공학석사 학위논문

Performance-Based Seismic Design considering Slab Flexural Stiffness and Arrangement of Vertical Reinforcement in Wall

슬래브 휨 강성 및 벽체 수직철근 배근 방식을
고려한 공동주택의 성능기반 내진설계 기법 연구

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김 환 철

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지도 교수 박 홍 근

이 논문을 공학석사 학위논문으로 제출함

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Abstract

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As a risk of earthquakes increases drastically in Korea, seismic performance evaluation and performance-based seismic design (PBSD) projects have been frequently. In the PBSD, slab is designed to resist gravity loads only and usually modeled as a rigid diaphram with no flexural stiffness. Because slab thickness of most residential buildings built in 1990s was 135~150 mm which was not thick enough to consider flexural stiffness of slab, so it has not been used as a lateral resistance component. However, after standard slab thickness of residential buildings was set to be 210 mm due to the tightening of noise regulations in apartment in 2009, there is a need to take into account the flexural stiffness of slab.

Considering the flexural stiffness of slab, seismic loads applied to slab of each floor are not same. This makes slab deisgn to be different and increases a workload compared with a typical design method. This also takes more time for modeling and analysis of buildings. Therefore, in this paper, the maximum flexural stiffness of slab that can be considered without changing design is found using FEA and PBSD considering flexural stiffness of slab is conducted. Also before applying the slab stiffness in the PBSD, response spectrum analysis is performed in order to investigate the effect of in-plane stiffness and out-of-plane stiffness of slab on the dynamic behavior of 20-story shear wall buildings.

In addition, PBSD is carried out considering an arrangement of vertical reinforcement in walls for more economical deisgn. Generally, the typical vertical reinforcement of walls is equally spaced. In this study, however, nonlinear static analysis and time history analysis are conducted for the shear wall buildings where vertical reinforcement of walls are concentrated at the ends remaining the same amount of reinforcement to evaluate a seismic performance and dynamic characteristics.

As an analysis result, the in-plane stiffness of slab is relatively large compared to the lateral stiffness of vertical members in concrete buildings, considering flexural stiffness of slab with rigid diaphram is more efficient than modeling the slab as shell element or plate element. Taking into consideration a flexural stiffness of slab, inter story drift ratio of the building is reduced and lateral loads are redistributed. Also shear force distribution between the upper and lower floors, reaction force distribution and the load difference acting on the large wall and the small wall become more uniform due to redistributed loads. The maximum flexural stiffness of slab which can be considered without design change is about 10%. When this is reflected in the PBSD, the bending strength of the building is increased more than 1.5 times and also plastic rotation angles of coupling beams is considerably decreased. Furthemore, it can become more economical by reducing the amount of reinforcement in total walls by about 6% compared with the existing design using rigid diaphram only.

When the vertical reinforcmenet of walls are concentrated at the ends, the bending strength of the whole building increases by 5~6% and inter story drift ratio decreases slightly. Also it can be possible to reduce the amount of reinforcement in all walls about 4% by downsizing the increased bending strength to the standard model level. But there is no big difference in dynamic characteristic change in nonlinear time history analysis.

Keywords : Performance-based seismic design, Slab flexural stiffness, Nonlinear analysis, Perform-3D, Wall reinforcement

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Contents

List of Tables

List of Figures

Chapter 1. Introduction

1.1 General

It is expected to break the belief that ‘Korea is the safe zone for earthquake’. Since 1978, when the earthquake observation began in Korea, the number of earthquake have been increasing according to the Korea Meteorological Administration. In particular, after the Gyeongju 912 earthquake of magnitude 5.8 on the Richter scale in September 12, 2016, the earthquake occurrence increased rapidly as like **Figure 1-1**. Pohang earthquake of magnitude 5.4 which occurred in November 11 last year caused the greatest damage to the country, resulting in damage of about 55 billion won. So earthquake is an important factor in a building design because it can cause huge damage if taking place in a densely populated city.

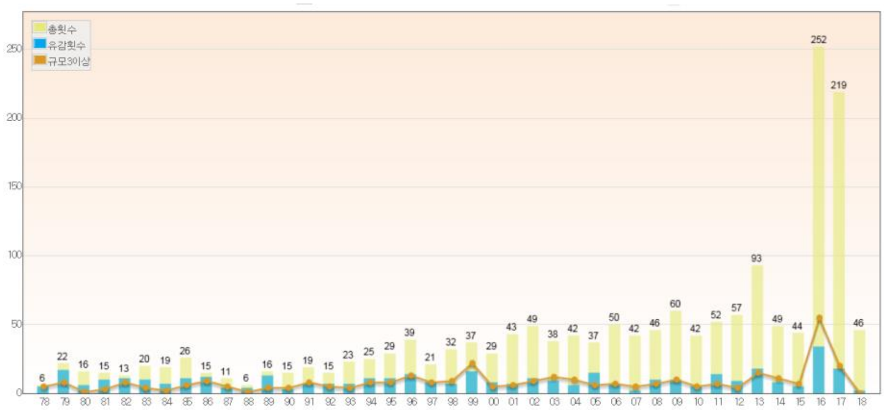


Figure 1-1 Earthquake record in Korea

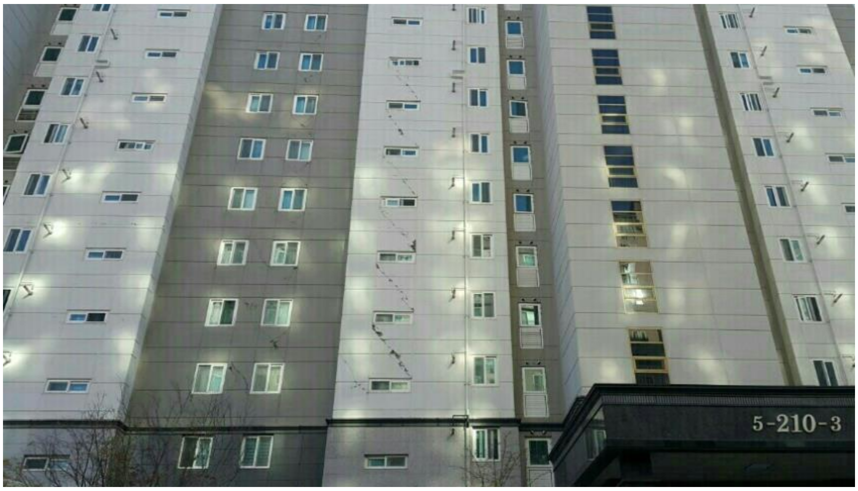


Figure 1-2 Damage of the Pohang earthquake

Generally, to contemplate the seismic load appropriately, response spectrum analysis is conducted on the assumption of elastic behavior. At this analysis, reduced earthquake load is used considering inelastic lateral deformation capacity of buildings using response modification factor(R). This factors are prescribed as an uniform value for a special structural system. In fact, however, the ductility of buildings can vary significantly according to the design method. Furthermore, in the case of large buildings or abnormal shaped buildings, such as Busan Cinema Center, the R factor can not be predicted accurately. Therefore, to assure the structural safety against earthquake, the actual performance of the building should be verified by performing inelastic nonlinear analysis. With this concept, performace-based seismic design(PBSD) is emerged. In the PBSD, target performance level of a building to be designed is determined, and then it verifies that the building meets the target performance level using inelastic nonlinear static analysis and nonlinear time history analysis.

When creating analysis model to perform response spectrum analysis or PBSD, the slab is usually modeled as a rigid diaphram with an infinite in-plane stiffness and no out-of-plane stiffness for simplicity of analysis. In fact, slab thickness of residential buildings that were built in the 1990s is about 135~150 mm which is not thick enough to consider the out-of-plane flexural stiffness of slab. However, the thickness of standard floor slab is set to be 210 mm due to block the inter-floor noise by the Korean code in 2009 and there is a need to take into account a flexural stiffness of slab because it is not thin now. But there are no standards for slab modeling method in Korea since the rigid diaphram has been used for a long time.

PBSD is also used not only for accuracy but also for economic efficiency in practice. Generally vertical reinforcement of wall is eually spaced. However, if the reinforcement is concentrated at the ends of the wall without changing an amount of reinforcement as shown in **Figure 1-3**, the bending strength of wall would be increasesd and it can possible to get better seismic performance with the same reinforcement. Or it can reduce the amount of reinforcement of walls by downsizing the increased bending strength to typical building level.

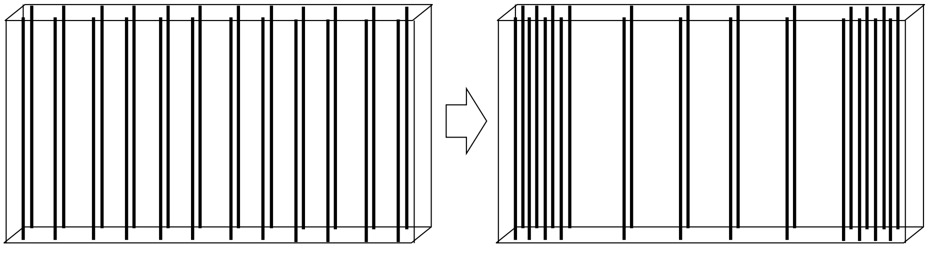


Figure 1-3 Change the arrangement of vertical reinforcement

1.2 Scope and Objectives

The purpose of this study is to investigate the influence of slab stiffness on the dynamic behavior of flat-type residential building using linear and nonlinear analysis and propese the appropriate modeling methods of slab for PBSD. Another objective is to confirm the effect of vertical reinforcement concentrated at the end of the walls and make an economic assessment for alternatives.

1.3 Organization

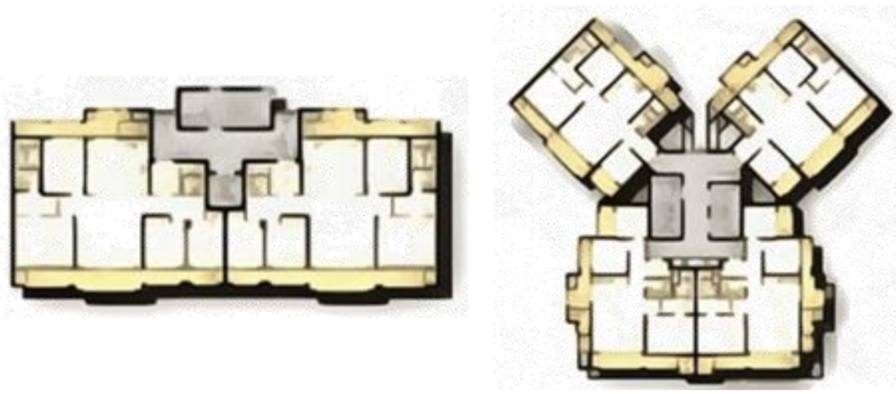
This thesis consists of six main chapters. The introduction of the study is briefly shown in **Chapter 1**. The literature reviews of the design codes and guidelines for PBSD, and the previous studies about the flexural stiffness of the slab are shown in **Chapter 2**. In **Chapter 3**, response spectrum analyses are conducted to check the effect of slab stiffness on the dynamic behavior for shear wall building and members are designed for PBSD. Also FEA of slab are carried out to find the maximum flexural stiffness of slab which can be considered without changing design. The nonlinear modeling, nonlinear static analysis and nonlinear time history analysis of the models considering out-of-plane flexural stiffness of the slab are performed in **Chapter 4**. The nonlinear modeling, nonlinear static analysis and nonlinear time history analysis of the model modified the spacing of the vertical reinforcement of wall are carried out in **Chapter 5**. Last, the summary and conclusions present in **Chapter 6**.

Chapter 2. Literature Review

2.1 Residential building design

2.1.1 Characteristic of residential buildings

As shown in **Figure 2-1**, there are two types of residential building: flat-type and tower-type. Nowadays, flat-type residential building is preferred since it is more sunny and well-ventilated than tower-types. Flat-type residential building has a lot of shear walls such as partition wall and long outer walls in the short side direction. Because of it's characteristic, it has a strong bending strength and stiffness in that direction and this makes the first mode of the building occur in the long side direction.



(a) flat-type (b) tower-type

Figure 2-1 Typical plans of residential building in Korea

2.1.2 Practical design

Generally, Shear wall system is used as a lateral-force-resisting system of residential buildings in Korea. Shear walls mainly resist lateral loads such as wind load and earthquake load. Both load cases are considered in the building design, but generally earthquake load has more impact on buildings with 30 stories or less. The earthquake load is usually evaluated by equivalent static analysis or response spectrum analysis using Midas programs.

In the modeling process, slab is assumed to be a rigid diaphram which has an infinite in-plane stiffnes and no out-of-plane stiffness. This means slab does not resist lateral loads. So, slab is designed to resist gravity loads only with the finite element analysis results. There are two main reasons for this, one of which is for the convenience of analysis process and the other is to reduce the amount of slab design using reference story slab. For the simplicity of analysis and the safety, wall is usually modeled as a membrane element with no out-of-plane stiffness. And vertical reinforcement of wall arranges in an equal spacing to construct more easily.

2.1.3 Performance-based seismic design

In practice, design projects using performance-based seismic design(PBSD) are becoming frequent because it is more reliable and economical than strength design using response spectrum analysis. Furthermore, PBSD is more advantageous for seismic design since it can determine a member yielding first and induce ductile failure mechanism.

As shown **Figure 2-2,** the first thing to do in PBSD is to determine the performance objective, which is considering the use and the importance of the building. Performance criteria can be categorized as **Operational, Immediate Occupancy, Life Safety** and **Collapse Prevention**. For example, essential facilities or harzardous facilities with operational limit state should function normally after an earthquake. Basic facilities with performance goal for life safety, such as apartment, permit buildings to be damaged moderately but structure must remain stable.

After the target performance is set, basic design of building is conducted using linear elastic analysis. This is because it is practically very difficult to determine the size of structural members and a number of reinforcement using nonlinear inelastic analysis.

When making a nonlinear analysis model, material nonlinearity like expected strength and degradation is considered. And members is modeled as continuum, distributed inelasticity(fiber element) or concentrated hinge model which can depict approximate nonlinear behavior. Viscous damping and p-delta effect are also taking into account in the model.

In the PBSD, the seismic performace of the building is evaluated through two kinds of nonlinear inelastic analysis: Nonlinear static analysis and nonlinear time history analysis. Before confirming the results of the nonlinear time history analysis, the nonlinear static analysis called push-over analysis is performed to ensure the reliability of the analysis model by checking the overstrength, location of performance point and interstory drift ratio at the point. In the nonlinear time history analysis, the performance evaluation is carried out by the average response of ground motions. As verifying that interstory drift ratio and plastic rotation angle of members would be within the target performance criteria, the design is ended.

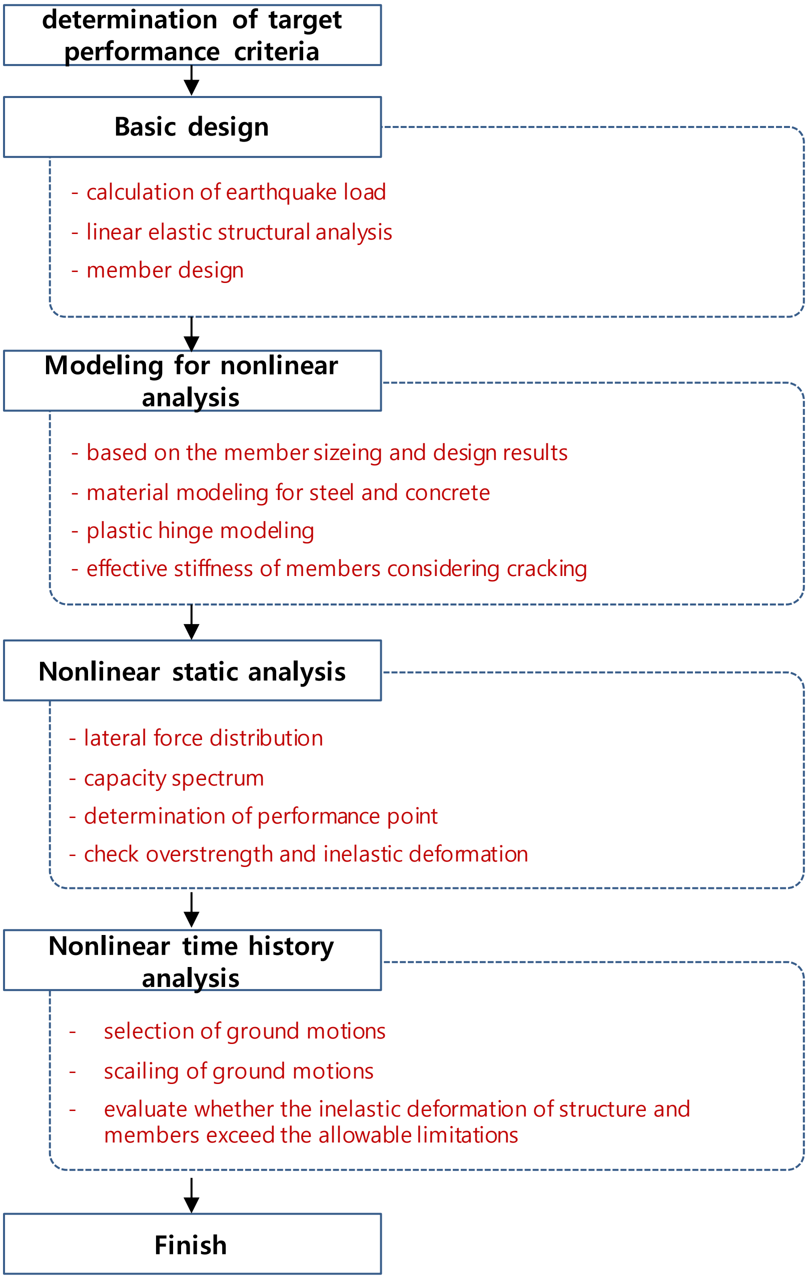


Figure 2-2 The flow of performance-based seismic design

2.2 Design code and guidelines

2.2.1 AIK-G-001-2015

As an interest in PBSD has increased, the Architectural Institute of Korea published the Guidelines for Performance-Based Seismic Design of Residential Buildings(AIK-G-001-2015) in 2015. This permits to design a building based on various performance objectives using nonlinear analysis when it is difficult to apply the design coefficients and factors in KBC code. The guideline is affected on the ATC72-1, FEMA 440(2013) and ASCE 41-13(2014), which have the similar procedure of design. But modeling parameter is revised more conservatively considering the condition of Korea industry.

In the guideline, the target performance of residential buildings in Korea should satisfy the life safety level of class 1. **Table 2-1** shows the performance objectives suggested by the guideline.

Table 2-1 Performance objectives

Seismic use group	Performance objectives	
	Performance level	Seismic harzard
S	Operational (or immediate occupancy)	**1.0** times design effective ground acceleration
	Life safety & collapse prevention	**1.5** times design effective ground acceleration
1	Life safety	**1.2** times design effective ground acceleration
2	Life safety	**1.0** times design effective ground acceleration

When designing performace-based seismic design in accordance with the guideline, the magnitude of the base shear force used in the basic design shall be more than 75% of the force calculated though the equivalent static analysis. Since nonlinear analysis is more accurate and reliable than elastic analysis, it allows to use 10% lower design seismic loads than the 85% used in the response spectrum analysis.

Also the guideline presents the parameters and details of the modeling for nonlinear static analysis and nonlinear time history analysis like expected strength of material and effective stiffness of members modified for the domestic situation.

The seismic performance evaluation of nonlinear static analysis procedure is based on the equivalent linearization method in FEMA 440. The performance of the building is evaluated by the state of global system and individual members at the performance point. At this point, the interstory drift ratio of building shall not be exceed 1.5% to satisfy LS according to the guideline.

In the nonlinear time history analysis, seven ground motions are sacled so that the average spectra of ground motions are not less than 90% of 1.3 times the target spectra. The performance evaluation is conducted by the average response of ground motions and the same interstory drift limit of nonlinear static analysis is applied to the seismic evaluation of the building. The individual members are evaluated by dividing into the force-controlled members or deformation-controlled members, which is the same as ASCE 41-13.

2.2.2 ASCE 41- 13

ASCE 41-13 is one of the most widely used guidelines for nonlinear analysis and seismic performance evaluation. It consists of nonlinear anlysis procedure like NSP and NDP and acceptance criteria for beam, column and wall. Also it includes general considerations of concrete shear wall.

In the nonlinear static analysis, the seismic performance of building is evaluated at the performance point, which is the intersection of the capacity curve and demand curve. And the seismic performance in nonlinear time history analysis is evaluated directly by the responses of a series of ground motionse.

The guideline also deals with various considerations that are used in nonlinear analysis such as force-deformation relations of the component, effective stiffness, strain limits, chord rotation of the member, diaphrams and damping. In the code, the effect of diaphram flexibility shall be considered where the length-to-width ratio of diaphram exceeds 2.0.

2.2.3 PEER/ATC 72-1

The PEER/ATC 72-1 is specialized in the nonlinear modeling of tall buildings. It presents recommended modeling approaches for shear wall components and diaphram.

Fiber element model which is commomly used for nonlinear modeling of shear walls involves subdividing the wall section into concrete and steel fiber and these are defined individually. In this model, effective stiffness values are not used since load versus deformation response of a fiber model depends on the uniaxial material stress-strain relations specified for the concrete and steel fibers, level of axial load and current condition of the elements.

From the guideline, diaphram can be modeled as **rigid**, **semi-rigid**, or **flexible**. Rigid diaphram that is widely used in practice is assumed to be infinitely rigid compared to the vertical elements of the seismic-force-resisting system. Distribution of lateral force is based on the relative stiffness of the vertical elements and differences between center of mass and center of rigidity cause plan torsion that is distributed to vertical elements. This is the most common approach for modeling concrete diaphram. Semi-rigid diaphram includes finite stiffness in the analysis model and stiffness is computed based on slab thickness, dimensions and material properties. It is the most realistic model but more time consuming and difficult to apply. And flexible diaphram is assumed to be infinitely flexible compared to the vertical elements and this is typically not applicable for concrete.

2.3 Previous studies

2.3.1 Kim, Lee and Kim

Lee et al. has performed many analytical studies on the dynamic behavior of the structures considering the effect of slab stiffness in the seismic analysis. Slab is classified into three types; rigid diaphram, semi-rigid diaphram and flexible diaphram to evaluate the influence of in-plane stiffness and out-of-plane stiffness of slab.

According to the analysis results, Considering the out-of-plane flexural stiffness of slab, the natural period of the structure is shortened and this induce the increase of earthquake load. Because slab is usually modeled as rigid diaphram which does not consider the flexural stiffness, it is possible to underestimate seismic load.

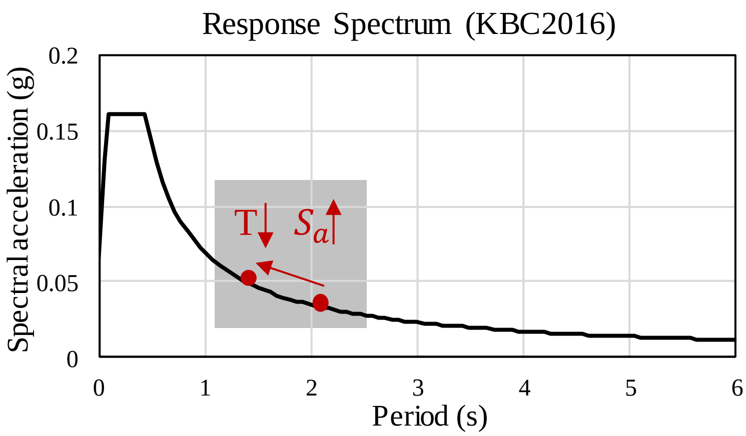


Figure 2-3 Relationship between natural period and seismic load

2.3.2 Saffarini et al. (1992)

In the study, the assumption of in-plane floor rigidity, commomly used in the analysis of reinforced concrete multistory buildings subjected to lateral loads, is examined by analytically investigating 37 buildings. These buildings are including parameters such as number of stories, story height, slab type and the size and spacing of columns and shear walls. The slabs of these buildings are modeled as rigid diaphram or plate elements and the analysis results are compared. The assumption that in-plane stiffness of slab is infinite is found to be excellent for framed buildings. For buildings containing shear walls as part of the lateral load-resisting system, a few error does resulting from the use of the assumption.

Chapter 3. Influence of Slab Stiffness on the Dynamic Behavior

3.1 Synopsis of analysis model

3.1.1 Overview

Based on the actual floor plan of residential building in Korea, elastic analysis model, OD20, is modeled as shown in **Figure 3-1**. It is a non-expandable flat-type that has been popular in the conuntry and the private area of one generation is 84 m^2. The number of the stories of the model is 20 and the story height is 3 m.

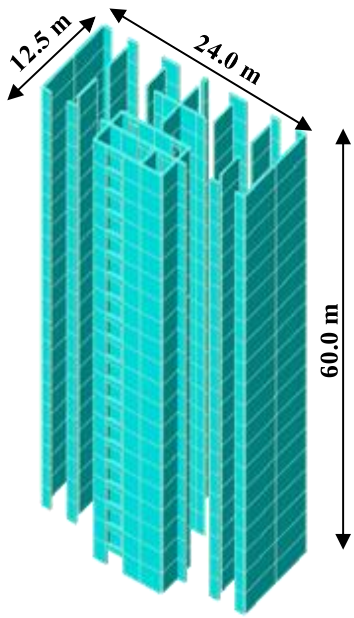


Figure 3-1 Three-dimentional elastic analysis model

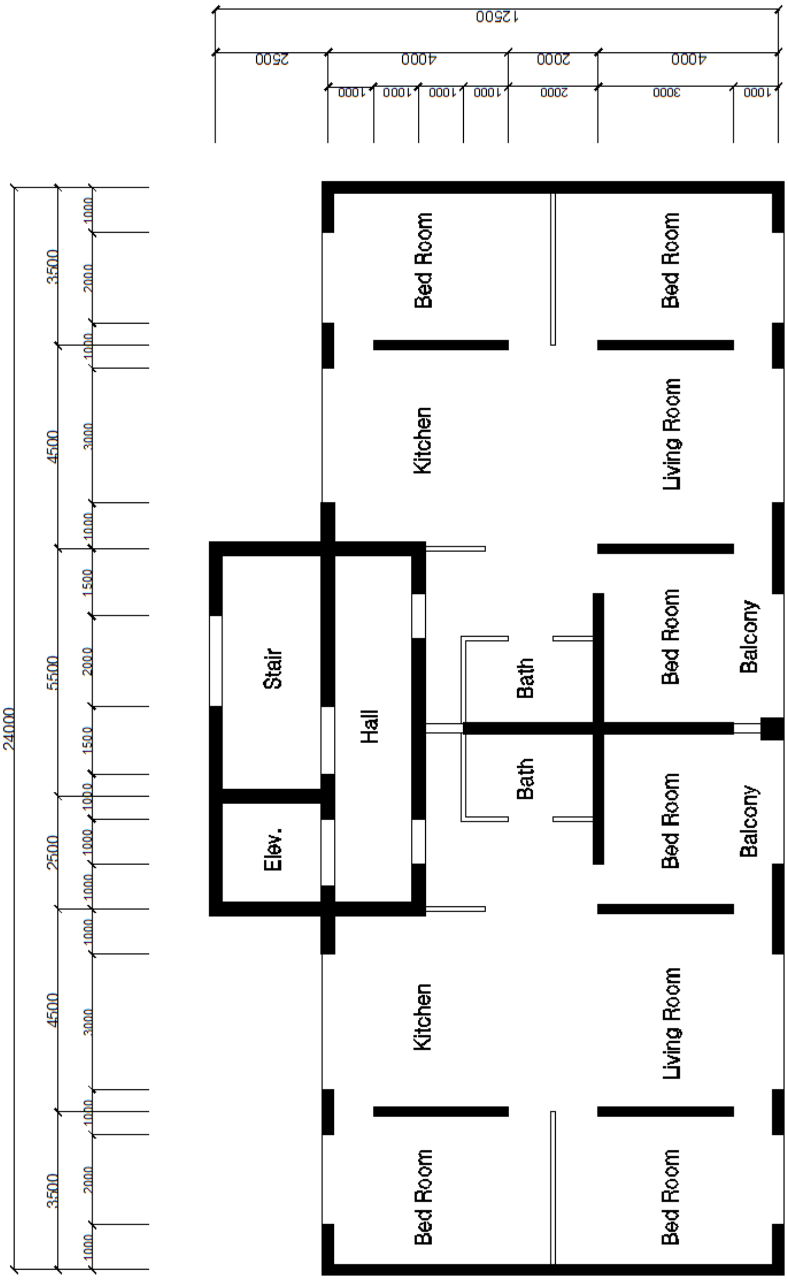


Figure 3-2 The architectural plan of models

With modeling of slab as a variable, the analysis models are divided into OD20, ODS20 and OS20 as shown in **Table 3-1**. Slab is assumed as rigid diaphragm in OD20 and that is modeled as plate element in OS20. In the ODS20, slab is modeled as plate with rigid diaphragm.

The design compressive strength of concrete is 24 MPa and the yield strength of reinforcement is 400 MPa for slabs and is 500 MPa for walls, beams and columns. Response modification coefficient(R) is 4 and important coefficient(I) is 1.2 since analysis models are ordinary reinforced concrete shear wall system of class 1. The basement is not modeled.

In this study, short side direction and long side direction are x-direction and y-direction, respectively.

The response spectrum analysis is performed using Midas ADS, and the finite element analysis of slab is conducted using Midas SDS. Best. Pro is also used to design members.

Table 3-1 Classification of analysis models

Model	OD20	ODS20	OS20
In-plane stiffness	∞	∞	○
Out-of-plane stiffness	x	○	○

3.1.2 Design loads

Table 3-2 and **Table 3-3** show the assumption for the gravity loads. Dead loads and live loads are considered and self-weight of the members is also taken into account as dead loads. In other to reflect the seismic loads significantly, site class is assumed as S_d and the other conditions are shown in **Table 3-4**. Response spectrum used for linear elastic analysis is also shown in **Figure 3-3**. The wind load is not taken into consideration to confirm the influence of the seismic load only on the analysis model.

Table 3-2 Assumption for the line loads (kN/m)

	list	D.L	
Masonary wall	floor tile & mortar (t=20)	0.40	
	brick	3.80 (1.0B), 1.90 (0.5B)	
	insulator (t=65)	0.07	
	plaster board (t=10)	0.10	
	tile (t=10)	0.20	
	Sum (1.0B)	**4.57**	**12.80 (considering story height)**
	Sum (0.5B)	**2.67**	**7.50 (considering story height)**

Table 3-3 Assumption for the area loads (kN/m^2)

	list	D.L	L.L	D+L	1.2D+1.6L
Roof floor (t=210)	mortar (t=80)	1.84	1.00		
	Insulator (t=120)	0.10			
	waterproofing	0.10			
	con'c slab (t=210)	5.04			
	CEILING	0.20			
	sum	**7.28**	**1.00**	**8.28**	**10.34**
ELEV. hall (t=150)	mortar (t=30)	0.60	3.00		
	con'c slab (t=150)	3.60			
	sum	**4.20**	**3.00**	**7.20**	**9.84**
Stair pace	floor finishing material (t=30)	0.60	3.00		
	con'c slab (t=150)	3.60			
	sum	**4.20**	**3.00**	**7.20**	**9.84**
Stair	floor finishing material (t=30)	0.60	3.00		
	con'c slab (t=210)	5.04			
	Sum 1/cos32	5.64 **6.2**	**3.00**	**9.20**	**12.24**
Living room Bed room Kitchen (t=210)	mortar & finishing material (t=45)	0.90	2.00		
	lightweight aerated con'c (t=40)	0.24			
	buffer material (t=35)	0.18			
	con'c slab (t=210)	5.04			
	CEILING	0.20			
	sum	**6.56**	**2.00**	**8.56**	**11.07**
Bath room (t=180)	floor tile & mortar (t=80)	1.60	2.00		
	con'c slab (t=180)	4.35			
	CEILING	0.20			
	sum	**6.15**	**2.00**	**8.15**	**10.58**
Balcony (t=210)	floor tile & mortar (t=120)	2.40	3.00		
	con'c slab (t=210)	5.04			
	sum	**7.44**	**3.00**	**10.44**	**13.73**

Table 3-4 Assumption for the seismic load

Seismic load parameters	Factor
Seismic zone	1
Zone factor (S)	0.22
Site class	S_d
F_a	1.46
F_v	1.58
Importance factor (I)	1.2
Response modification coefficient (R)	4
Seismic design category	D

※ $S_{ds} = 2.5 \times S \times F_a \times 2/3 = 0.5353g$

※ $S_{d1} = S \times F_v \times 2/3 = 0.2317g$

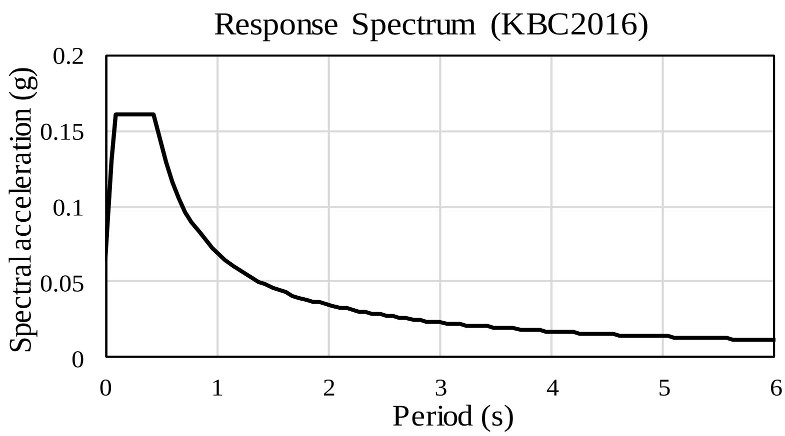


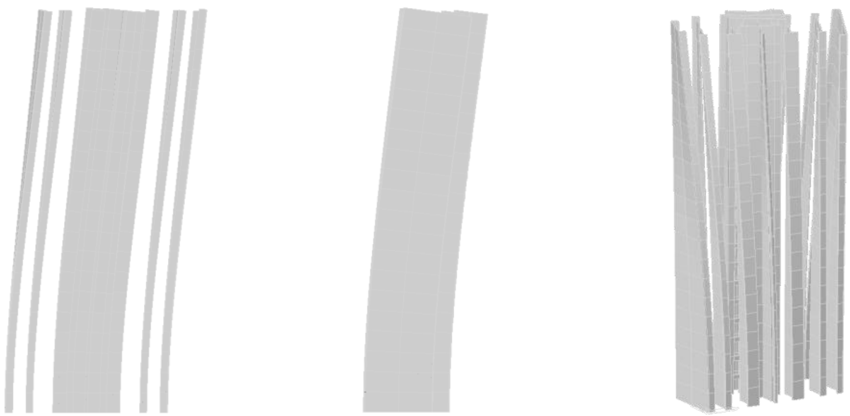
Figure 3-3 Response spectrum of DBE

3.2 Response spectrum analysis

3.2.1 Natural period

Modal analysis results are shown in **Figure 3-4** and **Figure 3-5**. Because of the small amount of walls in the x-direction, first mode of all models proceeds in that direction. And the second mode goes on in the y-direction with strong outer walls.

Since the story mass is constant regardless of the slab modeling metod, the natural period is different depending on the stiffness of slab. The results of eigenvalue analysis show that the period becomes shorter when the flexural stiffness of slab is considered. In addition, as the more flexural stiffness is taken into account, the natural period becomes more shorter. And the natural periods of ODS20, OS20 are almost the same, which means that the effect of in-plane stiffness of the slab is not large.



(a) mode 1 (b) mode 2 (c) mode 3

Figure 3-4 mode shapes of elastic models

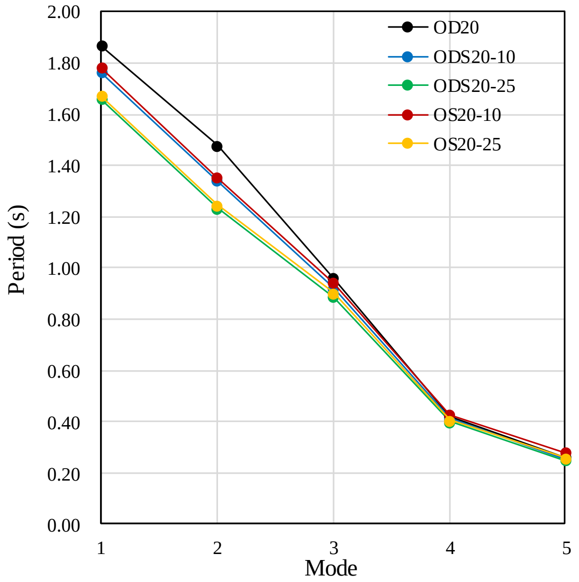


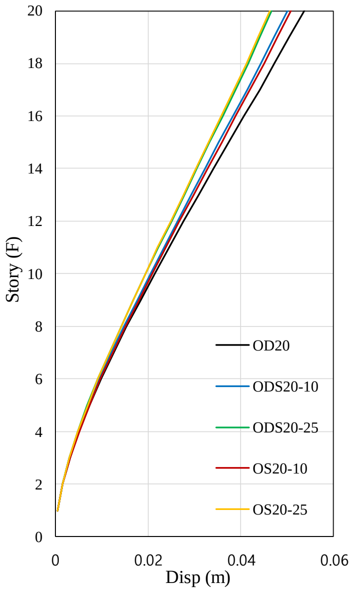
Figure 3-5 Natural periods of models

Table 3-5 Modal analysis results

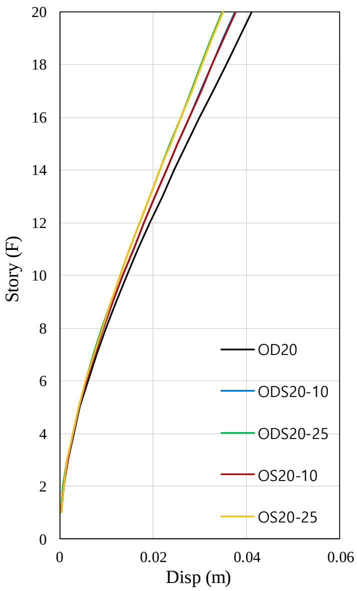
Mode	OD20			ODS20-10%			OS20-10%		
	T (s)	Mass X(%)	Mass Y(%)	T (s)	Mass X(%)	Mass Y(%)	T (s)	Mass X(%)	Mass Y(%)
1	1.87	**65.55**	0.02	1.76	**66.21**	0.01	1.78	**66.60**	0.01
2	1.48	0.04	**63.54**	1.34	0.02	**64.26**	1.35	0.03	**64.35**
3	0.96	1.09	0.08	0.92	0.99	0.12	0.94	1.06	0.11
4	0.42	**17.51**	0.01	0.41	**17.11**	0.01	0.43	**17.09**	0.01
5	0.26	0.02	**20.20**	0.26	0.02	**19.57**	0.28	0.02	**18.75**
6	0.19	0.84	0.05	0.18	0.64	0.05	0.20	**5.42**	0.01
7	0.18	**5.16**	0.02	0.18	**5.28**	0.01	0.19	0.11	0.05
8	0.11	3.10	0.09	0.10	3.07	0.08	0.14	0.01	**5.25**
9	0.10	0.02	**6.75**	0.10	0.02	**6.69**	0.13	2.69	0.01
10	0.08	0.03	0.02	0.08	0.03	0.02	0.12	0.06	0.01
Sum		93.36	90.78		93.39	90.82		93.09	88.56

3.2.2 Lateral displacement and interstory drift ratio

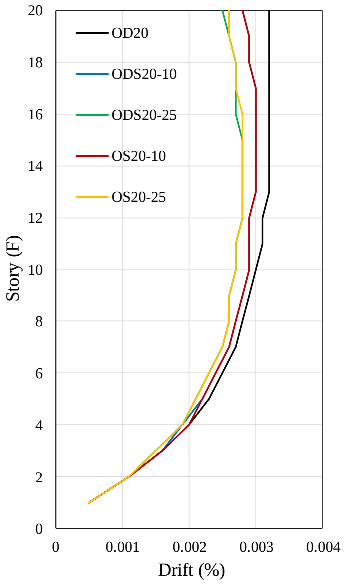
Fig. 3-6 shows the lateral displacement and interstory drift ratio of the models by the response spectrum analysis. In all cases, maximum lateral displacement goes down when the flexural stiffness of the slab is considered. It decresases by 7% and 9% in the x-direction and the y-direction respectively when the flexural stiffness of the slab is taken into account by 10%. The maximum displacement of OS20 is little larger than that of ODS20, but the difference is meaningless. Interstroy drift ratio is the same aspect as maximum lateral displacement.



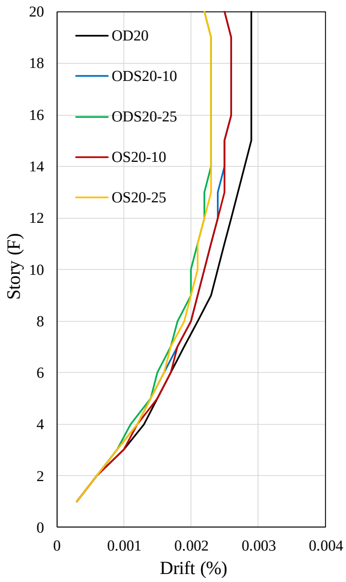
(a) story displacement (x-dir)



(b) story displacement (y-dir)



(c) story drift ratio (x-dir)



(d) story drift ratio (y-dir)

Figure 3-6 Story displacement and drift ratio of elastic models

3.2.3 Shear force distribution

Considering flexural stiffness of slab in the response spectrum analysis, the natural period of the building is shortened and this causes an increase in seismic load. As shown in **Figure 3-7**, base shear force increases about 5% and 10% when flexural stiffness of slab is considered as 10% and 25% respectively. It means that the more flexural stiffness is taking into account, the more seismic load is applied. So it can underestimate the risk of earthquakes when slab is modeled as rigid diaphram.

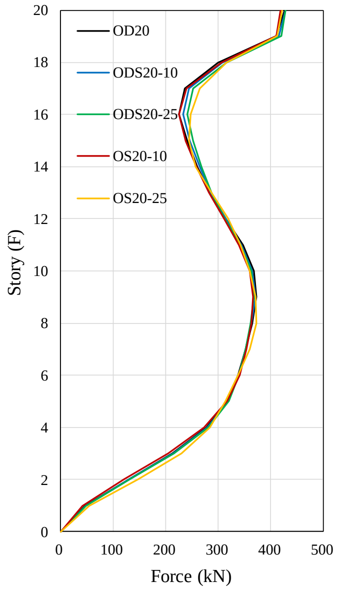
In fact, if base shear force(V_t) obtained using the response spectrum analysis is smaller than 85% of the base shear force(V_o) calculated by the equivalent static analysis, the correction coefficient(C_m) should be considered according to the KBC 2016 section 0306.7.3.5. In this study, base shear force increased by taking into account the flexural stiffness of slab is less than 85% of the base shear force by the equivalent static analysis. Because this factor increases the seismic load, seismic load underestimated due to rigid diaphragm can be somewhat compensated.

$$C_m = 0.85\frac{V_o}{V_t} \geq 1.0 \tag{3-1}$$

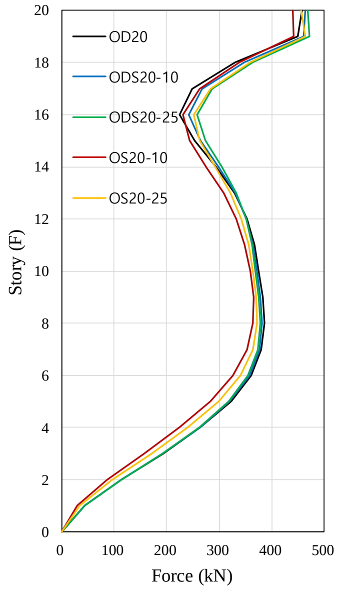
Table 3-6 Base shear force of models

Model	direction	V_t (kN)	C_m
OD20	x-dir	2740	1.278
	y-dir	3236	1.135
ODS20-10	x-dir	2790	1.255
	y-dir	3353	1.095
ODS20-25	x-dir	2857	1.225
	y-dir	3480	1.055
OS20-10	x-dir	2773	1.263
	y-dir	3240	1.134
OS20-25	x-dir	2851	1.228
	y-dir	3423	1.073

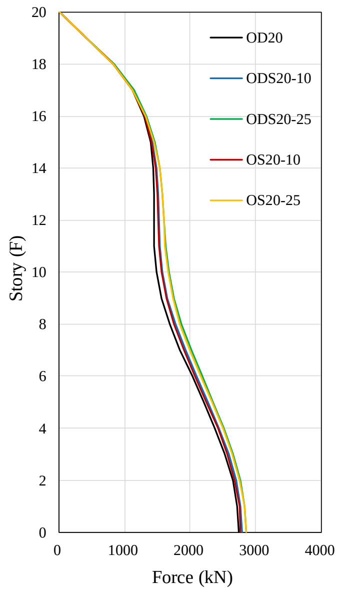
※ $0.85V_o =$ 3501 kN (x-dir), 3673 kN (y-dir)



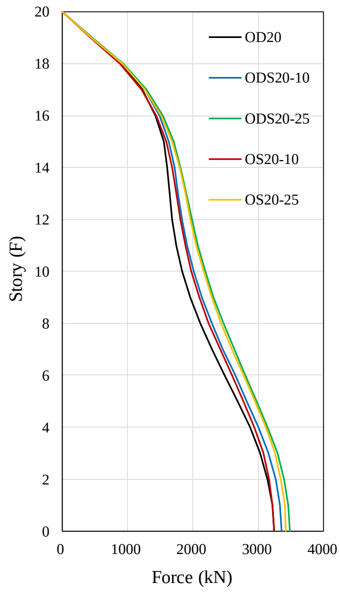
(a) lateral force distribution (x-dir)



(b) lateral force distribution (y-dir)



(c) story shear distribution (x-dir)

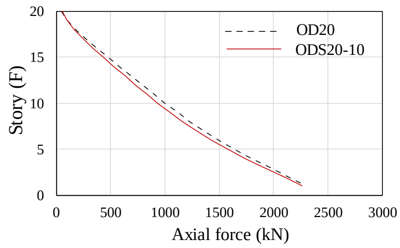


(d) story shear distribution (y-dir)

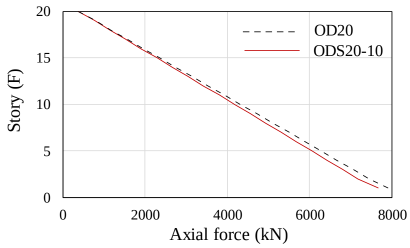
Figure 3-7 Force distribution of elastic models (SRSS)

3.2.4 Wall force and moment

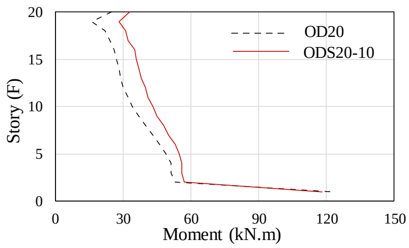
Slab redistributes lateral loads to the vertical members which resist the loads. As shown in **Figure 3-8**, difference between the loads acting on the large member and small member is decreased because of redistribted lateral loads. Also the gap between the loads acting on each story decreases a little.



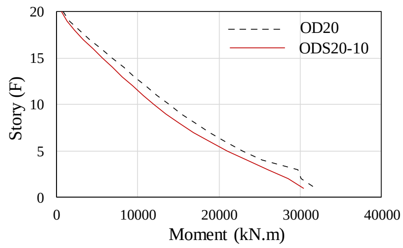
(a) HW5-axial force



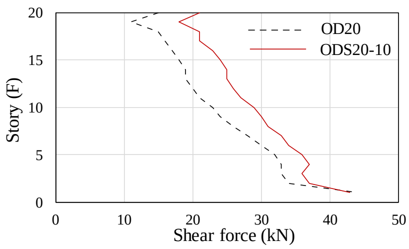
(b) W5-axial force



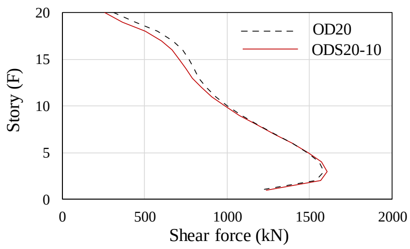
(c) HW5-moment



(d) W5-moment



(e) HW5-shear force

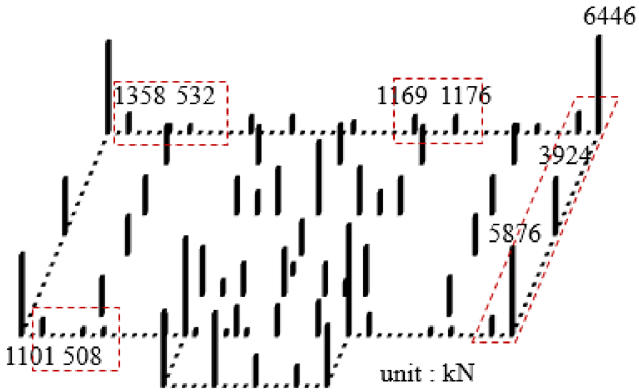


(f) W5-shear force

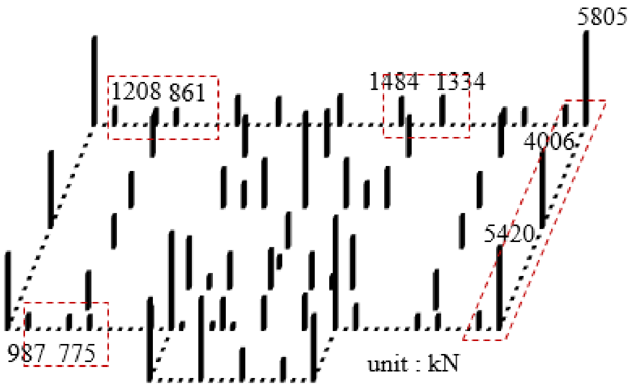
Figure 3-8 Wall forces and moments

3.2.5 Reaction

Reaction force is also redistributed as like member force because of slab effect. As shown in **Figure 3-9**, reaction force is concentrated on a specific point on OD20 that slab is modeled as rigid diaphram, but the force is relatively uniformed when the flexural stiffness is considered.



(a) OD20



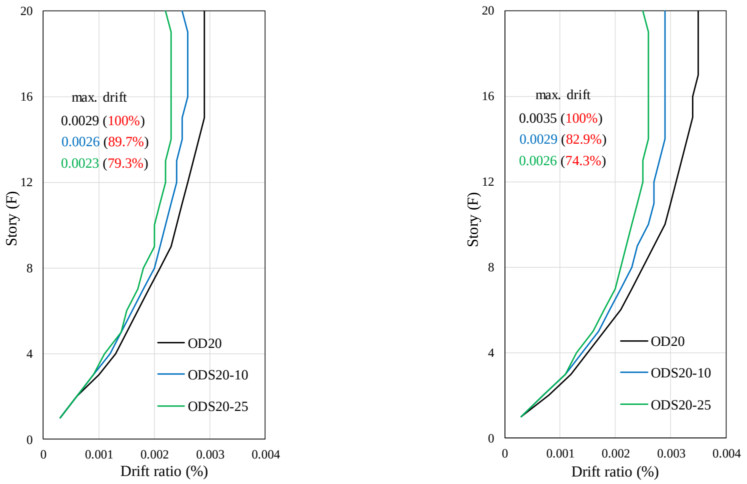
(b) ODS20-25%

Figure 3-9 Reaction force distribution chart

3.2.6 Application of effective stiffness

It is not easy to prevent cracks completely in reinforced concrete structures, so there is a need to consider degradation of stiffness appropriately in the structural analysis step. According to the KBC 2016 section 0503.4.6, the moment of inertia of 70% ($0.7I_g$) is used for vertical member like columns and walls to consider a cracking.

As a result of reflecting effective stiffness of walls, decrement of natural period and interstory drift ratio is increased when considering slab stiffness. When the slab flexural stiffness is 25%, the natural period of the 1st mode is reduced by 12% but it increases up to 14% taking into account effective stiffness of wall. And interstory drift ratio is decreased by 22% but it is changed by 27% when considering $0.7I_g$. This means that the influence of slab stiffness is increased if the cracking is taken into consideration.



(a) case of applying I (b) case of applying 0.7I

Figure 3-10 Difference of interstory drift ratio (y-direction)

3.3 Basic design

The slab, wall, beam and column are designed by applying the limit-states design based on the analysis results of OD20. All members should satisfy the strength limit state and serviceability limit state. To satisfy the former limit state, Design strength of all member at all sections shall satisfy **Eq.(3-2)** to **Eq.(3-4)**.

$$\phi P_n \geq P_u \tag{3-2}$$

$$\phi M_n \geq M_u \tag{3-3}$$

$$\phi V_n \geq V_u \tag{3-4}$$

KBC 2016 specifies that it should be designed for the worst condition of the required strength calculated by applying load factors and load combinations. Because wind load is not considered in this study, the following load combinations are considered in member design.

$$1.4\ \mathrm{D} \tag{3-5}$$

$$1.2\ \mathrm{D} + 1.6\ \mathrm{L} \tag{3-6}$$

$$1.2\ \mathrm{D} + 1.0\ \mathrm{L} \pm 1.0\ \mathrm{E} \tag{3-7}$$

$$0.9\ \mathrm{D} \pm 1.0\ \mathrm{E} \tag{3-8}$$

And Buckling has a great influence on vertical member design. When the column and wall are designed, they are assumed to be restrained and it is discriminated whether to consider the slenderness effect using the **Eq. (3-9)**. KBC 2016 section 0506.5.1 states that it is permissible to neglect slenderness effect if,

$$\frac{kl}{r} \leq 34 - 12(\frac{M_1}{M_2}) \tag{3-9}$$

where

M_1/M_2 : ratio of the moments at the two ends of the vertical member

The bending moment is calculated considering the moment amplification factor when the slenderness effect is considered. The moment amplication factor, δ , is determined by the following equation.

$$\delta = \frac{C_m}{1 - P_u/0.75P_c} > 1 \tag{3-10}$$

where

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \tag{3-11}$$

$$P_c = \frac{\pi^2 EI}{(kl)^2} \tag{3-12}$$

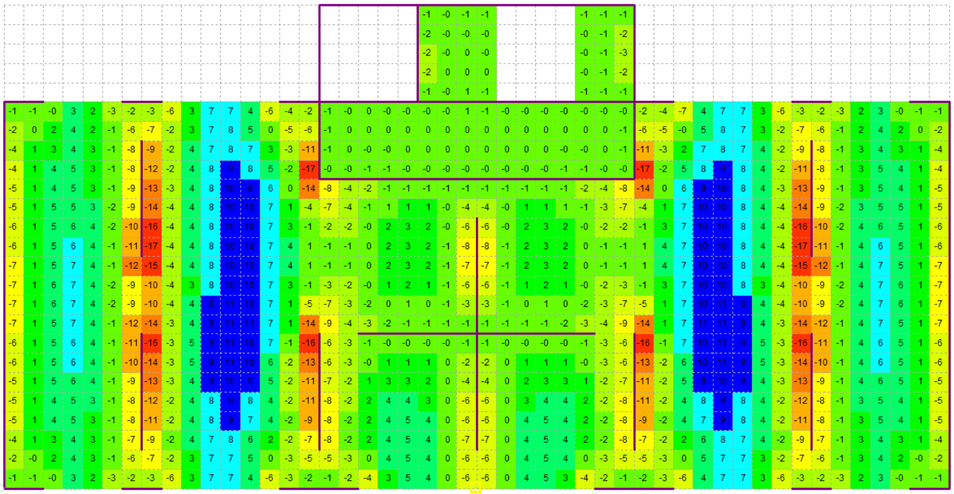
3.3.1 Slab design

Slab is desiged to resist gravity only, so the following two load combinations are considered.

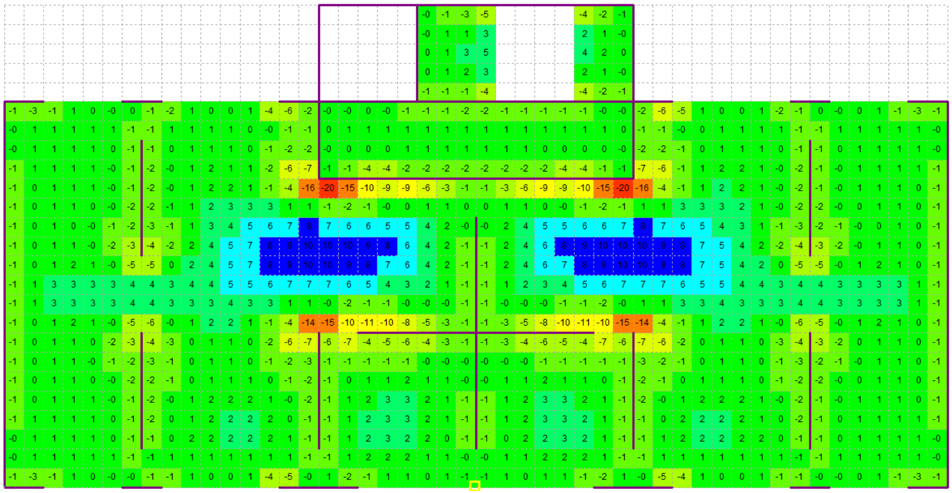
$$1.4\ D \tag{3-5}$$

$$1.2\ D + 1.6\ L \tag{3-6}$$

Floor slab is divided into some areas with similar flexural behavor. And then minimum reinforcement is designed in each direction considering drying shrinkage. After that, comfirming the ϕM_n of the slab where the minimum reinforcement is designed, additional flexural reinforcement is designed only the region where the bending moment exceeds the flexural strength of the slab. The maximum shear force acting on the slab do not exceed ϕV_c, so no additional shear reinforcement is designed. The cover thickness of a slab is a 20 mm generally and is a 40 mm on the surface in contact with the outside. **Figure 3-10** shows the FEA results of typical floor slab and reinforcement detail is shown in **Figure 3-11**.

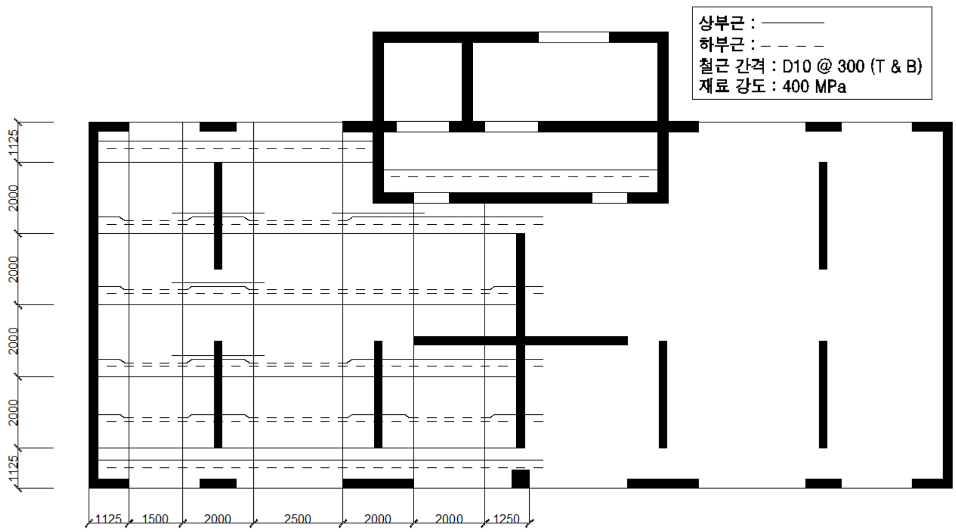


(a) Bending moment in the x-direction

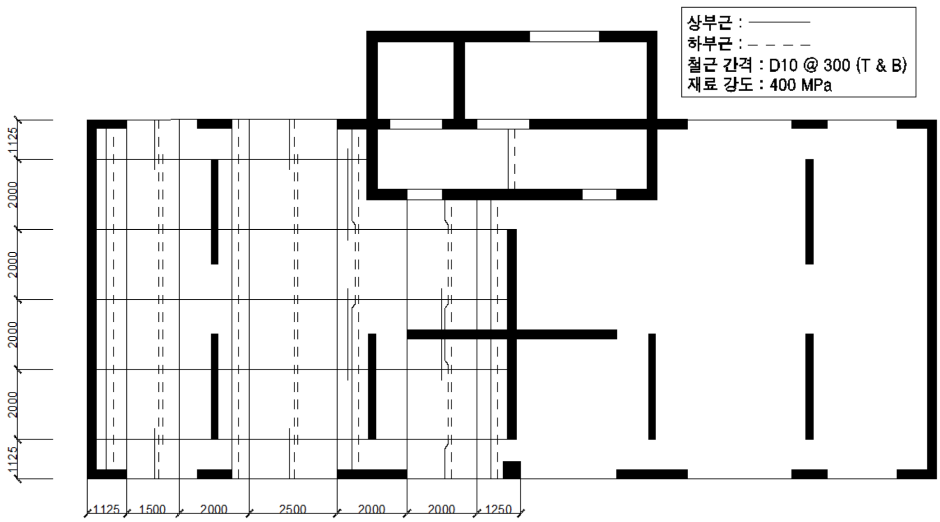


(b) Bending moment in the y-direction

Figure 3-11 FEA results of typical floor slab



(a) reinforcement detail in the x-direction



(b) reinforcement detail in the y-direction

Figure 3-12 Slab design of typical floor

3.3.2 Wall design

Walls are designed with the same details in two stories and designed considering the continuity of the vertical reinforcement between the upper and lower floors. The maximum spacing of vertical reinforcement of roof floor walls, outer walls and walls with a length of 1 m is designed to be 300mm. The cover thickness of wall is 20 mm generally and is 40 mm on the surface in contact with the outside.

As like column design, walls are designed for the axial force and bending moment using P-M curve. And for the shear design, walls are designed in accordance with KBC 2016 section 0507.10. The nominal shear strength of a wall shall be calculated by the **Eq. (3-13)** and

$$V_n = \phi[V_c + V_s] \tag{3-13}$$

the shear strength carried by the concrete is calculated by the next eqations.

$$V_c = mim[V_{c1}, V_{c2}] \tag{3-14}$$

where

$$V_{c1} = 0.28\lambda\sqrt{f_{ck}}hd + \frac{N_u d}{4L_w} \tag{3-15}$$

$$V_{c2} = [0.05\lambda\sqrt{f_{ck}} + \frac{L_w(0.1\lambda\sqrt{f_{ck}} + 0.2\,N_u/L_w h}{M_u/V_u - L_w/2}]hd \tag{3-16}$$

And if $V_u > 1/2\,\phi V_c$, minimum ρ_h and ρ_l shall be satisfied the following equations.

$$\rho_h \geq 0.0025 \tag{3-17}$$

$$\rho_l \geq 0.0025, 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_h - 0.0025) \tag{3-18}$$

where

ρ_h : ratio of area of transverse reinforcement to gross concrete area

ρ_l : ratio of area of longitudinal reinforcement to gross concrete area

Where the V_u exceeds ϕV_c, the shear strength, V_s, carried by the reinforcement should be calculated by the following equation.

$$V_s = \frac{A_{vh} f_y d}{s_h} \tag{3-19}$$

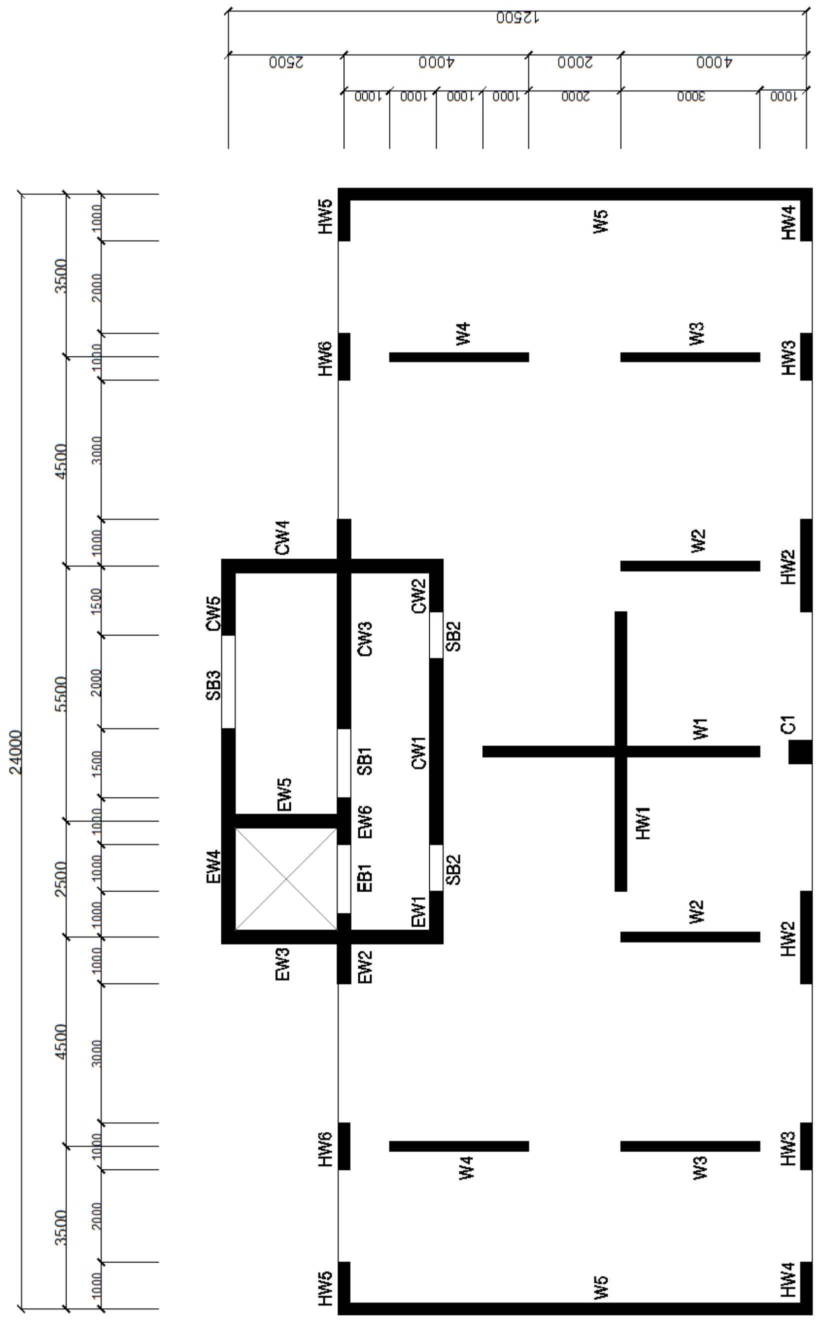


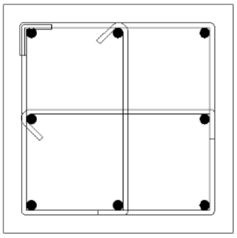
Figure 3-13 Structural plan of OD20

Table 3-7 Wall list of OD20

Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)	Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)
W1	20F	250	D10@300	D10@280	EW2	11 - 16F	300	D13@250	D10@230
	7 - 19F		D10@450	D10@280		7 - 10F		D13@150	D10@230
	1 - 6F		D10@200	D10@200		1 - 6F		D16@100	D10@190
W2,3,4	20F	200	D10@300	D10@350	EW3	17 - 20F	300	D10@300	D10@230
	1 - 19F		D10@450	D10@350		15 - 16F		D10@250	D10@230
W5	17 - 20F	250	D10@300	D10@280		13 - 14F		D10@200	D10@230
	13 - 16F		D10@300	D10@220		9 - 12F		D13@300	D10@230
	1 - 12F		D10@200	D10@220		7 - 8F		D13@250	D10@230
HW1	20F	250	D10@300	D10@280		3 - 6F		D16@250	D10@190
	7 - 19F		D10@450	D10@280		1 - 2F		D16@150	D10@190
	1 - 6F		D10@200	D10@220	EW4	7 - 20F	300	D10@300	D10@230
HW2	5 - 20F	250	D10@300	D10@280		3 - 6F		D10@300	D10@190
	3 - 4F		D10@300	D10@230		1 - 2F		D13@250	D10@190
	1 - 2F		D10@200	D10@220	EW5,6	1 - 20F	300	D10@300	D10@230
HW3	1 - 20F	250	D10@300	D10@280	CW1	5 - 20F	300	D13@300	D10@230
HW4	15 - 20F	250	D10@300	D10@190		3 - 4F		D13@250	D10@190
	11 - 14F		D13@300	D10@190		1 - 2F		D13@150	D10@190
	7 - 10F		D16@300	D10@190	CW2	11 - 20F	300	D13@150	D10@190
	5 - 6F		D16@200	D10@190		1 - 10F		D13@100	D10@190
	3 - 4F		D16@150	D10@190	CW3	17 - 20F	300	D10@300	D10@230
	1 - 2F		D16@100	D10@190		11 - 16F		D10@200	D10@230
HW5	15 - 20F	250	D10@300	D10@190		5 - 10F		D10@150	D10@190
	11 - 14F		D13@300	D10@190		3 - 4F		D10@200	D10@190
	7 - 10F		D16@300	D10@190		1 - 2F		D10@150	D10@190
	5 - 6F		D16@200	D10@190	CW4	15 - 20F	300	D10@300	D10@230
	3 - 4F		D16@150	D10@190		11 - 14F		D13@200	D10@230
	1 - 2F		D16@100	D10@190		7 - 10F		D13@150	D10@190
HW6	1 - 20F	250	D10@300	D10@280		1 - 6F		D13@100	D10@190
EW1	17 - 20F	300	D13@300	D10@230	CW5	15 - 20F	300	D10@300	D10@230
	11 - 16F		D13@250	D10@230		11 - 14F		D13@200	D10@230
	1 - 10F		D13@100	D10@190		7 - 10F		D13@200	D10@190
EW2	17-20F	300	D13@300	D10@230		1 - 6F		D13@100	D10@190

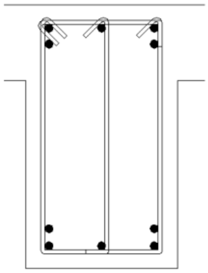
3.3.3 Beam and column design

According to the KBC 2016, beams and columns are designed as like **Figure 3-13**.



< ALL SECTION>

NAME : C1
SIZE : 500 x 500
RE-BAR : 8 – D22
STR : D10@300



< ALL SECTION>

NAME : EB1, SB1~3
SIZE : 300 x 700
TOP : 5 – D19
BOT : 5 – D19
STR : D10@125(3-LEG)

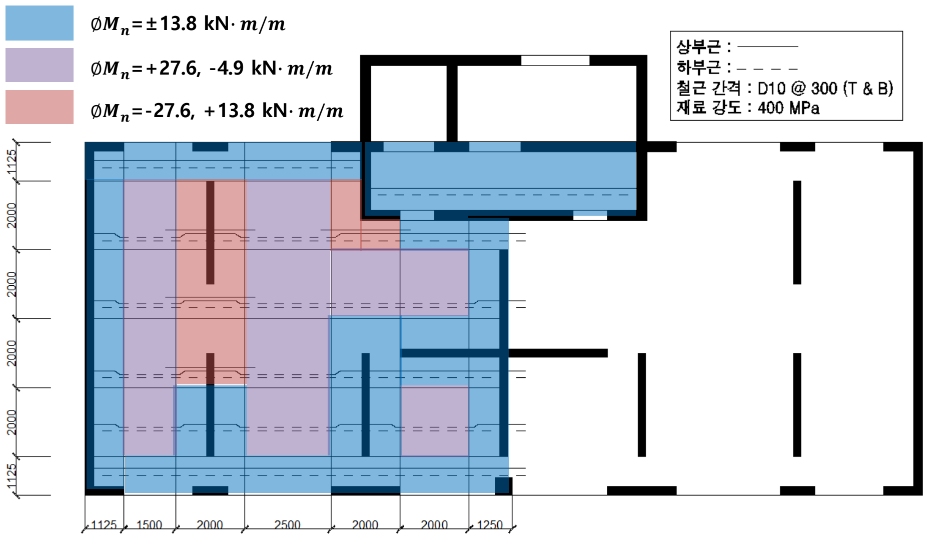
Figure 3-14 Beam and column design section

3.4 Finite element analysis of slab

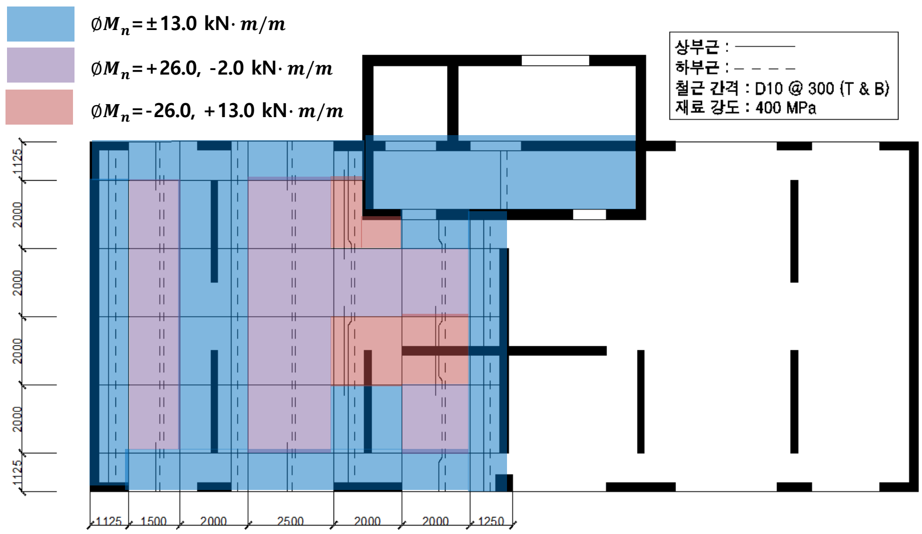
3.4.1 Overview

As shown in the previous study, if considering flexural stiffness of slab, the load acting on the wall decreases and this makes wall reinforcement reduced. On the contrary, reinforcement of slab can be increased to resist more lateral loads and slab design should vary from floor to floor since the lateral loads acting on each floor slab is different. So it is necessary to find the maximum flexural stiffness of slab without changing design in order not to undergo these disadvantages.

Verification procedure proceeds as follows. First check the design bending strength of slab for each zone as like **Figure 3-14**. Then, compare the maximum bending moment acting on each slab with bending strength by considering the flexural stiffness of slab as a variable. All load combinations are used to calculate the maximum bending moment. Shear force also verified in the same way as bending moment.



(a) design bending strength in the x-direction



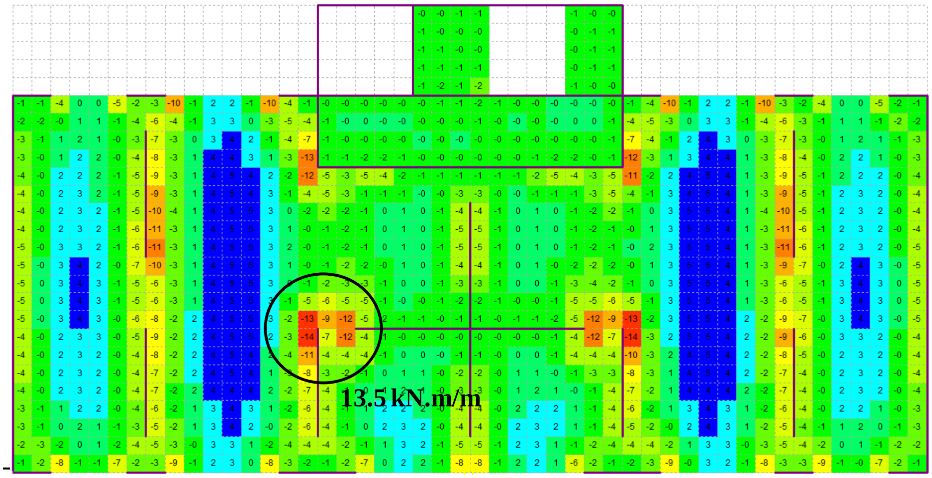
(b) design bending strength in the y-direction

Figure 3-15 Design bending strength of typical floor slab

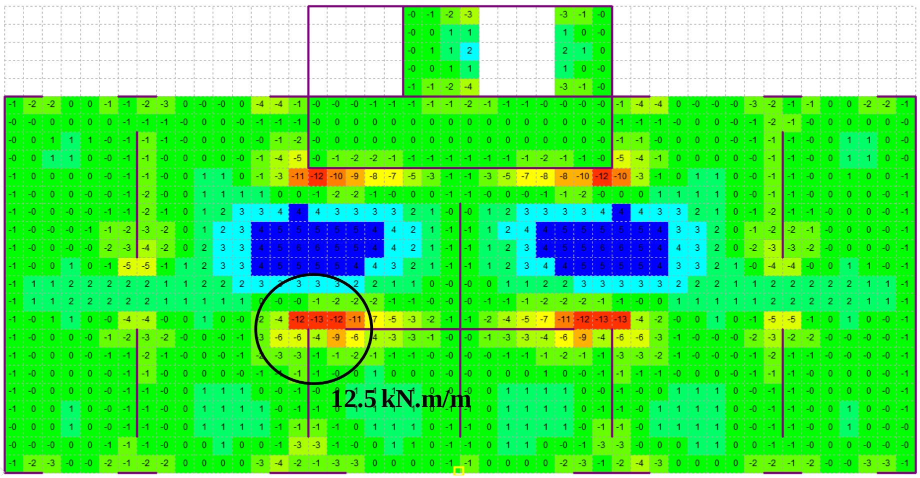
3.4.2 Examination for bending moment

Design bending strength of slab which resists gravity loads only is shown in **Figure 3-14**. Mesh size of finite element is 0.5 x 0.5 m and bending moment and shear force are considered in unit width. Also the flexural stiffness of slab is examined at 5% intervals.

As a result, seismic load increases as going up to higher floors in the same load combination. As shown in **Figure 3-15**, when the flexural stiffness of slab is 10%, the maximum bending moment at the 18th floor is 13.5 kN·m/m, which is smaller than design strength of slab. However, if the flexural stiffness is taken into condieration by 15%, it is exceeded. So, the maximum flexural stiffness of slab that can be reflected without any design change of slab is 10%. But this can be changed as the following conditions : the magnitude of seismic load, story height of buildings and shape of architectural plan.

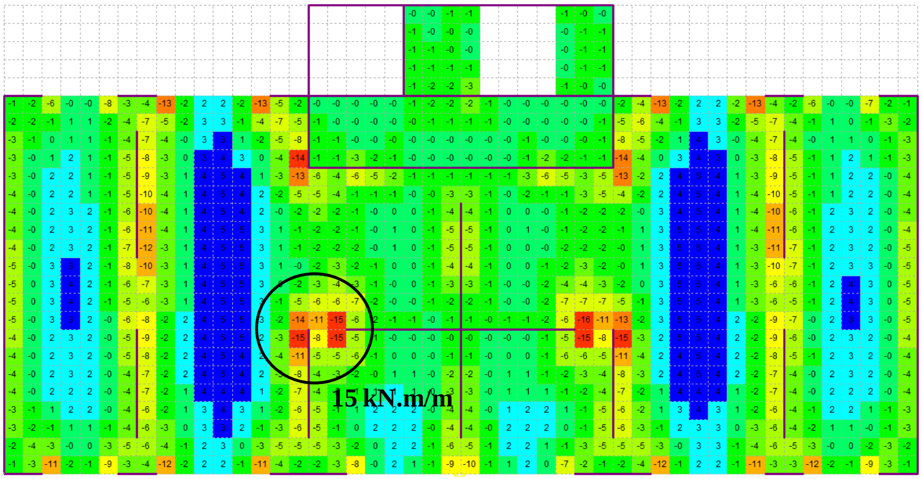


(a) Bending moment in the x-direction (18F, 0.9D-1.255(RS+ES)X +1.0L)

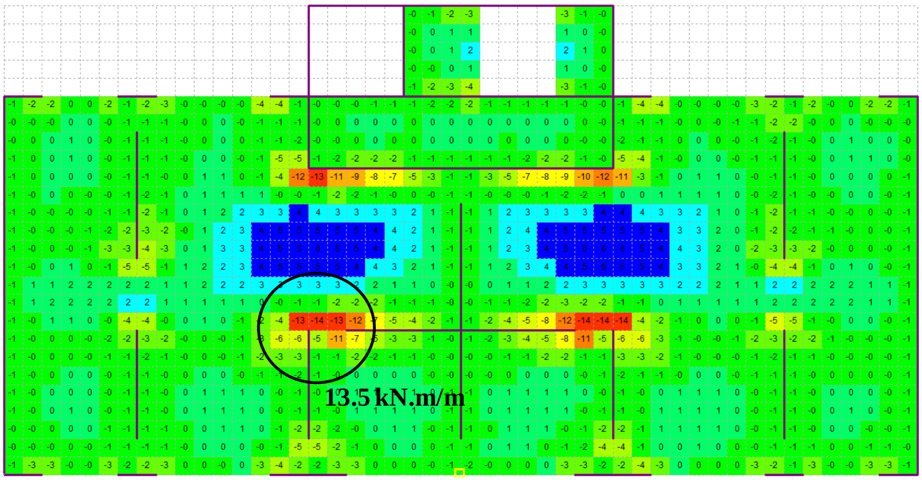


(b) Bending moment in the y-direction (18F, 0.9D-1.255(RS+ES)Y +1.0L)

Figure 3-16 FEA results of slab (10% flexural stiffness)



(a) Bending moment in the x-direction (18F, 0.9D-1.245(RS+ES)X +1.0L)



(b) Bending moment in the y-direction (18F, 0.9D-1.245(RS+ES)Y +1.0L)

Figure 3-17 FEA results of slab (15% flexural stiffness)

3.5 Discussion

In this chapter, response spectrum analysis for 20-story shear wall buildings and finite element analysis for slab are performed to investigate the effect of slab stiffness on the dynamic behavior. The results are summarized as follows:

1) The natural period of building becomes shorter if considering flexural stiffness of slab. This becomes more outstanding as the stiffness is more taken into account. When the slab is modeled as rigid diaphragm with no flexural stiffness in practice, seismic load may be underestimated. However, the underestimation can be compensated somewhat because of Cm factor which is used to design a building

2) There is no big difference between the behavior of ODS20 and OS20. This means that in-plane stiffness of slab is relatively bigger than flexural stiffness of vertical members even in a shear wall system. So it is more advantageous that slab is modeled as plate with rigid diaphragm rather than plate only because of decreasing degree of freedom.

3) Since slab redistributes the seismic load, lateral displacement is decreased and and reaction force distribution is relatively uniformed. Also the load acting on large walls decreases and this can be possible to reduce the amount of reinforcement in walls slightly.

4) Maximum flexural stiffness of slab that can be considered without changing slab design is about 10%. This value can be different if seismic load, story height and shape of architectural plan of building is modified.

Chapter 4. Nonlinear Analysis considering Flexural Stiffness of Slab

4.1 Nonlinear modeling

4.1.1 Material model

Sturctural models should incorporate realistic estimates of stiffness and strength considering the anticipated level of excitation and damage. In the nonlinear analysis, expected material properties shall be utilized throughout as opposed to nominal or specified properties. In lieu of detailed justification, values provided **Table 4-1** is used to determine the expected strength by the Guidelines for Performance-Based Seismic Design of Residential Buildings (2015). And elastic modulus of concrete is determined by the following equations from KBC 2016.

$$E = 8{,}500\sqrt[3]{f_{cu}} \tag{4-1}$$

$$f_{cu} = f_{ck} + \Delta f \tag{4-2}$$

where f_{ck} is the specified compressive strength of concrete; if f_{ck} is less than 40 MPa, Δf is 4 MPa, and if f_{ck} exceeds 60 MPa, Δf is 6 MPa. Values between 40 MPa and 60 MPa shall be determined by linear interpolation.

Table 4-1 Expected strength factor

Material properties	Nominal strength	Expected strength factor
Concrete compressive strength	$f_{ck} \leq 21\ MPa$	1.2
	$21\ MPa\ \leq\ f_{ck} \leq 40\ MPa$	1.1
	$f_{ck} > 40\ MPa$	1.0
Yield and tensile strength of reinforcement	$400\ MPa\ \leq\ f_y < 500\ MPa$	1.1
	$500\ MPa\ \leq\ f_y < 600\ MPa$	1.05
	$f_y \geq 600\ MPa$	1.0

In the nonlinear analysis, which is using fiber elements, stress-strain curves of material are required because stiffness of members is determined by the stress-strain curve of fiber element. In this study, simplified concrete stress-strain curve is used. As shown in **Figure 4-1**, the initial elastic modulus of concrete is assumed to remain until 85% of the maximum expected stress. The maximum compressive stress of unconfined concrete is assumed when strain is 0.002, and 10% of the maximum stress is assumed to be a residual stress at the strain of 0.004. The tensile stress of concrete is ignored.

Guidelines for the Performance-Based Seismic Design of Residential Building(2015) suggests that elastic modulus of reinforcement is 200,000 MPa and the maximum tensile strain shall be 0.1. As shown in **Figure 4-2**, the material model for reinforcement is assumed to be trilinear model. Bucking is also considered since the cover of wall is thin in residential building

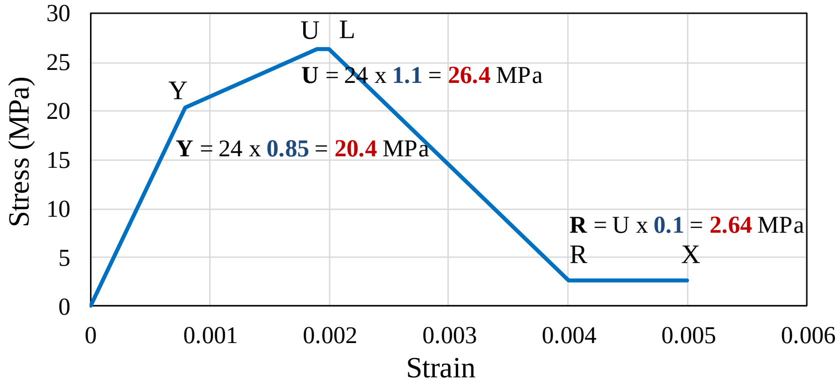


Figure 4-1 Simplified stress-strain curve of concrete

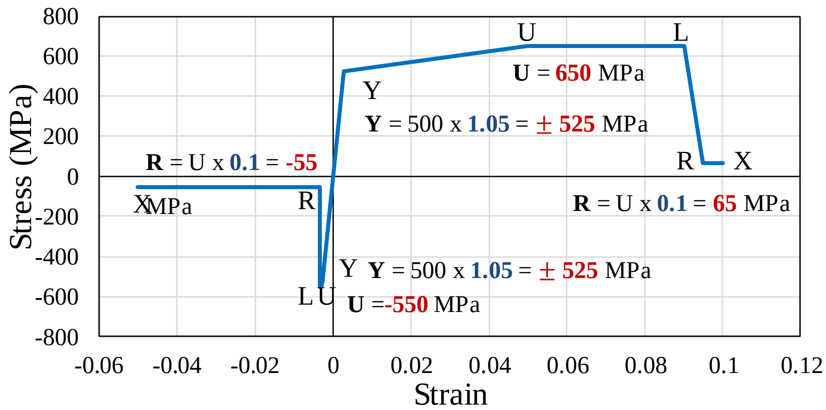


Figure 4-2 Simplified stress-strain curve of reinforcement

4.1.2 Slab

According to the AIK 2015, slab shall be modeled as a rigid diaphram in the nonlinear modeling of residential buildings. In this study, however, elastic shell element and rigid diaphram are used at the same time for slab modeling. And 10% of flexural stiffness of slab is used considering cracking and inelastic behavior based on the analysis result of Chapter 3. The slab is determined along the wall lines and the range of mesh size is from 0.5 x 0.5 m to 1.5 x 1.5 m as shown in **Figure 4-3**.

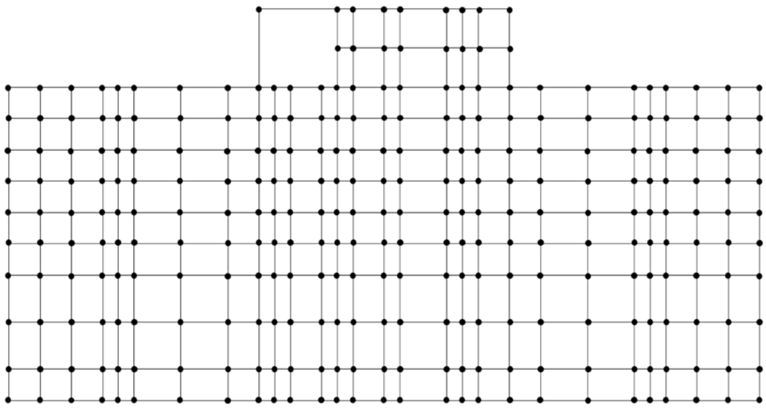
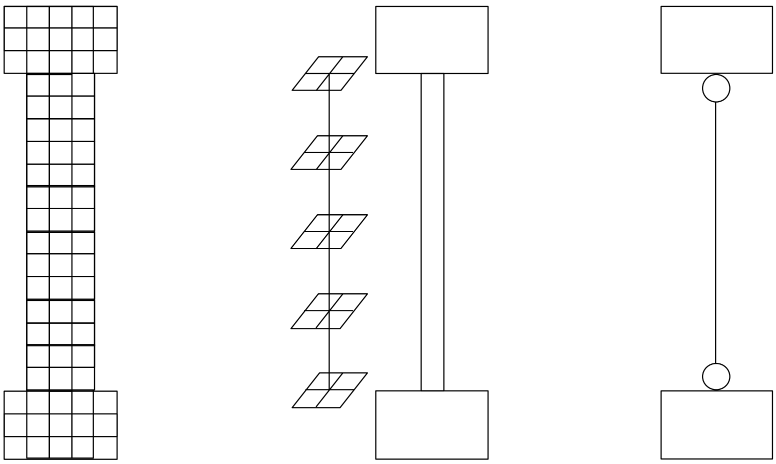


Figure 4-3 slab of nonlinear analysis model

4.1.3 Wall

As shown in **Figure 4-4**, there are three types of inelastic models; 1) continuum model, 2) distributed inelasticity model and 3) concentrated hinge model. First one describes the inelastic behaviors based on the physics of materials. But it is not appropriate for modeling of a whole structure due to the large amount of computational efforts. Second one called as fiber model is usually used for modeling of inelastic behavior of walls. In a fiber model, the cross-section geometry is prescribed and concrete and reinforcement are individually defined. Also its effective stiffness is calculated based on the stress-strain relationship of materials in the fiber element. In this study, fiber model is used for nonlinear modeling of flexural behavior in walls. Because this can consider only the in-plane nonlinear behavior, the effective stiffness of $0.1EI_g$ is used for the out-of-plane flexural stiffness of walls.



(a) continuum model (b) distributed inelasticity (c) concentrated hinge

Figure 4-4 Types of inelastic models

The shear behavior of walls is assumed to be elastic and uncracked shear stiffness(effective shear stiffness) is typically taken as the following equation according to the PEER/ATC 72-1 section 4.2.1.4.

$$G_c A = \frac{E_c}{2(1+v)} A = 0.4 E_c \mathrm{A} \tag{4-3}$$

where

v : Poisson's ratio

A : cross sectional area of the web

E_c: modulus of elasticity for concrete

G_c: shear modulus of elasticity for concrete

4.1.4 Coupling beam

Coupling beam is modeled as concentrated hinge model and hinge properties is shown in **Figure 4-5**. As like wall modeling, shear behavior of coupling beam is assumed to be elastic under the assumption that stirrup is a lot sufficiently.

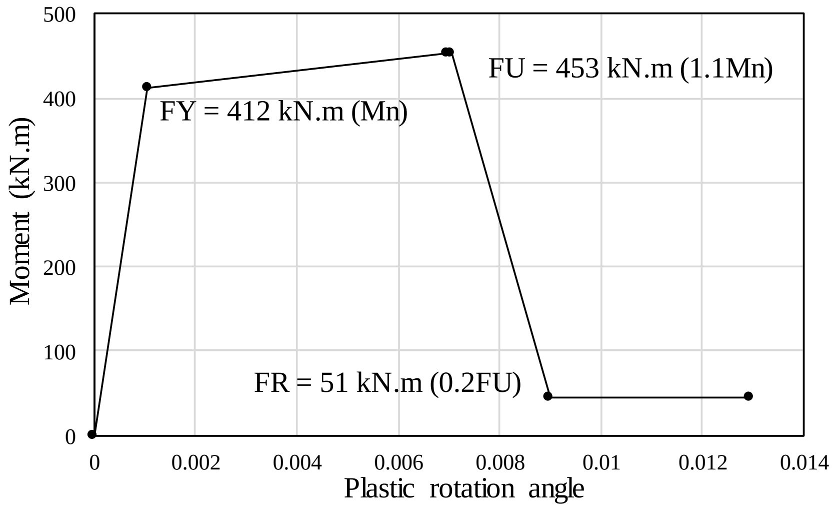


Figure 4-5 Force-deformation relationship of beam

4.1.5 Damping

Responses of a nonlinear time history analysis are sensitive to damping which is generally assosicated with reduction in dynamic vibration due to energy dissipation in structural and nonstructural components of the building, so it is important to select of damping ratio. In linear elastic analysis, damping values is typically 5% in the primary vibration modes. Unlike the linear elastic analysis, where the elastic stiffness and percentage of critical modal damping remain constant, in the nonlinear analysis the stiffness matrix softens due to inelastic effects, and the relative significance of damping can change dramatically during the analysis. Because inelastic effect produces hysterectic damping in the nonlinear analysis, the damping ratio should be less than that of linear analysis. According to the ASCE 41-13, the target damping ratio shall not exceed 3% for nonlinear dynamic analysis.

Also PEER/ATC 72-1 suggests the following values of equivalent viscous damping as appropriate for use in nonlinear time history analysis.

$$D = \alpha / 30 \text{ (for } N < 30) \tag{4-4}$$

$$D = \alpha / N \text{ (for } N > 30) \tag{4-5}$$

where

D : the maximum critical damping (%)

N : the number of stories

α : the coefficient for structural system $(60 \leq \alpha \leq 120)$

α = 60 for steel structures and =120 for reinforced concrete structures

From the equations, the equivalent viscous damping ratio is calculated as 4% but the modal damping of 0.5% and Rayleigh damping of 2.5% are used conservatively in this study. If using only the Rayleigh damping, damping ratio of higher order modes and since the damping is not reflected in the higher order modes when using modal damping only, two types of damping are used so that they are mutually complementary. Rayleigh damping value is calculated using the following equations and the result is shown in **Figure 4-6**.

$$\xi_n = \frac{\alpha_M T_n}{4\pi} + \frac{\alpha_K \pi}{T_n} \tag{4-6}$$

$$\alpha_M = 4\pi\xi \frac{1}{(T_i + T_j)}, and\ \alpha_K = \frac{\xi}{\pi} \frac{T_i T_j}{(T_i + T_j)} \tag{4-7}$$

where

ξ_n : the critical damping for the n^{th} vibration mode with the period T_n

α_M and α_K : the proportionality for mass and stiffness

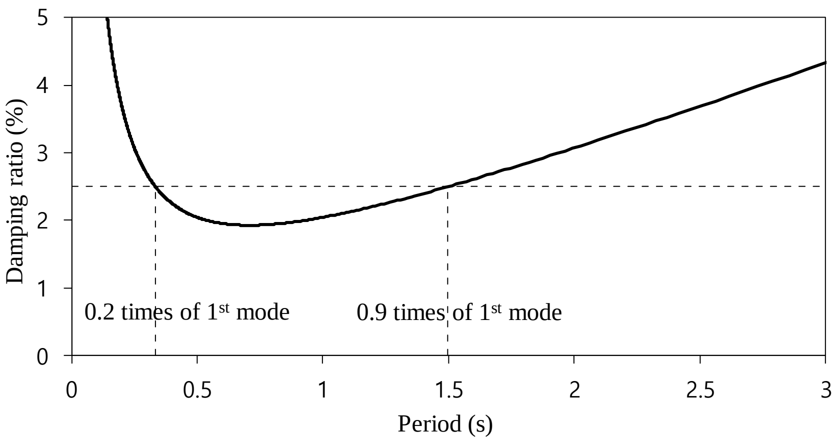


Figure 4-6 Rayleigh damping

4.2 Nonlinear Static analysis

4.2.1 Load set

Before performing the pushover analysis, a nonlinear static analysis for gravity load is conducted for the initial condition of pushover analysis. The load combination of expected gravity loads from the Guidelines for Performance-Based Seismic Design of Residential Buildings (2015) is used. The expected gravity load combination is determined by the following equation.

$$\text{Expected gravity load} = 1.0\,(W_D + W_S) + 0.25\,W_L \qquad (4\text{-}8)$$

Two types of lateral force distributions are used for the analysis in x- and y-directions. 1) Mode superposition force distribution considering the sum of over the 90% mass participation factor and 2) uniformly lateral force distribution. In this paper, only the results of the former are shown. The reference drift is the roof drift relative to the base and the maximum allowable drift is 5%. The P-delta effect is also considered.

4.2.2 Mode properties

For the intial conditions, the natural period of nonlinear model is shorter than that of elastic model due to the expected strength and reinforcement modeling. Considering the flexural stiffness of slab, the stiffness of the whole building increases and the natural period becomes shorter as shown in **Table 4-2**.

In the case of mass participation factor, it increases in the 1st and 2nd mode and decreases in the 4th and 5th mode when the flexural stiffness of slab is considered. But the difference is not large.

Table 4-2 Mode properties for the nonlinear models

Mode	Rigid diaphram model (nonlinear)			Slab model (nonlinear)		
	T (s)	Mass X (%)	Mass Y (%)	T (s)	Mass X (%)	Mass Y (%)
1	1.67	63.96	0.55	1.54	65.23	1.06
2	1.46	0.61	62.94	1.27	1.27	64.48
3	0.81	0.09	0.01	0.77	1.01	0.01
4	0.36	19.14	0.03	0.35	18.51	0.03
5	0.26	0.01	20.36	0.25	0.01	19.50
6	0.18	0.53	0.04	0.19	0.54	0.01

4.2.3 Overstrength factor

As shown in **Figure 4-7** and **Figure 4-8** , overstrength of the residential building increases 1.5 to 2 times when slab is modeled. In OD20 models, overstrength is decreasing after flexural failure of coupling beam in the x-direction, but it is continuously increasing in ODS20-10 because slab takes the role of coupling beams after flexural failure of them. Slab remains elastic since the maximum bending moment acting on the core slab is smaller than design strength of slab at the pick point. Also walls are connected to the slab to exhibit the coupling effect which significantly increases overstrength of the building in the y-direction. Force-deformation curves are similar in the case of mode superposition shape distribution and uniformly shape.

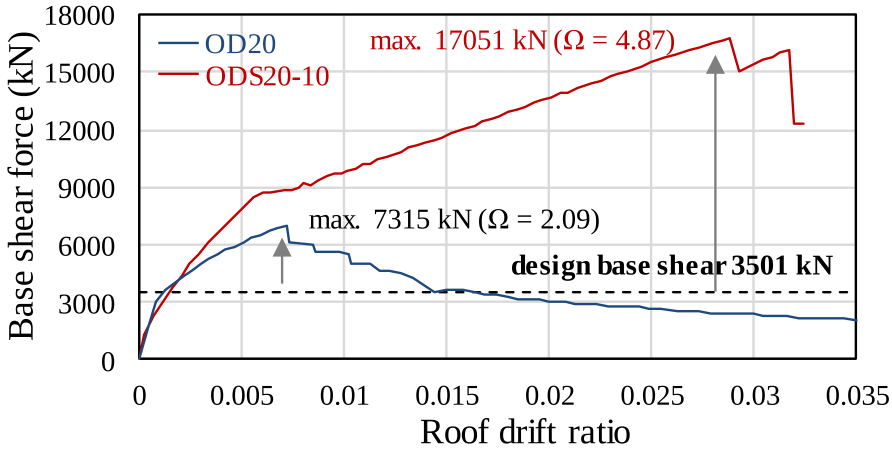


Figure 4-7 Overstrength factor (x-direction)

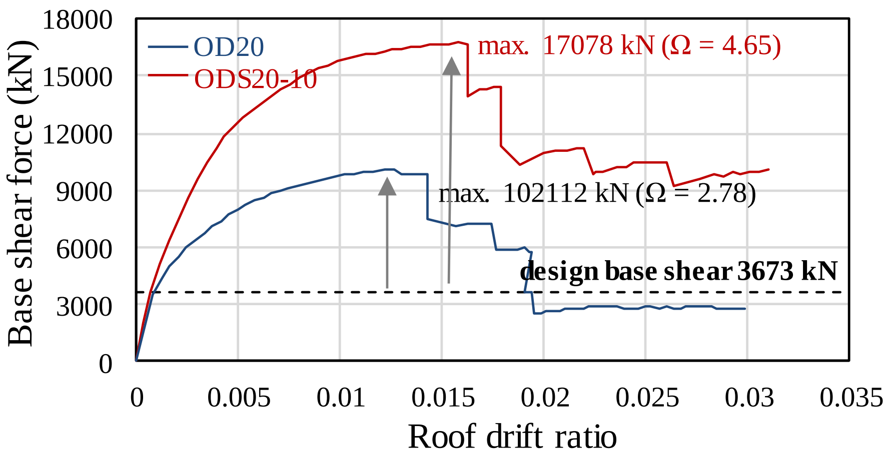


Figure 4-8 Overstrength factor (y-direction)

4.2.4 Perfermance point

The seismic performance of building is evaluated at the performance point, which is the intersection of the capacity curve and demand curve. The point satisfying LS level is formed at a relatively stable position after cracking occurred and stiffness of the entire building is reduced. In the x-direction, the performance point is formed at similar position because the initial stiffness of buildings between two models is almost sames. On the other hand, the point of ODS20-10 occurred at relatively early displacement compared to that of OD20 due to the increase of initial stiffness in the y-direction.

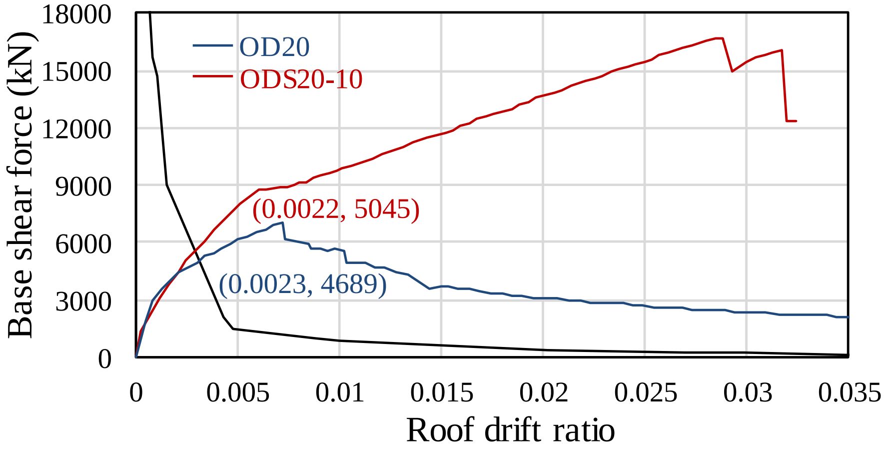


Figure 4-9 Performance point (x-direction)

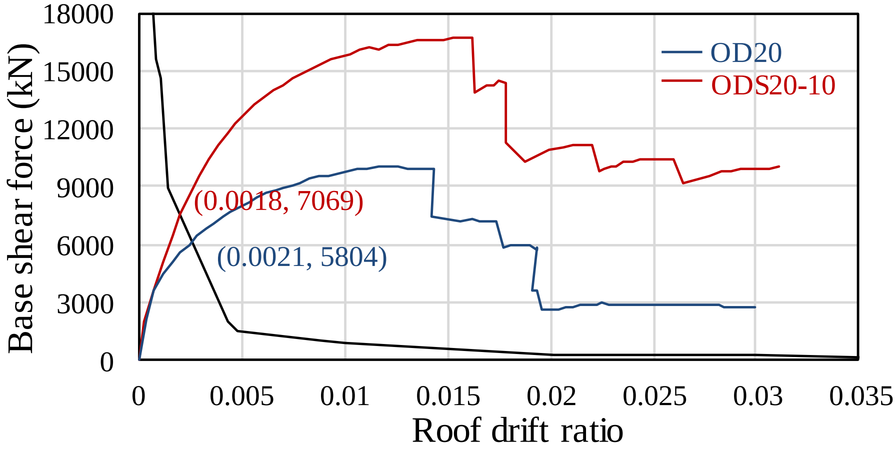
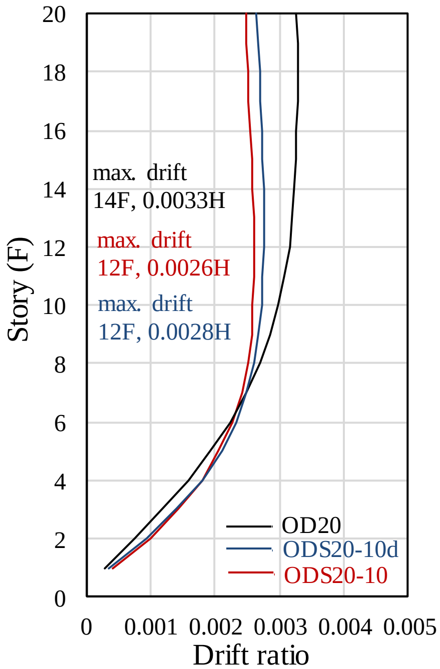


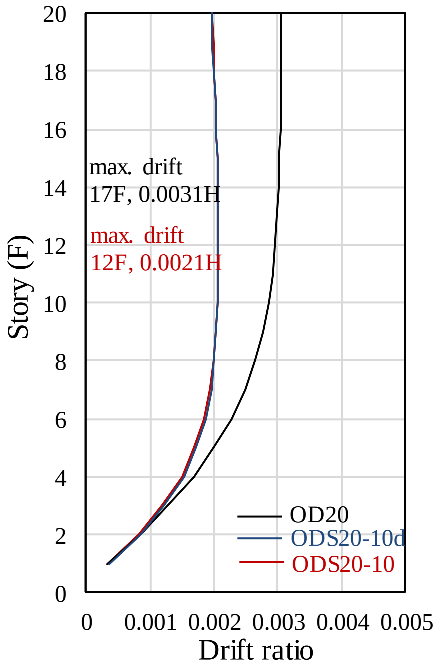
Figure 4-10 Performance point (y-direction)

4.2.5 Interstory drift ratio

To satisfy the seismic performance for Life Safety, story drift ratio shall not exceed 0.015H according to the Guidelines for Performance-Based Seismic Design of Residential Buildings. As shown in **Figure 4-11**, the maximum story drift ratio is 0.0033H which satisfies the target performance. When the flexural stiffness of slab is considered, the drift is decreased by 15~30% in high stories as like the results of elastic analysis.



(a) uniform shape - x direction



(a) uniform shape - y direction

Figure 4-11 Story drift ratio at performance point

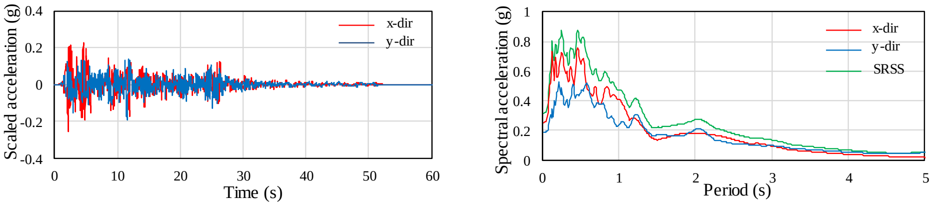
4.3 Nonlinear time history analysis

4.3.1 Ground motion records of DBE

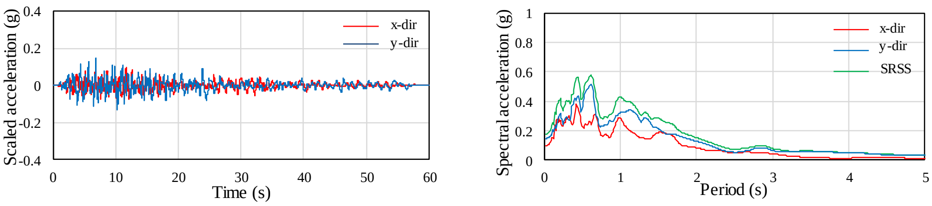
Responses of a nonlinear time history analysis are sensitive to a characteristic of ground motion records, so KBC 2016 requires using not less than three ground motion records for time history analysis. The maximum response shall be used for three ground motion records, and the average response can be permitted if seven or more ground motion records are used. In selecting the ground motion records, there are many deficiencies in the number and size of earthquakes measured in Korea, so it is possible to select foreign measurement wave and apply it. According to the Guidelines for Performance-Based Seismic Design of Residential Building, the following conditions are taken into account : Magnitude, Vs30 and Far-Field Record set. In this study, seven ground motion records satisfying magnitude 5 to 7 and Vs30 of 180 m/s to 360 m/s were selected at the PEER Ground Motion Database site. These are shown in **Table 4-3.**

Table 4-3 Selected ground motion records

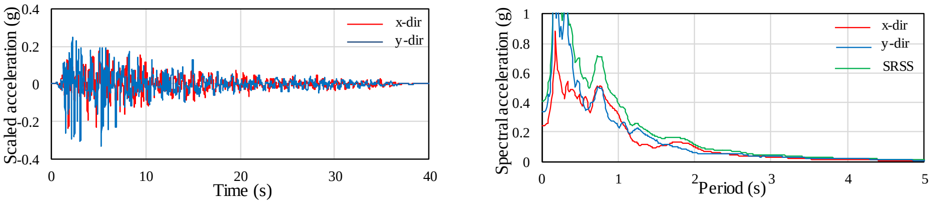
ID	EQ Name	Station	Year	M	Rrup (km)	Vs30 (m/s)
EQ1	Imperial Valley-02	El Centro Array #9	1940	6.95	6.1	213.4
EQ2	El Alamo	El Centro Array #9	1956	6.80	121.7	213.4
EQ3	Friuli_Italy-01	Conegliano	1976	6.50	80.4	352.1
EQ4	Northridge-01	San Bernardino	1994	6.69	103.2	336.9
EQ5	Kobe_Japan	Tadoka	1995	6.90	31.7	312.0
EQ6	Chi-Chi_Taiwan-03	CHY065	1999	6.20	105.5	250.0
EQ7	Chuetsu-oki_Japan	FKS023	2007	6.80	104.6	182.3



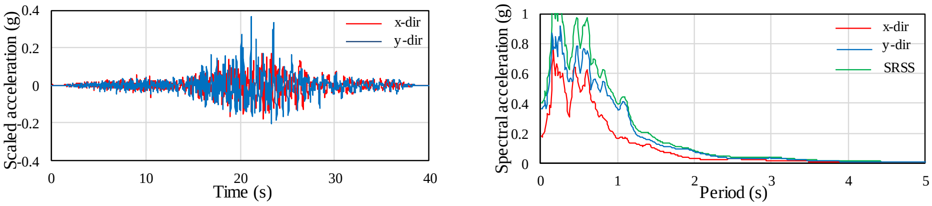
(a) DBE EQ1 Imperial Valley-02



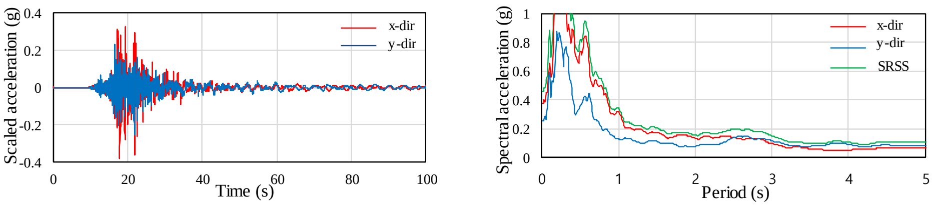
(b) DBE EQ2 El Alamo



(c) DBE EQ3 Friuli_Italy-01

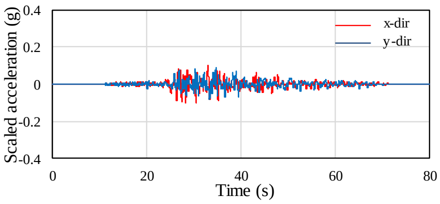


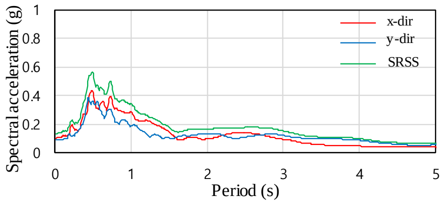
(d) DBE EQ4 Northridge-01



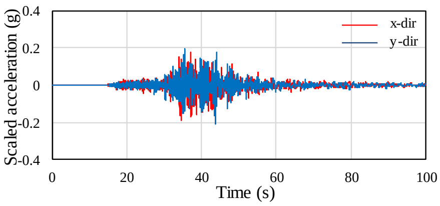
(e) DBE EQ5 Kobe_Japan

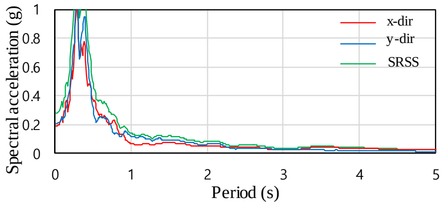
Figure 4-12 Ground motion records (EQ1-5)





(d) DBE EQ4 Northridge-01





(e) DBE EQ5 Kobe_Japan

Figure 4-13 Ground motion records (EQ6-7)

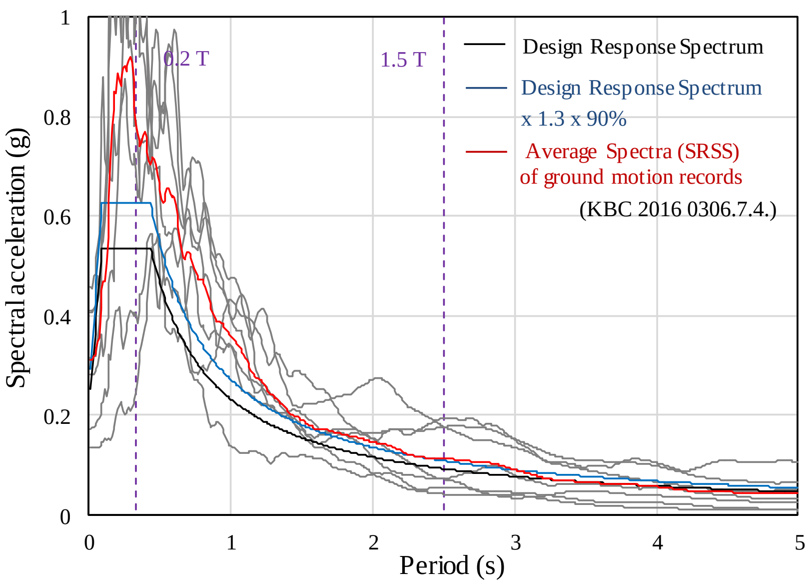
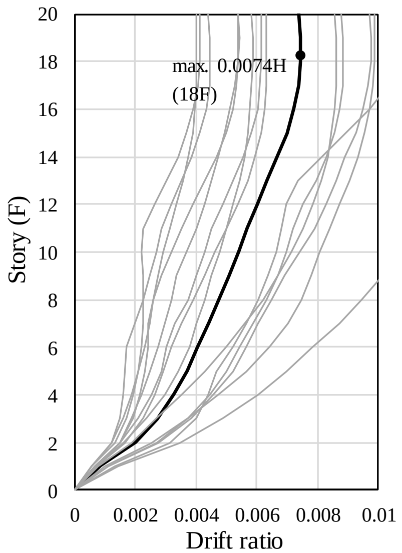


Figure 4-14 Average spectra of ground motion records

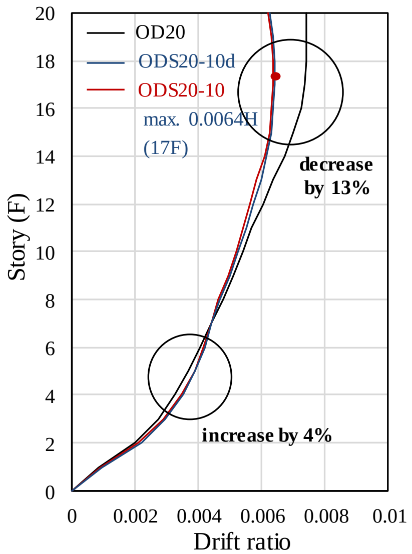
4.3.2 Interstory drift ratio

According to the AIK 2015, the interstory drift ratio of the building is calculated as the mean value of maximum response for each ground motion when seven ground motion records are used. And the maximum interstory drift ratio of the building shall not be exceed 1.5%(0.015H) to satisfying the Life Safety.

As a result, both OD20 and ODS20-10 models meets the LS criteria for the maximum interstory drift ratio. Maximum interstory drift ratio is reduced by about 13% considering slab stiffness. And as the ratios in the lower floors increases about 4%, the difference of the ratio between the upper and lower floors decreases.



(a) story drift ratio of OD20

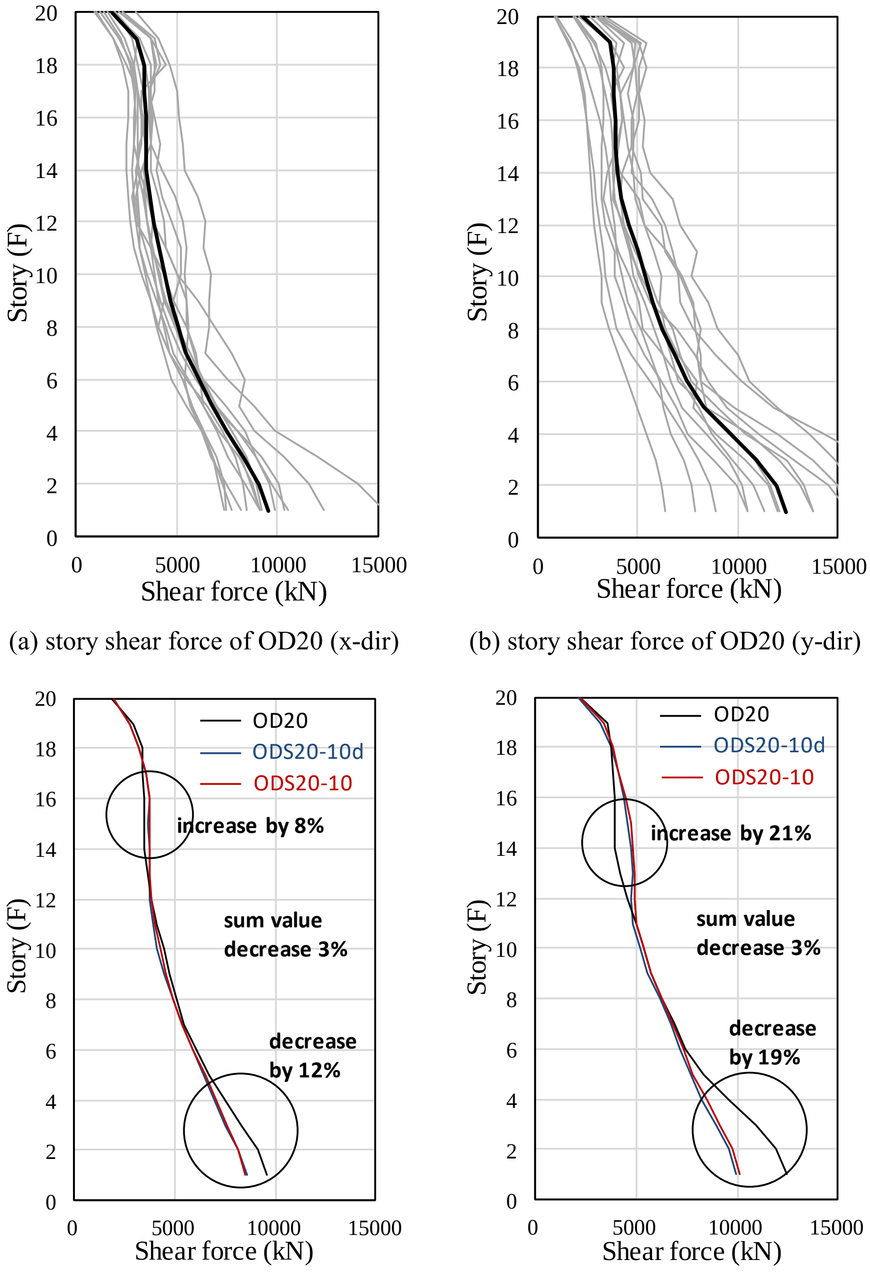


(b) comparison of story drift ratio

Figure 4-15 Story drift ratio of nonlinear models (SRSS)

4.3.3 Story shear force distribution

Story shear force distributions are similar to the interstory drift ratio. As shown in **Figure 4-16**, the story shear force of upper floors increases somewhat and that of lower floors decreases since seismic loads is redistributed by the slab. This makes the difference of story shear forces between the upper and lower floor diminished. And difference of shear force distributions in the x-direction and y-direction is reduced when considering slab stiffness. Also the sum of the story shear force acting on the building is reduced by 3% in all directions unlike elastic analysis.



(a) story shear force of OD20 (x-dir)

(b) story shear force of OD20 (y-dir)

(c) comparison of shear force (x-dir)

(d) comparison of shear force (y-dir)

Figure 4-16 Story shear force of nonlinear models

4.3.4 Wall rotation

Seismic performance of members is evaluated based on the deformation in PBSD. By confirming the plastic rotaion anlge, it is possible to check whether walls satisfy the performance level. The roatation angles are calculated as the average value of 14 response of ground motion records. All walls of models are met the objectives for LS. As shown in **Figure 4-17**, wall rotations of upper and lower floors increase by 5~10% and these of middle floors decrease by 15~25% due to redistribution of seismic loads when slab is modeled.

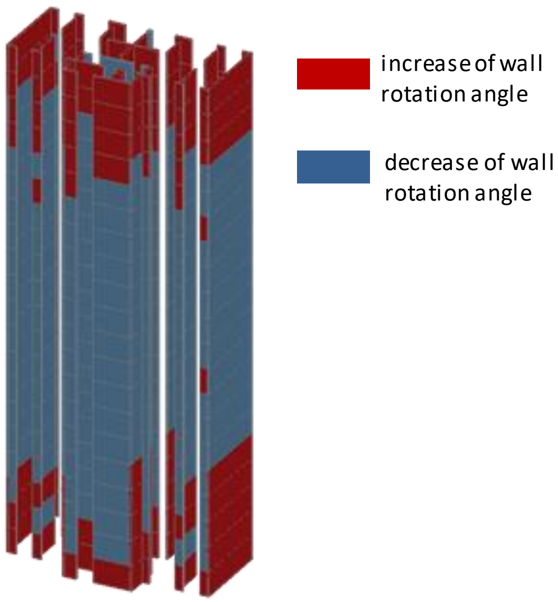
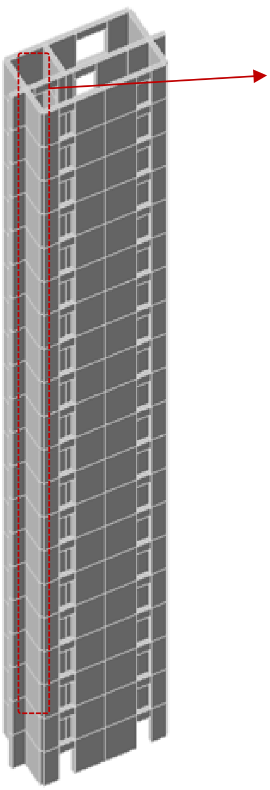


Figure 4-17 Walls with changed plastic rotation angles

4.3.5 Beam rotation

As like walls, beam roation angle is also evaluated to confirm the seismic performance using the average value of response of ground motions. As shown in **Figure 4-18**, all beams of OD20 and ODS20-10 are satisfied the objectives for LS and the plastic rotation angles for all beams of ODS20-10 are reduced by 50 to 70% compared to those of OD20. This is because core slab around beams resists to the seismic loads with beams.



unit : rad

	OD20	ODS20-10	for LS	Difference
EB1-1F	0.00033	0.00019	0.006	57%
EB1-2F	0.00033	0.00016	0.006	49%
EB1-3F	0.00034	0.00017	0.006	51%
EB1-4F	0.00035	0.00018	0.006	53%
EB1-5F	0.00034	0.00018	0.006	53%
EB1-6F	0.00035	0.00018	0.006	52%
EB1-7F	0.00033	0.00017	0.006	52%
EB1-8F	0.00031	0.00016	0.006	52%
EB1-9F	0.00030	0.00016	0.006	51%
EB1-10F	0.00030	0.00014	0.006	48%
EB1-11F	0.00030	0.00014	0.006	47%
EB1-12F	0.00030	0.00014	0.006	45%
EB1-13F	0.00031	0.00014	0.006	45%
EB1-14F	0.00033	0.00014	0.006	44%
EB1-15F	0.00034	0.00014	0.006	42%
EB1-16F	0.00035	0.00014	0.006	41%
EB1-17F	0.00034	0.00014	0.006	41%
EB1-18F	0.00033	0.00013	0.006	40%
EB1-19F	0.00032	0.00012	0.006	38%
EB1-20F	0.00022	0.00009	0.006	38%

Figure 4-18 Difference in beam rotation angle

4.4 Economy evaluation

As noted in the preceding chapter, seismic loads acting on the core wall, long-outer walls decreases slightly when flexural stiffness of slab is considered. If taking into account the maximum stiffness without changing slab design, 10% of flexural stiffness, the amount of reinforcement in walls can be reduced as much as possible.

Wall list of OD20 and ODS20-10d are shown in Appendix and **Table 4-4** represents the reducing of vertical reinforcement of walls in ODS20-10d. As a results, the amount of reinforcement decreases by about 6.7% from 46.0 ton to 42.9 ton only considering vertical and horizontal reinforcement of walls.

Table 4-4 Decrease of vertical reinforcement in walls

Name	Story	OD20	ODS20-10d
W1	1F – 6F	D10@200	D10@450
W5	1F – 12F	D10@200	D10@300
EW3	15F – 16F	D10@250	D10@300
	13F – 14F	D10@200	D10@300
	9F – 12F	D13@300	D10@300
CW1	3F – 4F	D10@250	D10@300
	1F – 2F	D10@150	D10@200
CW2	1F – 10F	D13@100	D13@150
CW3	11F – 16F	D10@200	D10@300
	5F – 10F	D10@150	D10@200
CW4	1F – 6F	D13@100	D13@150
CW5	7F – 14F	D13@200	D13@300
	1F – 6F	D13@100	D13@150

4.5 Discussion

In this chapter, Seismic performance evaluation and dynamic behavior of the model(OD20) that slab is assumed as rigid diaphram and the model(ODS20-10) considering 10% flexural stiffness of slab with rigid diaphram are compared through nonlinear static analysis and nonlinear time history analysis. The seismic peroformance and dynamic behavior of ODS20-10d which is designed considering 10% flexural stiffness of slab is almost same as ODS20-10.

1) Overstrength of ODS20-10 is 1.5 to 2 times higher than that of OD20 in the x-direction since core slab plays a role of coupling beam continuously after the beams is flexural failure. In the y-direction, the initial stiffness of building and overstrength are also increased significantly because of coupled action.

2) As like response spectrum analysis result, the maximum inter story drift ratio is decreased by more than 10% when flexural stiffness of slab is considered. This is very adavantageous in controlling the lateral displancement at the analysis steps.

3) Difference of seismic load between upper and lower floors is reduced because slab redistributes the lateral force. Also difference in seismic load in x-direction and y-direction is diminished.

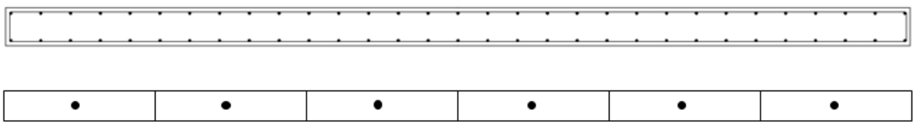
4) As considering 10% of flexural stiffness of slab which is not changed slab design, it can be possible to reduce the amount of reinforcement in walls by about 6.7%.

Chapter 5. Nonlinear Analysis considering Wall Reinforcement Details

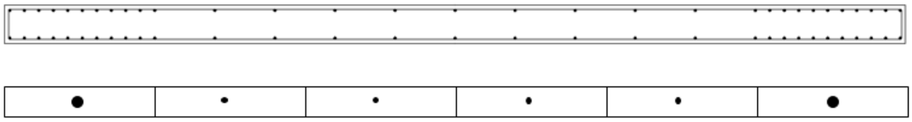
5.1 Design Concept

5.1.1 Overview

Generally vertical reinforcement of wall is equally spaced in practice. If it placed a lot at the ends of the wall with the same amount of reinforcement as shown in **Figure 5-1**, the bending strength of wall would increase and this makes the earthquake resistance more advantageous. But its details can not be applied to the wall which has minimum spacing or maximum spacing of vertical reinforcement. Also if the length of wall is too short, the effect may be meaningless. So this is applied the long outer wall, intergeneration wall and core wall in this study.



(a) before design change



(b) after design change

Figure 5-1 Vertical reinforcement concentrated at the ends of wall

In order to better understand the influence of the wall where vertical reinforcement is concentrated at the ends, nonlinear static analysis and time history analysis is performed for two models: OD20-C and OD20-200C. The difference of between two models is the wall thickness. The characteristic of the models is shown in Appendix.

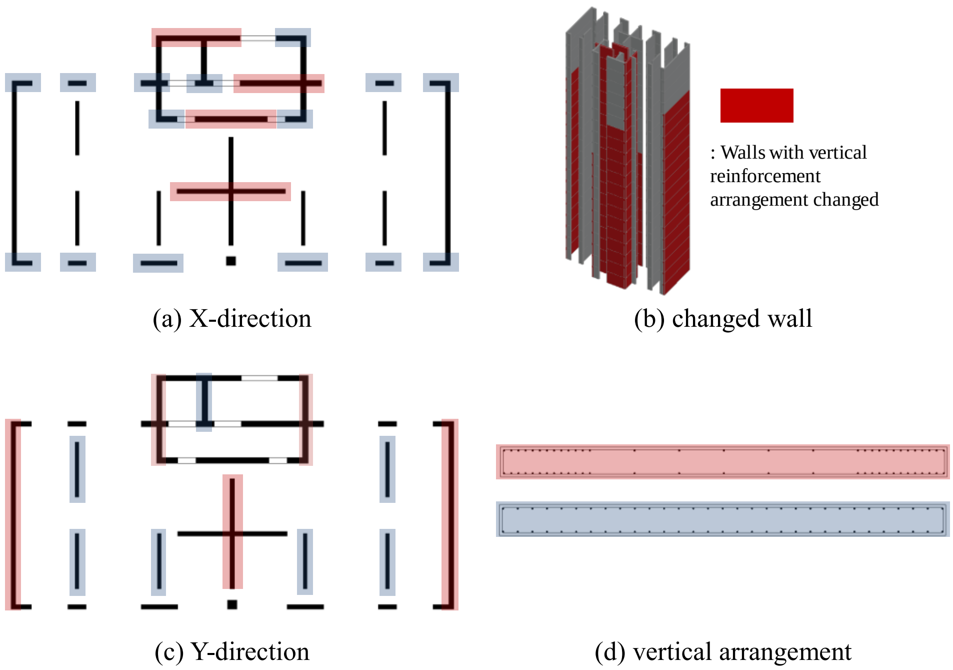
In fact, the modified details can have more outstanding effect in high-rise building or the building in hige seismic zone since they can be used largely. In addition, this is best applied the wall which has a 200~250 mm spacing of vertical reinforcement.

5.2 Nonlinear modeling

Nonlinear modeling parameter (material nonlinearity, member nonlinearity etc) used in Chapter 5 is the same as Chpater 4. Difference is the amount of reinforcement in fiber model.

5.2.1 Wall model

Walls with changed vertical reinforcement arrangement are shown in **Figure 5-2**. The minimum spacing of vertical reinforcmenet at the end is set to be over 40 mm considering the construction and the maximum reinforcement ratio of the vertical reinforcement is not less than 1% not to use the additional reinforcement.



(a) X-direction

(b) changed wall

(c) Y-direction

(d) vertical arrangement

Figure 5-2 Location of walls changed arrangement

5.3 Nonlinear static analysis

Load setting (expected gravity load, lateral force distribution etc) used in Chapter5 are the same as in Chapter 4.

5.3.1 Mode properties

Even though the position of the vertical reinforcement of walls is altered, the change of stiffness of the entire building is not large, so the natural period and mass participation factor do not differ greatly.

Table 5-1 Mode properties for the nonlinear models

Mode	OD20			OD20-C		
	T (s)	Mass X (%)	Mass Y (%)	T (s)	Mass X (%)	Mass Y (%)
1	1.67	63.96	0.55	1.66	63.80	0.61
2	1.46	0.61	62.94	1.46	0.68	62.59
3	0.81	0.09	0.01	0.80	1.02	0.01
4	0.36	19.14	0.03	0.34	19.03	0.01
5	0.26	0.01	20.36	0.25	0.01	20.55
6	0.18	0.53	0.04	0.18	0.54	0.02

5.3.2 Overstrength factor

For core walls, the arrangement of vertical reinforcement in walls do not affect the increase of bending strength. In the case of OD20-C, which vertical reinforcement of walls are concentrated at the ends, the maximum bending strength of the entire building slightly increases by 1.6% in the y-direction and it does not increase in the x-direction. Because the thickness of core wall and long outer walls is 300 mm and 250 mm in OD20-C, respectively, the contribution of core walls to the bending strength of the entire building is relatively greater than that of long outer walls. That is why the increase is not great. On the contrary, it is increased by 6.7% in OD20-200C where the portion of core wall is relatively reduced compared to the OD20-C.

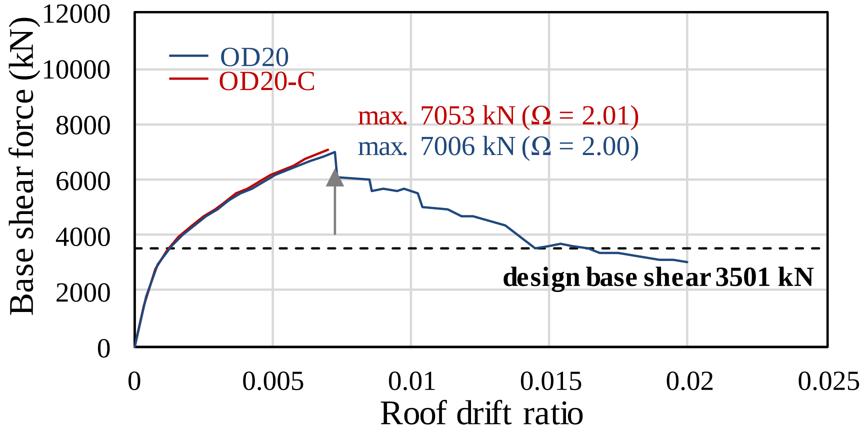


Figure 5-3 Overstrength factor of OD20-C (x-direction)

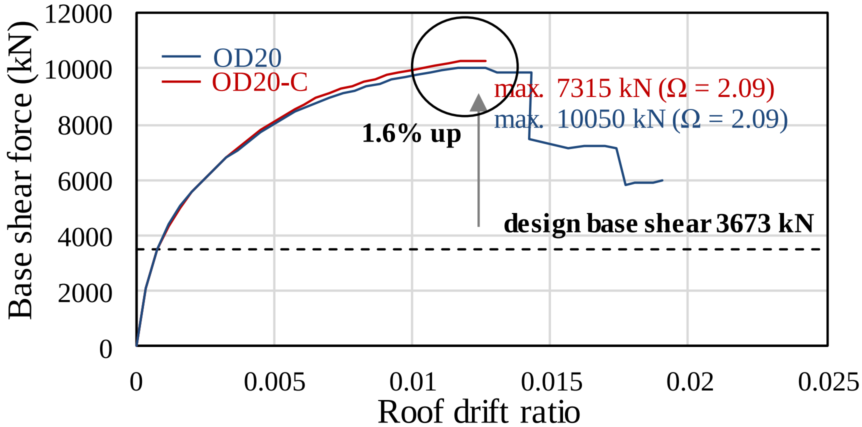


Figure 5-4 Overstrength factor of OD20-C (y-direction)

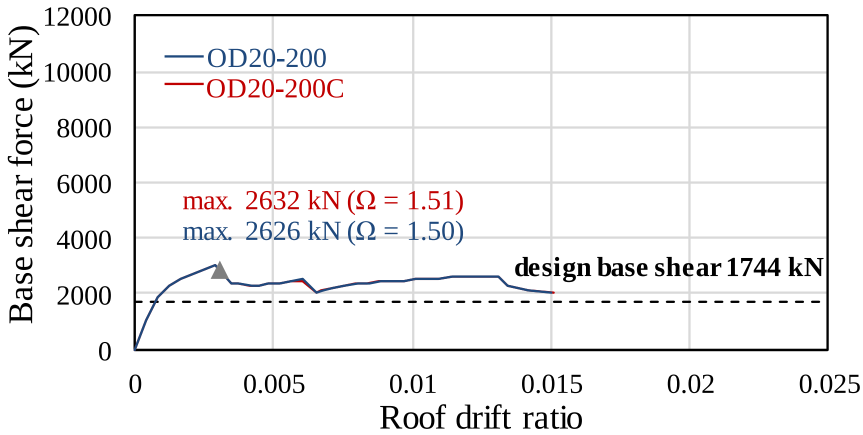


Figure 5-5 Overstrength factor of OD20-200C (x-direction)

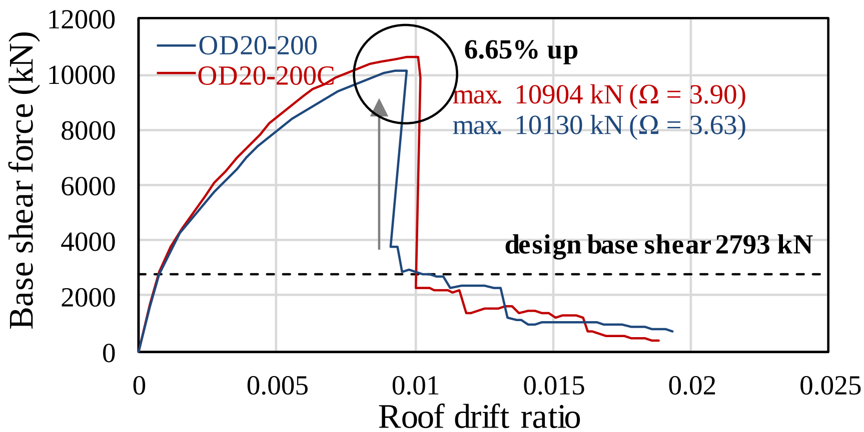


Figure 5-6 Overstrength factor of OD20-200C (y-direction)

5.3.3 Performance point

The seismic performance of building is evaluated at the performance point, which is the intersection of the capacity curve and demand curve. As shown in **Figure 5-7** and **Figure 5-8**, the point satisfying LS is formed at a relatively stable position after cracking occurred and stiffness of the entire building is reduced. In the both directions, the performance point is formed at similar position because the initial stiffness of buildings between two models is almost sames.

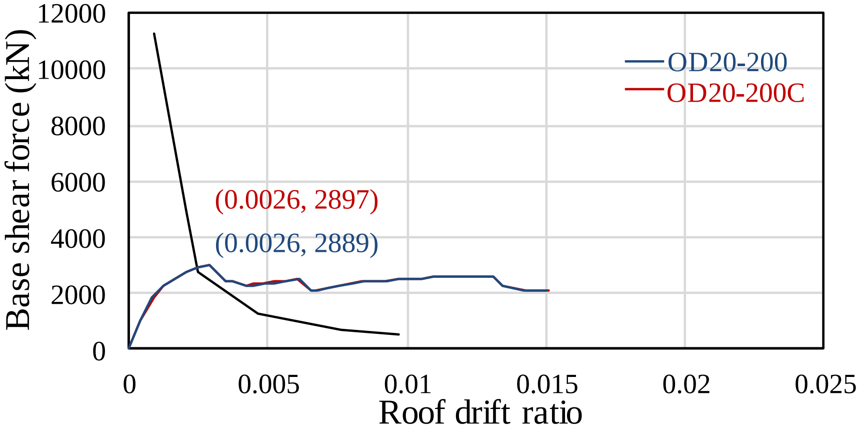


Figure 5-7 Performance point of OD20-200C (x-direction)

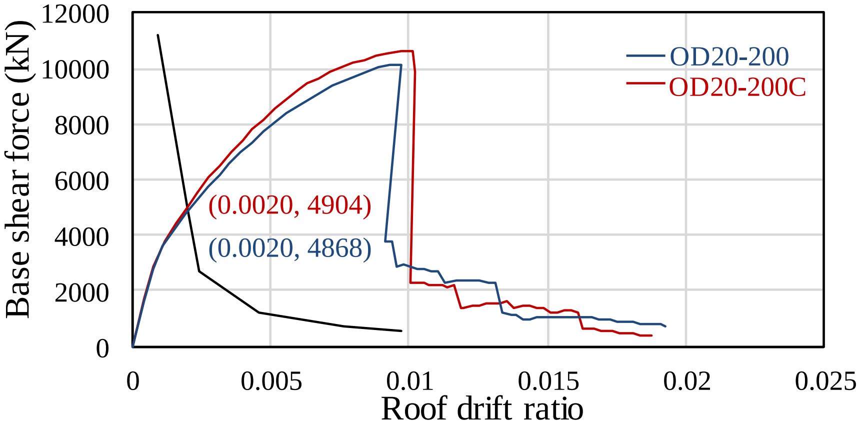
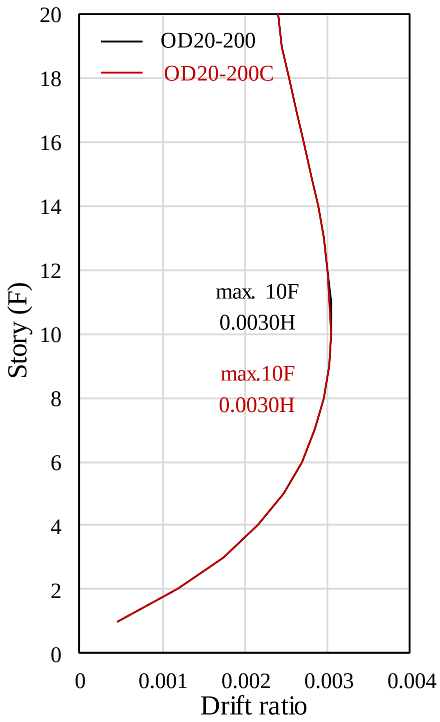


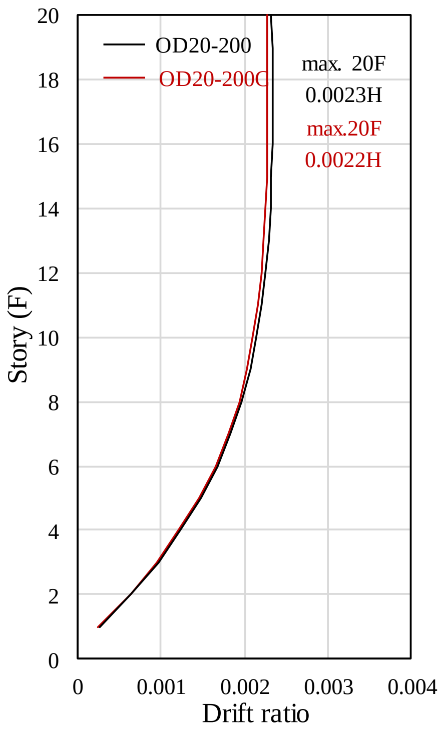
Figure 5-8 Performance point of OD20B-C (y-direction)

5.3.4 Interstory drift ratio

To satisfy the seismic performance for Life Safety, story drift ratio shall not exceed 0.015H according to the Guidelines for Performance-Based Seismic Design of Residential Buildings. As shown in **Figure 5-9**, the maximum story drift ratio is 0.0033H, so it satisfies the target performance. Since the performance point is formed at similar locations, the difference in inter story drift ratio at the point is not large.



(a) uniform shape - x direction



(a) uniform shape - y direction

Figure 5-9 Story drift ratio at performance point

5.4 Nonlinear time history analysis

In the nonlinear time history analysis, interstory drift ratio, shear force distribtution and wall rotation is almost same not only between OD20 and OD20-C and also between OD20-200 and OD20-200C, so the result is not contained in this paper.

5.5 Economic evaluation

Generally, vertical reinforcement of wall is designed to be equally spaced. When reinforcement is moved to both ends with only minimum reinforcement at the center of the wall, bending strength of the wall increases and this makes seismic performance improved. Or it is possible to reduce the amount of reinforcement while maintaining the same seismic performance.

As shown in **Figure 5-10**, When the vertical reinforcement of wall is concentrated at the ends, as like OD20-200C, bending strength of entire building increases by 5~6%. Wall reinforcement can be reduced with the same seismic performance. As a result, vertical reinforcement of both ends in W5 walls of 1^{st} to 12^{th} stories can be reduced by 4 EA. So the amount of reinforcement decreases by about 3.8% from 50.4 ton to 48.5 ton.

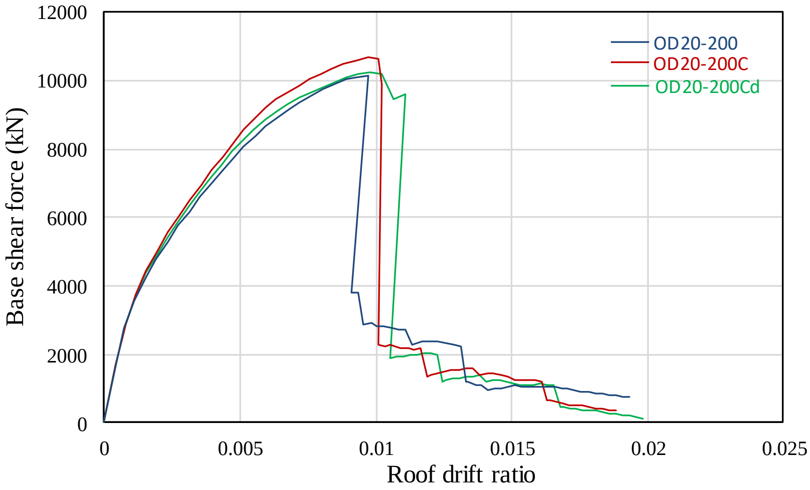


Figure 5-10 Bending strength of OD20B-Cd

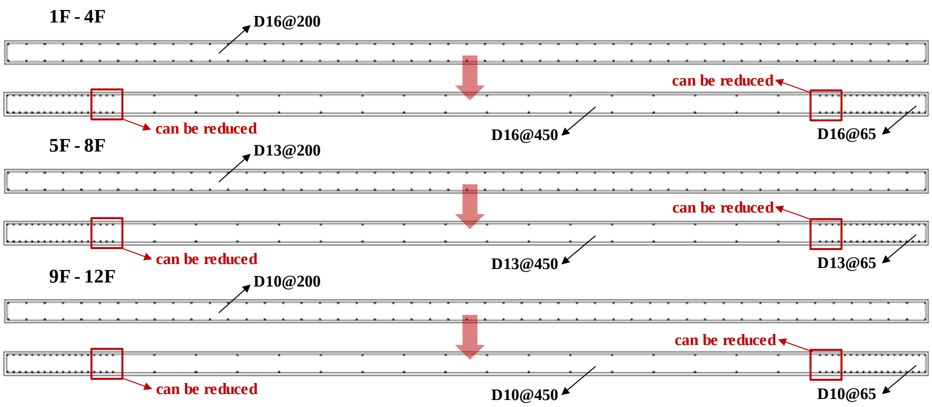


Figure 5-11 Reduced reinforcement of W5 in OD20-200Cd

5.6 Discusstion

In this chapter, nonlinear static analysis and time history analysis are conducted to check the effect of the wall that vertical reinforcement is concentrated at the ends without changing the amount of reinforcemet. The main results are summarized as follows:

1) Bending strength of the building is increased when vertical reinforcement of walls are concentrated at the ends of the walls but this do not affect for core walls.

2) The more contribution of core walls to the bending strength of the entire building is decreased, the more effect of concentrated walls is increased.

3) is possible to curtail the amount of reinforcement while maintaining the same seismic performance. In this study, the amount of reinforcement in walls reduced by about 3.8% if the reinforcement is concentrated at the ends.

Chapter 6. Conclusion

In this study, response spectrum analsysis for 20-story shear wall building and finite element analysis for typical floors are performed to investigate the influence of slab stiffness on the dyanamic behavior and to propose the appropriate slab modeling method for performance-based seismic design of residential building. Also PBSD considering arrangement of vertical reinforcement in wall is conducted. The conclusions are summarized as follows:

4) If the flexural stiffness of slab is considered, lateral displacement of building is decreased in both elastic analysis and lnelastic analysis. When the storng earthquake load are expected, it can be helpful to control lateral displacement.

5) The in-plane stiffness of slab is very large compared with the lateral stiffness of vertical members in concrete structures. So, it is more advantageous to consider out-of-plane flexural stiffness with rigid diaphram rather than modeling a slab as plate or shell element only.

6) Maximum flexural stiffness of slab that can be considered without changing slab design is about 10%. However, this value can be changed if the conditions such as seismic load, story height, and architectural plan of building are modified.

7) Because slab redistributes the seismic load, the load acting on the wall cuts down and the load on the slab increases on the contrary. When wall is designed for the reduced load by considering 10% of the flexural

stiffness of slab, it is possible to curtail the amount of wall reinforcmenet by about 6% without increasing the reinforcement of slab.

8) As a nonlinear static analysis result, bending strength of the entire building is 1.5 to 2 times higher when flexural stiffness of slab is considered by 10% since core slab plays a role of coupling beams continuously after flexural failure of beam in the x-direction. Coupling effect of slab is also appears in the y-direction.

9) The arrangement of vertical reinforcement in walls do not affect the increase of bending strength for core walls. However, the effect of vertical reinforcement concentrated at the ends of the wall is outstanding in the long outer walls or intergeneration walls. Generally most of the long walls exist in the y-direction, they are effective only in that direction in flat-type apartment.

10) Concentrating vertical reinforcement at the end of the walls increases the bending strength of the building. For economical design, it is possible to reduce the amount of reinforcement concentrated at the end to have the same bending strength as typical residential building where the vertical reinforcement of wall is equally spaced. In this study, it can reduce the amount of wall reinforcement by 5%.

11) When the contribution of core walls is relatively small, the effect of concentrated reinforcement at the end of walls becomes large. The bending strength of entire building increses by 6.7% in this study.

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Appendix A : Wall list

1. Characteristic of Models

OD20 : Elastic or inelastic analysis model that slab is modeled as rigid diaphram with infinite in-plane stiffness.

ODS20-10(25) : Elastic or inelastic analysis model considering out-of-plane flexural stiffness of slab by 10%(25%) with rigid diaphram.

OS20-10(25) : Elastic analysis model that slab is modeled as plate considering in-plane stiffness and out-of-plane flexural stiffness by 10%(25%).

ODS20-10d : Inelastic analysis model with the same characteristic of ODS20-10, but the member is designed considering flexural stiffness of slab.

OD20-C : Inelastic analysis model which has rigid diaphram and consider the arrangement of vertical reinforcement in wall.

OD20-200 : Inelastic analysis model with rigid diaphram and the thickness of all walls is 200 mm.

OD20-200C : Inelastic analysis model with the same characteristic of OD20-200 but it is considering the arrangement of vertical reinforcement in wall.

OD20-200Cd : Inelastic analysis model with the same characteristic of OD20-200C but the amount of reinforcement in wall is reduced.

2. Wall list of OD20

Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)	Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)
W1	20F	250	D10@300	D10@280	EW2	11 - 16F	300	D13@250	D10@230
	7 - 19F		D10@450	D10@280		7 - 10F		D13@150	D10@230
	1 - 6F		D10@200	D10@200		1 - 6F		D16@100	D10@190
W2,3,4	20F	200	D10@300	D10@350	EW3	17 - 20F	300	D10@300	D10@230
	1 - 19F		D10@450	D10@350		15 - 16F		D10@250	D10@230
W5	17 - 20F	250	D10@300	D10@280		13 - 14F		D10@200	D10@230
	13 - 16F		D10@300	D10@220		9 - 12F		D13@300	D10@230
	1 - 12F		D10@200	D10@220		7 - 8F		D13@250	D10@230
HW1	20F	250	D10@300	D10@280		3 - 6F		D16@250	D10@190
	7 - 19F		D10@450	D10@280		1 - 2F		D16@150	D10@190
	1 - 6F		D10@200	D10@220	EW4	7 - 20F	300	D10@300	D10@230
HW2	5 - 20F	250	D10@300	D10@280		3 - 6F		D10@300	D10@190
	3 - 4F		D10@300	D10@230		1 - 2F		D13@250	D10@190
	1 - 2F		D10@200	D10@220	EW5,6	1 - 20F	300	D10@300	D10@230
HW3	1 - 20F	250	D10@300	D10@280	CW1	5 - 20F	300	D10@300	D10@230
HW4	15 - 20F	250	D10@300	D10@190		3 - 4F		D10@250	D10@190
	11 - 14F		D13@300	D10@190		1 - 2F		D10@150	D10@190
	7 - 10F		D16@300	D10@190	CW2	11 - 20F	300	D13@150	D10@190
	5 - 6F		D16@200	D10@190		1 - 10F		D13@100	D10@190
	3 - 4F		D16@150	D10@190	CW3	17 - 20F	300	D10@300	D10@230
	1 - 2F		D16@100	D10@190		11 - 16F		D10@200	D10@230
HW5	15 - 20F	250	D10@300	D10@190		5 - 10F		D10@150	D10@190
	11 - 14F		D13@300	D10@190		3 - 4F		D10@200	D10@190
	7 - 10F		D16@300	D10@190		1 - 2F		D10@150	D10@190
	5 - 6F		D16@200	D10@190	CW4	15 - 20F	300	D10@300	D10@230
	3 - 4F		D16@150	D10@190		11 - 14F		D13@200	D10@230
	1 - 2F		D16@100	D10@190		7 - 10F		D13@150	D10@190
HW6	1 - 20F	250	D10@300	D10@280		1 - 6F		D13@100	D10@190
EW1	17 - 20F	300	D13@300	D10@230	CW5	15 - 20F	300	D10@300	D10@230
	11 - 16F		D13@250	D10@230		11 - 14F		D13@200	D10@230
	1 - 10F		D13@100	D10@190		7 - 10F		D13@200	D10@190
EW2	17-20F	300	D13@300	D10@230		1 - 6F		D13@100	D10@190

3. Wall list of ODS20-10

Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)	Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)
W1	20F	250	D10@300	D10@280	EW2	7 - 10F	300	D13@150	D10@230
	1 - 19F		D10@450	D10@280		1 - 6F		D16@100	D10@190
W2,3,4	20F	200	D10@300	D10@350	EW3	17 - 20F	300	D10@300	D10@230
	1 - 19F		D10@450	D10@350		9-16F		D10@300	D10@230
W5	17 - 20F	250	D10@300	D10@280		7 - 8F		D13@250	D10@230
	1 - 16F		D10@300	D10@220		3 - 6F		D16@250	D10@190
HW1	20F	250	D10@300	D10@280		1 - 2F		D16@150	D10@190
	7 - 19F		D10@450	D10@280	EW4	7 - 20F	300	D10@300	D10@230
	1 - 6F		D10@200	D10@220		3 - 6F		D10@300	D10@190
HW2	5 - 20F	250	D10@300	D10@280		1 - 2F		D13@250	D10@190
	3 - 4F		D10@300	D10@230	EW5,6	1 - 20F	300	D10@300	D10@230
	1 - 2F		D10@200	D10@220	CW1	3 - 20F	300	D10@300	D10@230
HW3	1 - 20F	250	D10@300	D10@280		1 - 2F		D10@200	D10@190
HW4,5	15 - 20F	250	D10@300	D10@190	CW2	1 - 20F	300	D13@150	D10@190
	11 - 14F		D13@300	D10@190	CW3	13 - 20F	300	D10@300	D10@230
	7 - 10F		D16@300	D10@190		5 - 12F		D10@200	D10@190
	5 - 6F		D16@200	D10@190		3 - 4F		D10@200	D10@190
	3 - 4F		D16@150	D10@190		1 - 2F		D10@150	D10@190
	1 - 2F		D16@100	D10@190	CW4	15 - 20F	300	D10@300	D10@230
HW6	1 - 20F	250	D10@300	D10@280		11 - 14F		D13@200	D10@230
EW1	17 - 20F	300	D13@300	D10@230		1 - 10F		D13@150	D10@190
	11 - 16F		D13@250	D10@230	CW5	15 - 20F	300	D10@300	D10@230
	1 - 10F		D13@100	D10@190		11 - 14F		D13@300	D10@230
EW2	17 - 20F	300	D13@300	D10@230		7 - 10F		D13@300	D10@190
	11 - 16F		D13@250	D10@230		1 - 6F		D13@150	D10@190

4. Wall list of OD20-200

Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)	Name	STORY	THK. (mm)	VER. (mm)	HOR. (mm)
W1,2,3,4	20F	200	D10@300	D10@350	EW3	5 - 8F	200	D16@200	D10@280
	1 - 19F		D10@450	D10@350		1 - 4F		D16@150	D10@280
W5	17 - 20F	200	D10@300	D10@280	EW4	17 - 20F	200	D10@300	D10@280
	13 - 16F		D10@300	D10@220		9 - 16F		D13@300	D10@280
	9 - 12F		D10@200	D10@220		5 - 8F		D13@200	D10@280
	5 - 8F		D13@200	D10@220		1 - 4F		D16@200	D10@280
	1 - 4F		D16@200	D10@280	EW5,6	5 - 20F	200	D10@300	D10@280
HW1	20F	200	D10@300	D10@280		1 - 4F		D13@300	D10@280
	7 - 19F		D10@450	D10@280	CW1	17 - 20F	200	D10@300	D10@280
	1 - 6F		D10@200	D10@280		9 - 16F		D13@300	D10@280
HW2,3	1 - 20F	200	D10@300	D10@350		5 - 8F		D13@200	D10@280
HW4,5	13 - 20F	200	D13@300	D10@190		1 - 4F		D16@200	D10@280
	9 - 12F		D16@300	D10@190	CW2	17 - 20F	200	D10@300	D10@190
	5 – 8F		D16@200	D10@190		9 - 16F		D13@300	D10@190
	1 - 4F		D16@100	D10@280		5 - 8F		D16@300	D10@190
HW6	1 - 20F	200	D10@300	D10@350		1 - 4F		D16@250	D10@190
EW1	17 - 20F	200	D10@300	D10@190	CW3	17 - 20F	200	D10@300	D10@280
	9 - 16F		D13@300	D10@190		5 - 16F		D13@300	D10@280
	1 - 8F		D16@300	D10@190		1 - 4F		D16@250	D10@280
EW2	17 - 20F	200	D10@300	D10@350	CW4	17 - 20F	200	D10@300	D10@280
	9 - 16F		D13@300	D10@350		13 - 16F		D10@200	D10@280
	5 - 8F		D13@200	D10@280		5 - 12F		D13@200	D10@280
	1 - 4F		D16@200	D10@190		1 - 4F		D16@200	D10@280
EW3	17 - 20F	200	D10@300	D10@280	CW5	13 - 20F	200	D10@300	D10@280
	13 - 16F		D13@300	D10@280		5 - 12F		D13@300	D10@280
	9 - 12F		D13@200	D10@280		1 - 4F		D16@250	D10@280

Appendix B : Matlab code for response spectra

```
clear all
clc
%% 지진파 기록

% 변수이름=xlsread(엑셀파일 명,sheet number,불러올 영역)

data=xlsread('선정지진파.xlsx',9,'C:C') ;

N=length(data) ;


%% TT(total time), dt(time step)
TT=5 ;
dt=0.02 ;
ddt=0.002 ;


%% Response Spectra
ACC=interp1(0.02:dt:N*dt,data(:,1),0.02:ddt:N*dt) ;
for z=1:1:TT/0.05


    % T, w, m, k, c, p, dr(damping ratio)
    T=z*0.05 ;
    w=2*pi/T ;
    dr=0.05 ;
    m=17500 ;
    k=m*w^2 ;
    c=dr*2*m*w ;
    p=-m*ACC(1,:) ;


    % initial condition
```

```
    u(1)=0 ;
    du(1)=0 ;
    ddu(1)=(p(1)-c*du(1)-k*u(1))/m ;

    % parameter
    a=1/2 ;
    b=1/6 ;
    a1=1/(b*ddt^2)*m+a/(b*ddt)*c ;
    a2=1/(b*ddt)*m+(a/b-1)*c ;
    a3=(1/(2*b)-1)*m+ddt*(a/(2*b)-1)*c ;
    k_tilda=k+a1 ;

    for i=1:length(ACC)-2
        p_tilda(i+1)=p(i+1)+a1*u(i)+a2*du(i)+a3*ddu(i) ;
        u(i+1)=p_tilda(i+1)/k_tilda;
        du(i+1)=a/(b*ddt)*(u(i+1)-u(i))+(1-a/b)*du(i)+ddt*(1-a/(2*b))*ddu(i) ;
        ddu(i+1)=1/(b*ddt^2)*(u(i+1)-u(i))-1/(b*ddt)*du(i)-(1/(2*b)-1)*ddu(i) ;
    end

    % max value
    max_disp(z)=max(abs(u)) ;
    max_vel(z)=max(abs(du)) ;
    max_acc(z)=max(abs(ddu)) ;
    max_pseudo_vel(z)=max_disp(z)*w ;
    max_pseudo_acc(z)=max_disp(z)*w^2 ;
end
```

초 록

슬래브 휨 강성 및 벽체 수직철근 배근 방식을 고려한 공동주택의 성능기반 내진설계 기법 연구

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요즘 국내 지진 발생 횟수가 증가함에 따라 건물의 내진성능평가와 성능기반 내진설계 관련 업무가 많아지고 있는 추세이다. 일반적으로 벽식구조 아파트의 성능기반 내진설계에서 슬래브는 중력하중만 저항하도록 설계되며, 풍하중이나 지진하중등의 횡 하중 저항요소로는 사용되지 않는다. 과거 1990년대 지어진 대다수 아파트의 슬래브 두께는 135~150 mm로, 슬래브의 휨 강성이 크지 않아 구조해석 단계에서 슬래브는 휨 강성이 없는 강막가정으로 모델링 되어왔다. 하지만 2009년 공동주택 층간소음 규정 강화로 인해 표준 슬래브 두께가 210 mm로 증대된 이후, 상대적으로 두꺼워진 슬래브는 횡 하중에 대해 충분히 저항할 수 있게 되었다. 따라서 최근에 설계되는 아파트의 경우 기존의 방식대로 강막가정만 사용하고 슬래브의 휨 강성을 고려하지 않는다면 건물의 실제 동적거동과 동적해석 결과가 큰 차이가 발생할 가능성이 있다.

초 록

슬래브의 휨 강성을 고려하게 되면, 각 층의 슬래브에 작용하는 지진하중의 크기가 달라 층마다 슬래브의 설계가 달라지게 된다. 이는 기준층 슬래브를 설계한 후 그 슬래브를 다른 층에 적용하는 기존의 설계 방법과 비교하여 업무량이 많아지게 되고, 또한 건물의 모델링과 해석에 걸리는 시간도 늘어나게 된다. 따라서 본 논문에서는 20층 높이의 벽식구조 아파트를 대상으로 하여 중력하중만 저항하도록 설계된 슬래브의 설계 변경 없이 반영 가능한 휨 강성을 유한요소해석을 통해 찾고, 이를 성능기반 내진설계에 적용하는 연구를 수행하였다. 또한 슬래브의 휨 강성을 벽식구조 아파트의 성능기반 내진설계에 적용하기에 앞서, 응답스펙트럼해석을 이용하여 슬래브의 면내 강성과 면외 휨 강성이 벽식구조 건물의 동적거동에 미치는 영향을 파악하고, 슬래브의 모델링과 해석시간을 감소시킬 수 있는 방법에 대한 연구가 선행되었다.

추가적으로 벽체의 수직철근 배열을 고려한 성능기반 내진설계에 관해 연구가 수행되었다. 일반적으로 벽체의 수직철근은 등간격으로 설계가 되는데, 본 연구에서는 수직철근량은 동일하게 유지하면서, 벽체 중앙 부분에는 최소한의 수직철근만 배근하고, 이외의 수직철근은 벽체 양 단부에 집중시킨 벽체를 가진 건물에 대해 비선형 정적해석과 시간이력해석을 수행하였다.

연구 결과 콘크리트 구조에서 슬래브의 면내 강성은 수직부재의 횡 강성에 비해 상대적으로 커 강막가정과 큰 차이가 발생하지 않았다. 따라서 강막가정을 그대로 적용한 채 슬래브의 휨 강성만 고려하는 방법이 슬래브를 플레이트요소나 쉘요소로 모델링 하는 방법보다

자유도를 줄일 수 있어 해석결과의 큰 차이 없이 해석시간을 단축시킬 수 있었다. 슬래브의 휨 강성을 고려했을 때 지진하중에 의한 건물의 층간변위가 줄어들고, 슬래브의 하중 재분배 효과로 인해 상하층간의 층전단력 분포, 반력 분포, 큰 벽체와 작은 벽체에 작용하는 하중 차이가 보다 균등해졌다. 중력하중에만 저항하도록 설계된 슬래브의 설계 변경 없이 고려가능한 면외 휨 강성은 약 10% 정도로 나타났고, 이를 성능기반 내진설계에 반영하였을 때 건물의 휨강도가 크게 증가했으며 인방보의 소성회전각이 감소하여 건물의 내진성능이 증가하였다. 또한 벽체의 철근량을 약 6% 정도 감소시킬 수 있어 강막가정에 의한 현행 설계법과 비교하여 경제성을 확보할 수 있었다.

벽체의 수직철근을 양 단부에 집중시킨 건물의 비선형 정적해석 결과 건물 전체의 휨강도가 5~6% 증가되었고 층간변위도 소폭 감소하였다. 벽체 수직철근이 등간격으로 된 모델과 동일한 휨 강도를 가지도록 벽체의 철근을 줄인 결과 약 4% 정도의 벽체 철근량을 감소시킬 수 있었다. 하지만 비선형 시간이력해석에서 건물의 동적 특성 차이는 거의 없었다.

주요어 : 공동주택 성능기반 내진설계, 슬래브 휨 강성, 비선형 해석, Perform-3D, 벽체 수직철근 배열

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감사의 글

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김 환 철