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Evaluation of Expected Annual Loss Probabilities of RC Moment Resisting Frames

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***Abstract*—**Building loss estimation methodologies which have been advanced considerably in recent decades are usually used to estimate socio and economic impacts resulting from seismic structural damage. In accordance with these methods, this paper presents the evaluation of an annual loss probability of a reinforced concrete moment resisting frame designed according to Korean Building Code. The annual loss probability is defined by (1) a fragility curve obtained from a capacity spectrum method which is similar to a method adopted from HAZUS, and (2) a seismic hazard curve derived from annual frequencies of exceedance per peak ground acceleration. Seismic fragilities are computed to calculate the annual loss probability of a certain structure using functions depending on structural capacity, seismic demand, structural response and the probability of exceeding damage state thresholds. This study carried out a nonlinear static analysis to obtain the capacity of a RC moment resisting frame selected as a prototype building. The analysis results show that the probability of being extensive structural damage in the prototype building is expected to 0.01% in a year.

***Keywords*—**Expected annual loss, Loss estimation, RC structure, Fragility analysis.

I. Introduction

SEISMIC fragility studies of building structures have been increasingly carried out to estimate the earthquake losses resulting from their structural damage. The seismic loss estimation requires to identify the seismic vulnerability of a building structure and to describe resulting structural performance quantitatively.

There are numerous low-rise reinforced concrete (RC) buildings in Korea. Such low-rise RC buildings could be vulnerable to horizontal ground accelerations and suffered serious structural damages, which causes significant national-wide seismic losses. This paper has evaluated a probability of expected annual loss for low-rise RC moment resisting framed building. The annual seismic loss probability is calculated by the fragility curve of a building and seismic hazard in a certain area.

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II. Description of Prototype Building

A. Design of Example Building and Analytical Model

A prototype structure, a 5-story reinforced concrete framed building shown in Fig. 1 is chosen to expect the annual loss probability of typical RC intermediate moment-resisting frames (IMRF). The plan of prototype building consists of three 10 m bays and the height of each story is 3.6 m. The building is seismically designed according to a current Korean building code, KBC 2009 [1]. For gravity loads, uniform dead loads of 5 kPa and live loads of 4 kPa are applied to each floor. It is assumed that the building is located in Seoul, Korea of which the site class is assigned to SD. The design spectral response acceleration parameters at short period, S_{DS} and at 1s period, S_{D1} are, respectively, 0.50, and 0.29. The response amplification factor, R of the prototype RC IMRF is 5.0. It is found from a preliminary eigenvalue analysis that the first mode period of the building, T_1 is 1.29 second. Using these values, the design base shear, V_d is calculated as 500.6 kN that is vertically distributed according to a rule prescribed in KBC 2009 [1] which is similar to the American seismic design code, ASCE 7 [2]. Table I summarises structural dimensions and reinforcement arrangement of column and beam members of which the locations are shown in Fig. 1. The compressive strength of concrete is 28 MPa and the yield strength of a steel rebar is 400 MPa.

The prototype two-dimension (2D) RC IMRF is modeled using a non-linear analysis simulation software, Ruaumoko 2D [3]. The strength and stiffness degradation according to the corresponding ductility are adopted using a modified Takeda hysteresis rule. Reinforced concrete beam-column joints are modeled according to a hysteretic rule suggested by [4] which has been long known to properly capture the shear behavior of RC beam-column joints.

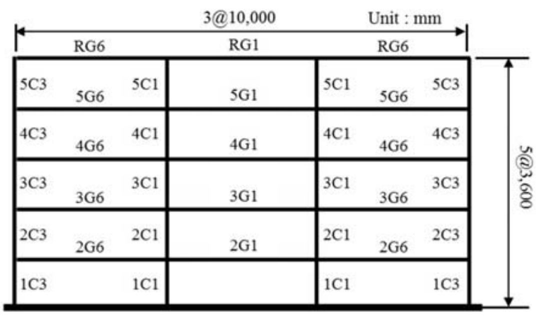

Fig. 1 Elevation of the prototype building

TABLE I
STRUCTURAL MEMBER SIZING AND REINFORCEMENT ARRANGEMENTS

Member		Story	Section	Rebar	Stirrup
Columns	C1 (interior)	4-5	550x550	6-2-D25	2-D10@400
		2-3	600x600	8-3-D25	2-D10@400
		1	700x700	10-4-D25	2-D10@400
	C3 (exterior)	4-5	500x500	6-2-D25	2-D10@210
		2-3	550x550	6-3-D25	2-D10@240
		1	550x550	6-3-D25	2-D10@400
Beams	G1 (interior)	4-R	400x700	6-D22, 3-D22	2-D10@300
		2-3	400x800	7-D22, 3-D22	2-D10@350
	G6 (exterior)	4-R	400x700	6-D22, 3-D22	2-D10@300
		2-3	400x800	7-D22, 3-D22	2-D10@350

B. Seismic Hazard in Seoul, Korea

The prototype building site, Seoul is a seismologically low or moderate seismicity zone. The probabilistic seismic hazard analysis uses the uniform hazard spectra developed by Ministry of Construction and Transportation in Korea [5]. The scenarios have 50, 100, 200, 500, 1000, 2400, and 4800 years return periods. The seismic hazard in Seoul is shown in Fig. 2, derived from mean annual frequency of exceedance which is evaluated by ground motion intensity.

III. EVALUATION OF ANNUAL LOSS PROBABILITY

A. Results of Fragility Analysis with CSM

The fragility analysis methodology suggested by HAZUS [6] has been widely accepted to investigate the vulnerability of existing buildings. In this study, a fragility curve is obtained from the capacity spectrum method (CSM) based on the results of nonlinear static analysis, which follows the methodology of HAZUS. The capacity spectrum for the prototype building is presented in Fig. 3. Using the yield capacity (D_y, A_y) and ultimate capacity (D_u, A_u) obtained from a nonlinear static analysis of the prototype building, the capacity spectrum is, as shown in Fig. 3, idealized and is presented in the figure as a dashed line passing through the yield and ultimate capacity points. The yield capacity is defined as the onset where the first noticeable stiffness reduction is observed. After the ultimate capacity representing the maximum strength of the prototype building, it is assumed that the global structural system has reached a fully plastic state without strength degradation. This assumption significantly decreases the computation time in finding a performance point for a given seismic hazard level.

The fragility curve describes probabilities of exceeding performance criteria, so-called damage state, *ds*, at different levels of seismic input intensity, and expresses in a form of a log-normal cumulative distribution function.

The conditional probability of being in or exceeding *ds* given spectral displacement, S_d, is defined from:

$$\mathrm{P}(\mathrm{ds}|S_d) = \Phi\left[\frac{1}{\beta_{ds}} ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \quad (1)$$

$$\beta_{ds} = \sqrt{(CONV[\beta_C, \beta_D])^2 + (\beta_M)^2} \quad (2)$$

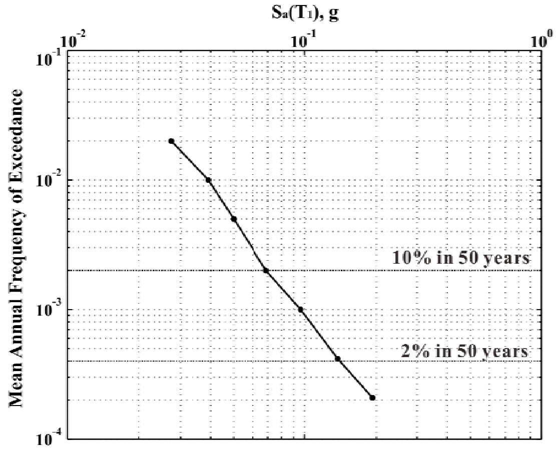

Fig. 2 Hazard curve in Seoul, Korea

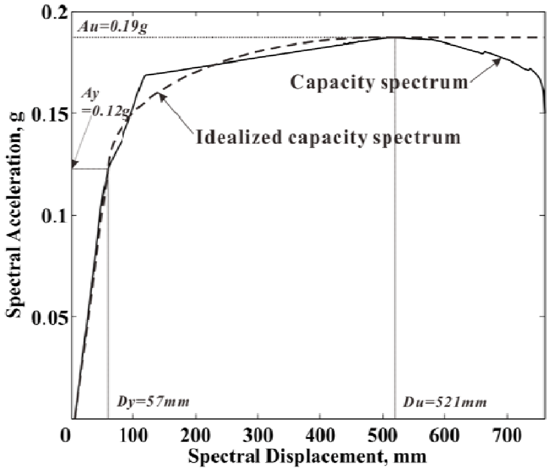

Fig. 3 Capacity spectrum of the prototype building

where Φ is the standard lognormal cumulative distribution function, $\bar{S}_{d,ds}$ is the median value of S_d at the threshold of *ds*, β_{ds} is the log-standard deviation of S_d for *ds*. Specifically, the value of β_{ds} is evaluated using a SRSS method of uncertainties consisting of the variability of β_M in the damage state threshold, the uncertainty of β_C in the structural capacity and the uncertainty of β_D in seismic demand. The term $CONV[\beta_C, \beta_D]$ in (2) represents the combined uncertainty of capacity and demand of the structure. This uncertainty is obtained through a convolution process, which is produced by intersection points, namely performance points, of the capacity spectrum and demand spectrum [7]. In this paper, the performance points are obtained by the intersection points of the sampled capacity and demand spectra in tandem with a Monte Carlo simulation (MCS) based on a Latin hypercube sampling (LHS).

The thresholds of damage states (Slight, Moderate, Extensive, and Complete) as indicated in NEMA report [8] are calibrated by damage state criteria (Operational, Immediate Occupancy, Life Safety, and Collapse Prevention) in FEMA 356 [9]. The damage state thresholds of a RC moment resisting frame are summarized in Table II. The probability of the structural responses exceeding damage state threshold is used to estimate the term of $CONV[\beta_C, \beta_D]$, and the result of β_{ds} in Table II is calculated by SRSS of the term and the value of β_M(=0.4). The results of the fragility analysis for the prototype RC moment frame are shown in Fig. 4.

B. Expected Annual Loss Probability

The ground motion intensity in a certain area can be quantitatively expressed as mean annual frequency of exceedance which is described in the hazard curve. The probability of expected annual loss is estimated using the hazard curve, $\lambda(S_d)$ and the fragility curve of the building modeled by a lognormal probability density function, $f(ds|S_d)$ which is defined on [10]:

$$\mathrm{P(loss)} = \int f(ds|S_d)\lambda(S_d)\, dS_d \quad (3)$$

The standard elastic acceleration response spectrum can be converted to an Acceleration-Displacement Response Spectrum (ADRS) format [11]. The expected annual loss probability for the RC moment framed building is given in Fig. 5 and Table III. As shown in the table, the expected annual loss probability of exceeding each damage state is less than 0.1%. The expected annual probability of exceeding the extensive damage state is 0.01%. Table III also lists the probabilities of loss in 40 years that is assumed as the life duration of typical RC buildings. The expected probability of exceeding the Extensive damage state is 0.33% in 40 years.

IV. Conclusion

This paper describes the seismic hazard in Seoul, Korea and evaluates the probability of expected annual loss for low-rise reinforced concrete moment framed buildings. The fragility curve is obtained from the Capacity Spectrum Method (CSM) through the results of nonlinear static analysis, and thereby the annual loss probability is derived from the fragility analysis and the hazard curve.

From the evaluation, the probability of being Extensive structural damage (i.e. Life Safety level in FEMA 356) of prototype building is expected to 0.01% in a year and 0.33% in 40 years. This study considers only seismic losses resulting from damage in structural members. It is noted that the most extent of seismic risk on the building is caused by the performance of nonstructural components (e.g. mechanical, electrical, plumbing, and architectural components) [12]. Due to lack of knowledge and practice on the seismic performance of nonstructural components, this study does not properly evaluate seismic losses caused from nonstructural and facility damage. Therefore, in-depth researches and experiments on nonstructural and facility seismic damage will be required for accurate assessment of the seismic risk.

TABLE II
MEDIAN AND LOG-STANDARD DEVIATION OF PROTOTYPE BUILDING FRAGILITY CURVE

	Slight	Moderate	Extensive	Complete
$\bar{S}_{d,ds}$, mm	57.3	91.6	183.3	366.5
β_{ds}	0.48	0.53	0.64	0.69

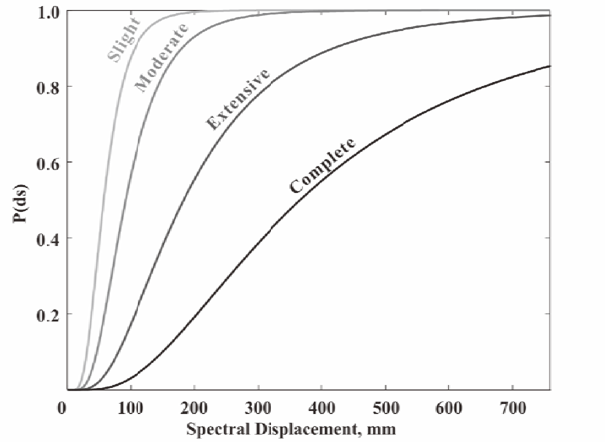

Fig. 4 Results of fragility analysis for the prototype building

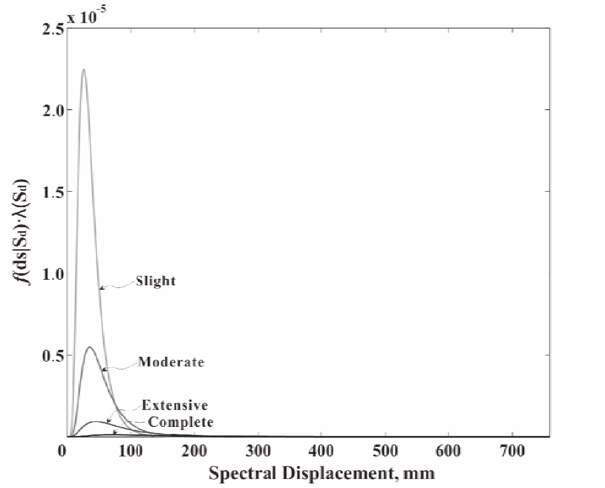

Fig. 5 Estimation of annual loss probability

TABLE III
EXPECTED ANNUAL LOSS PROBABILITIES OF EXCEEDANCE FOR PROTOTYPE BUILDING

Loss probability, %	Slight	Moderate	Extensive	Complete
In a year	0.08	0.03	0.01	0.002
In 40 years	3.13	1.18	0.33	0.08

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