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ANALYSIS OF STRUCTURAL IRREGULARITIES IN AN ELEVEN-STORY REINFORCED CONCRETE HOTEL BUILDING IN PONTIANAK USING LINEAR TIME HISTORY ANALYSIS

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ABSTRACT

Structural irregularity is a factor that can significantly affect a building's response to seismic loads. Therefore, an analysis that more realistically represents structural behavior under earthquake excitation is required. This study aims to analyze the irregularity of a reinforced concrete structure in an eleven-story hotel building with a height of 48 m located in Pontianak City using the linear time-history analysis method. The earthquake ground motion records employed in this study were obtained from the Imperial Valley, Morgan Hill, Whittier Narrows, and Chichi earthquakes. Structural modeling and analysis were performed using RSAP (Robot Structural Analysis Professional) 2024 and SeismoMatch in accordance with SNI 1726:2019 and SNI 2847:2019. The structure uses a reinforced concrete Special Moment-Resisting Frame (SMRF) system and considers Site Class SE and seismic design parameters of S_{DS} = 0.281 g and S_{D1} = 0.143 g. The maximum interstory drift was found to be 44.836 mm under the Morgan Hill Y (-) case, corresponding to a base shear of 813.887 kN. The structure exhibited Horizontal Irregularity Types 1a and 1b, with the most critical condition caused by the Chichi X (+Y) case. The minimum ratio of story lateral strength was found to be 99.369% on the 10th floor; therefore, the structure does not exhibit Vertical Irregularity Type 5b. The results indicate that structural irregularities significantly influence the seismic performance of the building and should be carefully considered in the design and evaluation of reinforced concrete structures subjected to earthquake loads.

Keywords: linear time-history analysis, reinforced concrete structure, SMRF, structural irregularity.

1. INTRODUCTION

The construction of high-rise buildings in Indonesia continues to increase in line with the growth of the economic, service, and tourism sectors. One of the rapidly growing types of buildings is high-rise hotels, which require structural designs that are safe, strong, and meet seismic resistance requirements. Although the city of Pontianak is located in an area with a relatively lower seismic hazard level compared to some other regions in Indonesia, earthquake-resistant structural design remains a critical aspect to ensure the safety of the building and its occupants in accordance with the provisions of SNI 1726:2019 [1].

Seismic loads are the most critical lateral loads in the design of multi-story buildings. A structure's response to earthquakes is influenced by various factors, such as mass, stiffness, ductility, and the building's geometric configuration. Structural irregularities, both horizontal and vertical, can cause an uneven distribution of seismic forces, thereby increasing the likelihood of stress concentrations and damage to specific structural elements [2]. Irregular buildings exhibit different seismic responses compared to regular buildings, particularly in terms of displacement parameters, interstory drift, base shear, and torsional effects [3], [4].

Structural irregularities generally result from changes in floor plan shape, uneven mass distribution, and differences in stiffness between floors. These conditions can cause the dynamic behavior of a structure

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to become more complex when subjected to seismic loads. Therefore, the seismic design standard SNI 1726:2019 pays special attention to buildings with such irregularities, as they have the potential to exhibit a greater response compared to regular structures. The presence of irregularities can increase lateral deflections and heighten the risk of earthquake damage if not properly accounted for in the design [5], [6]. For example, if the structural stiffness is uneven across floors, the weaker floors may experience greater lateral displacement (soft story), which poses a high risk of local collapse. Similarly, if the mass distribution or floor plan is asymmetrical, the center of mass and the center of stiffness will be misaligned, causing torsional forces during seismic vibrations.

To evaluate a structure's behavior under seismic loads, dynamic analysis is a more representative method than equivalent static analysis, particularly for multi-story buildings and structures with irregular configurations. One widely used method of dynamic analysis is time-history analysis, which accounts for variations in seismic acceleration over time and thus provides a more realistic representation of the structure's response [7]. This method evaluates structural parameters such as displacement, acceleration, internal forces, and interstory deflection throughout the duration of an earthquake.

Time-history analysis can be performed using either linear or nonlinear methods. This study employs the linear time-history analysis method, which assumes that the material behavior remains within the elastic range. Elastic analysis has limitations in representing structural behavior when the response enters an inelastic state. During a strong earthquake, structural members may experience cracking, reinforcement yielding, the formation of plastic hinges, degradation of stiffness and strength, and the redistribution of forces among members. These phenomena cannot be explicitly represented in elastic analysis because the force–strain relationship of the members is assumed to remain linear. Although simpler than nonlinear analysis, this method is still capable of providing a reasonable representation of the structure's dynamic response characteristics and the effect of irregularities on the building's behavior when subjected to seismic loads [8]. In addition, linear time-history analysis is still widely used in the initial evaluation of the performance of reinforced concrete structures because it is efficient in terms of modeling and computation time.

Various studies have been conducted to evaluate the seismic behavior of reinforced concrete buildings with structural irregularities. Simbolon et al. [9] evaluated horizontal irregularities in the Simpang Temu MRT Dukuh Atas Building using RSA (Response Spectrum Analysis) and LTHA (Linear Time-History Analysis). The study focused on evaluating displacement, interstory drift, structural stability, and the identification of horizontal irregularities based on SNI 1726:2019. The results showed that the time-history method can produce different structural responses compared to the response-spectrum method. However, the study only addressed horizontal irregularities and therefore did not comprehensively evaluate vertical irregularities.

Therefore, this study aims to address this gap by analyzing the structural irregularities of a 48-meter-tall, eleven-story hotel building in Pontianak using the LTHA method. This study uses four real earthquake records—Imperial Valley, Morgan Hill, Whittier Narrows, and Chichi—to evaluate the structural response in terms of interstory drift and base shear, while also identifying horizontal and vertical irregularities based on the provisions of SNI 1726:2019 and determining which earthquake record produces the most critical condition. Thus, this study is expected to contribute to understanding the influence of structural irregularities on the seismic performance of reinforced concrete buildings in areas with relatively low earthquake hazard levels.

Reinforced concrete was chosen because it is the most commonly used system in multi-story buildings in Indonesia due to its ability to effectively resist both gravitational and lateral loads. Given indications of irregularities in the building's configuration, a more in-depth analysis is needed to determine their impact on the structure's response to an earthquake.

SMRF (Special Moment-Resisting Frame) is one of the earthquake-resistant structural systems widely used in reinforced concrete buildings in Indonesia. SMRF is designed to have a high degree of ductility, enabling it to undergo significant inelastic deformation without sudden collapse when subjected to strong seismic loads. The main concept of this system is the structure's ability to absorb and dissipate seismic energy through the formation of plastic hinges in the beam elements, while the column elements maintain their capacity in accordance with the strong-column–weak-beam principle.

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In the design of high-rise buildings, the use of SMRF is expected to result in structures that are safe, stable, and meet performance requirements for both gravitational loads and lateral loads caused by earthquakes. Therefore, the application of SMRF principles is one of the key aspects of modern reinforced concrete structural design in Indonesia [10].

This study focuses on the response of structurally irregular buildings to earthquakes and the design of structural elements to meet SMRF requirements. The results of this study are expected to provide information on the dynamic behavior of irregular buildings and serve as a reference for structural engineers in designing buildings that are safe, reliable, and compliant with applicable regulations. The design of the hotel building's structure is shown in Figure 1.

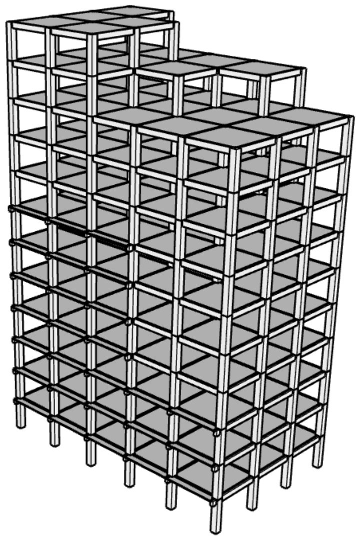

Figure 1. Structural Design of Hotel Building

2. RESEARCH METHOD

2.1 Data Collection

The required data, including architectural data, site soil data, and ground motion data, were used for the design and structural calculations of the building. Ground motion data were selected based on the disaggregation map [11]; the city of Pontianak has an R value ranging from 60 to 70 km and an M value ranging from 5.2 to 5.4. According to SNI 1726:2019, Table 5, sites classified as soft soil (SE) have a $\bar{V}_s$ value ranging from 0 to 175 m/s. Once the parameters were determined, ground motion records with matching or closely matching parameters were obtained from https://ngawest2.berkeley.edu.
In accordance with SNI 1726:2019, section 7.9.2.3, the dataset must consist of at least 3 pairs of seismic records (each pair consisting of two mutually perpendicular horizontal components, such as X and Y). The design results must be based on the maximum response value from all records.

2.2 Preliminary Design

Preliminary design is the initial design phase used to determine the conceptual design of a building's structural elements so that it can be used for more detailed calculations. Material properties of the structural elements are f_c' = 32 MPa and f_y = 420 MPa. The floor slab was modeled as a full-rigid diaphragm and the beams were modeled as fixed at both ends. The base supports were assumed to be fixed (rigid). The damping ratio for reinforced concrete structures was taken as 5%. The modeling of the structural elements was based on the following applicable provisions:

1. Beam. Beams were modeled as frames with a moment of inertia of $0.35I_g$ in accordance with Section 6.6.3.1.1 of SNI 2847:2019.

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2. Column. Columns were modeled as frames with a moment of inertia of $0.7I_g$ in accordance with Section 6.6.3.1.1 of SNI 2847:2019.
3. Floor Slab. Floor slabs were modeled as thin shells with a moment of inertia of $0.25I_g$ in accordance with Section 6.6.3.1.1 of SNI 2847:2019.
4. Stair. Stair slabs were modeled as thin shells with a moment of inertia of $0.25I_g$ in accordance with Section 6.6.3.1.1 of SNI 2847:2019.

Accidental eccentricity of ±5% is applied to each seismic load direction, assumed to act at locations shifted by ±5% from the building's plan dimensions perpendicular to the seismic load direction, resulting in two loading conditions: +5% and −5%. For example, for an X-direction earthquake, eccentricity is applied in the Y-direction by ±0.05B (where B is the building width in the Y-direction), while for a Y-direction earthquake, eccentricity is applied in the X-direction by ±0.05L (where L is the building length in the X-direction). This results in an additional torsional moment of T = V × e, where V is the floor seismic force and e is the accidental eccentricity value. Analysis is then performed for both eccentricity conditions (positive and negative), and the response resulting in the most critical condition is used as the basis for design evaluation, including in the check for Type 1a and 1b torsional irregularities (Figure 2).

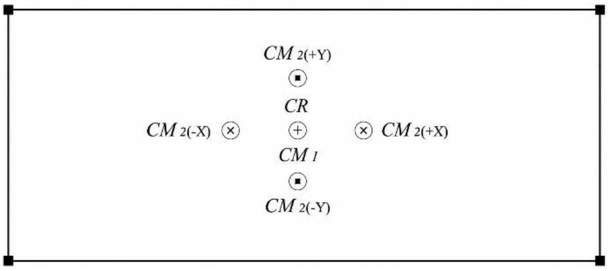

Figure 2. Accidental Torsion Eccentricity

The values for superimposed dead loads (SDL) and live loads (LL) refer to SNI 1727:2020 [12], as shown in Figure 3 (SDL = 1.83 kN/m^2). Considering the building's geometry and function, the effective seismic weight combination is 1.0D + 0.32L.

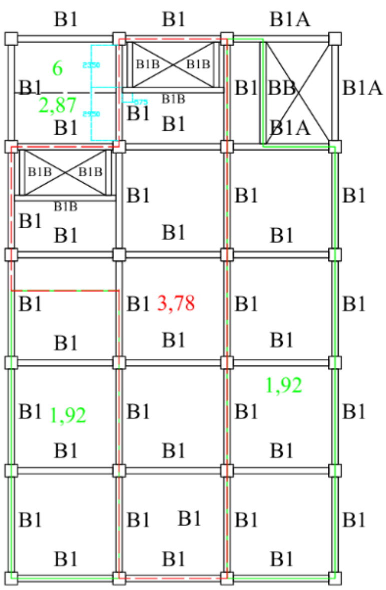

Figure 3. Live Load on the Third to Sixth Floors

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2.3 Structural Analysis

The hotel building structure was analyzed using RSAP (Robot Structural Analysis Professional) 2024 and the spectral matching was performed using SeismoMatch based on the method by Hancock et al. (2006). The building was modeled, and the resulting outputs were the internal forces, interstory drift, and base shear.

In Section 7.9.1.1 of SNI 1726:2019, modal response spectrum analysis is performed by examining the number of modes. The analysis must include a sufficient number of modes to achieve a combined modal mass of at least 90% of the actual mass in each orthogonal horizontal direction of the response being analyzed by the model.

2.4 Structural Irregularity Evaluation

A. Horizontal Irregularity

Horizontal irregularity refers to irregularities in a building's floor plan. These irregularities are generally caused by an asymmetrical floor plan, uneven mass distribution, variations in stiffness in certain directions, or the presence of large openings in floor diaphragms. These conditions can cause the center of mass and the center of stiffness to be misaligned, resulting in a significant torsional response during an earthquake [13]. The types of horizontal irregularity are shown in Table 1.

Table 1. Types of Horizontal Irregularity

Type of Horizontal Irregularity	Applicable SDC
1a. Torsional Irregularity	C, D, E, F
1b. Extreme torsional irregularity	C, D, E, F
2. Reentrant Corner Irregularity	D, E, F
3. Diaphragm Discontinuity Irregularity	D, E, F
4. Out-of-Plane Offset Irregularity	C, D, E, F
5. Non-Parallel System Irregularity	C, D, E, F

B. Vertical Irregularity

Vertical irregularity is a condition in which there are significant variations in the distribution of mass, stiffness, strength, or structural geometry along the height of a building. This irregularity is often found in multi-story buildings that have soft stories, differences in structural systems between floors, or an uneven distribution of mass. The types of vertical irregularity are shown in Table 2.

Table 2. Types of Vertical Irregularity

	Type of Vertical Irregularity	Applicable SDC
1a.	Stiffness–Soft Story Irregularity	D, E, F
1b.	Stiffness–Extreme Soft Story Irregularity	D, E, F
2.	Mass Irregularity	D, E, F
3.	Geometric Irregularity	D, E, F
4.	In-plane discontinuity in vertical lateral force-resisting element irregularity	C, D, E, F
5a.	Discontinuity in Lateral Strength–Weak Story Irregularity	C, D, E, F
5b.	Discontinuity in Lateral Strength–Extreme Weak Story Irregularity	C, D, E, F

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3. RESULTS AND DISCUSSION

The building structure model is shown in Figure 4.

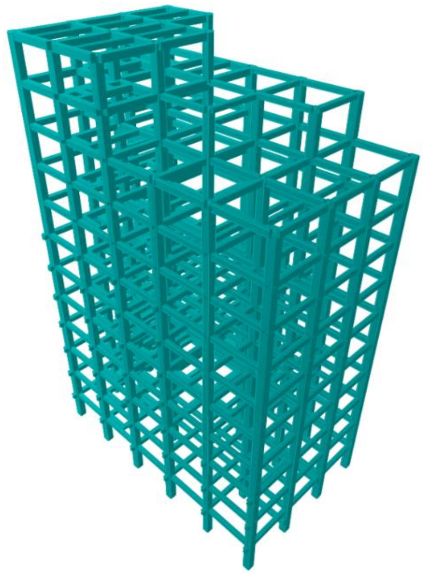

Figure 4. 3D Model

3.1. Preliminary Design

The dimensions of the beam were determined in accordance with the provisions of Section 9.3.1.1, SNI 2847:2019 [14], as shown in Table 3.

Table 3. Dimensions of the Beam

Beam	Length (mm)	Dimension (mm)
B1	6000	300 x 550
B1A	6000	350 x 600
B1B	3000	300 x 400
B2A	6000	400 x 650
B2B	6000	350 x 600
BB	6000	300 x 500

The column was designed to be 4 m tall, and its cross section was calculated based on all loads applied to the column using the tributary area method. The vertical loads acting on the column are dead loads and live loads, while the horizontal loads are wind loads and seismic loads. For the preliminary design, the column dimensions were 700 x 700 mm. The floor slab thickness design is based on SNI 2847:2019 (Table 4).

Table 4. Floor Slab Thickness

Slab of	Thickness (mm)
1^{st}-10^{th} Floor	140
11^{th} Floor	200
Top Floor	180

The minimum floor slab thickness is 139.333 mm, but because of the heavy loads on the 11th floor and the top floor, the floor slabs were designed to be thicker. Using the same method as for the floor slab design, the stair slab thickness was 125 mm.

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Pontianak City has an S_{D1} = 0.143 g and S_{DS} = 0.281 g; therefore, the hotel building falls under Risk Category II. Based on these data, it can be concluded that the building falls under SDC (Seismic Design Category) C. In accordance with Tables 1 and 2, irregularities that need to be evaluated for buildings with SDC C are horizontal irregularities of types 1a and 1b, types 4 and 5, as well as vertical irregularities of types 4 and 5b.

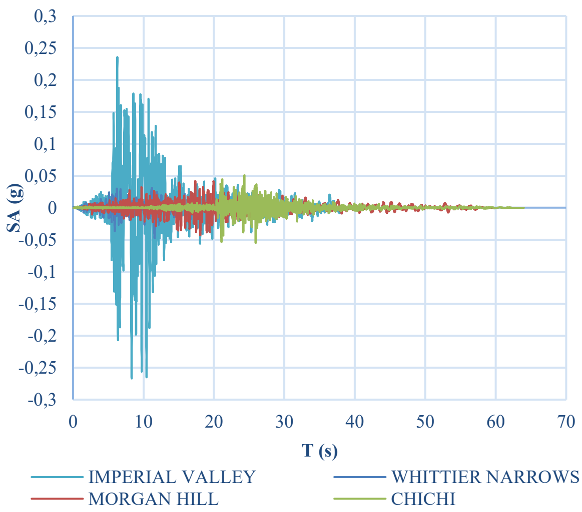
3.2. Linear Time-History Analysis

1. Time-History Data Scaling

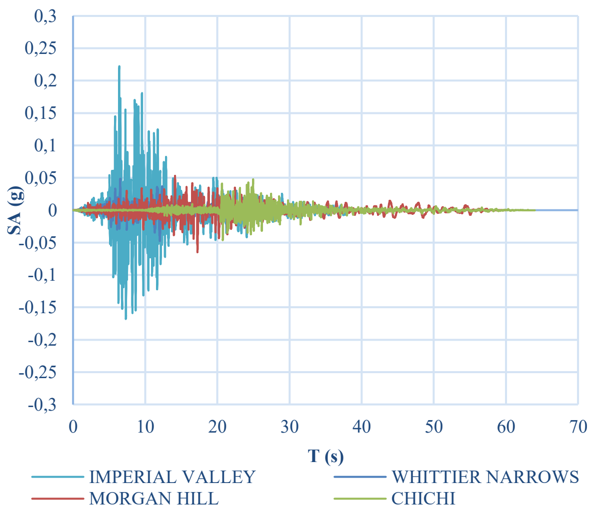
In accordance with Subsection 2.1, this study used four pairs of earthquake records. The maximum response from all records was used for each irregularity analysis, as shown in Table 5 and Figure 5.

Table 5. Selected Ground Motion Data Records

Name	M	R (km)	Vs (m/s)	Year	Station	RSN
IMPERIAL VALLEY	6.53	12.85	162.94	1979	El Centro Array #3	178
MORGAN HILL	6.19	53.89	116.35	1984	Foster City - APEEL 1	759
WHITTIER NARROWS	5.27	29.09	160.58	1987	Carson - Water St.	3697
CHICHI	5.9	68.12	169.52	1999	CHY047	2175



(a)



(b)

Figure 5. Original Time-History Accelerograms (a) X-Direction and (b) Y-Direction

The selected time-history records were matched to the spectral response of Pontianak, as shown in Figure 6. This matching was performed to scale the earthquake magnitude so that the simulation results would be representative of the actual location.

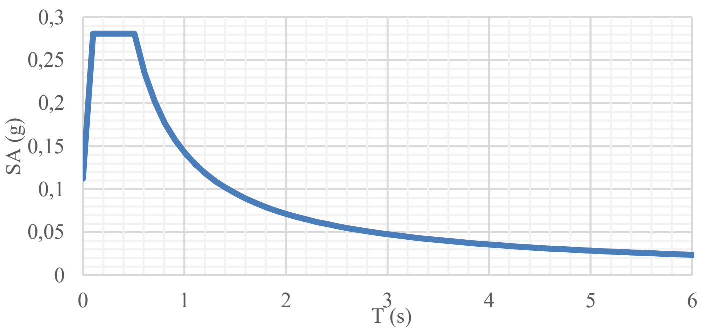


Figure 6. Response-Spectrum Curve of Pontianak

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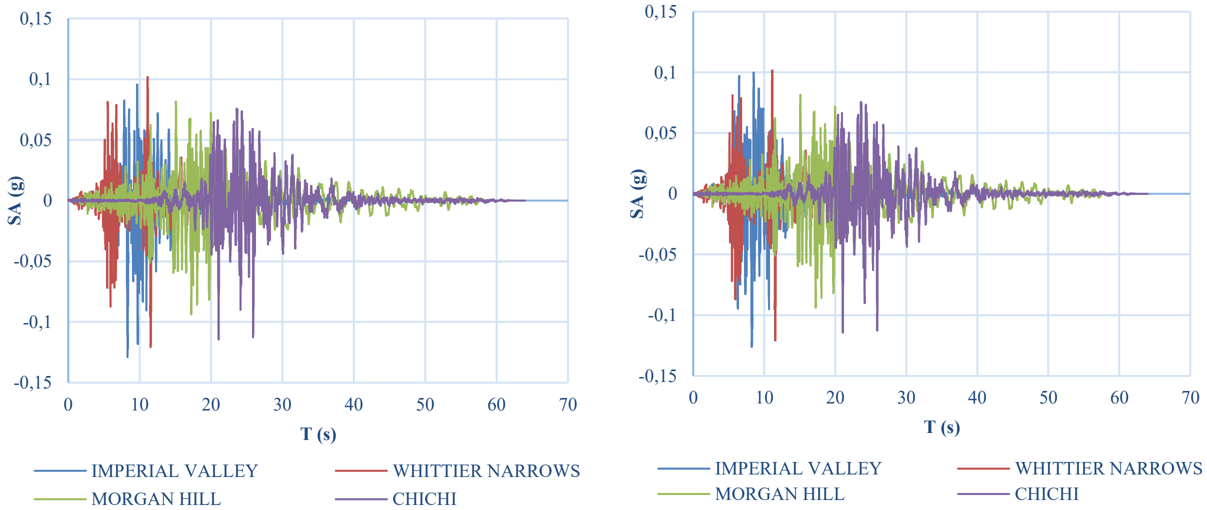
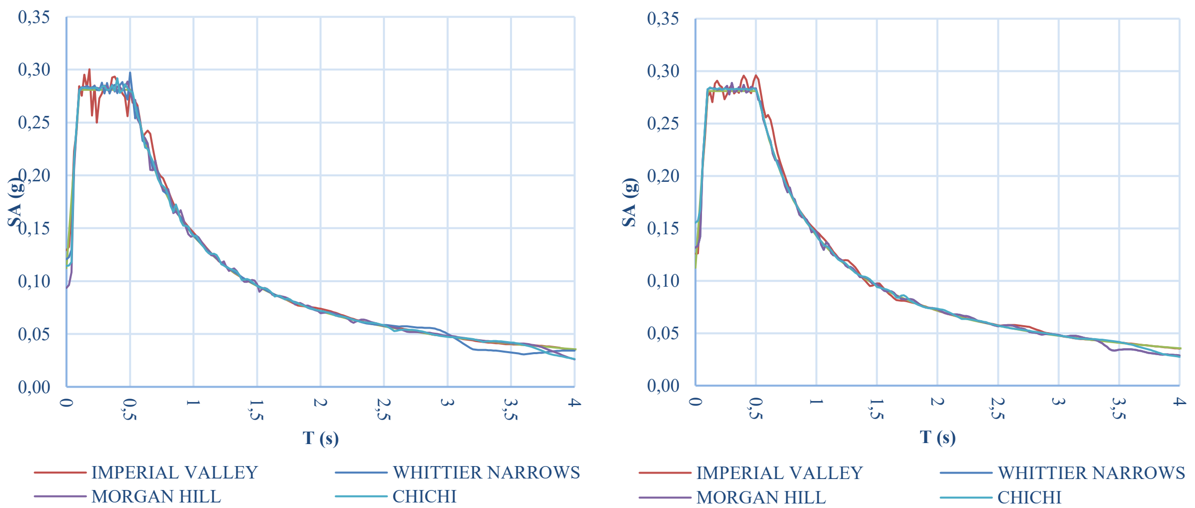
The results of matching the time-history accelerograms to the response-spectrum curve are shown in Figure 7.



(a) (b)

Figure 7. Matched Time-History Accelerograms (a) X-Direction and (b) Y-Direction

The matched time-history accelerograms were converted into response-spectrum curves and then averaged. As a condition for using the time-history data, the mean response-spectrum curve must fall between 90% and 110% of the target curve in the range of $0.8T_{lower}$ to $1.2T_{upper}$ (Figures 8a and 8b).



(a) (b)

Figure 8. Response-Spectrum Curves of Matched Time-History Records (a) X-Direction and (b) Y-Direction

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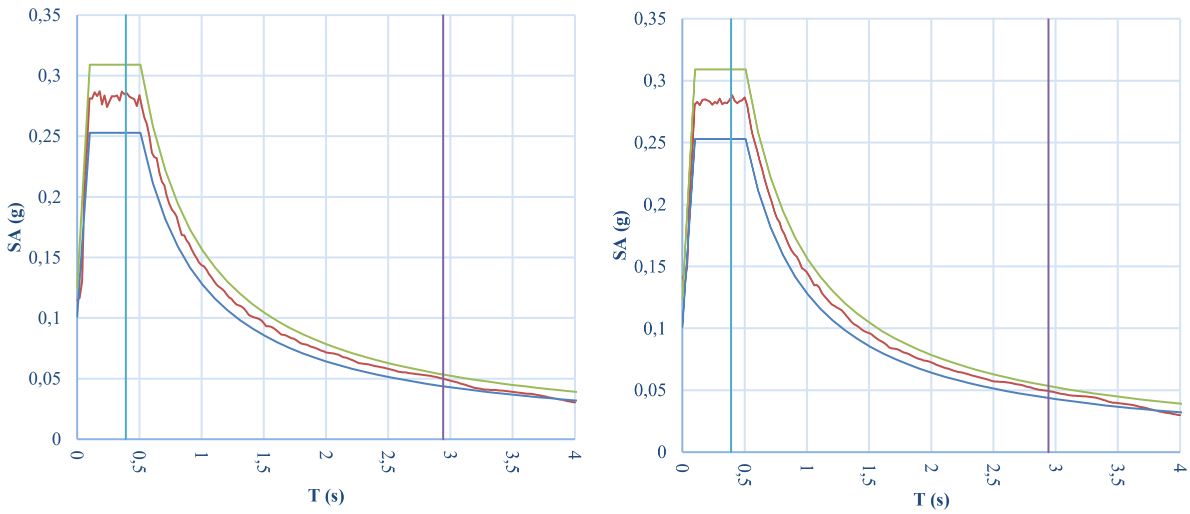
The value of $0.8T_{lower}$ is 0.381 s, and the value of $1.2T_{upper}$ is 2.942 s (see Table 8), as shown in Figure 9.



(a) (b)

Figure 9. Mean Response-Spectrum Curves of Matched Time-History Records (a) X-Direction and (b) Y-Direction

2. Base Shear Scaling

Based on the analysis, the structural weight (W) is 65,836.54 kN (Table 6).

Table 6. Effective Seismic Weight

Story	UX	UY
	kN	kN
Top Floor	2,449.84	2,449.84
Story11	5,478.18	5,478.18
Story10	5,505.94	5,505.94
Story9	5,738.11	5,738.11
Story8	5,738.11	5,738.11
Story7	6,076.62	6,076.62
Story6	5,807.28	5,807.28
Story5	5,807.28	5,807.28
Story4	5,807.28	5,807.28
Story3	5,850.42	5,850.42
Story2	5,794.51	5,794.51
Story1	5,782.97	5,782.97
Total	65,836.54	65,836.54

The seismic response coefficient (C_s) is 0.0124; therefore:
$V = C_s W = 0.0124 \times 65{,}836.54 = 813.887$ kN
Then, the base shear values were scaled using the Imperial Valley in the X direction as an example, as follows:

$$\textit{Base shear in the X direction} = \frac{\text{Scale Factor} \times \text{V}}{V_t} = \frac{1{,}125 \times 813.887}{324.817} = 3{,}070 \text{ mm/s}^2$$

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$$\textit{Base shear in the Y direction} = \frac{30\%\text{Scale Factor} \times \text{V}}{V_t} = \frac{368 \times 813.887}{324.817} = 923 \text{ mm/s}^2$$

The scaled seismic-force data are shown in Table 7.

Table 7. V_t with the Seismic Force Scale Factor (V/V_t)

Name	Max/Min	Fx (kN)	Fy (kN)	Vt (kN)	V/Vt	X	Y
IMPERIAL VALLEY X	Max	324.817	117.161	324.817	2.506	3,070	923
IMPERIAL VALLEY X	Min	-493.904	-126.961				
IMPERIAL VALLEY Y	Max	94.718	372.021	372.021	2.188	806	2,680
IMPERIAL VALLEY Y	Min	-138.911	-374.492				
WHITTIER NARROWS X	Max	376.076	107.835	369.282	2.204	2,700	812
WHITTIER NARROWS X	Min	-369.282	-101.714				
WHITTIER NARROWS Y	Max	109.813	370.631	357.906	2.274	837	2,786
WHITTIER NARROWS Y	Min	-103.976	-357.906				
MORGAN HILL X	Max	312.051	78.141	312.051	2.608	3,196	960
MORGAN HILL X	Min	-322.911	-77.991				
MORGAN HILL Y	Max	78.852	316.679	316.679	2.570	946	3,149
MORGAN HILL Y	Min	-83.405	-322.241				
CHICHI X	Max	374.870	110.849	374.870	2.171	2,660	799
CHICHI X	Min	-398.133	-108.565				
CHICHI Y	Max	117.553	375.431	374.341	2.174	801	2,664
CHICHI Y	Min	-115.806	-374.341				

3.3. Structural Analysis

1. Modal Participating Mass Ratio

The results of the structural analysis using 12 modes are presented in Table 8.

Table 8. Modal Participating Mass Ratio

Case	Mode	Period sec	SumUX	SumUY	SumRZ
Modal	1	2.589	0.6977	0.0378	0.057
Modal	2	2.521	0.7559	0.7368	0.0865
Modal	3	2.18	0.7875	0.7865	0.7865
Modal	4	0.81	0.8006	0.8661	0.7945
Modal	5	0.801	0.8871	0.882	0.7958
Modal	6	0.69	0.8914	0.8876	0.8887
Modal	7	0.45	0.8917	0.9258	0.8905
Modal	8	0.434	0.9299	0.9258	0.8917
Modal	9	0.385	0.9314	0.9282	0.9292
Modal	10	0.294	0.9326	0.9485	0.93
Modal	11	0.281	0.9521	0.9495	0.9303
Modal	12	0.257	0.9522	0.9507	0.9504

Table 8 shows that the combined modal mass participation with a total of 12 modes satisfies the requirements of Section 7.9.1.1 of SNI 1726:2019, with values of 95.22% in the X direction, 95.07% in the Y direction, and 95.04% in the Z direction. It can also be concluded that the first modal shape of the structure is translational in the X direction, followed by translation in the Y direction and then rotation about the Z axis.

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2. SMRF Requirements Check

Structural elements were checked in accordance with SMRF requirements as specified in SNI 2847:2019.

a. Floor Slab

The minimum thickness of the floor slab is 139.333 mm, with a minimum reinforcement ratio (ρ_{min}) of 0.18%. As shown in Table 9, the floor slab meets the requirements.

Table 9. Floor Slab Reinforcement

Slab of	Thickness (mm)	Location	Direction		Ratio (%)	
			X	Y	Top	Bottom
1^{st}-10^{th} Floor	140	End	M10-150	M10-150	0.51	0.54
		Middle	M10-150	M10-150	0.27	0.24
11^{th} Floor	200	End	D14-150	D14-150	0.48	0.57
		Middle	M10-150	M10-150	0.2	0.28
Top Floor	180	End	M10-150	M10-150	0.29	0.35
		Middle	M10-150	M10-150	0.13	0.15

b. Beam

To meet the SMRF requirements, a beam must have a clear span of at least 4 times its effective height, a width of at least 30% of the beam's height or 250 mm, and, since the columns are square, the beam's width must not exceed 75% of the column's cross-sectional dimension. The effective length of all beams is 5,300 mm. The complete results can be found in Table 10.

Table 10. Beam SMRF Check

Beam	Length (mm)	Dimension (mm)	4d (mm)	0.3h or 250 (mm)	0.75c (mm)	SMRF Check
B1	6000	300 x 550	1,970	250	525	OK
B1A	6000	350 x 600	2,156	250	525	OK
B1B	3000	300 x 400	1,368	250	525	OK
B2A	6000	400 x 650	2,350	250	525	OK
B2B	6000	350 x 600	2,162	250	525	OK
BB	6000	300 x 500	1,770	250	525	OK

The longitudinal reinforcement ratio of the beam for the SMRF is 0.34%. Based on Table 11, the beam reinforcement meets the requirements.

Table 11. Beam Reinforcement

Beam	Length (mm)	Dimension (mm)	Location	Rebar		Ratio (%)	
				Top	Bottom	Top	Bottom
B1	6000	300 x 550	End	4D22	3D19	0.75	0.36
			Middle	2D22	3D19	0.34	0.34
B1A	6000	350 x 600	End	5D22	3D22	0.76	0.34
			Middle	3D22	3D22	0.34	0.34
B1B	3000	300 x 400	End	3D16	3D16	0.34	0.34
			Middle	3D16	3D16	0.34	0.34
B2A	6000	400 x 650	End	5D22	3D22	0.77	0.34
			Middle	3D22	3D22	0.34	0.41
B2B	6000	350 x 600	End	5D22	5D19	0.8	0.34
			Middle	3D22	5D19	0.34	0.5
BB	6000	300 x 500	End	3D19	3D19	0.34	0.34
			Middle	3D19	3D19	0.34	0.34

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c. Column

For square columns, the minimum cross-sectional dimension is 300 mm, with reinforcement ranging from 1% to 6%. The column dimensions are 700 x 700 mm with a 20D19 reinforcement configuration, resulting in a reinforcement ratio of 1.157%. Another criterion for SMRF columns is the SCWB (Strong-Column-Weak-Beam) principle, as shown in Table 12.

Table 12. SCWB Check

Beam	$\Sigma M_{nc\text{-}Y}$ kNm	$1.2\Sigma M_{nb\text{-}X}$ kNm	Check	$\Sigma M_{nc\text{-}X}$ kNm	$1.2\Sigma M_{nb\text{-}Y}$ kNm	Check
B1	2,371.42	548.04	OK	2,403.15	548.04	OK
B1C	2,371.42	316.04	OK	2,403.15	316.04	OK
B2A	2,371.42	1,083.09	OK	2,403.15	1083.09	OK
B2B	2,371.42	476.16	OK	2,403.15	476.16	OK

Therefore, it can be concluded that all elements of the hotel building comply with SMRF requirements.

3.4. Interstory Drift

Absolute displacement (δ) is the displacement of a floor relative to its initial position or undeformed state due to loading, whereas interstory drift (Δ) is the difference in displacement between two consecutive floors. Thus, δ describes the absolute displacement of a floor as a whole, while Δ describes the relative displacement change between floors, indicating the deformation at a given floor level of a building. Therefore, the value of Δ is obtained from the difference between the absolute displacements of the floor above and the floor below. The parameters used to calculate the interstory drift are I_e, C_d, and SDC, with the following values:
- Seismic Importance Factor (I_e), apartment/hotel building = Risk Category II; therefore, Ie = 1.
- Deflection amplification factor (C_d), SMRF; therefore, Cd = 5.5.
- Allowable interstory drift (Δa), $\Delta a = 0.02 h_{sx} = 0.020 \times 4{,}000$ mm = 80 mm.
- SDC C, therefore Δa = 80 mm.

The most critical interstory drift was caused by the Morgan Hill earthquake in the Y direction, which resulted in a southward deflection of the building (negative), as shown in Table 13 and Figure 10.

Table 13. Interstory Drift (MORGAN HILL Y (-))

Story	δx (mm)	δy (mm)	Δx (mm)	Δy (mm)	hsx (mm)	Δxe (mm)	Δye (mm)	Check X & Y
Top Floor	-16.581	-37.354	3.003	-8.152	4,000	16.517	-44.836	OK
11	-19.584	-29.202	-1.050	-1.335	4,000	-5.775	-7.343	OK
10	-18.534	-27.867	-1.408	-1.789	4,000	-7.744	-9.840	OK
9	-17.126	-26.078	-1.703	-2.266	4,000	-9.367	-12.463	OK
8	-15.423	-23.812	-1.953	-2.731	4,000	-10.742	-15.021	OK
7	-13.470	-21.081	-2.155	-3.131	4,000	-11.853	-17.221	OK
6	-11.315	-17.950	-2.296	-3.432	4,000	-12.628	-18.876	OK
5	-9.019	-14.518	-2.365	-3.608	4,000	-13.008	-19.844	OK
4	-6.654	-10.910	-2.347	-3.643	4,000	-12.909	-20.037	OK
3	-4.307	-7.267	-2.164	-3.454	4,000	-11.902	-18.997	OK
2	-2.143	-3.813	-1.580	-2.825	4,000	-8.690	-15.538	OK
1	-0.563	-0.988	-0.563	-0.988	4,000	-3.097	-5.434	OK
Ground Floor	0.000	0.000	0.000	0.000	0.000	0.000	0.000	OK

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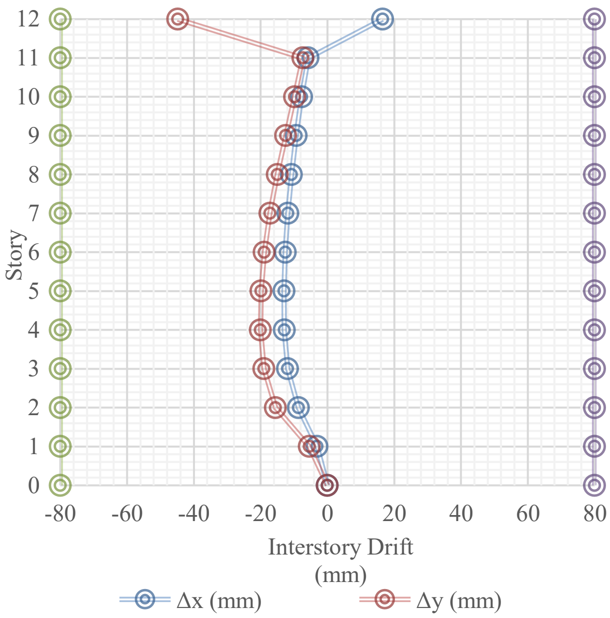

Figure 10. Interstory Drift (MORGAN HILL Y (-))

The largest interstory drift is 44.836 mm, which is less than the allowable interstory drift of 80 mm and therefore meets the requirements.

3.5. Horizontal Irregularity

1. Type 1a and 1b

Type 1 horizontal irregularity, or extreme torsional irregularity, exists if the maximum interstory drift including the effect of accidental torsion with Ax = 1.0 at one end of the structure transverse to an axis is more than 1.2 times the average interstory drift for Type 1a and 1.4 times for Type 1b at both ends of the structure. This criterion applies only to structures with rigid or semi-rigid diaphragms.
Torsional irregularity checks were performed on each floor, taking into account all seismic loading conditions, including accidental torsional eccentricity of ±5%. For each loading direction, the interstory deflections at both ends of the diaphragm were compared to determine the maximum and average deflections.
Type 1 horizontal irregularity with the largest drift ratio was caused by the Chichi earthquake in the X direction, with a center of mass eccentricity of 5% toward the north of the building (+Y), as shown in Table 14.

Table 14. Horizontal Irregularity Type 1a and 1b (Chichi X (+Y))

Story	Step Type	Max Drift	Avg Drift	Ratio	Type	Step Type	Max Drift	Avg Drift	Ratio	Type
		mm	mm				mm	mm		
Story1	Max	2.905	2	1.453	1b	Min	2.914	2.054	1.418	1b
Story2	Max	5.03	3.664	1.373	1a	Min	5.047	3.761	1.342	1a
Story3	Max	5.336	4.064	1.313	1a	Min	5.367	4.144	1.295	1a
Story4	Max	5.001	4.055	1.233	1a	Min	5.031	3.999	1.258	1a
Story5	Max	4.406	3.733	1.180	OK	Min	4.632	3.723	1.244	1a
Story6	Max	4.247	3.583	1.185	OK	Min	4.245	3.414	1.243	1a
Story7	Max	3.958	3.365	1.176	OK	Min	3.896	3.126	1.246	1a

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Story	Step Type	Max Drift	Avg Drift	Ratio	Type	Step Type	Max Drift	Avg Drift	Ratio	Type
		mm	mm				mm	mm		
Story8	Max	3.682	3.066	1.201	1a	Min	3.768	2.888	1.305	1a
Story9	Max	3.419	2.709	1.262	1a	Min	3.512	2.533	1.386	1a
Story10	Max	2.925	2.22	1.318	1a	Min	2.945	2.029	1.452	1b
Story11	Max	2.067	1.61	1.284	1a	Min	2.016	1.545	1.305	1a
Top Floor	Max	1.256	1.064	1.181	OK	Min	1.189	1.067	1.114	OK

The building exhibits Type 1a and 1b horizontal irregularities. This may be attributed to the building's vertical setback, and torsional irregularities occur on all floors except the top floor. According to SNI, the required procedure is a three-dimensional structural analysis with three degrees of freedom. This procedure includes diaphragm classification as rigid or flexible, accidental torsional moment amplification, verification of compliance with the allowable interstory drift, and a 5% eccentricity shift. All of these criteria have been met. However, torsional amplification is not mandatory for linear dynamic analysis using linear time-history analysis, as stated in SNI 1726:2019, Section 7.9.2.2.2.

2. Type 4

Irregularities resulting from out-of-plane offset displacement are defined as existing if there is a discontinuity in the path of lateral force resistance, such as a perpendicular displacement relative to a plane in at least one vertical element that carries lateral forces. Based on the floor plan of this hotel building, all column elements are continuous from the ground to the top floor; therefore, the structure does not exhibit any out-of-plane offset irregularities (Figure 11).

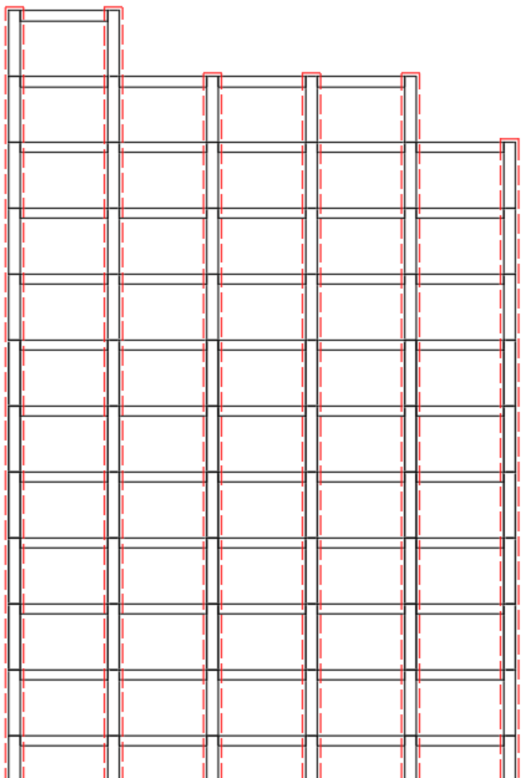

Figure 11. Building Lateral Force-Resisting Elements

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3. Type 5

Nonparallel system irregularity is defined as existing when the vertical elements carrying lateral forces are not parallel to the principal orthogonal axes of the seismic load-bearing system. Based on the floor plan of this hotel building, all column elements are parallel to the X-axis and Y-axis; therefore, it can be concluded that the structure does not exhibit nonparallel system irregularity (Figure 12).

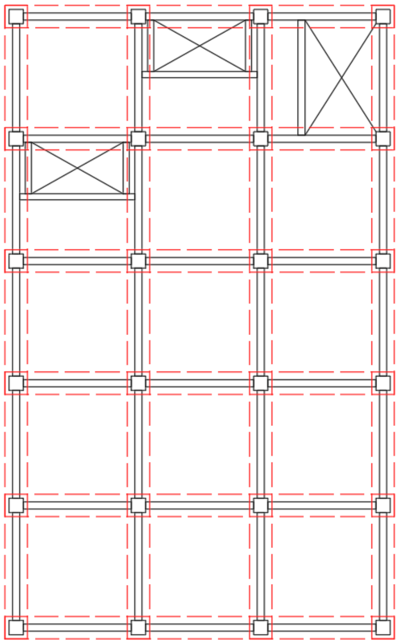

Figure 12. Building Elements Plan

3.6. Vertical Irregularity

1. Type 4

In-plane discontinuity in vertical lateral force-resisting element irregularity exists when there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements. Based on the structural plan of this hotel building (see Figure 11), all column elements are continuous from the ground floor to the top floor; therefore, the structure does not exhibit Type 4 vertical irregularity.

2. Type 5b

An extreme weak story is defined as a story whose lateral strength is less than 65% of the lateral strength of the story above it. The lateral strength of a story is defined as the total strength of all seismic-resisting elements that share the story's shear in the direction under consideration.
The lateral strength of the story is calculated using the following formula [15]:

$$G = \frac{12E}{h_i\left(\frac{1}{\Sigma\frac{I_y}{L_y}}+\frac{1}{\Sigma\frac{I_c}{h_i}}\right)} \tag{1}$$

where:
G = Story lateral stiffness (kN)
E = Elastic modulus (MPa)
h_i = Story height (m)
I_y = Moment of inertia of beam (m^4)

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L_y = Beam length (m)
I_c = Moment of inertia of column (m^4)

As shown in Table 15, the lowest lateral strength ratio is 99.369%, occurring on the 10th floor of the building. Therefore, the structure does not exhibit extreme weak story irregularity.

Table 15. Vertical Irregularity Type 5b

Story	Vx (kN)	Vy (kN)	65% X-Direction Check		65% Y-Direction Check	
Top Floor	713.378	637.152				
11	954.036	936.867	133.735%	OK	147.040%	OK
10	948.016	1014.177	99.369%	OK	108.252%	OK
9	948.016	1014.177	100.000%	OK	100.000%	OK
8	948.016	1014.177	100.000%	OK	100.000%	OK
7	948.016	1014.177	100.000%	OK	100.000%	OK
6	948.016	1014.177	100.000%	OK	100.000%	OK
5	948.016	1014.177	100.000%	OK	100.000%	OK
4	948.016	1014.177	100.000%	OK	100.000%	OK
3	948.016	1014.177	100.000%	OK	100.000%	OK
2	948.016	1014.177	100.000%	OK	100.000%	OK
1	948.016	1014.177	100.000%	OK	100.000%	OK

4. CONCLUSION

Based on the results of the hotel building's structural analysis, the interstory drift remains within the limits specified by SNI 1726:2019 (44.836 mm ≤ 80 mm). Regarding horizontal irregularities, there are Type 1a and 1b horizontal irregularities caused by the Chichi earthquake in the X direction with the highest ratio of 1.453 (≥ 1.4). There are no Type 4 or Type 5 horizontal irregularities. As for vertical irregularities, there are no Type 4 or Type 5b vertical irregularities (99.369% ≥ 65%). Thus, the existing irregularities are horizontal irregularities of types 1a and 1b. The results are valid within the assumptions of elastic linear analysis. It can be concluded that seismic evaluation of high-rise structures is crucial for structural stability, especially in buildings with asymmetrical and unique geometries.

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