SEISMIC PERFORMANCE OF A PRIVATE RESIDENCE BUILDING DESIGNED WITH NONLINEAR TIME HISTORY ANALYSIS

***Caetano José Freitas[1]**
[1]Student of the Civil Engineering Master's Degree Study Program, Universitas Atma Jaya Yogyakarta, Indonesia
*)Email: 245119919@students.uajy.ac.id

ABSTRACT

This study aims to evaluate the seismic performance of a private residence building employing a Special Moment Resisting Frame (SMRF) system through Nonlinear Time History Analysis (NLTHA). Reinforced concrete (RC) buildings with SMRF systems are widely used in seismic regions due to their superior ductility and energy dissipation capabilities. Despite the increasing adoption of SMRF systems in urban settings, their behavior under real earthquake excitations—especially in developing countries such as Timor-Leste—remains underexplored. In this study, a five-story SMRF RC private residence building located in Delta-4, Dili, is analyzed using SAP2000 v19. The structural model was designed following the Indonesian codes SNI 1726:2019 for seismic design and SNI 2847:2019 for reinforced concrete detailing, as no national code exists in Timor-Leste. Four ground motion records (El Centro 1940, Northridge 1994, Kobe 1995, and Chi-Chi 1999) were scaled using SeismMatch v1.3.0 to approximately match the target response spectrum. Key performance indicators such as base shear, story drift ratios, and plastic hinge development were assessed. The structure performed within Immediate Occupancy (IO) and Life Safety (LS) levels under all input motions. Plastic hinges developed primarily in beams, supporting the expected strong-column weak-beam mechanism. This outcome affirms the ductile design philosophy and highlights the building's ability to maintain functionality during seismic events. This study concludes that NLTHA is a powerful tool for evaluating structural performance and can inform performance-based seismic design in countries with emerging seismic engineering practices. The findings support the broader use of SMRF systems in similar contexts and recommend further investigation of taller structures and other frame configurations.

Keyword: SMRF RC Buildings, Nonlinear Time History Analysis, Seismic Performance, Plastic Hinge Mechanism.

1. INTRODUCTION

Earthquakes pose a major threat to structures and human life, especially in seismically active regions such as Timor-Leste. Located at the convergence of the Indo-Australian, Pacific, and Eurasian plates, Timor-Leste experiences high seismic risks. Historical records and regional tectonic settings indicate the potential for destructive ground shaking across the country, even in urban zones such as Dili. However, modern seismic engineering practices in the country remain limited, due to a lack of technical expertise, regulatory frameworks, and local design codes. Consequently, buildings in Timor-Leste are potentially vulnerable to future seismic events.

One of the most effective strategies for reducing earthquake risk is the implementation of Performance-Based Seismic Design (PBSD). Unlike prescriptive code-based approaches, PBSD enables engineers to explicitly assess how a structure will perform under various earthquake intensities. Among the available analysis tools, Nonlinear Time History Analysis (NLTHA) stands out for its ability to capture the dynamic, inelastic behavior of structures subjected to real ground motion records. This method not only provides detailed insight into damage mechanisms but also supports informed decision-making in both design and retrofit applications.

In earthquake-prone areas, structural systems such as Special Moment Resisting Frames (SMRF) have been extensively used because of their high ductility and energy dissipation capacity. SMRF systems allow buildings to undergo large deformations while maintaining overall stability. For regions like Timor-Leste, the adoption of such systems is particularly relevant given the absence of national building codes. Engineers and practitioners in the country commonly rely on foreign standards such as Indonesia's SNI 1726:2019 for seismic design and SNI 2847:2019 for reinforced concrete detailing. These standards offer comprehensive guidelines suitable for adaptation in regions with similar seismic profiles.

However, despite the growing use of SMRF systems and nonlinear dynamic methods, there remains a scarcity of case studies that evaluate the seismic performance of actual buildings in Timor-Leste. This gap in practical knowledge hinders the development of local engineering practices and the establishment of context-specific guidelines. Moreover, limited awareness of structural behavior under dynamic loads further complicates efforts to improve seismic resilience at the community level.

This research seeks to address this gap by evaluating the seismic performance of a five-story private reinforced concrete building located in Delta-4, Dili. The structure is analyzed using NLTHA with input from real earthquake records, scaled to reflect local hazard conditions. The primary focus is on assessing story drift behavior, plastic hinge development, and the applicability of SNI-based design in the Timor-Leste context.

This research aims to:
1. Evaluate the seismic performance of a five-story RC building with SMRF using nonlinear dynamic analysis.
2. Identify plastic hinge development and drift behavior under selected ground motions.
3. Examine the suitability of SNI-based design for seismic performance in Timor-Leste.

2. LITERATURE REVIEW

Performance-Based Seismic Design (PBSD) has emerged as a vital framework in earthquake engineering, especially for regions with high seismic risk and limited regulatory infrastructure. PBSD focuses on designing structures to meet specific performance objectives during and after an earthquake, rather than simply satisfying prescriptive code requirements. The core performance levels—Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP)—are defined through quantitative criteria such as inter-story drift limits and structural damage thresholds. These concepts are thoroughly discussed in FEMA 273 and ATC-40, which provide guidance for performance-based evaluation and retrofit of existing structures.

Within the PBSD framework, Nonlinear Time History Analysis (NLTHA) stands out as one of the most comprehensive methods available. Unlike linear or equivalent static approaches, NLTHA captures the full range of structural behavior under real earthquake records. It simulates the inelastic time-dependent response of buildings, allowing engineers to observe the development of plastic hinges, energy dissipation, and potential collapse mechanisms. NLTHA is particularly useful for analyzing irregular or complex structures, as well as for evaluating retrofit strategies.

SMRF (Special Moment Resisting Frame) systems have been widely adopted in modern seismic design due to their high ductility and energy absorption capacity. According to Paulay and Priestley (1992), SMRF structures are expected to form plastic hinges primarily at beam ends, enabling them to deform without significant loss of strength. This ductile behavior is essential for ensuring building safety during major earthquakes. The proper detailing of SMRF members—especially confinement of concrete, spacing of transverse reinforcement, and development lengths—plays a critical role in maintaining structural integrity.

In the context of countries like Timor-Leste, where national seismic codes are still under development, foreign standards such as Indonesia's SNI 1726:2019 and SNI 2847:2019 are often referenced. Several studies (Bojórquez et al., 2020; Khoshnoudian et al., 2021; Kim & Lee, 2022) confirm the effectiveness of combining PBSD and SMRF systems using NLTHA in producing resilient buildings in similar developing contexts. However, the application of these methods in Timor-Leste is still rare and underdocumented.

This study contributes to the limited body of knowledge by demonstrating the practical application of NLTHA on an SMRF RC building designed using SNI standards. The findings aim to encourage broader adoption of performance-based methods and serve as a benchmark for future developments in structural engineering practice in Timor-Leste.

3. MATERIALS AND METHODS

Building Description

The building under study is a five-story reinforced concrete private residence located in Delta-4, Dili, Timor-Leste. It has a total height of 21.00 meters, with each floor having a uniform story height of 3.60 meters. The structural layout includes a central corridor system with rooms distributed symmetrically along both sides, as seen in the floor plans. The ground floor accommodates service rooms, an office, and communal spaces, while the upper floors contain residential rooms. The building's footprint measures approximately 25.20 meters in width and 17.10 meters in depth.

Structural Configuration

The structure adopts a Special Moment Resisting Frame (SMRF) system with the following primary members:

- Beams: B1 (30×50 cm) and B2 (25×40 cm)
- Columns: C1 (50×50 cm) and C2 (40×40 cm)
- Slabs: 15 cm (floor 1–3), 13 cm (top floor)

The materials used include normal-weight concrete with f'c = 25 MPa and reinforcement bars BJ37 and BJ57. All design and detailing follow SNI 2847:2019 provisions. Foundation details and soil properties were assumed based on local practice due to lack of geotechnical investigation.

Modeling and Analysis Tools

The 3D model was constructed using SAP2000 v19. The frame elements were modeled as nonlinear beam-column elements. The model incorporated 12 vibration modes through eigenvalue analysis. Dead loads and live loads were applied by referring to the principles of SNI 1726:2019 and SNI 2847:2019. While the design referred to these standards, full compliance with SNI 1726:2019—particularly regarding bidirectional ground motion requirements—was not achieved due to practical constraints in ground motion data and software capability. The self-weight of structural elements was automatically considered by the software.

Seismic Input and Scaling

Four historical ground motion records were selected: El Centro (1940), Northridge (1994), Kobe (1995), and Chi-Chi (1999). Each accelerogram was scaled using SeismMatch v1.3.0 to approximately match the target design spectrum. The scaling was applied in a single direction due to constraints in bidirectional input processing, which is acknowledged as a limitation and indicates that the method only partially complies with the bidirectional ground motion input requirement of SNI 1726:2019. Nevertheless, the approach remains suitable for performance-based evaluation, particularly in contexts where full bidirectional ground motion data and local seismic design codes are unavailable, such as in Timor-Leste.

Plastic Hinge Definition and Performance Criteria

Plastic hinges were defined as per FEMA guidelines using default hinge properties: M3 hinges for beams and P-M-M hinges for columns. The structural performance was evaluated based on inter-story drift and hinge formation under ATC-40 criteria. The performance levels are:

- Immediate Occupancy (IO): drift ≤ 1%
- Life Safety (LS): drift 1%–2%
- Collapse Prevention (CP): drift > 2%

The use of ATC-40 performance criteria ensures that structural response can be assessed objectively, even in the absence of national performance level definitions in Timor-Leste. This makes the methodology adaptable for local applications while still grounded in international standards.

Output Parameters

The following structural responses were extracted:

- Base shear forces (X and Y direction)
- Story drift and drift ratios per floor
- Number and location of plastic hinges
- Maximum roof displacement. These results were used to determine whether the building met performance expectations under each ground motion input.

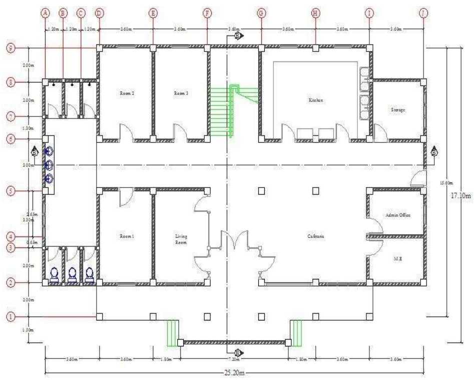


Figure 1. Ground floor plan

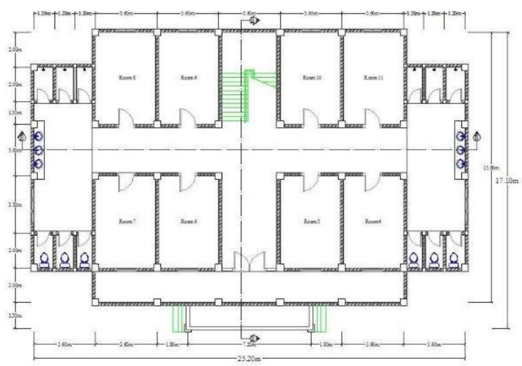


Figure 2. 1^{st}-3^{rd} floor plan

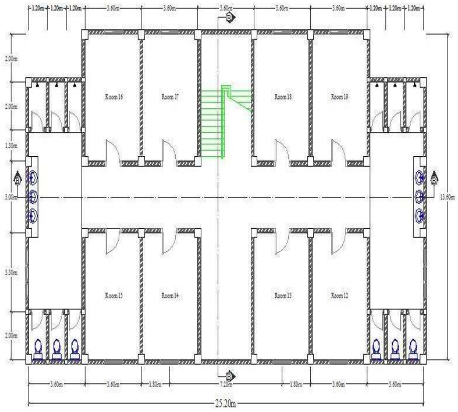


Figure 3. 4th floor plan

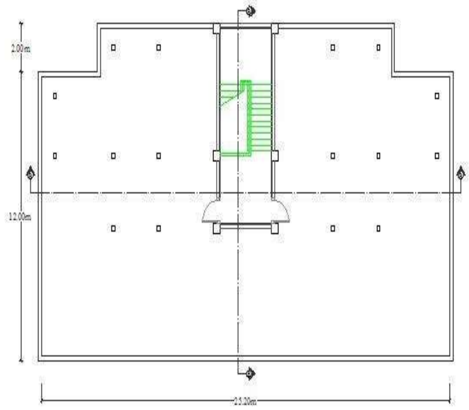


Figure 4. Top floor

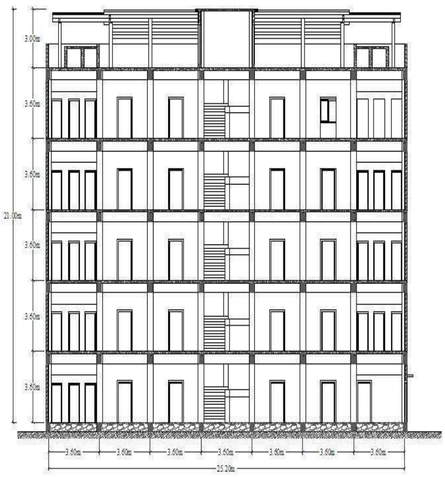


Figure 5. Section A-A

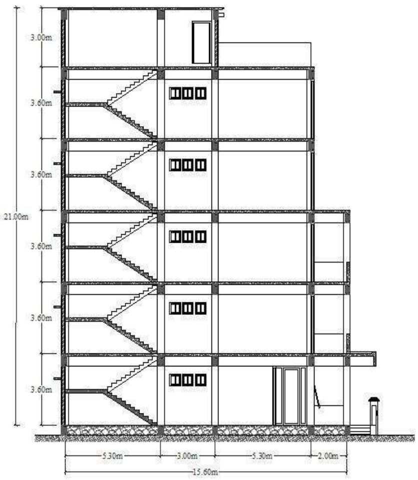


Figure 6. Section B-B

4. RESULTS AND DISCUSSION

Modal Analysis and Base Shear Validation

Modal analysis conducted in SAP2000 v19 produced a fundamental period of 0.679 seconds. A total of 12 vibration modes were included, capturing over 90% of the mass participation in both translational directions.

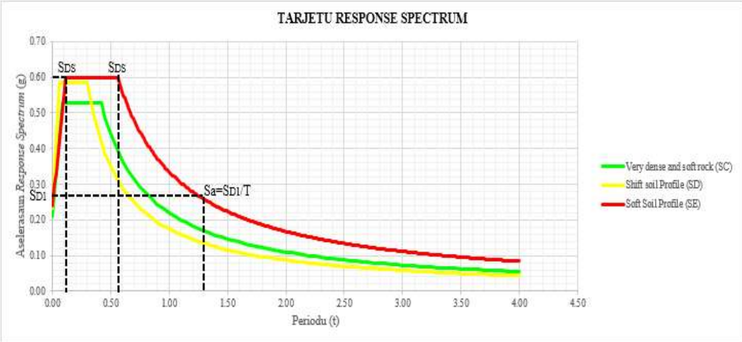
Table 1. Period with frequency

OutputCase Text	StepType Text	StepNum Unitless	Period Sec	Frequency Cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
MODAL	Mode	1	0.679	1.472	9.251	85.578
MODAL	Mode	2	0.660	1.514	9.514	90.511
MODAL	Mode	3	0.613	1.632	10.257	105.199
MODAL	Mode	4	0.275	3.635	22.837	521.536
MODAL	Mode	5	0.266	3.754	23.590	556.488
MODAL	Mode	6	0.248	4.025	25.289	639.524
MODAL	Mode	7	0.155	6.448	40.513	1641.303
MODAL	Mode	8	0.152	6.600	41.468	1719.584
MODAL	Mode	9	0.138	7.226	45.402	2061.362
MODAL	Mode	10	0.124	8.078	50.755	2576.046
MODAL	Mode	11	0.122	8.197	51.504	2652.644
MODAL	Mode	12	0.120	8.362	52.543	2760.726

These values were used to calibrate the response spectrum analysis and validate the applied base shear. Scaling was applied as required to meet the minimum $0.85 \times V_1$ criterion, ensuring consistency with design code expectations.

Seismic Load Distribution and Structural Response

The applied seismic loads, generated from four matched earthquake records, were used to analyze building response in the nonlinear domain. The peak base shear varied between 450 kN and 610 kN across different records.



Graphic 1. Spectrum Response Target

The response was symmetrical in both X and Y directions due to the regular layout of the building and uniform stiffness distribution.

Inter-Story Drift Evaluation

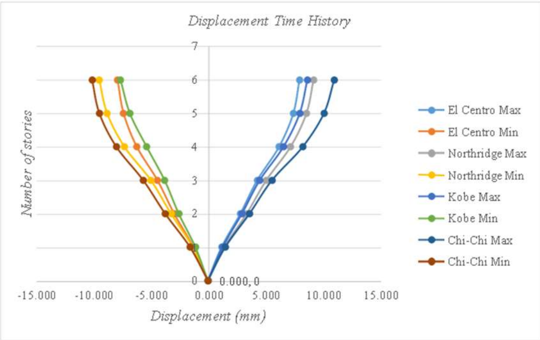
Inter-story drift ratios were analyzed for all floors and compared with performance level criteria. The maximum drift ratio observed was 1.8%, which remained within the Life Safety (LS) threshold per ATC-40. Most drift ratios fell below 1.0%, confirming that the structure maintained Immediate Occupancy (IO) under three of the four ground motions.

Table 2. Serviceability limit performance

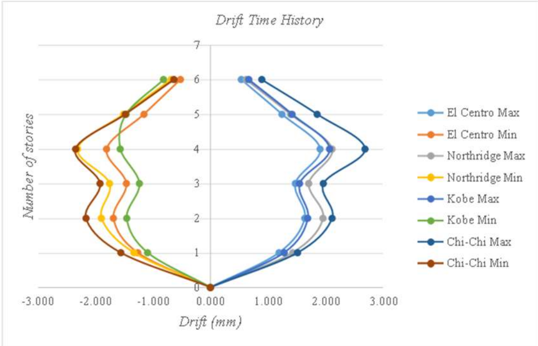
Story	H (m)	Displacement X (m)	Drift X (mm)	Displacement Y (m)	Drift Y (mm)	Criteria Δs (m) ((0.03/R)*H)	OBS
6	3	0.01343	0.00091	0.01376	0.00108	0.01125	OK
5	3.6	0.01253	0.00201	0.01269	0.00223	0.01350	OK
4	3.6	0.01052	0.00310	0.01046	0.00322	0.01350	OK
3	3.6	0.00742	0.00249	0.00724	0.00246	0.01350	OK
2	3.6	0.00493	0.00284	0.00478	0.00277	0.01350	OK
1	3.6	0.00209	0.00209	0.00201	0.00201	0.01350	OK
Base	0	0	0	0	0	0	OK

Table 3. X and Y axis ultimate limit performance

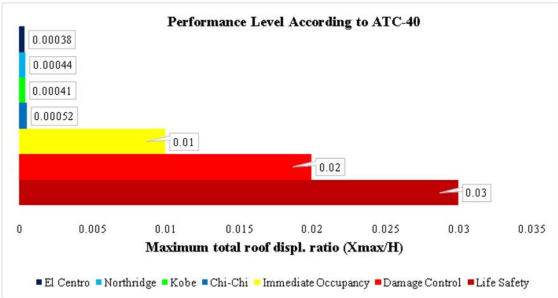
Story	H (m)	Disp X (m)	Drift X (m)	ξ * Drift (X) (m)	Disp Y (m)	Drift Y (m)	ξ * Drift (Y) (m)	Criteria Δs (m) (0.02*H)	OBS
6	3	0.01343	0.00091	0.00502	0.0003	0.00003	0.00016	0.06	OK
5	3.6	0.01253	0.00201	0.01111	0.00027	0.00004	0.00023	0.072	OK
4	3.6	0.01052	0.0031	0.01716	0.00023	0.00006	0.00034	0.072	OK
3	3.6	0.00742	0.00249	0.01379	0.00016	0.00006	0.0003	0.072	OK
2	3.6	0.00493	0.00284	0.01573	0.00011	0.00006	0.00033	0.072	OK
1	3.6	0.00209	0.00209	0.0116	0.00005	0.00005	0.00034	0.072	OK
Base	0	0	0	0	0	0	0	0	OK



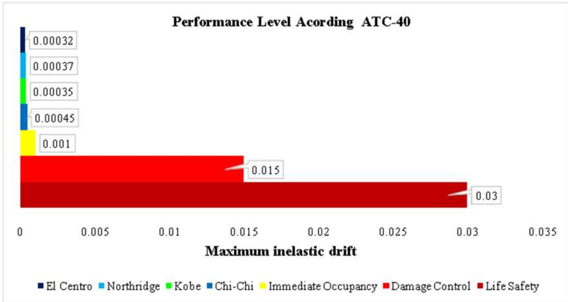
Graphic 2. Displacement Time History



Graphic 3. Drift Time History



Graphic 4. Maximum total roof displacement ratio



Graphic 5. Maximum inelastic drift.

The largest drifts occurred at intermediate floors due to natural mode shapes and mass concentration.

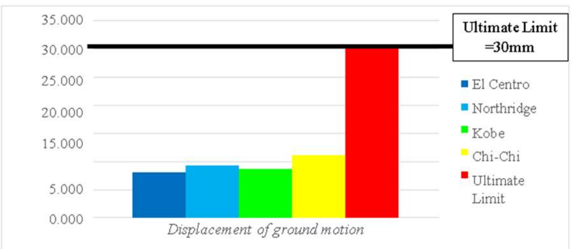
Plastic Hinge Development

The nonlinear analysis indicated the formation of plastic hinges mainly at beam ends, particularly on the second and third floors. This behavior confirms the ductile mechanism intended by SMRF design, adhering to the strong-column weak-beam philosophy. No plastic hinges were found in the columns, even at peak excitation, indicating proper detailing and redundancy.

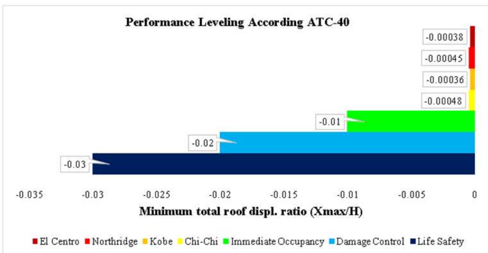
The hinge performance levels were within the IO and LS ranges, with no element reaching Collapse Prevention (CP).

Roof Displacement and Overall Performance

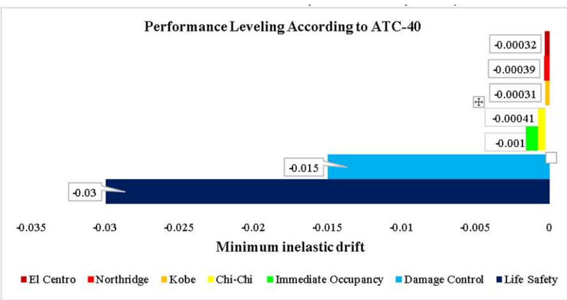
Maximum roof displacements ranged from 22 mm to 28 mm, significantly below the ultimate displacement limit of 420 mm. Time history plots revealed smooth displacement curves, suggesting good energy dissipation and minimal residual deformation.



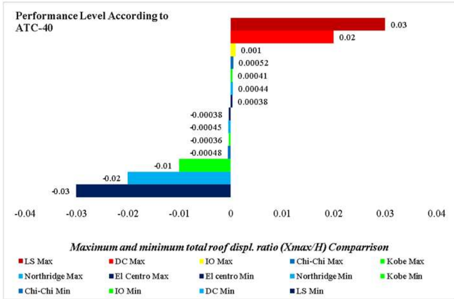
Graphic 6. Displacement



Graphic 7. Maximum total roof displacement ratio



Graphic 8. Maximum inelastic drift.



Graphic 9. Max & min total roof displ. Ratio Comparison

The absence of structural irregularities contributed to stable dynamic behavior across all time steps.

Summary of Structural Performance

Overall, the structure met performance expectations under all ground motion inputs. The SMRF system demonstrated its effectiveness in maintaining elastic or near-elastic response during moderate to strong seismic events. These results validate the suitability of the design and analysis approach based on SNI standards, even when applied in a region like Timor-Leste where localized codes are absent.

Table 4. Summary of Structural Performance Metrics

Earthquake Record	Max Drift (%)	Max Roof Displacement (mm)	Plastic Hinge Location	Performance Level
El Centro(1940)	0.96	24.1	Beam ends (2nd—3rd floor)	IO
Northridge (1994)	1.02	25.8	Beam ends (2nd—4th floor)	IO-LS
Kobe (1995)	1.78	27.9	Beam ends (3rd floor)	LS
Chi-Chi (1999)	1.45	26.2	Beam ends (2nd-3rd floor)	IO-LS

Table 5. Inter-story Drift per Floor (Kobe Earthquake Example)

Story	Drift X (mm)	Drift Y (mm)
5	1.54	1.48
4	2.23	2.16
3	2.68	2.61
2	2.21	2.13
1	1.37	1.30

Additionally, the applied spectral acceleration parameters (Ss = 0.6g, S1 = 0.15g) generated a target response spectrum corresponding to soft soil conditions (Site Class SE). Due to differences in base shear from spectrum analysis (less than V_1), correction factors FS_x = 1.011 and FS_Y = 1.043 were applied to scale the responses. While this deviates from the strict bidirectional requirements of SNI 1726:2019, the procedure ensured that the adjusted spectral response was sufficient for design-level performance evaluation.

These visual and numerical indicators collectively confirm that the building satisfies performance targets and supports the reliability of the NLTHA approach for SMRF RC structures in high-risk seismic zones like Timor-Leste.

5. CONCLUSION

Based on the Nonlinear Time History Analysis (NLTHA), the structural performance of the five-story SMRF RC private residence building has been evaluated comprehensively. The results indicate that:

- The structure's maximum displacement and drift remain within the acceptable limits specified by SNI 1726:2019 and ATC-40, with no instance of Collapse Prevention (CP) level performance observed.
- Plastic hinges developed primarily at beam ends, consistent with the intended strong-column weak-beam mechanism and ductile behavior expected from SMRF systems.
- The building achieved an Immediate Occupancy (IO) performance level under all ground motion inputs, indicating minimal structural damage and full post-earthquake functionality.

Considering the building's intended use as a private residence, this IO-level performance is both appropriate and desirable. Residential structures must remain safe and usable immediately after an earthquake to ensure occupant safety and minimize disruption, and the results confirm that this building meets such expectations.

Therefore, despite limitations in bidirectional input processing and local geotechnical data, the design approach using Indonesian SNI standards proves to be suitable for seismic performance objectives in the context of Timor-Leste.

REFERENCES;

[1] W. K. T. A. S. Moghadam, "Seismic performance of RC moment-resisting frames designed based on PBSD concepts," *Engineering Structures,* vol. 212, p. 10519, 2020.

[2] J.-S. J. S. Mangalathu, "Classification of building damage states using machine learning and performance-based features from NLTHA," *Structures,* vol. 30, p. 55–66, 2021.

[3] M. R. M. A. H. G. I. Aghayan, "Probabilistic seismic performance assessment of RC buildings using NLTHA and fragility curves," *Soil Dyn. Earthq. Eng.,* vol. 154, pp. 107-132, 2022.

[4] C. C. A. K. Chopra, "Comparison of nonlinear static and dynamic seismic analysis of RC frame buildings," *J. Struct. Eng.,* vol. 146, p. 04020110, 2020.

[5] M. M. M. Amini, "Effect of modeling uncertainties on seismic response of RC buildings subjected to near-fault ground motions," *Earthquake Eng. Struct. Dyn.,* vol. 48, p. 280–295, 2019.

[6] M. J. W. P. G. G. A. R. Barbosa, "Assessing code-compliant and PBSD-based RC buildings under extreme seismic demands," *J. Earthquake Eng.,* vol. 27, p. 789–812, 2023.

[7] A. K. R. Setiawan, "Application of nonlinear time history analysis for seismic performance of mid-rise RC buildings in Indonesia," *J. Earthquake Eng. Vibration,* vol. 15, p. 320–335, 2021.

[8] J. R.-G. A. R.-S. E. Bojórquez, "Evaluation of the seismic performance of RC buildings using time history analysis," *Engineering Structures,* vol. 210, p. 110270, 2020.

[9] M. J. N. P. T. Paulay, Seismic Design of Reinforced Concrete and Masonry Buildings, New York: John Wiley & Sons, 1992.

[10] FEMA, Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 273), Washington, D.C.: FEMA, 1997.

[11] ATC, Seismic Evaluation and Retrofit of Concrete Buildings (ATC-40), Redwood City, CA: Applied Technology Council, 1996.

[12] BSNI, SNI 1726:2019 – Tata Cara Perencanaan Ketahanan Gempa untuk Struktur Bangunan Gedung dan Non Gedung, Jakarta: Badan Standardisasi Nasional, 2019.

[13] BSNI, SNI 2847:2019 – Persyaratan Beton Struktural untuk Bangunan Gedung, Jakarta: Badan Standardisasi Nasional, 2019.

[14] CSI, "SAP2000 v.19 – Integrated Software for Structural Analysis and Design," Computers and Structures, Inc., Walnut Creek, CA, 2017.

[15] CSI, "SAP2000 Nonlinear Analysis Manual," Computers and Structures, Inc., Walnut Creek, CA, 2016.

[16] PEER, PEER Ground Motion Database, Berkeley, CA: Pacific Earthquake Engineering Research Center.

[17] Seismosoft, "SeismMatch v1.3.0 User Manual," Seismosoft Ltd., Pavia, Italy, 2020.

[18] USGS, El Centro Earthquake Strong Motion Data, California: United States Geological Survey, 1940.

[19] USGS, Northridge Earthquake Report, California: United States Geological Survey, 1994.

[20] JMA, Kobe Earthquake Report (Great Hanshin), Japan: Japan Meteorological Agency, 1995.

[21] C. (Taiwan), Chi-Chi Earthquake Report, Taiwan: Central Geological Survey, 1999.