MPa, ρ_h =0.68 %, ρ_v =0.70 %), the load-displacement relationships were similar to those of NF2. The peak strength was V_{test} =+1494, and -1490 kN, which was close to those of NF2. The yield drift (Δ_u = +1.02 and -0.81 %), and ultimate drift ratio (Δ_y =+2.45, and -2.23 %) were also close to those of NF2. The lateral drift ductility was estimated as 2.4 (Δ_u / Δ_y =2.45/1.02) in the positive direction, and 2.75 (Δ_u / Δ_y =2.23/0.81) in the negative direction. Specimens HF2 500 and HF2 600 (Fig. 4(c) and (d)), using 500 MPa (72.5 ksi) (f_{yh} =563 MPa, ρ_h =0.80 %, ρ_v =0.81 %) and 600 MPa (f_{yh}=637 MPa, ρ_h =0.72 %, ρ_v =0.62 %) bars, respectively, showed the same behavior as that of HF2 with Grade 550 MPa web bars. In all specimens, the test strength V_{test} was greater than the flexural strength predictions of ACI 349 (V_{test} / V_f =1.01 to 1.15).

Table. 2 Ductility of specimens

Specimen	V_f kN	Test results				Ratios	
		V_{test} kN	Failure mode	Δ_y	Δ_u	Δ_u/Δ_y	V_{test}/V_f
NF2	1341	1429	CC + DT	26.6	64.7	2.42	1.06
HF2	1290	1492		36.4	82.9	2.27	1.15
HF2 500	1323	1400		26.7	62.3	2.33	1.06
HF2 600	1277	1405		25.8	65.3	2.53	1.10

3.3 Strains of Reinforcing Bars

In Fig. 5, strains of horizontal bars at the center of wall were greater than those at the upper and lower part of wall. At maximum load V_{test}, yielding occurred in all specimens despite different yield strains of each specimen.

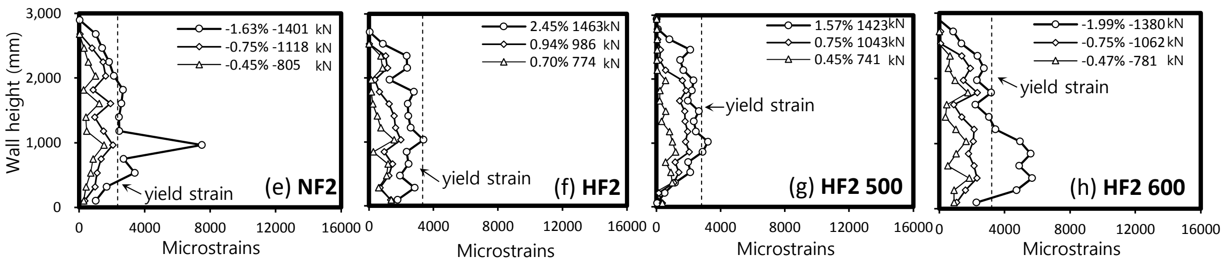


Fig. 5 Measured strains of horizontal bars in specimens

3.4 Average Diagonal Crack Width

In Fig. 6, the average diagonal crack width at the higher wall panel was shown with respect to the magnitude of lateral load. The average diagonal crack width was estimated using the length change measured by the diagonal LVDTs at the higher part of the walls (L11 to L12, see Fig. 2). The average diagonal crack width was calculated by dividing the maximum length change by the number of diagonal cracks during each cycle. At initial load, the average diagonal crack widths were closed each other. However, at maximum load, in HF2 and HF2 600, with comparably higher yield strength of web shear bars (667 and 637 MPa, respectively), the average diagonal crack width were greater than those of NF2 and HF2 500 with the yield strength of 470 and 563 MPa.

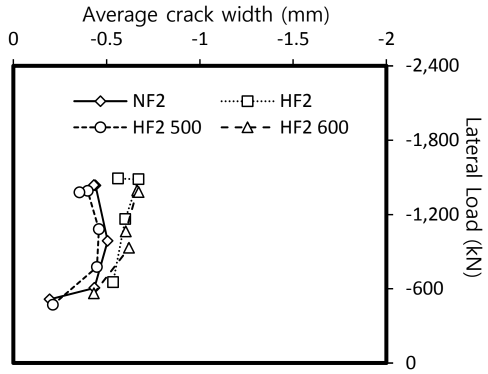


Fig. 6 Average diagonal crack width at each load increment

4. CONCLUSIONS

To investigate the validity of Grade 550 MPa bars for shear reinforcement of walls, four wall specimens with aspect ratio h_w/l_w=2.0 were tested under cyclic lateral loading. The test results of the walls with Grade 550 MPa web bars were directly compared with those of walls with Grade 420, 500, and 600 MPa web bars. Specimens NF2, HF2, HF2 500, and HF2 600 were tested to investigate the shear strength and ductility after flexural yielding. The major findings of the present study are summarized as follows.

1. The failure mode of NF2 using 420 MPa was the same as those of HF2, HF2 500, and HF2 600 using higher strength bars.
2. Bar-fracture occurred in several walls using 550MPa bars. However, bar-fracture also occurred in the wall with 420MPa bars. Further, bar-fracture did not occur in the walls with 500 MPa bars and 600 MPa bars. This result indicates that the bar-fracture is not related to the steel grade, but to the material property and/or the failure mode.
3. In all specimens, the test strength V_{test} was greater than the flexural strength predictions of ACI 349 (V_{test} / V_f =1.06 to 1.15). The lateral displacement ductility of the walls with different yield strength of shear web bars showed narrow difference: 2.27 to 2.53.
4. In all specimens, yielding occurred in the horizontal bars, which indicates that the horizontal bars including 550MPa bars developed their yield strength.
5. At initial load, the average diagonal crack width were similar despite the different grade of shear bars used in the test walls. However, at maximum load, the average diagonal crack width increased as the yield strength of shear bars used in walls became greater.

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REFERENCES

ACI Committee 349 (2014), "*Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349M-13) and Commentary*," *Farmington Hills*, p.196.

Hawkins, N.M., and Ghosh, S.K. (2003), "Acceptance Criteria for Special Structural Walls Based on Validation Testing," *PCI Journal*, **49**(5), 78-92.

Park, R. (1988) "Ductility evaluation from laboratory and analytical testing," in *Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan*, Vol. 8, 605-616.