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Assessment of seismic collapse risk for old reinforced concrete building according to seismic design categories

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ABSTRACT

Reinforced concrete buildings designed before adoption of modern seismic codes are vulnerable to earthquakes. In FEMA P695 (2009), it is suggested that the probability of collapse due to Maximum Considered Earthquake (MCE) ground motions be limited to 10%. The objective of this study is to identify the damage potential of old reinforced concrete building in different seismic categories by evaluating collapse probabilities. FEMA P695 (2009) is used to estimate the collapse probabilities. For this purpose, three story office building designed with gravity loads is used. It is shown that in seismic design category the probability of collapse was less than 10%, and in other design categories the probabilities of collapse were exceeding 10%.

1. INTRODUCTION

The introduction of the seismic design standard in Korean design code was carried out in 1988, but the middle and low-rise buildings were not included in the subject of seismic design. The introduction of seismic design compulsory for the middle and low-rise buildings was a relatively recent revision. In 1988, when seismic design standards were introduced, seismic design was not required for buildings under 5 stories. Subsequently, in 2006 and 2017, it was expanded to perform seismic design on buildings of more than 3 stories and 2 stories, respectively, but majority of low-rise reinforced concrete structures prior to the revision were designed without considering the resistance to earthquake loads (Lee 2018; Lee 2019; Moon 2012).

Due to the recent earthquakes in Gyeongju and Pohang, many buildings in Korea have been damaged, resulting in economic losses. In most cases, these economic losses and damages occurred in low-rise old reinforced concrete structures, which led to the need for evaluating seismic performance and establishing seismic countermeasures for domestic buildings. However, many buildings in Korea have not been subjected to seismic design, and it is necessary to ensure the safety of the structure through performance evaluation of the seismic performance of old reinforced

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concrete structure (Lee 2019).

The poor seismic performance of old reinforced concrete frames designed without considering earthquake loads can be examined through earthquakes in the United States and Japan (Galanis 2015; Han 2004). Since the old reinforced concrete frame is designed using only gravity load, it exhibits non-ductile behavior such as shear failure with respect to lateral forces due to inadequate rebar details on the column (Galanis 2015; Lee 2018; Shin 2018).

This study aims to evaluate the safety of low-rise old reinforced concrete structures according to seismic zones based on seismic performance evaluation. For this, selection of old reinforced concrete frame without seismic design, establishment of numerical analysis model, and seismic performance evaluation according to FEMA P695 (2009) were performed based on previous studies. To perform quantitative evaluation of seismic performance of old reinforced concrete frame according to seismic zones, the secure of target seismic performance of the old reinforced concrete frame located in Seismic design category A, B, C, and D region proposed in ASCE 7-16 (2016) was evaluated.

2. BUILDING SELECTION

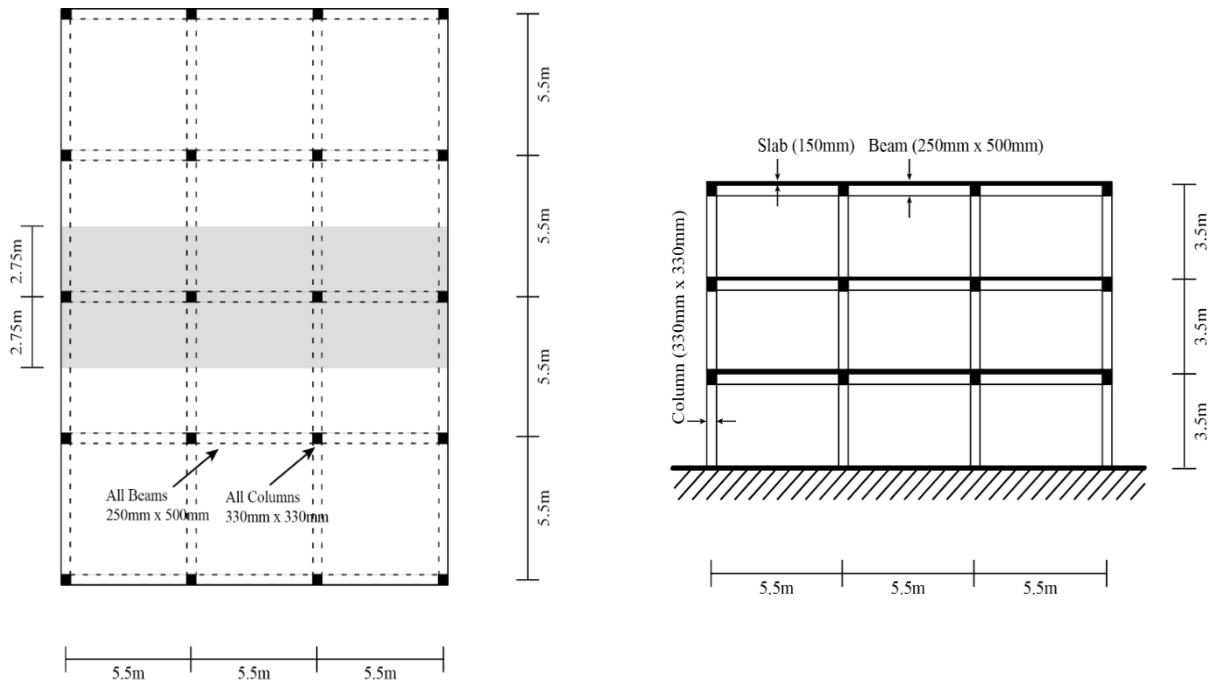
In order to select an old reinforced concrete building suitable for the purpose of this building, physical properties of old reinforced buildings and existing buildings by age, and the target building was selected based on this. Old reinforced concrete buildings show poor behavior due to improper column details because design for lateral loads has not been performed (Shin 2018). Lee (2018) summarized the properties of a column located in old reinforced concrete building as follows: (1) $1.23 \le a/d \le 3.76$, (2) $0.11 \le s/\mathrm{d} \le 1.16$, (3) $0 \le \nu \le 0.62$, (4) $21.0 \le f_c{'} \le 43.6$, (5) $318 \le f_{yl} \le 496$, (6) $249 \le f_{yt} \le 476$, (7) $0.013 \le \rho_l \le 0.033$, (8) $0.001 \le \rho_t \le 0.015$. Here, a, d, and s respectively denote shear span, depth, and transverse reinforcement spacing, and ν, axial force ratio, can be expressed as $\nu = P/(A_g\, f_c{'})$. In addition, f_c', f_{yl}, and f_{yt} mean strength of concrete (MPa), yield strength of longitudinal and transverse reinforcement (MPa), and ρ_l and ρ_t represent ratio of longitudinal and transverse reinforcement, respectively. In this study, a reinforced concrete building with column details within this range was selected as the target building.

In addition, as mentioned above, old reinforced buildings are mostly composed of buildings designed before 1988 or 6 or less floor buildings (before 2005) designed prior to the improvement of seismic design target. Therefore, in this study, the current status of concrete buildings (Hong 2015) was consulted to select target building. As shown in Table 1, the strength of concrete and rebar materials according to the completion year was invested. According to the survey, it can be seen that the strength of concrete ranges from 15 to $24\,MPa$, and the strength of the rebar ranges from 235 to $500\,MPa$, for the buildings competed before establishment of the seismic design provisions. Therefore, in this study, as a target building, a building having a column within the range of physical properties of old reinforced building component and simultaneously having material strength within a range shown in Table 1 was selected.

Table 1 the strength of materials according to the completion year

completion year	Concrete		Rebar	
	Min (MPa)	Max (MPa)	Min (MPa)	Max (MPa)
1960~1969	21	21	400	400
1970~1979	15	24	235	500
1980~1989	18	21	235	300
1990~1999	18	24	240	400
2000~2010	21	27	300	400

Based on this procedure, in this study, a three-story reinforced concrete ordinary moment frame (Han 2004), which was designed only considering gravity load(1.4D+1.7L), was selected as a target building. D and L are dead load and live load, respectively. The reasons for selecting the three-story reinforced concrete ordinary moment frame (Han 2004) as the target building are as follows: (1) seismic design of the target building has not been carried out because the design was performed by excluding the seismic loads according to ACI 318-99 (1999), (2) the material properties and sections considered in the design process of the target building are similar to the old reinforced buildings shown in previous studies.



(a) plan (b) elevation

Fig. 1 Plan and elevation of three-story reinforced concrete ordinary moment frame

Fig. 1 shows dimensions of the building. The building has 3 bays in the E-W direction and 4 bays in the N-S direction. The height of each floor is $3.5\ m$, the width of each bay is $5.5\ m$, and the total height of the building is $10.5\ m$. The beam and column details were designed following the design procedure for the ordinary frame in ACI 318-99 (1999), and slab was designed using the direct design method of ACI 318-99 (1999). The strength of concrete was $24\ MPa$, and the strength of longitudinal and transverse reinforcement were $294MPa$ and $392MPa$. The dead load and live load of each floor is $5.2\ kN/m^2$ and $2.45\ kN/m^2$, and the period is 1.23 sec.

3. ANALYTICAL MODELING

In this study, in order to evaluate the seismic performance of the target building, an analytical model was constructed along E-W direction of three bay building, shaded from Fig. 1(a). The analytical model was constructed using the OpenSees software (McKenna 2011) with nonlinear model to simulate the behavior after yielding. The structural components of the moment frame were constructed with concentrated plastic hinge model which simulates the nonlinearity of components based on the behavior of the plastic hinge at both ends. The plastic hinges were constructed with IMK model (Ibarra, 2005), and for each property parameter that determines the strength, stiffness, deformation and dissipation of component, parameters proposed by Haselton (2016) were used.

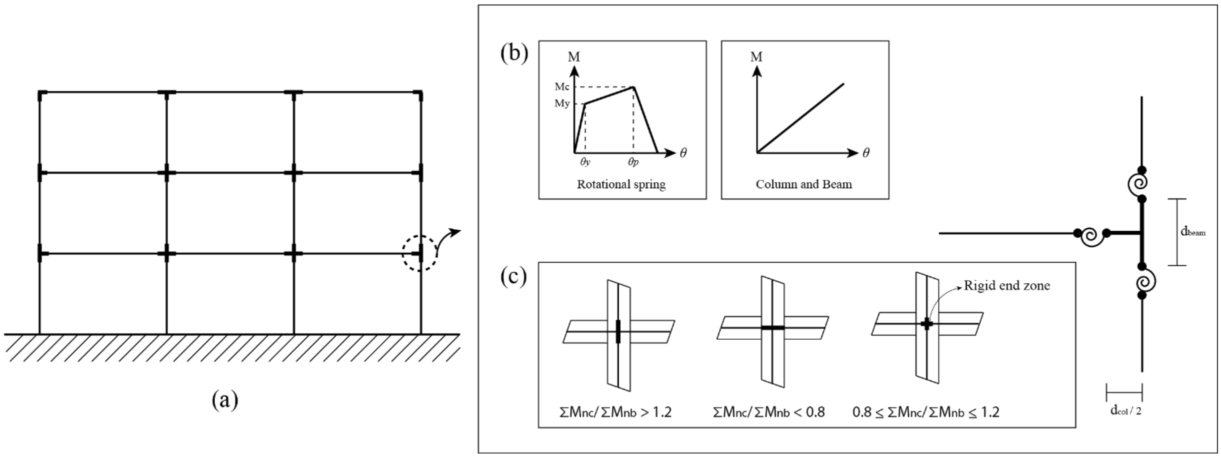


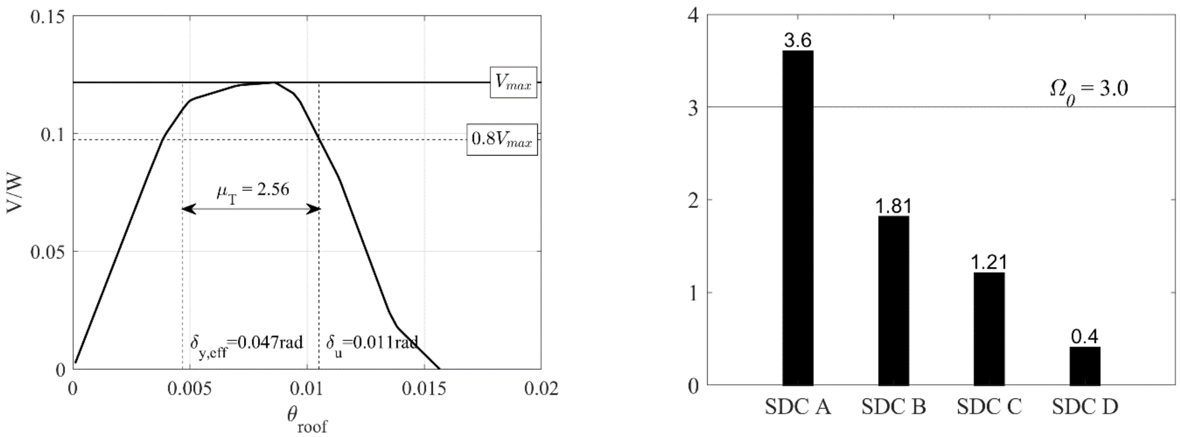
Fig. 2 Analytical model of reinforced concrete ordinary moment frame

Elwood (2007) proposed joint rigid modeling to consider the shear deformation at the joint according to the ratio of the moment of the column and beam as shown in Fig. 2(c). In addition, for beams, the stiffness and strength of the component change due to slab effect. To perform this contribution, the effective width was defined according to NIST GCR 17-917-47 (2017), and the beam was defined to have strength for effective width equal to 1/5 of beam span.

ATC 72-1 (2009) proposed a damping ratio of 2-3% for reinforced concrete structures. Therefore, in this study, an analytical model was constructed assuming that the target building had damping ratio of 2%, and Rayleigh damping was applied to the damping ratio in consideration of the first mode and third mode. In addition, in order to prevent the unrealistic damping force that may occur in the concentrated plastic hinge model, damping matrix is constructed using mass matrix by lumped mass and stiffness matrix by considering only elastic element (Zareian 2010). Additionally, geometric transformation was applied to each column component to reflect $P-\Delta$ effect by gravity, and the effective seismic weight was applied as 1.05D+0.25L as suggested in FEMA P695 (2009).

4. COLLAPSE PROBABILITY

In order to evaluate the lateral force performance of the target building, nonlinear static analysis (pushover) was performed according to the procedure proposed in FEMA P695 (2009). FEMA P695 (2009) requires the use of lateral force distribution according to 1st mode shape to perform pushover analysis. Fig. 3 shows the pushover curve and overstrength factor (Ω) of model building. In the Fig. 3(a), the horizontal axis and vertical axis represent the roof drift ratio (θ_{roof}) which is roof displacement divided by height of the roof and the normalized lateral force (V/W) which is base shear force (V) divided by weigh of the structure (W). As shown in Fig. 3(a), despite the fact that the target building is a low-rise building with relatively small $\mathrm{P}-\Delta$ effect caused by small gravity loads, a sudden drop of strength occurred after the maximum base shear force. The normalized maximum base shear force (V_{max}/W) and maximum roof drift ratio of the target building were evaluated to be 0.121 and 0.011 rad. Here, θ_u was defined as the roof drift ratio at the point of 20% strength loss according to FEMA P695 (2009).

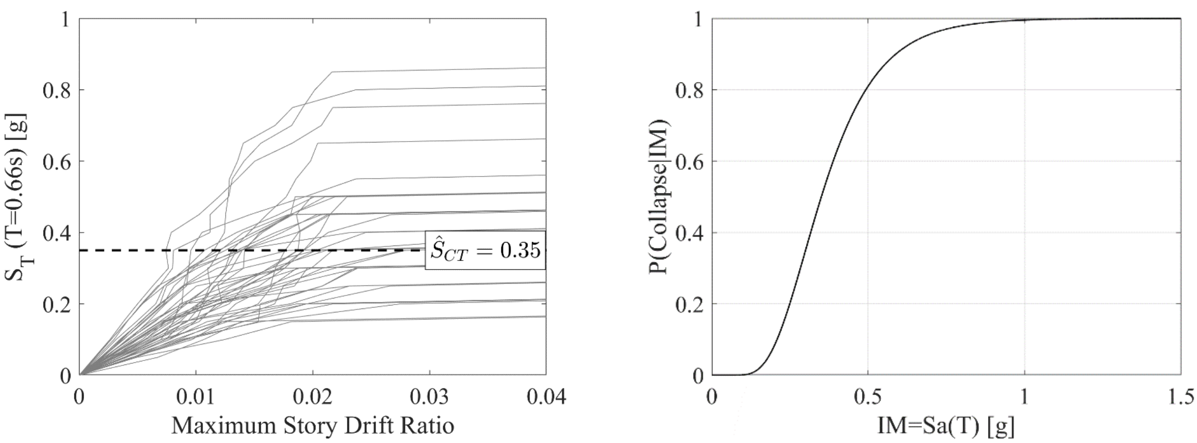


(a) pushover curve (b) overstrength factor

Fig. 3 Result of pushover analysis

The overstrength factor (Ω) according to seismic design category (SDC) of the target building was evaluated to have values of 3.6, 1.8, 1.2, and 0.4 in $\mathrm{A_{max}}$, $\mathrm{B_{max}}$, $\mathrm{C_{max}}$, and $\mathrm{D_{max}}$ regions as shown in Fig. 3(b). Except for SDC $\mathrm{A_{max}}$ region, it was evaluated to have Ω less than 3.0, which is the system overstrength factor (Ω_0) proposed for target building in current seismic standards. The SDC $\mathrm{A_{max}}$ region has a design spectrum acceleration at short- (S_{DS}) and 1-second period (S_{D1}) of 0.167g and 0.067g, which is smaller than the design spectrum acceleration used in low-seismicity region according to KDS 41 17 00 (2019). Therefore, in the case of old reinforced concrete building, the target seismic performance proposed in KDS 41 17 00 (2019) was not secured, and the stability of the building cannot be guaranteed.

Incremental dynamic analysis (IDA; Vamvatsikos 2002) of the target building was additionally performed according to the procedure proposed in FEMA P695 (2009) to evaluate the resistance performance of the target building to dynamic load. 44 Far-field ground motions sets proposed in FEMA P695 (2009) were used to perform the IDA, and the IDA curve and collapse fragility curve are shown in Figure 4. The horizontal and vertical axes in Fig. 4(a) represent the maximum inter-story drift ratio (θ_{max}) and the intensity of ground motion for the period T, as a result of dynamic analysis. Here, the period T represents the upper limit value ($C_u T_a$) of the period according to the current standard. When the maximum inter-story drift ratio (θ_{max}) due to ground motion occurs more than 10%, the analysis model does not converge due to the extreme nonlinearity, or each plastic hinge experiences a deformation of 1.2 times or more of the fracture, it is defined that collapse occurs.



(a) IDA curve (b) collapse fragility curve

Fig. 4 Result of IDA

As a result of performing IDA, the target structure behaves elastically when the maximum inter-story drift ratio (θ_{max}) is less than 0.6%, and collapse rapidly when it is more than 2%. In case of the target structure, it was evaluated that when the maximum inter-story drift ratio (θ_{max}) of 2% or more occurs, severe nonlinearity or collapse due to fracture of the plastic hinge occurs. The median collapse intensity ($\hat{S}_{CT}$) of target structure was evaluated to be 0.35g. The collapse fragility curve for evaluating the distribution of collapse intensity for each ground motion is shown in Fig. 4(b).

In order to evaluate whether the target structure secured the target seismic performance according to SDC, the conditional collapse probability according to FEMA P695 (2009) was calculated. ASCE 7-16 (2016) requires the conditional collapse probability given maximum considered earthquake (MCE) to be less than 10% for structures in Risk category I/II. The conditional collapse probability ($\mathrm{P_C}$) given MCE can be calculated according to Eq. (1).

$$\mathrm{P_c} = p(Collapse|S_{MT}) = \Phi\left(\frac{\ln(S_{MT}) - \ln(\hat{S}_{CT} \times SSF)}{\beta_{TOT}}\right), \tag{1}$$

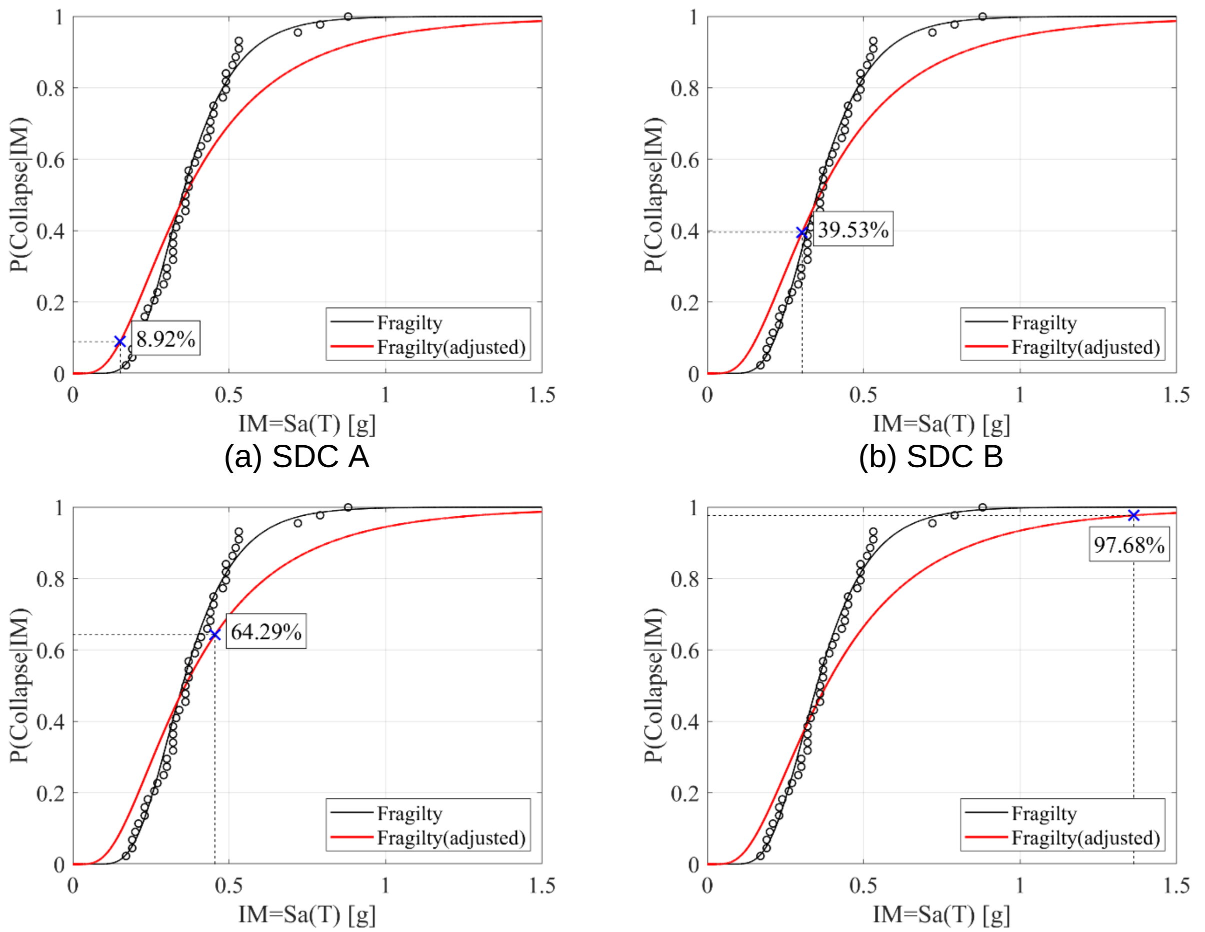
In Eq. (1), the spectral shape factor (SSF) is a coefficient that adjusts the collapse probability by adjusting the spectral shape of the ground motions with low incidences (Baker and Cornell, 2006), β_{TOT} means uncertainty that can occur in simulation. β_{TOT} is defined by record-to-record (β_{RTR}), design requirement-related (β_{DR}), test data-related (β_{TD}), and modeling (β_{MDL}) uncertainty, and it can be calculated according to Eq. (2). In this study, β_{RTR} was calculated according to FEMA P695 (2009), and β_{DR}, β_{TD}, and β_{MDL} were defined as 0.35, 0.2, and 0.35.

$$\beta_{\mathrm{TOT}} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}, \tag{2}$$

Table 2. Result parameters of seismic performance evaluation according FEMA P695

	Overstrength and collapse margin parameters						Acceptance check	
SDC	$\mathrm{S_{MT}}$	Ω	μ_T	SSF	$\hat{S}_{CT}$	β_TOT	$\mathrm{P(C\|S_{MT})}$	$\mathrm{P_C} \leq 0.1$
A	0.156	3.599	2.56	1.027	0.35	0.642	0.089	Pass
B	0.303	1.813	2.56	1.027	0.35	0.642	0.394	Fail
C	0.455	1.206	2.56	1.027	0.35	0.642	0.643	Fail
D	1.364	0.402	2.56	1.086	0.35	0.642	0.977	Fail

Table 2 summarizes each parameter calculated according to FEMA P695 (2009) methodology. Also, the collapse fragility curve based on IDA shown in Fig. 4(b) and the collapse fragility curve adjusted using SSF and β_TOT are shown in Fig. 5 as black and red solid lines. The black circle in Fig. 5 represents the collapse intensity of each ground motion. For the target structure, adjustment by SSF does not significantly affect the results (Table 2). In addition, in the case of the target structure, β_TOT was evaluated as a large value, and uncertainty has a great influence on the collapse probability because the design criteria and the quality rating of the modeling were defined as low levels. The target structure has $\mathrm{P_c}$ of 8.9%, 39.53%, 64.3%, and 97.7% for SDC A, B, C, and D regions. This tends to be similar to the result of the pushover analysis mentioned above, which means that the target structure does not have the target seismic performance on most of the region in Korea.



(a) SDC A (b) SDC B

(c) SDC C (d) SDC D

Fig. 5 Adjusted collapse fragility curve according to SDC

As a result of performing the collapse evaluation of the old reinforced concrete building in the method suggested by FEMA P695 (2009) in the SDC A, B, C, and D region, it was evaluated that the target building in the SDC A region had a target seismic performance, but it was evaluated to have high collapse probability in other regions. The majority of seismic design category in Korea corresponds to SDC B and C. Based on this, it can be confirmed that repair and reinforcement of old reinforced concrete buildings are required. However, it should be noted that this is a study conducted on a three-story building, which is the majority of old reinforced concrete building in Korea, an evaluation for accurate seismic performance should be performed through additional research.

5. CONCLUSIONS

The purpose of this study was to evaluate the stability of target building through the seismic performance evaluation of old reinforced concrete building. For quantitative evaluation, the conditional collapse probability for the maximum consideration earthquake was evaluated based on FEMA P695 (2009), and the seismic performance of the target building according to region was evaluated through comparison with the target seismic performance of ASCE 7-16 (2016). The target building was selected from Han (2004) based on the material and structural properties of old reinforced building. Seismic load was not considered in the design, and all components were located within the properties of old reinforced building. As a result of performing pushover analysis, it is evaluated that the old reinforced concrete building has an overstrength factor (Ω) less than the system overstrength factor (Ω_0) required for the seismic resisting frame when it is located in the region above SDC B. This indicates that old reinforced concrete building exhibits similar performance to the moment frame with seismic design in the SDC A region, but may exhibit a weak response in the region above SDC B. In order to evaluate the performance of the old reinforced building against dynamic load, seismic performance evaluation according to FEMA P695 was performed. In order to quantitatively represent the target seismic performance, the target reliability for risk category I/II structures from ASCE 7-16 (2016) was defined as the target seismic performance. As a result of seismic performance evaluation, old reinforced concrete buildings were evaluated as failing to seismic performance in regions other than SDC A. SDC A is a weak region, and it is required to design for larger earthquake loads in most regions of Korea. Therefore, in the case of old reinforced concrete building located in Korea, the target seismic performance required by KDS 41 17 00 (2019) is not secured, and it is considered that seismic reinforcement or reconstruction is required.

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