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Structural Response of a Reinforced Concrete Building to Earthquake Excitation: A Nonlinear Time-History Analysis Case Study in Palu, Indonesia

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Abstract

Palu is one of the regions with high seismicity due to its location directly on the Palu-Koro Fault, the longest active fault in Indonesia. A post-earthquake structural response evaluation was conducted on a reinforced concrete office building in Palu, Indonesia, using nonlinear time-history analysis to assess its performance under seven recorded ground motions. This evaluation serves as a reference for determining the structural functionality, safety against collapse, and the need for structural reinforcement due to earthquake loads. The primary focus was to assess the serviceability limit state and ultimate limit state in accordance with SNI 1726:2019 and to determine the performance level category based on ATC-40. Modeling was conducted using ETABS software with input from earthquake accelerograms of the Superstition Hills, San Fernando, Kobe, Loma Prieta, El Centro, Caldiran, and Manjil earthquakes. The results show significant variations among the earthquake scenarios. The highest displacement and drift values occurred in the Y-direction due to the El Centro earthquake, while the lowest values were observed in the X-direction during the Caldiran earthquake. Only the Caldiran earthquake met the limit criteria in both directions. Based on ATC-40, three earthquakes (Caldiran, Loma Prieta, and San Fernando) reached the Immediate Occupancy (IO) and Life Safety (LS) levels. In comparison, the other four earthquakes showed levels exceeding Collapse Prevention (>CP), indicating potential building failure and high hazard risks. Therefore, further evaluation and retrofit recommendations are necessary to enhance the structure's capacity and stiffness, thereby achieving the desired performance level in all earthquake scenarios.

Keywords

Performance level, Serviceability limit state, Ultimate limit state, Time-history analysis, Earthquake

INTRODUCTION

Indonesia is situated at the confluence of three major tectonic plates: the Indo-Australian Plate, Eurasian Plate, and Pacific Plate, making it one of the countries with high seismic activity. One of the areas significantly affected is Palu City in Central Sulawesi Province, due to its location right on the Palu-Koro Fault line, which is the longest active fault in Indonesia. On September 28, 2018, Palu City experienced a destructive earthquake with a moment magnitude of 7.5 Mw that not only generated a massive tsunami and liquefaction but also caused extensive damage to infrastructure and buildings. This condition underscores the importance of conducting seismic reviews as a key measure for mitigating the impact of earthquake disasters today.

Basically, in the concept of earthquake-resistant building structures, buildings are designed not only to maintain elastic conditions but are allowed to experience damage within acceptable levels according to applicable regulations[1]. In other words, this concept emphasizes that the structure may be damaged, but should not collapse suddenly, allowing evacuation measures to be carried out and minimizing existing casualties during an earthquake.

Performance-based Seismic Design (PBSD) is an approach to planning design and structural evaluation that focuses on the overall performance aspects of the building[2]. Building performance serves as a benchmark to assess the condition of the structure and its potential collapse pattern during and after an earthquake[3]. D is becoming an increasingly applied standard because it provides references regarding the feasibility of structural functions, safety against collapse, and the need for structural reinforcement due to earthquake loads. In this approach, the review of structural performance against earthquakes is not only in terms of safety or strength, but also in terms of how well the structure meets specific performance objectives during earthquakes with different strength levels.

Analysis of structures under earthquake loads can be conducted using either static analysis methods or dynamic analysis methods[4]. Dynamic analysis for earthquake-resistant structures can be carried out when a more accurate assessment of the earthquake forces acting on the structure is required, and to understand the behavior of the structure due to the influence of the earthquake[5]. This method is carried out by taking into account the dynamic influence of ground motion on the structure under review to obtain the distribution of earthquake shear forces at all levels[6]. In the dynamic analysis itself, several methods can be used, including Time-History Analysis and Response Spectrum Analysis[7]. Time-history analysis is an analytical method used to determine the response of a construction structure by simulating earthquake loads in stages[8]. In time-history analysis, a structural model is analyzed using earthquake acceleration data as input to simulate ground motion acceleration due to earthquakes, and structural responses are then calculated step by step at specific intervals[9]. In other words, the ground surface at the location of the structure is accelerated according to the acceleration record against the time of the earthquake data that has occurred before, so that the structural response to the earthquake due to the acceleration can be observed as a basis for structural behavior.

Non-linear time-history analysis is a dynamic analysis approach that is increasingly used for performance-based design and evaluation purposes, as it is the most advanced analysis available today[9]. In more detail, non-linear time-history analysis is performed by feeding ground motion into a structural computational model, which then explicitly models the non-linear forces and deformation responses of structural components[10].

Structural evaluation of public buildings is a crucial step in disaster risk mitigation efforts, ensuring that buildings remain capable of withstanding earthquake loads in accordance with Performance-Based Seismic Design criteria. Strategic public buildings, such as schools, universities, hospitals, government offices, and other public facilities, play vital roles in people's lives. Therefore, building structures must have a high level of structural safety to ensure they can survive and operate in emergency conditions, such as earthquakes.

This study evaluates the performance of an existing structure with a reinforced concrete portal system using the time-history dynamic analysis method with seven different earthquake records. The research object is a reinforced concrete institutional building located in Palu City, an area classified as an earthquake-prone zone. This study reviews the primary parameters, including base shear force, story drift, and displacement of structural elements, to assess the level of building performance criteria after an earthquake using ETABS software modeling and analysis.

RESEARCH SIGNIFICANCE

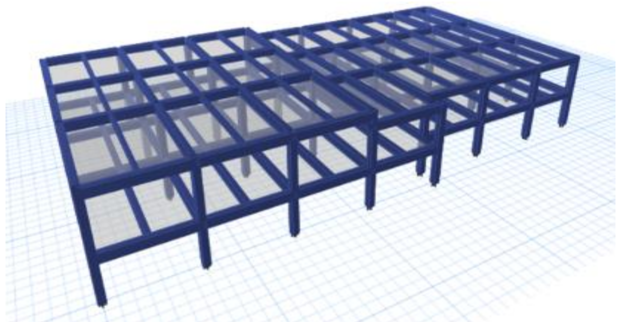
This study aims to evaluate the seismic response of reinforced concrete buildings more accurately, enabling a more precise determination of the post-earthquake performance of the building structure. This study focuses on determining the displacement and drift values to obtain service limit performance and ultimate limit performance using a nonlinear time-history analysis approach. The structural performance results obtained will be determined based on the assessment criteria listed in the ATC-40 standard. This evaluation is then expected to be the basis for assessing the safety and feasibility of structures after experiencing seismic events.

METHODOLOGY

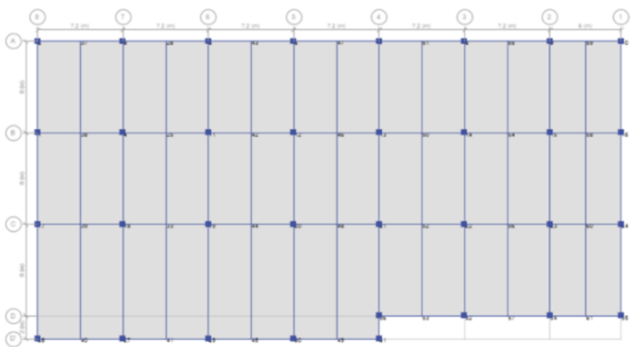
This research employs a numerical analysis method, utilizing ETABS software to model the existing building structure. The analyzed structural model comprises main elements, including columns, beams, floor slabs, and roof slabs. The application of loads on the structure includes gravity (dead load, additional dead load, and live load) and earthquake loads in the form of subgrade acceleration. The parameters supporting earthquake loads and structural performance evaluation utilize applicable standards and requirements based on SNI 1726:2019.

A. BUILDING DESCRIPTION

The building reviewed in this study is a two-story reinforced concrete lecture building with two roof decks, located in Palu City. The site is characterized by dense silty sand soil conditions. The building functions as a lecture facility with a total height of approximately 9 meters and a plan dimension of 48 x 26 square meters. The building structure is a reinforced concrete type of Special Moment Bearing Frame System – SRPMK, with a concrete compressive strength of 30 MPa and a tensile yield strength of reinforcing steel of 400 MPa for longitudinal reinforcement and 240 MPa for shear reinforcement.



(a)



(b)

Figure 1 Modelling of Existing Buildings in ETABS:
(a) 3D view, (b) Plan view

Table 1 Configuration and Effective Seismic Weight

Floor Level	Story Height (m)	Elevation (m)	Effective Seismic Weight (kN)
Roof Slab 2	1	9	4240.50
Roof Slab 1	4	8	4923.51
Second Floor	4	4	12233.62
Effective Seismic Weight			21397.63

B. DYNAMIC INPUT MOTION

In the time-history method for earthquake acceleration analysis, accelerogram data should be selected from earthquake records that have similar characteristics to the location of the building structure under review, taking into account the scaling process of the input earthquake acceleration and the scaling of the structural base shear force[11]. This dynamic analysis was performed using accelerograms from seven real earthquake records with response spectrum and moment magnitudes comparable to the Palu earthquake: Superstition Hills (USA, Mw 6.54), San Fernando (USA, Mw 6.6), Kobe (Japan, Mw 6.90), Loma Prieta (USA, Mw 6.93), El Centro (USA, Mw 6.95), Caldiran (Turkey, Mw 7.21), and Manjil (Iran, Mw 7.37) sourced from PEER pacific earthquake engineering research center (PEER) ground motion database. These accelerograms are ground acceleration (PGA) data, presented in a graph that compares the ground acceleration to the duration of the earthquake[5]. The accelerogram data were then scaled to approximate the design response spectrum of the Palu area, particularly in the period range relevant to the structure's dynamic behavior. Ground motion scaling with response spectrum is the process of comparing the magnitude of each ground motion to be adjusted to the magnitude of the design response spectrum, as specified in SNI 1726:2019[12].

In structures modeled in ETABS, accelerogram data is simulated to generate earthquake loads in the form of ground acceleration versus time. The results of the analysis are used to evaluate the response of the structure to earthquake loads, as well as to assess the performance level of the structure after it has been subjected to earthquake forces.

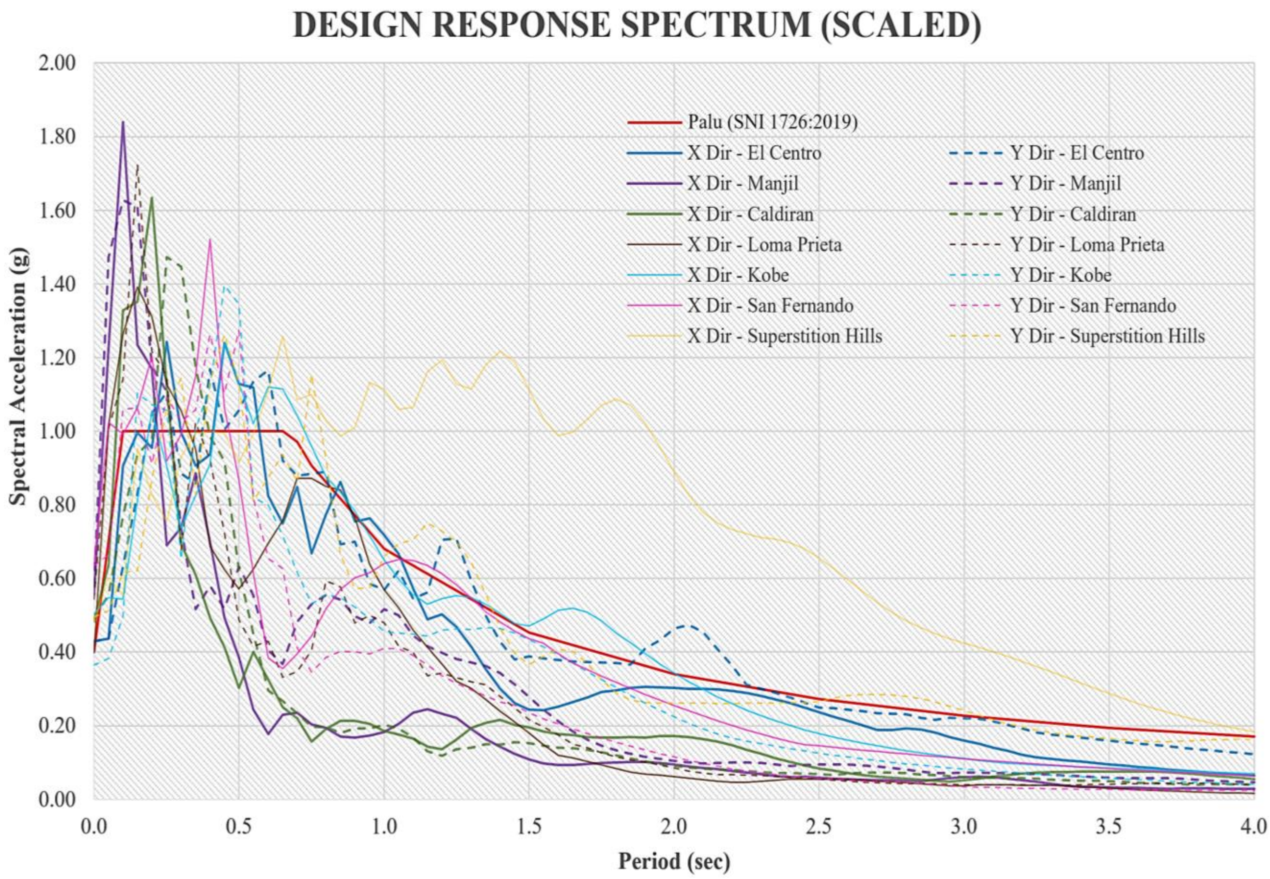


Figure 2 Scaled Response Spectrum of Seven Ground Motion matched to the Design Response Spectrum of Palu (SNI 1726:2019)

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Figure 3 Ground Motion Records for Nonlinear Time-History Analysis

C. STRUCTURAL PERFORMANCE

This research was conducted by paying attention to the level of damage that occurs when the structure is given a certain earthquake load that has occurred before. From the results of the analysis, a structural performance evaluation is carried out to determine whether the structure is safe or not according to the inter-level deviation limits listed in SNI 1726: 2019 and the structural performance level according to Applied Technology Council-40[13].

Structures are reviewed against two limits, namely service limit performance and ultimate limit performance[14]. The service limit performance of the building structure is determined by the inter-level deviation due to the influence of the plan earthquake while the ultimate limit performance of the building structure is determined by the maximum deviation and inter-level deviation of the building structure due to the influence of the Plan Earthquake in the condition of the building structure on the verge of collapse[15]. Furthermore, the structural performance level is determined based on the ATC-40 document which is categorized by taking into account the post-earthquake drift ratio value[16].

RESULTS AND DISCUSSIONS

The nonlinear time-history dynamic analysis of the lecture building was performed using seven recorded ground motions from previous earthquakes as input data variations, intended to obtain a more thorough comparison of the structural response to earthquake loads, thereby ensuring more accurate analysis and further assessing the consistency of structural performance.

A. DYNAMIC PROPERTIES

Modal analysis is performed after structural modeling and loading are complete, as a control to the applicable standards and requirements. The analysis results are reviewed for the participation of the modal mass and the fundamental period of the structure (T).

Table 2 Modal Participating Mass Ratios

Mode	Ux	Uy
1	0.0009	0.6744
2	0.8595	0.6778
3	0.8643	0.8586
4	0.8643	0.9674
5	0.9987	0.9674
6	0.9987	0.9989
7	0.9987	0.9997
8	0.9999	0.9997
9	0.9999	1
10	1	1
11	1	1
12	1	1

The results of the ETABS analysis show that the total response for the fifth mode with Sum-UX = 0.9987 and Sum-UY = 0.9674, respectively, has met the minimum requirement of 90%, indicating that the mass of the structure has been appropriately accommodated.

Table 3 Periods and Frequencies

Mode	Period (sec)	Frequency (cyc/sec)
1	0.690	1.449
2	0.636	1.572
3	0.589	1.697
4	0.23	4.346
5	0.218	4.587
6	0.194	5.162
7	0.161	6.205
8	0.128	7.786
9	0.104	9.572
10	0.074	13.472
11	0.055	18.258
12	0.046	21.524

According to SNI 1726:2019, the fundamental period of the structure (T) is not allowed to exceed the product of the coefficient for the upper limit at the calculated period (C_u) and the fundamental period of the approach (T_a), so that it can be expressed in terms of $T_{a\ min} < T < T_{a\ max}$.

For the natural vibration period (T_c), the first variation obtained from the ETABS analysis is 0.690 sec. The $T_{a\ min}$ and $T_{a\ max}$ values obtained from calculations based on tables 17 and 18 in SNI 1726:2019 are 0.337 sec and 0.471 sec, respectively. Because the value of $T_c > T_{a\ max}$, the value of T used for dynamic analysis is 0.471 sec.

B. SEISMIC LOAD ANALYSIS

Determination of earthquake load analysis parameters calculated based on SNI 1726:2019, including seismic response coefficient (Cs), seismic base shear force (V), and vertical distribution of lateral seismic force (F), which is then applied to the structure to evaluate its lateral response.

From the calculation results, a Cs value of 0.1061 seconds is obtained, which is then multiplied by the effective seismic weight (W) to obtain a seismic base shear force (V) value of 2269.88 kN. Meanwhile, the vertical distribution value of lateral seismic force (F) is divided based on the mass proportion and floor height, as shown in the following table.

Table 4 Vertical Distribution of Lateral Forces (F)

Floor Level	Elevation h_i^k (m)	Weight W_i (kN)	Moment $W_i\ h_i^k$ (kN-m)	Lateral Force (kN)
Roof Slab 2	9	4240.50	38164.53	684.88
Roof Slab 1	8	4923.51	39388.10	706.84
Second Floor	4	12233.62	48934.48	878.16
Total			126487.11	2269.88

C. STRUCTURAL RESPONSES TO SEISMIC LOAD

In the nominal base shear force control, the dynamic shear force ($V_{dynamic}$), must be greater than 85% of the static shear force (V_{static})[17]. From the results of the analysis performed, it is not necessary to scale the value of the base shear force because the $V_{dynamic}$ exceeds the V_{static} by 2269.88 kN. This value meets the requirements specified for the base shear force in the SNI 1726:2019 standard.

The analysis results are further reviewed based on two parameters: displacement and drift, which represent the structural response to seismic loads in terms of overall movement and inter-story deformation. The displacement and drift values are presented in Table 5 and Table 6.

Table 5 Story Displacement Results

Floor Level	X-Direction Displacement, *D* (mm)						
	El Centro	**San Fernando**	**Caldiran**	**Superstition Hills**	**Loma Prieta**	**Manjil**	**Kobe**
Roof Slab 2	331.896	130.532	63.635	381.718	176.665	356.356	271.336
Roof Slab 1	294.292	114.629	54.506	341.621	153.134	311.39	244.556
Second Floor	153.796	60.4	33.885	161.289	81.949	167.338	117.003

Floor Level	Y-Direction Displacement, *D* (mm)						
	El Centro	**San Fernando**	**Caldiran**	**Superstition Hills**	**Loma Prieta**	**Manjil**	**Kobe**
Roof Slab 2	462.187	167.156	102.769	287.243	169.035	277.476	210.31
Roof Slab 1	303.98	96.514	67.336	161.915	101.901	163.748	118.533
Second Floor	183.355	68.828	44.653	99.704	60.116	122.681	94.563

Table 6 Story Drift Results

Floor Level	X-Direction Story Drift, Δx (mm)						
	El Centro	**San Fernando**	**Caldiran**	**Superstition Hills**	**Loma Prieta**	**Manjil**	**Kobe**
Roof Slab 2	42.802	17.331	11.05	46.542	23.917	49.089	34.122
Roof Slab 1	43.982	17.554	10.153	47.752	24.001	49.32	34.896
Second Floor	38.427	15.09	8.462	40.304	20.478	42.233	29.237

Floor Level	Y-Direction Story Drift, Δy (mm)						
	El Centro	**San Fernando**	**Caldiran**	**Superstition Hills**	**Loma Prieta**	**Manjil**	**Kobe**
Roof Slab 2	60.926	46.542	16.075	38.847	25.459	38.798	30.049
Roof Slab 1	40.605	47.752	10.806	25.712	17.918	26.19	19.746
Second Floor	46.224	40.304	11.163	25.744	15.107	30.67	23.641

D. PERFORMANCE EVALUATION

1. Serviceability Limit State (SLS) Control

Based on the results of ETABS analysis for several earthquake scenarios, the values of inter-story drift vary significantly. For example, the highest story drift on Roof Slab 2 in X-direction was obtained due to the Manjil earthquake at 49.089 mm and exceeded the permissible limit according to SNI 1726:2019. Whereas the lowest drift value obtained due to the Caldiran earthquake of 11.05 mm meets the criteria of the drift (Δ) limit requirement <0.03/R*h. This extreme difference implies the need for attention and follow-up on critical areas of the structure.

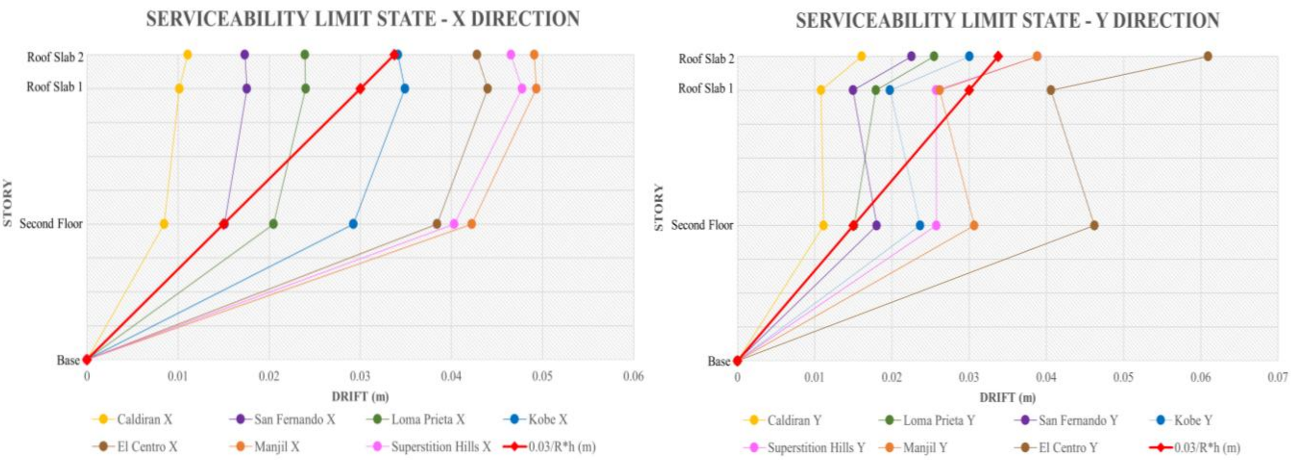

Figure 4 SLS Comparison under Seven Earthquake Scenarios

Table 7 Serviceability Limit State Control

Design Earthquake	Floor Level	Height (mm)	Story Drift (mm)		Requirement *0.03/R*h* (mm)	Remarks	
		h	Δx	Δy		X-Dir	Y-Dir
El Centro	Roof Slab 2	9000	42.802	60.926	33.75	NOT OK	NOT OK
	Roof Slab 1	8000	43.982	40.605	30	NOT OK	NOT OK
	Second Floor	4000	38.427	46.224	15	NOT OK	NOT OK
San Fernando	Roof Slab 2	9000	17.331	22.544	33.75	OK	OK
	Roof Slab 1	8000	17.554	14.962	30	OK	OK
	Second Floor	4000	15.09	18.032	15	NOT OK	NOT OK
Caldiran	Roof Slab 2	9000	11.05	16.075	33.75	OK	OK
	Roof Slab 1	8000	10.153	10.806	30	OK	OK
	Second Floor	4000	8.462	11.163	15	OK	OK
Superstition Hills	Roof Slab 2	9000	46.542	38.847	33.75	NOT OK	NOT OK
	Roof Slab 1	8000	47.752	25.712	30	NOT OK	OK
	Second Floor	4000	40.304	25.744	15	NOT OK	NOT OK
Loma Prieta	Roof Slab 2	9000	23.917	25.459	33.75	OK	OK
	Roof Slab 1	8000	24.001	17.918	30	OK	OK
	Second Floor	4000	20.478	15.107	15	NOT OK	NOT OK
Manjil	Roof Slab 2	9000	49.089	38.798	33.75	NOT OK	NOT OK
	Roof Slab 1	8000	49.32	26.19	30	NOT OK	OK
	Second Floor	4000	42.233	30.67	15	NOT OK	NOT OK
Kobe	Roof Slab 2	9000	34.122	30.049	33.75	NOT OK	OK
	Roof Slab 1	8000	34.896	19.746	30	NOT OK	OK
	Second Floor	4000	29.237	23.641	15	NOT OK	NOT OK

2. Ultimate Limit State (ULS) Control

For the performance criteria at the ultimate limit, the story drift value multiplied by the primacy factor ζ = 0.7R obtained also tends not to meet the requirement <0.02h according to SNI 1726:2019 for several earthquake records. The overall qualified ultimate limit state was only obtained for the Caldiran earthquake in the X-direction and Y-direction, while the Loma Prieta and San Fernando earthquakes were also qualified in two directions but only for roof slabs 1 and 2. In addition, the Superstition Hills, Manjil, and Kobe earthquakes each had qualified ultimate limit state values in the Y-direction only. The analyzed ultimate limit state values in each direction can be seen in Table 8 and Figure 5.

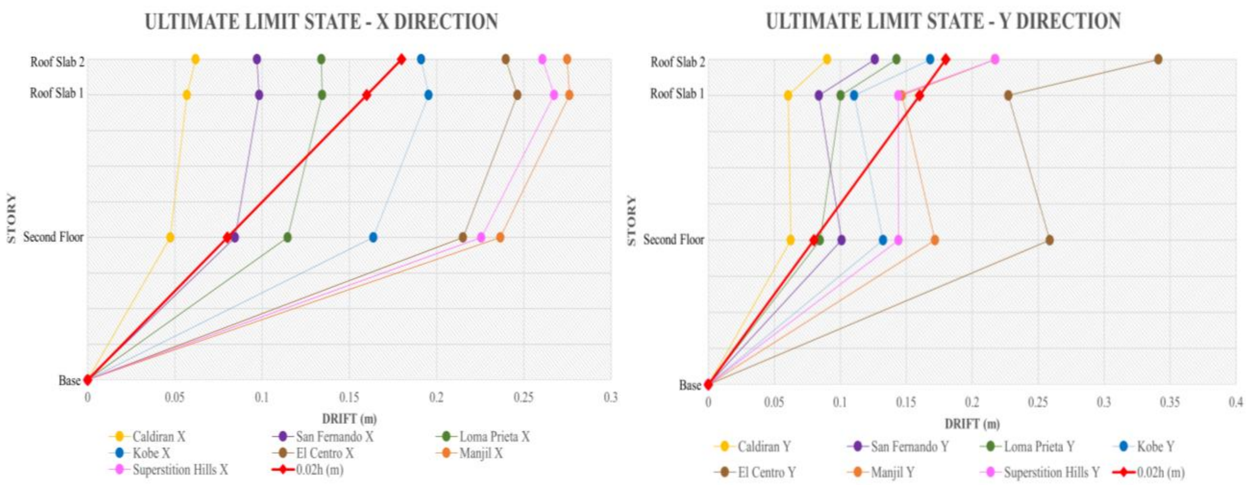

Figure 5 ULS Comparison under Seven Earthquake Scenarios

Table 8 Ultimate Limit State Control

Design Earthquake	Floor Level	Height (mm)	ζ.Δ (mm)		Requirement 0.02*h (mm)	Remarks	
		h	ζ.Δ*x*	ζ.Δ*y*		X-Dir	Y-Dir
El Centro	Roof Slab 2	9000	239.691	341.186	180	NOT OK	NOT OK
	Roof Slab 1	8000	246.299	227.388	160	NOT OK	NOT OK
	Second Floor	4000	215.191	258.854	80	NOT OK	NOT OK
San Fernando	Roof Slab 2	9000	97.054	126.246	180	OK	OK
	Roof Slab 1	8000	98.302	83.787	160	OK	OK
	Second Floor	4000	84.504	100.979	80	NOT OK	NOT OK
Caldiran	Roof Slab 2	9000	61.88	90.02	180	OK	OK
	Roof Slab 1	8000	56.857	60.514	160	OK	OK
	Second Floor	4000	47.387	62.513	80	OK	OK
Superstition Hills	Roof Slab 2	9000	260.635	217.543	180	NOT OK	NOT OK
	Roof Slab 1	8000	267.411	143.987	160	NOT OK	OK
	Second Floor	4000	225.702	144.166	80	NOT OK	NOT OK
Loma Prieta	Roof Slab 2	9000	133.935	142.57	180	OK	OK
	Roof Slab 1	8000	134.406	100.341	160	OK	OK
	Second Floor	4000	114.677	84.599	80	NOT OK	NOT OK
Manjil	Roof Slab 2	9000	274.898	217.269	180	NOT OK	NOT OK
	Roof Slab 1	8000	276.192	146.664	160	NOT OK	OK
	Second Floor	4000	236.505	171.752	80	NOT OK	NOT OK
Kobe	Roof Slab 2	9000	191.083	168.274	180	NOT OK	OK
	Roof Slab 1	8000	195.418	110.578	160	NOT OK	OK
	Second Floor	4000	163.727	132.39	80	NOT OK	NOT OK

3. Building Performance Level Based on ATC-40

Based on ATC-40, structural performance is analyzed by comparing the most significant displacement value or displacement at the point under review to the height of the building. Based on an analysis of seven earthquake records, the structural performance is categorized into three categories: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). For the IO category, where the drift ratio <1% is only obtained in the X-direction of Caldiran Earthquake loading, while for the LS category, a drift ratio of 1-2% is obtained in the Y-direction of Caldiran Earthquake loading, as well as the Loma Prieta Earthquake and San Fernando Earthquake in two directions. In addition, the four earthquakes namely Kobe, El Centro, Manjil, and Superstition Hills, each obtained a drift ratio value >2.5% which is categorized into the >CP category.

Table 9 Structural Performance Level ATC-40

Design Earthquake	*h* (mm)	*D* (mm)		Drift Ratio (%)		Performance Level	
		X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
El Centro	9000	331.896	462.187	3.688	5.135	>CP	>CP
San Fernando	9000	130.532	167.156	1.450	1.857	LS	LS
Caldiran	9000	63.635	102.769	0.707	1.142	IO	LS
Superstition Hills	9000	381.718	287.243	4.241	3.192	>CP	>CP
Loma Prieta	9000	176.665	169.035	1.963	1.878	LS	LS
Manjil	9000	356.356	277.476	3.960	3.083	>CP	>CP
Kobe	9000	271.336	210.31	3.015	2.337	>CP	>CP

CONCLUSIONS

This study shows a comparison of the response and performance levels of structures to seven different earthquake loadings. Based on the results of the nonlinear dynamic time-history analysis, it can be concluded that:

1. The displacement and drift values at the roof level have significant variations. The most considerable displacement value is obtained in the loading due to the El Centro Earthquake in the Y direction, which is 462.187 mm. In contrast, the lowest value is very different, namely 63.635 mm, due to the Caldiran earthquake loading in the X direction. The drift ratio value also yields the same results; the largest drift value of 5.135% occurs in the Y direction due to the El Centro earthquake loading, while the lowest value of 0.707% occurs in the X direction due to the Caldiran earthquake loading.
2. For the SLS and ULS control, each obtained results that tend to fall short of the standards required in SNI 1726:2019. The overall qualified value is only in 1 of the seven earthquake loading scenarios, namely the Caldiran earthquake in both X and Y directions. This indicates that the structure has the potential to fail in the event of a large earthquake, necessitating further evaluation and possible strut to achieve the expected performance level.
3. The results of the structural performance level analysis, based on ATC-40, are divided into three distinct categories. Broadly speaking, only three earthquake loading conditions —namely, Caldiran, Loma Prieta, and San Fernando — meet the performance level targets of IO and LS in both the X-direction and Y-direction. Meanwhile, the loading due to the El Centro, Kobe, Manjil, and Superstition Hills earthquakes each obtained performance level results >CP, which indicates that the deviation and deformation that occurs can cause significant structural damage or failure, so that the risk of danger is considerable and can threaten the safety of building users and post-earthquake operations. Therefore, structural retrofitting or strengthening recommendations are required to increase the building's stiffness and deformation capacity to meet the target level of performance in all earthquake scenarios analyzed.

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