Master's Thesis of Engineering

Ductility of Boundary Element of Walls with Simplified Confinement Details

간략 선조립 상세을 적용한 RC 휨 벽체
경계요소의 연성능력

August 2021

Graduate School of Enginerring

Seoul National University

Architercture and Archterctural Engineering

Mok In Park

Ductility of Boundary Element of Walls with Simplified Confinement Details

Advisor: Hong Gun Park

Submitting a Master's thesis of
Architecture and Architectural Engineering

August 2021

Graduate School of Engineering
Seoul National University
Architecture and Architectural Engineering

Mok In Park

Confirming the Master's thesis written by
Mok In Park

July 2021

Chair	Sung Gul Hong
Vice Chair	Hong Gun Park
Examiner	Cheol Ho Lee

Abstract

Ductility of Boundary Element of Walls with Simplified Confinement Details

Mok In Park
Department of Architecture and Architectural Engineering
College of Engineering
Seoul National University

The recent major earthquakes, Gyeongu and Pohang Earthquake, occurred in succession, causing large damage to many residential buildings. In recent years, as the frequency of earthquakes and the risk of earthquake load in korea increase, the importance of seismic performance of RC wall structure, which is mostly used in residential building, is emphasized.

Accordingly, the walls of apartment buildings that are becoming taller have large compressive force corresponding 15 to 30% of the wall compression capacity due to the gravity load. When excessive axial force is applied to the shear wall, an increase of the compressive strain at the end-region of wall cause the concrete to collapse during earthquake load. And it increases the risk of collapse due to the decrease in the ductility of bearing walls.

In addition, the use of high-strength rebar for longitudinal reinforcement of the bearing wall is increasing. But, high-strength rebar has lower ductility capacity and yield ratio, so low-frequency fatigue and buckling of rebar may occure under earthquake load, which may degrade the performance of the wall. Therefore, the use of seismic rebar with excellent ductility is required. The recently revised Building Seismic Design Code(KDS 41 17 00) prescribed the use of seismic rebar for medium and high ductility structural types.

In accordance with the design code, it is stipulated that special confinement detail consisiting of closed hoop and cross-tie used for the boundary element of RC bearing wall system that is more than height of 60m and belongs to Seismic Design Category(SDC) D. However, domestic apartment walls are thin. And if such a thin wall is designed as special shear wall, it may lead to a decrease in constructability. Therefore, it is necessary to develop the seismic detail that verify constructability and cost reduction, it is essential to improve the ductility of the high-rise bearing wall.

Therefore, in this study, transverse reinforcement detail that enhances constructability and economics is developed through simplified confinement detail, and cyclic axial loading test for boundary element was performed to pre-verify the effectiveness of the simplified confinement detail. In addition to evaluate the ductility capacity of RC wall with the detail, cyclic lateral loading test was performed. The main variables were the type of rebar, the type of transverse reinforcement details, the vertical spacing of the transverse reinforcement, and the load history.

As a result of the boundary element test, in the load history that simulates the hysteretic behavior of walls subjected to relatively high axial force., the deformation capacity increased as the spacing of the transverse reinforcement detils narrowed. Also, in the load history that simulates the hysteretic behavior of walls with low axial force and reinforcement ratio, the performance of simplified confinement detail showed the same deformation capacity as special seismic detail, and the effectiveness of the deformation capacity as an alternative ductile detail was verified. In addition, as a result of cyclic lateral loading test, the narrower the spacing of the transverse detail, the higher the lateral drift ratio, and the core concrete of the boundary element was well constrained. Because the seismic rebar has high tensile to yield ratio, the excess strength ratio to the nominal flexural strength was large in the specimen.

Therefore, if the simplified confinement detail is used, it can be expected that the costuctability and economics are better than the current seismic detail of the special RC shear wall. In addition, it is expected to increase structural safety from seismic load by showing superior ductility than ordinary shear wall, preventing brittle failure of concrete that may occur at the end-region of RC wall.

Keywords : RC shear wall, Simplified detail of boundary confinement, Seismic reinforced bar, Boundary element

Student Number : 2019-29053

Contents

List of Tables

List of Figures

Chapter 1. Introduction

1.1 Background

The recent major earthquakes, Gyeongu and Pohang Earthquake, occurred in succession, causing large damage to many residential buildings. In recent years, as the frequency of earthquake earthquakes and the risk of earthquake load in korea increase, the importance of seismic performance of RC wall structure, which is mostly used in residential building, is emphasized.

Currently, more than 91% of Korean population lives in urban areas. Therefore, the density and height of buildings are gradually increasing more and more, and the proportion of apartment building is close to 60% of total domestic houses. In order to efficiently utilize the floor plan and shorten the construction period, the structural type of high-rise apartment building is mainly bearing wall system.

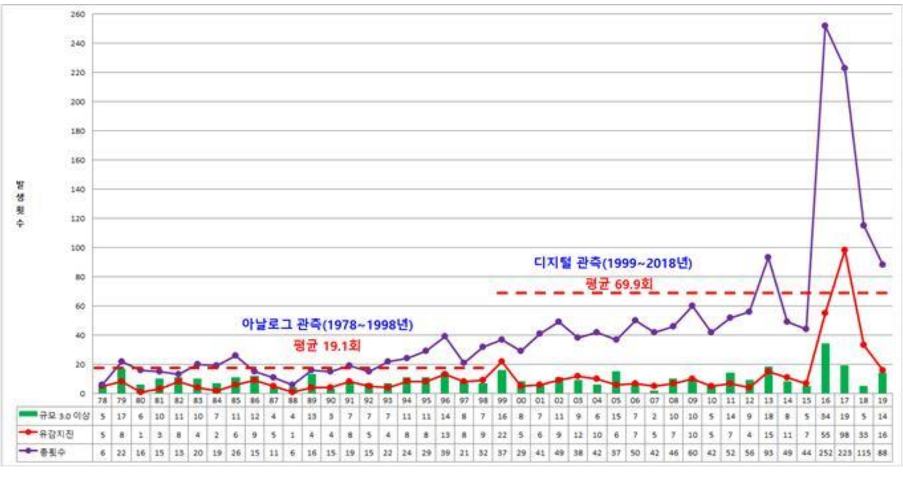


Figure 1-1 Earthquake records in Korea

Accordingly, the walls of apartment buildings that are becoming taller have large compressive force corresponding 15 to 30% of the wall compression capacity due to the gravity load. When excessive axial force is applied to the shear wall, an increase of the compressive strain at the end-region of wall cause the concrete to collapse during earthquake load. and it increses the risk of collapse due to the decrease in the ductility of bearing walls. In addition, the use of high-strength rebar for longitudinal reinforcement of the bearing wall is increasing. But, high-strength rebar has lower ductility capacity and yield ratio, so low-frequency fatigue and buckling of rebar mayoccure under earthquake load.

The proportion of a bearing wall system structure in residential buildings is 99.3% for private apartment and 96.8% for public apartment as shown in Table 1-1. In terms of urban overpopulation and efficient use of terriroty, the demand for residential apartment building will countinue to increase, and the use of bearing wall system structure is expected to maintain same ratio as now. In addition, the seismic design of residential apartment building is essential due to the mandatory requirement for buildings with two or more floors.

Figure 1-2 Damages of the earthquake in Gyeongju and Pohang

Table 1-1 Structure type of residential apartment with 500 or more households completed in the last 10 years(as of 2017)

Region	Private Apartment			Republic Apartment		
	Sum	Bearing wall system	Moment-resisting frame system	Sum	Bearing wall system	Moment-resisting frame system
Seoul	128,421	126,724	1,697	54,590	37,116	17,474
Pusan	105,693	105,045	648	19,956	19,956	-
Deagu	90,124	90,124	-	31,072	31,072	-
Incheon	107,294	107,294	-	39,223	39,087	136
Gwangju	44,593	44,593	-	23,811	23,811	-
Daejein	33,653	33,653	-	26,300	26,300	-
Ulsan	45,164	45,164	-	7,372	7,372	-
Gyeonggi	374,456	370,789	3,667	233,540	233,540	-
Kangwon	26,372	26,372	-	11,157	11,157	-
Chungbuk	42,033	37,913	4,120	21,326	21,030	296
Chungnam	85,262	85,262	-	22,344	22,344	-
Cheonbuk	34,927	34,927	-	22,482	22,482	-
Cheonnam	27,702	27,702	-	14,012	14,012	-
Sejoig	44,617	44,617	-	9,447	8,283	1,146
Gyeongbuk	67,916	67,916	-	17,865	17,865	-
Gyeongnam	117,171	117,171	-	31,348	31,348	-
Jeju	3,788	3,788	-	6,628	6,628	-
Sum	1,379,186	1,369,054	10,132	592,473	573,403	19,070
	100%	**99.3%**	**0.7%**	**100%**	**96.8%**	**3.2%**

Figure 1-3 Details of reinforced concrete shear wall

Chapter 1. Introduction

In accordance with the current Building Seismic Design Code(KDS 41 17 00), special secismic detail consisting of rectangular closed hoop and cross-tie is stipulated for the wall boundary element of reinforcement concrete bearing wall system with height of 60m or more and belonging to Seismic Design Category(SDC) D. Also, the longitudinal reinforcement of boundary element requires the use of seismic rebar. However, the walls of domestic residential apartment building use thin wall of 150mm to 300mm. When designing such a thin wall as special RC shear wall, it is difficult to manage the quality of the wall due to the complex and densely transverse reinforcement, and the constructability and economic feasibility are deteriorated. Therefore, it is necessary to reduce the amount of reinforcement and simplify the details so as to be suitable for the construction of bearing wall system for residential apartment buildings.

1.2 Scope and Objectives

The purpose of this study is to develop simplified confinement detail and verify its deformation capacity in order to efficiently secure the proper seismic performance of the structural RC wall of high-rise apartment. In addition, it intends to enhance constructability and economic feasibility through the development of simplified confinement detail.

In order to achieve the purpose of the study, a modular transverse reinforcement was developed by supplementing the shortcomings of the previously simplified details. To verify the deformation capacity of the proposed seismic detail and the effect of seismic reinforced bar, cyclic axail load test for the boundary element was first conducted. In this experiment, to compare and verify the performance of simplified confinement detail, the test was performed on the boundary element specimen without transverse reinforcement and the boundary element with special seismic detail. Also, load history was planned to examing the stability of the wall boundary element when tensile strain may be excessively generated under the seismic load. Based on the results of the cylic axial load test, the evaluation and improvement of the developed boundary element detail were carried out.

To verify the ductility, deformation capacity, and ultimate behavior when the simplified confinement detail of boundary element was applied to the actual RC wall, not the idealized test of wall boundary element, cyclic lateral load test for RC wall was conducted. After the cyclic lateral load test, the structural safety of the wall using the simplified confinement detail was evaluated. The performance of the detail was experimentally verified, and when applying performanc-based seismic design, the possibility as an alternative seismic detail rather than special seismic detail was reviewed.

1.3 Outline of the Master's Thesis

In chapter 2, current design codes and previous studies were reviewed. The scope of the review is codesfor the installation of boundary elelemts of special RC shear wall, transverse reinforcement detail, the standard for structural system. The current codes were examined because the purpose was to suggest simplified confinement detail for resolving over-reinforcement of transverse details installed on the boundary element of special RC shear wall. In addition, studies on the previously developed simplified detail and idealized boundary element test were investigated.

In Chpater 3, the assumption of the internal stress of the wall where plastic hinge occurs and the development of simplified confinement detail were studied for cyclic axial loading test of specimnes. The experimental test was conducted on the deformation capacity of the detail under cyclic axial loading and the concrete confinement effect. Also, in order to verify the structural performance of the speccimens, compression and tesnsile deformation, failure mode, and ultimate strength, etc according to each variable were evaluated.

In Chapter 4, cyclic lateral loading test was performed by applying the simplified confinement detail verified in the boundary element test to the RC wall specimens. The RC wall specimens was evaluated for ductility, deformation capacity, ultimate behavior, compressive depth zone, failure mode, peak strength, etc. The effectiveness of the boundary element test was verified by comparing and analyzing the actual wall behavior and the boundary element test results.

Finally, summary and conclusions presented in Chapter 5.

Chapter 2. Literature Review

2.1 Current Design Codes

2.1.1 KDS 41 17 00

KDS 41 17 00 [1] addresses the installation of special boundary element in wall seismic design for structures located in seismic design category D with height of 60m or more. In addition, KDS 41 17 00 addresses to use seismic reinforced rebar for vertical reinforcement of special boundary element. In KDS 41 17 00, different from ACI318-19 special structural wall, the transverse detail of special boundary element can be permitted to consist of U-shpaed stirrup and cross-tie, not rentangular closed hoop and cross-tie.

Design requirement for special boundary element is as follows.

(1) Compression zones shall be reinforced with special boundary elements where the following is satisfied.

$$c \geq \frac{l_w}{600(\delta_u/h_w)}$$

where δ_w/h_w shall not be taken less than 0.007

(2) Where special boundary elements are required by (1). Reinforcement of the special boundary elements shall extend vertically from the critical section at least the greater of l_w and $M_u/4V_u$.

where special boundary elements are required by 4.7.6 (1) through (6) shall be satisfied.

(1) The boundary element shall exten horizontally from extreme compression fiber a distance not less than **the larger of $\mathbf{c - 0.1}\boldsymbol{l_w}$ or $\boldsymbol{c/2}$**.

(2) For walls of flanged sections, the boundary element shall include the effective flange width in compression and extend at least 300 mm into the web.

(3) Transverse reinforcement of the boundary element shall satisfy 4.5.4(1) through 4.5.4(3). Eq. (4.5-3) need not be satisfied, and spacing of transverse reinforcement shall be **1/3 of the least dimension** of the boundary element. Transverse reinforcement conforming to 4.5.4(1)③ shall be in the form of closed hoops enclosing the edges of walls and shall be permitted to consist of **U-stirrups and cross-ties** extending a length equal to the development length beyond the boundary element in toe wall web.

(4) Transverse reinforcement shall extend into the support a distance not less than the development length in tension of the largest longitudinal reinforcement in the special boundary element. If the special boundary element connects with a footing or mat, transverse reinforcement shall extend at least 300mm into the footing or mat.

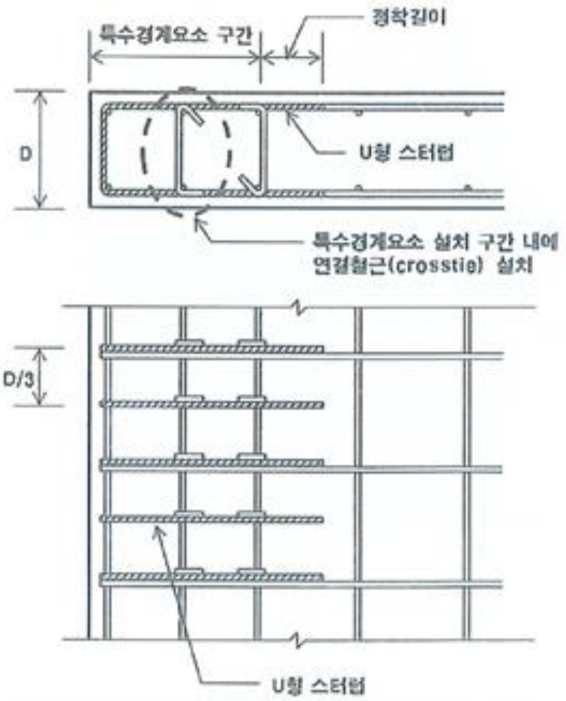


Figure 2-1 Transverse reinforcement detail of special boundary element(KDS)

Table 2-1 Design Coefficients and factors for bearing RC wall(KDS)

<table>
<tr><th rowspan="3">Seismic Force-Resisting System</th><th colspan="3">Design Coefficient factor</th><th colspan="3">Structural System Limitations Including Structural Height, (m) Limits</th></tr>
<tr><th rowspan="2">Response Modification Coefficient R</th><th rowspan="2">Overstrength Factor Ω_0</th><th rowspan="2">Deflection Amplification C_d</th><th colspan="3">Seismic Design Category</th></tr>
<tr><th>A or B</th><th>C</th><th>D</th></tr>
<tr><td>Special RC shear wall</td><td>5</td><td>2.5</td><td>5</td><td>NL</td><td>NL</td><td>NL</td></tr>
<tr><td>Ordinary RC shear wall</td><td>4</td><td>2.5</td><td>4</td><td>NL</td><td>NL</td><td>60</td></tr>
</table>

*NP : NOT PERMMITED / *NL : NOT LIMITED

2.1.2 ACI318-19

A boundary element is a portion along a structural wall edge of opening that is strengthened by longitudinal and transverse reinforcement. Where combined seismic and gravity loaing results in high compressive demands on the end-region of wall, ACI318-19 [2] requires a special boundary element. Where compressive demnads are lower, special boundary element are not required, but boundary element transverse reinforcement still is required if the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, psi [3].

Design requirement for boundary element of special walls is as follows.

The need for special boundary elements at the edges of structural walls shall be eavaluated in accordance with 18.10.6.2 or 18.10.6.3. The requirments of 18.10.6.4 and 18.10.6.5 shall also be satisfied.

Walls or wall piers with $\boldsymbol{h_{wcs}/l_w \geq 2.0}$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads shall satisfy (a) and (b) :

(a) Compression zones shall be reinforced with special boundary elements where

$$\frac{1.5\delta_u}{h_{wcs}} \geq \frac{l_w}{600\mathrm{c}}$$

and **c** corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement δ_u. Ratio δ_u/h_{wcs} shall not be taken less than 0.005.

(b) If special boundary elements are required by (a), then (i) and either (ii) or (iii) shall be satisfied.

(i) Special boundary element transverse reinforcement shall exten vertically above and below the critical section a least the greater of l_w and $M_u/4V_u$ except as permitted in 18.10.6.4(i)

(ii) $b \geq \sqrt{0.0025lc_w}$

(iii) $\frac{\delta_u}{h_w} = \frac{1}{100}(4 - \frac{1}{50}\left(\frac{l_w}{\mathrm{b}}\right)\left(\frac{c}{b}\right) - \frac{V_e}{8\sqrt{f_c'}A_{cv}})$

The value of δ_u/h_w need not be taken less than 0.015.

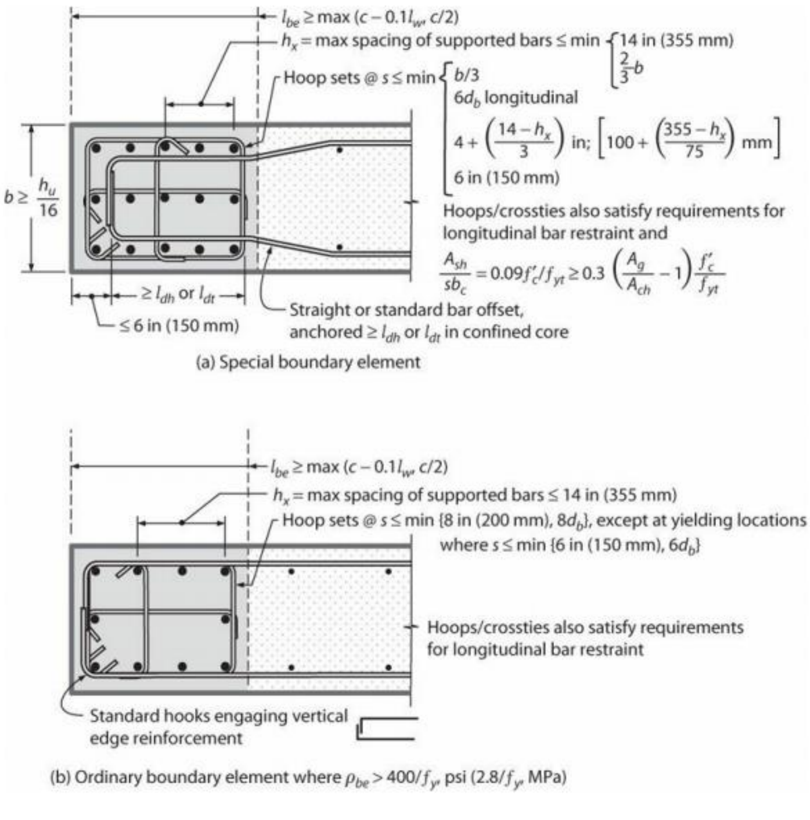


Figure 2-2 Special and ordinary boundary element according to ACI318

2.1.3 ASCE/SEI 7-16

For buildings in which special structural walls are the sole seismic-force-resisting system, ASCE/SEI 7 [4] limits height to 160ft (48.8m) in Seismic Design Category(SDC) D and E and 100ft (30.5m) in SDC F, These heights can be increased to 240ft (73.2m) and 160ft (48.8 m) if there is no extreme torsional irregulariry(as defined in ASCE 7) and the shear in any line for a dual system combining walls with special moment frames capable of resisting at least 25% of prescribed seimic forces[3].

Unlike ASCE/SEI 7, KDS 41 17 00 doesn’t have SDC E and F. In the case of US, the ground conditions are disadvantageous, and the Maximum Considered Eearthquake Acceleration is significantly higher than South Korea. Therefore, SDC E and F are determined using design earthquake spectral response acceleration parameters at 1-s periods and seismic importance Factor, which differs from Korean code. The method of calculating the seismic load is is almost similar.

Table 2-2 Design coefficients and Factor for RC shear wall(ASCE)

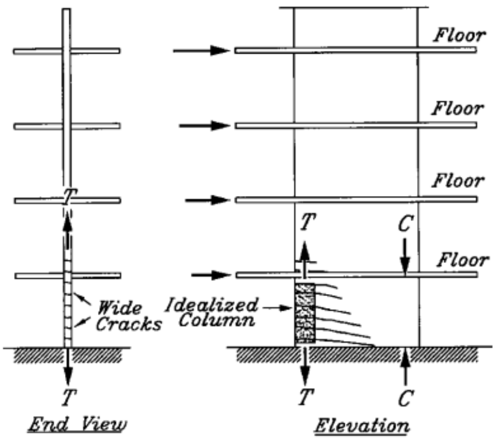
Seismic Force-Resisting System	Design Coefficient factor			Structural System Limitations Including Structural Height, (m) Limits				
	R	Ω_0	C_d	Seismic Design Category				
				A or B	C	D	E	F
Special RC shear wall	5	2.5	5	NL	NL	48.8	48.8	48.8
Ordinary RC shear wall	4	2.5	4	NL	NL	NP	NP	NP

*NP : NOT PERMMITED / *NL : NOT LIMITED

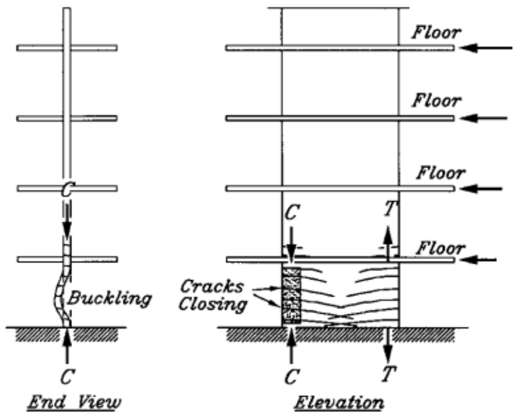
2.2 Review of previous research

2.2.1 Y.H. Chai and D.T.Elayer (1999)

Chai and Elayer conducted the experiment on the axial reversed cyclic response of well-confined reinforced concrete columns. And a phenomenological model based on the observed test behavior was proposed for estimating the maximum tensile strain.



(a)Opening of crack under tension cycle



(b) closing of cracks under compression cycle

Figure 2-3 Idealization of reinforced concrete wall in end-regions

The lateral stability of a ductile reinforced concrete wall was studied by idealizing the end-region of the wall as an axially loaded reinforced concrete column, as shown in Figure 2-3. And the basic behavior of a reinforced concrete column under an axial tension and compression cycle may be described by a plot of the nominal axial strain versus the out-of-plane displacement, and a plot of the nominal as shown in Figure 2-4.

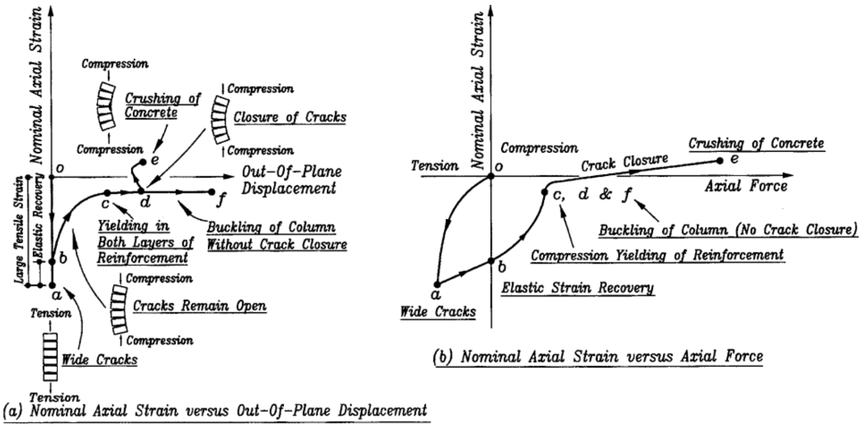


Figure 2-4 Axial reversed cyclic response of reinforced concrete column

They suggested a limiting condition Eq. (2.1) for the maximum tesile strain that may be imposed on a reinforced concrete column while insuring the lateral stability of the column.

$$\varepsilon_{sm} = \frac{\pi^2}{2}\left(\frac{b}{L_o}\right)^2 \xi_c + 3\varepsilon_y \tag{2.1}$$

Where the ε_{sm} is the maximum tensile strain, the b is the wall thickness, L_o is the length of the concrete column, the ξ_c is the critical normalized out-of-plane displacement and ε_y is the yield strain of the reinforcement.

Figure 2-5 shows the plots of the maximum tensile strains, as predicted by Eq (2.1), versus the height-to-thickness ratio, and the explerimentaaly measured peak tensile strains for the specimens with the longitudinal reinforcement ratio $\rho_v = 2.1\%$ and $\rho_v = 3.8\%$, respectively. A kinematic relation between the nominal axial strain and the out-of-plane displacement of the column, represented the end-regions of a ductile panar reinforced concrete wall, were presented. And the Eq (2.1) can predict the lateral buckling failure of the reinforced column according to the maximum tensile strain, but it showed conservative values rather than experimental result.

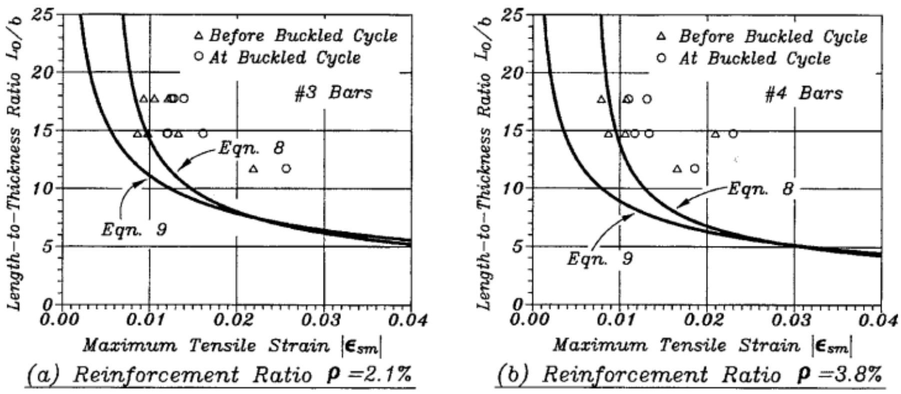


Figure 2-5 Comparison between predicted and experimental maximum tensile strains

2.2.2 Dazio at el. (2009)

Dazio et al. [18] conducted a cyclic lateral loading test to a wall using high-strength rebar(500 MPa). The WSH1 and WSH2 specimens have the same details as the rebar ratio except for the ultimate strength to yield strength ratio of rebar as shown in Figure 2-6. The longitudinal reinforcement ratios of the boundary element and the web are 1.31% and 0.30%, respectively, and the horizontal reinforcement ratios are both 0.25%.

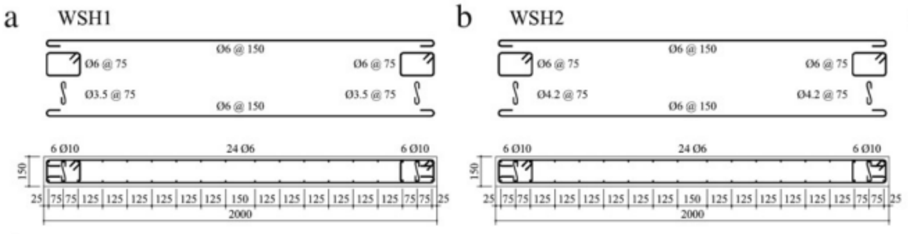


Figure 2-6 Reinforcement layout of WSH1 and WSH2

The strength ratios of the boundary element and the web rebar of WSH1 are 1.13 and 1.03, respectively, and in particular, the web rebar has poor strain-hardening. On the other hand, the strength ratios of the boundary element and web rebars of WSH2 are 1.28 and 1.10, respectively, which have higher ductility than WSH1. The axial force ratio of the specimens is 5%, and the result of the lateral cyclic loading test is shown in Figure 4.5.

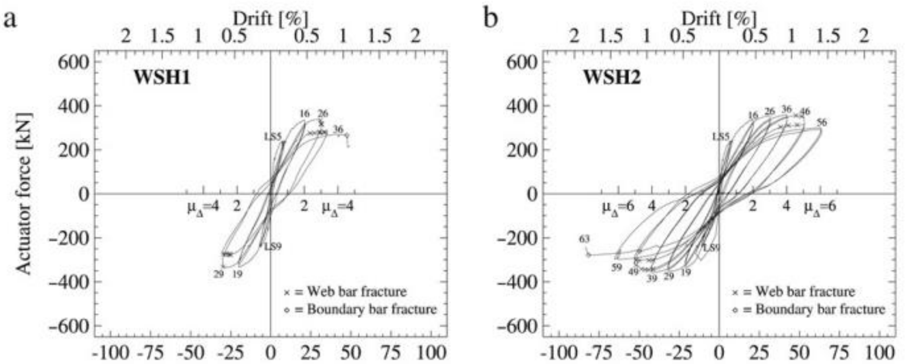


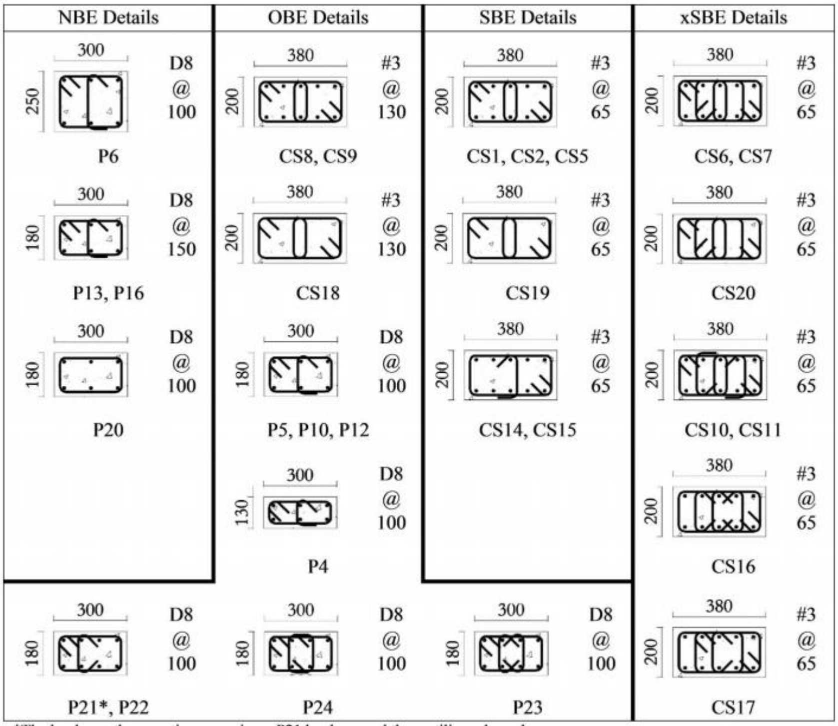
Figure 2-7 Force-displacement hystereses of WSH1 and WSH2

In both specimens, yielding occurred the same at the drift ratio 0.24%, but there was a big difference in the behavior after yielding. In WSH1, where the vertical rebar with the relatively low strength ratio were used, rebar fracture occurred at the drift ratio 0.68% when the vertical rebar of the web exceeded the ultimate strain, and then fracture occurred at the boundary element rebars at 1.04%. On the other hand, in WSH2, where rebar with a relatively high ultimate strength – yield strength ratio was used, the vertical rebars of the web broke at the drift ratio 1.05%, and all boundary element rebars buckled at 1.38%.

Through this, it can be seen that the low strength ratio of the web rebar increases the strain concentration in cracks, causing premature failure and reducing the deformability of the wall. In addition, when a rebar with the high ultimate strength to yield strength ratio was sed in the boundary element, the ductility of the RC wall can be increased.

2.2.3 Welt at el. (2017)

Welt at el. performed the test using rectangular RC prism specimens with experimental parameters including transverse reinforcement ratio, transverse reinforcement detailing configuration, longitudinal reinforcement, tensile strain prior to peak compressive strength, and cross-sectional aspect ratio. Specimen were either tested in reversed cyclic or monotonic loading. Some specimens were loaded in tension prior to commencing the cyclic or monotonic loading protocol. And nearly all of the specimens meet the minimum ACI318-14 detailing requirements for boundary elements of special RC structural walls. In addition, some specimens meet and exceeding the ACI318-14 requirements for special boundary elements as shown in Figure 2-8.



*The hooks on the crosstie on specimen P21 hook around the rectilinear hoop legs

Figure 2-8 Specimen cross section designs

Figure 2-9 shows the compressive response of monotonic tests. All specimens shown in Figure 2-9 were considered to be ordinary boundary elements with a transverse reinforcement ratio 0.5-0.6% in x and y directions. The results didn't show a significant variation in performance. A comparison of specimen P5 with specimens P22, P23 and P24 shows that inclusion of more longitudinal reinforcement may result in a slightly enhanced deformation capacity. However, this could also be attributed to the change in confining steel and is therefore inconclusive. P21 was constructed with the crosstie hooks wrapping around the rectangular hoop, which appeared to be slightly better in terms of peak strength, compared with P22.

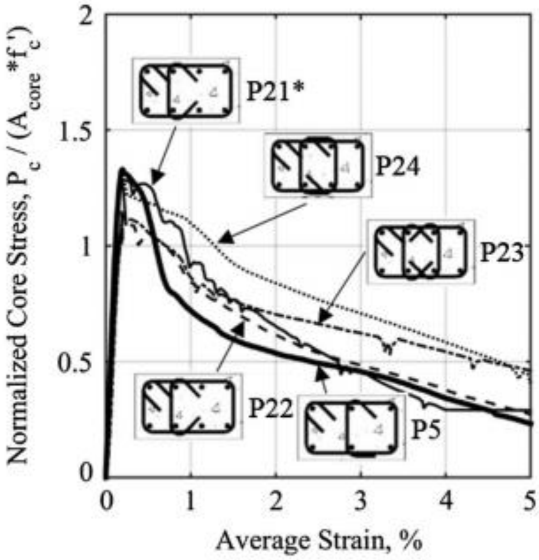


Figure 2-9 Effectiveness of various crosstie configurations

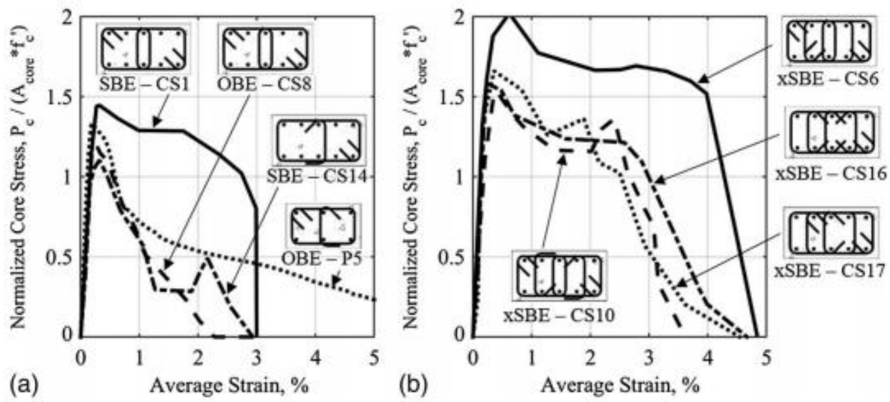


Figure 2-10 Comaprison of crossties versus rectangular hoops

The effectiveness of crossties to restrain longitudinal reinforcement in a manner similar to that of rectangular hoops was presented in Figure 2-10. The ordinary boundary element and special boundary element specimens were shown in Figure 2-10 (a), and the xSBE specimens are shown in (b). In this figure, although specimens CS8 and P5 are both OBE, specimen P5 appeared to perform better. And crossties with 135°–135° standard hooks didn't appear to provide any additional capacity as compared to crossties with 90°–135° standard hooks.

2.2.4 Kim at el. (2021)

To verify the effect of various transverse details and seismic grade vertical rebar on the ductility of the wall, kim at el. conducted a cyclic lateral loading test for RC wall.

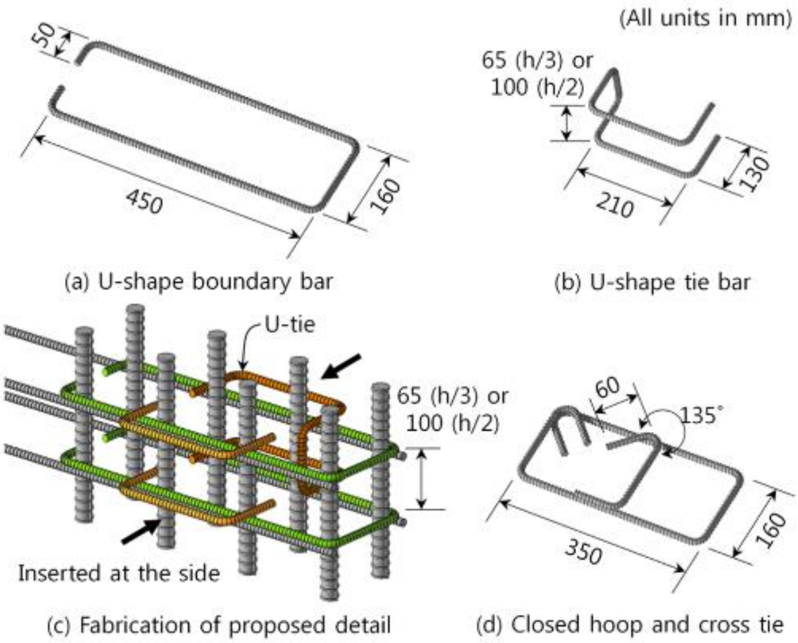


Figure 2-11 Details of boundary confinement reinforcement

The details shown in Figure 2-11 were used for the RC wall boundary elements. And the type of boundary confinement details, spacing of the details, and the type of vertical rebars were used as experimental variables. Figure 2-12 shows the results of the wall test with seismic rebar and non-seismic rebar.

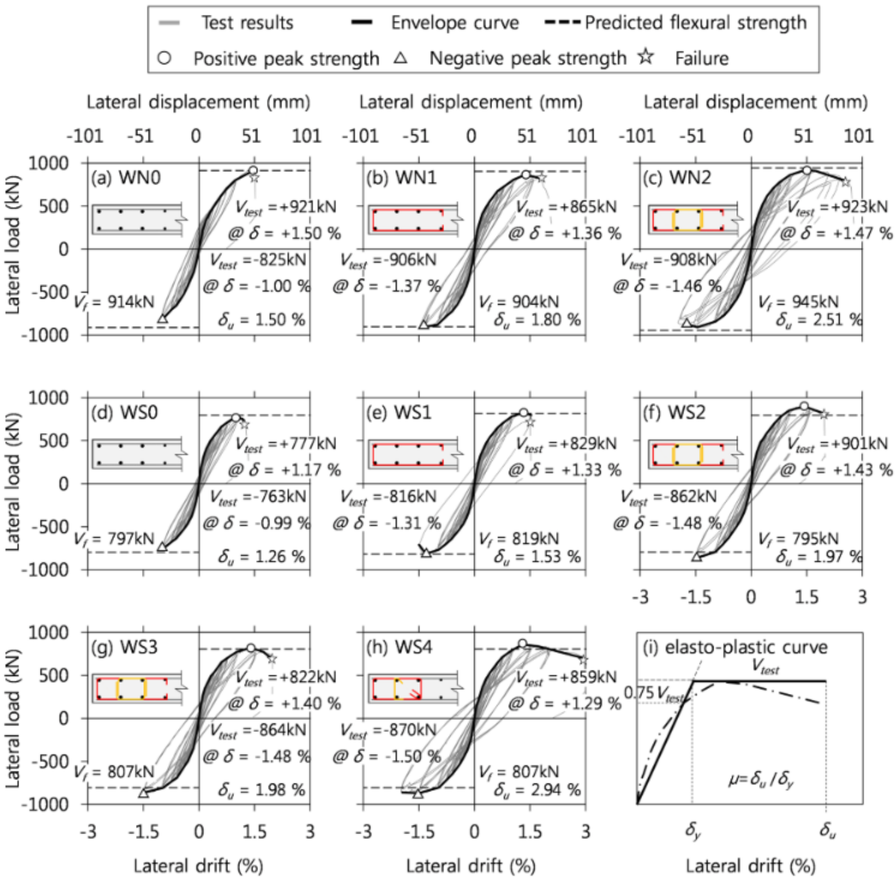


Figure 2-12 Lateral load-displacement relationships of test specimens

As a result of using only U-shpaed boundary bar, the deformation capacity of the WN1 and WS1 was increased by 1.2 times that of the non-confinement detail wall, WN0, WS0. Also, when U-shaped crossties were added, the ductility was improved about 1.6 times compared to the specimens without the transverse reinforcement detail. Special seismic detail specimen using crosstie and closed hoop showed the greatest ductility.

However, due to the high axial force and high longitudinal rebar ratio, the specimens failed by crushing concrete at the compressive zone, and no large inelastic deformation occurred in the flexural rebar of the boundary element. Therefore, it was not possible to compare the ultimate strength to yield strength ratio(TS/YS) as a parameter, and the difference between seismic rebar and non-seismic rebar could not be shown. Therefore, under the above conditions, the effect of proper transverse reinforcement detailing on the wall ductility was important than the ratio(TS/YS) of rebar.

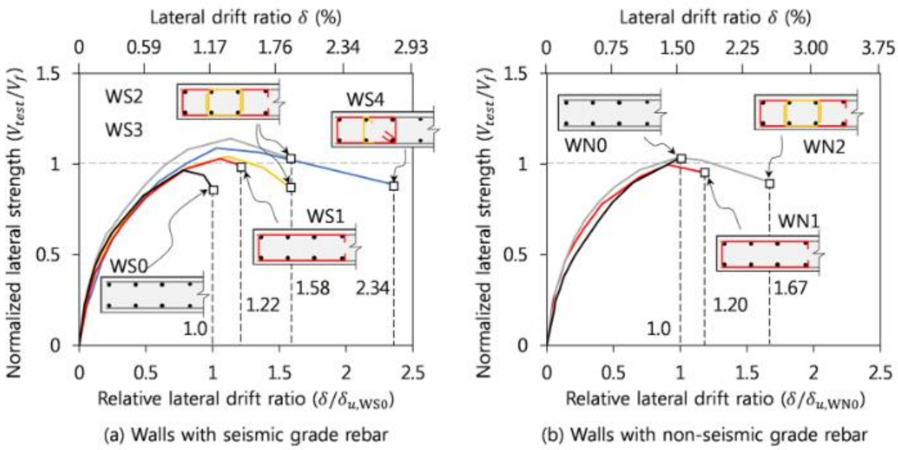


Figure 2-13 Normalized lateral strength-relative lateral drift ratio relationship

Chapter 3. Cyclic Axial Loading Test for RC Wall Boundary Element

3.1 Introduction

Many RC walls that resist seismic loads consist of the web and the boundary elements. The web primarily resists the shear stresses, and the boundary elements is related to the bending performance of the walls. The stress of the boundary elements in the lower part of structural walls, which is used especially in high-rise residential buildings, is almost uniaxial compression-tensile cyclic loading under seismic load. In failure behaviors of the previous research(2019). the deformation capacity of the boundary element is mainly due to the buckling of the rebar and the concrete crushing that occurred while the boundary element at end-region of the specimens was in compressive stress and the transverse reinforcement was pulled out. However, in order to implement the stress of the actual boundary element in a linear state, a super-large experimental specimen is required, and it is difficult to actually realize it.

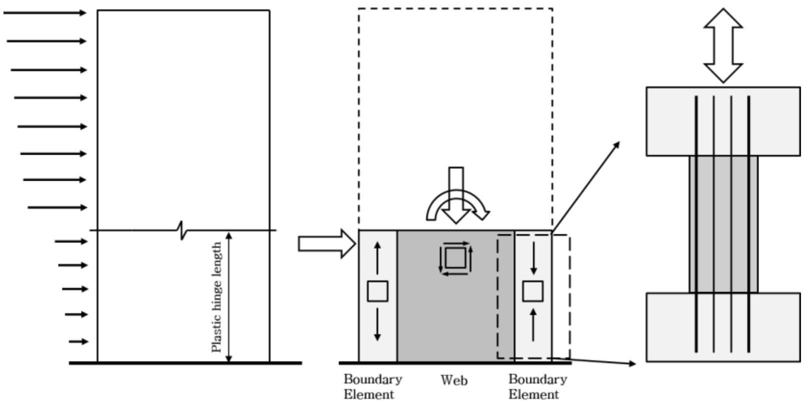


Figure 3-1 Component of RC seismic resisting wall

In this chapter, therefore, the cyclic tension and compression loading test was planned for a boundary element specimen that simulates the boundary element of bearing walls(Figure 3-1). The cyclic axial loading test is to simulate the boundary element of the walls and to quantitatively measure the deformation capacity of the boundary element with lateral confinement details. Through this test, the deformability of the boundary elements was verified, and the ductility of the RC wall with the verified confinement detail was evaluated.

3.2 Development of simplified boundary confinement detail

The walls of high-rise apartment buildings are designed to bend and behave under the control of the overturning moment during earthquake loads. When a lateral load occures due to an earthquake, earthquake load causes the high tension and compression at the boundary elements of the flexure-dominated wall. If the compressive strain at the boundary element of wall due to the compressive force is larger than the ultimate strain of concrete, failure by the concrete crushing occurs and the stability due to the seismic load is dramatically reduced. To prevent the sudden failure and improve the compressive deformation capacity, it is essential to reinforce the boundary element by the lateral confinement details. According to the current Building Seismic Design Code(KDS 41 17 00), the boundary element of RC bearing wall system that is more that 60m in height and belongs to the Seismic Design Category(SDC) D requires seismic reinforced rebar for the longitudinal reinforcement and special seismic details composed of 135 degree closed hoop and cross tie.

However, the wall in korean apartments are as thin as 150~300mm. The special seismic details specified in the design code are too complicated and require excessive lateral confinement details, which degrades constructability and economy. In addition, structural member construction period and labor costs are factors that have a great influence on entire construction cost. Shortening the construction period and improving constructability lead to the reduction of costs, many constructors endeavor to reduce the period of construction through precast or new construction method.

Therefore, the simplified confinement detail to improve constructability and economy will prevent the transverse reinforcement from loosening and enhance the ductility capacity of the RC wall while ensuring sufficient anchorage. In this detail, relatively long U-tie and short U-tie are spot-welded to each other to form a set. This set is assembled by forming modules in the height direction and sandwiching them from the out-of-plane direction of the RC wall. Each U-tie meshes with each other to form a hoop shape, which increase the confined concrete core area while each U-tie ensures sufficient anchorage length. In addition, U-end ties are assembled by putting them into the U-tie set form the end of the wall, and devised to prevent buckling of the outermost vertical rebar(Figure 3-2). Since the simplified confinement detail can be assembled by inserting the pre-fabricated manufactures, the time required to arrange the confinement can be greatly reduced compared to the special seismic details and constructability is increased.

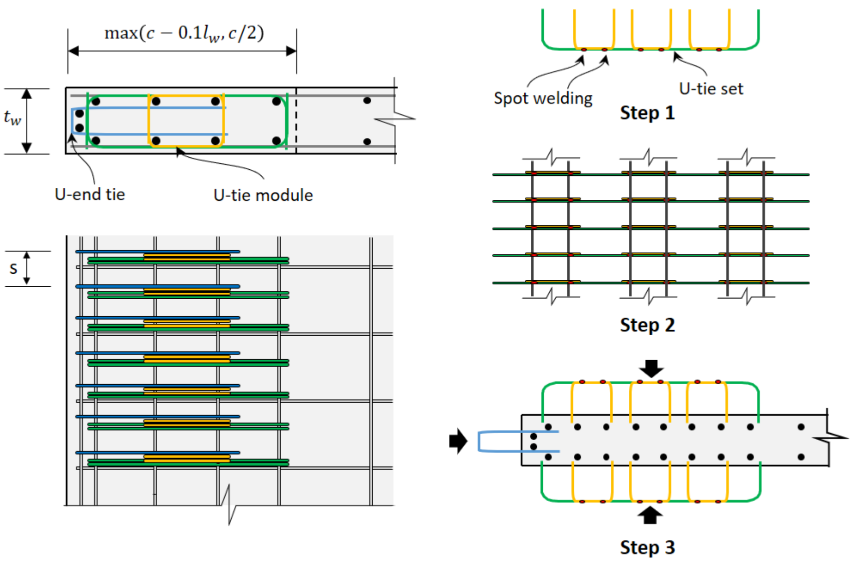
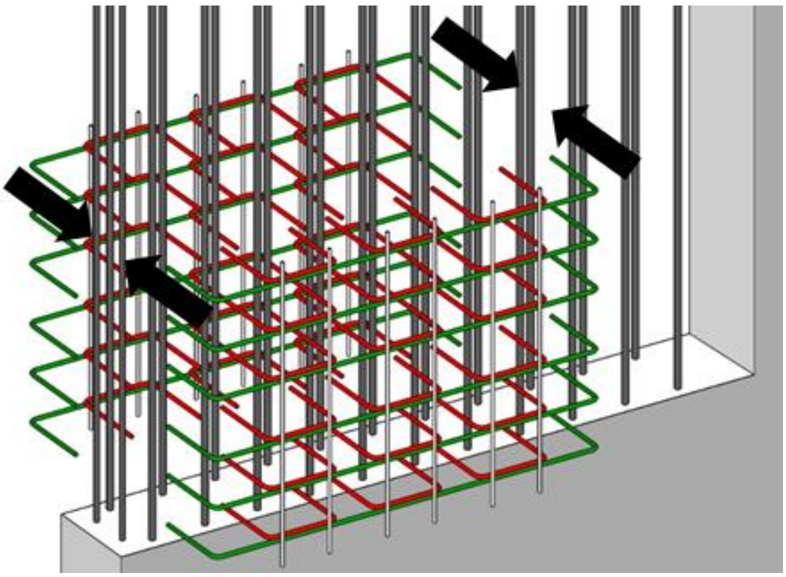
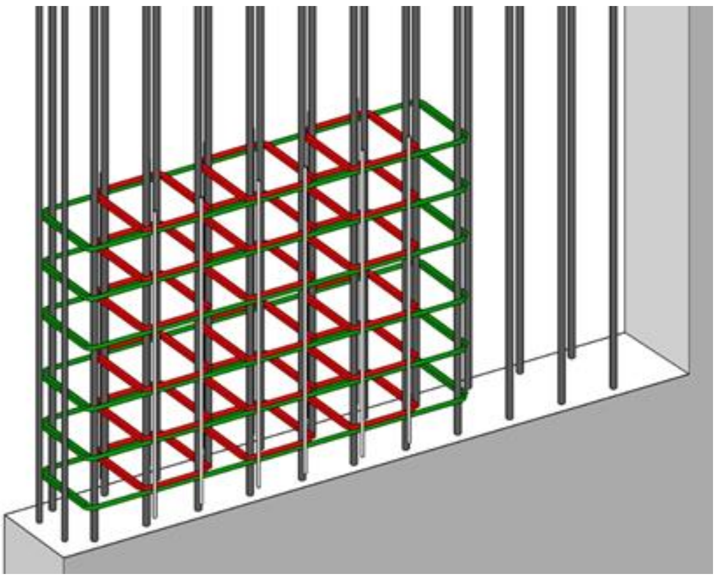


Figure 3-2 Development of simplified confinement detail

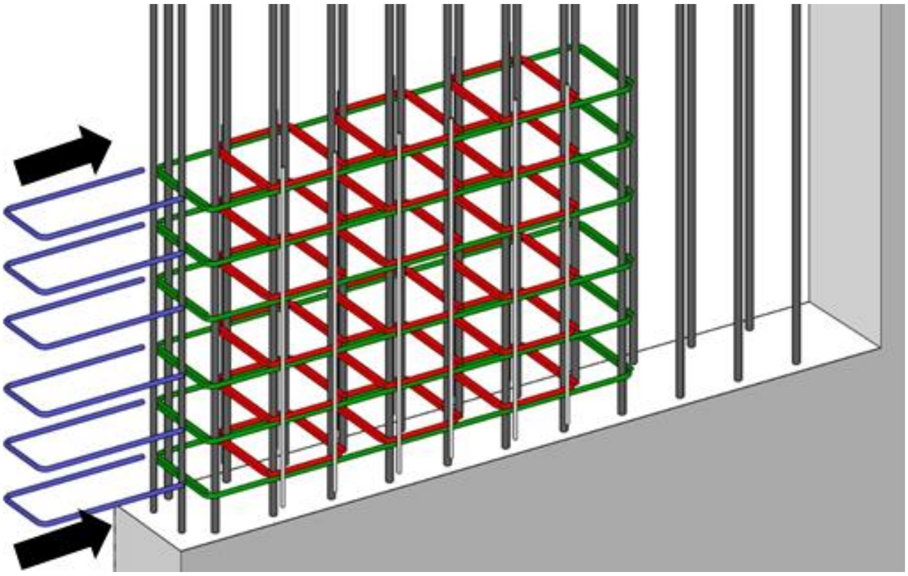


(b)-1

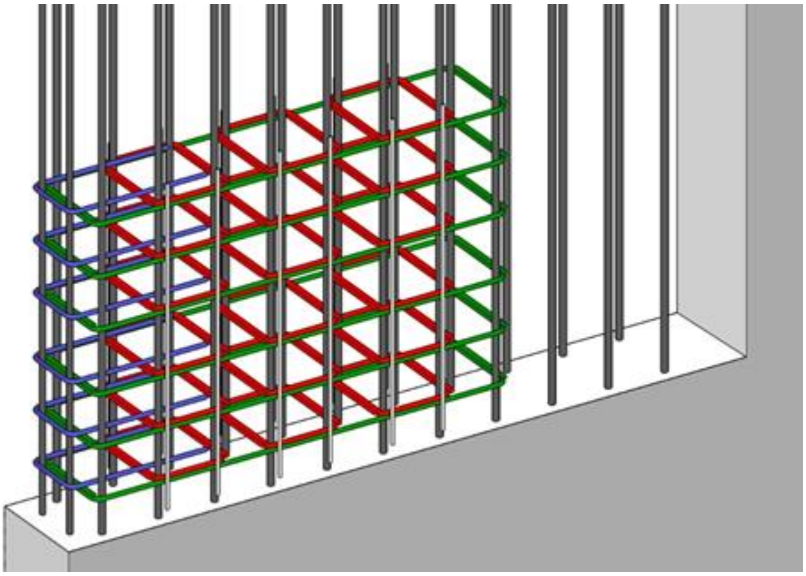


(b)-2

Figure 3-2 Development of simplified boundary confinement



(b)-3



(b)-4

Figure 3-2 Development of simplified boundary confinement

3.3 Test plan

3.3.1 Test variables and details of specimens

Since the large gravity load and seismic load on lower floor of domestic high-rise residential building using reinforced concrete shear wall system, the vertical reinforcement ratio of the wall boundary element is relatively high. in the view of situation, cyclic axial loading test specimens use high vertical reinforcement ratios.

In this experiment, in order to study the performance of the boundary of the wall, assuming that the boundary of the wall on the lateral load is in a uniaxial stress, the specimen is designed so that cyclic axial load can be applied. Total number of the boundary specimen is ten, Table 3-1 shows test variables. To compare the deformation capacity of each confinement type, All test specimens were made at 400 mm(length) x 200 mm(thickness) x 800 mm(height) for direct comparison of each detail. Experimental variables include detail type, vertical spacing of transverse reinforcement, rebar type and loading history.

The names of the specimens in Table 3-1 summarized the experimental variables. The first letter B meant the boundary element specimen. The second letters N and S indicated the type of vertical rebar in Normal rebar and seismic rebar. The numbers indicate the type of the sectional detailing of confinement. Number 0 is boundary element with no detailing, number 1, 2 were boundary element with simplified confinement detail at a spacing of two-third and half of the thickness, i.e., 130 mm and 100 mm. Number 3 is boundary element specimen in which the detailing of the special shear wall system specified in

Korean Buildings Seismic Design Code(KDS 41 17 00) was arranged at a spacing of one-third of the thickness, 65 mm, and it is composed of a 135 degree closed hoop and cross tie.

T1 and T2 show loading history of the specimens. T1 shows the ratio of compressive strain to tensile strain 1:5, and T2 shows the ratio 1:10. The ratio compressive strain to tensile strain is related to the compression zone depth of the wall on the earthquake load. Section analysis using a fiber model was performed to calculate the strain distribution and the compression zone depth. Prototype residential building walls of 5m and 10m were selected, and vertical rebar concentrated at the boundary element of wall. Nonlinear section analysis using a fiber model can obtain strain distribution, stress, compressive force, tensile force. In non-linear section analysis using the fiber model, it is possible to easily obtain strain, stress, compressive force, tensile force generated in each fiber without complicated mathematical calculation such as solving a higher-order equation. For nonlinear section analysis, it is assumed that the cross section before and after applying the load remains flat and the cross-section strain distribution is linear. It is also assumed that vertical rebar and concrete are completely bonded and strain of concrete and rebar fibers at the same position is equal. Concrete fiber was divided into 50mm width in section and the rebar was arranged in the same position as the actual wall section.

The strain distribution of the wall cross section due to lateral load changes according to the vertical reinforcement ratio. T1 replicated the strain distribution of a wall over 5 m while having higher rebar ratio, and T2 replicated the strain distribution when a wall with rebar ratio of less than 1% was subjected to lateral load. T1 and T2 simulated each wall with compression zone depth of 1/6 and 1/11 of the wall length.

concrete compressive strength was designed to be 30MPa. The strength became almost uniform at 27.1 to 31.7MPa except for the BS3-T2 specimen. Because of long concrete curing period, BS3-T2 had relatively high strength compared to other specimens. The vertical reinforcement with nominal strength of 600MPa and the transverse rebar with nominal strength of 400MPa was used for all specimens. The specimens to which the lateral confinement was not applied used normal reinforcement and seismic reinforcement of D22, D16. The specimens to which the special seismic detailing was applied used seismic reinforcement of D16 and D22. The Specimens with the simplified confinement detail used only seismic reinforcement of D16. and all specimens used normal reinforcement as transverse rebar. The seismic reinforcement(SD600S) had above an yield ratio standard of 1.25(tensile strength/yield strength = 1.29 - 1.34). On the other hand, the yield ratio of normal reinforcement(SD600) is 1.15-1.17. Since the boundary element cyclic axial loading test idealizes the end region of the wall on the lateral load, the vertical rebar arrangement was different for each specimen, but the rebar ratio was designed to be almost the same, and the lateral confinements were designed differently as a test variable.

The longitudinal rebar configuration of the specimens is as follows in Figure 3-3 to Figure 3-6. In the specimens(BS0, BS3) with no transverse reinforcement details and special seismic detail, Total 8 of rebar were placed at 100 mm intervals, D22 was placed on both outer sides, and D16 was placed on the center. The specimens(BS1,BS2) with simplified confinement detail used the the rebar configuration different from that of BS0 and BS3, total 12 of vertical rebar were placed. Eight vertical rebars were placed on both outer sides at 40mm intervals, and four rebars were placed at 80mm intervals in the center. The spcaing of between the lateral confinement is 130mm(2t/3) for the BS1 specimen, 100mm(t/2) for the BS2 specimen, and 65mm(t/3) for the BS3 specimen. The yield strength and tensile strength of the reinforcement are shown in Table 3-2.

The nominal compressive strength P_{nc} and nominal tensile strength P_{nt} of the specimen were calculated based on the following current KBC2016.

$$P_{nc} = 0.85 f_{ck}\left(A_g - A_s\right) + f_y A_s \tag{3.1}$$

$$P_{nt} = f_y A_s \tag{3.2}$$

Figure 3-3 shows the specimens with the no transverse reinforcement detail, BN0 and BS0. Due to idealize the end-region of the wall, the length and the thickness of the specimen was determined as 400mm and 200mm. the transverse rebar resisting the shear force was not placed because it received only compression and tension. The nominal strengths of BN0-T1 and BN0-T2 are 3389kN and 3521kN, respectively, and the nominal tensile strengths are 1573kN. The nominal compressive strength of BS0-T1 and BS0-T2 are 3296kN and 3521kN, respectively, and the nominal tensile strengths are 1494kN. The difference in strength was shown due to the difference in compressive strength of concrete and the yield strength of normal rebar and seismic rebar.

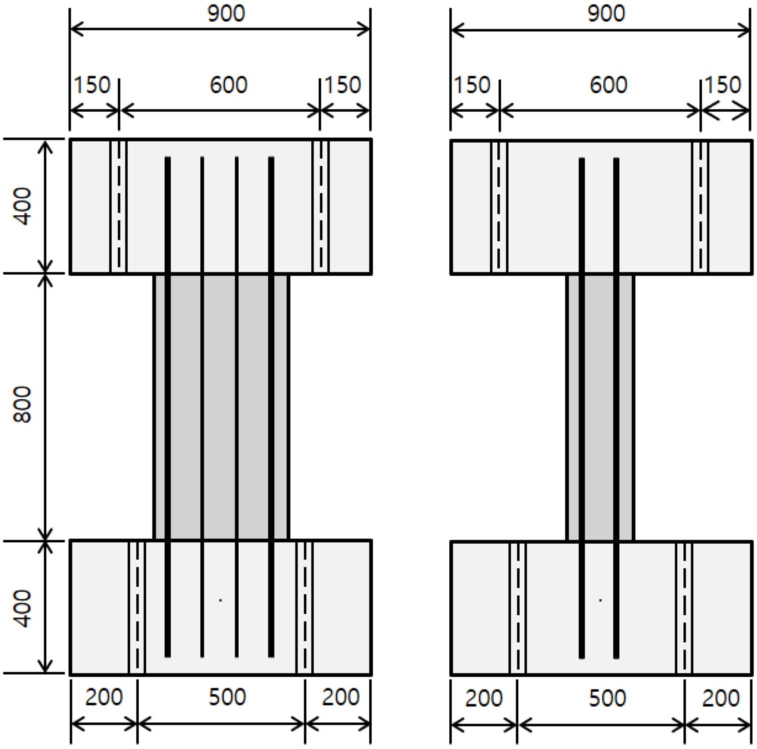
It was assumed that difference between the compressive strength and the tensile strength was that the compressive force due to the gravity load acts on the wall. The difference between the nominal compressive strength and the nominal tensile strength of the boundary element specimen represents about 15% of the nominal compressive strength of the cyclic lateral loading test specimen. However, the difference between the compressive and tensile strength, which reflects the results of the cyclic axial loading test for the boundary element, was about 10% of the nominal compressive strength of the wall specimen.

Figure 3-4 and Figure 3-5 show the cross sections of BS1-T1, BS1-T2, BS2-T1 and BS2-T2 with simplified confinement detail. This specimen was designed to evaluate the effects of prefabricated detail U-shaped tie and U-end tie. As shown in the figure, a long U-shaped tie and a short U-shaped tie were welded to each other and assembled in both out-of-plane direction. U-shape ties engaged with each other to form a single hoop that increases the confinement effect of concrete. Unlike the existing special seismic detail, there was no 135-degree seismic hook in the detail, so it can be easily assembled. In addition, it is possible to make the detail in one module and assemble them at the same time instead of individually assembling them. The U-end tie also has no 135-degree seismic hook and can be easily assembled in in-plane diction of specimen. It also prevents the local buckling and enhances the confinement of concrete.

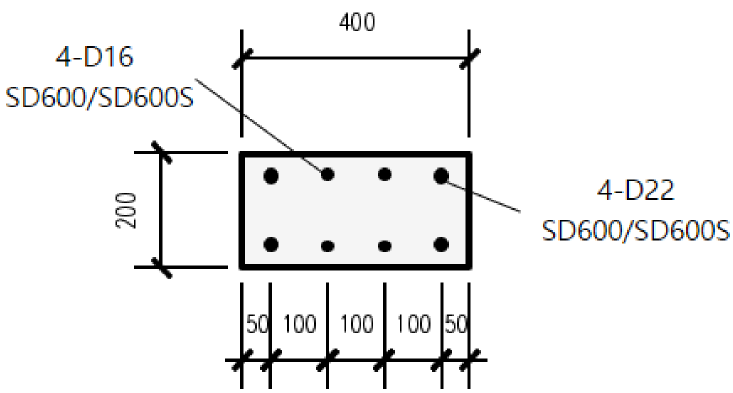
Twelve long U ties, short U ties, and U-end ties were arranged on BS1. Since BS2 had a narrower vertical spacing than BS1, the 16 details that make up the simplified confinement detail were arranged. The nominal compressive strengths of BS1-T1 and BS1-T2 were 3449kN and 3475kN, and the nominal tensile strength was 1522kN, respectively. The nominal compressive strength

P_{nc} of BS2-T1 and BS2-T2 were 3541kN and 3614kN, and the nominal tensile strengths P_{nt} were 1522kN, respectively.

Figure 3-6 shows the cross-sections of BS3-T1 and BS3-T2 with the special seismic detail specified by the Building Seismic Building Code(KDS 41 17 00). For the BS3 specimens, closed hoops and cross-ties were used and 135-degree hooks were installed for closed hoop according to the design code. It is difficult to assemble these details because it must be inserted from the top of the vertical rebar, making difficult to insert them if the vertical rebar was already arranged. The vertical spacing of the special transverse reinforcement was 65mm, one-third of the wall thickness. The nominal compressive strengths P_{nc} of BS3-T1 and BS3-T2 were 3567 kN and 3838 kN, and the nominal tensile strengths P_{nt} were 1494 kN, respectively.

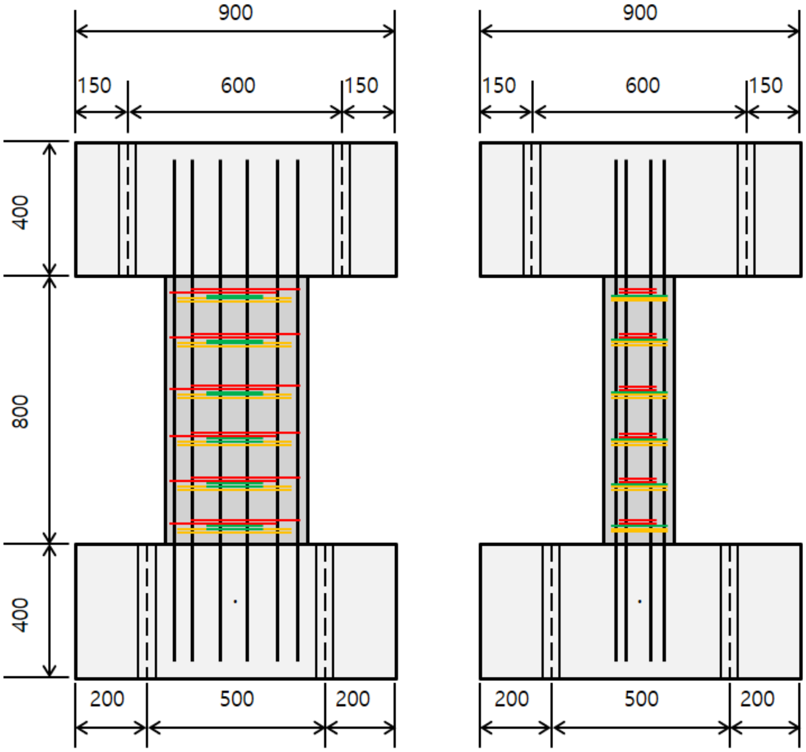


(a) Elevation of **BN0** and **BS0**

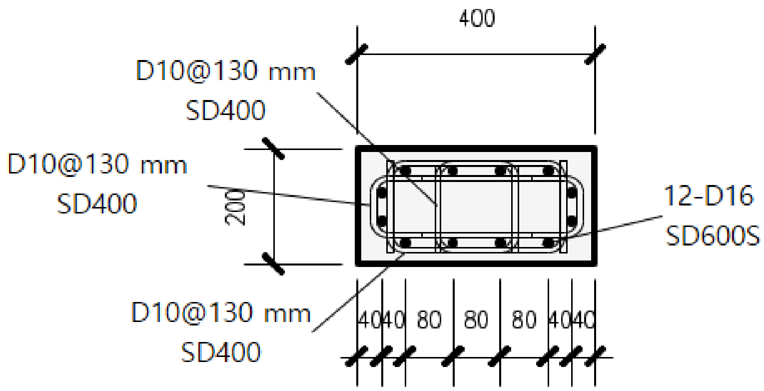


(b) Sectional Detail of **BN0** and **BS0**

Figure 3-3 Detail of BN0 and BS0

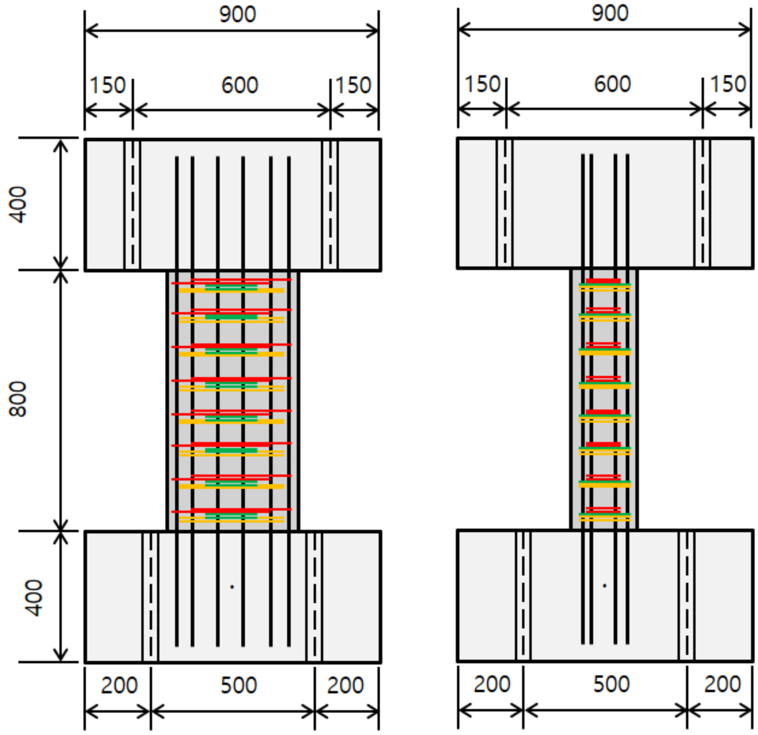


(a) Elevation of **BS1-T1** and **BS1-T2**

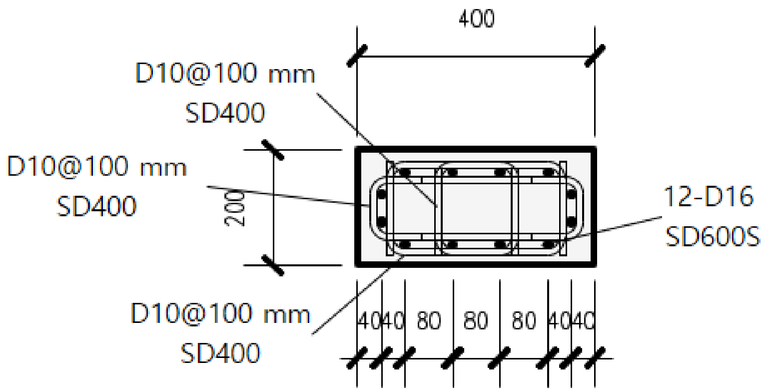


(b) Sectional Detail of **BS1-T1** and **BS1-T2**

Figure 3-4 Detail of BS1

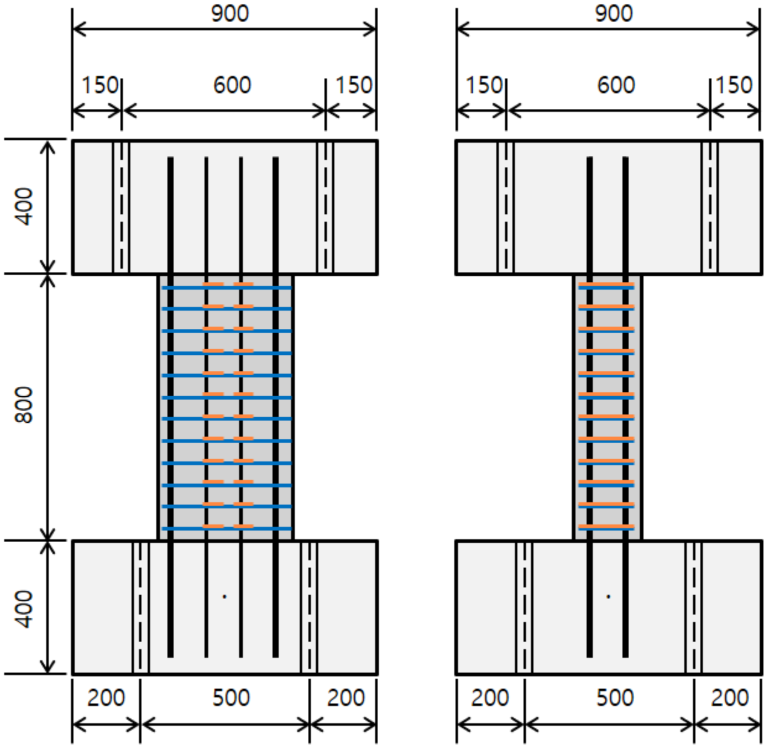


(a) Elevation of **BS2-T1** and **BS2-T2**

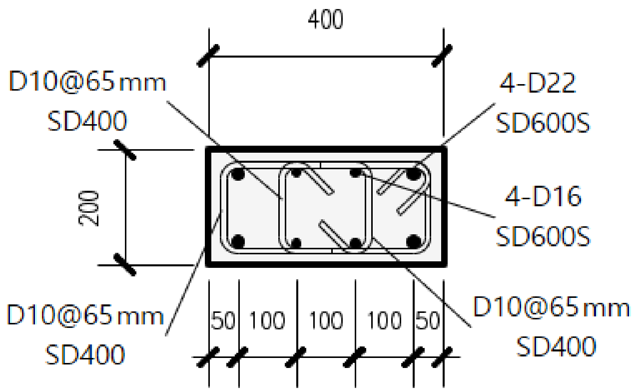


(b) Sectional Detail of **BS2-T1** and **BS2-T2**

Figure 3-5 Detail of BS2



(a) Elevation of **BS3-T1** and **BS3-T2**



(b) Sectional Detail of **BS3-T1** and **BS3-T2**

Figure 3-6 Detail of BS3

(a) Long U-tie

(b) Short U-tie

(c) U-end tie

(d) Module of U-tie

(e) Closed hoop

(f) Cross-tie

Figure 3-7 Transverse reinforcement details

Table 3-1 Test parameters

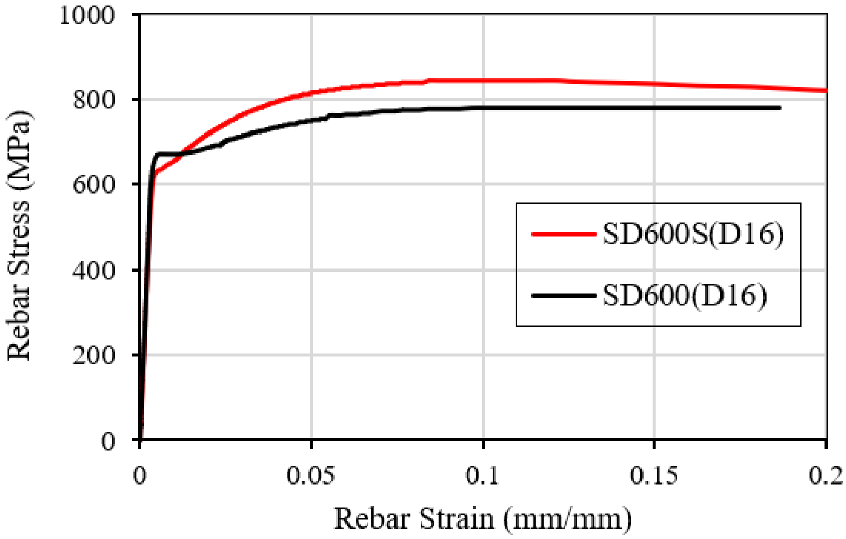
Speci-mens	Hysteric Behavior	Rebar Type	Concrete Strength f_c' [MPa]	Reinforcement: Boundary region: Horizontal f_{yh} [MPa]	Horizontal ρ_h [%]	Vertical f_{yb} [MPa]	Vertical ρ_b [%]	Re-bar Detail	Strength prediction: Compression strength P_C [kN]	Tension strength P_t [kN]	$\frac{P_c}{P_t}$
BN0-T1	T1	SD600	27.5	-	-	673	2.93	-	3389.2	1573.4	2.15
BS0-T1		SD600S	27.3	-	-	638	2.93	-	3296.3	1493.7	2.21
BS1-T1			29.2	491	1.23	639	2.98	U end bar + U cross tie	3448.5	1522.1	2.27
BS2-T1			30.6	491	1.62	639	2.98		3540.9	1522.1	2.33
BS3-T1			31.4	491	1.98	638	2.93	Special seismic detail	3567.0	1493.7	2.39
BN0-T2	T2	SD600	29.5	-	-	673	2.93	-	3521.3	1573.4	2.24
BS0-T2		SD600S	30.7	-	-	638	2.93	-	3520.8	1493.7	2.36
BS1-T2			29.6	491	1.23	639	2.98	U end bar + U cross tie	3474.9	1522.1	2.28
BS2-T2			31.7	491	1.62	639	2.98		3613.5	1522.1	2.37
BS3-T2			35.5	491	1.98	638	2.93	Special seismic detail	3837.7	1493.7	2.57

Table 3-2 Summary of reinforcement properties

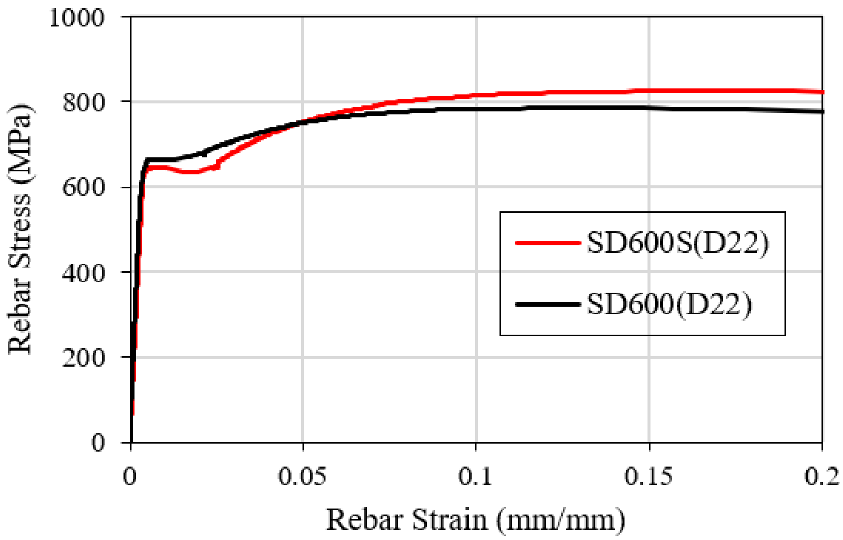
Rebar type		Yield Strength [MPa]	Tensile Strength [MPa]	Tensile to Yield Ratio	Ratio Average
D16 (SD600)	rebar1	678	778	1.147	1.154
	rebar2	674	784	1.163	
	rebar3	674	776	1.151	
D22 (SD600)	rebar1	675	792	1.173	1.177
	rebar2	668	787	1.178	
	rebar3	666	785	1.179	
D16 (SD600S)	rebar1	637	849	1.333	1.330
	rebar2	632	844	1.335	
	rebar3	647	855	1.321	
D22 (SD600S)	rebar1	635	828	1.304	1.292
	rebar2	643	826	1.285	
	rebar3	633	815	1.288	

Table 3-3 Summary of concrete properties

Specimens	Hysteric Behavior	Concrete Strength f_c' [MPa]	Concrete Strain at Maximum Stress
BN0-T1	T1	27.5	0.00214
BS0-T1		27.3	0.00302
BS1-T1		29.2	0.00273
BS2-T1		30.6	0.00185
BS3-T1		31.4	0.00368
BN0-T2	T2	29.5	0.00259
BS0-T2		30.7	0.00304
BS1-T2		29.6	0.00205
BS2-T2		31.7	0.00342
BS3-T2		35.5	0.00291



(a) D16 reinforcement



(b) D22 reinforcement

Figure 3-8 Reinforcement strain-stress relationship

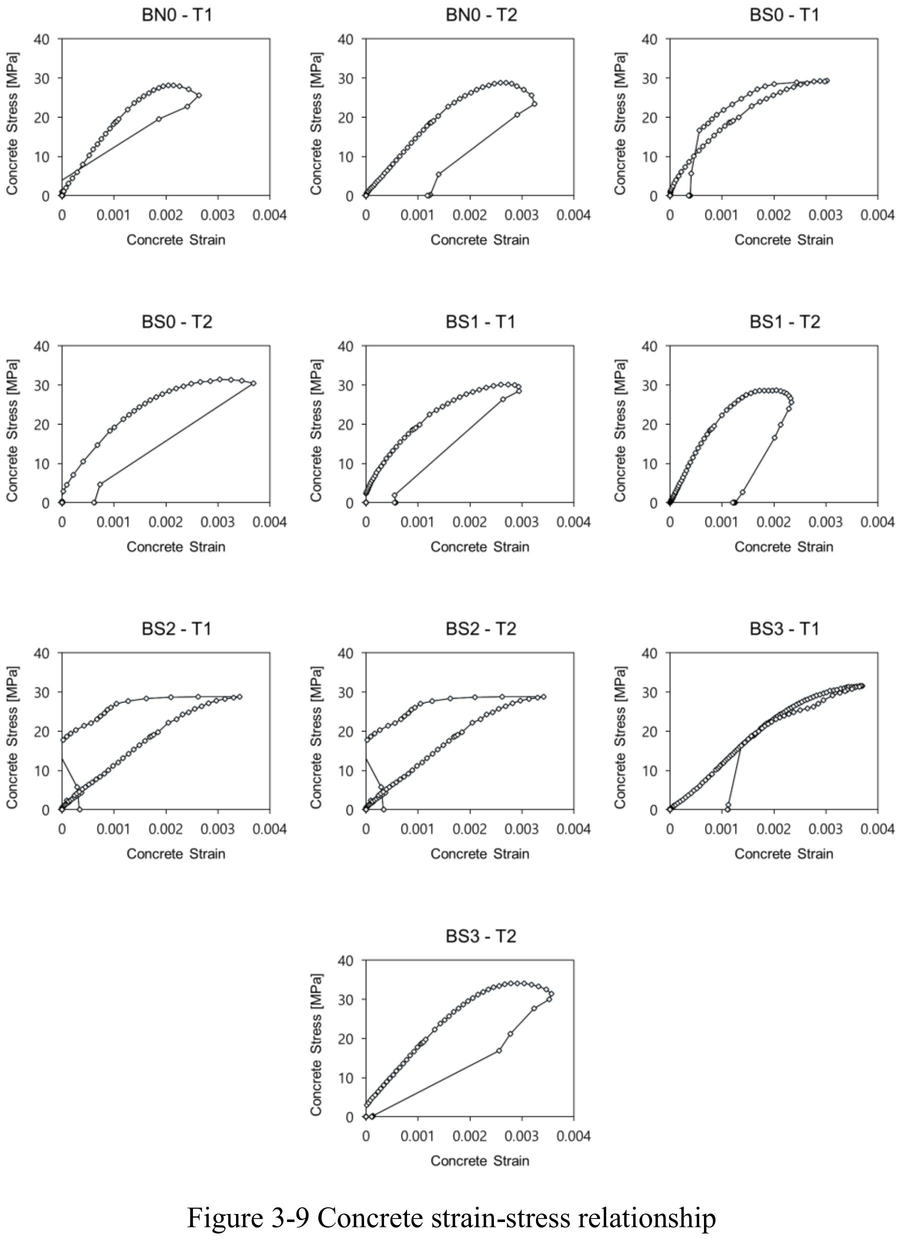


Figure 3-9 Concrete strain-stress relationship

3.3.2 Test setup and loading plan

Figure 3-10 represents the loading method of the specimen, the setup for displacement measurement and Linear Variable Displacement Transformers(LVDTs). 5000kN Universal Testing Machine(UTM) was used, and after fixing the specimen on the upper and lower parts of the UTM, cyclic axial loading was applied to the upper head of the specimen. The displacement due to the applied force was recorded every 4 seconds, and the cyclic load was applied by displacement control until the specimen was destroyed. Concrete compressive strength was determined by the average strength of the three concrete cylinder specimens. Figure 4-8 and 4-9 show the axial loading plans T1 and T2, which idealize the strain history of the wall boundary element on the earthquake load, and compressive strain : tensile strain was planned as 1:5 and 1:10. total displacement of the specimen(L1-L2) was measured using LVDTs. The axial deformation of the specimen was measured separately for the upper, central, and lower part(L3-L8). In addition, the displacement in the out-of-plane direction of the specimen was measured for the upper, central, and lower part(L9-L11), respectively.

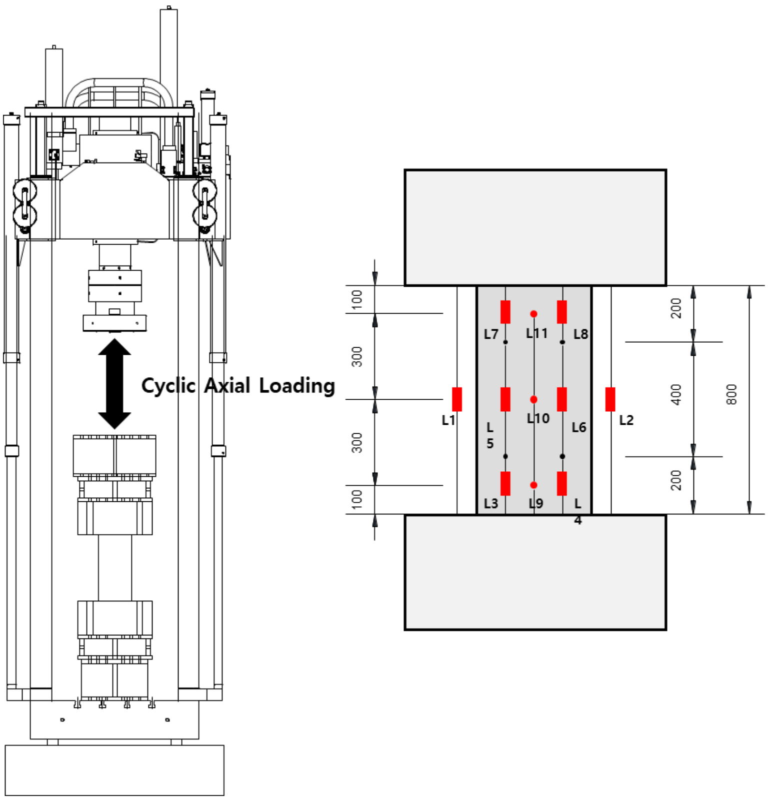


Figure 3-10 Test setup of specimen and LVDTs plan

Figure 3-11 Test setup

Table 3-4 Load protocol - T1

Load Step	Compressive strain	Compressive Deformation [mm]	Tensile Strain	Tensile Deformation [mm]
STEP1	-0.00025	-0.2	0.00125	1
STEP2	-0.0005	-0.4	0.0025	2
STEP3	-0.00075	-0.6	0.00375	3
STEP4	-0.001	-0.8	0.005	4
STEP5	-0.0015	-1.2	0.0075	6
STEP6	-0.002	-1.6	0.01	8
STEP7	-0.003	-2.4	0.015	12
STEP8	-0.004	-3.2	0.02	16
STEP9	-0.006	-4.8	0.03	24
STEP10	-0.008	-6.4	0.04	32
STEP11	-0.01	-8	0.05	40
STEP12	-0.015	-12	0.075	60

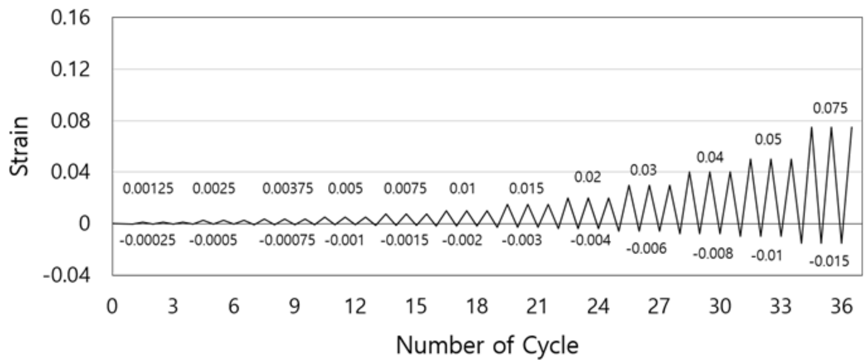


Figure 3-12 Load Protocol - T1

Table 3-5 Load protocol - T2

Load Step	Compressive strain	Compressive Deformation [mm]	Tensile Strain	Tensile Deformation [mm]
STEP1	-0.00025	-0.2	0.0025	2
STEP2	-0.0005	-0.4	0.005	4
STEP3	-0.00075	-0.6	0.0075	6
STEP4	-0.001	-0.8	0.01	8
STEP5	-0.0015	-1.2	0.015	12
STEP6	-0.002	-1.6	0.02	16
STEP7	-0.003	-2.4	0.03	24
STEP8	-0.004	-3.2	0.04	32
STEP9	-0.006	-4.8	0.06	48
STEP10	-0.008	-6.4	0.08	64
STEP11	-0.01	-8	0.1	80
STEP12	-0.015	-12	0.15	120

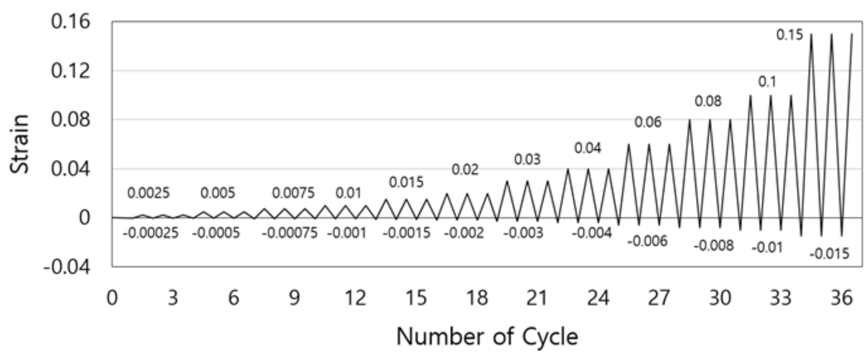


Figure 3-13 Load protocol - T2

3.3.3 Specimen

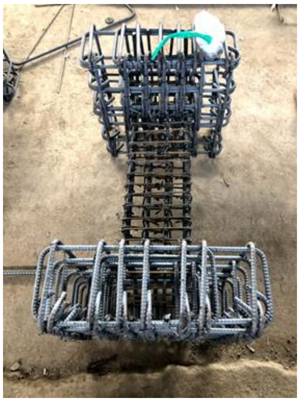
The size of the specimens is as follow. Boundary element was 400 mm(length) x 800 mm(height) x 200 mm(thickness), and the foundation and head were 900 mm(length) x 900 mm(thickness) x 400 mm(height). After assembling the foundation rebar, aligning the vertical rebar and placing the confinement details, a rebar strain gauge was attached. Because it was an axial loading test, concrete pouring was carried out at once for integrated behavior. In order to maximize the area of confined concrete, the thickness of the concrete cover for the boundary element specimen was designed to be 20mm. A 20mm spacer was installed between the formwork and the vertical rebar to maintain the cover thickness before concrete pouring. The compressive strength of concrete poured in the boundary element, the head part and foundation was 30MPa. The mixture ratio of concrete was cement 393 kg/m^3, water 193kg/m^3, fine aggregate 812 kg/m^3, coarse aggregate 926kg/m^3 and admixture 3.93 kg/m^3. And the concrete was an ordinary ready mixed concrete using Portland cement type 1 with the water-admixture ratio 44.0% and the aggregate ratio 48.5%. Table 3-3 shows the actual concrete compressive strength, and Table 3-2 shows the actual yield strength of the reinforcement used in specimen.

Figure 3-14 shows the manufacturing process of the specimens(See Figure 3-14 (a)-(f)). (a) Rebar processing, (b) assembling the rebar for foundation, head, boundary element, (c) fabrication and assembly of transverse details, (d) attaching the strain gauge, (e) concrete pouring and curing, (f) Completed specimen.

(a) Rebar processing

(b) Assembling the rebar for foundation, head and boundary element

(c) Fabrication and assembly of transverse details



(d) Attaching straing gauge

(e) Concrete pouring and curing

(f) Completed specimen

Figure 3-14 Test specimen construction procedure


(a) BN0 and BS0


(b) BS1 with simplified confinement detail


(c) BS2 with simplified confinement detail


(d) BS3 with special seismic detail

Figure 3-15 Arrangement of transverse reinforcement

3.4 Test result

3.4.1 Failure mode

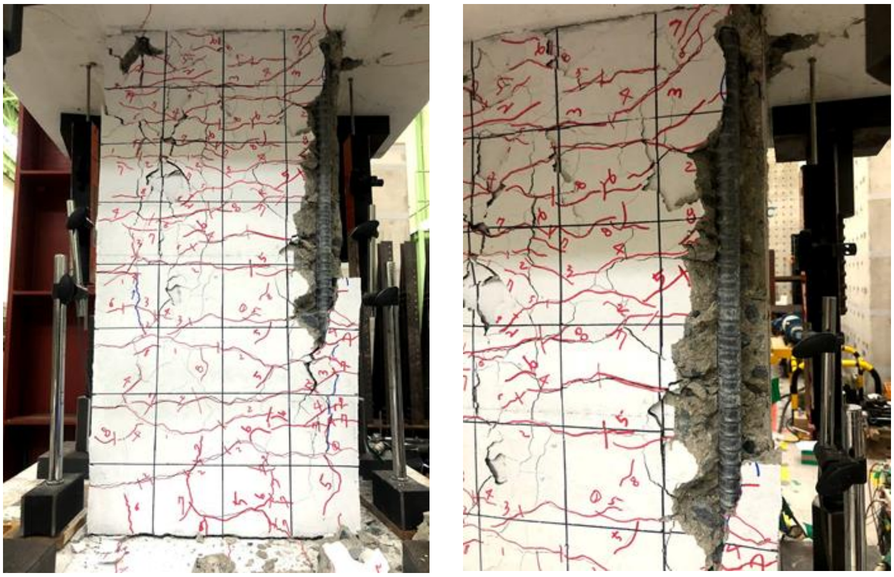
The failure mode of specimens is shown in Figure 3-16 to Figure 3-25, and Table 3-6 represents the final failure mode of the specimens and test results. The failure modes of the specimen were variously shown as concrete crushing, local buckling of rebar, and out-of-plane buckling. In BN0-T1 and BN0-T2 using normal reinforcement, where there is no the confinement detail, the concrete crushing occurred and buckling of the vertical rebar occurred together(Figure 3-16), and specimens were destroyed at the strength that is less than the nominal strength. This indicates that the brittle failure of concrete may occur in the compression-dominated sate without the moderate confinement details.

On the other hand, specimens BS0-T1 and BS0-T2 using seismic rebar, where there is no the confinement detail, were destroyed due to the buckling of the vertical rebar, unlike the specimens BN0 using normal rebar(Figure 3-17). In the case of BS0-T1, the rebar buckling occurred at the outermost rebar. Since BS0-T1 doesn’t have transverse reinforcements to restrain vertical rebar, it was destroyed by local buckling and spalling of concrete. Compared to BS0-T1, BS0-T2 was destroyed by buckling of whole specimen, not buckling of the outer rebar(Figure 3-22). The buckling of the rebar occurred suddenly, causing brittle failure of the specimen.

In specimen BS1-T1 and BS1-T2 with simplified confinement detail placed at 130mm intervals, buckling occurred in the vertical rebar between the transverse reinforcement(Figure 3-18). In BS1-T1, the transverse rebar didn't fall off, and the strength of the specimen dropped sharply due to the spalling of concrete along with the buckling of the rebar at the top. In BS1-T2, concrete spalling occurred along with buckling of the rebar at the bottom of the specimen(Figure 3-23). No drop-off of the rebar occurred, and buckling occurred between the transverse details. After the vertical rebar buckling, it fractured.

In the specimen BS2-T1 and BS2-T2 with simplified confinement detail placed at 100mm intervals, the fracture of the longitudinal rebar preceded and buckling occurred in both specimens. Fracture occurred between the transverse reinforcement, and the compressive load after the fracture caused the U-tie and U-end tie to lose their anchorage and buckling occurred(Figure 3-19). In BS2-T1, rebar buckling of out-of-plane and in-plane directions occurred together at the center, and the transverse reinforcement in the buckling area were dislocated from its position(Figure 3-24). BS2-T2 was preceded by the fracture of the vertical rebar at outer part of the lower. Buckling occurred as the anchorage of U-end tie was released in the part where the fracture occurred, and the specimen was destroyed when the concrete fell off.

Rectangular closed hoop and cross-tie of BS3 did not loosen or fracture after failure and well constrained core concrete of specimens. In BS3-T1, out-of-plane buckling occurred instead of local buckling, and the 90-degree hook of cross tie at the buckling area was released(Figure 3-20), but no major failure occurred in that part. However, in BS3-T2, the cross-tie at the area of failure was unraveled and fracture occurred, causing out-of-plane buckling, not local buckling of vertical rebar, and the strength decreased sharply(Figure 3-25).



(a) Crack (b) Failure mode

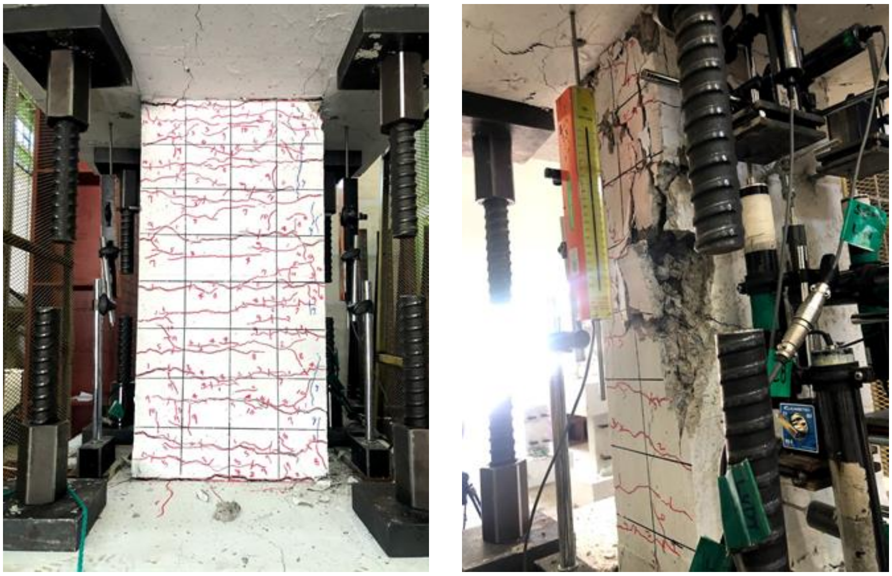
Figure 3-16 Crack and failure mode of BN0-T1

(a) Crack (b) Failure mode

Figure 3-17 Crack and failure mode of BS0-T1

(a) Crack (b) Failure mode

Figure 3-18 Crack and failure mode of BS1-T1



(a) Crack (b) Failure mode

Figure 3-19 Crack and failure mode of BS2-T1


(a) Crack

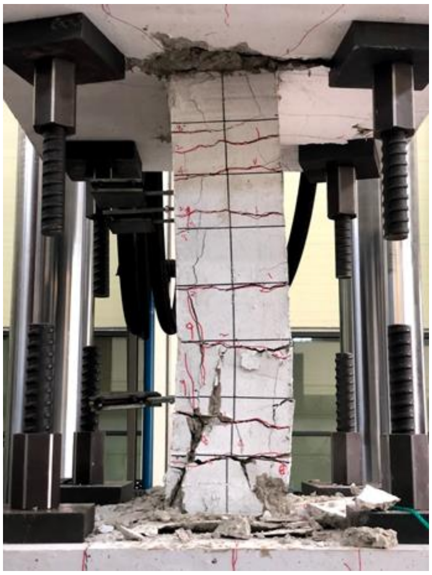
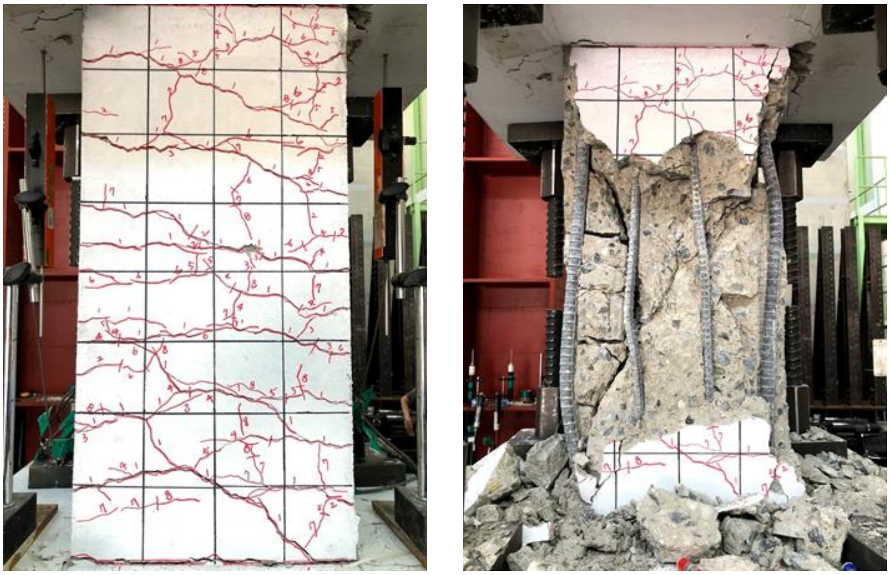

(b) Failure mode

Figure 3-20 Crack and failure mode of BS3-T1


(a) Crack


(b) Failure mode

Figure 3-21 Crack and failure mode of BN0-T2

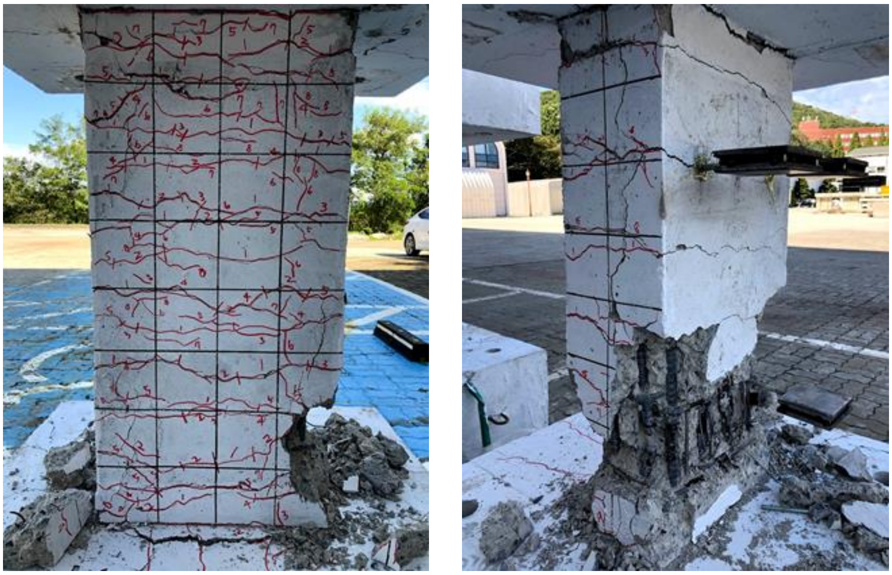


(a) Crack (b) Failure mode

Figure 3-22 Crack and failure mode of BS0-T2

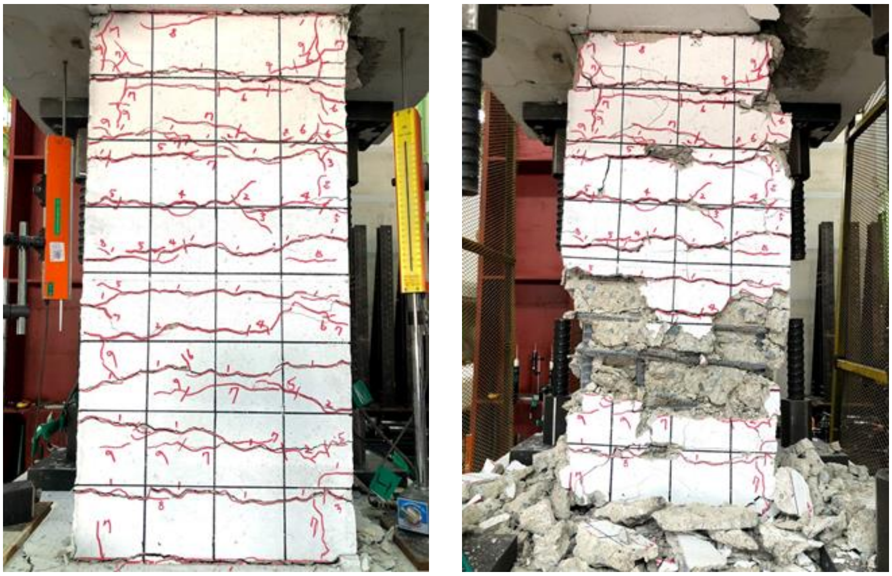
(a) Crack (b) Failure mode

Figure 3-23 Crack and failure mode of BS1-T2



(a) Crack (b) Failure mode

Figure 3-24 Crack and failure mode of BS2-T2



(a) Crack (b) Failure mode

Figure 3-25 Crack and failure mode of BS3-T2

Table 3-6 Summary of test result

Specimen	f_c' [MPa]	P_{nt} [A_s*f_y]	P_{nc1} [0.85(A_g-A_s)*f_{ck}+A_s*f_y]	P_{nc2} [0.85(A_{core}*f_{ck})+A_s*f_y]	P_{t_test} [kN]	P_{c_test} [kN]	P_{c_test}/P_{nc1}	P_{c_test}/P_{nc2}	P_{t_test}/P_{nt}
BN0-T1	27.5	1573.4	3389.2	-	1653	2631	0.78	-	1.05
BS0-T1	27.3	1493.7	3296.3	-	1717	3349	1.02	-	1.15
BS1-T1	29.2	1522.1	3448.5	2728.7	1762	3327	0.96	1.22	1.16
BS2-T1	30.6	1522.1	3540.9	2786.6	1839	3561	1.01	1.28	1.21
BS3-T1	31.4	1493.7	3567.0	2968.5	1751	3340	0.94	1.13	1.17
BN0-T2	29.5	1573.4	3521.3	-	1771	3101	0.88	-	1.13
BS0-T2	30.7	1493.7	3520.8	-	1700	3483	0.99	-	1.14
BS1-T2	29.6	1522.1	3474.9	2745.3	1916	3291	0.95	1.20	1.26
BS2-T2	31.7	1522.1	3613.5	2832.1	1888	3588	0.99	1.27	1.24
BS3-T2	35.5	1493.7	3837.7	3161.1	1856	3844	1.00	1.22	1.24

Table 3-7 Maximum strain evaluation - T1

Specimens	Concrete Strength f_c' [MPa]	Hysteric Behavior	Peak Strength [kN]		ε_{peak}		ε_{max}		$\varepsilon_{max}/\varepsilon_{BS1}$	$\varepsilon_{max}/\varepsilon_{BN0}$	Failure mode
			Compression	Tension	Compression	Tension	Compression	Tension	Compression	Tension	
BN0-T1	27.5	T1 (1:5)	2631	1653	0.0030	0.0287	0.0030	0.0300	0.3	1.0	CC
BS0-T1	27.3		3349	1717	0.0040	0.0403	0.0040	0.0410	0.4	1.4	RB
BS1-T1	29.2		3327	1762	0.0061	0.0386	0.0090	0.0400	1.0	1.3	RB
BS2-T1	30.6		3561	1839	0.0061	0.0489	0.0101	0.0501	1.1	1.7	RF -> RB
BS3-T1	31.4		3440	1751	0.0054	0.0406	0.0150	0.0750	1.7	2.5	RB

* RB : Reinforcement buckling / RF : Reinforcement Fracture / CC : Concrete Crushing

Table 3-8 Maximum strain evaluation - T2

Specimens	Concrete Strength f_c' [MPa]	Hysteric Behavior	Peak Strength [kN]		ε_{peak}		ε_{max}		$\varepsilon_{max}/\varepsilon_{BS1}$	$\varepsilon_{max}/\varepsilon_{BN0}$	Failure mode
			Compression	Tension	Compression	Tension	Compression	Tension	Compression	Tension	
BN0-T2	29.5	T2 (1:10)	3101	1771	0.0040	0.0393	0.0040	0.0401	0.6	1.0	CC
BS0-T2	30.7		3483	1700	0.0049	0.0395	0.0049	0.0400	0.8	1.0	RB
BS1-T2	29.6		3291	1916	0.0040	0.0604	0.0062	0.0606	1.0	1.5	RB -> RF
BS2-T2	31.7		3588	1888	0.0056	0.0600	0.0081	0.0800	1.3	2.0	RF -> RB
BS3-T2	35.5		3844	1856	0.0056	0.0792	0.0081	0.0800	1.3	2.0	RB

* RB : Reinforcement buckling / RF : Reinforcement Fracture / CC : Concrete Crushing

3.4.2 Load-strain relationship

Figure 3-26 and Figure 3-27 show the relationship between the axial load and strain of the specimen. The strain indicates that the displacement generated at the net height of the specimen divided by the height(H=800 mm). Axial displacement is defined as the average of axial displacement(L1-L2) measured on both sides. The net height, H, represents the vertical distance between the head and foundation. Figure 3-26 shows the load-strain graph and envelope curve together. Nominal compressive and tensile strength are indicated by dotted line in the graph. Table 3-6 shows the ratio of the maximum strength to the nominal strength. The actual material strength was used for the nominal strength, and the strength of the concrete was also indicated. Table 3-7 and Table 3-8 are values obtained by comparing the compression and tensile strain at maximum strength and strain. Load history T1 is compressive strain : tensile strain = 1:5, and T2 is 1:10.

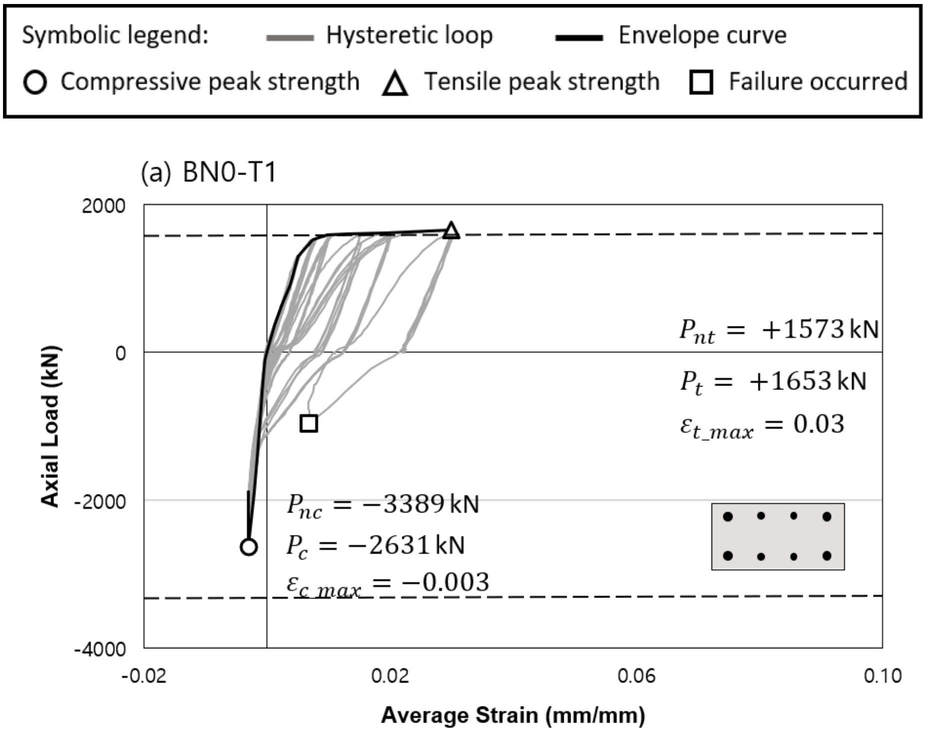
The maximum strength of BN0-T1 using normal rebar with no transverse reinforcement occurred at compressive strain 0.003 and tensile strain 0.029. The ultimate test strength P_c and P_t were -2631kN and 1653kN, respectively, and the nominal strength ratio were 0.78 and 1.05. The reason why failure occurred at strength less than the nominal compressive strength was that the strength increase ratio due to strain-hardening after yielding of normal rebar was small. In addition, the Bauschinger effect caused compressive stress smaller than yield strength under the cyclic load after tensile yield, and buckling easily occurred. For BN0 and BS0, since the type of rebar is a test variable, the test was conducted by increasing tensile strain only and fixing compressive strain when reaching the concrete crushing strain 0.003~0.004 in order to compare the tensile strain mainly. After the tensile strength of BN0-T1 reached the maximum strength, load capacity fell sharply due to the concrete crushing and the specimen was destroyed. In the case of BN0-T1, the maximum strain was equal to the strain at the maximum strength.

In BS0-T1 using seismic rebar with no transvers detail, the maximum strength occurred at compressive strain 0.004 and tensile strain 0.04 The ultimate test strength P_c and P_t were -3349kN and 1717kN, respectively, and the nominal strength ratios were 1.02 and 1.15. The maximum strain was the same as the strain at maximum strength. After the tensile strength reached the maximum, buckling occurred because there was no transverse reinforcement to restrain vertical rebars where the tensile deformation occurred greatly, and load capacity fell dramatically. The BS0-T1 using seismic rebar showed compressive and tensile strain more than those of BN0-T1 using normal rebar. This test result was judged to be due to excellent tensile deformation performance of seismic rebar.

In BS1-T1 with simplified confinement detail placed at 130mm interval and BS2-T1 at 100mm interval, the spacing of transverse reinforcement affected deformation capacity and strength. The ultimate test strength appeared at compressive strain 0.006 and tensile strain 0.04 and the maximum compressive strain was 0.009. The ultimate test strength P_c and P_t were -3327kN and 1762kN and the nominal strength ratio were 0.96 and 1.16. Due to local buckling of longitudinal rebar, specimen was destroyed by concrete spalling, and after reaching the maximum strength, the compressive strength gradually decreased. The ultimate strength of BS2-T1 occurred at compressive strain

0.006 and tensile strain 0.05, and the maximum compressive strain was 0.01. The ultimate test strength P_c and P_t were -3561kN and 1839kN, and the nominal strength ratios were 0.96 and 1.21. As the spacing of the transverse reinforcement narrowed, it showed greater strength and deformation capacity. BS2-T1, which was preceded by fracture of vertical rebar, showed tendency to gradually decrease in strength after reaching the maximum strength, represented smaller decrease in strength compared BS1-T1.

The ultimate test strength of BS3-T1 with Special seismic detail was shown at compressive strain 0.005 and tensile strain 0.04, and maximum compressive and tensile strain were 0.015 and 0.075, respectively. The ultimate test strength P_c and P_t were -3440kN and 1751kN. And the nominal strength ratios were 0.94 and 1.17. In BS3-T1, as the strain increases, the load capacity decreased and then increased again. This is because the strain increased and the area corresponding to concrete cover fell off and was not subjected to force, and the strength of the core concrete was developed accordingly.



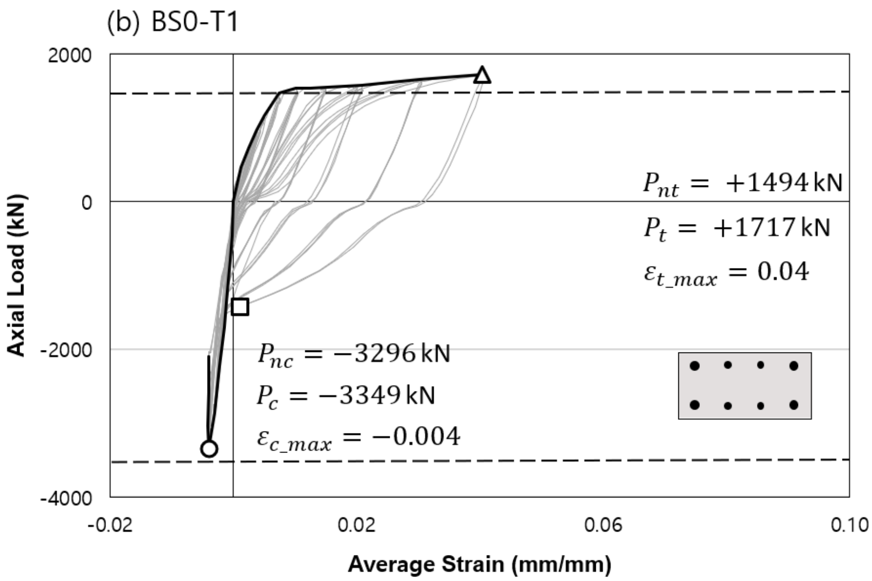


Figure 3-26 Axial load - axial strain relationship - T1

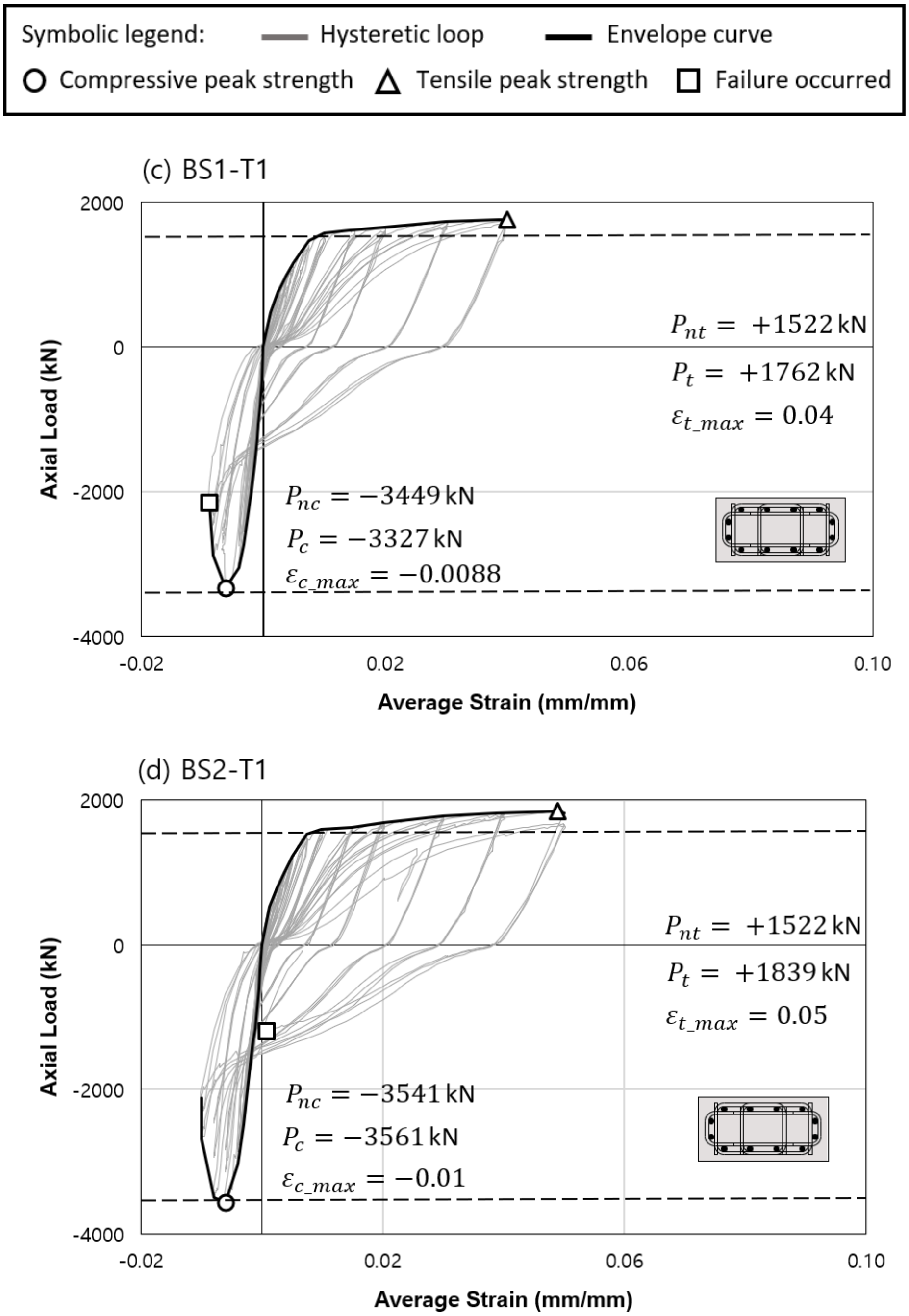


Figure 3-26 Axial load - axial strain relationship - T1

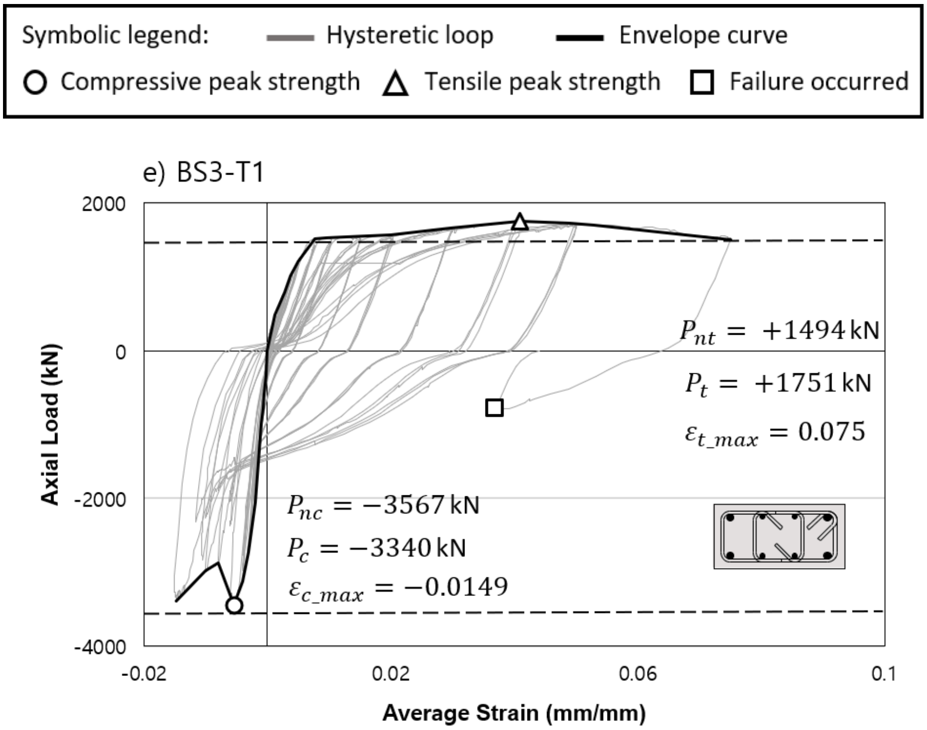


Figure 3-26 Axial load - axial strain relationship - T1

For load history T2, the specimens with the same cross-section as T1(1:5) were loaded as load history with compressive strain : tensile strain was 1:10. The ultimate test strength of the BN0-T2 using normal rebar appeared at compressive strain 0.004 and tensile strain 0.039. The ultimate test strength P_c and P_t were -3101kN and 1771kN, the nominal strength ratios were 0.88 and 1.13. The BN0-T2 was destroyed by sharp drop in strength due to concrete crushing after tensile strength reached the maximum strength. The maximum strain was the same as the strain at the maximum strength, showed similar pattern to the failure of BN0-T1.

The ultimate strength of BS0-T2 occurred at compressive strain 0.0049 and tensile strain 0.04. The ultimate test strength P_c and P_t were -3483kN and 1700kN, the nominal strength ratios were 0.99 and 1.14. The maximum strain was the same as the strain at the maximum strength, and when compressive strength reached the maximum, it was destroyed by buckling of the whole specimen, not by local buckling of rebar. BS0-T2 showed above-described failure pattern due to an error in loading when performing the test.

BS1-T2 with transverse reinforcement placed at 130mm interval and BS2-T2 with 100mm interval had a difference in the spacing affecting the deformation performance and strength, and showed high tensile strength unlike load history T1. The ultimate test strength of BS1-T2 was shown at compressive strain 0.004 and tensile strain 0.039, and maximum compressive and tensile strain were 0.062 and 0.04, respectively. The ultimate test strength P_c and P_t were -3291kN and 1916kN, And the nominal strength ratios were 0.95 and 1.26. it was destroyed due to the concrete spalling with buckling of vertical rebar in the lower part, and the compressive strength gradually decreased after reaching the maximum strength. In addition, since relatively high tensile strain was applied as compared the load history T1, sufficient strain-hardening occurred in vertical rebars of the specimen, resulting in high tensile strength.

The ultimate strength of BS2-T2 occurred at compressive strain 0.0056 and tensile strain 0.06, and the maximum compressive and tensile strain were 0.0081 and 0.08. The ultimate test strength P_c and P_t were -3588kN and 1888kN, And the nominal strength ratios were 0.99 and 1.24. As the spacing of the details narrowed, it showed higher compressive and tensile deformation capacity compared to BS1-T1. BS2-T2, which was preceded by the fracture of vertical rebar, showed tendency to gradually decrease after reaching the maximum strength, like BS1-T2. Because the cross-sectional force was lost in the vicinity of the fracture position of vertical rebar, it didn’t receive the axial load, and buckling occurred.

The ultimate strength of BS3-T2 occurred at compressive strain 0.0056 and tensile strain 0.079. The maximum compressive and tensile strain were 0.0081 and 0.08. The ultimate test strength P_c and P_t were -3844kN and 1856kN, And the nominal strength ratios were 0.99 and 1.24. Unlike BS3-T1, BS3-T2 showed the same maximum compressive and tensile deformation capacity as BS2-T2. Failure mode was destroyed by out-of-plane buckling with loosening of the cross-tie. Compared to BS3-T1, because of the large tensile deformation, the stability became poor, and the core concrete damage of the specimen was relatively large.

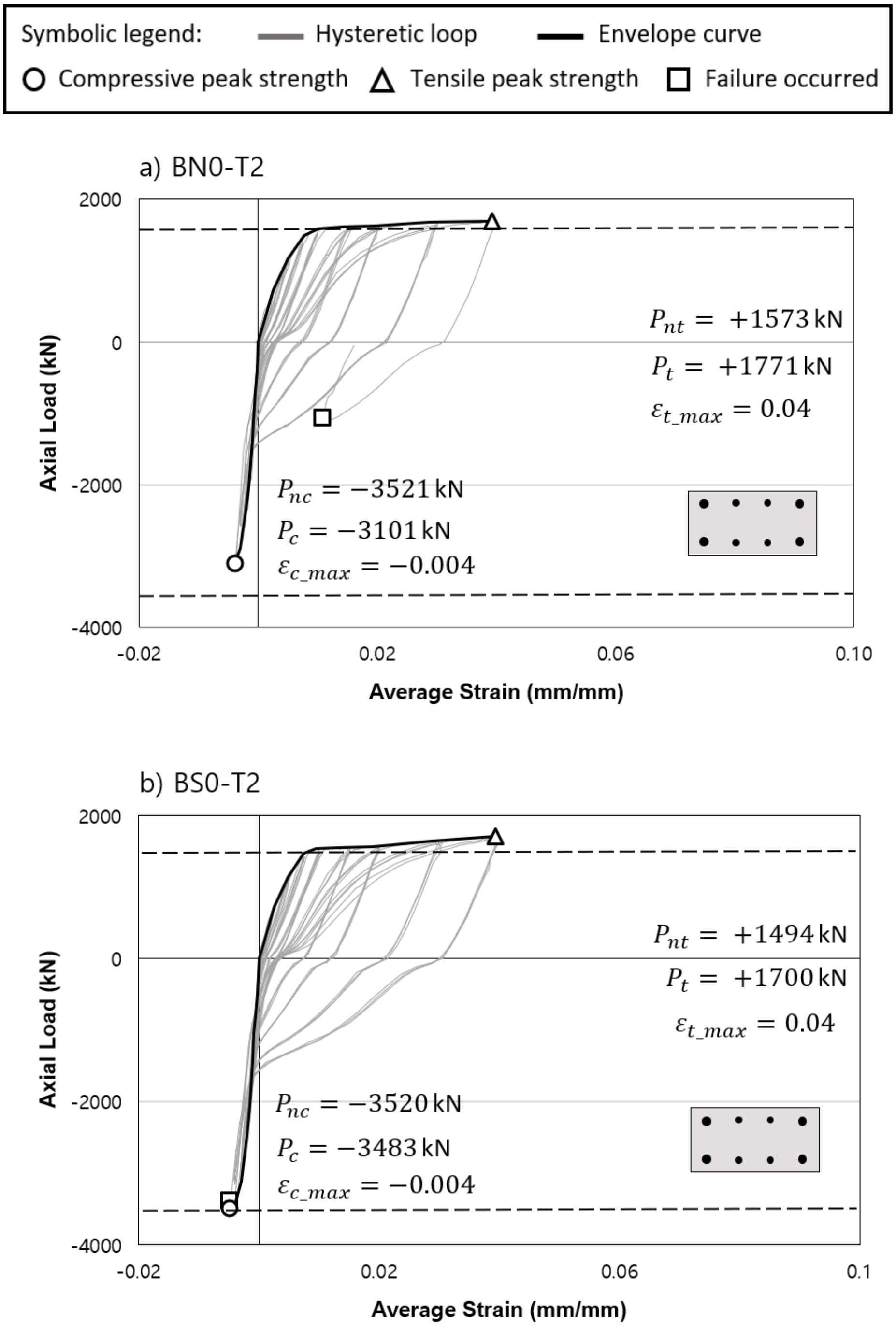


Figure 3-27 Axial load - axial strain relationship - T2

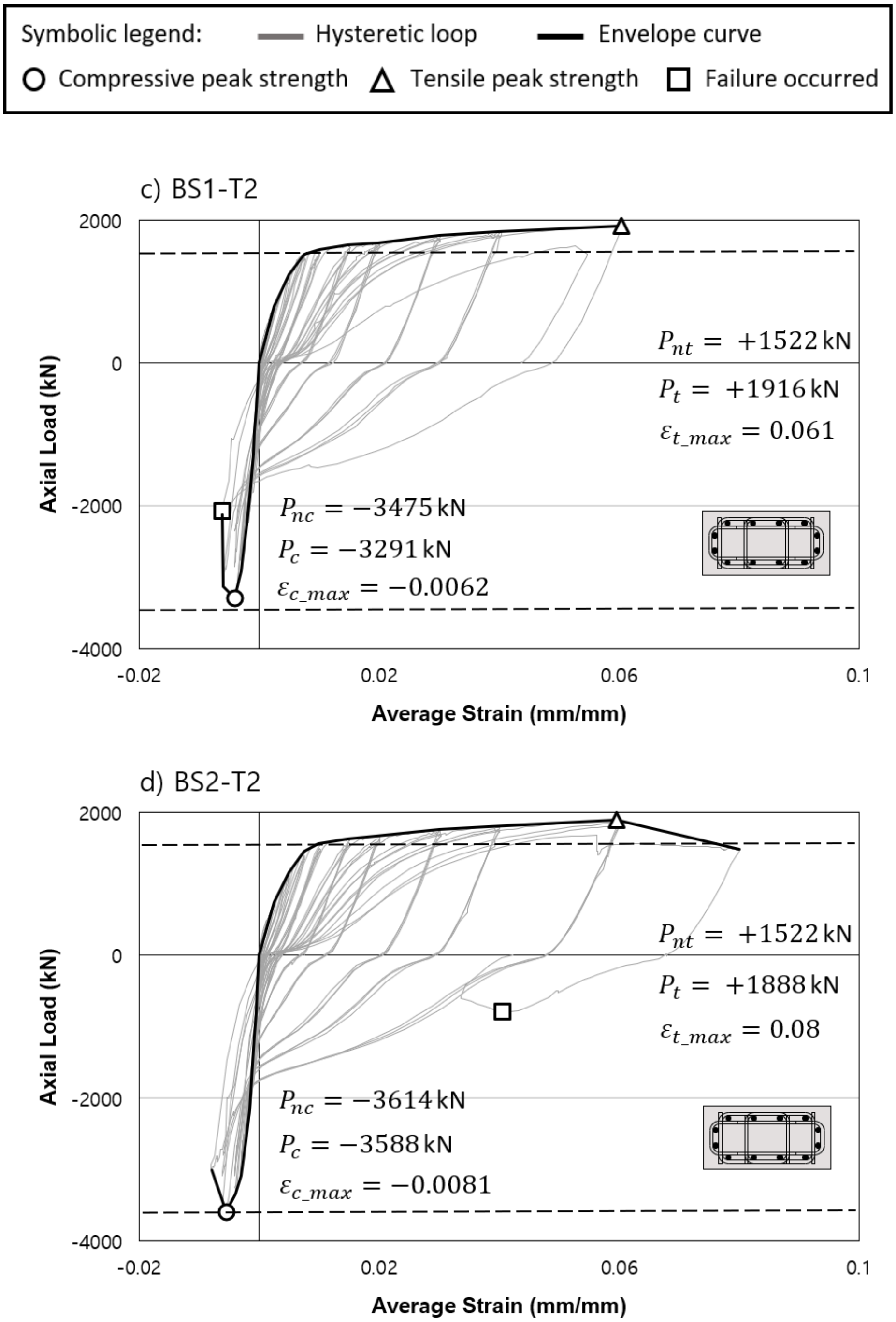


Figure 3-27 Axial load - axial strain relationship - T2

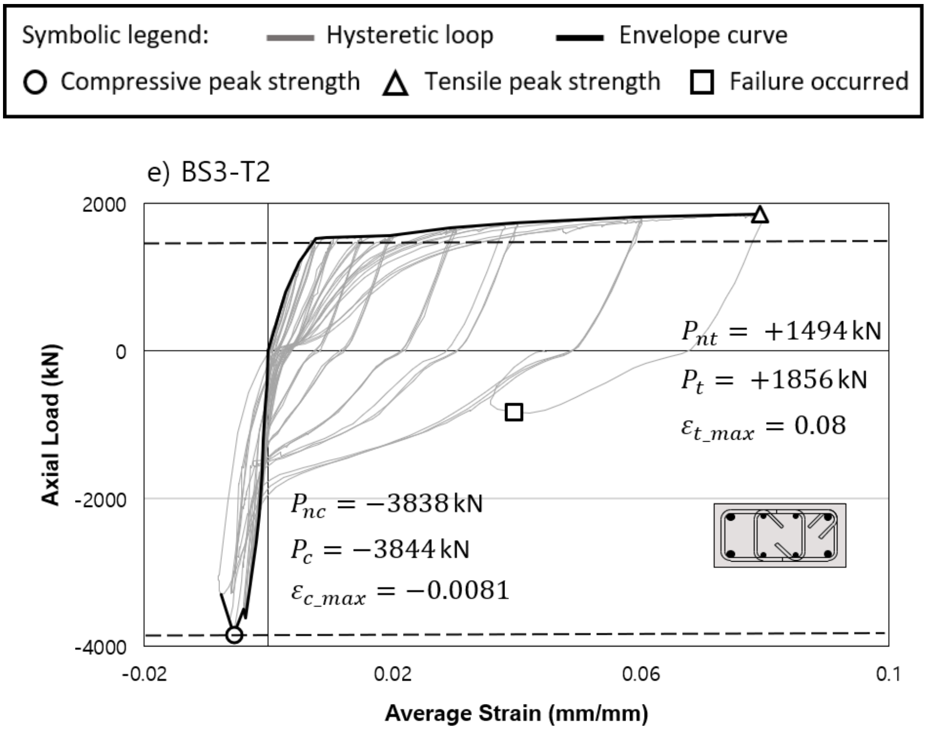


Figure 3-27 Axial load - axial strain relationship - T2

3.4.3 Strain of reinforcement

Figure 3-28 and Figure 3-29 show strain distribution of vertical rebars in the cross section of specimens. In the figures, strain distribution is indicated as 0.001(○ mark), 0.002(□ mark), 0.004(△ mark), 0.006(◇ mark), 0.008(X mark), 0.01(* mark). Dotted line represents the yield tensile strain of rebar.

Since vertical rebar deformation was governed by the axial force behavior of boundary element specimens, the strain increased approximately uniformly when compressive and tensile loads were applied. When the rebar was in plastic state, it represented relatively large strain even under compressive load because tensile residual strain remained. Despite the axial force, large strain can occur due to local stress concentration around cracks after yielding. In the case of BN0 and BS0 with no transverse reinforcement, axial load wasn't transmitted evenly and large deformation occurred partially. This was related the failure mode that occurred in no transverse reinforcement. In particular, the failure modes of BN0-T1 and BS0-T1 occurred due to concrete crushing on the outer side of specimen and buckling of outmost rebar. The cause was that axial force was not transmitted uniformly to the whole specimen section, leading to partial failure. However, BS0-T1 using seismic rebar showed superior deformation performance compared to BN0-T1 using normal rebar, and this can be seen from the test result and strain as shown in Figure 3-28.

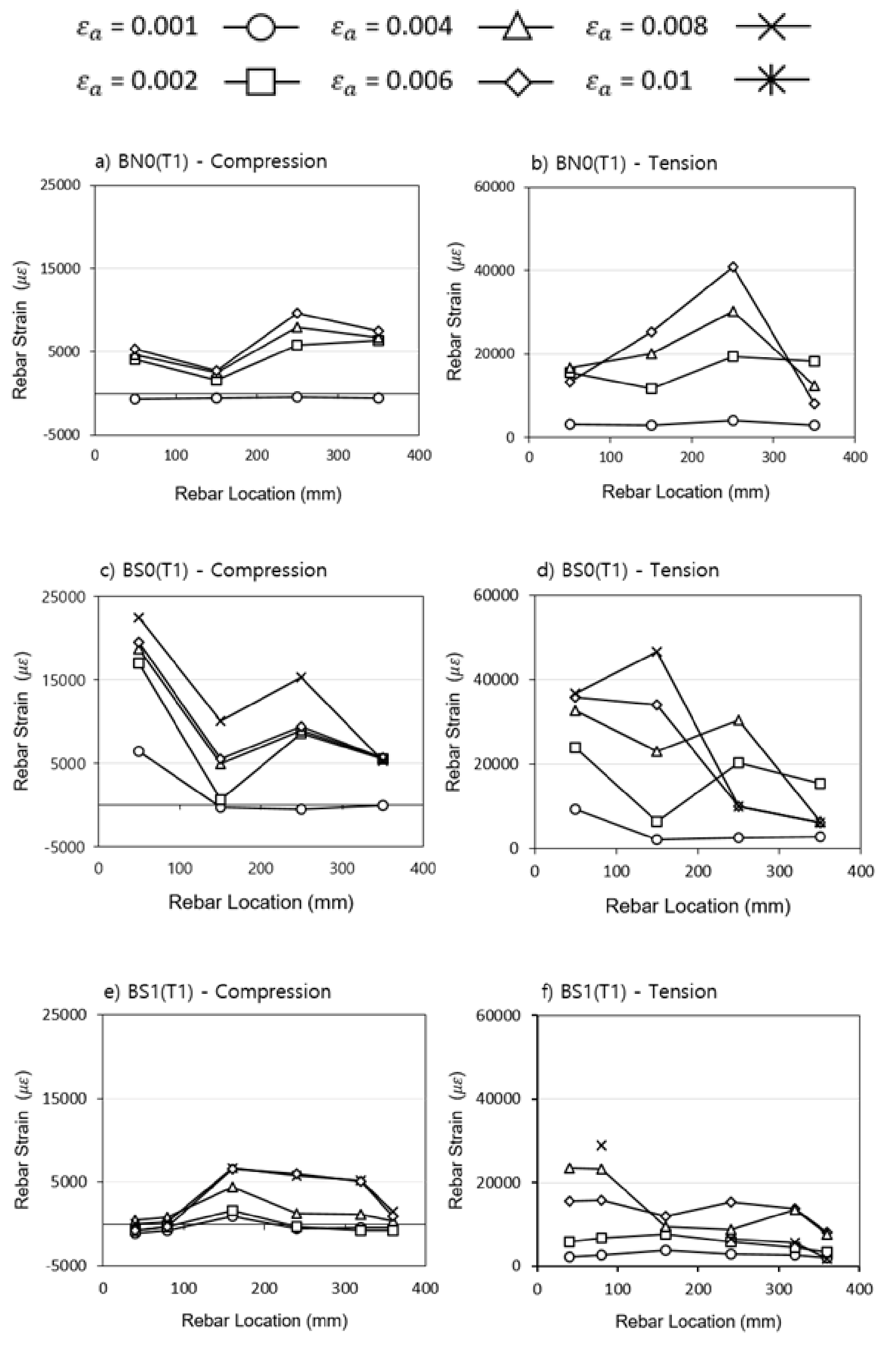


Figure 3-28 Strain distribution of longitudinal reinfocement - T1

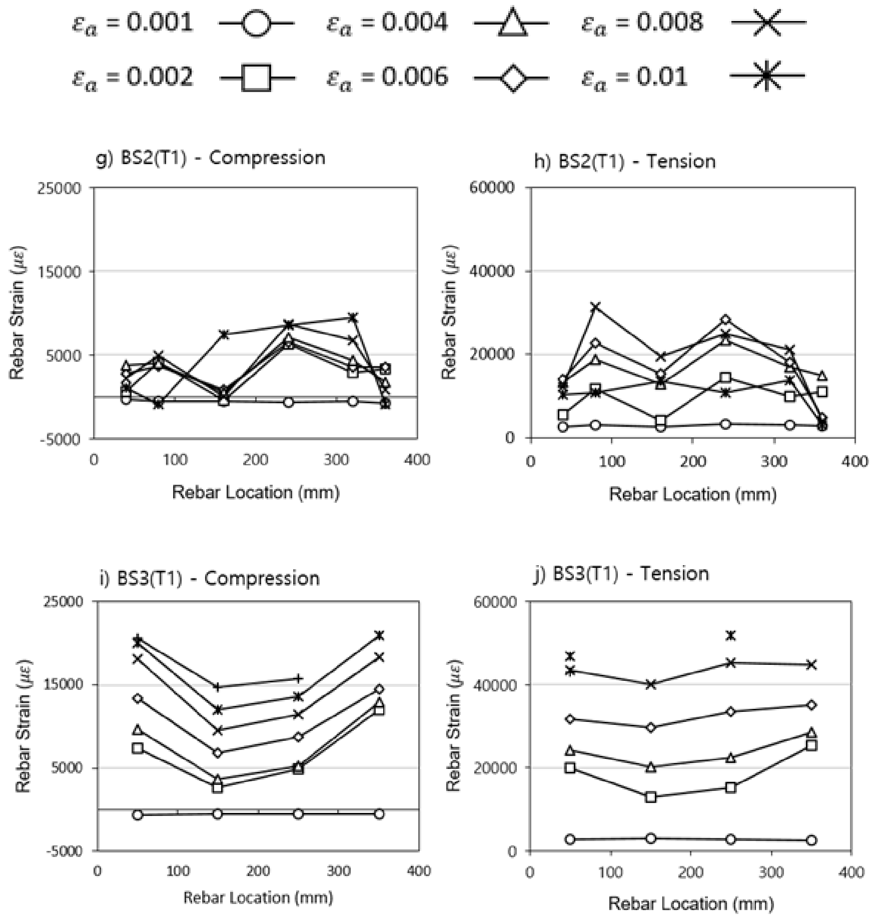


Figure 3-28 Strain distribution of longitudinal reinforcement - T1

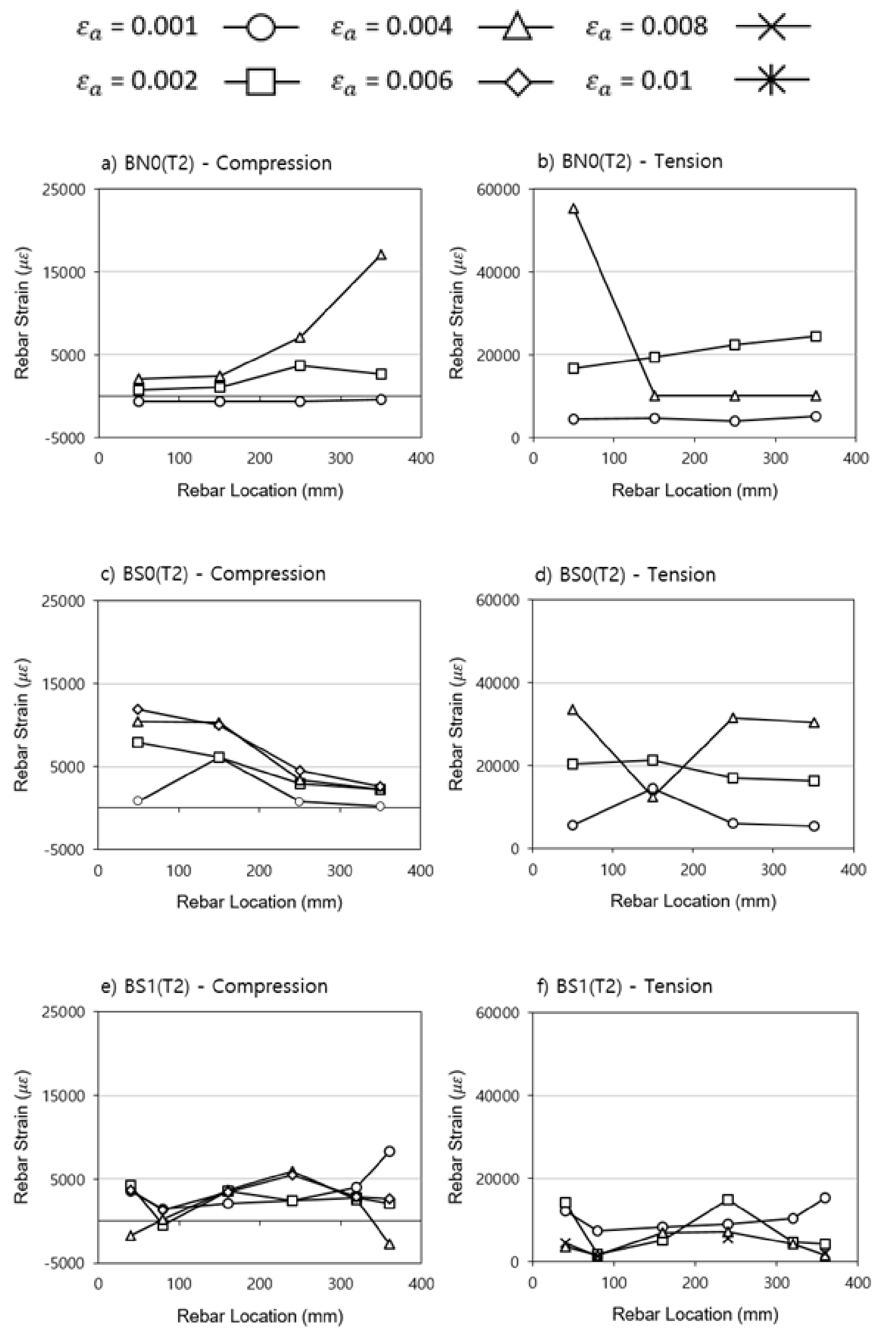


Figure 3-29 Strain distribution of longitudinal reinforcement - T2

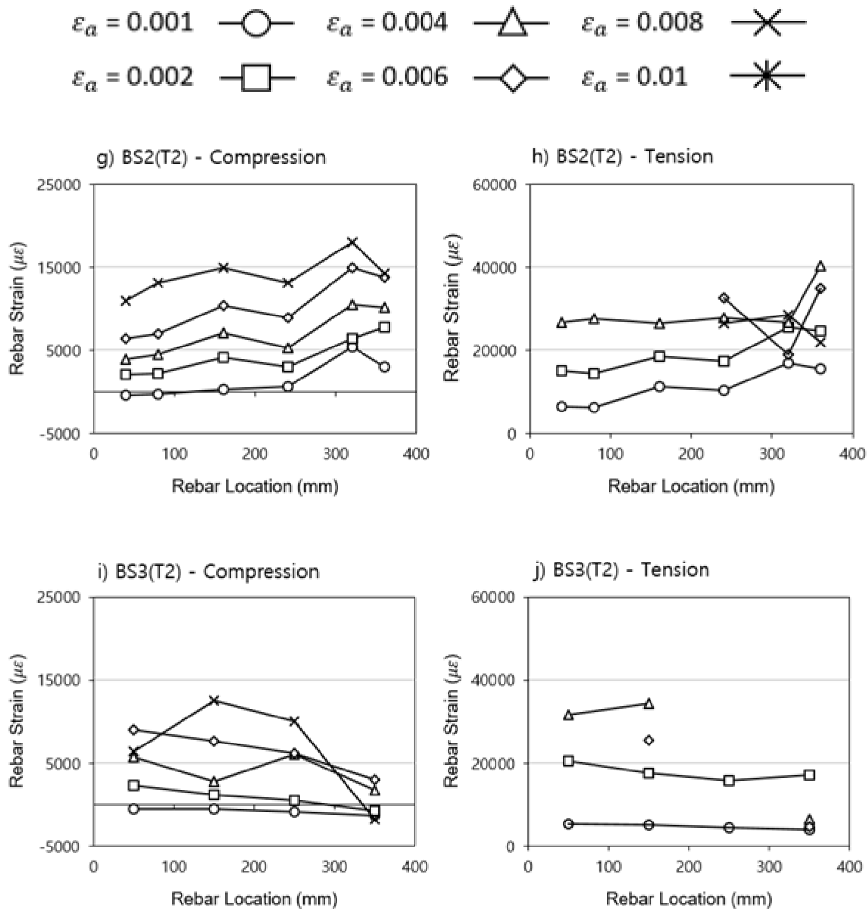


Figure 3-29 Strain distribution of longitudinal reinforcement - T2

3.5 Effect of test parameter

3.5.1 Reinforcement type

BN0 and BS0 are specimens without transverse reinforcement. Strain history was performed with compressive strain : tensile strain = 1:5 and 1:10. From the step when the strains of two specimens reached 0.003 to 0.004, which causes concrete crushing, the test were conducted while fixing the compressive strain and increasing the tensile strain only. The fixation of compressive strain was judged as the occurrence of vertical craking or load capacity decrease of the specimen when the strain was 0.002 more.

In Figure 3-30 (a), BN0-T1 was destroyed by concrete crushing under compressive stress due to load capacity decrease and vertical cracking when the compressive strain reached 0.003. BS0-T1 had compressive strain 0.004 and tensile stran 0.04, and it was destroyed by buckling of rebar. The maximum compressive strength ratio BS0-T1 to BN0-T1 was 1.27 times, and BS0-T1 using seismic rebar showed high strength. The maximum compressive strain ratio was 1.33 times, and BN0-T1 was destroyed at lower compressive strain because concrete crusing was preceded.

The maximum tensile strength ratio was 1.04 times, so there was no significant difference between the two specimens. The specimens without boundary confinement had no concrete confinement effect, so concrete crushing and bukling preceded. Therefore, the strength ratio between BN0-T1 and BS0-T1 was not large because vertical rebar didn’t sufficiently occure strain-hardening. However, the maximum tensile strain ratio was 1.33 times, showing the excellent deformability of seismic rebar, which showed the distinct difference between BN0-T1 and BS0-T1. In addition, the yield strength of seismic rebar was lower than normal rebar, but the tensile strength indicated by the actual boundary element test result was higher in BS0 using seismic rebar.

BN0 was destroyed by concrete crushing after the compressive strain and the tensile strain reached 0.004 and 0.04. BS0-T2 had compressive strain 0.0048 and tensile strain 0.04, and buckling of vertical rabar occurred and destroyed. The maximum compressive strength ratio of the specimens was 1.12, and BS0 -T2 showed high strength, and the maximum compressive strain ratio was 1.21. BS0-T2 was destroyed without developing tensile strain and strength than the test plan because the buckling was preceded at compressive strain 0.0048 due to error of loading. Therefore, direct comparison was difficult, but indirect comparison with BS3-T2 using the same vertical rebar configuration as BS0-T2 was required. The maximum tensile strength ratio is 1.02 times, when compared by replacing the load history of BS0-T2 with the load of the BS3-T2. It was judged that the difference of strength was not significant because of the effect of normal rebar with higher yield strength than seismic rebar in t he case of the same strain after yielding.

3.5.2 Transverse reinforcement type

According to Table 3-7, compressive strain of BS0-T1 was less than 0.004. On the other hand, when comparing BS0 with the specimens placed transverse reinforcement, BS1-T1 with simplified confinement detail (130mm) had maximum compressive strain ratio 2.18(0.009). BS2-T2 with developed detail(100mm) increased to 2.48(0.0101) compared with compressive strain at rebar fracture. BS3-T1 with closed hoop and crosstie improved maximum strain ratio by 3.66 times(0.015).

In Figure 3-30 Tensile strain of BS0-T1 was less than 0.04. When comparing BS0 with the specimens with details, BS1-T1 showed the same tensile strain as BS0 because buckling of rebar(the outermost rebars in both in-plane direction, between transverse reinforcement) occurred before sufficient tensile strain was developed. BS2-T2 increased tensile strain by 1.25 times compared to BS1-T1. In the case of BS3-T1, the maximum tensile strain was 0.075, but the strength gradually decreased after raching the maximum strength at tensile strain 0.053. When compared based on the strain of maximum strength, it increased 1.31 times(0.053), and when compared based on the maximum strain, it enhanced 1.86 times(0.075). This was that the performance of compressive deformation capacity increased due to the confinement, so strength hardening of vertical rebar occurred when applying the tensile load.

3.5.3 Vertical spacing of transverse reinforcement

Specimnes with transverse boundary confinement are BS1(simplified confinement detail, 130mm), BS2(simplified confinement detail, 100mm), and BS3(special seismic confinement, 65mm). Compared to the maximum compressive strain 0.009 of BS1-T1, BS2-T1 and BS3-T1 were 0.0101 and 0.015, showing 1.12 and 1.67 times better deformation capacity, respectively. As the spacing of transverse reinforcement is narrower, not only concrete confinement effect improved, but also the resistance to buckling showed better performance. In terms of tensile strain, BS2-T1 and BS3-T1 showed 1.25 and 1.88 times better performance than BS1-T1.

Also, compared to BS1-T2, BS2-T2 and BS3-T2 increassed maximum compressive strain and tensile strain enhanced 1.3 times. Although failure mode of BS2-T2 arose faster than buckling of BS3-T2 due to the rebar buckling after the rebar facture, there was no difference in terms of maximum strain. The effect of vertical spacing of details was less in the boundary elements where tensile strain greater than compressive strain occurred. As tensile deformation increases, concentrated strain occurs in crack, so the stability against compressive load decreases significantly. Therefore, in this case, the spacing of transverse reinforcement didn’t significantly affect the performance of the boundary element. The maximum strength of T2 specimens was not significantly different except for BS3-T2. Beacause BS3-T2 had 1.13 times greater concrete strength than BS3-T1, BS3-T2 had higher maximum load capacity.

3.5.4 Loading history

In the cyclic axial loading test, two loading histories were used to test for the same two specimens(See Figure 3-30). The deformation performance and behavior of the specimens were evaluated by setting up the magnitude of tensile strain that can occur at boundary element of wall on earthquake load as a variable. Comparing T1 and T2 in the compressive load – strain relationship showed different patterns. Specimens of T1 represented relatively higher strain than that of T2 in compression, but the loading history was not an important factor in terms of compressive strength. In the case of specimen without lateral detail, there was no significant difference due to the concrete crushing and local buckling between the compressive strain 0.003 ~ 0.004.

BS1, BS2, and BS3 with the transverse reinforcement represented 1.47, 1.27, and 1.93 times greater compression strain in T1 than T2, respectively. When loading history T2 was the same compressive strain as compared to T1, the load was applied at twice the tensile strain, which caused a large plastic strain on the vertical rebar and greatly reduced the stability. Although, the strength didn't sharply decrease, buckling and fracture of rebar occurd at the lower compressive strain of T2 specimens than T1 specimens.

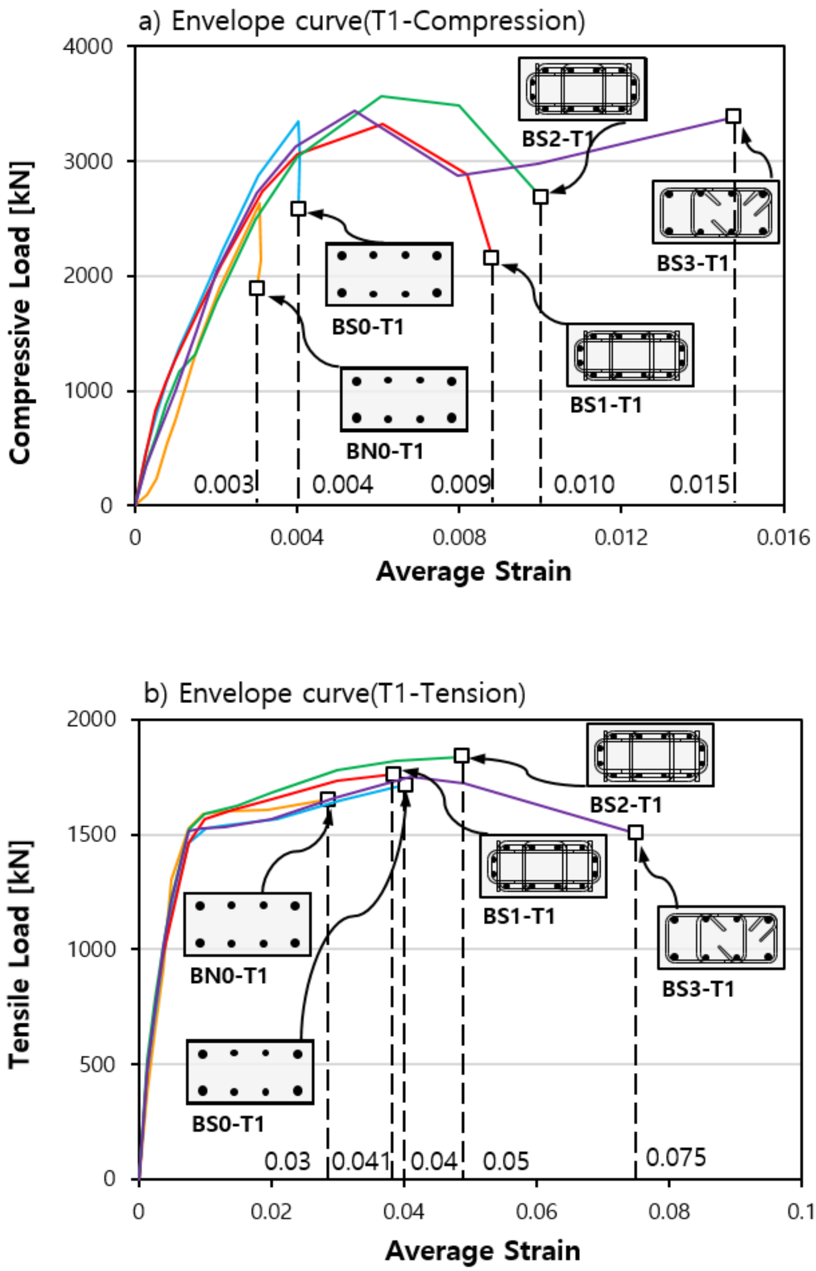
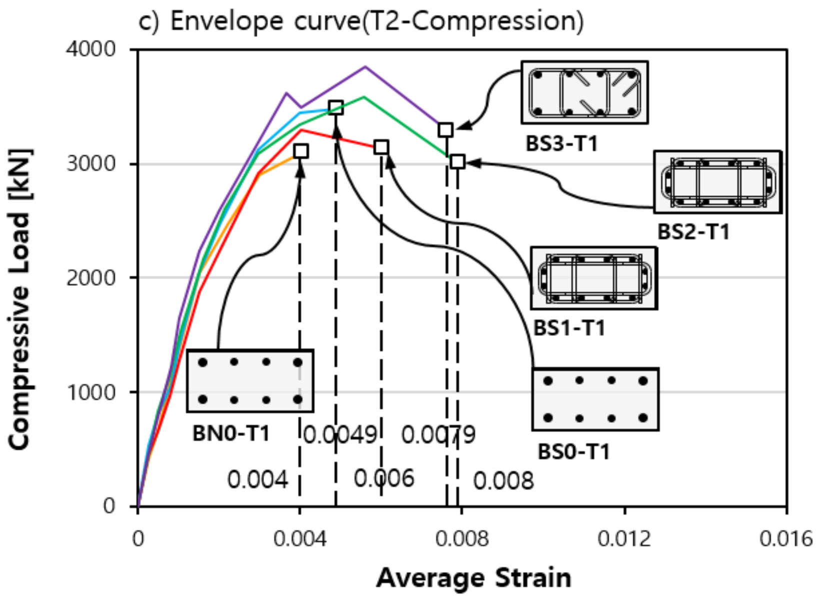


Figure 3-30 Strength - strain relationship



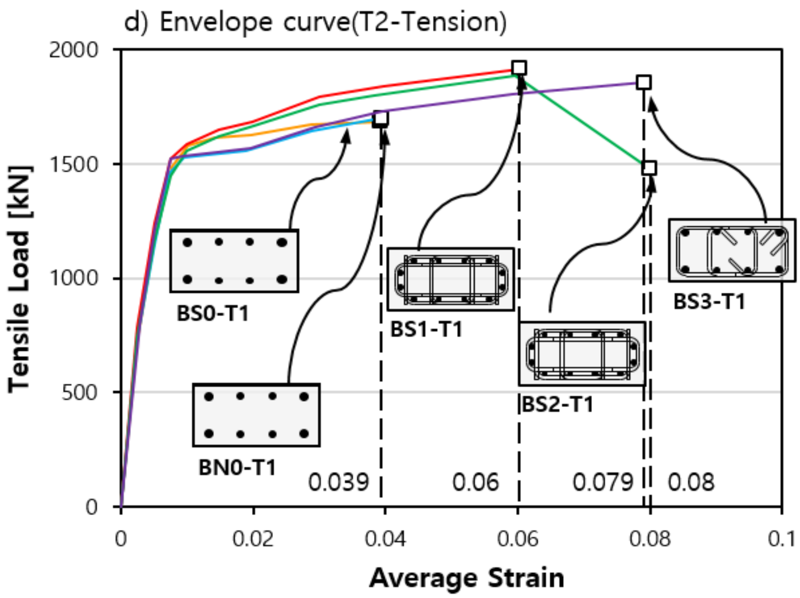


Figure 3-30 Strength - strain relationship

Chapter 4. Cyclic Lateral Loading Test for RC wall

4.1 Introduction

In Chpater 4, based on the results of cyclic axial loading test for boundary element specimen, the proposed transeverse reinforcement detail were applied to RC wall to verify the performance of the detail in RC wall specimen. Since the boundary element loading test was the test that simulated the edge of a flexural wall, there was limitation in accurately reproducing the mechanical behavior represented by RC wall.

In the details propsed in the previous study, the effect of side ties was not significant due to the loosening of U-shpaed details. In this study, an experiment on the flexural tensile yield wall was designed so that simplified confinement detail and seismic reinforced bar can be used. Cyclic lateral loading test for RC wall with simplified confinement detail were performed to review the ductility, deformation capacity, peak strength, ultimate behavior, failure mode, etc. of wall specimes, and the validity of the simplified confinement detail.

4.2 Test plan

4.2.1 Test variables and detail of specimens

In Cyclic lateral loading test for RC wall, the vertical spacing of the proposed detail placed on the wall specimen was adjustd by reflecting the result of cyclic axial loading test for boundary element specimen. Boundary element specimens BS1 and BS2 were arranged with $2t_w/3$(130mm) and $t_w/2$(100mm) vertical spacing of the detail, respectively, and boundary element test was performed. As a result of the test, the peak compressive strain and tensile strain of BS1-T1 were 0.009 and 0.04, which were 0.6 times and 0.53 times lower that that of BS3-T1. In addition, BS1-T2 showed the peak compressive strain 0.0062 and tensile strain 0.061, showing 0.75 times lower than BS3-T2. Therefore, the vertical spacing of the proposed detail applied to the RC wall test was placed in the same spacing as the special seismic detail specimen BS3. So $t_w/2$ and $t_w/3$ spacing were used to verify the performance of the proposed detail.

The vertical reinforcement raito at boundary element was 2.92% to perform the test under the same condition as the boundary element test. in oder to prevent premature failure in the web, the relatively high reinforcement ratio(=0.75%) was applied. To avoid shear failure of wall, the horizontal reinforcement ratio was designed to be 0.97%.

As a result of section analysis, the compressive zones of the two specimens at peak strength were 312mm and 301 mm, and the ratio of compressive strain and tensile strain in the wall section was 1:4.3. This test is an experimental setup without axial load, and since it was designed as a flexural yield wall, it showed a relatively short compressive zone depth. According to KDS 41 17 00, for the compressive zone depth, the code that the boundary element range must be at leat the larger of $c/2$ or $c-0.1l_w$ from the edge was satisfied, the boundary element of specimen was designed to be 400mm.

In order to study the effect of seismic transverse reinforcement detail on the lateral deformation capacity of the wall, the shear strength was design to be higher than the flexural strength so that the specimen was destroyed in the flexural failure mode. In addition, the foundation and head beam were designed to perform the test. The total number of wall specimens was 2, and the experimental parameters are shown in Table 4-1. The size of all the specimens was 1,600mm(length) x 3,125mm x 200mm(thickness). As a test variable, the vertical spacing of the transverse reinforcement was considered.

In Table 4-1, the name of the specimen represents the test parameter. The second letter S stands for seismic reinforcement. The third number represents the vertical spacing of the proposed transverse reinforcement. The number 1 represents the spacing 100mm of transeverse reinforcement, and number 2 means the spcaing 65mm.

The compressive strength of concrete was designed to be 30MPa. Concrete pouring of the specimnes was carried out on the same time, and both specimens showed concrete compressive strength 29.2MPa, which was close to the target strength. In the specimen, all verical reinforcement used seismic rebar. D16 and D19 rebars of SD600 were used for vertical reinforcement. except for the vertical reinforcement, horizontal reinforcement and transverse reinforcement were used for normal rebar. And D10 of SD400 rebar was used for transverse reinforcement details, D13 of SD400 rebar was used for horizontal reinforcement.

The arrangement of vertical reinforcement bar is as follows. In the boundary element, D16 seismic rebars were arranged at 50mm and 90mm intervals, and D19 seismic rebars were arranged at 90mm intervals, so that a total of 20 rebar was placed. A total of six D16 seismic rebar were arranged in the center of the wall at 215mm intervals. The arrangement of horizontal rebar is as follows. A total of 46 normal rebars were placed at 130mm intervals. D10 was tranverse reinforcement, and it was reinforced differently according to the specimen. Both specimens applied simplified confinement detail, the diameter and spacing of vetical and horizontal rebar were the same. Only the vertical spacing of the transverse reinforcement detail was designed differently. A groove join with depth of 70mm was installed to prevent shear slip on the upper surface of the foundation.

The nominal flexural strength and nominal shear strength of the specimen were calculated according to KDS 41 17 00.

$$\mathrm{V_f} = M/(h_w + 250mm) \tag{4.1}$$

$$\mathrm{V_c} = \frac{1}{6}\sqrt{f_{ck}}b_w d \tag{4.2}$$

$$\mathrm{V_s} = \frac{A_s f_y d}{s} \tag{4.3}$$

$$\mathrm{V_n} = V_c + V_S \tag{4.4}$$

Figure 4-2 and Figure 4-3 show WS1 and WS2 with proposed transverse reinforcement details. These specimnes were evaluated for the effects of U-tie and U-end tie. Short U-tie and long U-tie are welded together in the same direction and assembled in out-of-plnae dicrection of wall. U-ties were meshed with each other to form a hoop, exerting the concrete confinement effect. Figure 4-2 shows simplified confinement detail in the section of specimen.

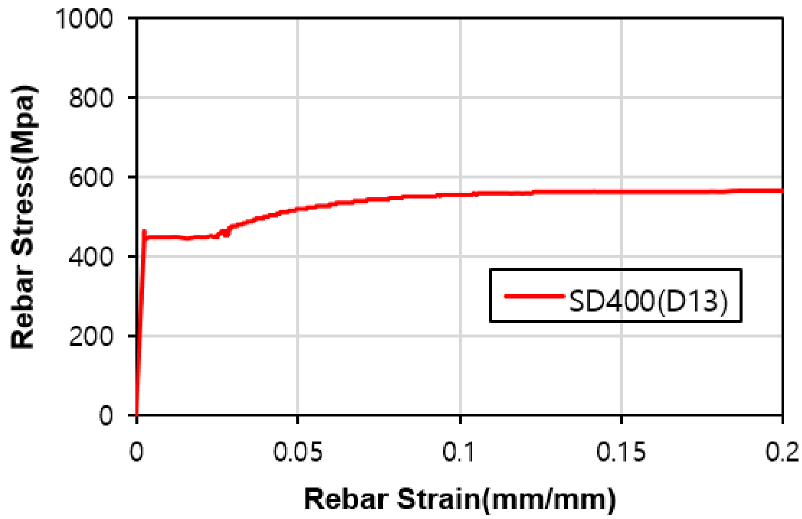
The boundary element length was designed to be 400mm, the seismic detail were applied, and the length of the web was designed to be 800mm. In order to prevent the occurrence of shear failure before yielding, the horizontal rebar ratio was designed as 0.97% and the shear strength/flexural strength was designed as 1.6. In the boundary element, the vertical reinforcement ratio was 2.92%, and the vertical reinforcement ratio of the web was 0.75%. The flexural strength of WS1 was 770.4kN, WS2 is 781.4kN, and the shear strength of WS1 and WS2 were 1231.9kN, respectively.

Table 4-1 Test parameter

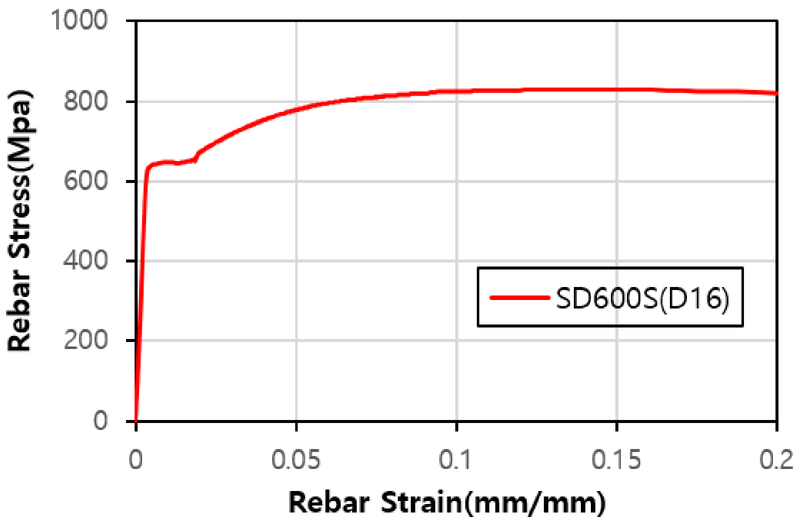
Specimens	Rebar type	Concrete Strength f_c' [MPa]	Reinforcement										Strength prediction		
			Web					Boundary region					Shear strength V_n [kN]	Flexural strength V_f [kN]	$\frac{V_n}{V_f}$
			Horizontal			Vertical		Horizontal		Vertical		Re-bar Detail			
			V_s/V_{smax}	f_{yh} [MPa]	ρ_h [%]	f_{yv} [MPa]	ρ_v [%]	f_{yh} [MPa]	ρ_h [%]	f_{yb} [MPa]	ρ_b [%]				
WS1	SD600S (Seismic)	29.2	1.07	459	0.97	642	0.75	459	1.33	642	2.92	U end bar + U cross tie	1231.9	770.4	1.60
WS2		29.2	1.07	459	0.97	642	0.75	459	2.05	642	2.92	U end bar + U cross tie	1231.9	781.4	1.58

Table 4-2 Summary of reinforcement properties

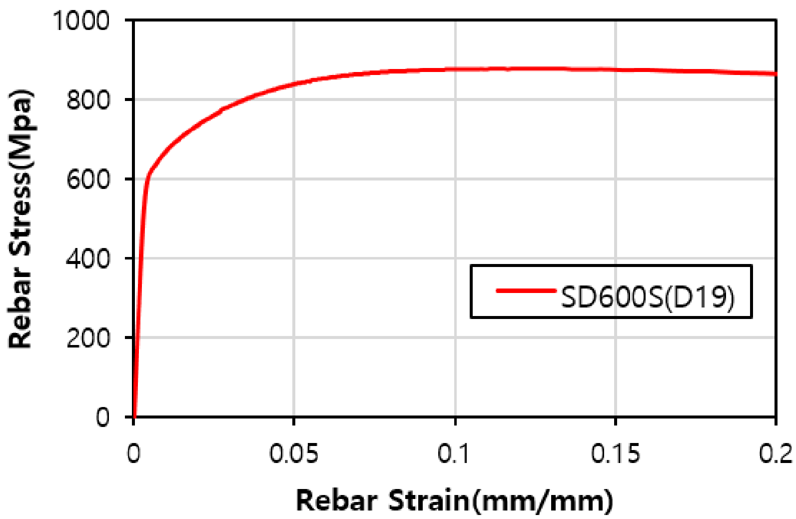
Rebar type		Yield Strength [MPa]	Tensile Strength [MPa]	Tensile to Yield Ratio	Ratio Average
D13 (SD400)	rebar1	462	584	1.264	1.254
	rebar2	450	567	1.260	
	rebar3	465	576	1.239	
D16 (SD600S)	rebar1	645	842	1.305	1.295
	rebar2	640	830	1.297	
	rebar3	642	824	1.283	
D19 (SD600S)	rebar1	650	875	1.346	1.347
	rebar2	646	876	1.356	
	rebar3	652	873	1.339	



(a) D13 reinforcement



(b) D16 reinforcement

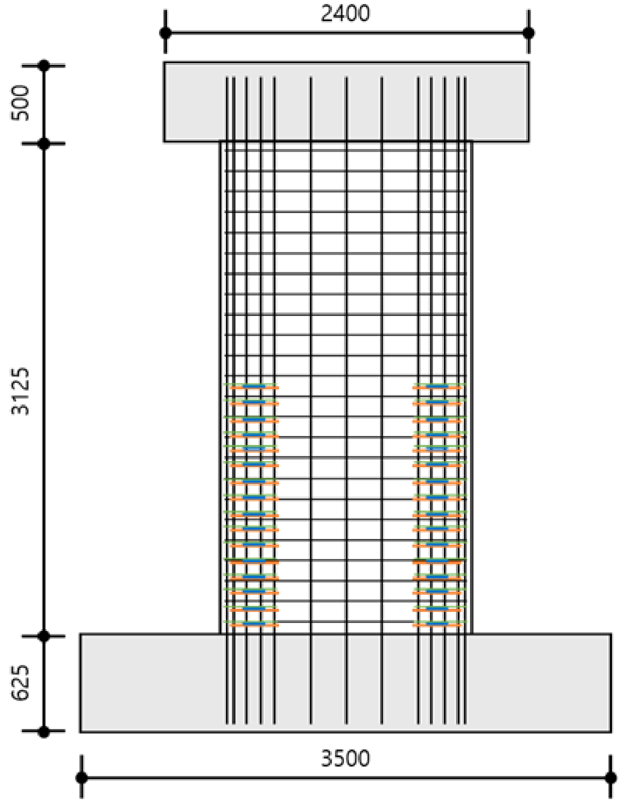


(c) D19 reinforcement

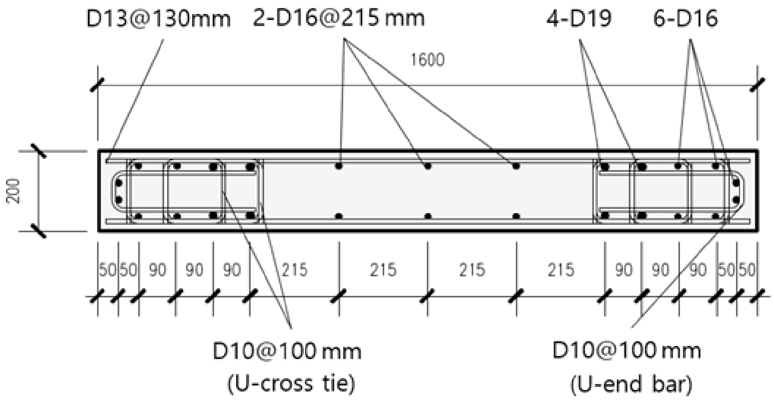
Figure 4-1 Reinforcement stress-strain relationship

4.2.2 Test setup and loading plan

Figure 4-6 shows the loading plan and the setup for displacement measurement and Linear Variable Displacement Transformers(LVDTs). After installing a lteral restraint guide to prevent twisting in the out-of-plane direction, the test performed by applying a cyclic lateral load to the specimen's head beam using a 1500kN Static Actuator. The displacement due to the load was recorded every 2 seconds, and the specimen was applied under the displacement control until the specimen was destroyed. The concrete compressive strength was determined as the average value by compressive strength test on the day of experiment for three concrete cylinder specimes. The lateral load loading plan in Table 4-3 was followed by Acceptance Criteria for Special Precast Concrete Structural Walls". The lateral displacement of the specimen was measured suing 200mm LVDT(L1) and 2000mm LVDT(1-1). The flexural deformation of the wall(L4 - L9) was measured in thress areas : plastic hinge area, middle area, and elastic area. Shear deformation(L10-L13) was measured in two areas by bisecting the wall. In addition, the slip(L2 – L3) and locking(L16-17) of the specimen foundation, the sliding of the wall(L14), and the displacement of the wall in the out-of-plane direction(L15) were measured.

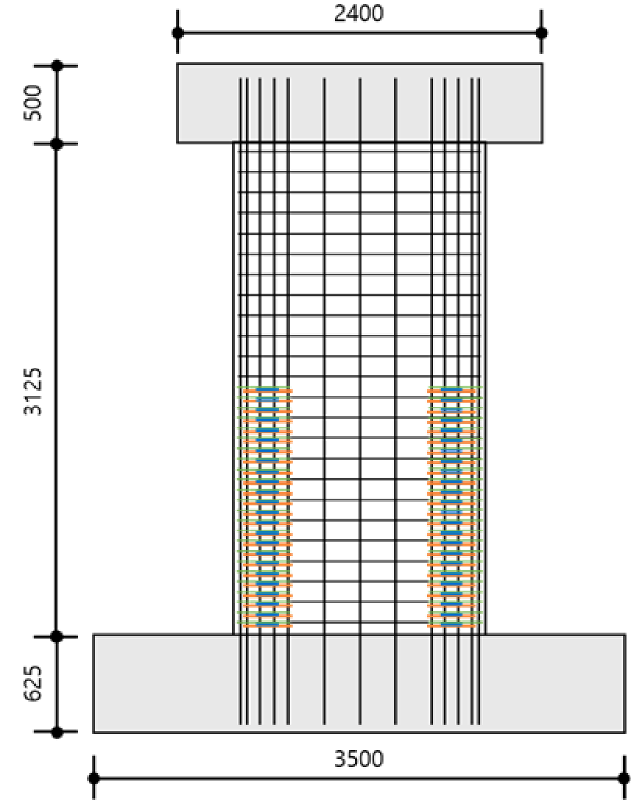


(a) Elevation detail of WS1

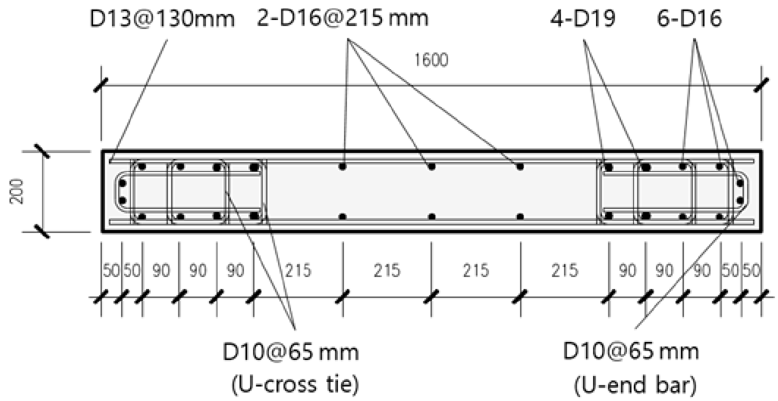


(b) Sectional detail of WS1

Figure 4-2 Detail of WS1



(a) Elevation detail of WS2



(a) Sectional detail of WS2

Figure 4-3 Detail of WS2

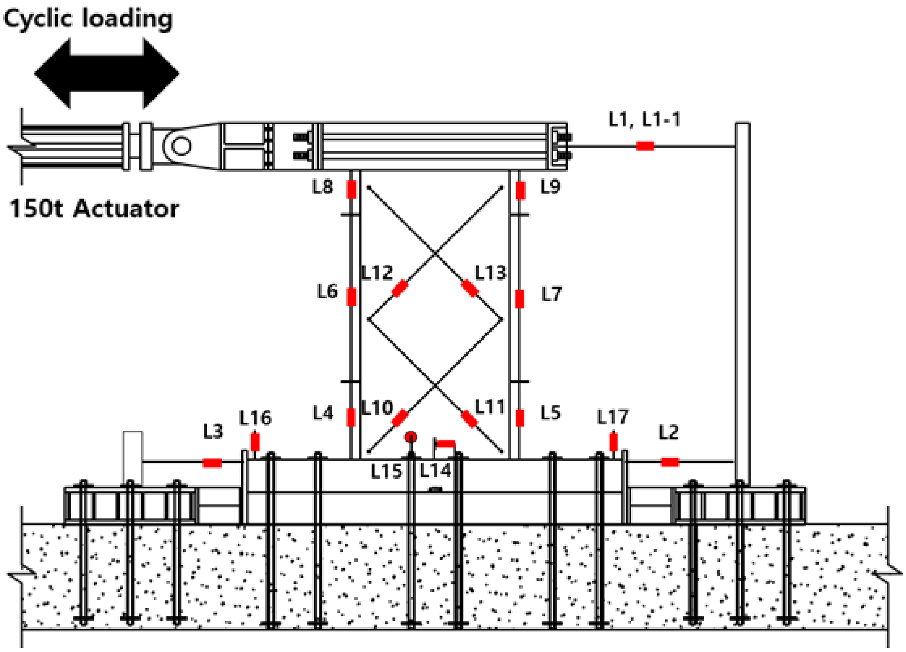


Figure 4-4 Test-setup and LVDTs plan

Figure 4-5 Test-setup

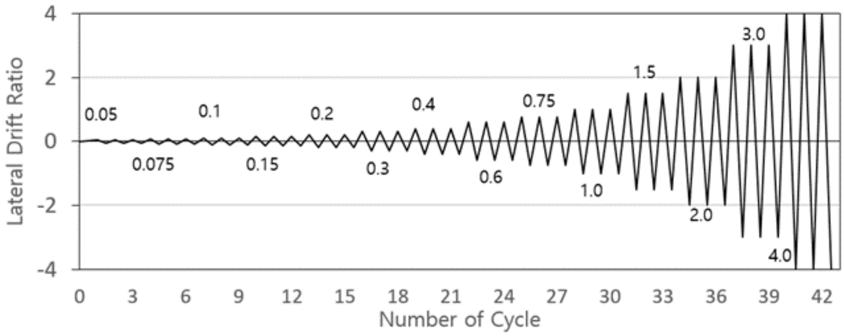


Figure 4-6 Load Protocol

Table 4-3 Loading plan

Load Step	Lateral Drift Ratio[%]	Lateral Deformation[mm]
1	0.05	1.7
2	0.075	2.5
3	0.1	3.4
4	0.15	5.1
5	0.2	6.8
6	0.3	10.1
7	0.4	13.5
8	0.6	20.3
9	0.75	25.3
10	1	33.8
11	1.5	50.6
12	2	67.5
13	3	101.3
14	4	135.0
15	6	202.5

4.2.3 Specimen

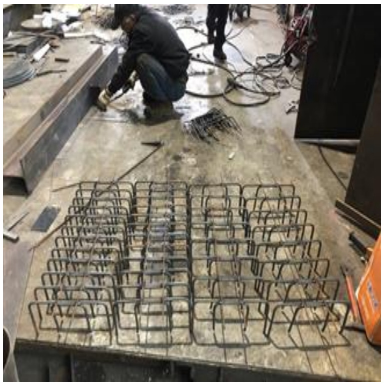
The size of the specimen is as follows. Wall as 1600mm(length) x 3125mm(height) x 200mm(thickness), foundation as 3500mm(length) x 625mm(height) x 1300mm(thickness), head beam as 2400mm(length) x 500mm(height) x 900(thickness). After assembling the foundation reinforcement, the wall reinforcement bar position was fixed to pour concrete. And the foundation concrete was cured for 7 days, the horizontal reinforcement and the transverse reinforcement were aseembeld to the vertical rebar. Then, strain gauages were attached. Concrete pouring was not carried out simultaneously, but in the order of foundation and wall, head beam.in order to maximize the area of confined concrete, the concrete cover thickness of the wall was 20mm. A 20mm spacer was installed between the wall formwork and the reinforcement to maintain the cover thickness before concrete pouring. Concrete compressive strength poured into the foundation and the girder beam was 40MPa, and the concrete strength poured into the wall was 30MPa. The mixture ratio of concrete was cement 395kg/m^3, water 115kg/m^3, fine aggregate 812 kg/m^3, coarse aggregate 981kg/m^3 and admixture 2.77 kg/m^3. And the concrete was an ordinary ready mixed concrete using Portland cement type 1 with the water-admixture ratio 41.8% and the aggregate ratio 45.8%.

Figure 4-7 shows the manufacturing process of the specimens. (a) Rebar processing, (b) assembling the rebar for foundation, head beam, wall, (c) foundation concrete pouring and curing, (d) fabrication and assembly of transverse reinforcement, (e) attaching the strain gauge, (f) concrete pouring into wall and curing, (g) concrete pouring into head beam and curing, (f) completed specimen.

(a) Rebar processing

(b) Assembling the rebar for foundation, head beam, wall

(c) Foundation concrete pouring and curing



(d) Fabrication and assembly of transverse reinforcement

(e) Attaching the strain gauge

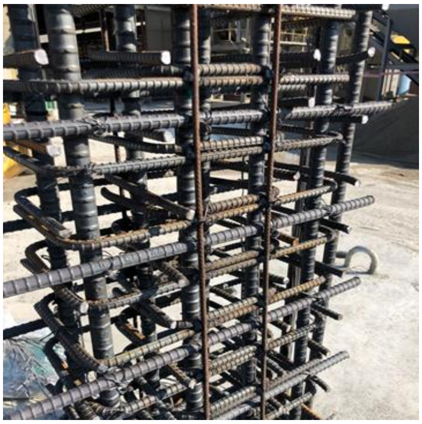
(f) Concrete pouring into wall and curing

Figure 4-7 Test specimen construction procedure

, (g) Concrete pouring into head beam and curing

(f) Completed specimen.

Figure 4-7 Test specimen construction procedure



, (a) Assembly of U-tie module

(b) U-tie module and U-end tie

Figure 4-8 Transverse reinforcement details

4.3 Test result

4.3.1 Failure mode

Figure 4-9, Figure 4-10 and Table 4-4 show the failure mode of specimens and test result. The shear failure was prevented because the nominal shear strength of two specimens was greater than the nominal flexural strength. The failure mode of WS1 was flexural compressive failure due to buckling of the outmost rebar in boundary element and concrete crushing in compressive zone. The failure mode of WS2 was flexural compression failure in which concrete crushing of boundary element and bouckling occurred.

WS1 was the specimen with the vertical spacing of 100mm of the proposed transverse reinforcement detail, and vertical cracks in the boundary element of wall began to occur at drift ratio 2%. After that, the stability of the vertical rebar in boundary element was greatly reduced in the compressive zone, and the concrete crushing and buckling occurred. Due to the buckling of rebar, the horizontal rebar and transverse reinforcement detail were dislocated, and the concrete confinement effect was sharply reduced, resulting in the destruction of core concrete. In addition, because the boundary element lost its resistance to load, the outermost rebar was buckling in the direction of lateral load, and the other D16 rebar in boundary element was buckling in out-of-plane direction. Besides, as the boundary element was destroyed, resistance to the load was concentrated on the web, and the specimen web was also destroyed.

WS2 was the specimen with the vertical spacing of 65mm of the simplified confinement detail, and the reinforcement fracture occurred after buckling of the vertical rebar and cover concrete spalling. In addition, only the cover concrete spalling proceded, and the core concrete was not destroyed by the confinement effect of the transverse reinforcement details. The rebar buckling was also small compared to WS1. Although buckling of the outermost rebar occurred in the direction of loading, the U-end tie did not dislocate in the concrete. U-tie and horizontal rebar were slightly dropped out, bout severe destruction was limited. However, as the lateral displacement increased, the web was destroyed and the concrete fell out.

Figure 4-9 Crack and failure mode of WS1

Figure 4-10 Crack and failure mode of WS2

Table 4-4 Summary of test result

Speci mens	Test results							Predicted strength		Vtest / Vpred		Actual failure mode of specimen
	Positive		Negative		Yield drift ratio δy (%)	Ultimate drift Ratio δu (%)	Drift ductility ratio μ	Vpred [kN]		Positive	Negative	
	V_{test} [kN]	at δ (%)	V_{test} [kN]	at δ (%)				Flexural strength V_f [kN]	Shear strength V_n [kN]			
WS1	789.1	1.52	792.5	1.48	0.56	3.01	5.38	770.4	1231.9	1.02	1.03	FY (RB)
WS2	802.1	3.01	846.7	3.01	0.60	4.00	6.67	781.4	1231.9	1.03	1.08	FY (RB)

* RB : Reinforcement buckling / RF : Reinforcement Fracture / CC : Concrete Crushing

4.3.2 Load-displacement relationship

Figure 4-11 shows the relationship between the lateral load and the lateral drift ratio of the specimen. The drift ratio indicated that the forward lateral displacement was divided by the net height of the wall(H = 3,125m). lateral displacement is defined as the difference between the lateral displacement measured at the head beam and foundation. Figure 4-11 shows the flexural strength together. Table 4-4 shows the ratio of the peak strength to the flexural strength measured as the result of the test. In both specimens, the maximum test strength reched the nominal flexural strength. The peak strength of WS1 was 789.1kN and 792.5kN at $\delta = +1.52\%$ and $\delta = -1.48\%$, respectively. The peak lateral drift ratio was 3.01%, and the strength ratio was 1.02 and 1.03 in the positive and negative directions. Almost the same value as the predicted flexural strength was showed. After the peak strength of WS1, the strength gradually decreased due to the fracture of the outermost vertical rebar of boundary element. WS1 is a wall dominated by the yield of flexural and tensile rebar, so the lateral drift ratio of wall was over 3%.

The specimen WS2, showed 802.1kN and 846.7kN at the peak strength $\delta = +3.01\%$ and $\delta = -3.01\%$. The maximum lateral drift ratio was 4.00%, and the strength ratio was 1.03 and 1.08 in the positive and negative direction. The lateral drift in which the peak strength of WS2 was shown was the lateral drift ratio in which WS1 was completely destroyed. This means that when the vertical spacing of the transverse reinforcement details is $t_w/3$, it had superior ductility compared to the vertical spacing $t_w/2$. The strength of WS2 continuously increased until the lateral drift ratio reached 3%. In addition, the confinement effect was sufficiently maintained at the boundary element, and excellent deformation capacity of the seismic rebar was shown, resulting in maximum lateral drift ratio of up to 4%. The peak strength was also 1.02 times and 1.07 times higher in the positive and negative directions than in WS1. And the smaller the spacing of the details, the greater the strength was represented.

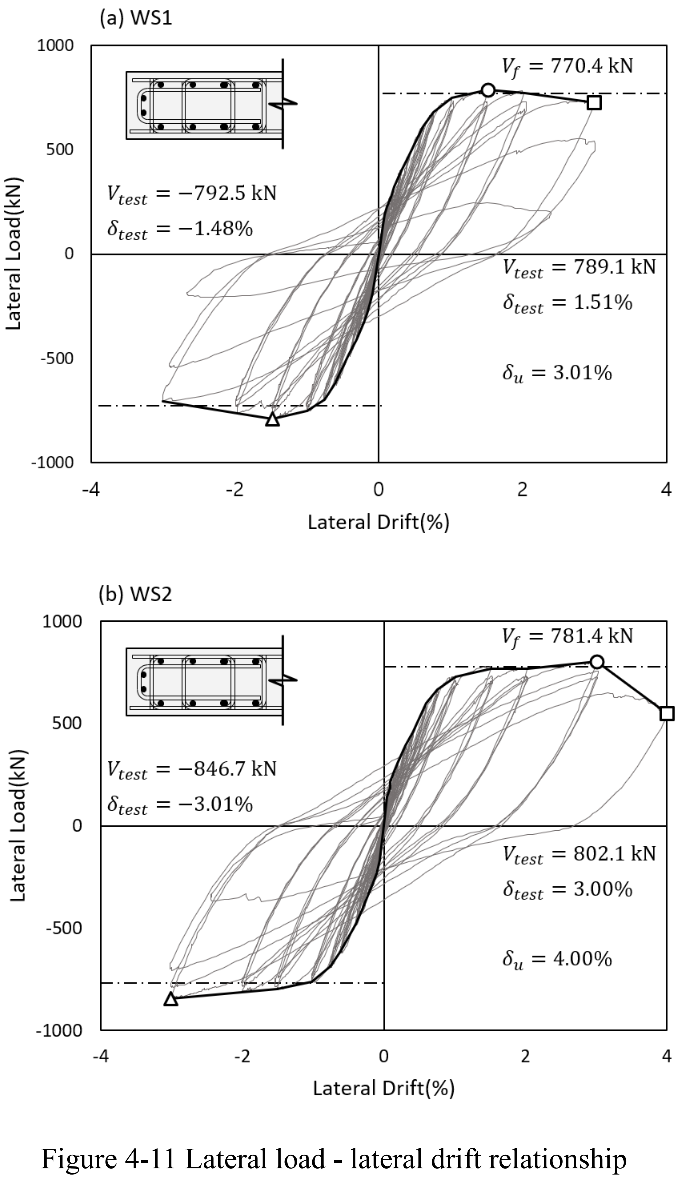


Figure 4-11 Lateral load - lateral drift relationship

4.3.3 Lateral displacement ductility

Figure 4-12 was based on the lateral load – lateral drift relationship graph, and the drift ductility ratio was calculated. The lateral yield drift was determined by assuming the equivalent elasticity of which the elastic stiffness was confimed at 75% of the maximum strength of the envelop curve obtained as a result of the actual test. The maximum drift was determined by the test result. the drift ductility ratio was defined as the ratio($= \delta_u/\delta_y$) of the ultimate drift to the yield drift. Table 4-4 shows the yield drift ratio, ultimate drift ratio, drift ductility ratio. The drift ductility ratio of the specimen was 5.38 and 6.67, respectively, in WS1 and WS2, and 1.24 times higher in the WS2 specimne with narrowe spacing. As a result of the test, the initial stiffness of WS2 was lower than that of WS1, but the increase in strength appeared until drift of about twice that of WS2 compared to WS1.

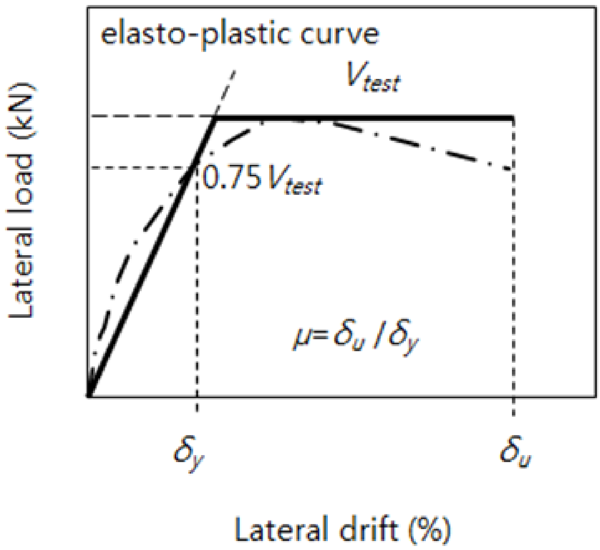
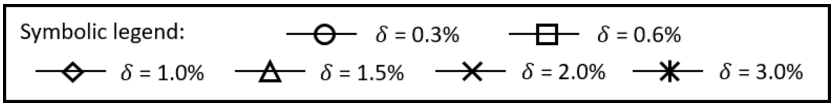
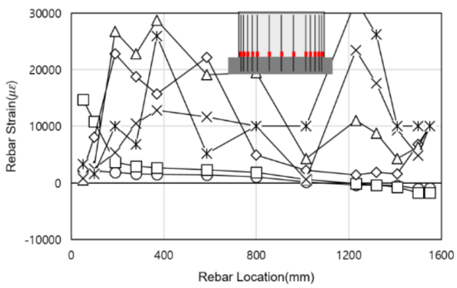


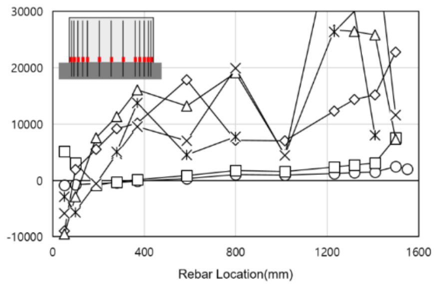
Figure 4-12 Envelop curve

4.3.4 Strain of reinforcement

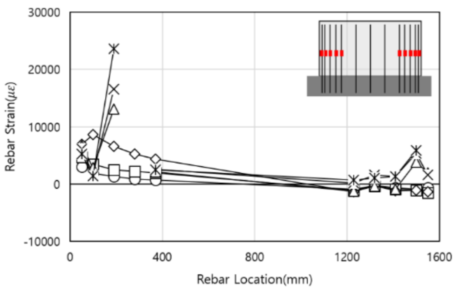




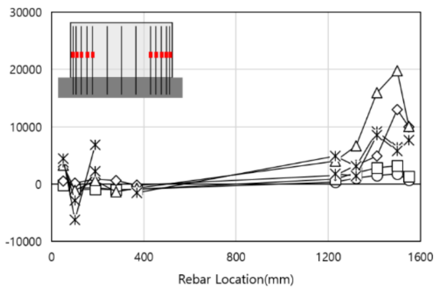
(a) WS1 – positive direction



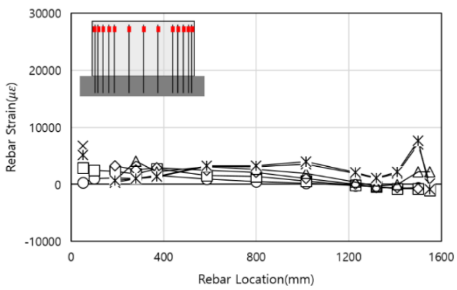
(b) WS1 – negative direction



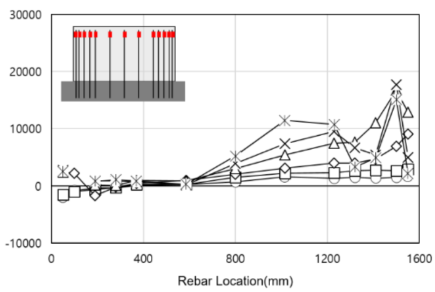
(c) WS1 – positive direction



(d) WS1 – negative direction

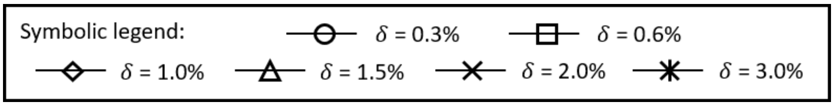


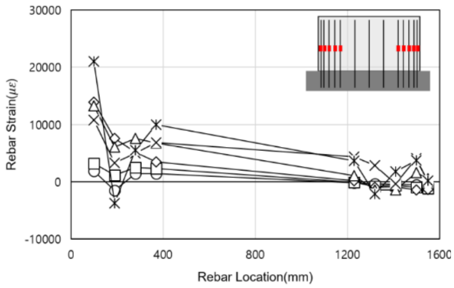
(e) WS1 – positive direction



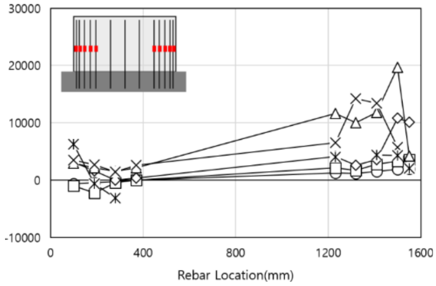
(f) WS1 – negative direction

Figure 4-13 Strain distribution of lower section vertical reinforcement

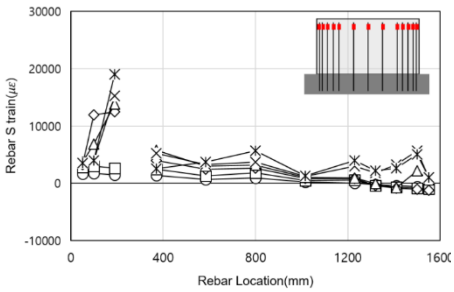




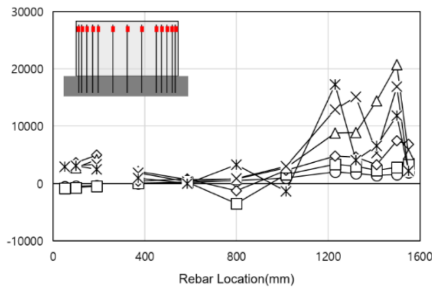
(a) WS2 – positive direction



(b) WS2 – negative direction

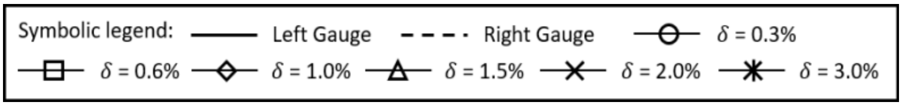


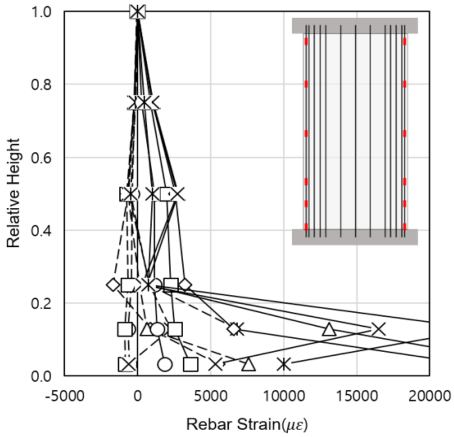
(c) WS2 – positive direction



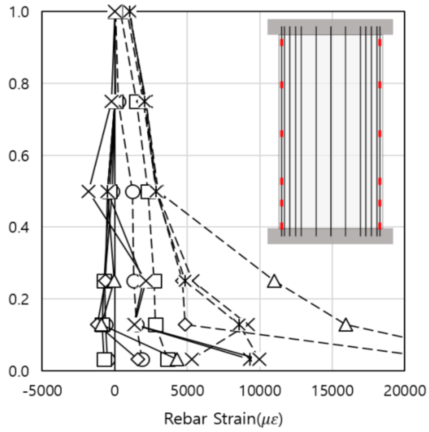
(d) WS2 – negative direction

Figure 4-13 Strain distribution of lower section vertical reinforcement

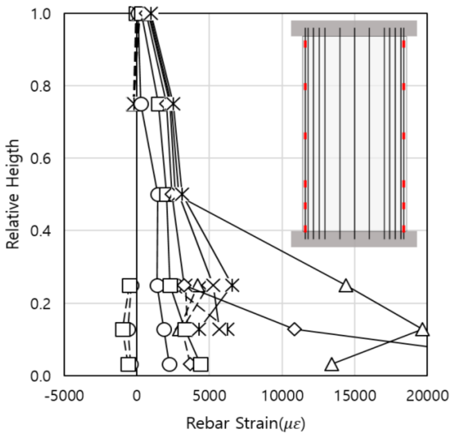




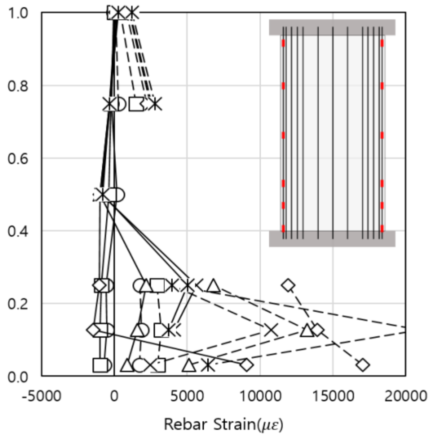
(a) WS1 – positive direction



(b) WS1 – negative direction

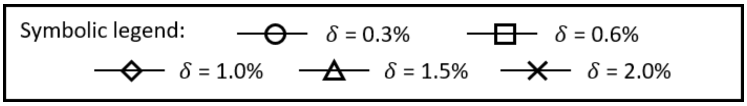


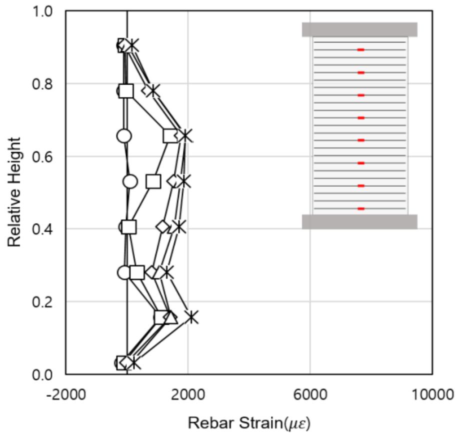
(c) WS2 – positive direction



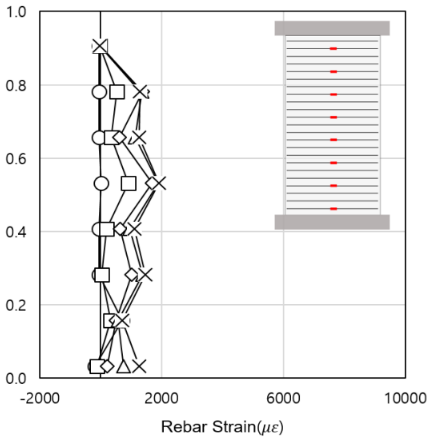
(d) WS2 – negative direction

Figure 4-14 Strain distribution of boundary element vertical reinforcement

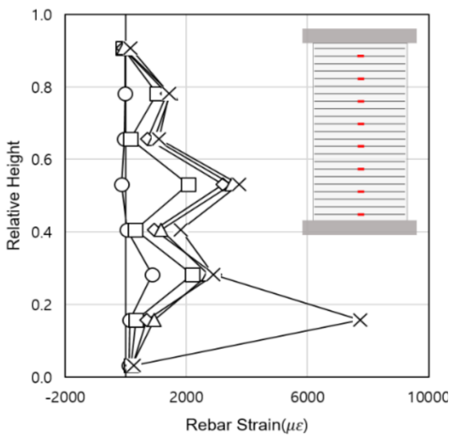




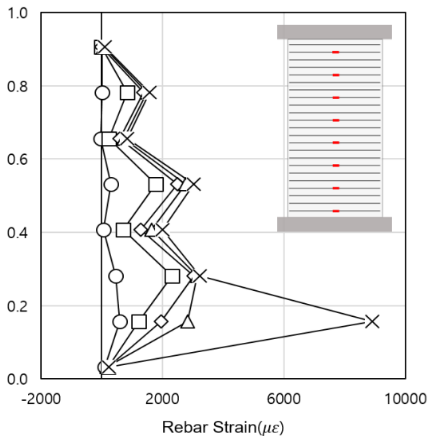
(a) WS1 – positive direction



(b) WS1 – negative direction



(c) WS2 – positive direction



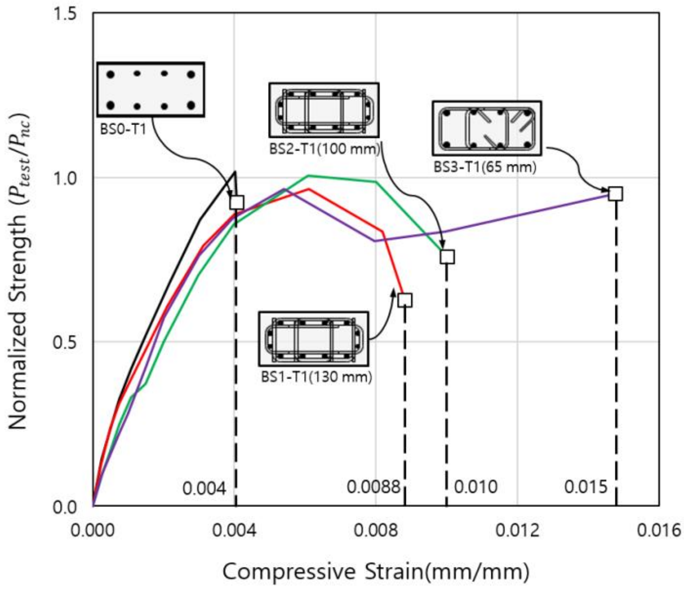
(d) WS2 – negative direction

Figure 4-15 Strain distribution of web horizontal reinforcement

4.3.5 Effect of test parameter

The parameter of the cyclic lateral loading test was the vertical spacing of the transverse reinforcement detail. For WS1 and WS2, the spacing of the simplified confinement detail was 100 mm and 65 mm, respectively, and a narrower spacing was applied than the spacing of the detail in the cyclic axial loading test. According to the results of previous studies, there was no significant difference in strength and lateral displacement ratio, although the spacing of the transverse reinforcement detail was reduced. However, the strength and lateral drift ratio increased as the spacing of the details decreased. For the specimen WS1, the maximum lateral drift ratio was 3.01 %, and the maximum strength was 789.1 kN and -792.5 kN in the positive and negative directions, respectively. And for the specimen WS2, the maximum lateral drift ratio was 4.00%, and the maximum strength was 802.1 kN and -846.7 kN in the positive and negative directions, respectively. Comparing WS1 and WS2, the maximum lateral drift ratio of WS2 was 1.33 times higher than that of WS1 (see Figure 4-11), and the maximum strength was 1.02 times and 1.07 times, showing better performance. In addition, the lateral drift ratio at maximal strength was about 2 times higher in WS2 than in WS1.

Figure 4-16 shows the normalized strength-compressive strain relationship for the results of the cyclic axial loading test and cyclic lateral loading test. As a result of the boundary element test, the specimens BS1, BS2, and BS3 with the transverse reinforcement details were 2.2 times and 2.5 times, respectively, compared to specimen BS0-T1 with non-transverse reinforcement detail, showed 3.75 times higher compressive strain. As a result of the lateral loading test, WS2 showed 1.66 times higher compressive strain than that of WS1.



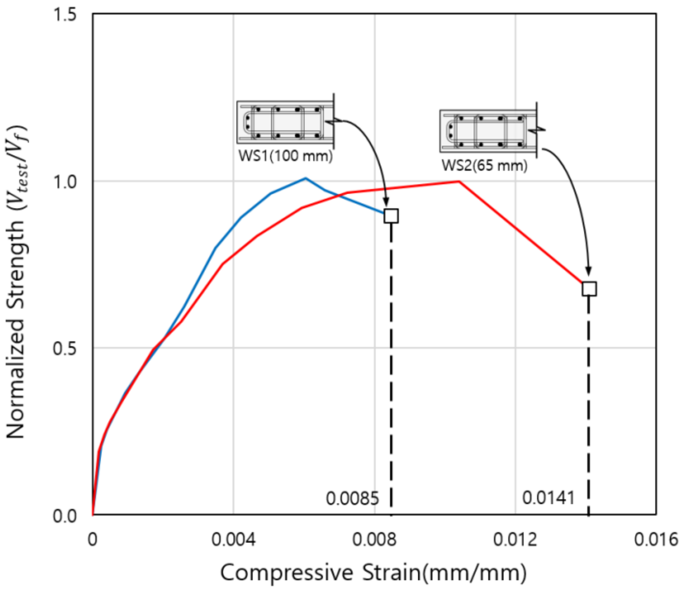


Figure 4-16 Comparison of cyclic axial loading test for boundary element and cyclic lateral loading test for RC wall

Chapter 5. Conclusion

In this study, the simplified confinement detail that can improve the deformation capacity and constructability of wall boundary elements were developed. To verify the performance of the simplified confinement detail, cyclic loading test and cyclic lateral loading test were performed. The main characteristics of the boundary elements applied on the simplified confinement detail according to the results of this study are summarized as follows.

[1] In the specimens without transverse reinforcement details of cyclic axial loading test, the specimen using seismic grade rebar had the maximum compressive strength ratio of 1.27 times when the load history was T1 (1:5) compared to the specimen using normal reinforcement bar. And the tensile strength ratio is 1.04 times. The deformation capacity was 1.33 times the compressive and tensile strains, respectively. Thus, As a result of the experiment, the excellent deformability of the seismic grade rebar was shown, and the increase in the deformation capacity was larger when seismic grade rebars were used.

[2] In the case of using the simplified confinement details, there was an effect according to the vertical spacing of the transverse reinforcement. The maximum compressive and tensile load ratios of specimens BS1 and BS2 tested by loading history T1 were 1.07 times and 1.05 times, respectively. Comparing the maximum compressive strain and maximum tensile strain ratios, BS2 showed better performance by 1.12 times and 1.25 times, respectively.

The maximum compressive and tensile load ratios of specimens BS1 and BS2 tested by load history T2 were 1.09 times and 0.99 times, respectively. Also, comparing the maximum compressive strain and the maximum tensile strain ratio, BS2, which has a narrower spacing, by 1.33 times each, has better deformability. Because this indicates that the simplified detail can have better deformation capacity as the spacing is narrower, the vertical spacing of the details was adjusted to 65 mm and 100 mm, respectively, by reflecting the results of the cyclic axial loading test in the cyclic lateral loading test. In addition, since the performance of simplified confinement detail showed intermediate deformation capacity between non-transverse reinforcement detail and special seismic detail in the load history T1, proper ductility can be secured when using simplified confinement detail.

[3] When the specimen with the same cross section and detail was tested with two loading histories, the compressive strains of BS1, BS2, and BS3 were 1.47 times that of T1(1:5) and T2(1:10), respectively, 1.27 times, 1.93 times. T2 showed a difference of 1.56 times, 1.63 times, and 1.06 times of tensile strain compared to T1, respectively. In the case of T2, the plastic deformation due to tension was larger than that of T1. Thus, buckling of the longitudinal rebar can occur relatively easily, which leads to concrete cover spalling and compression failure at low compressive strain. Therefore, it is possible to increase the stability of the boundary element with high tensile strain by adjusting the vertical spacing of the transverse reinforcement details and improving the confinement details.

[4] As a result of the experiment, the maximum lateral drift ratios of the two

specimens, WS1 and WS2, were 3.01% and 4.00%, respectively. And the maximum strengths were 789.1 kN and -792.5 kN for WS1 and 802.1 kN and -846.7 kN for WS2 in the positive and negative directions, respectively.

As for the failure mode, the crushing of the boundary element concrete started at drift ratio of 2% in WS1, and then the specimen was destroyed by buckling of the boundary element vertical rebar at lateral drift ratio of 3%, failing the core concrete, and breaking the web of the wall. In WS2, the strength of the specimen increased up to 3%, which is the failure drift ratio of WS1, and vertical cracks due to compression at the boundary element started to appear from 3%. At 4%, the strength was sharply lowered and destroyed due to buckling of the outermost rebar of the boundary element, dropping of covered concrete, and destruction of the web. In addition, the compressive strain at the end-region of the wall was 0.0141 for WS2, which was 1.66 times higher than that of WS1. In this study, it was possible to confirm the excellent deformation performance of seismic grade reinforcement, and in WS2, the simplified details did not loosen and the concrete was well restrained.

In conclusion, the developed simplified confinement detail can greatly increase the workability at the construction site, and as a result of the experiment, superior deformation performance and structural stability can be secured, compared to ordinary concrete shear wall. In the performance-based seismic design, the use of simplified confinement details can secure the ductility and structural safety of residential buildings instead of the special seismic details specified in the design standard.

References

[1] Korean Design Standard (2019). Seismic Design Code of Buildings (KDS 41 17 00), Korean Design Standard.

[2] ACI Committee 318 (2019). Building Code Requirment for Structural Concrete (ACI 318-19) and Commentary. American Concrete Institue, Farmington Hills, MI, p324

[3] ASCE. (2017). “Minimum design loads for buildings and other structures.” SEI/ASCE 7 -16, Reston, VA.

[4] Hawkins, N. M., & Ghosh, S. (2004). “Acceptance Criteria for Special Precast Concrete Structural Walls Based on Validation Testing”, PCI Journal, 49(5), 78−92

[5] Y.H. Chai and D.T. Elayer. (1999). “Lateral Stability of Reinforced Concrete Columns under Axial Reversed Cyclic Tension and Compression” ACI Struct J, 96(5), 780-790

[6] Travis. S. Welt, Leonardo M. Massone, James M. LaFave, Dawn E. Lehman, Steven L, McCabe & Pablo Poanco. (2017). “Confinement Behavior of Rectangular Reinforced Concrete Prisms Simulating Wall Boundary Elements.” J. Struct. Engrg., ASCE, 143(4).

[7] Dazio, A., Beyer, K., & Bachmann, H. (2009). “Quasi-static cyclic tests and plastic hinge analysis of RC structural walls.” Engineering Structures., 31(7), 1556-1571

[8] S.H. Kim, E.K. Lee, S.M. Kang, H.G. Park and J.H. Park. (2021), "Effect of boundary confinement on ductility of RC walls" Engineering Structures., 230

[9] Mander, J. B, Priestley, M. J. N., and Park R. (1988b). "Theoretical Stress-Strain Model for Confined Concrete." J. Struct. Engrg., ASCE, 114(8), 1804-1826

[10] R. Taleb, M. Tani and S. Kono. (2016). "Performance of Confined Boundary Regions of RC Walls under Cyclic Reversal Loadings" J. Advanced Concrete Technology. 14(4), 180-124

[11] C.W. Hilson, C.L. Segura and J.W. Wallce, "Experimental Study of Longitudinal Reinforcement Buckling in Reinforced Concrete Structural Wall Boundary Elements" Proceedings of the 10th National Conference in Earthquake Engineering, Earthquake Engineering Research Institute, Anchorage, AK, 2014

[12] Y.S. Chun, K.H. Lee, H.W. Lee, Y.E. Park and J.K. Song. (2013). "Seismic performance of Special Shear Wall Structural System with Effectively Reduced Reinforcement Detail", Journal of the Korea Concrete Institute, 25(3), 271-281

[13] J.W. Song, Y.S. Chun, J.K. Song and K.H. Yang. (2018) "Seismic Performance of Special Shear Wall with Special Boundary Element Confined by Overlapping Hoops", Journal of the Korea Concrete Institute, 30(1), 47-58

초 록

간략 선조립 상세를 적용한 RC 휨 벽체 경계요소의 연성능력

박 목 인

서울대학교 건축학과 대학원

최근 두 차례의 큰 지진인 경주지진 및 포항지진이 연달아 일어나 많은 주거 건물에 손상이 발생하였다. 최근 들어, 국내 지진발생 빈도 및 지진발생 빈도 및 지진하중의 위험이 증가함에 따라 공동주택에서 대부분 사용하고 있는 철근콘크리트 벽식구조의 내진성능의 중요성이 강조되고 있다.

고층화가 되어가는 공동주택 벽체는 중력하중에 의하여 벽체 압축성능의 15~30%에 해당하는 큰 축력이 작용한다. 전단벽에 과도한 축력이 작용하는 경우, 벽체 단부의 압축변형률 증가로 인해 지진하중 시 콘크리트 압괴가 쉽게 유발되며, 내력벽의 연성능력 감소로 붕괴 위험성이 증가된다. 또한 내력벽 수직철근에 고강도 철근의 사용이 증가하고 있지만, 고강도 철근은 연성능력이 저하되는 단점이 있다. 고강도 철근은 지진하중 하에서 저주파 피로파단 및 국부좌굴이 발생하여 벽체의 성능을 저하시킬 우려가 있기 때문에 연성능력이 뛰어난 내진용 철근의 사용이 요구되며, 최근 개정된

초 록

건축물내진설계기준(KDS 41 17 00)에서는 중연성도 및 고연성도 구조형식에 내진용 철근의 사용을 의무화하였다.

현행 건축물 내진설계기준에 따라서 높이 60m 이상이면서 내진설계범주 D에 속하는 철근콘크리트 내력벽 시스템의 벽체 경계요소에는 폐쇄형후프와 연결철근으로 이루어지는 특수내진상세를 사용하도록 규정한다. 그러나 국내 벽체는 얇은 벽체를 사용하며, 이와 같은 얇은 벽체를 특수전단벽으로 설계할 경우 배근과밀로 인하여 시공성과 경제성 저하로 이어질 수 있다. 그러므로 시공성과 경제성을 확보할 수 있는 벽체 내진 상세의 개발이 필요하며 이를 통해서 고층 내력벽의 연성능력을 향상시킬 필요가 있다.

따라서 본 연구에서는 내진 상세의 선조립을 통해 시공성과 경제성을 향상시키는 횡보강 상세를 개발하며, 선조립 횡보강 상세의 유효성을 선검증하기 위해 경계요소 1축반복가력 실험을 수행하였다. 또한 선조립 상세를 배근한 철근콘크리트 벽체의 연성성능을 평가하기 위해 반복 횡가력 실험을 수행하였다. 주요 변수는 철근의 종류, 횡보강 상세의 종류, 횡보강 상세의 간격, 하중이력이었다.

경계요소 1축 반복하중 실험결과, 상대적으로 높은 축력을 받는 벽체의 이력거동을 모사한 하중이력에서는 횡보강 상세의 간격이 좁아질수록 변형성능이 증가하였다. 또한 낮은 축력과 철근비를 가지는 벽체의 이력거동을 모사한 하중이력에서는 선조립상세의 성능이 특수내진상세와 동일한 변성성능을 나타내어 유효성을

대안연성상세로서의 변형성능을 검증하였다. 그리고 벽체 반복하중 실험결과, 상세의 간격이 좁아질수록 벽체의 횡변위비가 증가하는 결과를 나타냈으며, 경계요소의 심부콘크리트를 잘 구속하는 결과를 나타냈다. 또한, 내진용 철근은 높은 항복비를 나타내기 때문에 공칭 휨강도에 대한 초과 강도비가 벽체에서 크게 나타났다.

따라서 본 연구에서 제안된 선조립 상세를 사용하면, 현행 특수전단벽의 내진상세 보다 뛰어난 시공성과 경제성 향상을 기대할 수 있다. 또한 보통전단벽 보다 뛰어난 연성능력을 발휘하여 단부에서 발생할 수 있는 콘크리트 취성파괴를 방지하여 지진하중으로부터 구조적 안전성을 높일 수 있을 것으로 기대한다.

주요어 : 간략 선조립 상세, 철근콘크리트 휨 벽체, 경계요소, 내진용 철근

학 번 : 2019-29053