A Simplified SDOF Model for Loss Estimation Evaluating Seismic Resilience

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***Abstract*—This study introduces a resilience performance evaluation method for structures, which assesses their ability to recover following an earthquake disaster. The resilience evaluation approach quantifies recovery capacity through a loss estimation model and a recovery evaluation model. The recovery evaluation model considers both repair time and downtime, while the loss estimation model includes not only direct damage to structural and non-structural components but also indirect losses due to functional disruption. To develop an efficient loss estimation model, structural simplification is required. Accordingly, this study proposes an equivalent terminal induction system, applying both Cantilever-Type and Rahmen-Type Single-Degree-of-Freedom (SDOF) models. The evaluation results indicate a slightly conservative estimate compared to the example structure, suggesting that further improvements in hysteresis behavior could be achieved by incorporating a stiffness reduction factor into the SDOF model.**

***Keywords*—resilience based design, seismic fragility analysis, SDOF System, incremental dynamic analysis**

I. Introduction

Current seismic design standards for buildings utilize Performance-Based Design (PBD) to define performance levels—such as functionality, life safety, and collapse prevention—based on seismic recurrence intervals. However, since some structural damage is accepted under these criteria, there is no guarantee regarding the feasibility of repair or reuse of the structure following seismic events. As a response, Resilience-Based Design (RBD) has emerged as a new approach, focusing on quantitatively assessing a building's resilience and incorporating this assessment into seismic design to ensure recovery to its pre-event condition. In the context of architecture, resilience refers to the ability to recover swiftly to the pre-disaster state and, on a broader scale, represents the capacity to address disaster through effective recovery strategies within social systems. When the resilience concept is applied to seismic performance evaluation of buildings, it enables the prediction of repair costs and restoration periods, facilitating efficient and cost-effective reinforcement strategies during seismic retrofitting. This approach promotes structural designs that prioritize rapid functionality recovery following seismic damage.

To quantitatively assess structural resilience, it is essential to predict both the loss and the recovery period following a seismic disaster. Loss prediction models for seismic events should encompass not only direct losses, such as damage to structural and non-structural components and human casualties, but also indirect losses, such as economic impacts due to building downtime [1]. The recovery period must account for repair time of structural and non-structural elements as well as various delay factors, including interruptions in power and water supply, inspection periods prior to retrofitting, and redesign periods, all of which contribute to total downtime. Therefore, to achieve accurate resilience assessments, it is crucial to establish a quantifiable evaluation method and build a recovery scenario database suited to local conditions. Resilience-based seismic design methodologies have been explored, allowing resilience ratings to be integrated into building design by anticipating seismic disaster scenarios. However, resilience-based design remains a relatively unfamiliar concept in South Korea, with limited research in this area. Accordingly, this study aims to examine resilience-based performance evaluation and proposes a simplified single-degree-of-freedom (SDOF) structural analysis model for loss estimation, with the objective of exploring pathways for its application within South Korea.

II. Introduction Overview of Resilience Based Performance Evaluation

The concept of resilience refers to a structure's ability to recover its pre-disaster performance level within a control period (TLC) designated by the building owner. In the study by C. P. Cimellaro *et al.* (2010) [2], resilience of buildings is quantitatively evaluated using numerical methods, as illustrated in Fig. 1 and expressed through Eqs. (1) to (2).

$$R = \int_{t_{OE}}^{t_{OE}+T_{LC}} Q(t)/T_{LC}dt \tag{1}$$

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$$Q(t) = [1 - L(I, T_{RE}][H(t - t_{OE}) - H(t - (t_{OE} + T_{RE}))] \times f_{Rec}(t, t_{OE}, T_{RE}) \quad (2)$$

where, $L(I, T_{RE})$ represents the loss function, $f_{Rec}(t, t_{OE}, T_{RE})$ denotes the recovery function for the recovery period, $H(\,)$ is the step function for each phase, T_{RE} indicates the recovery period following the disaster, and t_{OE} marks the disaster occurrence time.

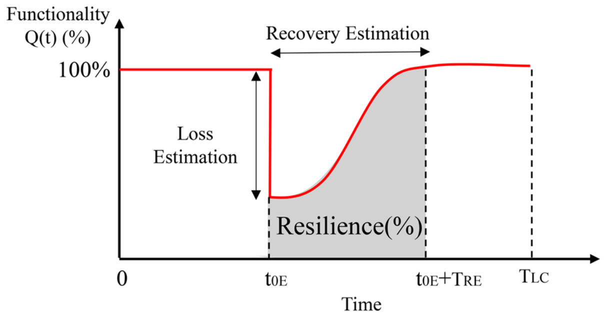


Fig. 1. Schematics of a functionality function for resilience system.

Resilience can be evaluated in buildings based on four components: robustness, rapidity, resourcefulness, and redundancy [3]. Rapidity refers to a structure's ability to restore its original condition and functionality following a seismic disaster, which can be represented by the rate of change in the resilience function ($\frac{dQ(t)}{dt}$) over the recovery period ($t_{OE} \leq t \leq t_{OE} + T_{RE}$). Robustness is defined as the ability of a structure's system and components to withstand a seismic disaster and can be quantified as the residual resilience ($1 - L(m_L, \sigma_L)\%$) at the time of the earthquake. Resourcefulness and redundancy refer to resources or elements available to restore building functionality if it is lost due to seismic damage. These components are interrelated and influence the shape and slope of the resilience function over the recovery period ($t_{OE} \leq t \leq t_{OE} + T_{RE}$).

Resilience-based performance design for structures begins by predicting the seismic intensity based on the seismic hazard map of the structure's location and selecting an appropriate design earthquake ground motion. A nonlinear dynamic analysis is then conducted on the target structure's analytical model. Through this analysis, dynamic responses—such as inter-story drift ratio, base shear, and acceleration—are extracted. These responses are used to determine seismic vulnerability by calculating damage probabilities based on damage criteria, and these probabilities are applied to the loss evaluation and recovery period functions to quantify the resilience score. This resilience-based performance evaluation incorporates three key elements—life safety, damage, and recovery—into the design, allowing for the estimation of repair costs and recovery time. The design aims to restore the structure's functionality as quickly as possible following an earthquake. Consequently, strategies for enhancing resilience should focus on minimizing structural losses and ensuring a short repair period by selecting effective seismic retrofitting methods. Additionally, the application of base isolation and damping systems can further improve structural resilience against seismic events [4].

III. Loss and Recovery Period Prediction Model

To evaluate the resilience of structures after seismic disasters, it is possible to employ probabilistic seismic vulnerability analysis, which enables the prediction and assessment through Loss Evaluation Models and Recovery Evaluation Models. The Loss Evaluation Model estimates both direct and indirect losses resulting from seismic events. It calculates probabilistic loss costs by multiplying the aggregated total costs of losses by the probability of damage occurrence. Direct losses (L_D) refer to direct structural and non-structural damages (L_{DE}) as well as casualties (L_{DC}) due to seismic disasters. Indirect losses (L_I) include disruptions to business, rental income losses, and casualties resulting from the functional shutdown of medical facilities like hospitals due to seismic events. The total loss prediction can be expressed as shown in Eq. (3).

$$L_{I,D} = L_{DE,IE} + L_{DC,IC} \quad (3)$$

$$L_{DE}(I) = \sum_{j=1}^{n}[\frac{C_{S,j}}{I_s} \cdot \prod_{i=1}^{T_i} \frac{(1+\delta_i)}{(1+r_i)}] \cdot P_j\{\cup_{i=1}^{n}(R_i \geq \frac{r_{lim}}{I}\} \quad (4)$$

$$L_{DC}(I) = \frac{N_{in}}{N_{tot}} \quad (5)$$

where, P_j represents the probability of exceeding a certain Damage State given a seismic intensity I. $C_{s,j}$ denotes the repair cost for a building at Damage State j, while I_s stands for the construction cost of a new building. The parameter r_i is the annual discount rate, t_i is the repair period after the disaster, and δ_i indicates the annual depreciation rate. Additionally, N_{tot} refers to the number of casualties caused by the disaster, and N_{in} represents the number of fatalities due to the disaster.

Indirect losses refer to those incurred when a building or structure fails to function properly, such as relocation costs or loss of rental income, and are generally more challenging to quantify compared to direct losses. However, they are just as significant as direct losses. For instance, if a bridge collapses due to a seismic disaster, the repair costs for structural damage would not be the only concern. There would also be indirect losses, such as the loss of revenue generated by the bridge's functionality during the repair period, and the disruption to the flow of traffic dependent on the bridge. Therefore, these indirect losses must be accounted for in resilience-based performance assessments. In the case of medical facilities like hospitals, the loss of building functionality could lead to indirect casualties that may be as substantial as the direct casualties caused by structural damage.

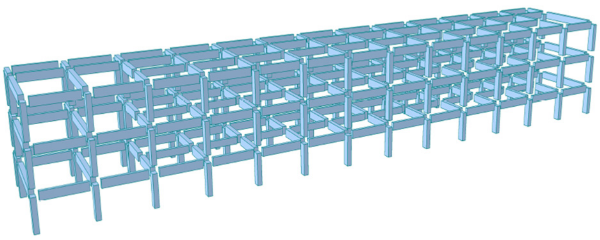
The Recovery Evaluation Model is a crucial factor for quantitatively assessing the resilience of structures, as it represents the time required to restore a structure to its pre-earthquake, as-new condition. Currently, predictions of repair time are primarily based on the methodology outlined in FEMA P-58 [5], and quantitative estimates can be made using the PACT (Performance Assessment Calculation Tool) software distributed by the Applied

Technology Council (ATC). However, the FEMA P-58 [6] methodology for estimating repair time is limited to direct losses and repair periods for structural and non-structural components, with inadequate consideration of downtime during seismic events.

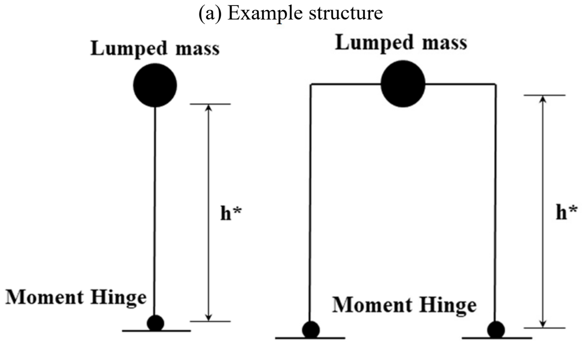
In response to these limitations, Company has developed a new method for predicting repair time that incorporates downtime due to seismic disasters [1]. Accurate prediction of repair time should account not only for the period required to repair damaged structural and non-structural elements but also for downtime caused by impeded factors, such as post-earthquake inspections, retrofitting design periods, and the restoration of power and water supply facilities. These downtime considerations have a significant impact on resilience-based recovery time predictions.

IV. Introduction Equivalent Single-Degree of Freedom System for Loss Cost Estimation

To develop predictive models for loss and recovery time, it is essential to estimate the probability of damage occurrence, which can be derived through seismic vulnerability analysis. In seismic vulnerability assessments, the Capacity Spectrum Method using the nonlinear static method allows for effective analysis of a structure's performance and the demand level against seismic loads. However, its accuracy is relatively lower for structures significantly influenced by higher modes or with irregular geometries [6]. In contrast, nonlinear dynamic analysis is a more accurate and reliable method, as it involves applying actual seismic waves to structures to understand their dynamic responses.



(a) Example structure



(b) Cantilever SDOF (c) Rahmen SDOF

Fig. 2. Equivalent SDOF analytical model.

As a result, Incremental Dynamic Analysis (IDA), which incrementally increases the seismic acceleration until the structure reaches a collapse level, is commonly used to evaluate seismic vulnerability. Despite its accuracy, IDA is often impractical for application in real-world scenarios due to the lengthy analysis time required and the inefficiency of capturing critical dynamic responses through repeated simulations. Therefore, a more efficient approach involves substituting real structures with an equivalent Single-Degree-of-Freedom (SDOF) system for analysis. In this study, an equivalent SDOF system in the form of a cantilever and frame is proposed [7]. A reinforced concrete structure with a beam-column frame system, similar to school facilities, is selected as the example structure, as shown in Fig. 2. The proposed cantilever-type equivalent single-degree-of-freedom (SDOF) system can be established following the procedure outlined in Fig. 3.

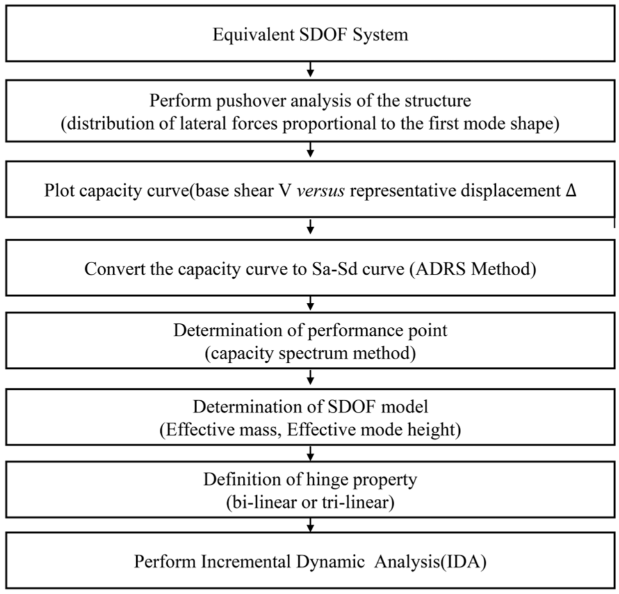


Fig. 3. Equivalent SDOF analytical mode.

Initially, a pushover analysis is performed using the lateral force distribution (s_1) for the primary mode of the example structure, as shown in Eq. (6). This analysis allows for the derivation of the capacity curve, which represents the relationship between the base shear force and the roof displacement.

$$s_1 = m\phi_1 \tag{6}$$

The performance curve must be determined by comparing it to the demand spectrum in order to represent the required strength against seismic loads. To achieve this, the performance point is calculated using the ADRS (Acceleration-Displacement Response Spectrum) method, as outlined in Eqs. (7–9). The ADRS method allows for the conversion of the base shear force (V_{b1}) and roof displacement (Δ_1) into a capacity spectrum in the form of spectral acceleration and spectral displacement, using the mass participation factor (β_1) and mode vector (Φ_1) for the primary mode. These values are then expressed on a single coordinate system. For the example structure in this study, the demand spectrum is calculated based on KDS 41 17 00 [6], and the performance point corresponds to a 1.5x DBE spectrum demand level. The coordinates of this point are considered as the structure's ultimate strength point.

$$\beta = \frac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[M]\{\phi_1\}} \tag{7}$$

$$V_{b1} = M_1 S_{a1} \tag{8}$$

$$\triangle_1 = \beta_1 \phi_{i1} S_{d1} = S_{d1} \tag{9}$$

where, β_1 is the mass participation factor, M_1 is the effective mass for the primary mode of the structure, S_{a1} is the spectral acceleration for the primary mode, and S_{d1} is the spectral displacement for the primary mode.

The performance curve for the example structure is shown in Fig. 4, and through linearization of the performance curve, the nonlinear hysteretic characteristics of the equivalent single-degree-of-freedom (SDOF) structure were determined, including yield strength, post-yield stiffness, and ultimate strength. The equivalent SDOF analysis model was configured as a cantilever with a concentrated mass at the column base, and the mass participation rate for the primary mode was calculated to be 1513 tons. Additionally, the lateral stiffness of the cantilever and frame system was set to match the initial elastic stiffness of the performance curve. As a result, the natural period (0.7399 sec) for the primary mode of the structure was matched in the model. The height of the equivalent SDOF analysis model was determined using the effective height for the mode characteristics, as shown in Eq. (10), to reflect the structural characteristics according to the height of the example structure.

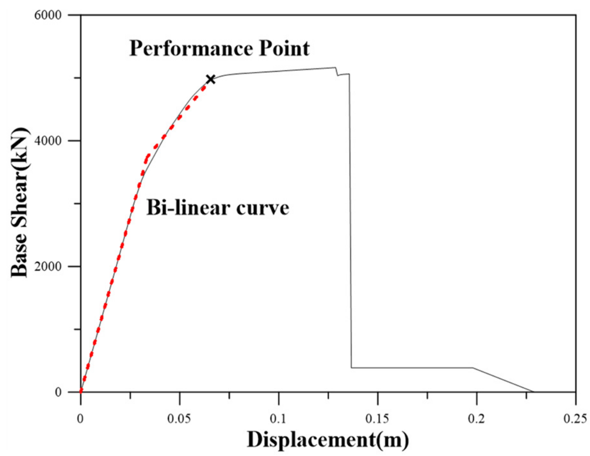


Fig. 4. Capacity curve of example structure.

$$h^* = \frac{L_n}{L_\theta} \tag{10}$$

$$L_n = \sum_{i=1}^{n} m_i \phi_i \tag{11}$$

$$L_\theta = \sum_{i=1}^{n} h_i m_i \phi_i \tag{12}$$

where, h^* is the effective height for the mode characteristics, m_i is the nodal mass, Φ_i is the modal vector at the node, and h_i is the height of the corresponding node.

The nonlinear characteristics of the single-degree-of-freedom (SDOF) analysis model were defined by applying moment-hinge elements (plastic hinge elements) at the column base. The flexural strength and rotation angles at the yield point and ultimate strength point of the performance curve were calculated and incorporated into the hinge properties. The equivalent SDOF system was verified by comparing performance curves using commercial analysis software, Perform 3D and Midas Gen, as shown in Fig. 5.

The typical process of Incremental Dynamic Analysis (IDA) involves selecting a set of ground motions and gradually increasing their intensity based on the spectral acceleration corresponding to the structure's first natural period. A time-history analysis is then conducted to extract dynamic responses, such as maximum base shear, maximum roof drift ratio, and inter-story drift ratios. These responses are plotted on an acceleration spectrum-dynamic response graph to generate the IDA curve. Due to the repetitive nature of this analysis procedure, it is more efficient to simplify the actual structure for analysis. In this study, incremental dynamic analyses were performed on the previously proposed cantilever-type and frame-type SDOF models, followed by a comparative analysis.

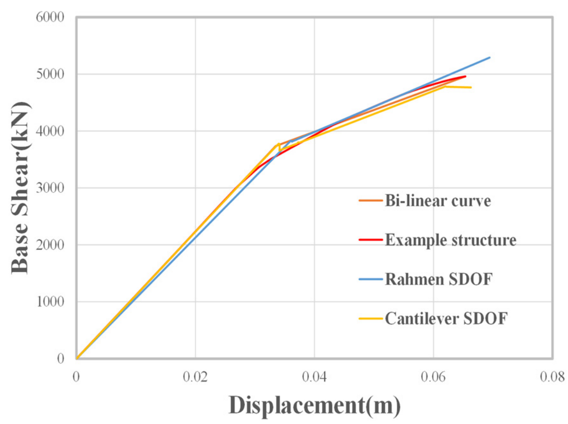


Fig. 5. Comparison of capacity curve.

TABLE I. SEISMIC RECORDS

Case	Earthquake	Max. Acc(gal)
EQ1	Northridge A	0.42
EQ2	Northridge B	0.41
EQ3	Duzce, Turkey	0.73
EQ4	Hector	0.27
EQ5	Imperial Valley A	0.24
EQ6	Imperial Valley B	0.36
EQ7	Kobe, Japan A	0.51
EQ8	Kobe, Japan B	0.24
EQ9	Kocaeli, Turkey A	0.31
EQ10	Kocaeli, Turkey B	0.22
EQ11	Landers A	0.24
EQ12	Landers B	0.28
EQ13	Loma Prieta A	0.53
EQ14	Loma Prieta B	0.56
EQ15	Manjil, Iran	0.51
EQ16	Superstition Hills A	0.36
EQ17	Superstition Hills B	0.45
EQ18	Cape Mendocino	0.39
EQ19	Chi-Chi, Taiwan A	0.35
EQ20	Chi-Chi, Taiwan B	0.47
EQ21	San Fernando	0.21
EQ22	Friuli, Italy	0.35

In this analysis, 22 seismic ground motion records were used for nonlinear dynamic analysis, as presented in Table 1. Each seismic dataset varies in magnitude, soil conditions, and epicentral distance, so the records were scaled to approximate the design spectrum of KDS 41 17 00, as shown in Fig. 6 The spectral acceleration corresponding to the natural period of the analysis model was incremented from 0.1g to 2.0g, and nonlinear dynamic analysis was conducted using the commercial structural analysis software, Perform 3D. Inter-story drift ratios were utilized as the primary dynamic response data, and the resulting IDA curves are shown in Fig. 6. In this analysis, 22 seismic ground motion records were used for nonlinear dynamic analysis, as shown in Table 1. Since each seismic dataset varies in magnitude, soil conditions, and epicentral distance, the records were scaled to closely match the design spectrum of KDS 41 17 00 [8], as illustrated in Fig. 6. The spectral acceleration corresponding to the natural period of the analysis model was incremented from 0.1g to 2.0g, and nonlinear dynamic analysis was conducted using the commercial structural analysis software, Perform 3D. Inter-story drift ratios were used as the primary dynamic response indicator, and the resulting IDA curves are presented in Fig. 7.

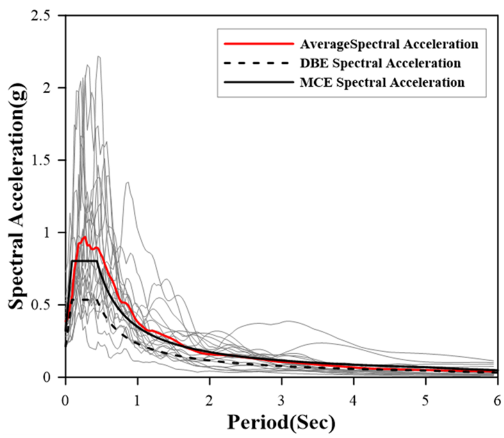
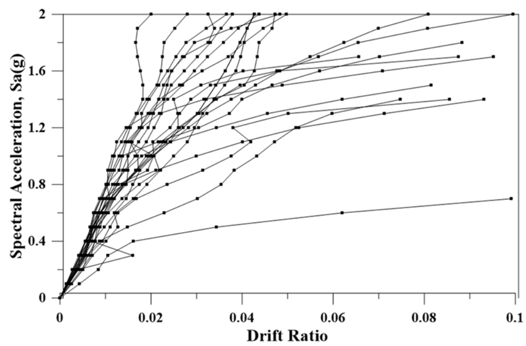


Fig. 6. Scaling of ground motions

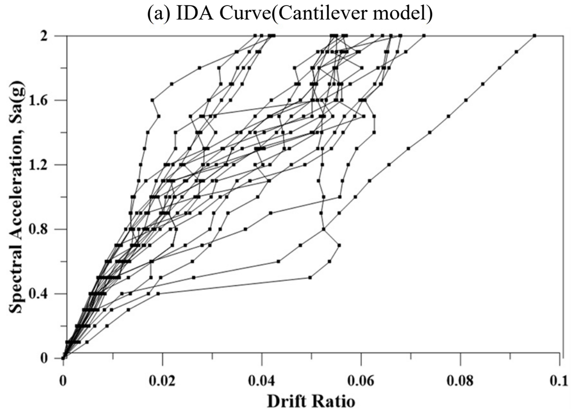
The Incremental Dynamic Analysis (IDA) results for the equivalent Single Degree of Freedom (SDOF) system showed that the cantilever-type equivalent SDOF system exhibited a relatively linear behavior up to an inter-story drift ratio of approximately 0.02. Beyond an inter-story drift ratio of 0.03, nonlinear deformation began to occur, and residual displacements started to appear in the SDOF system. When the inter-story drift ratio reached 0.1, no further increase in the drift ratio was observed, and significant residual deformation in the structure led to the termination of the analysis. Similarly, the frame-type equivalent SDOF system exhibited linear behavior up to an inter-story drift ratio of about 0.02, after which residual deformation became apparent. Unlike the cantilever-type SDOF system, the frame-type system experienced structural failure at an inter-story drift ratio of approximately 0.05, ceasing further analysis.

If an earthquake of a certain intensity occurs, the probability of specific levels of structural damage and their exceedance, based on seismic intensity, can be probabilistically evaluated using a seismic fragility function. This approach enables an assessment of seismic performance. To implement this, it is first necessary to define the damage state criteria for the equivalent Single Degree of Freedom (SDOF) system. According to HAZUS, damage state criteria are divided into four levels: Slight, Moderate, Extensive, and Complete.

Focusing on concrete frames, the characteristics of each damage level are as follows: Slight indicates minor cracking in some members; Moderate means that some members have reached a yielding state; Extensive signifies that most members have yielded, with some reaching their ultimate state; and Complete represents either structural collapse or a near-collapse condition.



(a) IDA Curve(Cantilever model)



(b) IDA Curve(Rahmen model)

Fig. 7. IDA curve.

In the study conducted by Barbat *et al.* [9], inter-story drift ratio criteria for each damage state were suggested based on the Capacity Spectrum, as shown in Table 2. For the "Slight" damage state, the drift ratio corresponds to 70% of the yield point (d_y) on the bi-linearized capacity curve. The "Moderate" damage state is defined as the inter-story drift ratio at the yield point (d_y). The "Extensive" damage state corresponds to 25% of the ductility capacity ($d_u - d_y$), and the "Complete" damage state is associated with the point of maximum strength (d_u).

TABLE II. DAMAGE STATE INDEX

Damage state	Thresholds
Slight	$0.7d_y$
Moderate	d_y
Extensive	$d_y + 0.25(d_u - d_y)$
Complete	d_y

Based on the aforementioned damage state criteria, the probability that the structural response exceeds the damage thresholds from the incremental dynamic analysis results can be expressed as a conditional function, as shown in Eq. (13).

$$P[C < D | SI = x] = 1 - \Phi[\frac{\ln(\hat{C}/\hat{D})}{\sqrt{\beta_{D|SI}^2 + \beta_C^2 + \beta_M^2}}] \quad (13)$$

where, Φ represents the area under the standard normal probability integral, $\hat{C}$ denotes the spectral acceleration that induces each damage state for the example structure across 11 ground accelerations, $\hat{D}$ is the required spectral acceleration for the structure, and $\sqrt{\beta_{D|SI}^2 + \beta_C^2 + \beta_M^2}$ represents the overall system uncertainty. The uncertainty of the structural system is outlined in FEMA P695 [10], and in this analysis, it was assumed to be 0.6, considering uncertainties that may arise during the modeling process.

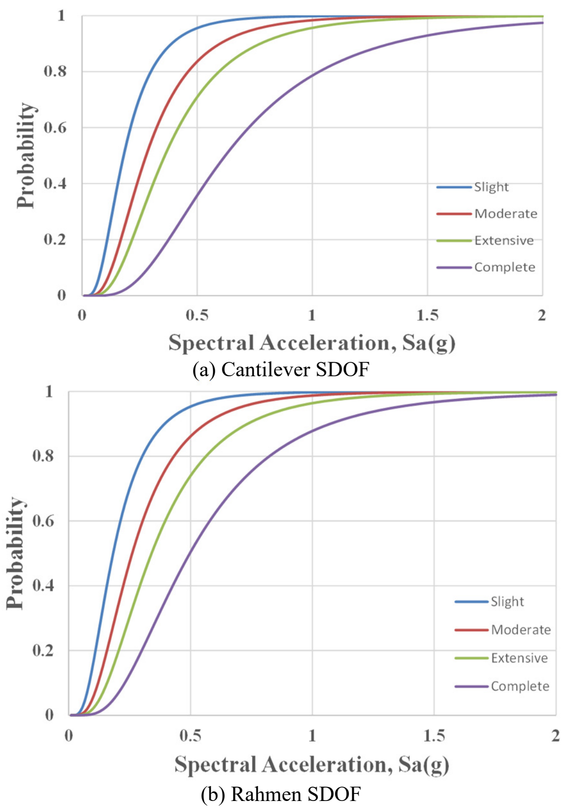


Fig. 8. Seismic fragility curve.

TABLE III. DAMAGE STATE WITH 50% PROBABILITY

Damage State	Cantilever $\hat{C}$	Rahmen $\hat{C}$
Slight	0.18018564	0.18231037
Moderate	0.277634274	0.26063549
Extensive	0.358974883	0.340203585
Complete	0.621511918	0.497132924

In this analysis model, the spectral acceleration values for each damage state at a 50% probability of damage occurrence, denoted as, $\hat{C}$ are shown in Table 3. Using Eq. (13), the earthquake vulnerability curves for each damage state are presented in Fig. 8. Both equivalent single-degree-of-freedom systems exhibit a probability of 90% or higher for the occurrence of Slight, Moderate, and Extensive damage levels at a spectral acceleration of 0.78g. However, the probability of Complete damage occurrence differs: for the cantilever-type equivalent single-degree-of-freedom system, it occurs at 1.35g, while for the frame-type equivalent system, it occurs at 1.08g, resulting in different slopes of the vulnerability curves. This difference is attributed to the distinct P-Δ effects between the cantilever and frame-type systems, which lead to different inter-story displacements at the collapse point of the structure.

V. CONCLUSION

Through resilience-based performance evaluation, the ability of a building to recover after an earthquake disaster can be assessed. As the need to incorporate this method into next-generation performance-based design approaches arises, this study presents the following conclusions regarding the consideration and implementation of resilience-based performance evaluation:

1) For resilience-based performance evaluation, it can be quantified through loss prediction models and recovery time evaluation models. In constructing these predictive models, it is essential to consider not only direct damage but also indirect damage.
2) The evaluation of recovery time for seismic damage to a structure should account for the repair periods of both structural and non-structural components, as well as the downtime, including pre-investigation, reinforcement design periods, and the recovery time for power and water supply facilities. It is necessary to establish a recovery time evaluation process suited to domestic conditions and represent it in a flowchart.
3) To build a loss prediction model, probabilistic seismic vulnerability analysis is needed. However, since it is repetitive and time-consuming, for practical applications, simplified equivalent models should be used to increase efficiency and reduce analysis time.
4) Accordingly, this study proposes equivalent single-degree-of-freedom systems in cantilever and frame forms. The results of the incremental dynamic analysis indicate linear behavior up to the inter-story drift ratio at the collapse state, suggesting the need for calculating a stiffness reduction factor for the hysteretic characteristics of the hinges. Moreover, although the seismic vulnerability analysis results show similar vulnerability curves for the two equivalent systems, the slope differs at the Complete damage state. This indicates the need for improvements to better simulate the behavior of the structure under gravitational loads, including P-Δ effects.
5) Future research will focus on specific studies for the domestic adoption of resilience-based performance evaluation methods. Additionally, plans are in place to refine the equivalent single-degree-of-freedom systems for loss prediction and propose simplified multi-degree-of-freedom systems.

CONFLICT OF INTEREST

The authors declare no conflict of interest.

AUTHOR CONTRIBUTIONS

All authors performed the experiment; Lee contributed to the analysis of experimental data and the writing of the paper; Kim contributed to finishing the paper; all authors had approved the final version.

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REFERENCES

[1] I. Almufti and M. Willford, "REDI rating system: resilience-based earthquake design initiative for the next generation of buildings (Version 1.0)," *ARUP*, 2013.

[2] G. P. Cimellaro, A. M. Reinhorn, and M. Bruneau, "Framework for analytical quantification of disaster resilience," *Engineering Structures*, vol. 32, pp. 3639–3649, 2010.

[3] Risk and Resilience Measurement Committee, "Resilience-based performance: Next generation guidelines for buildings and lifeline standards," *ASCE*, 2019.

[4] G. P. Cimellaro, A. M. Reinhorn, and M. Bruneau, "Introduction to resilience-based design (RBD)," *Geotechnical, Geological, and Earthquake Engineering*, vol. 33, pp. 151–183, 2014.

[5] S. A. Hadigheh, M. N. S. Hadi, and D. M. Leach, "A preliminary case study of resilience and performance of rehabilitated buildings subjected to earthquakes," *Earthquakes and Structures*, vol. 11, no. 6, pp. 967–982, 2016.

[6] Federal Emergency Management Agency (FEMA), *FEMA P-58: Seismic Performance Assessment of Buildings*, vol. 1, 2018.

[7] S. H. Han and H. S. Moon, "Equivalent SDF systems representing steel moment-resisting frames," *Earthquake Engineering Society of Korea*, vol. 12, no. 3, pp. 21–28, 2008.

[8] Architectural Institute of Korea (AIK), *Seismic Building Design Code and Commentary: KDS 41 17 00*, pp. 26–27, 2019.

[9] A. H. Barbet, G. Gutiérrez-Alonso, and D. Arosio, "Seismic damage evaluation in urban areas using the capacity spectrum method: Application to Barcelona," *Soil Dynamics and Earthquake Engineering*, vol. 28, pp. 851–865, 2008.

[10] Federal Emergency Management Agency (FEMA), *FEMA P-695: Quantification of Building Seismic Performance Factors*, 2009.