ECONOMIC SEISMIC LOSS ASSESSMENT OF RC SCHOOL BUILDINGS IN SOUTH KOREA


(1) Ph.D Candidate, Department of Architecture and Architectural Engineering, Yonsei University, Seoul 03722, Republic of Korea, insub@yonsei.ac.kr
(2) Associate Professor, Department of Architecture and Architectural Engineering, Yonsei University, Seoul 03722, Republic of Korea, junhkim@yonsei.ac.kr
(3) Ph.D Candidate, Department of Architecture and Architectural Engineering, Yonsei University, Seoul 03722, Republic of Korea, insub@yonsei.ac.kr
(4) M.S Student, Department of Architecture and Architectural Engineering, Yonsei University, Seoul 03722, Republic of Korea, kurtsohn@yonsei.ac.kr

Abstract

Economic seismic loss assessment for evaluating seismic risk is becoming important in South Korea, since earthquake damages had been occurred in building structures caused by major two earthquakes (2016 Gyeongju earthquake, ML 5.8; 2017 Pohang earthquake, ML 5.4). The main purpose of this study is to develop a methodology to evaluate customized economic seismic loss in South Korea. RC school buildings with masonry infills are selected as the case study model, because the past two earthquakes caused large economic loss in the school buildings than in other buildings. The FE model was presented to simulate the short column effect that causes premature failure of the columns by concentrating on shear stress. The seismic fragility curves for the structural and nonstructural components were derived from nonlinear time history analysis. The repair costs of the structural and nonstructural components were defined to produce a customized economic seismic loss function through statistical data on the construction cost of RC school buildings. Using the seismic fragility curves and the repair costs, this study developed a customized economic seismic loss function. In addition, loss-based seismic performance criteria were established to directly correlate the current code-defined (i.e. displacement-based) seismic performance criteria. Then, the loss-based seismic performance of RC school buildings in Pohang city were estimated. The proposed methodology can provide decision-making data for evaluating seismic performance based on seismic loss and has the potential to extend to loss assessment of other categories of buildings.

Keywords: economic seismic loss; RC school buildings; seismic resilience; loss estimation
1. Introduction

It is practically known from past earthquakes that earthquake disasters cause many economic losses in building structures. Seismic risk assessment of the building structures is essential to secure seismic resilience of global or local communities in the future by predicting the seismic losses induced by the earthquake disasters. The seismic risk assessment of the building structures can be evaluated on the basis of the seismic loss assessment of the building structures, which are expected to be caused by the earthquake disasters.

The performance-based seismic design methods [1, 2], developed and started to be used in the mid-1990s, divides the performance of the building structures into four categories (operational, immediately occupancy, life safety, and collapse prevention) based on structural damages. Although the performance level of the building structures was estimated probabilistic methods by fragility analysis for structural components [3], it is difficult to directly link the damage probability and the seismic losses of the building structures. In fact, according to the Earthquake Damage Survey report [4, 5], it is found that the damage to nonstructural components accounts for more than the direct economic loss of buildings rather than structural damages. Therefore, in order to derive decision-making corresponding to the observed loss, it is necessary to consider the direct economic loss comprised of the structural and nonstructural components [6].

In South Korea, two major earthquakes (5.8M<sub>L</sub> Gyeongju earthquake in 2016 and 5.4M<sub>L</sub> Pohang earthquake in 2017) have increased interest in ensuring the safety of the building structures against the earthquake disasters. The survey of the building structures damaged by the two earthquakes revealed that school building structures belonging to the essential facility were vulnerable to the earthquakes [7, 8]. The building structures that have suffered significant damages from the two earthquakes have almost no seismic design applied, and generally the building structures with no seismic design are known to be vulnerable to the earthquakes due to premature shear failure of column. In other words, it is important to evaluate seismic losses for school buildings that are not seismically designed for future seismic events.

Seismic vulnerability function (i.e. seismic loss function) can be used as a tool to evaluate the monetary losses resulted from the physical damages. The seismic losses of nonstructural components account for more than 80% of the total seismic losses of building structures in the case of commercial building structures [9]. Therefore, it is necessary to evaluate the seismic fragility of nonstructural components to assess the seismic losses. Analyzing the existing research [10-13] to estimate the seismic fragility for various nonstructural components, most research have been conducted to define the engineering demand parameters (EDPs) and limit states of nonstructural components through static or dynamic experimental test. These existing research provide the analytical methods and experimental data for deriving the seismic fragility functions of the structural and nonstructural components, but there are some limitations to overcome: (i) it is necessary to define the seismic fragility functions that reflect the structural characteristics of school building in South Korea; (ii) integrated seismic fragility functions are required for structural and nonstructural components; (iii) the seismic fragility functions for each components can be defined for each story level, not for the building level, to evaluate precise seismic damage evaluation; (iv) the seismic vulnerability function for individual building is required to estimate the earthquake-induced loss.

Therefore, this study aimed to develop a framework for evaluating the seismic vulnerability function considering the fragility function of structural and nonstructural regarding each story level. The school building structures are selected the case study model. Nonlinear model for the column elements was presented to simulate the flexure-shear behavior of the building structure and was verified through comparison with the existing experimental data. The probabilistic seismic demand models for each story were established by using the maximum interstory drift ratio and the maximum peak floor acceleration through time history analysis. The seismic fragility functions for the case study model were estimated using the seismic demand model for each component. The seismic vulnerability function for the prototype model was defined using the seismic fragility functions obtained by simulation method. Also, loss-based seismic performance criteria were defined by correlation analysis with the current code-defined (i.e. displacement-based) seismic performance criteria to evaluate the loss-based seismic performance of the building structures.
2. Prototype Model Description

The seismic losses due to the 2017 Pohang earthquake were analyzed to select the prototype model that is vulnerable to earthquakes. Since public buildings are classified as essential facilities, which should be secured structural safety during disasters such as earthquakes, the seismic losses of public facilities were analyzed as shown in Table 1. According to the analysis of the loss of public facilities in the Pohang earthquake in 2017, school buildings account for about 48.2% of the total public loss. Securing seismic performance is important because school buildings are used as shelters during earthquakes. However, school buildings located the region are vulnerable to earthquake disaster. So, the school building structures were selected as the case study building structures in this study.

Table 1 – Seismic loss data of public facilities induced by Pohang earthquake (source from [14])

<table>
<thead>
<tr>
<th>Division</th>
<th>Seismic Loss</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infrastructures</td>
<td>4,993,712,000</td>
<td>18.6</td>
</tr>
<tr>
<td>School buildings</td>
<td>12,932,970,000</td>
<td>48.2</td>
</tr>
<tr>
<td>Hospital buildings</td>
<td>2,133,661,000</td>
<td>7.9</td>
</tr>
<tr>
<td>ETC†</td>
<td>6,802,731,000</td>
<td>25.3</td>
</tr>
</tbody>
</table>

†ETC includes military and small facilities.

Since the school buildings were classified as essential facility and applied seismic design code after revision of seismic design code in 2005, most school buildings located South Korea do not have sufficient seismic performance. In order to determine the prototype model of the school building, reference (school building standard blueprint [15]) was investigated. A three-story and seven-span RC moment frame with unreinforced masonry infills was selected as the prototype model, as shown in Fig. 1. The properties of the concrete and steel materials were determined considering the construction year (1973-1988) of the real building structures as specified in a guideline for seismic performance evaluation [16]. Concrete compressive strength is $f'_c = 15$MPa and steel yielding strength is $f_y = 240$MPa. The dead load is 3.7kN/m² and the live load is 3.0kN/m².
3. Numerical Model of the School Building

3.1. General

The nonlinear modeling for structural elements was performed to simulate the time history analysis of the case study building structure. OpenSees [17], finite element simulation program, was used to perform nonlinear modeling and time history analysis of the prototype model. School buildings are more likely to cause premature shear failure due to the short column effect resulted from the masonry infills between the columns.

The frame elements are modeled to simulate the flexure-shear behavior of the column elements as shown in Fig. 2. One of the typical structural characteristic of school building is that the masonry infills between columns classified as nonstructural components affects the structural behavior. Therefore, even though the masonry infills are classified as nonstructural components, nonlinear analysis is performed by adding modeling of masonry infills to the numerical model. The consideration of numerical model in columns and masonry infills is describe as follow.

Columns. The flexural-shear model composed of fiber element and shear spring was used to simulate the seismic behavior of the column elements considering the premature shear failure. In this model, the flexural behavior of the column is simulated by the fiber element, and the shear behavior of the column is simulated by the shear spring. The details of the column modeling will be discussed in Section 3.2.

Masonry infills. Equivalent diagonal struc element (i.e. lumped plasticity model) with nonlinear axial spring and linear truss elements was used. Existing studies [18, 19] have reported that the equivalent diagonal struct suggested by FEMA 356 [20] simulates the behavior of masonry infills closest to the actual behavior. The modeling parameters for the equivalent diagonal struc are discussed in Section 3.3.

(a) A frame module
(b) Detail of the numerical model

Fig. 2 – Total response of column element considering flexure-shear behavior

3.2. Column flexure-shear model

The fiber element that can express the flexural behavior of the column elements is modeled as a five node element by dividing the confined and unconfined regions (see Fig. 3(a)). A linear tension softening model (Concrete02) by Hisham M. was used as the concrete material model, and the Giuffrè-Menegotto-Pinto model (Steel02) was used as the steel material model in OpenSees.

The shear spring model to simulate the shear behavior of the column elements is shown in Fig. 3(b). Shear strength, \( V_s \), was estimated using equation (1) presented in FEMA 356 [20], and shear deformation, \( \Delta_s \), was calculated by considering the effective stiffness of the column. The detail information can be founded in FEMA 356.
The maximum shear strength, \( V_m \), of the column was estimated to be 1.137 times that of \( V_y \), based on the literature of Ahmed and Tan [21]. Shear deformation, \( \Delta_m \), was 8 times \( \Delta_y \) at maximum shear strength, residual shear strength \( V_r \) was approximately 0.227 times \( V_y \), and shear deformation, \( \Delta_r \), was 16 times of \( \Delta_y \) when residual shear strength was reached. Verification of the flexural-shear model of column elements is summarized in section 3.4.

3.3. Equivalent diagonal struct model

The masonry infills installed between the columns were modeled with equivalent diagonal compression strut as shown in Fig. 4. The parameters that determine numerical modeling for equivalent diagonal strut are the effective width \((W_{ef})\) of the equivalent compression brace, the effective stiffness \((K_e)\), and the maximum strength \((F_{max})\). The modeling parameters for the equivalent diagonal strut were calculated by using equation (2) to equation (4) on the basis of 1.0B stacking. The crack load \((F_{cr})\) is assumed to be 0.55\( F_{max} \), the tensile strength \((F_t)\) and the residual strength \((F_r)\) are 0.2\( F_{max} \). The displacement at the maximum strength \((\delta_{cap})\) is assumed to be 2\( \delta_{cr} \) (crack displacement), and the displacement at the residual strength \((\delta_r)\) is assumed to be \( \delta_{cap} \).

\[
W_{ef} = 0.175(\lambda_h H)^{0.4} \sqrt{H^2 + L^2} 
\]

\[
K_e = \frac{E_w W_{ef} t_w}{\sqrt{H^2 + L^2}} \cos^2 \theta 
\]

\[
F_{max} = 0.818 \frac{L_{in} t_w F_p}{C_t} \left( 1 + \sqrt{C_t^2 + 1} \right) 
\]

where, \( \lambda_h = \sqrt{\frac{E_w I_{in}}{4 E_t I_{in} H_{in}}} \), \( C_t = 1.925 \frac{L_{in}}{H_{in}} \), \( E_w \) is elastic modulus of masonry infills (MPa), \( E_c \) is elastic modulus of concrete (MPa), \( t_w \) is thickness of masonry infills (mm), \( I_c \) is second moment of inertia of...
column element (mm$^4$), $H$ is height of story (mm), $L$ is column span (mm), $H_{e}$ is effective height of masonry infills (mm), $L_{e}$ is effective length of masonry infills (mm), $f_{cr}$ is crack strength of masonry infills (mm), and $\theta$ is slop of diagonal struct.

![Diagram of masonry infills and backbone curve](image)

(a) Dimension of masonry infills  
(b) Backbone curve of nonlinear axial spring

**Fig. 4 – Detail of equivalent diagonal struct model**

3.4. Numerical model validation

The existing experimental data were investigated to validate the nonlinear column modeling method by comparing the experimental response and the simulation results. The structural properties used for model validation are given in the paper [22].

Fig. 5 represents the comparison of cyclic response of column elements between simulation and experiment. The numerical model used for the column elements well simulates the overall behavior of the experimental specimens. The maximum variations of the initial stiffness and the maximum strength are -3.2% and 5.4% for BG-1 specimen, respectively. A minus sign means that the simulation results are lower than the experimental results. For BG-2 specimen, the maximum variations of the initial stiffness and the maximum strength are -2.6% and -3.2%, respectively. So, it can be concluded that the column model accurately simulates the experimental results in terms of the initial stiffness and maximum strength.

![Cyclic response comparison between simulation and experiment](image)

(a) BG-1  
(b) BG-2

**Fig. 5 – Cyclic response comparison between simulation and experiment [22]**
4. Evaluation of Seismic Vulnerability Function

4.1. Evaluation of the seismic fragility based on probabilistic demand model analysis

The seismic fragility functions for the structural and nonstructural components are defined using the seismic demand and seismic capacity. When using the probabilistic seismic demand models based on bilinear cloud analysis, the seismic fragility of structural and nonstructural components is evaluated using equation (5).

\[ p_f = \Phi \left[ \ln \left( \frac{EDP_{im}}{C} \right) \right] \]

\[ \sqrt{\beta_{EDP}^2 + \beta_C^2} \]

where, \( p_f \) is the damage probability representing the seismic fragility, \( \Phi[\cdot] \) is cumulative distribution of log-normal distribution, \( EDP_{im} \) is the engineering demand parameters such as interstory drift ratio (IDR) or peak floor acceleration (PFA), \( C \) is capacity of building components, \( \beta_{EDP} \) is the dispersion of seismic response, and \( \beta_C \) is the dispersion of capacity of building components.

As shown in equation (5), the probabilistic seismic demand models are needed to develop the seismic fragility functions. Generally, the maximum responses of building structure (i.e. maximum interstory drift ratio or maximum peak floor acceleration) had been used as the EDPs for establishing the probabilistic seismic demand model regardless of the number of stories. However, if the damage probabilities of a building structure are measured as a function of seismic fragility using the maximum value, there is a possibility of overestimating the seismic losses resulting from the seismic damage occurring in each story [23]. Therefore, in this study, the seismic demands were assessed using the probabilistic seismic demand model using the bilinear least square fitting method (i.e. the bilinear cloud analysis [24]). Also, a total of 44 far-field ground motion data (22 pairs) were used to perform the nonlinear time history analysis for the prototype model. The probabilistic seismic demand models for IDR and PFA are established using the results of the time history analysis.

Fig. 6 showed the probabilistic seismic demand models of IDR and PFA for 1st floor. While the linear cloud method is presented by a power function, the bilinear cloud method used in this study is presented by two power function. Typically, if the building experiences a certain EDP (i.e. IDR or PFA) enough to failure the structural or the nonstructural components, the EDP was rapidly increased with respected to the seismic intensity. As shown in Fig. 6, it can be seen that there is a rapid increase in IDR and PFA when \( S_a \) is 0.4g or more. In other words, the bilinear method is more appropriate to estimate the seismic response than the linear method in case of major damage to the building structure.
The correlation analysis with simulation data were performed to statistically evaluate the reliability of the probabilistic seismic demand models obtained the two methods. The simulation data were used as the reference data. The results of the correlation analysis are summarized in Table 2. The coefficient of determination ($R^2$) was introduced as a measurement to quantitatively evaluate the correlation between seismic demand models and simulation data. The $R^2$ derived from the linear cloud method for the IDR on the first story is 0.5944 and the $R^2$ derived from the bilinear cloud method is 0.7819, which shows that the bilinear method well estimates the seismic response with higher reliability than the linear cloud method. Therefore, the probabilistic seismic demand models based on bilinear cloud analysis are used for seismic fragility analysis.

<table>
<thead>
<tr>
<th>Engineering Demand Parameter</th>
<th>Story Level</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Linear Cloud</td>
</tr>
<tr>
<td>Interstory drift ratio</td>
<td>1</td>
<td>0.5944</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.6512</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.6848</td>
</tr>
<tr>
<td>Peak floor acceleration</td>
<td>1</td>
<td>0.6352</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.6634</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.6766</td>
</tr>
</tbody>
</table>

The limit states for the structural and architectural nonstructural components are determined to define the seismic fragility functions. The median and the dispersion of structural and nonstructural components must be defined to estimate the seismic fragility functions (see equation (5)). Usually, the median value of the structural components (i.e. damage level according to the limit states) is defined as a maximum interstory drift ratio. The damage states of school building structures used in this study are summarized in Table 3 through the existing literature review [16, 25].

<table>
<thead>
<tr>
<th>Building Component</th>
<th>Parameter</th>
<th>Unit</th>
<th>DS1</th>
<th>DS2</th>
<th>DS3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$C$</td>
<td>$\beta_C$</td>
<td>$C$</td>
</tr>
<tr>
<td>Structural component</td>
<td>IDR</td>
<td>-</td>
<td>0.0035</td>
<td>0.40</td>
<td>0.0070</td>
</tr>
<tr>
<td>Masonry infills</td>
<td>IDR</td>
<td>-</td>
<td>0.0021</td>
<td>0.60</td>
<td>0.0071</td>
</tr>
<tr>
<td>Partitions</td>
<td>IDR</td>
<td>-</td>
<td>0.0064</td>
<td>0.30</td>
<td>-</td>
</tr>
<tr>
<td>Tiles</td>
<td>IDR</td>
<td>-</td>
<td>0.0021</td>
<td>0.60</td>
<td>0.0071</td>
</tr>
<tr>
<td>Ceilings</td>
<td>PFA</td>
<td>g</td>
<td>0.7</td>
<td>0.25</td>
<td>1.2</td>
</tr>
<tr>
<td>Floor finishing</td>
<td>PFA</td>
<td>g</td>
<td>0.5</td>
<td>0.40</td>
<td>0.7</td>
</tr>
</tbody>
</table>

The seismic fragility functions for the structural and nonstructural components were derived from the seismic demand and the limit states to probabilistically evaluate the physical damage of the school building structures (see Fig. 7). The exceedance probabilities of structural components reaching DS1 are similar in the first and second stories, and those of structural components reaching DS2 and DS3 are highest in the first story. According to the earthquake reconnaissance of the school building structures [7], the major damages were observed in the column elements of the first story, and no significant damages were observed as the number of stories increased. Therefore, the seismic fragility functions presented in this study reflect well the damage patterns observed in the school building structures. In section 4.2, the seismic vulnerability function...
was evaluated to analysis the seismic loss of school building using the structural and nonstructural seismic fragility functions representing the physical damages.

![Graphs showing seismic fragility functions for different components](image)

(a) Structural component  
(b) Masonry infills  
(c) Partitions  
(d) Tiles  
(e) Ceiling  
(f) Floor finishing

Fig. 7 – Structural and nonstructural seismic fragility functions for each story

### 4.2. Seismic loss analysis of the school building

The seismic vulnerability function for the school building must be defined in order to estimate the direct loss induced by earthquake. The repair cost regarding each damage state for structural and nonstructural components were investigated to define the seismic vulnerability function from literature [25] and summarized in Table 4.

<table>
<thead>
<tr>
<th>Building Component</th>
<th>Unit of Measurement</th>
<th>Quantity</th>
<th>Mean Repair Cost (₩/unit of measurement)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>DS1</td>
</tr>
<tr>
<td>Structural component</td>
<td>m²</td>
<td>55</td>
<td>2,300,172</td>
</tr>
<tr>
<td>Masonry infills</td>
<td>m</td>
<td>83.1</td>
<td>233,200</td>
</tr>
<tr>
<td>Partitions</td>
<td>m</td>
<td>204</td>
<td>311,175</td>
</tr>
<tr>
<td>Tiles</td>
<td>m</td>
<td>55.9</td>
<td>777,331</td>
</tr>
<tr>
<td>Ceilings</td>
<td>m²</td>
<td>43.9</td>
<td>280,500</td>
</tr>
<tr>
<td>Floor finishing</td>
<td>m²</td>
<td>537</td>
<td>72,149</td>
</tr>
</tbody>
</table>

Table 4 – Loss estimation parameters for structural and nonstructural components

Fig. 8(a) showed the seismic vulnerability function using simulation data. The vertical axis represents the damage ratio defined by the ratio of repair cost and replacement cost. Each point in Fig. 8(a) are the
result calculated from the mean repair costs summarized in Table 5 and the seismic fragility functions summarized in Section 4.1. The median and the standard deviation (i.e. vulnerability function parameter) are 0.8912, and 0.4257, respectively. The damage ratio of individual building can be evaluated from the defined seismic vulnerability function a particular scenario earthquake. For example, in the case of Gyeongju Earthquake ($S_a$ is 0.3g), the damage ratio is 1.32% (see Fig. 8(b)). So, it is possible to estimate the damage ratio per each component for each story.

4.3. Evaluation of loss-based seismic performance of the school building

In order to assess the seismic performance of building structures based on seismic losses, it is necessary to define a loss-based seismic performance criteria. Analyzing the trend of damage ratio for seismic intensity, the damage ratio of school building increases sharply from 8.79% at 0.5g to 60.79% at 1.0g (see Fig. 8(a)). The main reason for this increase of damage ratio is the high interstory drift ratio caused by the shear failure of the column on the 1st and 2nd stories. Therefore, it is possible to define the loss-based seismic performance criteria through correlation analysis between the interstory drift ratio (physical damage) and the damage ratio (economic loss).
Current seismic performance evaluation method suggests the performance limit based on the interstory drift ratio of the structural components. In this study, the loss-based performance limit criteria are presented as shown in Figure 9 by directly connecting the displacement-based performance limit suggested in the current code with the seismic loss. The performance limits based on interstory drift ratio for the school buildings (PL) are divided into PL=0.35%, PL=0.70%, and PL=1.20%, and the corresponding loss-based performance limit (LD) can be defined as LD=7.36%, LD=37.39%, and LD=69.25%. The damage ratio and corresponding current seismic performance of the school building are summarized in Table 5. The estimated seismic loss for S=0.494g, which corresponds to the design load level, is 39.01%, and the seismic performance of the school building that caused these seismic losses is interpreted as the collapse prevention from the current seismic performance level.

Table 5 – Loss-based performance evaluation of the school building

<table>
<thead>
<tr>
<th>Divisions</th>
<th>20%</th>
<th>40%</th>
<th>Design (66.7%)</th>
<th>80%</th>
<th>MCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic intensity (S)</td>
<td>0.150</td>
<td>0.300</td>
<td>0.494</td>
<td>0.600</td>
<td>0.750</td>
</tr>
<tr>
<td>Damage ratio (%)</td>
<td>7.60%</td>
<td>19.50%</td>
<td>39.01%</td>
<td>48.56%</td>
<td>67.70%</td>
</tr>
<tr>
<td>Corresponding current performance</td>
<td>LS</td>
<td>LS</td>
<td>CP</td>
<td>CP</td>
<td>CP</td>
</tr>
</tbody>
</table>

5. Conclusions
In this study, a simulation-based framework for evaluating the seismic vulnerability function is presented and applied to the school building. The flexure-shear behavior of column elements is presented to simulate the premature shear failure and validated with the experimental data. The seismic fragility functions for structural and nonstructural elements are evaluated to assess seismic vulnerability function. Using repair cost ratio that quantify structural and nonstructural damage as a percentage of total replacement cost, the seismic vulnerability function regarding the seismic intensity is defined. Also, the loss-based performance criteria are proposed through the correlation analysis between the current displacement-based performance limit and the seismic loss. The seismic losses of the school building are 39.01% for design load level and 67.70% for maximum considered earthquake (MCE) load level. In conclusions, it is found that the proposed methodology can provide decision-making data for evaluating seismic performance based on seismic loss and has the potential to extend to loss assessment of other categories of buildings.

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7. References


