Nonlinear Seismic Assessment of Irregular Coupled Wall Systems Using High Performance Fiber-Reinforced Cement Composites

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Abstract

Reinforced concrete (RC) coupled wall systems that consist of multiple shear walls linked by coupling beams are known to be very effective for resisting lateral loads in high-rise buildings. The response of irregular tall coupled wall systems is sophisticated to understand when subjected to seismic ground motions. Thus, nonlinear history analysis is necessary for accurate seismic performance assessment. With regard to improving the capacity of coupled wall systems, high-performance fiber reinforced cement composites (HPFRCCs) are recently being considered. These materials are characterized by a strain-hardening behavior in tension that can improve ductility and toughness of structures subjected to reversed cyclic loading. In this study, nonlinear finite element (FE) analyses for coupled wall specimens and irregular tall buildings with such systems are conducted using PERFORM-3D software in order to predict the nonlinear behavior of this system including HPFRCCs more accurately. In the FE models, the coupling beams are modeled using Moment Hinge Elements and the structural walls are modeled using Fiber Elements. In case of the HPFRCCs beam, Moment Hinge Elements have the increased strength of 20% and 2 times of ductility compared with the normal concrete beams. The strain-hardening behavior of HPFRCCs material model for Fiber Elements is represented such as a confined concrete material model. From the comparisons between modeling and test results, the proposed modeling methods well represent the overall behavior of the reviewed coupled wall specimens such as the strength, stiffness, and energy dissipation. Also, the nonlinear FE models well demonstrate potential improvements in the seismic performance of the considered specimens resulted from the use of HPFRCCs. Subsequently, the nonlinear analysis was performed to observe the behavior of a 56-story tall building with coupled wall systems depending on the used materials. The system responses are discussed through the comparisons of the results including interstory drift, base shear, overturning moment, and failure process. In spite of the use of HPFRCCs in all coupling beams and structural walls in 1/4 height from the base, an unacceptable system performance such as the shifted plastic hinges is simulated in the tall building model, and the reason can be assumed due to a large differences of wall capacities resulted from the use of HPFRCCs. Therefore, when designing a coupled wall system using HPFRCCs, it is recommended that a proper ultimate shear stress at any horizontal section shall be considered to prevent these results.

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Nomenclature

A_c	area of concrete section
A_{cw}	area of concrete section of coupling beam resisting shear
A_g	gross area
A_{sd}	area of each group of the diagonal bars
d	distance from extreme compression fiber to centroid of longitudinal tension reinforcements
d'	length of the clear cover
E_c	secant modulus of elasticity of concrete
f'_c	specified compressive strength of concrete
f_y	specified yield strength of reinforcement
f_{yt}	specified yield strength of transverse reinforcement
h	height of the beam cross section;
h_w	height of wall
I_{cr}	cracked moment of inertia of the cross section
I_g	moment of inertia of gross concrete section
l_p	plastic hinge length
l_w	length of wall
M_n	nominal flexural strength at section
M_o	overturning moment
P	axial force
t_w	thickness of wall
Z	slope of the descending branch
α	angle with the horizontal axis of the beam;
ϵ_0	strain at the f'_c
ϵ_c	strain in the concrete
ρ_l	ratio of area of distributed longitudinal reinforcement to gross concrete area
ρ_t	ratio of area of distributed transverse reinforcement to gross concrete area
ν	poisson ratio

Chapter 1. Introduction

1.1 Background and Motivation

Tall buildings are expected to contribute greatly to creation of eco-friendly high-tech space by concentrating land use. Besides, because the tall buildings have the meaning of future-oriented landmark, the demand for these buildings is gradually increasing. Reinforced concrete (RC) coupled wall systems are known as the effective structural system resisting lateral loads like earthquakes for the tall buildings. These structural systems generally consist of two shear walls linked by coupling beams. Especially, properly designed coupled wall systems contribute greatly to dissipate a substantial amount of energy with inelastic behavior. For this reason, many extensive researches about the coupling beams and shear walls are continuously conducted.

Because shear forces are transferred between coupling beams and shear walls, the coupling beams are required to withstand the high shear forces. Various experiments have been also conducted to analyze the behavior of coupling beams depending on the aspect ratio and the arrangement of reinforcement (Paulay 1971, Paulay and Binney 1974, Barney et al. 1980, Tassios et al. 1996, Xiao et al. 1999, Galano and Vignoli 2000, Kwan and Zhao 2002, Fortney 2005). These studies usually had the low aspect ratios (the short beam clear span to the beam total depth) which are differ from the coupling beams in the tall buildings. Recently, Naish et al. (2009) experimented the larger aspect-ratio coupling beams satisfied with the 2008 ACI Code requirement about simplified reinforcement details.

The continuous researches about development and applicability of construction materials have been also conducted to get the better durability and structural performance. With regard to the coupled wall systems, high-performance fiber reinforced cement composites (HPFRCCs) are recently being assessed as the material with high applicability. In the early years of 1918, the adding of fiber to improve the weak tensile behavior of concrete was beginning of fiber reinforced concrete (FRC). Based on this, the HPFRCCs were appeared through a various researches and this material has a multiple crack characteristic and a strain-hardening characteristic (Naaman and Reinhardt, 1996). Over the past decade, the experimental works about building component including HPFRCCs are proceeding. The HPFRCCs beam and wall components showed good performance under the reversed cyclic displacement loading despite the reductions of reinforcement detailing (Canbolat et al. 2005, Parra-Montesinos et al. 2006). Lequesne (2011) showed that the coupled wall systems with HPFRCCs exhibited a ductile flexural behavior in the walls and the improved damage tolerance in beams. From the previous studies, the use of HPFRCCs can reduce the requirement of reinforcements and it can

simplify the construction work. Because few experimental researches about the coupled wall systems including HPFRCCs are conducted, the reliable studies about the response of the structure scale according to the materials are required.

Performance based design is known as the most used method to design of the tall buildings with coupled wall systems in these days. The important goal of this design is to evaluate the behavior of structures at different seismic risk levels. The nonlinear analysis must be considered to select the acceptable behavior of structure for use of the design method. The structure systems basically have two type of nonlinearity as follows. First one is a material nonlinearity which is usually caused by inelastic behavior and it is the material property such as yielding, cracking, crushing, fracture, and so on. The geometric nonlinearity usually caused by change in shape of the structure and it includes P- Δ effects and true large displacement effects. Although the experimental work for nonlinear analysis is most accurate way to measure these nonlinear behavior of the structures, it needs to a lot of times and efforts. Therefore, modeling method for nonlinear analysis is increasingly demanded. The nonlinear analysis is not for the exact prediction of the structural behavior, but it is to get the useful information about the structures. For the reliable and economical modeling analysis of the reinforced concrete structure, proper models of materials or geometry of the structural systems are essential.

1.2 Objectivities

In this study, PERFORM-3D developed by Professor Powell will be used for nonlinear seismic analysis of a tall building with the coupled wall systems as modeling software.

It was the purpose of this study to:

- Review the macroscopic modeling methods for effective nonlinear analysis.
- By the application of effective modeling methods to some specimens, assesse the behavior of the structure system model.
- Compare between modeling results and test results, and discuss the reliability of the modeling and a better method.
- Assesse and compare the performance of structures depending on the materials in coupling beams and the hinge of walls; normal concrete and HPFRCCs.
- Apply the reliable modeling method to tall building in structural scale, and investigate the behavior of an actual core wall system composed of the coupled wall systems as the different properties of coupling beams and lower sections of the walls through the nonlinear analysis.

Chapter 2. Literature Review of Coupled Shear Walls

2.1 Behavior of Coupled Shear Walls

When the shear walls are connected with the beams, a coupling effect increases the stiffness and strength of the shear wall system under the lateral loads as shown in Figure 1. This provides additional resistance to overturning moment M_o , which is calculated by Eq. (1) where M_1 is the moment from the left wall, and M_2 is the moment from the right wall. This coupling action by transferring shear between the walls through the coupling beams is represented with the TL and this effect reduces the required flexural stiffness and strength of the individual walls. The axial force T can be calculated with the sum of shear forces at the each coupling beam, and the magnitude of each shear force is decided by the stiffness and strength of the beam. Therefore, the proper design of coupling beams is important to the coupled wall systems.

$$M_{o(\text{coupled})} = M_1 + M_2 + TL \quad \text{Eq. (1)}$$

Figure 1 - A Coupled Wall System

The Figure 2 shows the internal forces, bending moment and shear force in the coupling beam. When the coupling beams cannot transfer the shear force by some damage, the stiffness of the coupled wall system will be decreased and the deflection will be larger than before.

Figure 2 - Deformation of Coupling Beams under lateral loads

Conventionally reinforced concrete coupling beams have the longitudinal flexural reinforcement with stirrups in early experiments of coupling beams (Paulay, 1969). Some shear forces of these beams resisted by the concrete and it will be failed by the diagonal tension while the force makes a principal diagonal crack as shown in Figure 3 (a). To prevent this diagonal tension failure, the beams are reinforced by more conventional shear reinforcement using capacity design principles. Although these beams can improve some limited ductility, it makes a sliding shear failure under a few load reversals as shown in Figure 3 (b). Because the sliding shear failure suddenly occurs, diagonal reinforcement was developed to improve this phenomenon. The diagonal bars bracing with the diagonal tension and compression like truss (Figure 3 (c)) make the ductile behavior, and these coupling beams dissipate a substantial amount of energy under the earthquake. However, because a buckling of the bars in compression can occur, a proper transverse reinforcement should be considered.

Figure 3 - Failure Modes of Coupling Beams

Definition of the structural walls in ACI Section 2.2 is the wall proportioned to resist combination of shears, moments, and axial forces. A shear wall is a structural wall, and mostly the shear walls resist lateral loads due to wind or earthquakes to the building in addition to the gravity

loads from the floors and roofs. These walls have a larger stiffness and a relatively smaller displacement by shear forces than frame structures. When the height-to-length aspect ratio (h_w/l_w) of the walls is less than or equal to 2, the walls are called ‘short or squat walls’. The shear walls have an aspect ratio of greater than or equal to 3, and these are called slender or flexural walls. The squat walls respond with a shear behavior in general, and the behavior of slender walls is dominated by flexure. Figure 4 shows this behavior with exaggerated expressions. The walls in the tall buildings typically are the slender walls with flexure behavior.

Figure 4 - Deformed Shape of Walls Subjected to Lateral Loading

The failure modes of structural walls can be classified into flexural and shear failures. Figure 5 shows the flexural failure of the walls, and Figure 5 (a) occurs when a vertical reinforcement ratio is low. The flexural reinforcing bars in tension part are gradually extended and fracture serially from the outside at the base of the wall. Before the reinforcements fracture, only one main flexural crack is formed. Figure 5 (b) shows a shear failure after the occurrence of several flexural cracks, and the failure mode is due to diagonal tension or diagonal compression caused by shear. Figure 5 (c) is a general failure mode, and the concrete at the lowest part of wall is finally crushed as yields of flexural reinforcement are expanded. Figure 6 shows the phenomena of the shear failure mode. Figure 6 (a) occurs when shear reinforcements are insufficient, and the reinforcing bars crossed the diagonal cracks resist the diagonal tension. When the diagonal tension failure of Figure 6 (a) is controlled by the sufficient shear reinforcements and high shear forces can be resisted by these, Figure 6 (b) occurs if the compressive stress at the compression strut of the wall is over the compressive stress of concrete. The compression strut will be crushed. Figure 6 (c) represents the sliding shear failure along the

construction joint when the (a) and (b) cases are controlled and the aspect ratio is lower.

Figure 5 - Flexural Failure Modes of Structural Walls (Cho, et al. 2007)

Figure 6 - Shear Failure Modes of the Structural Walls (Cho, et al. 2007)

2.2 Design Criteria of Coupled Shear Walls

In common with the shear walls, the design methods of coupling beams follow the ACI Code in this study. ACI Code Section 21.9.7 states that coupling beams with aspect ratio $(l_n/h) \geq 4$ shall satisfy the requirements of Section 21.5 as the flexural member of a special moment frame with conventional reinforcement. Coupling beams with $l_n/h < 2$ and with $V_u > 0.33\sqrt{f'_c}A_{cw}$, where A_{cw} is the coupling beam area, shall be reinforced with two intersecting groups of diagonally placed bars. The other coupling beams can select the reinforcement type between the diagonally placed bars and the special moment frame detailing. As a horizontal wall segments, the nominal shear strength of coupling beam shall be smaller than $0.83A_{cw}\sqrt{f'_c}$.

Proper design methods are also necessary for shear walls to prevent the brittle fracture behavior and perform the basic function of resisting lateral loads. The desirable behavior of the walls develops the flexural hinge at the base of the wall. It means that the flexural reinforcement should be yielded in the plastic hinge regions to provide ductility of the wall. Generally, the shear walls are designed in accordance with ACI Code Section 11.9 and Chapter 14. The ultimate shear stresses at any horizontal section shall be taken smaller than $0.83\sqrt{f'_c}$. The design methods of special structure walls with high ductility are given in the ACI Code Section 21.9. A structure which consists of several walls to resist the factored shear force has the average unit strength limited to $0.66\sqrt{f'_c}$. For any one of the individual vertical walls in this structure, the unit shear strength shall not exceed $0.83\sqrt{f'_c}$. After the wall cross section is detailed, the parts to be considered for seismic design are boundary elements. For the walls with a high compression demand at the edge, boundary elements are required and these can be developed with extra confinement or longitudinal bars at the end of wall or widened end with confinements.

2.3 Previous Experimental Works of Coupled Shear Walls

The response of coupled systems is not simple to be accurately predicted. While the individual wall or beam components have been actively experimented, relatively few experimental works were conducted to analyze the response of coupled wall systems. Table 1, Table 2, and Table 3 show the design details of these several previous experiments, and the results of these experiments are summarized as follows.

Hisatoku and Matano (1972) tested the initial coupled wall systems of 1/20th scale model with 20 story to investigate the stress distribution and deformation properties of the system, degree of rigidity contribution of connecting beams, and structural failure process. From the test, following remarks were obtained. First, the deformation patterns and stress distribution were similar to the behavior of one-story-one-bay rigid frame. The plastic hinges were formed at the end of coupling beams and base of the wall, and the bending and shear failure of the top coupling beams caused collapse of the system. However, because this paper was not written in English, the detail data could not be obtained.

In 1974, Santhakumar tested two 7-story coupled walls of 1/4 scale. The specimens Shear Wall A and Shear Wall B had similar reinforcement in the two walls, and the main variation of the test was reinforcement of beams. The beams of Shear Wall A had conventional reinforcement for shear and flexure, and the coupling beams of Shear Wall B had diagonal bars together with nominal secondary reinforcement. All the coupling beams in Shear Wall A failed by sliding shear, and the beams in Shear Wall B had much less damage than in Shear Wall A despite large yield. The results of the quasi-static loaded test indicated that the higher ductility by diagonal reinforced coupling beams effected to improve overall behavior of system response. The energy dissipation of Shear Wall A localized at the base of walls, and the energy of Shear Wall B was dissipated over the full height of the system. The Shear Wall B absorbed the twice cumulative energy than that of Shear Wall A.

Aristizabal-Ochoa and Sozen (1976) worked dynamic tests of 10-story coupled wall systems with small scale to observe the response of reinforced concrete structures resisting earthquake and the results of initial yielding location. Two types of specimen were built; type D (the standard test frame) and type M (the modified test frame). The main difference between these specimens was the amount of steel in the beams. The failure between two type of specimens was different with each other. The main failure process of type D was the flexural cracks at the ends of the beam connections and the additional damage at the bases of walls. In case of type M, the critical damage was observed on the exterior edges of the wall base. A critical result of the dynamic response was that the maximum top displacements of the structures were dominated by the first mode, and the contribution of higher mode was less than 7%. The cracks in the walls and beams reduced the natural frequencies inferred from the

displacement and acceleration waveforms. A triangular distribution of lateral load was recommended to the most proper vertical distribution, because the centroid of lateral action was located at 0.7 times of the wall height.

Other tests of small-scale coupled systems with 6 stories were conducted by Lybas and Sozen (1977). The five reinforced concrete coupled wall systems were subjected to earthquake base motions, and an additional specimen was subjected to static lateral loading. The principal variable in the specimens was the strength and stiffness of the connecting beams. The specimens were grouped into three classes (Type A, Type B, and Type C) according to their beam cross-section. As results from the experiment, type A under the dynamic loading was failed by the development of the maximum tensile capacity at the wall bases without any yields of beams. The failure mechanisms of Type B and C under the dynamic loads were similar to each other, and these specimens had a flexural yielding at the ends of the beams first and at the bases of walls subsequently. The tests showed that the natural frequency of the specimens decreased under the increased and continuous base motion. As the strength and stiffness of the coupling beams were decreased, the relative contribution to base shears and moments of higher modes increased. As the results of the static hysteretic analysis, the beams significantly contributed to the overall hysteresis of the structure.

Shiu et al. (1981) conducted the two coupled wall tests with 6 story of 1/3 scale to determine effects of coupling beam on the coupled wall. A system with weak coupling beams (CS-1) and a system with strong repaired beams (RCS-1) were used for this test. The coupling beams in CS-1 was yielded relatively early in the test, and the beams could not dissipate a sufficient energy. This resulted in the separate behavior of the walls. The RCS-1 resulted in large axial stresses which caused web crushing in the base of the walls, because the beams had relatively too strong than the walls. This was noted that beams require a sufficient deformation capacity to remain the coupling action. At the same time, the walls should maintain the axial load when the coupling beams dissipate the sufficient energy.

Three coupled structural walls with approximately 1/30th scale of a 15-story building were tested to investigate failure modes (Subedi, 1991). The different dimensions of the specimens were chosen for the different failure mode; flexural, shear, and composite. Two types of loadings were applied. Model 1 and 2 were tested under a point load at the top and Model 3 was tested under a four-point system for triangular distribution over the height. The three specimens represented the intended modes of failure. Model 1 was failed with flexure behavior at nearly all coupling beams and crushing of compression wall. The walls of Model 2 were failed compositely with minor damage in coupling beams. The failure mechanism of Model 3 consisted of shear failure in beams followed by crushing of compression wall. Although the size of three specimens was a small range of practical structure, it was clear that the coupling beam greatly affected on the behavior of the coupled wall systems.

Xillin et al. (2005) tested three coupled walls of 1/4 scale with 5 stories, named CW-1, CW-2,

and CW-3. The height of coupling beams was the variables of this test, and it made different coupling ratio between the walls. The horizontal loading was applied to the specimens. CW-1 and CW-3 were loaded at two points to represent the inverted triangular load, and CW-2 was loaded at one point of the top. The results of cracking procedure were very similar to each other, and the failures of all specimens occurred at the bottom of the walls. However, CW-1 was dominated by flexure failure, CW-3 was dominated by shear failure, and the both failure modes were represented in CW-2. Although the more damages of coupling beams were expected to prevent the failure of walls according to the design code (JGJ 101-96 in China), the different results were occurred at this experiment. There were no details about these results in this paper because the purpose of the study was the verification of a nonlinear model.

The main purpose of the previous tests was to understand the behavior of coupled wall structures depending on the design of coupling beams. As the writer's knowledge, there is one of experimental works for coupled wall systems using HPFRCCs. This was conducted by Lequesne in 2011, and the details will be considered in Chapter 4 to compare the results by nonlinear analysis.

Table 1 Design Details of Previous Experimental Works for Coupled Walls

Reference			Santhakumar (1974)				Aristizabal-Ochoa et al. (1976)					
Wall	Specimen		Shear Wall A & Shear Wall B				type-D			type-M		
	Scale		1/4				1/12					
	Shape		Rectangular				Rectangular					
	t _w	mm	102				25.4					
	l _w	mm	610				178					
	A _g	mm ²	61900				4520					
	Floor		1~3F	4F	5F	6, 7F	1~3F	4, 5F	6~10F	1~3F	4, 5F	6~10F
	h _w	mm	3960	762	762	1520	686	457	1240	686	457	1240
	ρ _t (%)		3.49 (end), 0.700 (middle)	2.79 (end), 0.700 (middle)	2.09 (end), 0.700 (middle)	1.39 (end), 0.700 (middle)	3.30 (end), 0.61 (middle)		1.65 (end), 0.600 (middle)	3.30 (end)		
	ρ _t (%)		0.88	0.67	0.67	0.44	0.310 (whole) 1.33 (confinement)	0.31		0.410 (whole) 1.33 (confinement)	0.41	
	f' _c	MPa	36.4 (A), 38.2 (B)				32.5			32.8		
	f _y	MPa	305				496 (#8 wire), 731 (#16 wire)					
f _{yt}	MPa	352				731						

	Load type		static test (under three-point system, triangular distribution)				dynamic test (with base motion)			
	Axial load		25 kips (111kN) using two prestressed cables at the centroid of each wall				1000 lb (4.45 kN) at each of the ten stories		1000 lb (4.45 kN) at each of the ten stories	
	P/A_cf_c'		0.0257 (A), 0.0245 (B)	0.0256 (A), 0.0244 (B)	0.0254 (A), 0.0242 (B)	0.0252 (A), 0.024 (B)	0.155 (at the base)	0.0767 (at the 6th story)	0.154 (at the base)	
	P/A_gf_c'		0.0247 (A), 0.0235 (B)	0.0247 (A), 0.0235 (B)	0.0247 (A), 0.0235 (B)	0.0247 (A), 0.0235 (B)	0.152 (at the base)	0.0758 (at the 6th story)	0.150 (at the base)	
Beam	Specimen		Shear Wall A		Shear Wall B		type-D		type-M	
	Reinf. type		conventional		diagonal		conventional			
			doubly reinforced		doubly reinforced					
	t	mm	76.2		25.4		25.4			
	h	mm	305		38.1		38.1			
	A_g	mm²	23200		968		968			
	I_n	mm	381		102		102			
	I_n/h		1.25		2.67		2.67			
	Flexural reinf.		2 #3 bars		2 #3 bars & 2 Φ6.35 mm bars		1 #8 gage wire		4 #8 gage wire	
	Shear reinf.		Φ6.35 mm @ 50.8 mm		Φ4.76 mm @ 114 mm		10 #16 gage wire @ 11.43 mm		20 #16 gage wire @ 5.334 mm	
	f_c'	MPa	36.4		38.2		32.5		32.8	
	f_y	MPa	315		315 & 346		496		496	
f_{vt}	MPa	346		230		731		731		

Table 2 Design Details of Previous Experimental Works for Coupled Shear Walls

Reference		Lybas et al. (1977)						Shiu et al. (1981)
Wall	Specimen	D1	D2	D3	S1	D4	D5	CS-1, RCS-1
	Scale	1/11						1/3
	Shape	Rectangular						Rectangular

	t_w	mm	25.4						102					
	l_w	mm	178						1910					
	A_g	mm²	4520						194000					
	Floor		6						1	2	3	4	5	6
	h_w	mm	1520						5490					
	ρ_l (%)		0.980						6.15 (end), 0.370 (middle)					
	ρ_t (%)		1.11						0.550 (whole), 0.410 (confinment)		0.550 (whole), 0.140 (confinment)			
	f'_c	MPa	31.4	39.4	31.8	36.0	34.2	28.8	30.5	23.4	25.8	25.0	21.0	25.9
	f_y	MPa	291	302	312	305	307	308	434 (#4), 531(Φ6)					
	f_{yt}	MPa	291	302	312	307	308	305	531 (Φ6), 510 (D3)					
	Load type		dynamic test (with base motion)				static test (5P at 6th story, 3P at 4th story, and P at 2nd story)	dynamic test (with base motion)	static test (at the top)					
	Axial load		2000 lb (8.90 kN) at each of 2nd, 4th, and 6th story						0					
	P/A_cf'_c		0.0951	0.0758	0.0939	0.0829	0.0873	0.104	0					
	P/A_gf'_c		0.0942	0.0751	0.0930	0.0821	0.0864	0.103	0					

Beam	Specimen		Type A	Type B			Type C			CS-1		RCS-1	
	Reinf. type		Conventional						conventional				
			doubly reinforced						doubly reinforced				
	t	mm	25.4	25.4			25.4			102		254	
	h	mm	57.2	38.1			38.1			170		203	
	A_g	mm²	1450	968			968			17300		51600	

I_n	mm	102	102			102		423	211
I_n/h		1.78	2.67			2.67		2.49	1.04
Flexural reinf.		2 #8 gage wire (steel ratio of 3.7%)	1 #11 gage wire (steel ratio of 1.52%)			1#13 gage wire (steel ratio of 0.88%)		2 Φ 6 mm	6 #3
Shear reinf.		#13 gage wire @ 9.53 mm	#13 gage wire @ 25.4 mm			#13 gage wire @ 15.9 mm		D3 wire @ 33.8 mm	D3 wire @ 33.8 mm
f'_c	MPa	31.4	39.4	31.8	36.0	34.2	28.8	25.2	24.1
f_y	MPa	295	302	312	305	288	288	531	483
f_{yt}	MPa	269	269	269	269	269	269	510	510

Table 3 Design Details of Previous Experimental Works for Coupled Walls

Reference		Subedi et al. (1991)			Xillin et al. (2005)			
Wall	Specimen	Model 1	Model 2	Model 3	CW-1,2,3			
	Scale	1/30			1/4			
	Shape	Rectangtular			Rectangtular			
	t_w mm	20.0		40.0	70.0			
	l_w mm	212		300	600			
	A_g mm ²	4240		12000	42000			
	Floor	15			1	2	3	4, 5
	h_w mm	1500			700	700	700	1400
	ρ_l (%)	1.66 (end), 0.81 (middle)		3.25	2.42 (end), 1.01 (middle)			
	ρ_t (%)	-	-	-	0.898			
	f'_c MPa	48.3	44.4	40	38.6	38.7	40.1	47.4
	f_y MPa	385	298	450	311			
	f_{yt} MPa	385	298	277	311			
	Load type	static test (at the top)		static test (under four-point system, triangular distributio n)	static test (CW1,3: 2P at the top, P at the 3rd story, CW2: at the top)			
	Axial Load	0			200 kN at the top			

	$P/A_c f'_c$		0			0.0625 (at the base)	0.0624 (at the base)	0.0602 (at the base)	0.0509 (at the base)
	$P/A_g f'_c$		0			0.0617 (at the base)	0.0615 (at the base)	0.0594 (at the base)	0.0502 (at the base)
Beam	Specimen		Model 1	Model 2	Model 3	CW-1	CW-2	CW-3	
	Reinf. type		conventional			conventional			
			doubly reinforced			doubly reinforced			
	t	mm	20.0	20.0	40.0	70.0	70.0	70.0	
	h	mm	20.0	50.0	30.0	200	250	300	
	A_g	mm²	400	1000	1200	14000	17500	21000	
	I_n	mm	75.0	75.0	90.0	400	400	400	
	I_n/h		3.75	1.50	3.00	2.00	1.60	1.33	
	Flexural reinf.		2 SWG14	2 SWG10	2 Φ 4.75mm	4 Φ 8, (2 Φ 8 at middle)	4 Φ 8, (2 Φ 8 at middle)	4 Φ 8, (2 Φ 8 at middle)	
	Shear reinf.		7 SWG19 @ 12.5mm	4 SWG19 @ 25mm	7 Φ 2.0mm @ 15mm	Φ 4 @ 50mm	Φ 4 @ 50mm	Φ 4 @ 50mm	
	f'_c	MPa	48.3	44.4	40.0	38.6	38.6	38.6	
	f_y	MPa	385	298	450	278	278	278	
	f_{yt}	MPa	385	298	277	797	797	797	

2.4 Modeling Methods of Coupled Wall Systems

2.4.1 Coupling Beam Models

The response of coupled wall systems is not simple to experiment the real scale structure. In order to predict the nonlinear behavior of these systems more accurately, extensive analytical researches are conducted. The modeling methods can be classified into microscopic models and macroscopic models. This study will mainly deal with the macroscopic modeling methods using simplified member of the structure to apply it for the analysis of a tall building.

The linear element models were studied by Aristizabal-Ochoa (1983) and this model was compared with the experimental results. The massless line elements were used instead of the members of structure at the centroid, and the joints had finite dimensions with rigid links at the each end of the beams. This model analyzed the flexural deformations and shear deformations. The rotational inertia was not considered, only the inertia forces paralleled to the plane were considered.

Nayar and Coull (1976) studied the linear element with hinged plastic ends to present the elastic-plastic analysis. The stiffness of beams is reduced due to the cracking and this paper represented the effect by using the hinges. The hinges were formed in an intersection between coupling beam and walls after the beam reached the ultimate moment or shear capacity. The members had a bilinear moment-rotation relationship.

Filippou's model (1999) discussed subelement with varied stiffness in different regions to represent the effect of reinforcement slip in critical region. The subelements described the force-displacement relationships. Three subelements were used in the paper such as elastic subelement, spread plastic subelement, and interface bond-slip subelement as shown in Figure 7. The elastic subelement modeled the linear elastic behavior of the element until yielding behavior, the spread plastic subelement modeled the post-yield behavior of the member, and the interface bond-slip subelement modeled the fixed-end rotations at the interface by the bond deterioration and reinforcement slip. The joint panel zone was assumed to be rigid.

Figure 7 - Decomposition of a RC Beam Model (Filippou et al., 1999)

Naish et al. (2010) studied the modeling of reinforced concrete coupling beams to assess the current modeling approaches. Especially, this paper focused on the modeling parameters about effective elastic stiffness, deformation capacity, and residual strength. The effective elastic bending and shear stiffness values are required for elastic analysis. The effective bending stiffness at yield has to consider the slip/extension impact of the reinforcement. ASCE 41-06 (2007) defined this; $0.12E_cI_g$ for the test beams. The load-deformation backbone relations for full scale beams were shown in Figure 8 describing the results of tests and ASCE 41-06 model. The total chord rotations of test beams at yield, strength degradation, and residual strength were 0.7%, 6.0%, and 9.0% respectively. The residual strength for tests was $0.3V_n$ up to 10 ~ 12% rotation and it was $0.8 V_n$ at plastic rotation of 5% for ASCE 41-06. This study included the application of the models to computer modeling (CSI, PERFORM3D). There were two types of model as shown in Figure 9. First models (Figure 9 (a)) had the moment hinge using a rotational spring at each end of beam to describe the nonlinear deformations. Figure 9 (b) model had the shear-displacement spring at the middle of the beam to model the nonlinear deformations. Both models incorporated the elastic slip/extension springs at each end and elastic beam cross-section.

Figure 8 - Load-Deformation Backbone Curves of Coupling beams by Naish et al. (2010)
(dotted line: 1/2-scale)

Figure 9 - Coupling Beam Models (Naish et al., 2010)

2.4.2 Shear Wall Models

Various models for walls have been proposed to improve some drawbacks by limited conditions. In the early stage of modeling, an important object of modeling methods was to develop more simple and accurate model, and each element of the coupled systems was replaced with flexural elements. The general modeling approach for wall hysteretic behavior was the Beam-Column Element Model (Takayanagi and Schnobrich, 1976) as shown in Figure 10. The main components of this model are an elastic flexural element, two nonlinear axial springs, and two nonlinear rotational springs. The elastic flexural element is account for flexural behavior of the wall, nonlinear axial springs represent the inelastic behavior at the end of the wall, and nonlinear rotational springs at each end describe inelastic behavior related with plastic hinge and fixed-end rotation of wall element. Although the modeling method is simple and easy, it cannot describe the important features of wall behavior like the variation of neutral axis of the wall cross section, rocking of the wall, and interaction with the frame members around the the wall.

Figure 10 - Beam-Column Element Model for RC Walls (Takayanagi and Schnobrich, 1976)

The Three-Vertical-Line-Element Model (TVLEM) was proposed by Kabeyasawa et al. (1983) as shown in Figure 11 to explain the variation of neutral axis and interaction with the frame members connected with the wall. The vertical springs and rotational spring account for the flexural behavior of the wall, and the horizontal spring accounts for the shear behavior of the wall. The axial force-deformation relation of the three vertical line elements was modeled with an axial-stiffness hysteresis model (ASHM, Figure 12 (a)), and the force-deformation relation of both the rotational and horizontal springs at the wall centerline was modeled with an origin-oriented hysteresis model (OOHM, Figure 12 (b)). This modeling method assumes that the shear stiffness degradation is independent of the axial force and bending moment. Therefore, the TVLEM cannot precisely describe the actual structure behavior when the wall systems are governed by shear.

Figure 11 - TVLEM for RC Shear Walls (Kabeyasawa et al., 1983)

(a) ASHM (Kabeyasawa et al., 1983)

(b) OOHM (Kabeyasawa et al., 1983)

Figure 12 - Force-Deformation Relations of Hysteresis Models for RC Members

Vulcano et al. (1988) improved the flexural modeling of wall systems to describe the yielding process of reinforcement by proposing the Multi-Vertical-Line Element Model (MVLEM) as shown in Figure 13. Two external vertical elements have the axial stiffnesses of boundary columns, and the two or more interior elements represent the axial and flexural behavior of the web. However, the MVLEM still has weakness in predicting nonlinear shear behavior, and it is hard to predict the relative contributions between shear deformation and flexural deformation.

Figure 13 - MVLEM for RC Shear Walls (Vulcano et al., 1988)

Colotti (1993) modified the MVLEM by substituting the horizontal spring with a two-dimensional nonlinear panel as shown in Figure 14. The panel involves a relation between the axial and shear components of RC wall response. Although this model produces more accurate results, the drawback related to the relative contributions of shear and flexural deformations on wall displacements still could not be resolved, as it just considered the shear and axial interactions.

Figure 14 - Modified MVLEM for RC Shear Walls (Colotti, 1993)

Fiber Element Model (Petrangeli et al., 1999a) is a modeling method that analyzes the section of a structure component divided into fibers which can only lead to axial deformation. This model can predict a moment-curvature relationship of the section based on the material stress-strain relationships of fibers and an assumption of strain distribution shape of the section. In case of wall element modeling, a section of the wall is subdivided into multiple fibers that are not necessary to have an equal area, and each fiber can be assigned concrete, structural steel, or reinforcing bar material properties (Figure 15). The strain of a fiber (Eq. (2)) is found from section strain variables (ϵ_0 , ϕ_x , and ϕ_y) based on the section deformation using a strain-deformation relationship and a

shape function. The stress of a fiber (Eq. (3)) is calculated from the fiber strain using relevant constitutive models, and the stresses are integrated over the cross-sectional area to obtain stress resultants such as force or moment.

$$\varepsilon_i = \varepsilon_0 + y_i \phi_x + x_i \phi_y \quad \text{Eq. (2)}$$

$$\sigma_i = E_i \varepsilon_0 + E_i y_i \phi_x + E_i x_i \phi_y \quad \text{Eq. (3)}$$

Once, the section forces that are associated with the section deformation are known, the corresponding wall forces can be calculated using the virtual work principle. However, since this model is also used restrictively in the shear deformation analysis, the Fiber-Spring Element Model (FSEM) was proposed by combining a shear spring with a fiber element to reflect the shear deformation (Lee et al., 2011).

Figure 15 - Fiber Element Model for RC Shear Walls (Petrangeli et al., 1999a)

Previously proposed nonlinear modeling methods for coupled wall systems describe the actual behavior well, in the case of structures governed by flexure. The main flaw of macroscopic modeling methods is that the general shear force/shear deformation relation does not coincide with the shear behavior at the plastic hinge region after flexural yielding. In other words, although the shear deformation is affected by the flexural cracks that occur before shear cracking, most models could not reflect this. If the shear deformation is predicted as a function of the wall aspect ratio, the FSEM can obtain more accurate results than the Fiber Element Model. However, since the experimental results or precise finite element analysis are required for the parameter setting of the shear springs, it takes substantial efforts to implement the FSEM. When the RC shear wall is expected to show the shear dominant behavior, a more simple and reasonable analytical model is required to predict the interaction between the flexural and shear deformations.

Chapter 3. Development of Analysis Methods

3.1 Force-Deformation Relationship in PERFORM-3D

Figure 16 shows the force-deformation model for most of the inelastic components, and it can describe the hysteric property in detail. The point Y is about yielding stress and strain. The point U and L as a peak stress means the ultimate stress and strain of the materials and can express a ductile limit. The point R can consider a remaining stress of material, and point X is about maximum strain of material. This model is trilinear relationship with strength loss, and the bilinear or no strength loss model also can be constructed according to user selection.

Figure 16 – Nonlinear Force-Deformation Relationship in PERFORM-3D

3.2 Modeling Methods of Coupling Beams

The coupling beams have significant inelastic deformation capacities, and the beams must maintain high stiffness and strength for meaningful degree of coupling. The conventional reinforcement is commonly used in the coupling beams, and it consists of the longitudinal reinforcement for flexural and transverse reinforcement for shear. In case of the coupling beams using diagonal reinforcement, the flexure and shear is resisted by this diagonal reinforcement which have better retention of strength, stiffness, and energy dissipation than the conventional reinforcement.

As the nonlinear modeling is required for the tall buildings design under the seismic loads, Naish, at al. (2010) model which is the relatively simple and accurate application method will be

used in the coupling beam modeling to estimate and develop the modeling methods using computer program. The ASCE 41-06 modeling parameters for coupling beams will be used in this paper.

The coupling beams used in this study are expected to develop flexural yielding at their both ends. Accordingly, the model of the coupling beam consists of a ‘Moment Hinge of Rotation Type’ on each end and an ‘Elastic Beam Section’ arranged in series as shown in Figure 17. The rigid-plastic moment hinges in PERFORM-3D indicate the inelastic bending behavior. The hinges can rotate when the moment of beams reaches the yield moment. Before the yielding of the beam, the stiffness of beam is expressed by the elastic segment and the deformation of this element is in the elastic part. And then, after the yield moment is reached, the rigid-plastic moment hinges represent the plastic deformation with hinge rotation. Naish et al. (2010) recommended the conservative effective elastic stiffness with $0.20E_cI_g$ in the cross section instead of slip/extension hinge. In other words, because this parameter includes the effect of slip/extension deformations on the overall load-deformation behavior, the slip/extension hinge could be excluded. This makes the coupling beam models more efficient in a tall building model that needs many components.

Figure 17 - Coupling Beam Model

The plastic hinge model assumed that all inelastic deformation is concentrated in zero length plastic hinges. ASCE/SEI 41-06 suggested the use of tabulated modeling parameter (Table 4) to approach backbone curves (Figure 18) for the nonlinear behavior of the coupling beam. The generalized force is yield moment and the generalized deformation for the beams is the rotation in the flexural plastic hinge zone. In Figure 18, A is an unloaded component, B is an effective yield point, significant strength degradation begins at point C, and the degraded strength up to D is maintained up to E. The parameters ‘a’ and ‘b’ are the portions of deformation after yielding, and the parameter ‘c’ is the reduced resistance. The parameters ‘d’ and ‘e’ are the total deformation from origin. Figure 19 shows the force-deformation model to represent this code in PERFORM-3D. The U point in this figure corresponds to the B point in the code figure. The other points C, D, and E were matched with L, R, and X respectively.

Figure 18 - Generalized Force-Deformation Relation for Concrete Elements or Components (ASCE 41-06)

Table 4 Modeling Parameters for Coupling Beams in ASCE 41-06

Reinforcement configuration	$\frac{V}{b_w h \sqrt{f'_c}}$	Controlled by flexure			Controlled by shear		
		a	b	c	d	e	c
With conforming transverse reinforcement	≤ 0.250	0.0250	0.0500	0.750	0.0200	0.0300	0.600
	≥ 0.500	0.0200	0.0400	0.500	0.0160	0.0240	0.300
With nonconforming transverse reinforcement	≤ 0.250	0.0200	0.0350	0.500	0.0120	0.0250	0.400
	≥ 0.500	0.0100	0.0250	0.250	0.00800	0.0140	0.200
Diagonal reinforcement	n.a.	0.0300	0.0500	0.800	-	-	-

Figure 19 - Force-Deformation Model of Coupling Beams in PERFORM-3D

The confinement effect by using HPFRCCs in beams can help to prevent buckling, and HPFRCCs can improve toughness and increase the energy dissipation capacity of the coupling beams. HPFRCCs increase the maximum shear stress of coupling beams from ACI Code with approximately 40% (Canbolat et al., 2005). The HPFRCCs coupling beams may have increased the flexural strength by the strain hardening of HPFRCCs and can also improve the ductility through distributing damage over multiple cracks. These will be modeled with 1.2 times of flexural strength and 2 times of ductility in this study.

3.3 Modeling Methods of Shear Walls

In the PERFORM-3D software, two kinds of wall elements are usually considered to predict the wall behavior. A 'Shear Wall Element' model is mainly for the relatively slender shear wall structures, and a 'General Wall Element' model is to analyze the complex reinforced concrete walls with irregular openings (Computer and Structures Inc., 2006). A General Wall Element consists of all of the parallel five layers as shown in Figure 20 to describe the axial-bending, shear and diagonal compression behavior. However, the Shear Wall Element consists of the (a) and (c) layers in Figure 20 to consider the vertical axial-bending and shear behavior.

Because the flexural failures of the specimens were expected due to the large slenderness ratio of the example structures used in this study, the Shear Wall Element can be enough to describe this structural behavior rather than the General Wall Element. An elastic shear deformation is considered using a shear material model. The horizontal stiffness and the out-of-plane stiffness are assumed to be elastic in the Shear Wall Element.

Figure 20 - Parallel Layers in General Wall Element on PERFORM-3D

The shear wall element bases on the ‘Inelastic Fiber Element’ for the vertical axial-bending behavior. The wall element is comprised of the fiber element which performs only axial deformation. This fiber model can consider the variation of neutral axis by the axial force. As the fiber model bases on a material stress-strain relation of each fiber and an assumption of the deformed distribution shape of the cross section, a relatively exact moment-curvature relation of the cross section can be predicted. Therefore, when the wall element was properly constructed with the material model in fiber, the nonlinear behavior such as the confinement effect by transverse steels, the concrete crushing, the tensile cracks of the concrete, the tensile yielding of the steel and the fractures can be precisely described. In other words, it can describe some complex behavior occurred at the reinforcement and concrete more exactly.

After dividing the walls with this shear wall element, the each elements of wall was divided again with fiber unit about the horizontal cross-section of shear wall as shown in Figure 21. The shear wall model consists of a combination of multiple discretized ‘concrete fiber elements’ and ‘steel fiber elements’, which are embedded in a series of shear wall elements.

Figure 21 - Shear Wall Fiber Model

In case of steel fiber element, the stress-strain relationship was simply constructed using a trilinear curve as shown in Figure 22. The initial modulus of elasticity for all steels E_s is modeled 200 GPa, and the slope in a hardening range is assumed $0.02E_s$.

For concrete material, Figure 23 shows the model shape of the stress-strain relationship and three type values were considered depending on materials property; unconfined concrete, confined concrete, and HPFRCCs. In compression, the each peak compressive stress (f'_c) of the unconfined concrete was taken for the point FU. Before the compressive stresses reach the specified f'_c , Eq. (4) proposed by Hognestad (1952) was used to represent the compressive constitutive response of concrete. In this equation, ϵ_0 is the strain at the f'_c , and it is assumed to be 0.002. Eq. (5) proposed by Kent and Park (1971) represents the linear descending part after the post-peak response.

$$f_c = f'_c \left[\frac{2\epsilon_c}{\epsilon_0} - \left(\frac{\epsilon_c}{\epsilon_0} \right)^2 \right] \quad \text{Eq. (4)}$$

$$f_c = f'_c [1 - Z(\epsilon_c - \epsilon_0)] \quad \text{Eq. (5)}$$

Here, the ϵ_c is the strain in the concrete, ϵ_0 is the strain at the f'_c , and Z is the slope of the descending branch. Z is assumed to be 150 for unconfined concrete and 50 for both confined concrete and HPFRCCs, because HPFRCCs behaves like confined concrete with greater ductility in compression. Normal concrete was assumed to have zero tensile strength as typical, but HPFRCCs show strain hardening response in tension by the fiber bridging action. This is modeled using the tested values by the paper of Lequesne (2011). As the existing researches about the stress-strain

relationship of concrete, the confined concrete surrounded with stirrup has an increased strength and ductility. This confinement effect can be calculated according to the proposal of Chang and Mander (1994).

To consider the reduced stiffness of RC walls due to concrete cracking, ACI Code Section 10.10.4.1 specifies coefficients applying to the moment of inertia I_g ; $0.70I_g$ (uncracked walls), $0.35I_g$ (cracked walls). These coefficients influence on the stiffness EI . In this modeling, the reduced stiffness is considered in the modulus of elasticity (E) of the concrete material instead of I_g .

The energy degradation factors at each model point were used to consider the reduced unloading and reloading stiffnesses under the cyclic loading. These factors adjust the reduced area inside the load-displacement loop, and the reduced area means the dissipated inelastic energy. The most energy dissipation is occurred at the reinforcements and it is assumed with 0.5 after the reinforcement reaches the ultimate strength in this study.

$$\text{Area of degraded loop} = (\text{non - degraded area}) \times (\text{energy degradation factor})$$

Figure 22 - Stress-Strain Model of Steel Material

Figure 23 - Stress-Strain Model of Concrete Material

Chapter 4. Analysis of Individual Coupled Shear Walls

4.1 Example Structures

The coupled wall specimens tested by Lequesne (2011) were used to evaluate the proposed modeling methods. Confinement with shear reinforcement can lead to some difficulties in construction work. Instead of this, the use of HPFRCCs can reduce these troubles because the fibers impede the crack opening of concrete and improve the tensional behavior of concrete like the confinement effect. This paper studied the impact that the ductility exhibited by HPFRCCs components in the critical regions has on the system performance. For this purpose, there are approximately 1/3 scale two specimens (CW-1 and CW-2) applied cyclic displacement reversals (Figure 27) on 4th floor. Both four-story specimens consisted of two T-shaped reinforced walls, four precast coupling beams, and slabs at the second and forth levels. The dimension and details of the specimen are shown in Figure 24 and Figure 25. The shaded areas in these figures were constructed with HPFRCCs. The coupling beams of the example structures designed diagonal reinforcement to prevent sliding shear failures and ensure ductile behavior. Figure 26 shows the dimension and details of the diagonal reinforced coupling beams with an angle of approximately 28°.

The differences between CW-1 and CW-2 specimens are shown in Table 5. The main difference is the material used at the first two stories of each wall. Instead of normal concrete in CW-1, HPFRCCs were used in CW-2. The wall details of CW-1 designed in accordance with ACI Building Code 318-08. The first two stories of CW-2 wall were made with HPFRCCs, and designed with a higher concrete shear stress contribution. Therefore, reduced amount of vertical and confinement reinforcement was designed. Another difference about detailing is that the dowel bars at the base of CW-2 were designed to supplement the flexural reinforcement at the cold joint. The longitudinal reinforcements of CW-1 coupling beams were terminated in the wall by 75 mm from the beam-to-wall interface. Because this made the damage localization at the interface, there was limited the ability of coupled effect. The CW-2 had the fully developed coupling beam reinforcement.

This RC coupled wall tested the used the conventional concrete, and the results of concrete material tests are shown in Table 6. Another material HPFRCCs was the mixture of concrete with randomly distributed fibers. Table 7 shows the average results of HPFRCCs properties from the cylinders test. Grade 60 mild-steel properties are shown in Table 8.

Figure 24 - Example Structures CW-1 Tested by Lequesne (2011)

Figure 25 - Example Structures CW-2 Tested by Lequesne (2011)

Figure 26 - Coupling Beams in CW-1 & CW-2 (Lequesne, 2011)

Figure 27 - Cyclic Loading History (Lequesne, 2011)

Table 5 Test Variables in Two Specimens

Variation	CW-1	CW-2
Material of first two stories of walls	Normal concrete	HPFRCCs
Reinforcement in first two stories of walls	Designed & detailed to satisfy Chapter 21 of ACI 318-08	Reduced confinement reinforcement for boundary elements & vertical reinforcement
Dowel bars at base	X	Placed at wall-to-foundation Interface along the cold joint
Horizontal reinforcement in coupling beams	Terminated in the walls by only 3 in. from the interface	Fully developed into the walls

Table 6 Properties of Normal Concrete

	Location of Pour	Specified f'_c (MPa)	28-Day f'_c (MPa)	Test Day f'_c (MPa)
CW-1	Beam-2	41.0	37.0	68.0
	Foundation	28.0	34.0	53.0
	Wall 1st story	28.0	37.0	48.0
	Wall 2nd story	28.0	28.0	46.0
	Slab #1	28.0	25.0	37.0
	Wall 3rd story	28.0	38.0	46.0
	Wall 4th story	28.0	48.0	66.0
	Slab #2	28.0	51.0	66.0
	Wall 5th story	28.0	46.0	61.0
CW-2	Beam-2	41.0	46.0	63.0
	Foundation	28.0	50.0	52.0
	Slab #1	28.0	41.0	46.0
	Wall 3rd story	28.0	54.0	57.0
	Wall 4th story	28.0	45.0	48.0
	Slab #2	28.0	50.0	53.0
	Wall 5th story	28.0	53.0	56.0

Table 7 Properties of HPFRCCs

	Location of Pour	28-Day Tests					Test Day f'_c (MPa)
		f'_c (MPa)	ASTM 1609 Flexural Tests				
			σ_{fc} (MPa)	σ_{peak} (MPa)	$\sigma_{(\delta=L/600)}$ (MPa)	$\sigma_{(\delta=L/150)}$ (MPa)	
CW-1	Beam-1	38.0	4.90	7.10	6.70	3.60	71.0
	Beam-3	38.0	4.90	7.10	6.70	3.60	71.0
	Beam-4	41.0	5.70	7.70	7.20	4.10	74.0
CW-2	Beam-1	41.0	5.70	7.70	7.20	4.10	72.0
	Beam-3	38.0	4.90	7.10	6.70	3.60	72.0
	Beam-4	41.0	5.70	7.70	7.20	4.10	72.0
	Wall 1st Story	50.0	5.50	7.50	7.20	5.10	51.0
	Wall 2nd Story	46.0	5.80	7.20	7.00	3.90	50.0

Table 8 Results from Tension Tests of Steel Reinforcement

	Location	Bar Size	Yield Stress (MPa)	Ultimate Stress (MPa)
CW-1	Coupling Beams	#2 (D6)	440	505
		#3 (D10)	525	815
		#4 (D13)	525	795
	Structural Wall	#2 (D6)	440	505
		#3 (D10)	510	770
		#5 (D16)	465	750
		#6 (D19)	470	750
CW-2	Coupling Beams	#2 (D6)	440	505
		#3 (D10)	525	815
		#4 (D13)	525	795
	Structural Wall	#2 (D6)	440	505
		#3 (D10)	465	810
		#4 (D13)	415	670
		#5 (D16)	465	750
		#6 (D19)	470	750

4.2 Modeling Processes

4.2.1 Coupling beam Modeling

The coupling beam was modeled with moment hinge element and elastic cross section in chapter 3. The strength of these coupling beams could be calculated according to ACI Code. The flexural strength of the diagonal reinforced beam is assumed to be only dependent on the diagonal reinforcement, and the nominal moment can be calculated using Eq. (6), where A_{sd} is the area of each group of the diagonal bars; f_y is the yield strength of the bars; α is angle with the horizontal axis of the beam; h is the height of the beam cross section; and the d' is the length of the clear cover. In Specimen CW-1, the nominal moment of a coupling beam was calculated using Eq. (6). However, because all longitudinal bars of the coupling beams in CW-2 are fully developed, Eq. (7) was used instead.

$$M_n = A_{sd} f_y \cos \alpha (h - 2d') \quad \text{Eq. (6)}$$

$$M_n = (A_s + A_{sd} \cos \alpha) (f_y) (0.9 \times d) \quad \text{Eq. (7)}$$

And the chord rotation at yield could be calculated using Eq. (8), where M_y is the yield moment; E_c is the secant modulus of elasticity of concrete; I_{cr} is the cracked moment of inertia of the cross section; and l_p is the plastic hinge length (the beam flexural depth/2).

$$\theta_y = \frac{M_y}{E_c I_{cr}} l_p \quad \text{Eq. (8)}$$

These calculated flexural strengths and chord rotations for each beam were used to construct the coupling beam model with the modeling parameters. Table 9 represents the results of structural calculation and application of modeling parameters, and it is used for the moment hinge property of rotation type. In this table, U is the ultimate strength point which corresponds the calculated M_n and θ_y ; L is the ductile limit point and DL can be obtained with $DY + a$; R is the residual strength point in which FR is FU multiplied by c; and X point is the maximum deformation point corresponding to $DY + b$.

Table 9 Modeling Values for Coupling Beams in CW-1 and CW-2

Specimen	Element	Point	U	L	R	X
CW-1	CB1,3	F (kN-m)	36.4	36.4	29.1	29.1
		D (rad)	0.00605	0.0361	0.0368	0.0561

	CB2	F (kN-m)	36.4	36.4	29.1	29.1
		D (rad)	0.00618	0.0362	0.0370	0.0562
	CB4	F (kN-m)	36.4	36.4	29.1	29.1
		D (rad)	0.00593	0.0359	0.0367	0.0559
CW-2	CB1,3,4	F (kN-m)	53.7	53.7	42.9	42.9
		D (rad)	0.00885	0.0389	0.0396	0.0589
	CB2	F (kN-m)	53.7	53.7	42.9	42.9
		D (rad)	0.00946	0.0395	0.0403	0.0595

4.2.2 Shear Wall Modeling

The walls of CW-1 and CW-2 were divided with Shear Wall element, and then the each wall elements were divided again with fiber unit about the horizontal cross-section of shear wall as shown in Figure 28. This figure is for a web part of the shear wall, and a flange part is also modeled with the same method.

Figure 28 - Shear Wall Fiber Model of Individual Coupled Shear Walls

Tested stress-strain relationships of materials were considered in each fiber element to reflect the nonlinear behavior of concrete and steel in the same strength of the test day. The steel material model used in this structures have the trilinear curve as shown in Figure 22, in which the yield stresses at the point FY and ultimate stresses at point FU were taken as the measured values in Table 8. The calculated values for the steel material model are shown in Table 10.

Table 11 and Table 12 show the modeling values for unconfined concrete materials in

compression and tension using the measured values in the experimental works. The calculated result for confined concretes shows in Table 13.

Table 10 Modeling Values for Steel (E = 200GPa) in CW-1 and CW-2

Specimen		Steel	FY (MPa)	FU (MPa)	DU	DX
CW-1	CB	#2 (D6)	440	505	0.0185	0.100
		#3 (D10)	525	815	0.0751	0.100
		#4 (D13)	525	795	0.0701	0.100
	Wall	#2 (D6)	440	505	0.0184	0.100
		#3 (D10)	510	770	0.0675	0.100
		#5 (D16)	465	750	0.0736	0.100
		#6 (D19)	470	750	0.0723	0.100
CW-2	CB	#2 (D6)	440	505	0.0184	0.100
		#3 (D10)	525	815	0.0751	0.100
		#4 (D13)	525	795	0.0701	0.100
	Wall	#2 (D6)	440	505	0.0184	0.100
		#3 (D10)	465	810	0.0886	0.100
		#4 (D13)	415	670	0.0658	0.100
		#5 (D16)	465	750	0.0736	0.100
		#6 (D19)	470	750	0.0724	0.100

Table 11 Modeling Values for Unconfined Concrete in Compression in CW-1 and CW-2

Specimen	Material	Element	Point	U	L	R	X
CW-1	HPFRCCs	CB1,3	F (MPa)	71.0	71.0	14.2	14.2
			D	0.00178	0.00180	0.0178	0.0250
		CB4	F (MPa)	74.0	74.0	14.8	14.8
			D	0.00182	0.00184	0.0178	0.0250
	Conventional Concrete	CB2	F (MPa)	68.0	68.0	13.6	13.6
			D	0.00174	0.00176	0.00708	0.0200
		Wall1st	F (MPa)	48.0	48.0	9.60	9.60
			D	0.00418	0.00423	0.00952	0.0200
		Wall2nd	F (MPa)	46.0	46.0	9.20	9.20
			D	0.00205	0.00207	0.00738	0.0200

		Wall3rd	F (MPa)	46.0	46.0	9.20	9.20
			D	0.00205	0.00207	0.00738	0.0200
		Wall4th	F (MPa)	66.0	66.0	13.2	13.2
			D	0.00245	0.00248	0.00779	0.0200
		Wall5th	F (MPa)	61.0	61.0	12.2	12.2
			D	0.00236	0.00238	0.00769	0.0200
CW-2	HPFRCCs	CB1,3,4	F (MPa)	72.0	72.0	14.4	14.4
			D	0.00179	0.00181	0.0178	0.0250
		Wall1st	F (MPa)	51.0	51.0	10.2	10.2
			D	0.00431	0.00436	0.0203	0.0250
		Wall2nd	F (MPa)	50.0	50.0	10.0	10.0
			D	0.00214	0.00216	0.0181	0.0250
	Conventional Concrete	CB2	F (MPa)	63.0	63.0	12.6	12.6
			D	0.00168	0.00169	0.00701	0.0200
		Wall3rd	F (MPa)	57.0	57.0	11.4	11.4
			D	0.00228	0.00230	0.00761	0.0200
		Wall4th	F (MPa)	48.0	48.0	9.60	9.60
			D	0.00209	0.00211	0.00743	0.0200
		Wall5th	F (MPa)	56.0	56.0	11.2	11.2
			D	0.00226	0.00228	0.00759	0.0200

Table 12 Modeling Values for Unconfined Concrete in Tension in CW-1 and CW-2

Specimen	Material	Element	Point	U	L	R	X
CW-1	HPFRCCs	CB1,3	F (MPa)	-3.40	-3.40	-0.567	-0.567
			D	-8.50E-05	-5.00E-03	-0.0190	-0.0200
		CB4	F (MPa)	-3.40	-3.40	-0.567	-0.567
			D	-8.40E-05	-5.00 E-03	-0.0190	-0.0200
CW-2	HPFRCCs	CB1,3,4	F (MPa)	-3.40	-3.40	-0.567	-0.567
			D	-8.50E-05	-5.00 E-03	-0.0190	-0.0200
		Wall1st	F (MPa)	-3.40	-3.40	-0.567	-0.567
			D	-29.0E-05	-5.00 E-03	-0.0190	-0.0200
		Wall2nd	F (MPa)	-3.40	-3.40	-0.567	-0.567
			D	-15.0E-05	-5.00 E-03	-0.0190	-0.0200

Table 13 Modeling Values for Confined Concrete in CW-1 and CW-2

Specimen	Element	Point	U	L	R	X
CW-1	Wall1st- outside	F (MPa)	56.4	56.4	11.3	11.3
		D	0.00454	0.00458	0.0206	0.0300
	Wall1st - inside	F (MPa)	59.0	59.0	11.8	11.8
		D	0.00464	0.00468	0.0207	0.0300
	Wall2nd	F (MPa)	51.8	51.8	10.4	10.4
		D	0.00217	0.00220	0.0182	0.0300
	Wall3rd	F (MPa)	51.8	51.8	10.4	10.4
		D	0.00217	0.00220	0.0182	0.0300
	Wall4th	F (MPa)	66.8	66.8	13.4	13.4
		D	0.00247	0.00249	0.0185	0.0300
	Wall5th	F (MPa)	72.5	72.5	14.5	14.5
		D	0.00257	0.00260	0.0186	0.0300
CW-2	Wall1st (left)	F (MPa)	52.2	52.2	10.4	10.4
		D	0.00436	0.00441	0.0204	0.0300
	Wall1st- outside	F (MPa)	56.1	56.1	11.2	11.2
		D	0.00452	0.00457	0.0206	0.0300
	Wall1st- inside	F (MPa)	57.2	57.2	11.4	11.4
		D	0.00457	0.00461	0.0206	0.0300
	Wall2nd (left)	F (MPa)	56.1	56.1	11.2	11.2
		D	0.00226	0.00229	0.0183	0.0300
	Wall2nd (right)	F (MPa)	50.8	50.8	10.2	10.2
		D	0.00215	0.00217	0.0182	0.0300
	Wall3rd	F (MPa)	63.2	63.2	12.6	12.6
		D	0.00240	0.00243	0.0184	0.0300
	Wall4th	F (MPa)	48.8	48.8	9.8	9.8
		D	0.00211	0.00213	0.0181	0.0300
	Wall5th	F (MPa)	67.3	67.3	13.5	13.5
		D	0.00248	0.00250	0.0185	0.0300

4.3 Correlationship of Modeling Results and Experimental Results

Figure 29 and Figure 30 show the result of overturning moment-wall drift relationship for CW-1 and CW-2 respectively. A solid line is the experimental result, and a dotted line is the modeling result. The proposed modeling methods well represent the overall behavior of the tested couple shear wall systems such as the strength, stiffness, and energy dissipation. The analysis well simulates strength increase observed in the measured strength of CW-2 by using HPFRCCs in comparison with CW-1. In the modeling results, although the initial strengths of both specimens are similar with the experimental result, the strengths after yielding have slightly different values. In the experiments, the slabs part of the beams at the second and forth levels were shown to develop reinforcement yielding and contribute to the coupling of the walls, which was not considered in the modeling. These could affect the smaller overturning moments. The modeling results couldn't express the strength reductions which were occurred after 2% wall drift. the sliding shear displacement of coupling beam at 2nd and 3rd floor was occurred in CW-1 during 2% wall drift and shear fail of the wall in CW-2 at -2.5% wall drift.

The stiffnesses of the tested specimen were well represented by the modeling results, and energy dissipations were also well fitted with somewhat exaggeration by the overestimated strengths after 2% wall drift. The amount energy degradation is related with the stiffness degradation in the hysteresis. The reduction of stiffness of walls can be reflected with the application of energy dissipation factors, and the shape of hysteresis loop is controlled by the unloading behavior. In this modeling, the minimum unloading elastic range is used.

Figure 29 - Overturning Moment vs. Wall drift for CW-1

Figure 30 - Overturning Moment vs. Wall drift for CW-2

Stain gauges were placed on the reinforcement throughout the CW-1 and CW-2, and this can be compared through the steel material strain capacity in wall and beam element result. In case of CW-1, the yield strains prior to 0.5% wall drift were observed in the all coupling beams and first floor of wall. The most of the first floor reinforcements in walls were yielded at 1.0% wall drift, and additional diagonal reinforcements in beams were also yielded in this drift. After 1.0% to end of test, the rest of flexure reinforcements in first floor wall almost yielded. The analyses well simulated that the all coupling beams first reached the yield moment at about 0.5% wall drift as shown in Figure 31, and the walls developed yield strains at the partial first story prior to 0.5% wall drift. The yield stains of all the first floor walls were observed during the 1.25% wall drift cycle.

By the experimental work of CW-2, the first yield strain was measured at the reinforcements in all coupling beams and first floor wall prior to 0.5% wall drift like as CW-1. When the CW-2 was pushed with 1.0% wall drift, the yield strains throughout the first floor walls were occurred. In contrast with CW-1, the horizontal reinforcements in the first floor wall were yielded prior to end of the test. This means that after ductile flexure behavior, the shear failure at approximately 2.5% wall drift. In the results of the CW-2 modeling, the yielding of coupling beams (Figure 32) and first partial floor walls were well modeled prior to 0.5% wall drift. When the wall drift was 1.25%, all first floor walls reached the yield strains. However, because the model of CW-2 was assumed to flexure behavior, the shear failure at 2.5% wall drift shown in the experiment could not be simulated.

Figure 31 - Coupling Beams of CW-1

Figure 32 - Coupling Beams of CW-2

Chapter 5. Analysis of a Tall Building with Coupled Shear Walls

5.1 Example Structures

The previous coupled wall examples were used to assess the proposed modeling methods. The purpose of the modeling in this chapter is to apply the modeling methods to a tall building and examine the behavior of the buildings depending on the use of the HPFRCs. The actual residential 56-story building (TB-1 and TB-2) was selected for nonlinear analysis using the proposed modeling methods. Figure 33 shows the plane view of this tall building and the labels are represented for the structural components of a core wall system. In the seismic design of building frame systems, the shear core is assumed to resist all of the lateral loads. Therefore, only the core wall system was modelled for the efficient analysis and the stiffness and strength of the columns were ignored to examine the response of the tall building under lateral loads.

Figure 33 - Plane View of the Tall Building

The core system consisted of 3-type coupling beams and 16-types of reinforced concrete walls, and the average story height is 3m. The cross sections of coupling beams in the tall buildings are shown in Figure 34. The all of the horizontal bars used in coupling beams are D19, and the all of stirrups used in coupling beams are D13 with the different spacing; 100mm for (a), (b), and (c), 150mm for (d), (e), and (g), and 250mm for (f).

Figure 34 - Cross Sections of CBs in Tall Buildings (unit: mm)

Table 14 shows the reinforcement of the walls, where t_w means the thickness of each walls, V,Bar is the information of vertical reinforcement, H,Bar is the information of horizontal reinforcement, and E,Bar is the information of the vertical reinforcement at the both end zones of the wall (Figure 35).

The concrete strengths for the modeling are 50MPa from 1F to 10F, 45MPa from 11F to 20F, and 35MPa from 21F to 56F for vertical members in the example buildings. The concrete strengths of horizontal members are assumed 1.4 times smaller than the materials of vertical members. The yield strengths have 400MPa for lower than D13 steel, and 600MPa for more than D16 steel. While the all structure members of the TB-1 are made of RC, the TB-2 has all HPFRCCs coupling beams and HPFRCCs walls from bottom to 14th floor (1/4 stories of the wall to cover the plastic hinge enough).

Figure 35 - Cross Section of Walls in Tall Buildings

Table 14 Reinforcement of Walls in TB-1 and TB-2 (unit: mm)

CW1					CW6				
Strory	t _w	V,Bar	H,Bar	E,Bar	Strory	t _w	V,Bar	H,Bar	E,Bar
51F~56F	600	D13 @300	D13 @200	4-D19	50F~56F	400	D10 @300	D13 @300	4-D13
49F~50F	600	D16 @250	D16 @250	4-D19	33F~49F	550	D10 @200	D13 @200	4-D13
2F~48F	600	D13 @300	D13 @200	4-D19	31F~32F	550	D16 @200	D16 @200	4-D19
1F	1000	D16 @300	D16 @190	4-D19	29F~30F	550	D16 @250	D16 @250	4-D19
CW1A					4F~28F	550	D10 @200	D13 @200	4-D13
Strory	t _w	V,Bar	H,Bar	E,Bar	2F~3F	550	D16 @300	D13 @200	4-D19
51F~56F	600	D13 @250	D13 @200	4-D16	1F	900	D19 @200	D19 @250	4-D22
49F~50F	600	D16 @300	D16 @250	4-D19	CW6A				
29F~48F	600	D13 @300	D13 @200	4-D16	Strory	t _w	V,Bar	H,Bar	E,Bar
27F~28F	600	D16 @200	D16 @250	4-D19	50F~56F	550	D10 @200	D13 @200	4-D13
3F~26F	600	D13 @300	D13 @200	4-D16	33F~49F	550	D10 @200	D13 @200	4-D13
1F~2F	1000	D16 @300	D16 @190	4-D19	31F~32F	550	D16 @200	D16 @200	4-D19

CW2					29F~30F	550	D16 @250	D16 @250	4-D19
Strory	t_w	V,Bar	H,Bar	E,Bar	4F~28F	550	D10 @200	D13 @200	4-D13
55F~56F	600	D13 @250	D13 @200	4-D16	2F~3F	550	D16 @300	D13 @200	4-D19
32F~54F	600	D13 @300	D13 @200	4-D16	1F	900	D19 @250	D19 @250	6-D22
30F~31F	600	D16 @250	D16 @250	4-D19	CW7				
2F~29F	600	D13 @300	D13 @200	4-D16	Strory	t_w	V,Bar	H,Bar	E,Bar
1F	1000	D16 @300	D16 @190	4-D19	50F~56F	400	D10 @300	D13 @300	4-D13
CW3					34F~49F	550	D10 @200	D13 @200	4-D13
Strory	t_w	V,Bar	H,Bar	E,Bar	28F~33F	550	D16 @250	D16 @250	4-D19
32F~56F	600	D13 @300	D13 @200	4-D16	6F~27F	550	D10 @200	D13 @200	4-D13
30F~31F	600	D16 @250	D16 @250	4-D19	2F~5F	550	D16 @250	D16 @250	6-D19
2F~29F	600	D13 @300	D13 @200	4-D16	1F	900	D19 @100	D19 @250	10-D22
1F	1000	D16 @300	D16 @190	4-D19	CW8				
CW3A					Strory	t_w	V,Bar	H,Bar	E,Bar
Strory	t_w	V,Bar	H,Bar	E,Bar	54F~56F	550	D16 @250	D13 @200	4-D13
32F~56F	600	D13 @300	D13 @200	4-D16	32F~53F	550	D10 @200	D13 @200	4-D16
30F~31F	600	D16 @300	D16 @250	4-D19	30F~31F	550	D16 @250	D16 @250	4-D19
19F~29F	600	D13 @300	D13 @200	4-D16	7F~29F	550	D10 @200	D13 @200	4-D13
17F-1F~18F	600	D16 @300	D16 @250	4-D19	5F~6F	550	D16 @250	D16 @250	4-D19
2F~17F	600	D13 @300	D13 @200	4-D19	2F~4F	550	D16 @250	D16 @250	6-D19

1F	1000	D16 @250	D16 @190	6-D19	1F	900	D19 @100	D19 @250	10-D22
CW4					W1				
Strory	t_w	V,Bar	H,Bar	E,Bar	Strory	t_w	V,Bar	H,Bar	E,Bar
32F~56F	600	D13 @300	D13 @200	4-D16	56F	400	D13 @200	D13 @300	4-D16
30F~31F	600	D16 @250	D16 @250	4-D19	31F~55F	400	D10 @300	D13 @300	4-D13
25F~29F	600	D13 @300	D13 @200	4-D16	17F-1F~30F	400	D13 @250	D13 @250	4-D16
2F~24F	600	D16 @250	D16 @250	4-D22	4F~17F	400	D10 @200	D13 @300	4-D13
1F	1000	D19 @200	D19 @200	6-D22	2F~3F	400	D16 @300	D13 @300	4-D19
CW4A					1F	400	D19 @150	D16 @300	6-D22
Strory	t_w	V,Bar	H,Bar	E,Bar	W2				
56F	600	D16 @250	D16 @250	4-D19	Strory	t_w	V,Bar	H,Bar	E,Bar
50F~55F	600	D13 @300	D13 @200	4-D16	31F~56F	250	D10 @450	D10 @280	4-D13
31F~49F	600	D16 @250	D16 @250	4-D19	29F~30F	250	D13 @200	D10 @200	4-D16
29F~30F	600	D19 @120	D16 @250	4-D22	20F~28F	250	D10 @450	D10 @280	4-D13
2F~28F	600	D16 @250	D16 @250	4-D19	18F~19F	250	D10 @200	D10 @200	4-D13
1F	1000	D19 @250	D19 @190	6-D22	8F~17F-1F	250	D10 @450	D10 @280	4-D13
CW5					2F~7F	250	D10 @200	D10 @200	4-D13
Strory	t_w	V,Bar	H,Bar	E,Bar	1F	250	D13 @300	D10 @220	4-D16
50F~56F	600	D10 @250	D13 @250	4-D16	W2A				
31F~49F	600	D13 @300	D13 @190	4-D16	Strory	t_w	V,Bar	H,Bar	E,Bar

29F~30F	600	D16 @250	D16 @250	4-D19	31F~56F	350	D10 @300	D10 @200	4-D13																					
2F~28F	600	D13 @250	D13 @190	4-D19	30F	350	D13 @200	D13 @250	4-D16																					
1F	1000	D16 @250	D16 @250	4-D22	2F~29F	350	D10 @300	D10 @200	4-D16																					
※ Size of Reinforcements <table><tr><th>Bar</th><th>Diameter (mm)</th><th>Area (mm²)</th></tr><tr><td>D10</td><td>9.53</td><td>71.3</td></tr><tr><td>D13</td><td>12.7</td><td>126.7</td></tr><tr><td>D16</td><td>15.9</td><td>198.6</td></tr><tr><td>D19</td><td>19.1</td><td>286.5</td></tr><tr><td>D22</td><td>22.2</td><td>387.1</td></tr><tr><td>D25</td><td>25.4</td><td>506.7</td></tr></table>					Bar	Diameter (mm)	Area (mm ²)	D10	9.53	71.3	D13	12.7	126.7	D16	15.9	198.6	D19	19.1	286.5	D22	22.2	387.1	D25	25.4	506.7	1F	350	D13 @200	D13 @250	4-D19
					Bar	Diameter (mm)	Area (mm ²)																							
					D10	9.53	71.3																							
					D13	12.7	126.7																							
					D16	15.9	198.6																							
					D19	19.1	286.5																							
					D22	22.2	387.1																							
					D25	25.4	506.7																							
					W3																									
					Strory	t _w	V,Bar	H,Bar	E,Bar																					
31F~56F	250	D10 @450	D10 @280	4-D13																										
30F	250	D10 @200	D10 @200	4-D16																										
2F~29F	250	D10 @450	D10 @280	4-D16																										
1F	250	D13 @300	D13 @250	4-D19																										

5.2 Modeling Processes

In this example of a tall building, the modeling process for coupling beams and shear walls was same with the previous example.

5.2.1 Coupling Beam Modeling

The coupling beams are expected to develop flexural yielding at their both ends. Therefore, the model of the coupling beam also consists of a ‘Moment Hinge of Rotation Type’ on each end and an ‘Elastic Beam Section’ as same with the previous example. The elastic beam section also has the $0.20E_cI_g$ effective elastic stiffness to include slip/extension effects. The calculated nominal moment and yield rotation values were used for the moment hinge model with the ASCE/SEI 41-06 parameter (Table 4) to construct the backbone curves (Figure 18). The flexural strength of the conventional reinforced beams is calculated using Eq. (9), where A_s is the area of tension bars; f_y is the yield strength of the bars; d is the distance from extreme compression fiber to centroid of longitudinal tension reinforcements, a is the depth of equivalent rectangular stress block in compression zone of the cross section, and $(d-a/2)$ is the moment arm. And the chord rotation at yield could be calculated using Eq. (6). Table 15 shows modeling values for coupling beams that used the results of structural calculation and application of modeling parameters in the same way of the previous example.

$$M_n = A_s f_y (d - a/2) \quad \text{Eq. (9)}$$

Table 15 Modeling Values for Coupling Beams in TB-1 and TB-2

Specimen	Element	Floor	Point	U	L	R	X
TB-1	CB1-1	1F~10F	F (kN-m)	1370	1370	687	687
			D (rad)	0.00890	0.0289	0.0295	0.0439
		11F~17F	F (kN-m)	1360	1360	678	678
			D (rad)	0.00936	0.0294	0.0299	0.0444
		18F~20F	F (kN-m)	1190	1190	596	596
			D (rad)	0.00814	0.0281	0.0287	0.0431
		21F~40F	F (kN-m)	1160	1160	578	578
			D (rad)	0.00917	0.0292	0.0298	0.0442
		41F~56F	F (kN-m)	996	996	498	498
			D (rad)	0.00778	0.0278	0.0283	0.0428
	CB1-2	1F~10F	F (kN-m)	1370	1370	1030	1030
			D (rad)	0.00545	0.0305	0.0311	0.0555

		11F~17F	F (kN-m)	1360	1360	1020	1020
			D (rad)	0.00573	0.0307	0.0313	0.0557
		18F~20F	F (kN-m)	1190	1190	894	894
			D (rad)	0.00499	0.0300	0.0306	0.0550
		21F~40F	F (kN-m)	1160	1160	867	867
			D (rad)	0.00562	0.0306	0.0312	0.0556
		41F~56F	F (kN-m)	996	996	747	747
			D (rad)	0.00477	0.0298	0.0304	0.0548
	CB1-3	1F~10F	F (kN-m)	1370	1370	687	687
			D (rad)	0.00782	0.0278	0.0284	0.0428
		11F~17F	F (kN-m)	1360	1360	678	678
			D (rad)	0.00823	0.0282	0.0288	0.0432
		18F~20F	F (kN-m)	1190	1190	894	894
			D (rad)	0.00715	0.0322	0.0328	0.0572
		21F~40F	F (kN-m)	1160	1160	867	867
			D (rad)	0.00806	0.0331	0.0337	0.0581
		41F~56F	F (kN-m)	996	996	498	498
			D (rad)	0.00684	0.0268	0.0274	0.0418
	CB2-1	1F~10F	F (kN-m)	801	801	400	400
			D (rad)	0.00786	0.0279	0.0284	0.0429
		11F~17F	F (kN-m)	792	792	396	396
			D (rad)	0.00827	0.0283	0.0288	0.0433
		18F~20F	F (kN-m)	612	612	459	459
			D (rad)	0.00630	0.0313	0.0319	0.0563
		21F~30F	F (kN-m)	598	598	449	449
			D (rad)	0.00710	0.0321	0.0327	0.0571
		31F~56F	F (kN-m)	520	520	260	260
			D (rad)	0.00611	0.0261	0.0266	0.0411
	CB2-2	1F~10F	F (kN-m)	801	801	400	400
			D (rad)	0.0140	0.0340	0.0347	0.0490
		11F~17F	F (kN-m)	792	792	396	396
			D (rad)	0.0148	0.0348	0.0355	0.0498
		18F~20F	F (kN-m)	612	612	306	306
			D (rad)	0.0113	0.0312	0.0319	0.0462

		21F~30F	F (kN-m)	598	598	299	299
			D (rad)	0.0127	0.0327	0.0333	0.0477
		31F~56F	F (kN-m)	520	520	260	260
			D (rad)	0.0109	0.0309	0.0315	0.0459
	CB3	1F~10F	F (kN-m)	397	397	198	198
			D (rad)	0.00723	0.0272	0.0278	0.0422
		11F~20F	F (kN-m)	392	392	294	294
			D (rad)	0.00760	0.0326	0.0333	0.0576
		21F~56F	F (kN-m)	380	380	285	285
			D (rad)	0.00856	0.0336	0.0342	0.0586
TB-2	CB1-1	1F~10F	F (kN-m)	1650	1650	824	824
			D (rad)	0.0107	0.0578	0.0590	0.0878
		11F~17F	F (kN-m)	1630	1630	813	813
			D (rad)	0.0112	0.0587	0.0599	0.0887
		18F~20F	F (kN-m)	1430	1430	715	715
			D (rad)	0.00981	0.0563	0.0574	0.0863
		21F~40F	F (kN-m)	1390	1390	693	693
			D (rad)	0.0108	0.0583	0.0595	0.0883
		41F~56F	F (kN-m)	1200	1200	597	597
			D (rad)	0.00929	0.0556	0.0567	0.0856
	CB1-2	1F~10F	F (kN-m)	1650	1650	1240	1240
			D (rad)	0.00657	0.0609	0.0621	0.111
		11F~17F	F (kN-m)	1630	1630	1220	1220
			D (rad)	0.00684	0.0615	0.0627	0.111
		18F~20F	F (kN-m)	1430	1430	1073	1073
			D (rad)	0.00601	0.0600	0.0612	0.110
		21F~40F	F (kN-m)	1390	1390	1040	1040
			D (rad)	0.00661	0.0612	0.0625	0.111
		41F~56F	F (kN-m)	1190	1190	896	896
			D (rad)	0.00569	0.0595	0.0607	0.110
	CB1-3	1F~10F	F (kN-m)	1650	1650	824	824
			D (rad)	0.00942	0.0556	0.0568	0.0856
		11F~17F	F (kN-m)	1630	1630	813	813
			D (rad)	0.00981	0.0565	0.0576	0.0865

		18F~20F	F (kN-m)	1430	1430	1070	1070
			D (rad)	0.00862	0.0643	0.0656	0.114
		21F~40F	F (kN-m)	1390	1390	1040	1040
			D (rad)	0.00948	0.0661	0.0674	0.116
		41F~56F	F (kN-m)	1190	1190	597	597
			D (rad)	0.00817	0.0537	0.0548	0.0837
	CB2-1	1F~10F	F (kN-m)	961	961	480	480
			D (rad)	0.00956	0.0557	0.0568	0.0857
		11F~17F	F (kN-m)	951	951	475	475
			D (rad)	0.00997	0.0565	0.0577	0.0865
		18F~20F	F (kN-m)	734	734	551	551
			D (rad)	0.00770	0.0626	0.0638	0.113
		21F~30F	F (kN-m)	718	718	538	538
			D (rad)	0.00854	0.0642	0.0655	0.114
		31F~56F	F (kN-m)	624	624	312	312
			D (rad)	0.00741	0.0522	0.0533	0.0822
	CB2-2	1F~10F	F (kN-m)	961	961	480	480
			D (rad)	0.0171	0.0681	0.0694	0.0981
		11F~17F	F (kN-m)	951	951	475	475
			D (rad)	0.0178	0.0695	0.0709	0.0995
		18F~20F	F (kN-m)	734	734	367	367
			D (rad)	0.0137	0.0625	0.0637	0.0925
		21F~30F	F (kN-m)	718	718	359	359
			D (rad)	0.0152	0.0654	0.0667	0.0954
		31F~56F	F (kN-m)	624	624	312	312
			D (rad)	0.0132	0.0618	0.0630	0.0918
	CB3	1F~10F	F (kN-m)	476	476	238	238
			D (rad)	0.00878	0.0545	0.0555	0.0845
		11F~20F	F (kN-m)	471	471	353	353
			D (rad)	0.00915	0.0652	0.0665	0.115
		21F~56F	F (kN-m)	456	456	342	342
			D (rad)	0.0101	0.0671	0.0685	0.117

5.2.2 Shear Wall Modeling

The ‘Shear Wall Element’ was used to model the slender shear wall components, because the flexural failures of the specimens were expected due to the large slenderness ratio. An elastic shear material model was assumed. After the walls were divided with this Shear Wall Element depending on the story height, the each elements of wall was divided again with Fiber Element unit. Table 16 and Table 17 show the stress-strain relationship of the steel and the concrete each for the ‘Steel Fiber Element’ and ‘Concrete Fiber Element’. The stress-strain relationship of HPFRCCs used in the Lequesne (2011) was considered in the analysis of TB-2, and it is shown in Table 18.

Since the walls are substantially slender, shear deformations may not be very significant. The elastic modulus E_c of concrete can be assumed with $4730 \times \sqrt{f'_c}$ (MPa), and the shear modulus G_c can be corresponding to $E_c/2(1 + \nu)$ with the poison ratio 0.25. However, this value may be much too large when there is significant cracking. For this example one quarter of shear modulus $G_c/4 = 3345$ MPa (1~10F), 3173 MPa (11~20F), and 2798 MPa (21~56F) are used in shear material models.

In shear wall elements, the behavior for out-of-plane bending is assumed to be elastic. Because the walls are much stiffer in-plane than out-of-plane, the stiffness contribution of the out-of-plane bending is only a small part of the total stiffness. For the out-of-plane cross sections of the wall in this modeling, the real wall thickness was used and one quarter of the modulus for concrete was used for Young's modulus to reflect the stiffness reduction when the concrete cracks.

Table 16 Modeling Values for Steels in TB-1 and TB-2 (E = 200GPa)

Steel	FY (MPa)	FU (MPa)	DU	DX
#2 (D6)	440	505	0.0185	0.100
#3 (D10)	525	815	0.0751	0.100

Table 17 Modeling Values for Concrete in TB-1 and TB-2

Material	Element	Story	Point	U	R	X
Normal Concrete	Wall	1F~10F	F (MPa)	50.0	10.0	10.0
			D	0.00214	0.00747	0.0200
		11F~20F	F (MPa)	45.0	9.00	9.00
			D	0.00203	0.00736	0.0200
		21F~56F	F (MPa)	35.0	7.00	7.00
			D	0.00179	0.00712	0.0200

	Beam	1F~10F	F (MPa)	35.7	7.14	7.14
			D	0.00842	0.0138	0.0200
		11F~20F	F (MPa)	32.1	6.43	6.43
			D	0.00799	0.0133	0.0200
		21F~56F	F (MPa)	25.0	5.00	5.00
			D	0.00705	0.0124	0.0200

Table 18 Modeling Values for HPFRCCs in TB-2

Compression					
Material	Story	Point	U	R	X
HPFRCCs	1F~10F	F (MPa)	50.0	10.0	10.0
		D	0.00150	0.0175	0.0200
	11F~14F	F (MPa)	45.0	9.00	9.00
		D	0.00142	0.0174	0.0200
Tension					
HPFRCCs	1F~14F	F (MPa)	3.40	0.570	0.570
		D	0.00405	0.0190	0.0200

5.2.3 Floor Slab and Columns

The floor slab is modeled as a rigid diaphragm using the slaving constraints, and the masses are lumped at each floor. The center of mass and the gravity load on the each wall from one floor slab were calculated using SAP2000 software. After modeling the one story of the tall building example through the SAP2000, the approximate results of translational and rotational masses (Table 19) could be applied to the center of mass.

Table 19 Results of Lumped Masses at each floor of Tall Building

Area	A (m ²)	692.2
Moment of inertia	I22 (m ⁴)	48100
	I33 (m ⁴)	61100
Dead Load	(kN/m ²)	20.7
Translational weight	(kN)	14300
Rotational weight	(kN-m ²)	2260000

	(kN-mm ²)	2.26E+12
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Although P-Δ effects can be considered in wall elements, this model used the "P-Δ column" elements. When the local buckling failure of the wall is occurred by the weak out-of-plane strength of wall, the structure model can no longer support the gravity loads, and the analysis do not have the solution. To prevent this error, the P-Δ column was considered at the center of mass with elastic 'Simple Bar' elements. P-Δ effects are considered through only this column. The gravity loads of each floor were applied this column with the slaving constraint. The mechanism of the P-Δ column is shown in the Figure 36. When the axial load P is applied to the column and there is horizontal load H in this structure, the effective shear force $P\Delta/h$ is occurred at this column. By the equilibrium, the shear force in the wall sections is equal to $H + P\Delta/h$. In addition, the accidental torsion for P-Δ effects can be modeled with the other P-Δ column at the eccentric location.

Figure 36 - Mechanism of P- Δ Column

5.2.4 Loads

For the gravity loads modeling, the dead loads (DL) include the self-weight of the floor slabs which is calculated with the weight density of materials and superimposed loads (1kN/m²). The design live load (LL) is 2kN/m² on the slabs. All floors were assumed to have same dead and live load, and the gravity loads of slabs were applied at the center of mass as mentioned above. However, the wall has different behavior depending on the existence of the vertical compression due to the delay of crack. Because the floor slabs of this example are not modeled, properly estimated gravity loads from slab to wall should be applied directly to the nodes of the core wall system. SAP2000 was used again

to calculate the reaction forces which are acting on the each nodes of the core wall system under a unit load 1kN/m^2 on the slab areas. These loads were scaled with $1.2\text{DL}+1.0\text{LL}$ when the load pattern was used in gravity load case for the PERFORM-3D modeling. Therefore, the gravity loads on walls consist of dead and live loads from the floor slabs and the wall self-weight. The self-weight of walls were applied by the data input of the weight density during the wall compound modeling process. Because the gravity loads about slabs were applied twice at the P- Δ column and walls, the result of P- Δ column should be excluded.

A vertical distribution of push-over load is fixed during the analysis as different with the actual earthquake, and it has a great effect on the structure behavior. This distribution can be based on inertia forces corresponding to displacement distributions over the building height in PERFORM-3D software. In this study, there are 15 load cases of static push-over load up to 10% drift to compare the effect of vertical distributions. The vertical distribution was modeled as depending on the uniform shape, triangular shape, and vertical distribution factor for the pseudo-lateral force as shown in Figure 37. Eq. (10) is for the vertical distribution factor C_{vw} , where $w_{x \text{ or } i}$ is portion of the effective seismic weight located on the floor level x or i , and the $h_{x \text{ or } i}$ is the height from the base to the floor level x or i . This software limits the number of input with 6 pattern, and Table 20 shows the calculated this factor along the floor. The other 9 load cases were modeled in accordance with each mode shape of the structure from first order to ninth order.

A cyclic push-over load is also applied with the vertical distribution factor C_{vw} for the pseudo-lateral force, and cyclic displacement reversals are same with Figure 27.

Figure 37 - Vertical Distributions for Static Push-Over Loads

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad \text{Eq. (10)}$$

Table 20 Vertical Distribution Factor C_{vx} in TB-1 and TB-2

Floor	h_i (mm)	C_{vx}
0	0	0
10	36100	0.0192
20	72400	0.0760
30	105700	0.1604
40	135800	0.2632
56	184300	0.4812

In this study, a dynamic earthquake load case was applied to compare and observe the nonlinear seismic behavior. For this nonlinear time history analysis, a ground motion considered for an earthquake that is El Centro earthquake at California (1940). This typical earthquake is known as the first recorded major earthquake and the time history of acceleration records is shown in Figure 38. The total duration is 29 seconds and the maximum acceleration is 0.32g. This earthquake load analysis was modeled with 5% Rayleigh damping.

Figure 38 - El Centro Earthquake

5.3 Analysis Results

5.3.1 Mode Property

When the masses of the tall buildings are defined, PERFORM-3D calculates the mode properties which are about mode shape and periods. The mode shapes are calculated to characterize displacement patterns in elastic state. In the push-over analysis, the structures commonly have fundamental mode dominant behavior. The effective mass factor represents the significant of the each mode, and the mode with higher effective mass factors has the relatively higher participation to the structural response under the seismic load. Mode shapes of TB-1 and TB-2 is shown in Figure 39 and Figure 40 respectively, and the each period for these models is expressed in Table 21 and Table 22. TB-1 and TB-2 have large torsional mode shapes due to the irregularity and both models have similar mode shapes except the 3rd, 5th, and 9th modes. The 5th and 9th mode shapes of TB-2 behave in the opposite direction with TB-1, and the 3rd mode shape of TB-2 seems to have smaller torsional behavior than that of TB-1. The periods and effective mass factors of each mode are almost similar in both models.

Figure 39 - Mode Shape of TB-1

Table 21 Mode Periods of TB-1

Mode	Period (s)	Effective Mass Factor	
		H1	H2
1	7.42	0.39	2.23E-05
2	6.45	0.0473	0.546

3	6.14	0.222	0.121
4	2.06	0.0110	4.45E-04
5	1.57	0.154	0.0198
6	1.51	0.0173	0.161
7	1.10	0.00184	1.92E-04
8	0.73	0.00168	8.34E-05
9	0.70	0.0349	0.0270

Figure 40 - Mode Shape of TB-2

Table 22 Mode Periods of TB-2

Mode	Period (s)	Effective Mass Factors	
		H1	H2
1	7.41	0.390	2.59E-05
2	6.45	0.0491	0.541
3	6.15	0.221	0.125
4	2.06	0.0104	4.41E-04
5	1.57	0.155	0.0191
6	1.51	0.0167	0.162
7	1.10	0.00171	1.84E-04
8	0.73	0.00163	4.44E-05
9	0.70	0.0350	0.0271

5.3.2 Push-Over Load

This section shows the results when the tall building is under various push-over loads in H1 direction. The strain gages were placed at the tips of the each wall element to check the yield of vertical reinforcement and compression level when concrete begins to lose strength. When the hinges form, a substantial yield of the reinforcement would be measured.

5.3.2.1 Story Drift

Figure 41, Figure 42, and Figure 43 show the interstory drift result of the two models depending on the vertical patterns of the static push-over load case. When both models are under the uniform and triangular load distribution up to 10% roof drift, they behave with almost same story drift way. The large story drift was occurred in 2nd and 3rd floors. However, the result of TB-2 under the C_{vx} factor pattern shows different with the other profiles of story drift. The large story drift occurred at 15th floor, and this means the weak story was shifted by using HPFRCCs.

Figure 41 - Story Drift under Static Push-Over Loads with Uniform Distribution

Figure 42 - Story Drift under Static Push-Over Loads with Triangular Distribution

Figure 43 - Story Drift under Static Push-Over Loads with C Factor Distribution

Figure 44 shows the interstory drift results at 10% roof drift depending on the vertical patterns that were modeled with the mode shapes of TB-1 and TB-2 from 1st mode to 9th mode. The results from first to fourth mode have similar. The relatively large story drifts were developed at 2nd and 3rd floors of both TB-1 and TB-2 in positive H1 direction. The results with 5th mode and 6th mode vertical distribution are also similar, and both TB-1 and TB-2 have large story drifts at 34th and 35th floors in negative H1 direction. When the vertical distribution was 7th mode shape, the result show large story drift occurred from 23rd floor to 25th floor in negative H1 direction. In case of 8th mode distribution, the sustainable story drift was developed from 41st floor to 44th floor in positive H1 direction. The result with 9th mode distribution also shows the large story drift at the upper part of the structures from 40th floor to 43rd floor in positive H1 direction.

Figure 45 shows the story drift of TB-1 and TB-2 under the cyclic load with C_{vx} factor distribution. At the initial cycles with 0.25% roof drift, the two models behaved with similar story drift. When the target roof drift of TB-1 reached at 1.5% drift, the story drift of 5th floor began to rapidly increase. At this cycle, the 4th and 5th floors of CW3 and CW3A wall showed yield stains in tension and the CW6A strain gage at the 4th floor reached at the DX state in compression. From this to 3.0% roof drift cycle, the TB-1 behaved with large story drift at 4th and 5th floor. The final result of this model shows that the 2nd and 3rd floor had large story drift in H2 direction which suddenly began from the -3.0% target drift cycle as shown in the figure. At the final cycle, the compression strain gages reached DX at the 2nd floor of CW1 and CW1A and it would have made the large H2 story drift.

In the case of TB-2, the weak story occurred at 15th and 16th floors from 1% roof drift. The a number of 15th floor strain gages indicated yield strains in tension and several compression strains of the walls reached at the DX level. The story drift in H2 direction began to sharply increased at 15th and 16th floors when 3% roof drift, and t at 3.5% roof drift, this model finally indicated a brittle fracture behavior and was stopped as the H2 story drift reached to 10%. These would be related to the measurement of all strain gages at 15th floor without the CW8 that indicated at the maximum deformation point.

TB-1 had a plastic hinge in lower stories and the use of HPFRCCs moved it to upper stories which used the normal concrete. At the later stage, the large displacement in H2 direction was shown in the both models even though the load direction is just H1.

Figure 44 - Story Drift under Static Push-Over Loads with Mode Shape Distribution

Figure 45 - Story Drift under Cyclic Push-Over Loads with C-factor Distribution

5.3.2.2 Base Shear

Figure 46 shows the relationship between base shear and roof drift when the static push-over loads are applied without the gravity load. U, T, and C in the legend of this figure represent the vertical patterns of lateral loads; uniform, triangular, and C_{vx} factor distribution respectively. TB-1 and TB-2 models with uniform distribution had largest base shear and stiffness, followed by the triangular distribution cases. TB-2 model indicate the increased base shear and stiffness responses compared with TB-1. However, a relatively larger strength reduction occurred in TB-2 model at lower roof drift than TB-1 case. The reason of a little strength reduction in TB-1 model was that the compressive stain at 5th floor of CW6A exceeded the defined maximum deformation that means the corresponding fiber element could not resist applied loads anymore. It also occurred at the 15th floor of CW6A in TB-2 model at the earlier roof drift.

Figure 47 shows the base shear results when TB-1 is subjected to the static push-over loads up to 10% roof drift without gravity and the load has vertical distributions with mode shapes. The relations of the base shear and roof drift from 1st mode to 3rd mode distribution show same result and the strength and stiffness are much smaller than that of the other modes. The 8th mode case has the largest stiffness, and the 9th mode has the largest strength.

The same load cases were applied to TB-2, and the results about the base shear are shown in Figure 48. The results of TB-2 from 1st mode to 3rd mode distribution are the same and the strength and stiffness are even smaller than the values from the other load cases such as TB-1 results. The result with 4th mode distribution shows the sustainable large strength in a moment before 1% roof drift. And the largest stiffness was modeled when 8th mode distribution case. The overall strength and stiffness of TB-2 were not much different from TB-1 except for 4th mode case in spite of the use of HPFRCCs. The base shear result under the load with C_{vx} factor distribution is similar with the result with 1st mode to 3rd mode distribution.

Figure 46 - Base Shear under Static Push-Over Load without Gravity: Uniform, Triangular and C Factor Distributions

Figure 47- Base Shear under Static Push-Over Load of without Gravity: Mode Shape Distribution of TB-1

Figure 48- Base Shear under Static Push-Over Load of without Gravity: Mode Shape Distribution of TB-2

As an existence of the gravity load, the results related to base shear have a lot of difference (Figure 49). The maximum base shear with gravity was about 3 times higher than the maximum strength without gravity. The substantial strength reductions are most notable feature in this figure. The initial strength reduction of TB-1 models was particularly influenced by yield of CB1 elements adjoining CW1 and CW2 in upper stories, the yielding of steel fibers of CW6 in lower stories, and the concrete fibers of CW6A passing the ultimate strength point in lower stories. In case of TB-2 model, the primary causes of the initial strength reduction were the yielding of CB1 beams adjoining CW1 and CW2 in upper stories and the yielding of steel fibers in CW3 at the 2nd floor for uniform and triangular distribution and at the 15th floor for C_{vx} factor distribution. Accordingly, the gravity load must be considered in the building modeling for more accurate analysis.

When the vertical distributions of lateral loads are mode shapes with gravity, the base shear results of TB-1 are developed in Figure 50. The same results are shown in the first three modes and these results are similar with the result of C_{vx} factor distribution. The yielding of CB1 beams in upper stories caused the initial yielding of the structure and the yields of steel in CW6 and maximum strength level of concrete fibers in CW6A in lower parts followed the initial behavior. And then, the nonlinear behavior occurred in all CB1 and lower wall elements as the roof drift increased. The 4th mode case shows the increased strength and stiffness than the lower order mode case. The initial yields occurred at lower part of CB1 beams, CW6 and CW6A wall elements. As the analysis

progressed the nonlinear behavior was developed at the wall elements in the middle stories and all beams elements. In case of 5th mode distribution, the result reached at the maximum strength level as CW1 elements in upper stories and CW6A in 33rd floor yielded. The strength reductions were progressed by the nonlinear behavior of wall elements in middle stories. And the structures were failed in the negative H1 direction at these stories. The similar behavior was shown in 6th mode shape case. When the load with 7th mode shape distribution was applied, the result of TB-1 shows the largest strength in early roof drift. The initial nonlinear behavior was due to the yielding of CW1-1L in upper and lower stories, the yields of steel fibers at CW6A, and the excess of ultimate compressive strength point of CW6 from 21st floor to 26th floor. The failures of CB1 beams from 21st floor occurred and the wall elements failed in the middle stories. The 8th mode case shows the main nonlinear behavior at the upper stories around 40th floor wall and beam elements in positive H1 direction, and the 9th mode case also had the main local behavior at around 40th floor with higher strength than the 8th mode result.

Figure 51 demonstrates the base shear result of TB-2 according to load case with the mode shape distribution after gravity load was applied. The first three mode cases show a similar behavior that have the initial yield by the yields of steel fibers in CW6 at 15th floor. The following nonlinear behavior occurred at CB1 beams and the wall elements around 15th floor and lower parts. The main reason of the final failure was the fails of the wall elements at 2nd floor. The 4th mode case has the increase strength and stiffness values. The initial nonlinear behavior of the model is due to the yield of beam elements, the yield of CW6 elements, and the passing ultimate strength of concrete fiber in CW6A at lower stories. After the progressing nonlinear behavior of lower wall elements, the continuous nonlinear behavior occurred at wall elements around 30th floor and all CB1 elements. The 5th mode and 6th mode cases show similar responses that have the large nonlinear behavior at around 33rd floor in negative H1 direction. When the vertical distribution was in accordance with 7th mode shape, the result shows the largest strength and stiffness. The main nonlinear behavior occurred at the walls and beams upper 21st floor in negative H1 direction. In case of the 8th mode and 9th mode distributions, the local behavior of TB-2 model is similar with TB-1.

Figure 49 - Base Shear under Static Push-Over Load with Gravity: Uniform, Triangular and C Factor Distributions

Figure 50- Base Shear under Static Push-Over Load with Gravity: Mode Shape Distribution of TB-1

Figure 51- Base Shear under Static Push-Over Load with Gravity: Mode Shape Distribution of TB-2

The comparison of hysteretic behavior between two models is shown in Figure 52. The maximum strength of TB-2 was increased by the use of HPFRCCs. However, while TB-1 models maintained the maximum strength from 0.75% to 1.5%, the initial strength reduction of TB-2 occurred at 1.12% drift just as soon as the base shear reached the maximum strength. Furthermore, although the ductile property of HPFRCCs, the large strength reduction of TB-2 occurred when the target drift cycle was 2.5% as different with TB-1 model in positive direction. At that drift, the axial strain gages at 15th floor in CW3 and CW3A reached the tensile yield strain DY, and the compressive strain in CW6A at same floor reached DR. The use of HPFRCCs from base to the 14th floor caused rather large strain at the 15th floor. On the opposite direction, when the target drift was -2.5% drift, TB-1 also showed a large strength reduction behavior as the compressive strain of CW3 and CW3A exceeded the maximum deformation. Overall, if the shift of weak story did not take place, the behavior of TB-2 could be better than the TB-1. When applying HPFRCCs in the critical region of the structural wall, the reduced reinforcement such as experimental work of Lequesne (2011) would be expected to avoid this shift of weak stories as well as the efficiency of construction work.

Figure 52 - Base Shear under Cyclic Push-Over Load

As described earlier, these models considered the P- Δ effects using the P- Δ column. Figure 53 demonstrates the effective shear force in this column with different load types. According to this, the minus values about shear are represented in these figures. The uniform vertical distribution cases also have the largest P- Δ shears followed by triangular distribution cases. The P- Δ shears of TB-1 have larger values than that of TB-2. In case of C_{vx} factor distribution, the proportions of the maximum P- Δ shear to the maximum base shear were about 2% for TB-1 and 1% for TB-2.

As shown in Figure 54 and Figure 55, the largest P- Δ shears are developed with the 4th mode case, and the results with 7th mode case are followed in both TB-1 and TB-2. The 5th, 6th, 8th, and 9th mode cases have relatively small P- Δ shears than that of others. The overall strength results of TB-2 have smaller values than the result of TB-1. Figure 56 shows the results from cyclic load case with C_{vx} factor distribution. In this case, the P- Δ shear of TB-2 is also smaller than that of TB-1 like static load case with C_{vx} factor distribution, but the large strength reduction occurred due to the cumulative damage by repeated loading.

Figure 53 - P-Δ Shear under Static Push-Over Load: Uniform, Triangular and C Factor Distributions

Figure 54 - P-Δ Shear under Static Push-Over Load: Mode Shape Distribution of TB-1

Figure 55 - P-Δ Shear under Static Push-Over Load: Mode Shape Distribution of TB-2

Figure 56 - P-Δ Shear under Cyclic Push-Over Load

5.3.2.3 Base Moment

The overturning moment result for TB-1 and TB-2 are shown in Figure 57 and Figure 60 according to the load type. The overall behavior is similar with base shear result, in which TB-2 model also has an increased strength about the overturning moment by using the HPFRCCs model. However, the maximum strengths depending on the vertical distribution shows almost same values as different with base shear results. TB-2 model has a relatively larger strength reduction than TB-1. The overturning moment results under the cyclic push-over load case have similar result about the base shear.

Figure 58 and Figure 59 show the results about the overturning moment of TB-1 and TB-2 respectively under the static push-over loads in accordance with the mode shapes. The largest overturning moment was developed with the first three mode cases in both TB-1 and TB-2, and the values are similar with the results from the uniform, triangular, and C_{vx} factor distribution cases. The 4th mode cases of both models also have the similar maximum strength values. However, the higher mode cases show the relatively smaller values.

Figure 57 - Overturning Moment under Static Push-Over Load: Uniform, Triangular, and C Factor Distributions

Figure 58 - Overturning Moment under Static Push-Over Load: Mode Shape Distribution of TB-1

Figure 59 - Overturning Moment under Static Push-Over Load: Mode Shape Distribution of TB-2

Figure 60 - Overturning Moment under Cyclic Push-Over Load

Figure 61 and Figure 64 represent the torsional moments of TB-1 and TB-2 under the static and cyclic push-over load respectively. A relatively larger torsional moment was indicated in TB-2 than TB-1, and the vertical pattern which has the largest torsional moment was the uniform distribution. The uniform distribution was required to larger force to push the building structure up to same drift ration with other vertical distribution, and this would cause larger torsion moment in same model. The results about the torsional moment were inversely proportional to the base shear results, and TB-2 had a slightly larger torsional moment at the same base shear than TB-1.

When the loads with mode shape distributions were applied, the results about the torsional moments of TB-1 and TB-2 are shown in Figure 62 and Figure 63 respectively. Both models have the largest torsional moments when 7th mode case and have the smallest torsional moments when the first three mode shape. The 4th mode case of TB-2 has a relatively large torsional moment unlike the 4th mode case of TB-1 while the overall results show similar trends with each mode shape case between TB-1 and TB-2.

Figure 61 - Torsional Moment under Static Push-Over Load: Uniform, Triangular, and C Factor Distributions

Figure 62 - Torsional Moment under Static Push-Over Load: Mode Shape Distribution of TB-1

Figure 63 - Torsional Moment under Static Push-Over Load: Mode Shape Distribution of TB-2

Figure 64 - Torsional Moment under Cyclic Push-Over Load

5.3.2.4 Details of Failure Process

The failure process of TB-1 and TB-2 can be examined through the local behavior that could be measured by rotation gages at the beam hinges and axial strain gage at the tips of the wall elements. Generally, the seismic behavior of structures can be described by the cyclic push-over load case rather than push-over load case, because this can represent the stiffness degradation and strength deterioration. These behavior properties can cause the reduction of deformation capacity under earthquake loading. In this section, the results under the cyclic load case will be shown. The rotation gage can show that which beam elements reach to the yielding point (Y), the strength reduction point (R), or the maximum deformation point (X) during each target drift level. The axial strain gage at the walls can measure the ultimate strength level (U), the strength reduction point (R), or the maximum deformation point (x) for the concrete or HPFRCCs in compression, and the yielding point of reinforcement (Y) in tension.

The local behavior of coupling beams in TB-1 was tabulated in Table 23. Here, the numbers following the CB1 and hyphen mean the adjoining wall element around the beams. Because there are two beams at the end of CW1, R is added to the coupling beams in the right of CW1 and L is added to the left coupling beams with reference to Figure 33. Because the cycling loading was repeated twice before 1.5% roof drift, the table shows repeated roof drifts with the number. The table indicates the target roof drift when each beam element reached the points Y, R, and X. When the target roof drift were 0.75% in positive and negative direction, the first yield rotation was measured at CB1-1R, CB1-1L and CB1-2 in upper stories. The yielding of coupling beams gradually spread to the beams in lower stories. At the same time, the partial CB1-1L hinges began to reach maximum rotation during -0.75% drift. Because the first floor walls that have large thickness and a large amount of reinforcement are considerable stiff than second floor, the weak story was occurred at these 2nd floor and the 1st floor beams behaved at the elastic range.

Table 24 shows the results measured from the rotation gages of coupling beams in TB-2. The initial yield of coupling beams also occurred at 0.75% drift. This yield was not indicated at the 56th floor, and the lower floor walls reached the Y point at the later drift level as different with TB-1. In addition to this, CB1-1R in lower floors of TB-2 did not yield. The hinge rotation of beam elements reached the DR and DX point after the target drift was -1.25%. Any CB1-2 did not attain the maximum deformation as well as CB1-1R in lower floors. The beam elements well modeled the ductile behavior by using of HPFRCCs.

Table 23 Failure Process of Coupling Beams in TB-1 under Cyclic Push-Over Load

TB-1	DY			DR			DX		
Floor	CB1-1L	CB1-1R	CB1-2	CB1-1L	CB1-1R	CB1-2	CB1-1L	CB1-1R	CB1-2
56	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
55	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
54	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
53	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
52	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
51	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
50	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
49	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
48	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
47	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
46	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
45	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
44	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
43	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
42	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
41	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
40	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
39	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
38	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
37	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
36	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
35	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
34	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	1.75%	-0.75%_1	1.00%_1	2.50%
33	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	2.50%
32	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	2.50%
31	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	2.50%
30	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	2.50%
29	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
28	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
27	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
26	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
25	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
24	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
23	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
22	-0.75%_1	0.75%_1	0.75%_2	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
21	-0.75%_1	0.75%_1	1.00%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
20	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
19	-0.75%_1	0.75%_1	0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%
18	-0.75%_1	0.75%_1	-0.75%_1	-0.75%_1	1.00%_1	2.00%	-0.75%_1	1.00%_1	3.00%

17_1	-0.75%_1	0.75%_1	0.75%_2	-0.75%_1	1.00%_1	2.00%	-1.00%_1	1.00%_1	3.00%
17	-0.75%_1	0.75%_1	1.25%_1	-0.75%_1	1.00%_1	2.00%	-1.00%_1	1.00%_1	3.00%
16	-0.75%_1	0.75%_1	1.25%_1	-0.75%_1	1.00%_1	2.50%	-1.00%_1	1.00%_2	3.00%
15	-0.75%_1	0.75%_1	1.25%_1	-0.75%_1	1.00%_1	2.50%	-1.00%_1	1.25%_1	3.00%
14	-0.75%_1	0.75%_1	1.25%_1	-1.00%_1	1.00%_1	2.50%	-1.00%_1	1.25%_1	3.00%
13	-0.75%_1	1.00%_1	1.25%_1	-1.00%_1	1.00%_2	2.50%	-1.00%_1	1.25%_1	3.00%
12	-0.75%_1	1.00%_1	-1.25%_1	-1.00%_1	1.00%_2	2.50%	-1.00%_1	1.25%_1	3.00%
11	-0.75%_1	1.00%_1	-1.25%_1	-1.00%_1	1.25%_1	2.50%	-1.00%_1	1.25%_2	3.00%
10	-0.75%_1	1.00%_1	-1.25%_1	-1.00%_1	1.25%_1	2.50%	-1.25%_1	1.50%	3.00%
9	-0.75%_1	1.00%_1	-1.25%_1	-1.00%_1	1.25%_1	2.50%	-1.25%_1	1.50%	3.00%
8	-0.75%_1	1.00%_1	-1.25%_1	-1.00%_1	1.50%	2.50%	-1.25%_1	1.50%	3.00%
7	-0.75%_1	1.00%_1	-1.25%_1	-1.25%_1	1.50%	2.50%	-1.25%_1	1.50%	3.00%
6	-0.75%_1	1.00%_1	-1.25%_1	-1.25%_1	1.50%	2.50%	-1.25%_1	1.50%	3.00%
5	-1.00%_1	1.00%_2	-1.25%_1	-1.25%_1	1.50%	2.50%	-1.25%_1	1.75%	-
4	-1.00%_1	1.25%_1	-1.50%	-1.25%_1	1.75%	3.00%	-1.25%_1	1.75%	-
3	-1.00%_1	1.50%	-1.50%	-1.25%_1	3.00%	3.50%	-1.25%_1	3.00%	-
2	-1.25%_1	3.00%	-1.50%	-1.25%_1	3.00%	3.50%	-1.25%_1	3.00%	-
1	-	-	-	-	-	-	-	-	-

Table 24 Failure Process of Coupling Beams in TB-2 under Cyclic Push-Over Load

TB-2	DY			DR			DX		
Floor	CB1-1L	CB1-1R	CB1-2	CB1-1L	CB1-1R	CB1-2	CB1-1L	CB1-1R	CB1-2
56	-1.00%_1	1.00%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
55	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
54	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
53	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
52	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
51	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
50	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
49	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
48	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
47	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
46	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
45	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
44	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
43	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
42	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
41	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
40	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
39	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
38	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
37	-0.75%_1	0.75%_2	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-

36	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
35	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
34	-0.75%_1	0.75%_1	0.75%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
33	-0.75%_1	0.75%_1	0.75%_2	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
32	-0.75%_1	0.75%_1	0.75%_2	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
31	-0.75%_1	0.75%_1	0.75%_2	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
30	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.25%_1	1.50%	-
29	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.25%_2	1.50%	-
28	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.25%_2	1.50%	-
27	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.50%	-
26	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.50%	-
25	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
24	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
23	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
22	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
21	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
20	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
19	-0.75%_1	0.75%_1	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
18	-0.75%_1	0.75%_2	1.00%_1	-1.25%_1	1.50%	-3.00%	-1.50%	1.75%	-
17_1	-0.75%_1	1.00%_1	1.25%_1	-1.25%_1	1.50%	-3.00%	-1.75%	1.75%	-
17	-0.75%_1	1.00%_1	1.25%_1	-1.50%	1.50%	-3.00%	-1.75%	1.75%	-
16	-0.75%_1	1.00%_1	1.25%_1	-1.50%	1.50%	-3.00%	-1.75%	1.75%	-
15	-0.75%_1	1.00%_1	1.25%_1	-1.50%	1.75%	-3.00%	-1.75%	2.00%	-
14	-1.00%_1	1.25%_1	-1.75%	-1.50%	-	-	-1.75%	-	-
13	-1.00%_1	1.25%_1	-1.75%	-1.75%	-	-	-1.75%	-	-
12	-1.00%_1	1.25%_1	-1.75%	-1.75%	-	-	-1.75%	-	-
11	-1.00%_1	-	-1.75%	-1.75%	-	-	-1.75%	-	-
10	-1.00%_1	-	-1.75%	-1.75%	-	-	-1.75%	-	-
9	-1.00%_1	-	-1.75%	-1.75%	-	-	-1.75%	-	-
8	-1.00%_1	-	-1.75%	-1.75%	-	-	-1.75%	-	-
7	-1.00%_1	-	-1.75%	-1.75%	-	-	-2.00%	-	-
6	-1.25%_1	-	-1.75%	-1.75%	-	-	-2.00%	-	-
5	-1.25%_1	-	-2.00%	-1.75%	-	-	-2.00%	-	-
4	-1.25%_1	-	-2.00%	-1.75%	-	-	-2.00%	-	-
3	-1.50%	-	-2.00%	-1.75%	-	-	-2.00%	-	-
2	-1.75%	-	-2.00%	-1.75%	-	-	-2.00%	-	-
1	-	-	-	-	-	-	-	-	-

Figure 65 describes the location and name of each strain gages at each node. The gage takes its name from the combination of each wall element and the direction of the location. The local behavior of the wall elements in TB-1 and TB-2 are shown in Table 25 and Table 26, respectively.

TB-1 model measured the first axial yield strain at 4th ~ 6th floors of the left ends of CW6 during the 1% drift, while this occurred earlier at 15th floor when 0.5% target drift in TB-2 model. The most tensile yield in TB-1 was developed at lower stories around the 2nd ~ 5th floors, TB-2 showed the earlier yield at around 15th floor that begin to use normal concrete after HPFRCCs. In spite of the ductile properties of HPFRCCs, the overall behavior was worse in TB-2 by the shift of weak story. In case of the compression, the initial behavior of TB-2 models was not significantly different with TB-1 up to -1% drift cycle. However, from 1.25% target roof drift, the compression strain gages at the lower stories around 2nd floor reached the DU, DR, and DX point in TB-1, while these were mainly happen at around 15th floor of TB-2 model. The failure of TB-1 wall systems was significantly influenced by the fracture of CW3, CW3A, CW1, and CW1A at 2nd floor after fractures of CW6 and CW6A. The failure of TB-2 began from fracture of CW6A at 15th floor, and this was propagated to CW4, CW4A, and the right side of CW1 and CW1A in 15th floor. And then, while the fracture was occurred at the left side of CW1A, CW3, and CW6 in 15th floor when -3.0% drift, the structure finally collapsed with H2 direction at this weak stories.

In case of TB-1, the initial nonlinear behavior was occurred at the coupling beam, and then the strain gages in wall elements reached the yielding of reinforcements and ultimate strength point of concretes. If the coupling beams failed, each shear wall is subjected to larger force than before. After all CB1-1L and a large part of CB1-1R were failed at -1.25% roof drift, the reinforcement in 2nd floor of CW1 and CW1AR yielded in tension and concrete at same floor of CW1 began to reach the ultimate strength level in compression.

The initial yielding was occurred at CW6W in TB-2 before the beams reached any yield point. And then, the yielding of wall and beam elements was measured around the same drift cycles. After the fails of CB1-1L and CB1-1R during -1.75% and -2.00% drift, the reinforcement was yielded and concrete reached at the DR point in second floor of CW1 and CW1A. Generally, the previous experiments for coupled walls demonstrate that ductile coupling beams improve some performances of the coupled wall system such as the ductility or energy dissipation. In spite of more ductile behavior of the coupling beams than that of TB-1, the performance of TB-2 was worsened due to the early development of a weak story.

Figure 65 - Location of Strain Gages for Tall Building

Table 25 Failure Process of Walls in TB-1 under Cyclic Push-Over Load

Tension Strain			Compression Strain					
DY			DU			DR		
Roof Drift	Element	Floor	Roof Drift	Element	Floor	Roof Drift	Element	Floor
1.00%_1	CW6W	4~6F	0.75%_1	CW6AE	2F	-1.25%_1	CW6W	2F
1.00%_2	CW6W	7~13F	-0.75%_1	CW6W	2~5F	1.50%	CW6AE	4F
-1.00%_2	CW6AE	4~7F	1.00%_1	CW6AE	2,4,11F	-1.50%	CW1W	2F
1.25%_1	CW6W	2,14~17F		CW6AE	5F	-2.50%	CW2LW	2F
-1.25%_1	CW1ARE	2F	-1.00%_1	CW6W	6,7,11F		CW3	2F
	CW1ALE	2F	1.25%_1	CW6AE	6~10F, 12~14F		CW3A	2F
	CW1E	2F		CW6AE	15F		CW6E	2F
	CW2RE	2F				3.00%	CW1ALE	2F

	CW4	2F	-1.25%_1	CW1W	2~4F	-3.00%	CW1E	2F	
	CW4A	2F		CW2LW	2F		CW7	2F	
	CW6AE	2,8,9F		CW3AS	2F		CW8AL	2F	
	CW6AW	2F	1.50%	CW1E	3~5F	3.50%	CW1AR	2F	
1.50%	CW1W	3,4F		CW2RE	4F		CW4N	2F	
	CW1AW	4F		CW4AS	4F		CW6A	2F	
	CW2LW	4F		CW6AE	16F	DX			
	CW3	4F	-1.50%	CW3AN	2F	Roof Drift	Element	Floor	
	CW3A	4F	1.75%	CW2R	3F				
	CW6W	1_1F		CW4AS	3F	-1.25%_1	CW6W	2F	
	CW6E	4,5F		CW7S	2F	1.50%	CW6AE	4F	
-1.50%	CW1ARE	3F	2.00%	CW2RE	5F	-1.50%	CW1W	2F	
	CW6AW	4F		CW4AS	5F	-2.50%	CW2LW	2F	
	CW8AL	2F		CW4AN	4F		CW3	2F	
1.75%	CW1ARW	4F	-2%	CW3S	2F		3.00%	CW3A	2F
	CW1ALW	5F		CW7S	4F	CW6E		2F	
	CW3N	5F	-2.50%	CW1AL	2F	3.00%	CW1ALE	2F	
	CW8AR	4F		CW3	2F		CW1E	2F	
-1.75%	CW6AW	4F		CW6E	2F		CW7	2F	
-2.00%	CW8L	2F	3.00%	CW1E	2F	3.50%	CW1AR	2F	
2.50%	CW1ALW	3F		CW1AW	2F		CW4N	2F	
	CW2LW	3,5F		CW6AW	4F		CW6A	2F	
	CW3N	3F		CW7N	2F		CW8AL	2F	
	CW3S	5F		CW8AL	2F				
	CW3A	5F		-3.00%	CW8AR				2F
	CW8R	4F		3.50%	CW4				2F
-2.50%	CW4AN	3F							
3.00%	CW1W	5F							

Table 26 Failure Process of Walls in TB-2 under Cyclic Push-Over Load

Tension Strain			Compression Strain					
DY			DU			DR		
Roof Drift	Element	Floor	Roof Drift	Element	Floor	Roof Drift	Element	Floor
0.50%_1	CW6W	15F	-0.75%_1	CW6W	2~4F	1.25%_1	CW6AE	15F
-0.50%_1	CW6AE	15F	1.00%_1	CW6AE	2~5F, 11~17F	-1.75%	CW1W	2F
0.75%_1	CW6E	15F	-1.00%_1	CW6W	5~8F, 11~14F		CW6W	2F
0.75%_2	CW6W	17F	1.25%_1	CW1E	15F	-2.00%	CW1E	15F
1.00%_1	CW3N	15F		CW2RE	15F	2.50%	CW1AR	15F
	CW6W	2,4,17_1		CW4AS	15F		CW2RE	15F

		F						
-1.00%_1	CW6AW	15F		CW6AE	6~8F,17_1F,21F		CW4	15F
	CW6AE	2,4,17F	-1.25%_1	CW1W	2F		CW4A	15F
1.25%_1	CW1ALE	15F		CW6W	15F		CW6AW	15F
	CW1W	15F	-1.25_2	CW1W	3F		CW7N	15F
	CW2LW	15F	-1.50%	CW1W	4F	-2.50%	CW1W	15F
	CW3R	15F		CW6W	16,17F	-3.00%	CW1AL	15F
	CW3A	15F	1.75%	CW4AN	15F		CW2LW	15F
	CW6W	16,18~20F	-1.75%	CW2LW	2,3F		CW3	15F
	CW8R	15F		CW3A	2F		CW3A	15F
	CW8AR	15F		CW3AS	3F		CW6	15F
1.50%	CW1ARW	15F		CW3S	2F		CW8AR	15F
	CW6E	16F		CW7S	15F		CW8AL	15F
-1.50%	CW4	15F	2.00%	CW4S	15F	DX		
	CW6AE	6,17_1,18F	2.50%	CW1AR	15F	Roof Drift	Element	Floor
-1.75%	CW2RE	2F		CW4N	15F			
	CW4	2F		CW6AW	15F	1.50%	CW6AE	15F
	CW4A	2F		CW7S	2F	-1.75%	CW6W	2F
	CW6AW	2F		CW7N	15F	2.50%	CW1ARE	15F
	CW8AL	2F		CW8R	15F		CW1R	15F
	CW1ALE	2F		CW1W	15F		CW2RE	15F
	CW1E	2F	-2.50%	CW1ALE	15F		CW4	15F
	CW1AR	2F		CW6E	2F		CW4A	15F
2.50%	CW7N	15F	3.00%	CW2RW	15F		CW6AW	15F
-2.50%	CW8L	2, 15F		CW2LE	15F		CW7	15F
-3.00%	CW2	15F		CW8A	15F	-2.50%	CW1L	2,15F
	CW4AR	15F	-3.00%	CW2LW	15F	-3.00%	CW1AL	15F
3.50%	CW3A	16F		CW3	15F		CW2LW	15F
	CW3AL	17F		CW3A	15F		CW3	15F
	CW2L	16F		CW6E	15F		CW3A	15F
	CW4AS	16F					CW6	15F
	CW8L	16,17F				3.50%	CW8AL	15F
	CW8AL	16,17_1,19F						
	CW8R	16,17F						
	CW8AR	15,16,17_1,19F						

DX				
Roof Drift	Element	Floor		
1.75%	CW6W	15F		
-2.00%	CW6AE	2F		
2.50%	CW2LW	15F		
	CW3A	15F		
	CW3S	15F		
	CW6E	15F		
-3.00%	CW8L	15F		
3.50%	CW8R	15F		

5.3.3 Earthquake Load

5.3.3.1 Story Drift

Figure 66 shows the interstory drift of TB-1 and TB-2 subjected to El Centro earthquake loads when each model has a maximum base shear. The interstory drift indicates the relative floor displacement divided by the floor height. The story drift of TB-2 has almost same results with the story drift of TB-1. Most of story drift occurred at lower part. The largest story drift is about 0.26% at 12th floor, and this value is remarkably small.

Figure 66 - Story Drift under an Earthquake Load at Instant of Maximum Base Shear

5.3.3.2 Base Shear

Figure 67 demonstrates the time histories of base shear in H1 direction for TB-1 and TB-2, and both models have similar results. For this earthquake load, the maximum base shear is about 45,000 kN in both models. At the instant of this maximum base shear, story by story lateral forces for TB-1 and TB-2 are shown in Figure 68 and the lateral forces are concentrated on the lower part. By the strain gage, TB-1 and TB-2 under this earthquake load have linear behaviors results in spite of the relatively large base shear compared with cyclic push-over load cases. These cyclic loads had a vertical distribution with C_{vx} factor and this distribution case was similar with the first three mode distribution cases. Therefore, the response of these structures follows higher modes and has a linear

behavior under the El Centro earthquake. As a result, the use of HPFRCC did not have a significant impact on the behavior of the structures.

Figure 67 - Base Shear under an Earthquake Load

Figure 68 – Story by Story Lateral Force under an Earthquake Load at Instant of Maximum Base Shear

5.3.3.3 Base Moment

Figure 69 represents hysteretic curves about the overturning moment of TB-1 and TB-2 under the El Centro earthquake loads. The two models have similar results, and the largest moment is about 2,500,000kN-m. This value is relatively smaller than the results from cyclic push-over load case. However, the maximum torsional moments of TB-1 and TB-2 are sustainably larger than that of cyclic push-over load case (Figure 70). For this ground motion, the structures have elastic behavior despite the large torsional moments.

Figure 69 - Overturning Moment under an Earthquake Load

Figure 70 - Torsional Moment under an Earthquake Load

Chapter 6. Conclusions

The main property of high performance fiber-reinforced cement composites (HPFRCCs) is characterized by the strain hardening behavior, and the enhanced response can be generally expected with the structure using this material compared to the normal concrete structure. This study is to investigate the seismic performance of a tall building with coupled wall systems using HPFRCCs. For this purpose, the first work is the development of analysis methods for the coupled wall systems based on the previously proposed methods to represent the nonlinear behavior of each component. The more reliable and economical modeling methods for coupling beams and shear walls was considered. The behavior of coupling beams was described with Moment Hinge Model including the reinforcement slip effect, and Fiber Model was selected for the wall modeling.

Two 4-story coupled wall specimens (CW-1 and CW-2) which were tested by Lequesne (2011) were used to check a reliability of the proposed modeling methods. Both specimens had the HPFRCCs coupling beams, and the walls of CW-1 followed a traditional RC design while the 1st floor wall of CW-2 used HPFRCCs materials with reduced reinforcement. The test results about overturning moment were compared with the analysis results to verify the overall behavior of the specimens under the reversed cyclic loading. The measured yielding process was also discussed for the comparison of the local behavior. The overall behavior of the tested couple wall systems was well simulated by the proposed modeling methods, especially such as the initial strength, stiffness, and energy dissipation. In case of the local behavior model, the location and time of the yielding occurrence almost corresponded with the test results. The analysis also well represents the strength increases observed in the measured strength of CW-2 by using HPFRCCs in comparison with CW-1. On the other hand, the models could not simulate the sliding shear displacement of coupling beam in CW-1 at 2% wall drift and shear fail of the wall in CW-2 at -2.5% wall drift.

These modeling methods for coupled wall system were applied with a 56-story tall building model (TB-1 and TB-2) to observe the effectiveness of HPFRCCs in coupling beams and walls 1/4 height of the building. The main analysis results with different conditions are summarized as follows:

- The constant gravity loads were applied considering the weight of the structure, assumed super imposed dead loads, and live loads. When the tall building was subjected to the static push-over loads with this gravity loads, the structure system had substantially larger base shear strength and strength reductions than the results without gravity loads. The differentials of the strength between two cases were even higher than P- Δ shear.
- Various types of the vertical distribution of lateral loads were modeled in this study. The result of the model with uniform distribution showed the largest base shear forces than the

triangular and C_{vx} factor distribution cases. However, they had a similar maximum overturning moment results. In case of TB-2, the model with C_{vx} factor distribution had a different interstory drift result that described a shift of weak story at the last wall drift.

- The results from lateral loads with modal distributions indicated that the first three mode cases had a similar result with the results from C_{vx} factor distribution case. The higher mode cases showed the larger base shear results and the smaller overturning moment results.
- The buildings under the reversed cyclic loading represented the large strength degradation at the latter cycles which is important properties of reinforced concrete members to account for the cumulated damage.
- The HPFRCCs coupling beams used in TB-2 showed more ductile behavior than the normal concrete coupling beams in TB-1.
- Failure processes of TB-1 and TB-2 were very different each other. The initial nonlinear behavior of TB-1 was dominated by the yielding of coupling beams, while the nonlinear behavior of walls in TB-2 at the weak story caused the initial response.
- The use HPFRCCs in all beams and the lower 1/4 stories (1~14F) of walls of TB-2 led to the earlier occurrence of weak story at 15th floor.
- For El Centro earthquake load cases, TB-1 and TB-2 had elastic behavior in spite of the large base shear. This response could be caused by the higher mode effects.

This study provides the information about the nonlinear analysis of tall coupled wall systems using HPFRCCs without design changes of the structure. When the better performance and construction work are intended due to the advantage of HPFRCCs material, the proper design changes are required in the HPFRCC walls such as the reduction of reinforcement quantity to prevent the occurrence of weak story in the higher floors before the HPFRCC coupling beams dissipate the sufficient energy.

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