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Performance-Based Seismic Design of High-Rise Timber Office Building

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ABSTRACT: In South Korea, the relaxation of regulations on timber buildings in 2020 has led to increased interest in mid- to high-rise timber structures. This study aims to establish a performance-based seismic design procedure by designing a 13-story hybrid office building composed of GLT frames and RC shear walls in accordance with the current structural standard, KDS 41 17 00, and performing nonlinear analysis. The performance objectives of the building are defined, and the design process is iterated until the structural members and overall building behavior satisfy all allowable criteria. The RC shear walls are designed to resist all lateral forces, and different design approaches are applied based on the type of structural member. A nonlinear analysis model must be developed based on both basic design principles and actual nonlinear behavior, and the structural performance of each member is evaluated through nonlinear analysis.

The results of the nonlinear static analysis indicate that RC shear walls reach their shear strength limit within the elastic range, causing many walls to fail to meet performance requirements. Additionally, GLT beams exceeded the allowable connection rotation angle at the performance point. Therefore, to achieve the building's performance objectives, the shear strength of the RC shear walls must be improved. Increasing the wall thickness and horizontal reinforcement is necessary for redesign, and an appropriate design approach must be considered to account for the stiffness difference between the GLT frame and RC shear walls.

KEYWORDS: Hybrid structure, GLT, Office building, Performance-based seismic design, Nonlinear analysis

1 INTRODUCTION

1.1 HIGH-RISE TIMBER BUILDINGS

Individuals, governments, and the international community are joining efforts to address the climate crisis. Countries participating in the Paris Climate Agreement have established Nationally Determined Contributions (NDCs) to reduce greenhouse gas emissions, aiming to achieve these targets by 2030 through efforts across various sectors. In response, the construction industry has shifted its perspective on building materials, focusing on timber as a renewable and highly effective carbon-reducing material. Compared to steel and concrete, timber generates fewer greenhouse gas emissions during production and construction, and it continues to store carbon throughout the building's lifespan, further reducing CO2 emissions. Additionally, the development of mass-timber products has made it possible to construct high-rise and large-scale timber buildings, overcoming challenges that were once considered difficult in the past.

The world's first high-rise timber building, "Stadthaus," was completed in 2009 in London, UK, standing at 29 m and constructed entirely from timber. Since then, timber-based construction has gained momentum worldwide, with high-rise timber buildings becoming iconic landmarks. In 2019, notable examples included Mjøstårnet Tower (Norway, 85.4 m), HoHo Wien (Austria, 84 m), and Sara Kulturhus Centre (Sweden, 75 m), all serving as multi-purpose complexes built with timber. Currently, the tallest timber building in the world is Ascent, a mixed-use residential tower in Milwaukee, USA, standing at 86.6 m. Completed in 2022, it features a timber-concrete hybrid structure. Around the globe, there is a growing competition to construct the tallest timber building.

1.2 TIMBER BUILDINGS IN SOUTH KOREA

Regulations related to the structural design of timber buildings in South Korea began with the draft of the Timber Structure Design Standards in 1999. Considering the structural safety of timber buildings, the Korean Building Code (KBC) 2005 imposed limitations on timber structures, restricting the building height to a maximum roof height of 18 m, eave height of 15 m, and total floor area of 3,000 m² (or 6,000 m² with a sprinkler system). Although the KBC was revised in 2009 and 2016, the size restrictions for timber buildings remained unchanged [1]. However, with advancements in engineered wood products and construction technology enabling larger timber structures, the size restriction clause for timber buildings was abolished in 2020 [2]. Following this revision, the Haedong Advanced Engineering Building at Seoul National University was constructed the tallest timber building in Korea. Standing

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at 29.9 m with 7 stories, it features a hybrid structure of reinforced concrete and timber. This marks the beginning of applying timber construction to public buildings in Korea, with efforts underway to expand its scale and scope to promote the widespread adoption of timber architecture.

Most office buildings in Korea are mid- to high-rise structures predominantly made of reinforced concrete. Replacing or combining these materials with engineered wood could significantly reduce carbon emissions. Therefore, establishing specific design procedures for mid- to high-rise timber buildings in Korea aims to contribute to the urbanization of timber construction.

2 PERFORMANCE-BASED SEISMIC DESIGN AND TIMBER STRUCTURE STANDARDS IN SOUTH KOREA

2.1 PERFORMANCE-BASED SEISMIC DESIGN

In 2016, Performance-Based Seismic Design (PBSD) was first introduced into the Korean Building Code, and in 2019, the Seismic Design Standards for Buildings (KDS 41 17 00) were officially announced. PBSD is a technique that designs structures to meet targeted seismic performance levels, utilizing nonlinear analysis to perform structural analysis and verification. The performance-based design method requires clear presentation of procedures and justifications, with the overall design process outlined in Fig. 1. When designing using the performance-based design method, performance objectives must be established to achieve the desired seismic performance level for the structure. The performance objectives according to the current standard (KDS 41 17 00) are presented in Table 1, and at least two of the minimum performance objectives must be satisfied. The performance level of a building must meet the performance requirements for both structural and non-structural elements. The 2400-year recurrence interval earthquake is defined as the Maximum Considered Earthquake (MCE), representing the largest earthquake considered in seismic design. The 1000-year recurrence interval earthquake is designated as the Basic Design Earthquake, with a spectral acceleration corresponding to two-thirds of the value of the MCE. The MCE must be included in the performance objectives for design purposes[3]. Basic design aims to configure the seismic force-resisting system and design its members through nonlinear structural analysis and verification, based on elastic analysis as per the Seismic Design Standards for Buildings. To ensure the accuracy of nonlinear analysis, a nonlinear property model is developed based on the results of elastic analysis. Nonlinear analysis is conducted using either nonlinear static analysis or nonlinear dynamic analysis, selecting the method most appropriate for the structure. The results of the nonlinear analysis are used to verify compliance with the target performance level, including the maximum inter-story drift ratio for each story, minimum strength requirements for base shear, allowable plastic rotation angles for each member by performance level, and allowable strength values for each member by performance level. If the acceptance criteria are not met, redesign must be performed until all items satisfy the target performance levels.

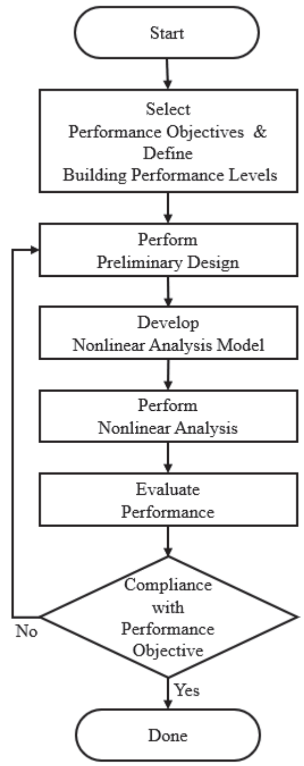

Figure 1: Performance-Based Seismic Design Process

Table 1: Seismic Performance Levels & Performance Objectives in KDS 41 17 00

Seismic Performance Levels	***Seismic Hazard Levels***	***Target Building / Structural / Nonstructural Performance Levels***
Special	2400-year return period earthquake (1.5 x Design Spectral acceleration)	Life safety / Life safety / Life safety
	1000-year return period earthquake (1.0 x Design Spectral acceleration)	Operatioal / Immediate Occupancy / Operatioal
I	2400-year return period earthquake (1.5 x Design Spectral acceleration)	Collapse Prevention / Collapse Prevention / -
	1400-year return period earthquake (1.2 x Design Spectral acceleration)	Life safety / Life safety / Life safety

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	100-year return period earthquake (0.43 x Design Spectral acceleration)	Operatioal / Immediate Occupancy / Operatioal
II	2400-year return period earthquake (1.5 x Design Spectral acceleration)	Collapse Prevention / Collapse Prevention / -
	1000-year recurrence interval (1.0 x Design Spectral acceleration)	Life safety / Life safety / Life safety
	50-year return period earthquake (0.3 x Design Spectral acceleration)	Operatioal / Immediate Occupancy / Operatioal

2.2 TIMBER STRUCTURE DESIGN STANDARDS

According to the National Construction Standards Center of South Korea, a total of 13 design standards related to timber structures have been established[4-8]. For the structural design of mid- to high-rise timber structures, the most commonly utilized standards include KDS 41 50 10 (Timber Structure Materials and Allowable Stresses), KDS 41 50 15 (Timber Structure Design Requirements), KDS 41 50 20 (Timber Structure Member Design), KDS 41 50 30 (Design of Timber Structure Connections), and KDS 41 50 80 (Heavy Timber Structures). Timber structure design follows the Allowable Stress Design (ASD) method, ensuring that structural timber and connections do not exceed their design allowable stresses. According to KDS 41 50 10, the seismic design of timber structures applies the equivalent static analysis method outlined in KDS 41 17 00 (7.2). This is due to timber structures generally being lightweight and exhibiting excellent seismic performance through vibration absorption. Additionally, the dynamic analysis method specified in KDS 41 17 00 (7.3) may also be used for design. Under the current standards, the only defined seismic force-resisting system for timber structures is the light-frame timber shear wall system. Heavy timber seismic force-resisting systems such as Glued Laminated Timber (GLT) moment frames, GLT braced frames, and (Cross-Laminated Timber (CLT) shear walls are not specified [3, 6].

While KDS 41 50 15 allows connections to be assumed as pinned or rigid, it does not provide criteria for a stiffness classification system. In practice, it primarily addresses shear connection design, making it challenging to approach the design of semi-rigid or rigid connections. The lack of clearly defined standards and components creates practical difficulties in the seismic design of high-rise and large-scale timber buildings using engineered wood. Therefore, this study aims to perform seismic design for timber buildings taller than 13 stories based on South Korea's current structural standards and analyze the results.

3 PROJECT DESCRIPTION

3.1 OVERVIEW OF THE TARGET BUILDING

The target building is a hybrid structure composed of a GLT frame and RC shear walls, with a total height of 55.4 m and 13 stories above ground, designed as an office building (Fig. 2). The first-floor has a height of 5 m, while the typical floor height is planned at 4.2 m. The core of the building consists of RC shear walls, with the remaining structure featuring a symmetrically arranged GLT frame. As shown in Fig. 3, the modular grid of the floor plan is 4.2 m in the X-direction and 6.6 m in the Y-direction. The compressive strength of the concrete is 27 MPa, and the yield strength of the reinforcement is 500 MPa. The timber used is structural glued laminated timber with a symmetrical differential grade of 10S-30B. The connections between the GLT beam-column and the RC shear wall-GLT beam utilize steel plate-inserted bolted joints.

In accordance with the current structural standards, the structural members of the target building will be designed. A nonlinear static analysis will be conducted to evaluate its seismic performance and establish a performance-based seismic design procedure.

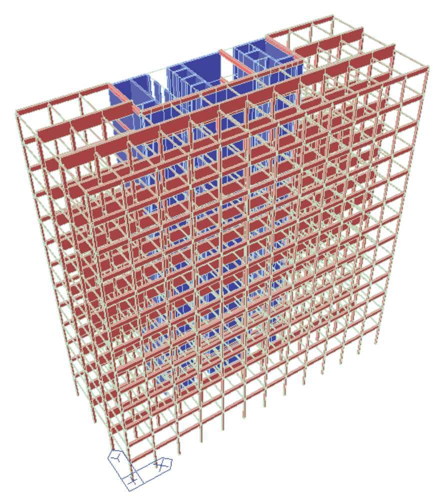

Figure 2: Target Building model

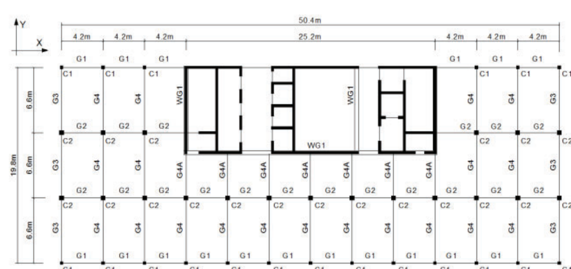

Figure 3: Plan of Target Building

3.2 LOAD CONDITIONS

The floor slab is assumed to be a composite floor system consisting of a 150 mm structural glued laminated timber

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panel (30 mm x 5 layers) topped with a 60 mm mortar finish. It is designed to bear a one-way load in the Y-direction of the plane. Dead loads were calculated considering the composite floor slab and other finishes, while live loads were determined as uniformly distributed loads, with additional partition loads reflecting the characteristics of the target building, as shown in Table 2. The seismic and wind load design conditions were established based on the building's location and importance. The target building is a general office structure with an importance category of 1. Accordingly, its seismic grade is Class I, with an importance factor (I_E) of 1.2. A seismic zone factor of 0.11g, corresponding to Seismic Zone 1, was applied. The soil type is classified as S4, and the seismic design category is D. The RC shear walls are planned to resist 100 % of the horizontal forces, and the seismic force-resisting system is designated as a building frame system with ordinary RC shear walls, using a response modification factor (R) of 5. For wind load design conditions, the basic design wind speed is 28 m/s, the exposure category is C, and the importance factor (I_W) is 1.0.

Table 2: Gravity loads for preliminary design (kN/m^2)

Story	RC Wall Core		GLT Frame	
	Dead Load	Live Load	Dead Load	Live Load
Typical	5.28	3.50	2.43	3.50
Roof	6.20	1.00	6.20	1.00

4 DESIGN PROCESS

4.1 PERFORMANCE OBJECTIVES

The seismic grade of the target building is Class I, with performance objectives set as Life Safety (LS) for a 1400-year return period earthquake and Collapse Prevention (CP) for a 2400-year return period earthquake. Since the target building does not include non-structural elements, the performance level of the structural elements is adopted as the building's performance objective. Referring to Chu et al. [9-10], the evaluation items and acceptance criteria for the performance assessment of the target building were determined (Table 3). The allowable inter-story drift was calculated based on the collapse prevention criterion of 3 % for the maximum considered earthquake under Seismic Grade II, as specified in KDS 41 17 00, divided by the importance factor, to establish the allowable inter-story drift values for LS and CP. The shear performance evaluation of the RC shear walls was determined by applying the design strength to set the acceptance criteria. The flexural performance evaluation referenced the Guidelines for Performance-Based Seismic Design of Reinforced Concrete Structures [11], calculating the allowable plastic rotation angle for each wall based on the axial force ratio and acting shear force [12]. For the GLT beam-column bolted connections, performance varies depending on the number of bolts used, and the rotation angle at the point where moment strength sharply decreases in the moment-rotation relationship was designated as the allowable criterion for collapse prevention. GLT columns were considered to exhibit force-dominated behavior with no deformation capacity after reaching maximum load, and the moment strength based on axial force was set as the allowable criterion for collapse prevention. The LS acceptance criteria for all evaluation items, except the plastic rotation angle of the RC shear walls, were set at 75 % of the CP values.

Table 3: Acceptance Criteria for Target Building

Parameters		Performance Level	
		LS	CP
Story Drift Ratio		2.0 %	2.5 %
RC Shear Wall	Shear Force	0.75 X Shear Strength	1.0 X Shear Strength
	Plastic Rotation angle	LS Level	CP Level
GLT Beam-Column Connection	Rotation angle	0.75 X CP Level	CP Level
GLT Column	Moment	0.75 X Moment Strength	1.0 X Moment Strength

4.2 PRELIMINARY DESIGN

The target building is designed such that the RC shear walls resist all lateral forces, with each structural member designed considering the applicable loads and design methods (Fig. 4). The design loads include gravity loads, seismic loads, and wind loads. The GLT frame was designed using the allowable stress design method, considering only gravity loads, while the RC shear walls were designed using the strength design method, accounting for both gravity loads and 100 % of the lateral loads. For the GLT frame, the design allowable stress was calculated by multiplying the reference allowable stress by all applicable adjustment factors. The cross-sections of the members were designed to ensure safety without exceeding the stresses induced by external forces.

For the connection design, the reference allowable strength of a single bolt was calculated in accordance with KDS 41 50 30, and the design allowable strength was determined by multiplying this value by all applicable adjustment factors. In the structural analysis, both ends of the beam members were assumed to be pinned connections. The required shear force at both ends of the beam, obtained through elastic analysis, was divided by the design allowable strength to calculate the number of bolts needed at each connection. The beam depth was then redesigned based on the number and arrangement of bolts in the designed connections.

Table 4 presents the member design results, showing that the load transfer direction of the floor slab and the connection design influenced the cross-section of the beam members. The vertical reinforcement ratio of the RC shear walls ranges from 0.25 % to 1.01 %, and the horizontal reinforcement ratio ranges from 0.16 % to 0.34 %, with each wall designed accordingly.

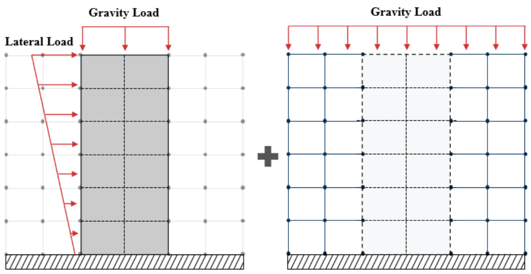

Figure 4: Load Assignment Based on Structural Elements

Table 4: Dimensions of section

Element	Section	Dimension (mm)	Material
Girder	G1	300 X 720	Wood
	G2	300 X 1830	
	G3	120 X 180	
	G4	120 X 180	
	G4A	120 X 180	
	WG1	400 X 600	RC
Column	C1	300 X 300	Wood
	C2	420 X 420	
Wall		250	RC

4.3 NONLINEAR MODEL DEVELOPMENT

The development and execution of the nonlinear static analysis were performed using the structural analysis program Perform-3D. The nonlinear analysis model was created based on the member design results from the elastic design. For the GLT frame, consisting of beams and columns, an elastic model was employed. At the beam ends, where bolted connections are located, a Moment Hinge model was used because the timber around the bolts experiences tearing under load, leading to progressive failure (Fig. 5 (a)). The moment-rotation relationship was derived using the method proposed by Awlaudin et al. [13]. The moment resistance capacity of the connection was calculated using the principle of energy conservation, which states that the external work done on the structure equals the internal work. Equation (1) represents the external work, defined as the work done by the bending moment applied to the connection with respect to the resulting rotation angle. Equation (2) represents the internal work, defined as the work done by the strength of each bolt with respect to its displacement. Accordingly, the moment strength equation can be formulated based on the bolt load-displacement relationship at the connection, as shown in (3).

$$W_E = \int_0^{\theta_{max}} M\, d\theta \tag{1}$$

$$W_I = \sum_{bolts}\left(\int_0^{s_{max}} F\, ds\right) \tag{2}$$

$$M = \frac{d}{d\theta}\sum_{bolt}\left(\int_0^{s_{max}} F\, ds\right) \tag{3}$$

Due to the anisotropic nature of wood, it is possible to determine load-displacement curves and connection coefficients by combining cases where the load is applied parallel (∥) and perpendicular (⊥) to the fiber direction.

$$k_{e\alpha} = \frac{k_{e\parallel}\, k_{e\perp}}{(k_{e\parallel} sin^m\alpha + k_{e\perp} cos^m\alpha)} \tag{4}$$

$$k_{p\alpha} = \frac{k_{e\parallel} + k_{e\perp}}{2} \tag{5}$$

The load-displacement experimental results for a single bolted connection exhibited a bilinear form, consisting of an elastic region and a plastic region.

$$F = k_{e\alpha} s\ ;\ s \le s_e \tag{6}$$

$$F = k_{e\alpha} s_e + k_{p\alpha}(s - s_e)\ ;\ s > s_e \tag{7}$$

Using this, the moment-rotation relationship can be predicted.

$$M = \theta \sum_{bolts}(k_{e\alpha} r_i^2)\ ;\ s \le s_e \tag{8}$$

$$M = \sum_{bolts}(k_{e\alpha} - k_{p\alpha}) s_e r_i + \theta \sum_{bolts}(k_{p\alpha} r_i^2)\ ;\ s > s_e \tag{9}$$

The moment-rotation relationship was predicted as shown in Fig. 5 (b), referencing the Load-Slip experimental results for a single bolt from Gattesco et al. [14]. The columns were modeled using a P-M2-M3 Hinge. Since the relationship between axial stress and flexural stress is defined for timber columns, it was converted into an axial force-moment relationship as shown in Fig. 6. The axial stress was multiplied by the cross-sectional area of the column, and the flexural stress was multiplied by the section modulus. In the standard, the allowable stress equation was multiplied by a safety factor of 2.1 for engineered wood to determine the axial stress and flexural stress values for glued laminated timber [15]. The flexural behavior of the RC shear walls was modeled using Fiber elements, composed of a combination of material models for concrete and reinforcement (Fig. 7 (a)). The material strengths of concrete and reinforcement were expressed as stress-strain relationships based on expected strengths. Referring to the nonlinear model for performance-based seismic design of reinforced concrete buildings [16], expected strength coefficients of 1.1 for concrete and 1.13 for reinforcement were applied(Fig. 7 (b), (c)). The nonlinear flexural behavior of the walls was effective only in the in-plane direction, while the out-of-plane direction used 0.25 of the elastic modulus of the in-plane direction. The shear behavior of the walls was modeled as elastic, with a reduction in shear stiffness due to

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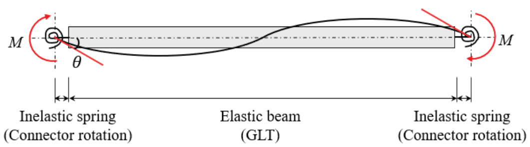
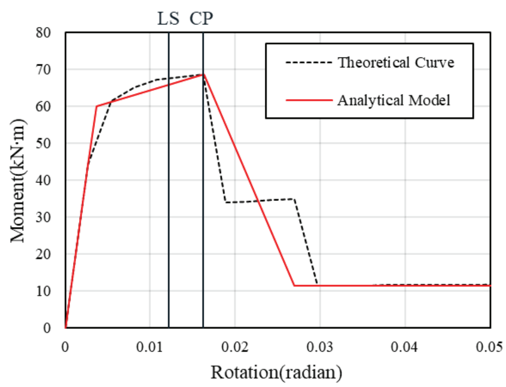
cracking reflected by applying 0.5 of the shear elastic modulus.



(a)



(b)

Figure 5: Properties of Connection: (a) Beam model (b) Moment-Ratation Curve

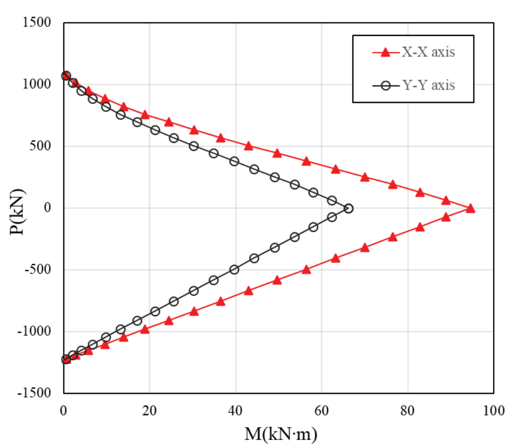
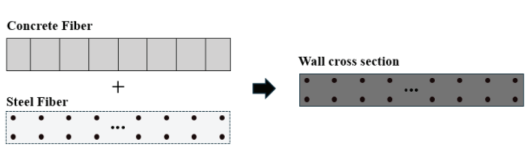


Figure 6: P-M Interacion Curve of GLT Column



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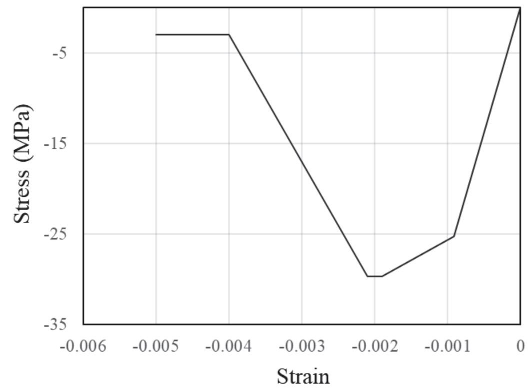

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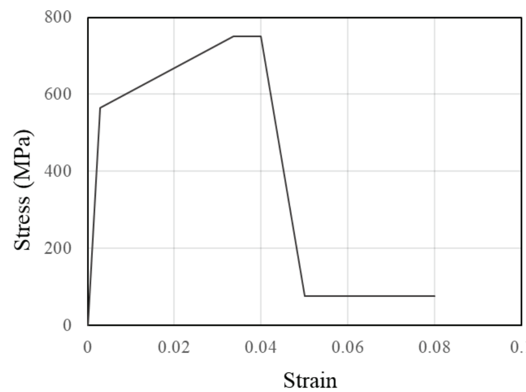


(c)

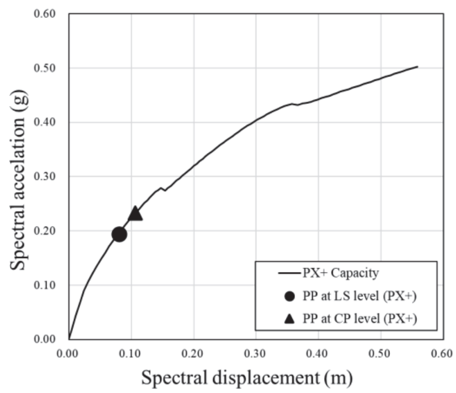
Figure 7: Properties of RC Shear Wall: (a) RC Shear Wall Fiber model (b) Concrete Stress-Strain Curve (c) Steel Stress-Strain Curve

5 PUSHOVER AND PERFORMANCE EVALUATION

5.1 PERFORMANCE CURVE

Nonlinear static analysis is a technique that considers the material nonlinear behavior characteristics of individual members, gradually increasing the lateral displacement of the system to determine the relationship between member strength and nonlinear deformation [11]. Seismic loads were applied to the target building in the X-direction (+PX, -PX) and Y-direction (+PY, -PY), with the reference point for lateral displacement in the pushover curve set as the center of mass at the top floor. The capacity curve, representing the relationship between base shear and roof displacement, was calculated and then converted into a capacity spectrum expressed as an acceleration-displacement response relationship. Based on an elastic demand spectrum that does not account for the response modification factor, a reduced inelastic demand spectrum was derived by considering the structure’s energy dissipation capacity.

The performance point was determined as the intersection where the demand spectrum, representing seismic demand, overlaps with the capacity spectrum, representing the structure's seismic capacity. Fig. 8 illustrates the performance points corresponding to LS and CP for the target building in the X-positive direction and Y-positive direction.



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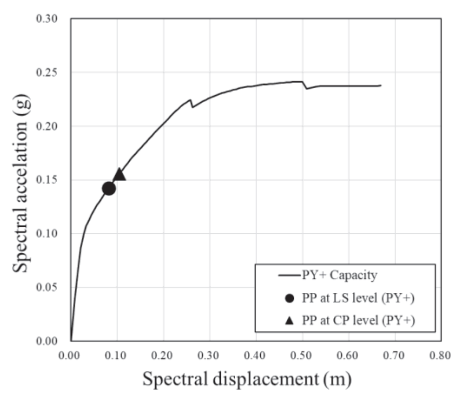


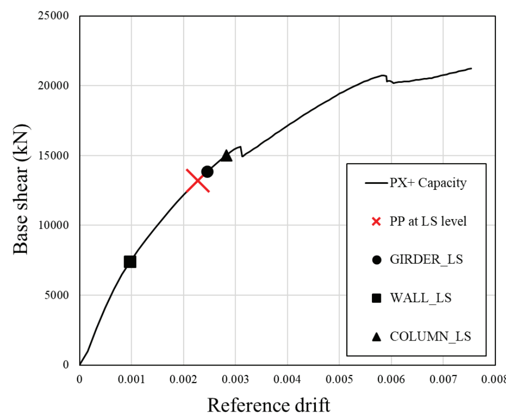
(b)

Figure 8: Pushover Curve: (a) X-Positive Direction (b) Y-Positive Direction

5.2 FAILURE MECHANISM

The target building is a hybrid structure composed of members made from different materials, the failure mechanism was analyzed to understand the building's behavior. Fig. 9 indicates the points at which failure occurs in the members, with performance objectives set for LS and CP in the X-positive direction. It can be observed that the members exceed their acceptance criteria in the order of RC shear walls, GLT beams, and GLT columns. For the RC shear walls, some walls reach their shear strength in the elastic region before the performance point is attained, with shear forces exceeding the shear capacity. As shown in Fig. 9 (a), the GLT beams and GLT columns reach the LS acceptance criteria after the performance point. In Fig. 9 (b), the CP acceptance criterion for the GLT beams aligns with the performance point, while the GLT columns satisfy the acceptance criterion at the performance point.

Fig. 10 illustrates the members that exceed the acceptance criteria at the CP performance level in the X-positive direction, showing that the walls fail from the lower floors to the upper floors, while the beams experience failure in the G2 members connecting to the shear walls on the 13th floor. As the load increases beyond the performance point, the failure of the beams is expected to propagate from the upper floors downward to the lower floors.



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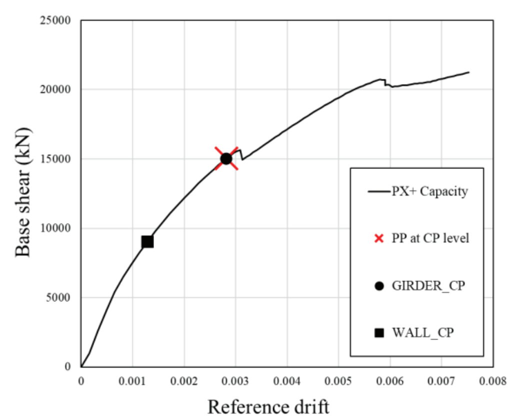

(b)

Figure 9: Collapse Mechanism in the X-Positive Direction (a) LS Level (b) CP Level

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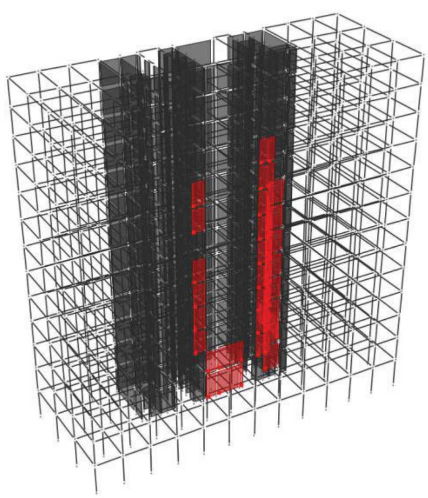

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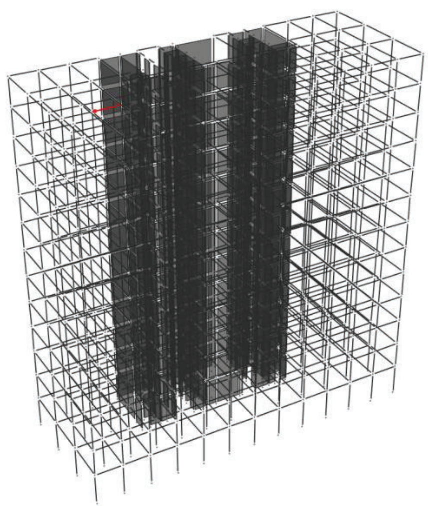

(b)

Figure 10: Failure of Structural Elements at the CP Level in the X-Positive Direction (a) RC Shear Wall (b) Beam

5.3 COMPLIANCE WITH ACCEPTANCE CRITERIA

The evaluation item for assessing the overall performance of the building is the inter-story drift ratio, while the performance evaluation items for individual members include the shear force, plastic rotation angle, and end compressive strain of the RC shear walls, the connection rotation angle at the ends of the GLT beams, and the moment strength of the GLT columns. Fig. 11 and Fig. 12 graphically represent the inter-story drift ratio and the allowable inter-story drift ratio for each direction. At the life safety level, the maximum inter-story drift is 0.31 % in the X-direction and 0.47 % in the Y-direction, while at the collapse prevention level, the maximum inter-story drift ratio is 0.40 % in the X-direction and 0.58 % in the Y-direction, all of which satisfy the criteria. The inter-story drift ratio increases toward the upper floors, suggesting that in a building composed of walls and frames, the walls dominate the overall behavior of the structure.

The compressive strain of the RC shear walls is within the allowable criterion of 0.002, satisfying the requirement. Tables 5 and 6 present the results for members with performance objectives of LS and CP, confirming compliance with the evaluation items. At the LS level, the GLT beams and GLT columns satisfy the allowable limits, but the RC shear walls exhibit a maximum Demand-Capacity Ratio (DCR) of 2.22 for shear in the X-direction, failing to satisfy the acceptance criteria in all directions. At the CP level, the connection rotation angle at the ends of the GLT beams and the shear performance of the RC shear walls do not meet the acceptance criteria in the X-direction. When comprehensively reviewing the results for members at both the LS and CP levels, the RC shear walls satisfy the performance criteria for flexure but fail to meet the criteria for shear. This indicates a significant load burden on the walls, as they are designed as the primary seismic force-resisting elements. The GLT columns, despite having their strength reduced by applying a safety factor, still satisfy the acceptance criteria.

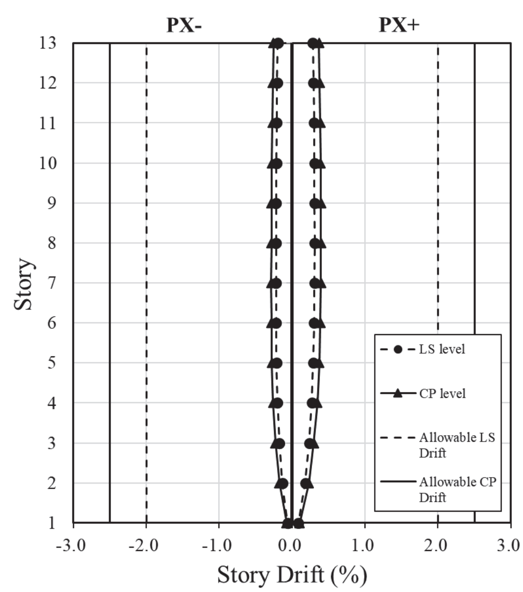

Figure 11: Story Drift Ratio in the X-Direction

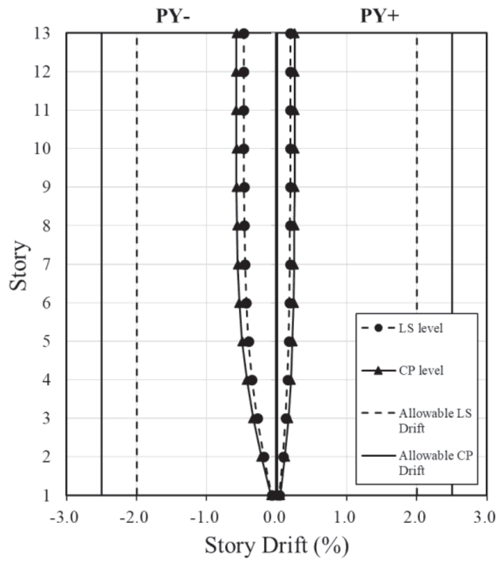

Figure 12: Story Drift Ratio in the Y-Direction

Table 5: Performance Level by Element in LS

Parameters	PX+	PX-	PY+	PY-	Result
GLT Beam-Column Connection	0.94	0.78	0.43	0.77	OK
GLT Column	0.83	0.20	0.49	0.57	OK
RC Shear Wall Strength	2.22	1.58	1.08	1.12	NG
RC Shear Wall Rotation	0.56	0.41	0.28	0.41	OK

Table 6: Performance Level by Element in CP

Parameters	PX+	PX-	PY+	PY-	Result
GLT Beam-Column Connection	1.06	0.68	0.33	0.66	NG
GLT Column	0.76	0.13	0.40	0.45	OK
RC Shear Wall Strength	1.91	1.43	0.90	0.92	NG
RC Shear Wall Rotation	0.41	0.30	0.21	0.29	OK

6 CONCLUSION

This study established a Performance-Based Seismic Design procedure in accordance with current standards for a 13-story office building composed of a GLT frame and RC shear walls. A nonlinear analysis model was developed through member design for the building with defined performance objectives, and the performance level of the building was assessed through nonlinear static analysis and evaluation. While the basic design of the target building can comply with current structural standards, the acceptance criteria for each member and the development of the nonlinear analysis model must reflect actual nonlinear behavior. Consequently, defining acceptance criteria and developing nonlinear analysis models for GLT members are necessary. In particular, since bolted connections can vary depending on timber species and bolt types, experimental results are critical for accurate analysis models.

The nonlinear static analysis results revealed that the shear strength of the walls and the connection rotation angle of the GLT beam members did not meet the acceptance criteria. As a result, the overall performance of the building was unsatisfactory, necessitating redesign to achieve the performance objectives. Since the failure of the RC shear walls was predominant compared to other members, redesign should prioritize increasing wall thickness and horizontal reinforcement to enhance shear strength. Additionally, an appropriate design method that accounts for the significant stiffness disparity between the GLT frame and RC walls needs to be considered.

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