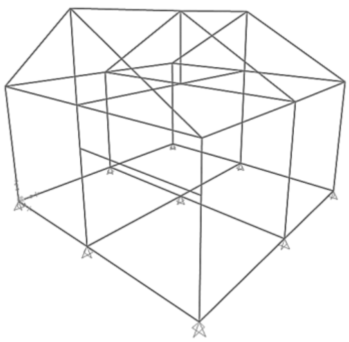


Figure 3 Numerical model of Non-Engineered Building

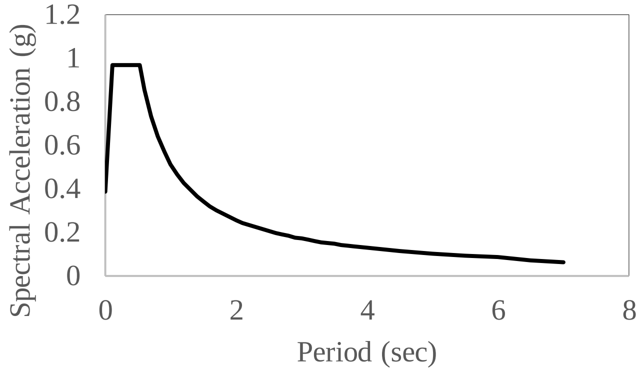


Figure 4 Load Response Spectrum used in this study

3. Results and Discussion

3.1 Linear Analysis

In this study, linear analysis method was firstly performed in SAP2000 [12] to investigate the demand to capacity ratio (DCR) of each main structural element. In the analysis, two different values of base shear are investigated: 1) based on analytical value provided in the Indonesian National Code for Earthquake (SNI 1726:2019) [3] 2) modal analysis. The fundamental period obtained from the modal analysis is compared with the range of acceptable natural period required by SNI 1726:2019 [3] to determine the design base shear.

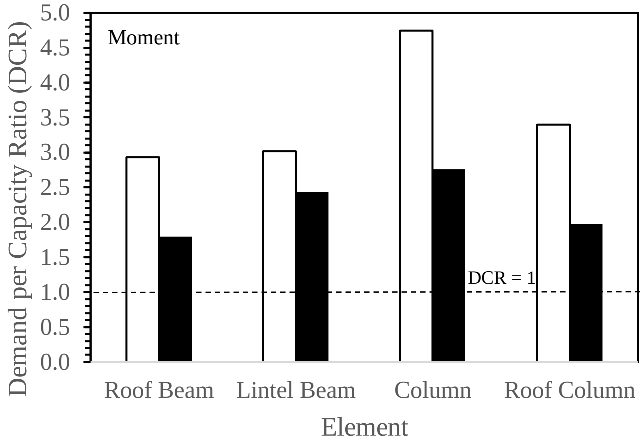
After determining the design base shear, the demand to capacity ratio (DCR) is calculated for each structural element: roof beam, lintel beam, column, and roof column. For conciseness, DCR calculation is only shown for one value of concrete strength, fc' 10 MPa with two different rebar strengths, fy = 240 MPa and fy=420 MPa. From the most critical DCR value, a reduction factor namely Response Modification Factor (R) is determined in order to reduce the elastic base shear to fulfill scenario: DCR = 1.0. In mathematical expression, the relationship can be written as:

$$DCR_{Service} + \frac{DCR_{Eq}}{R} = DCR = 1.0$$

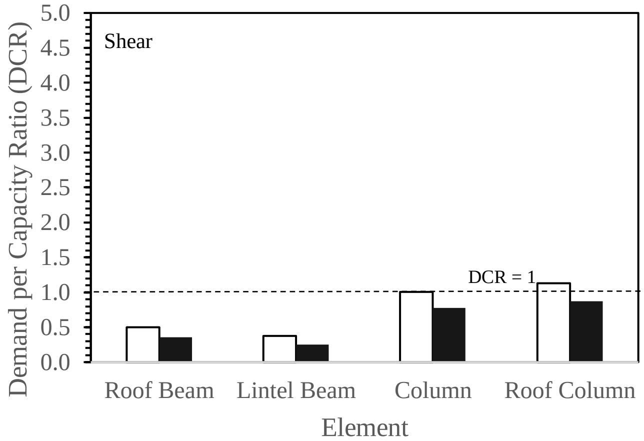
$DCR_{Service}$ is obtained from an analysis of service condition (1.4) of 1.0 D + 0.25 (Lr) whereas DCR_{Eq} is obtained from earthquake load combinations (2.1) and (2.2). This calculated R parameter gives a direct representation of the ductility requirement of the structure. Higher value of R indicates higher requirement for strict seismic detailing to ensure significant ductility of the structure.

Table 3 Base Shear Forces and Response Modification Factor (R) for Linear Analysis

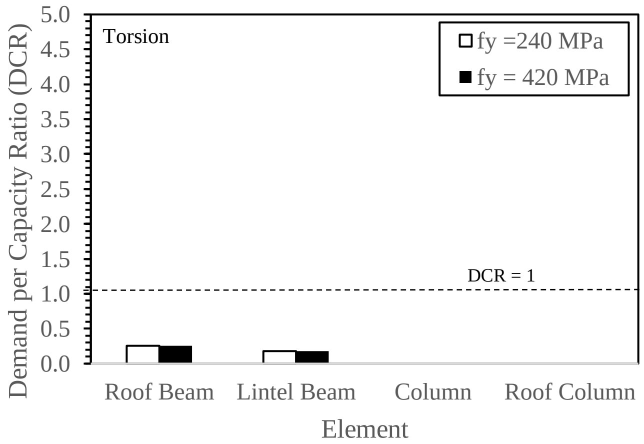
fc' (MPa)	fy (MPa)	Elastic Base Shear (kN)		Reduced Base Shear (kN)		R
		EQ X	EQ Y	EQ X	EQ Y	
10	240	83.00	83.02	9.26	9.26	8.96
	420	83.00	83.02	27.67	27.67	3.00



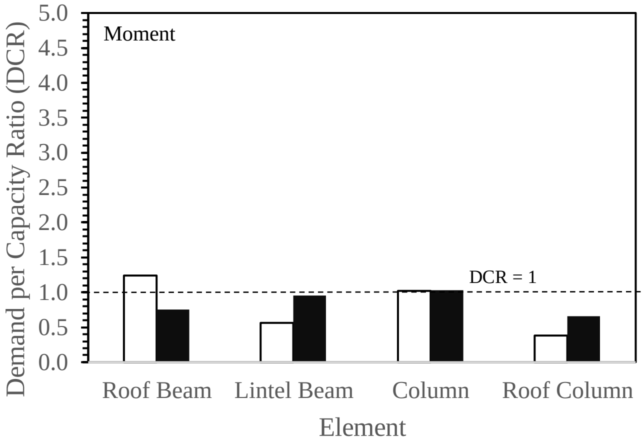
(a) Moment DCR for Elastic Base Shear



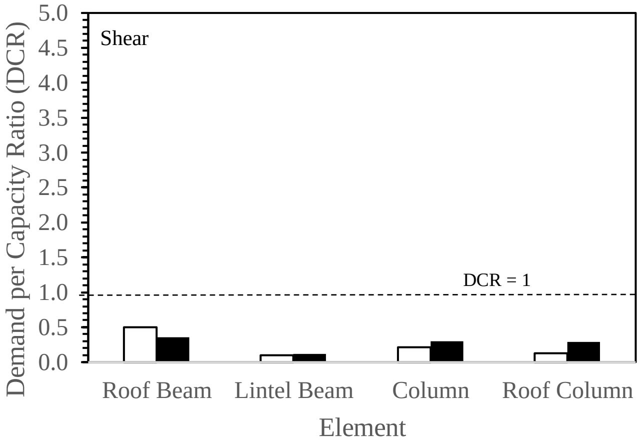
(b) Shear DCR for Elastic Base Shear



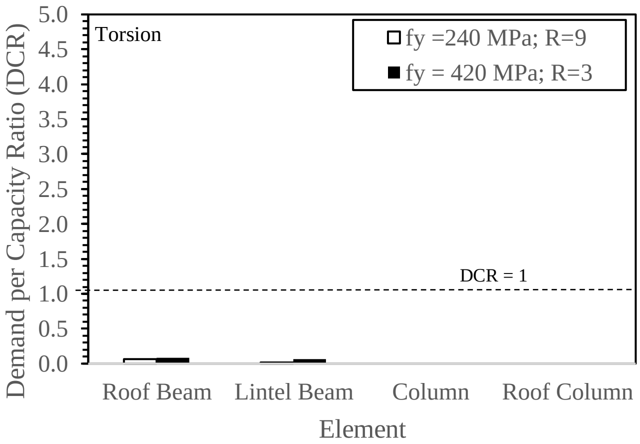
(c) Torsion DCR for for Elastic Base Shear



(d) Moment DCR for Reduced Base Shear



(e) Shear DCR for Reduced Base Shear



(f) Torsion DCR for Reduced Base Shear

Figure 5 DCR Value for Moment, Shear and Tension at various elements for unreduced and reduced seismic load

Table 3 shows value of the elastic base shear and the reduced value to satisfy DCR ≤ 1 for both fy 240 MPa and 420 MPa. The resulting DCR value of moment, shear and torsion at various elements at the structure can be seen in Figure 5. Figure 5 (a-c) show the DCR results for the obtained base shear seismic value, and it is clear from Figure 5 (a-c) that the DCR value for bending moment is critical. It can be seen that for model with fy=420 MPa, R equals 3. However, for fy=240 MPa, even with a seismic reduction as high as R=9, DCR value is not satisfied for the roof beam. This shows that rebar quality is more important than concrete strength and that with an fy of 420 MPa the structure can be considered an Ordinary Moment Frame. It is also notable that there is a change of critical element for fy=240 MPa with reduced seismic load, before reduction the critical element was at the columns, but after reduction the critical element is at the roof beam, this is because the roof beam capacity is not adequate to sustain gravity load alone, leaving no reserve capacity for seismic load combination. Thus, the DCR for roof beams always exceed 1 regardless of the R value taken.

Non-linear pushover analysis

Performance-based seismic engineering is focused on the definition of limit states to represent different levels of damage, which can be described by material strains, rotations, displacements or even changes in dissipating properties of the structure. In this study, the performance of the structure and failure mode is investigated by nonlinear static pushover analysis. The objective of the analysis is to determine the performance point, an intersection between the capacity curve and the demand curve, through iterations.

To obtain the capacity curve, the incremental lateral load (static pushover load) is applied to the structure. Then, the deformation of the structure is recorded according to its corresponding forces to obtain the force-deformation relation. In accordance with ASCE 41-17 [13], the gravity load combination (1.4) of 1.0 D + 0.25 (Lr) is applied to the structure prior to the static pushover load for X and Y directions. For this analysis, two different earthquake levels were investigated, namely the Design Based Earthquake (DBE), which is the design response spectrum used in linear analysis, and Risk-Targeted Maximum Considered Earthquake (MCE_R) with 475-year and 2475-year return period respectively.

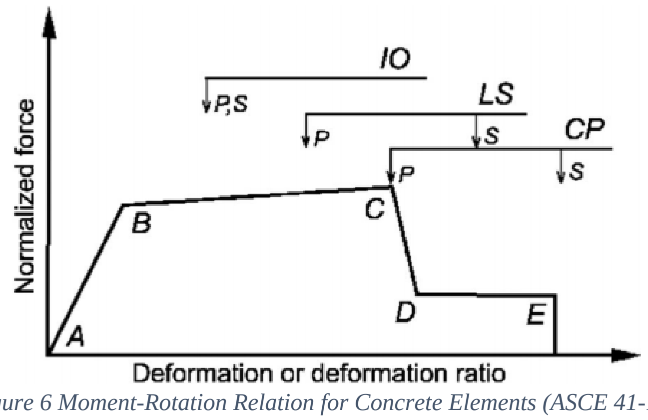
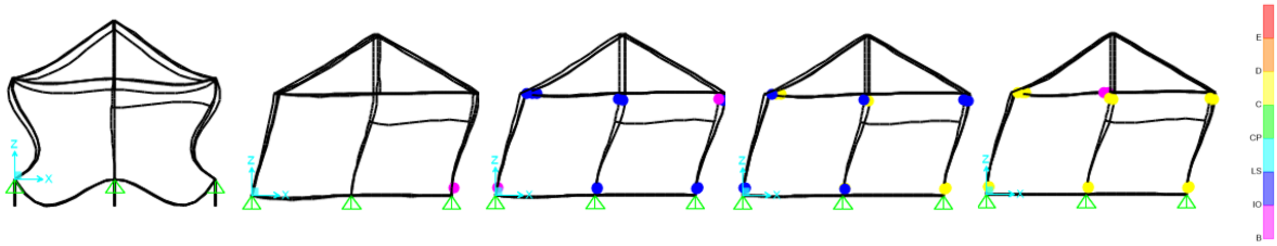


Figure 6 Moment-Rotation Relation for Concrete Elements (ASCE 41-17)

Table 4 Modelling Parameters and Acceptance Criteria for RC Beams in Nonlinear Procedures (ASCE 41-17)

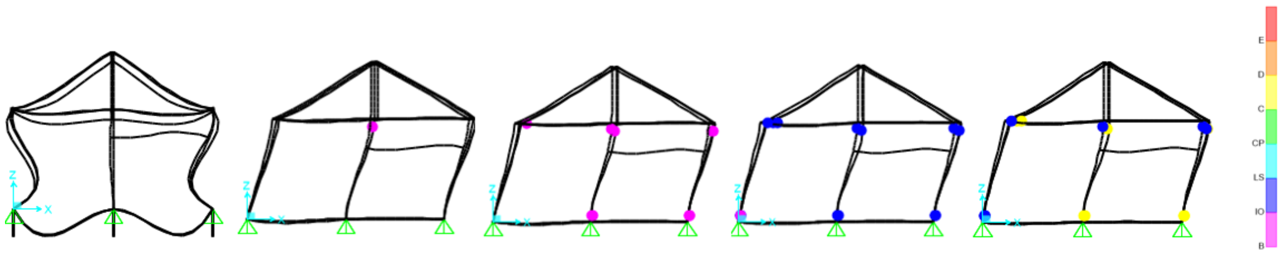
Modelling Parameters			Acceptance Criteria		
Plastic Rotation (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians)		
			Performance Level		
a	b	c	IO	LS	CP
0.02	0.03	0.2	0.005	0.02	0.03

Table 5 shows value of elastic base shear obtained from linear analysis and the yield base shear obtained from the nonlinear pushover analysis for both fy 240 MPa and 420 MPa. Although slightly lower, the response modification factor (R) from nonlinear analysis is in agreement with the value obtained from linear analysis (see Table 3). For model using fy 420 MPa, the R value represents the strength and ductility equivalent to that of Ordinary Moment Frames (OMF) specified in SNI 1726:2019 [3] (R = 3). Whereas for fy 240 MPa, the R value is equivalent to Intermediate Moment Frames (IMF) (R = 5). However, it is important to note that more stringent detailing requirement is needed for IMF to develop its expected ductility as a consequence of its reduced elastic strength. SNI 2847:2019 [15] code for structural concrete provides detailing requirement to ensure ductility for OMF and IMF. Data from field survey (see Table 1) shows that detailing of non-engineered houses do not fulfill the IMF requirement, mainly due to inadequate hoop spacing, incompliance of strong column-weak beam philosophy, and the use of plain rebars.



(a)	(b)	(c)	(d)	(e)
Gravity	*[Drift 5.56E-03] Plastic hinges form at bottom column*	*[Drift 1.47E-02] DBE Performance Point*	*[Drift 2.52E-02] MCER Performance Point*	*[Drift 6.67E-02] End of Static Pushover*

Figure 8 Failure Mode Behavior for Model with fy = 240 MPa in X-Direction



(a)	(b)	(c)	(d)	(e)
Gravity	*[Drift 1.06E-02] Plastic hinges form at top column*	*[Drift 1.39E-02] DBE Performance Point*	*[Drift 2.72E-02] MCER Performance Point*	*[Drift 3.23E-02] End of Static Pushover*

Figure 9 Failure Mode Behavior for Model with fy = 420 MPa in X-Direction

Failure Mode Behavior

The failure mode behavior of both the model using fy 240 MPa and 420 MPa with DBE and MCE for X can be seen in Figure 8. The plastic hinges firstly form at the bottom of the column for fy 240 MPa (see Figure 8) and at the top of the column for fy 420 MPa (see Figure 9), showing that the structure has more brittle behavior with fy 420 MPa. Plastic hinges then start to form at the beam once the plastic rotation at the bottom of the columns allow the structure to sway properly. After reaching the DBE and MCE_R performance of the elements consecutively, the plastic rotation at the beams and columns continue to increase until the end of the state where the elements drop to its residual strength of 0.2 of its initial strength (see Table 4). From Figure 5, it can be noted that the failure mode behavior in nonlinear analysis is aligned with DCR result in linear analysis where the columns, the first element reaching failure state in the static pushover, have the highest bending moment DCR compared to other structural elements, followed by beams.

A summary of the plastic hinge status for DBE and MCE_R for different fy, direction and elements can be seen in Table 6 (see Figure 6 for illustration). The lowest performance of element occurred with DBE is IO to LS (Damage Control). For MCE_R, the lowest performance is LS to CP (Limited Safety). Both local element performances are in agreement with the structural performance based on the global behavior of the structure (lateral drift ratio).

Table 6 Hinge State and Status for DBE and MCE at different direction, elements and fy.

fy (MPa)	Direction	Element	Hinge Status	
			DBE	MCE_R
240	X	Roof Beam	IO to LS	LS to CP
		Column	IO to LS	LS to CP
	Y	Roof Beam	A to IO	IO to LS
		Column	IO to LS	LS to CP
420	X	Roof Beam	A to IO	IO to LS
		Column	A to IO	IO to LS
	Y	Roof Beam	A to IO	IO to LS
		Column	IO to LS	IO to LS

Note: Hinge status definition can be referred to Figure 6.

In this study, the beam-column joint adequacy for seismic load is not investigated explicitly. In fact, lack of detailing in non-engineered structure can lead to early failures in these joints. In the case where the structure is in compliance with the seismic codes, the beam-column joints shall be the last element to fail, with the energy dissipation occur mainly on deformation-controlled element such as beams and columns. However, with the lack of shear capacity from the contribution of transverse rebars, shear failure mechanism can occur on these joints before the beams can deform properly and develop plastic hinges. This leads to a more brittle failure mode behavior with the failure hierarchy of bottom column at first, followed by joints and beams respectively.

4. Conclusion

In this study, an analysis of a non-engineered residential house in Indonesia is reported. Key findings from the study are summarized below:

1. Field survey reveals that the typical cross section for beam is larger than the column. This suggests that strong column-weak beam philosophy is not respected in the practice. In addition, the obtained concrete mixture provides lower compressive concrete strength than what was reported in previous studies.
2. Analysis results with varying concrete and rebar strength show that the variation in steel (rebar strength), represented by its yield strength (fy), has a more direct influence to seismic performance than the concrete strength. However, it is important to remark that this statement might not hold true if failure happens due to joint shear failure where the behavior is more controlled by the concrete strength.
3. From the DCR comparison in linear analysis, designated seismic-resisting elements such as roof beams do not have adequate flexural capacity for factored gravity load combination, leaving no reserve capacity for seismic load combination. Thus, it is important to highlight that in non-engineered houses, it is very possible for the structural elements to collapse due to gravity load alone, even without the occurrence of an earthquake.
4. The elastic force reduction for seismic-resistance structures, represented by response modification factor (R), from linear and nonlinear analysis has shown a similar pattern for both models with different yield

strength of steel (fy). For model using fy 240 MPa, the significantly high R value indicates that the structure will behave as intermediate or special moment frames where it relies mainly on the ductility of its elements in the event of a strong earthquake. By contrast, for model using fy 420 MPa, the structure falls into the ordinary moment frames category where it relies mainly on the strength and elastic stiffness with few permanent deformations occur on its elements hence less complicated seismic detailing is required.

5. The failure mode behavior and mechanism of the formation of plastic hinges in X and Y direction for fy 240 MPa comply with the expected failure modes for earthquake-resistance buildings where the bottom columns develop plastic hinges, followed by beams and other columns. However, structural model with fy 420 MPa has shown an early formation of plastic hinges at the top columns, leading to a more brittle behavior than fy 240 MPa.
6. For DBE, the non-engineered residential houses will sustain permanent deformations and moderate retrofitting scheme is sufficient to ensure the safety of the structure (IO to LS performance level). While for MCE_R, significant damages and permanent deformations will occur on the structural elements although the columns remain essentially working (LS to CP performance level), providing a minimal stiffness and strength and requiring heavy retrofitting scheme to prevent collapses and increase reserve capacity for future seismic events.

5. Acknowledgements

This research was done with the help of the P2MI-ITB (ITB Research, Community Service and Innovation Program) Grant Scheme 2021 from ITB.

5. Copyrights

6. References

[1] Departemen Pekerjaan Umum, Peraturan Muatan Indonesia 1970 NI-18, Bandung: Departemen Pekerjaan Umum, 1970.

[2] Direktorat Penyelidikan Masalah Bangunan, Peraturan Pembebanan Indonesia untuk Gedung, Bandung: Stensil, 1983.

[3] Badan Standarisasi Nasional, SNI 1726:2019 Tata cara perencanaan ketahanan gempa untuk, Jakarta: BSN, 2019.

[4] T. Boen, P. Suprobo, Sarwidi, K. S. Pribadi, A. Irmawan, Satyarno and A. Saputra, Persyaratan Pokok Rumah yang Lebih Aman Bangunan Tembokan dengan Bingkai Beton Bertulang, Padang: Departemen Pekerjaan Umum Republik Indonesia , JICA, Pemerintahan Propinsi Sumatera Barat., 2009.

[5] K. Okazaki, K. S. Pribadi, D. Kusumastuti and T. Saito, "Comparison of Current Construction Practices of Non-Engineered Buildings in Developing Countries," in *15th World Conference on Earthquake Engineering (15WCEE).*, Lisbao, 2012.

[6] S. Wijanto and R. Sinha, "World Housing Encyclopedia: A resource on construction in Earthquake Regions," Earthquake Engineering Research Institute and International Association for Earthquake Engineering.

[7] IFDR/AusAid/GA, "Austalia-Indonesia Facility for Disaster Reduction, Expert Engineering Team: West Sumatra Building Recommendations," IFDR/AusAid/GA, 2009.

[8] A. Masi, "Seismic Vulnerability Assesment of Gravity Load Deigned R/C Frames," *Bulletin of Earthquake Engineering*, vol. 1, pp. 371-395, 2003.

[9] A. Masi and M. Vona, "Vulnerability assessment of gravity-load designed RC buildings: Evaluation of seismic capacity through non-linear dynamic analyses," *Engineering Structures*, vol. 45, p. 257–269, 2012.

[10] D. Kusumastuti, M. Suarjana, K. S. Pribadi, Rildova and T. H. Lie, "Experimental Study on Typical Confined Masonry Structure under Cyclic Lateral Load," in *5th World Conference on Earthquake Engineering*, 2012, Lisboa.

[11] I. W. Sengara, M. Suarjana, D. Beetham, N. Corby, M. Edwards, M. Griffith, M. Wehner and R. Weller, "The 30th September 2009 West Sumatra Earthquake: Padang Region Damage Survey," Geoscience Australia, Canberra, 2010.

[12] Computers & Structures Inc., CSI Analysis Reference Manual for SAP2000, ETABS, SAFE, and CSiBridge, Computers & Structures Inc Developers., 2017.

[13] American Society of Civil Engineers, Seismic Evaluation and Retrofit of Existing Buildings (41-17), Virginia: ASCE, 2017.

[14] American Concrete Institute, ACI 374-2R-13 – Guide for Testing Reinforced Concrete Structural Elements Under Slowly Applied Simulated Seismic Loads, ACI Committee, 2013.

[15] Badan Standarisasi Nasional, SNI 2847:2019 Persyaratan beton struktural untuk bangunan, Jakarta: BSN, 2019.

[16] V. I. Novelli, R. D. Risi, I. Ngoma, I. Kafodya, P. Kloukinas, J. Macdonald and K. Goda, "Fragility curves for non-engineered masonry buildings in developing countries derived from real data based on structural surveys and laboratory tests," *Soft Computing*, 2020.

[17] Geoscience Australia, "Padang Earthquake Reconnaissance Workshop: Indonesian Building Regulation Development and Construction Practice," Geoscience Australia, Padang, 2010.

[18] Badan Standarisasi Nasional, SNI 1727:2020 Beban desain minimum dan kriteria terkait untuk bangunan dan struktur lain, Jakarta: BSN, 2020.

