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Modeling's Effect of Irregular Building Structure with Vegetated Roof on Seismic Evaluation per ASCE 41-17

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Abstract
This study examines the seismic vulnerability of an irregular educational building with a vegetated roof in Yogyakarta using linear procedures based on ASCE 41-17. Modelling approaches differ Model 1 treats skylights and planter boxes as loads with straight roof slabs, while Models 2 and 3 use shells with sloping roof slabs. Vegetated roofs are featured in Models 1 and 2. The analysis, conducted at seismic hazard levels BSE-1N and BSE-2N using the ETABS program, evaluates structural components and compares Response Spectrum (RS) and Linear Time History (LTH) methods. Results show seismic weight variations of 1.20% to 15.04% between models. Model 1 fails to meet the criteria for modal analysis, while Models 2 and 3 do. The structural performance evaluation based on average demands at BSE-1N and BSE-2N levels varied from Immediate Occupancy to Life Safety performance. The LTH method in all models had higher acceptance ratios than the RS method.

INTRODUCTION

The advancement of technology has led to significant ease in evaluating building structures, with various software programs and analytical tools now available to facilitate the process. However, users need to possess adequate knowledge and experience to make informed decisions in modeling and evaluation. One of the structural analysis programs frequently used in building evaluation is ETABS. Within ETABS, several inherent assumptions are applied to the model that users should consider before proceeding with the analysis. These include positioning insertion points for beams and slabs, dividing elements into meshes, and the selection of end-length offsets. Such considerations are crucial for modeling that closely aligns with real-world field conditions.

Sazzad et al. (2017) researched the optimum mesh size for achieving accuracy and optimal computational time. They concluded that selecting a specific mesh size is challenging without conducting a convergence study. Their results showed that using a mesh size of up to 1000 mm resulted in less than 1% errors, reducing the computational time required for analysis without sacrificing accuracy (Rudiyanto, 2023).

Critical factors when evaluating beam-column connections under horizontal loading, such as seismic loads, include self-weight control, stiffness, strength, and material ductility. These factors heavily depend on the nature of the connections (Bogatinoski et al., 2013). Another critical parameter in modeling is the application of end-length offsets in frames. Default software settings typically assume full-length offsets, while using clear length offsets often provides a more realistic representation of the structural response. The difference in weight between frames modelled with clear-length offsets and those with full-length offsets becomes more significant as the number of frames increases.

According to Satyarno et al. (2010), when modeling beams in SAP2000, the automatic setting positions beams at the centroid of the slab (i.e., the beam's axis aligns with the middle of the beam). A similar scenario may occur within ETABS, depending on the software version. However, beams are often placed at the top-center relative to floor plates, with the upper side of the beam aligning with the upper surface of the slab. This necessitates changing the beam placement from centroid to top center using the insertion point tools in the software. Support positioning significantly affects beam response and stiffness, while eccentricities at supports lead to horizontal reactions and negative moments (Turker, 2020).

In buildings with structural irregularities—where the center of mass does not coincide with the center of stiffness—eccentricities are introduced, resulting in torsion (Ahmed et al., 2016). Such irregularities often stem from non-uniform building shapes, and structural elements' position, size, and orientation significantly influence the induced torsion, which can cause damage (Zabihullah et al., 2020). Regularly arranged buildings tend to maintain consistent strength, stiffness, and mass distribution, whereas irregular buildings show variations in forces and deformations at locations of irregularity. This can lead to structural element failures and potentially building collapse (Shelke, 2017).

SNI 1726:2019 Article 7.7.2 mandates that calculating effective seismic weight must include the weight of landscaping and other loads on vegetated roofs or similar areas. This means the additional weight of vegetation on a roof significantly influences the seismic loads a structure experiences. Salamati et al. (2017) found that vegetated roofs can affect building response during earthquakes. The impact varies depending on plant type, substrate thickness, and the roof structure used.

According to ASCE 41-17, for new buildings, the basic performance level can only be applied to Tier 3 evaluations, where linear or nonlinear procedures may be used. Linear procedures are categorized into static and dynamic methods, but static procedures are only permitted for buildings with specific irregularities. While the base shear calculated using static analysis is not significantly affected by building irregularities, evaluations using response spectrum analysis and time history analysis show greater sensitivity (Nady et al., 2022). Complex buildings often require dynamic evaluation methods. Although more time-consuming, the linear time history method tends to provide more accurate results than the response spectrum method, as it accounts for a broader range of possible forces (Algamati et al., 2023).

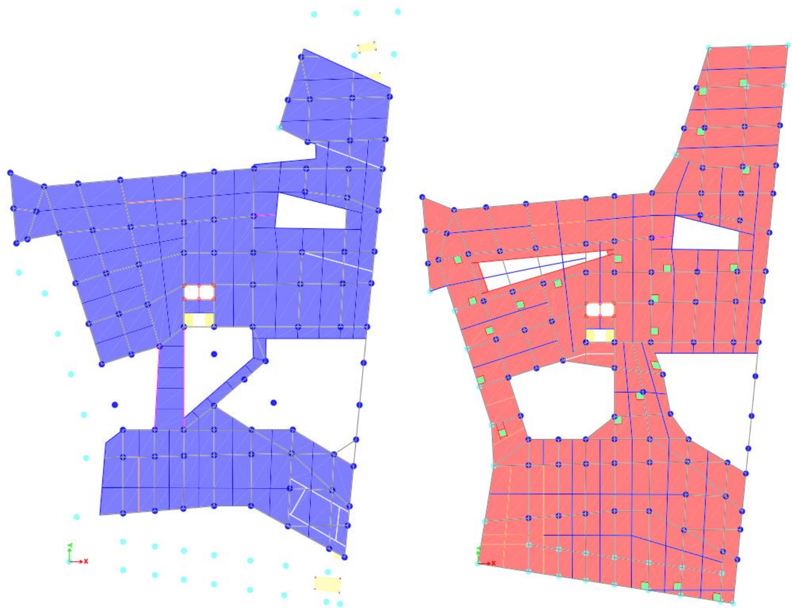
In recent years, the implementation of vegetated roofs has gained popularity due to their environmental benefits, such as thermal insulation and stormwater management. However, the additional weight and changes in structural stiffness introduced by vegetated roofs can significantly impact a building’s seismic response. Yogyakarta, located in a seismically active region of Indonesia, is an ideal location for this study due to its history of seismic activity and numerous educational buildings that prioritize sustainable design features. While vegetated-roof educational buildings may not be as prevalent today, understanding their seismic vulnerability is essential for future construction practices, retrofitting efforts, and implementing similar sustainable features in new buildings.

The difference between the research conducted by the author and previous research lies in the fact that the object and modelling parameters varied, as well as the analysis methods used. The modelling parameters compared by the researcher involve detailed modelling on the roof floor. The roof floor must be modelled with planter boxes, skylights, and sloping plates. These parameters will be modelled using shell elements instead of using only line loads, which are usually used to simplify modelling. By modelling using shell elements, it is hoped that the stiffness obtained will be more realistic in the following field conditions. After that, the effects of loads on the roof with vegetation and without vegetation will be compared. The object used in this research is an irregular building with a vegetated roof located in Yogyakarta. Additionally, in this research, the analysis method used is supported by the ETABS program. Therefore, this research is original and different from previous studies.

This research aims to compare seismic weight and modal analysis across all models. It evaluates building structural components using linear procedures according to ASCE 41-17 to assess the impact of vegetated roofs. Additionally, it compares evaluation results based on response spectrum (RS) and linear time history (LTH) methods across various building modelling variations.

RESEARCH METHODS

This study is numerical research to evaluate the influence of modelling assumptions on irregular buildings with vegetated roofs. The building under consideration is an educational facility in Yogyakarta situated on stiff soil with site class D. This building falls into risk category IV according to SNI 1729:2019. The building's performance level is evaluated based on ASCE 41-17 regulations. Based on primary performance objectives equivalent to new building standards for risk category IV, at seismic hazard level BSE-1N, the target performance level is Immediate Occupancy (IO), whereas at seismic hazard level BSE-2N, the target performance level is Life Safety (LS). The floor plan of this irregular building can be seen in Figure 1. The building has 2 floors with an additional lift cover roof floor, where the second floor will be utilized as a vegetated roof. Building modelling was conducted using ETABS.



(a) (b)

Figure 1. *Floor plan of the building (a) Level 1 and (b) Level 2.*

Modeling Parameters

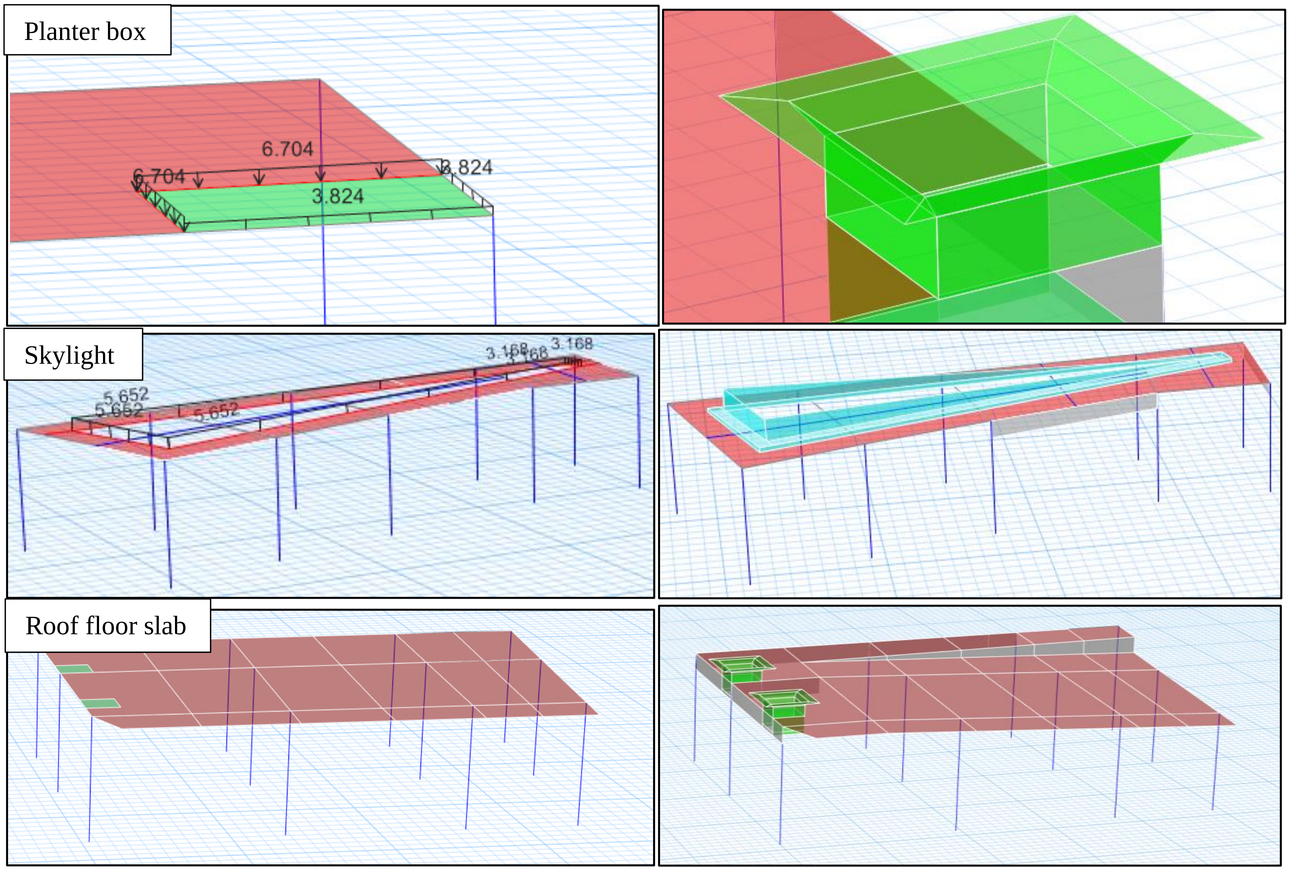
The parameters used for modeling variations in this study are as follows: (1) Modeling with basic model (Model 1) and detailed model (Model 2 and Model 3). In the basic model, the planter box and skylight modeling are only represented with uniform loads, while in the detailed model, they will be modeled with shell elements resembling the actual field conditions. As for the sloping slab on the roof floor, it will be modeled flat in the basic model, unlike the detailed model which will be modeled as is. The detailed differences between basic model and detailed model can be observed in Figure 2, along with added perspective images of the 3D models being studied shown in Figure 3. (2) Modeling for the detailed model will be evaluated by considering the influence of vegetated roofs. The detailed model will be compared with vegetated roof and without vegetated roof conditions.

Thus, in this study, three types of modeling will be conducted as shown in Table 1.

Table 1. *Variations in building modeling.*

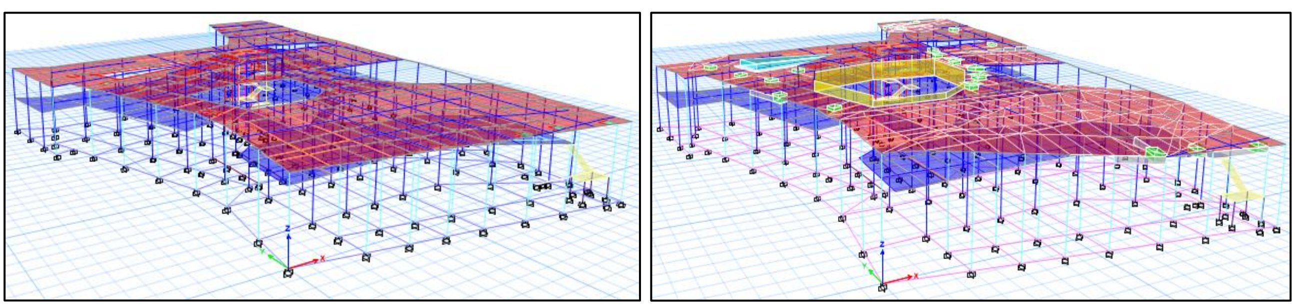
Model	Planter box	Skylight	Roof floor slab	Vegetated roof
Model 1	Modelled as load	Modelled as load	Modeled as a flat slab	Included
Model 2	Modelled as shell	Modelled as shell	Modeled as a sloped slab	Included
Model 3	Modelled as shell	Modelled as shell	Modeled as a sloped slab	Excluded

The components in the modelling will mesh using automatic meshing, where the maximum size of each mesh element should not exceed 1m. The insertion point for beams and slabs will be set to the top centre. Components using frame elements will have a rigid zone factor of one, with the length of the frame being the clear length.



(a) (b)

Figure 2. *The detailed differences between (a) basic model and (b) detailed model.*



(a) (b)

Figure 3. *Perspective images of the 3D models between (a) basic model and (b) detailed model.*

Loading

The loading is based on SNI 1726:2019, SNI 1727:2020, and ASCE 41-17. The building loads consist of: dead loads, additional dead loads, live loads, roof live loads, and seismic loads. The building is evaluated based on seismic hazard levels BSE-1N and BSE-2N. The seismic load direction is applied at 0°, 45°, 90°, and 135° angles to the floor plan. The application of the RS method utilizes simultaneous seismic effects with a combination of 100% in one direction and an additional 30% in other directions, while the LTH method applies orthogonal pairs of ground motion acceleration histories simultaneously.

The ground motion acceleration parameters are taken from SNI 1726:2019. The determination of seismicity levels based on ASCE 41-17, on parameters S_{DS} and S_{D1}, falls into the high seismicity level. The selection of ground motion data for the linear time history method is based on seismic site deaggregation, as indicated in Table 2, with 3 selected pairs of ground motions shown in The ground motion acceleration is spectrally matched within the range of 0.8 *Tlower* to 1.2 *Tupper*. The value of *Tlower* used for matching is the lowest period value at which 90% mass participation is achieved in both orthogonal directions of the building from all models, while the value of *Tupper* is the highest period value in the orthogonal direction of the building from all models. The value of *Tlower* is 0.500 seconds, while the value of *Tupper* is 0.681 seconds. The results of spectral matching of average pseudoacceleration (pSa) for the three pairs of ground motion recordings can be seen in Figure 4.
Table 3.

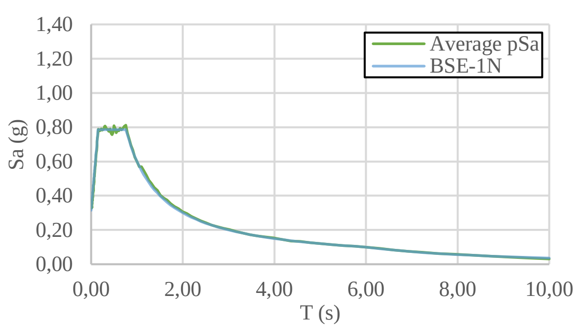
Table 2. *The disaggregation values of magnitude and earthquake source distance in Yogyakarta.*

Type	Magnitude (Mw)	Distance (km)
Benioff	7.2 - 7.4	104 - 125
Shallow crustal	6.2 – 6.4	19 - 28
Megathrust	8.6 - 8.8	104 - 125

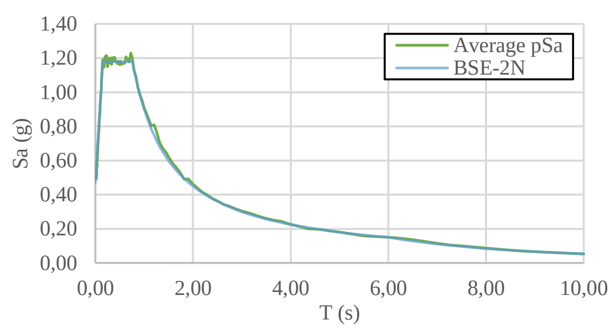
The ground motion acceleration is spectrally matched within the range of 0.8 T_{lower} to 1.2 T_{upper}. The value of T_{lower} used for matching is the lowest period value at which 90% mass participation is achieved in both orthogonal directions of the building from all models, while the value of T_{upper} is the highest period value in the orthogonal direction of the building from all models. The value of T_{lower} is 0.500 seconds, while the value of T_{upper} is 0.681 seconds. The results of spectral matching of average pseudoacceleration (pSa) for the three pairs of ground motion recordings can be seen in Figure 4.

Table 3. *The ground motion data was used*

Earthquake	Location	Year	Station	Magnitude M_W	Type	R Km	PGA (g) H_1	PGA (g) H_2	v_s m/s
Miyagi	Japan	2005	Sendai	7.22	Benioff	110.16	0.25	0.27	288.20
Chi-Chi	Taiwan	1999	TCU065	6.30	Shallow crustal	26.05	0.13	0.14	305.85
Vina Del Mar	South America	2010	Galeria Couve	8.81	Megathrust	120.52	0.33	0.22	282.00



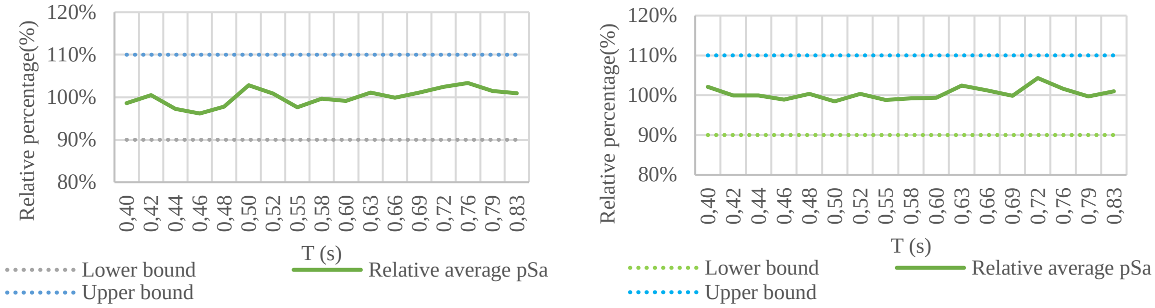
(a)



(b)

Figure 4. *The results of spectral matching of average ground motion acceleration at seismic hazard levels (a) BSE-1N and (b) BSE-2N.*

The graph showing the relative percentage of average spectral acceleration matching is displayed in Figure 5. The average ground motion recordings matched within the range of 0.8 (T_{lower} = 0.500) at 0.400 seconds and 1.2 (T_{upper} = 0.681) at 0.817 seconds are in accordance with the 10% boundary of the target spectrum requirements.



(a) (b)

Figure 5. *Graph of the relative percentage of average spectral acceleration matching at seismic hazard levels (a) BSE-1N and (b) BSE-2N.*

Linear Procedures

The fundamental period of the building based on empirical methods is calculated using Eq. (1), and this period value will be compared to the building period generated by the program.

$$T_a = C_t h_n^x \qquad (1)$$

T_a is fundamental period, C_t and x are the building period coefficient, and h_n is the structural height.

The criteria for using linear procedures are determined based on building irregularities and the demand-capacity ratio (DCR). The DCR values for components are calculated using Eq. (2).

$$DCR = Q_{UD}/Q_{CE} \qquad (2)$$

Where Q_{UD} is the deformation-controlled action due to gravity and seismic loads, and Q_{CE} is the expected strength of the deformation-controlled action element. Linear procedures are allowed for buildings without irregularities due to parallel or perpendicular discontinuities. If the DCR of a component exceeds the smaller of 3.0 and the m-factor value for component actions and there are one or more irregularities due to weak-story or torsional strength, then linear procedures do not apply.

Deformation-controlled action (DCA) is determined using Eq. (3), and force-controlled action (FCA) is determined using Eq. (4).

$$Q_{UD} = Q_G + Q_E \qquad (3)$$

Where Q_G is the action due to gravity loads and Q_E is the action due to seismic loads.

$$Q_{UF} = Q_G + \chi Q_E / C_1 C_2 J \qquad (4)$$

Where Q_{UF} is the force-controlled action due to gravity and seismic loads, χ is the factor to adjust actions caused by response at the selected performance level, C_1 is the modification factor to relate the estimated maximum inelastic displacement to the calculated displacement for elastic linear response, C_2 is the modification factor to describe the influence of pinned hysteresis shape, cyclic stiffness degradation, and strength degradation on maximum displacement response, and J is the force transfer reduction factor. Acceptance criteria for DCA are calculated using Eq. (5) and for FCA using Eq. (6).

$$m\kappa Q_{CE} > Q_{UD} \qquad (5)$$

Where m is the modification factor for component capacity based on ASCE 41-17, and κ is the knowledge factor (for new buildings κ = 1). Q_{CE} is the expected strength of the deformation-controlled action element.

$$\kappa Q_{CL} > Q_{UF} \qquad (6)$$

Where Q_{CL} is the lower bound strength of the force-controlled action element.
Parameters obtained from the linear procedure include the fundamental period of the building (T_a) and the acceptance criteria ratio of structural components. Components with DCA include flexural moment (M) in beams, combinations of tensile axial force (P) and flexural moment (M) in columns, while components with FCA include shear force (V) in beams and shear force (V) in columns. The target performance level is achieved when the acceptance criteria ratio does not exceed the target performance level.

RESULTS AND DISCUSSION

Modeling Effects

This study begins by comparing the three models, which will serve as the basis for selecting the model to be evaluated according to ASCE 41-17. The modeling effects are examined based on the differences in seismic weight and modal analysis for each model.

The influence of vegetated roofs on the seismic weight of the building will be compared for the three models. Model 1 and Model 2 are modeled considering the weight of wet soil, grass, and trees due to the presence of vegetated roofs. Model 3 is modeled with the assumption that the roof floor is not vegetated, so the weight of wet soil, grass, and trees is considered nonexistent. The comparison of seismic weights of the three models is shown in Table 4. The difference in seismic weight between Model 1 and Model 2 is only 1.20%, attributed to differences in the modeling of skylights, planter boxes, and sloping slabs. The difference in seismic weight between Model 2 and Model 3 is 15.04%, indicating the significant influence of vegetated roofs on the same modeling approach.

Table 4. *Percentage difference of seismic weight*

Model	Seismic weight (kN)	Percentage difference from Model 2
Model 1	141247.3	1.20%
Model 2	142962.2	-
Model 3	121460.7	15.04%

The modal analysis results for Model 2 and Model 3 have met the expected building response, where the first three modes are translation in the Y axis, translation in the X axis, and rotation in the Z axis, respectively. However, Model 1 did not meet the expected building response, where rotation in the Z axis occurred in the first mode, translation in the X axis in the second mode, and rotation in the Z d axis occurred again in the third mode. All models achieved a total mass participation of 90% in the third mode for both main orthogonal directions. The periods and mass participation for all building models can be seen in Table 5.

Table 5. *Period and mass participation of the building*

Model	Mode	Period *T* (s)	Dominant Mass Participation [Accumulation]		
			UX	UY	RZ
Model 1	1	0.613	0.290 [0.290]	0.293 [0.293]	0.366 [0.365]
	2	0.583	0.516 [0.806]	0.419 [0.711]	0.004 [0.369]
	3	0.549	0.133 [0.938]	0.228 [0.939]	0.568 [0.937]
Model 2	1	0.681	0.028 [0.028]	0.783 [0.783]	0.114 [0.114]
	2	0.671	0.893 [0.921]	0.025 [0.807]	0.004 [0.117]
	3	0.575	0.000 [0.921]	0.115 [0.922]	0.803 [0.921]
Model 3	1	0.579	0.020 [0.020]	0.827 [0.827]	0.074 [0.074]
	2	0.578	0.896 [0.916]	0.023 [0.851]	0.001 [0.075]
	3	0.500	0.003 [0.919]	0.071 [0.922]	0.845 [0.920]

With a seismic weight difference of 1.20% and a period difference of 9.99%, Model 2 is deemed to better represent Model 1, as there is no significant disparity between the two models. Additionally, Model 2 is characterized by a more realistic modeling approach. Model 2 and Model 3 have a seismic weight difference of 15.04% with a period difference of 14.98%. Model 2 and Model 3 will be evaluated based on ASCE 41-17 to determine the impact of the roof with vegetation and without vegetation.

Based on Eq. (1), the fundamental period of the building (T_a) for a concrete building with a height of 11.36 meters (37.27 feet) using a moment-resisting frame system is obtained.

$$T_a = 0.018 \times 37.27^{0.9} = 0.467\ s$$

The building period in all models obtained by the program is higher than the T_a value. This is because the empirical formula for T_a only uses the building height variable and can only depict the lower limit value for buildings.

Building Irregularity Check

The check for building irregularities is performed using the RS and LTH methods for seismic hazard levels BSE-1N and BSE-2N according to the requirements in ASCE 41-17. The results of the building irregularity check for Model 2 can be seen in Table 6, where there are no irregularities in the building for both RS and LTH methods at seismic hazard levels BSE-1N and BSE-2N. The results of the building irregularity check for Model 3 can be seen in

Table *7*, where there are building irregularities due to torsional strength in the RS method for seismic hazard levels BSE-1N and BSE-2N.

Table 6. *Checking the irregularity of the building in Model 2*

Seismic Hazard Level	Method	Building Irregularity			
		In-Plane Discontinuity	**Out-of-Plane Discontinuity**	**Weak Story** $DCR_n/DCR_{n+1} < 125\%$ $DCR_{n+1}/DCR_n < 125\%$	**Torsional Strength** $DCR_{Max.\ left}/DCR_{Max\ right} < 150\%$ $DCR_{Max.\ right}/DCR_{Max.\ left} < 150\%$
BSE-1N (IO)	RS	Did not exist	Did not exist	Did not exist (max. 113%)	Did not exist (max. 147%)
	TH	Did not exist	Did not exist	Did not exist (max. 120%)	Did not exist (max. 124%)
BSE-2N (LS)	RS	Did not exist	Did not exist	Did not exist (max. 106%)	Did not exist (max. 147%)
	TH	Did not exist	Did not exist	Did not exist (max. 123%)	Did not exist (max. 136%)

Table 7. *Checking the irregularity of the building in Model 3*

Seismic Hazard Level	Method	Building Irregularity			
		In-Plane Discontinuity	**Out-of-Plane Discontinuity**	**Weak Story** $DCR_n/DCR_{n+1} < 125\%$ $DCR_{n+1}/DCR_n < 125\%$	**Torsional Strength** $DCR_{Max.\ left}/DCR_{Max\ right} < 150\%$ $DCR_{Max.\ right}/DCR_{Max.\ left} < 150\%$
BSE-1N (IO)	RS	Did not exist	Did not exist	Did not exist (max. 109%)	Existed (max. 153%)
	TH	Did not exist	Did not exist	Did not exist (max. 111%)	Did not exist (max. 144%)
BSE-2N (LS)	RS	Did not exist	Did not exist	Did not exist (max. 102%)	Existed (max. 153%)
	TH	Did not exist	Did not exist	Did not exist (max. 105%)	Did not exist (max. 138%)

Structural Component Evaluation

The evaluation of structural components is conducted on beams and columns, where only primary components are evaluated. The evaluation of components using DCA is performed for moments in beams and combinations of axial forces with biaxial flexural moments in columns, calculated using Eq. (3). The evaluation of components using FCA is performed for shear forces in beam-columns, calculated using Eq. (4). The acceptance criteria ratio for DCA components is calculated based on Eq. (5), and for FCA components is calculated based on Eq. (6), with a maximum ratio limit of one. If the obtained ratio exceeds one, it is considered as not meeting the limit for ratio acceptance criteria. The building evaluation is conducted based on the RS and LTH methods at the BSE-1N (IO) and BSE-2N (LS) levels for Model 2 and Model 3.

Component capacity

A summary of beam capacity calculations is presented in Table 8, while a summary of column capacity calculations can be found in Table 9.

Table 8. *Beam capacity*

Beam	**Lower bound capacity**			**Expected capacity**		
	Moment +	**Moment -**	**Shear**	**Moment +**	**Moment -**	**Shear**
	$M_{CL(-)}$ **(kNm)**	$M_{CL(+)}$ **(kNm)**	V_{CL} **(kN)**	$M_{CE(-)}$ **(kNm)**	$M_{CE(+)}$ **(kNm)**	V_{CE} **(kN)**
B1 400x800	664.45	341.76	1118.97	838.51	429.18	1391.22
B2 350x700	563.74	292.76	935.18	713.75	368.23	1163.31
B3 300x600	355.44	184.87	761.46	450.26	232.58	947.73
B4 300x500	240.63	149.15	619.79	304.93	187.92	771.40

Table 9. *Column capacity*

Column	**Lower bound capacity**			**Expected capacity**		
	Axial	**Moment**	**Shear**	**Axial**	**Moment**	**Shear**
	N_{CL} **(kN)**	M_{CL} **(kNm)**	V_{CL} **(kN)**	N_{CE} **(kN)**	M_{CE} **(kNm)**	V_{CE} **(kN)**
K1 D1000	21024.29	3134.33	1988.32	30926.42	4256.91	2432.35
K3 L300x600	7731.62	750.49	1131.88	11272.57	1042.76	1394.74
K4 T300x600	7818.66	774.70	1098.60	11381.48	1075.47	1359.96

The *m-factor* values are calculated for all primary components to be evaluated. The smallest m-factor value for each component is presented in Table 10 for more conservative results.

Table 10. *Acceptance criteria for the m-factor*

Component		***m-factor***	
		BSE-1N (IO)	**BSE-2N (LS)**
Beam	B1 400x800	1.25	2.00
	B2 350x700	2.71	5.13
	B3 300x600	2.70	5.09
	B4 300x500	2.00	2.76
Column	K1 D1000	1.35	2.11
	K3 L300x600	1.37	2.15
	K4 T300x600	1.38	2.23

Evaluation of beam components

The evaluation of beam moments using DCA is performed for the RS and LTH methods in Model 2 and Model 3. The results of beam moment evaluation at the seismic level BSE-1N (IO) can be seen in Table 11, while for BSE-2N (LS) can be seen in table 12.*At* seismic hazard level BSE-1N, for the RS method, there are a total of 9.49% of beam components in Model 2 that do not meet the acceptance criteria for DCA, whereas for Model 3, it's only 2.37%. With the LTH method, there are a total of 19.87% of beam components in Model 2 that do not meet the acceptance criteria for DCA, while for Model 3, it's only 9.20%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of DCA beam moment at seismic hazard level BSE-1N are considered as components that have exceeded the Immediate Occupancy performance level.

Table 12

Table 11. *The acceptance criteria ratio of DCA beam moment at seismic hazard level BSE-1N*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.
1	0.39	5.36	0.53	7.42	0.41	4.10	0.51	5.04
2	0.25	1.03	0.30	2.19	0.26	0.66	0.30	0.94
3	0.20	0.92	0.24	0.91	0.16	0.61	0.19	0.70

At seismic hazard level BSE-1N, for the RS method, there are a total of 9.49% of beam components in Model 2 that do not meet the acceptance criteria for DCA, whereas for Model 3, it's only 2.37%. With the LTH method, there are a total of 19.87% of beam components in Model 2 that do not meet the acceptance criteria for DCA, while for Model 3, it's only 9.20%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of DCA beam moment at seismic hazard level BSE-1N are considered as components that have exceeded the Immediate Occupancy performance level.

Table 12. *The acceptance criteria ratio of DCA beam moment at seismic hazard level BSE-2N*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.
1	0.44	5.02	0.54	6.34	0.31	3.83	0.36	4.37
2	0.23	1.55	0.26	1.98	0.17	0.82	0.19	1.33
3	0.17	0.54	0.18	0.53	0.09	0.50	0.13	0.74

At seismic hazard level BSE-2N, with the RS method, 9.64% of beam components in Model 2 do not meet the acceptance criteria for DCA, whereas for Model 3, it's only 4.45%. With the LTH method, there are 12.16% of beam components in Model 2 that do not meet the acceptance criteria for DCA, while for Model 3, it's only 6.23%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of DCA beam moment at seismic hazard level BSE-2N are considered as components that have exceeded the Life Safety performance level.

Evaluation of beam shear using FCA for the RS and LTH methods in Model 2 and Model 3. The results of beam shear evaluation at the seismic level BSE-1N (IO) are shown in Table 13, while for BSE-2N (LS) can be seen in table 14.

Table 13. *The acceptance criteria ratio of FCA beam shear at seismic hazard level BSE-1N.*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.
1	0.29	2.52	0.35	3.06	0.26	1.89	0.33	1.42
2	0.32	1.21	0.36	1.47	0.21	0.84	0.23	1.02
3	0.16	0.55	0.19	0.58	0.10	0.30	0.11	0.33

At seismic hazard level BSE-2N, with the RS method, there are a total of 30.81% of beam components in Model 2 that do not meet the acceptance criteria for FCA, whereas for Model 3, it's only 17.49%. With the LTH method, there are a total of 38.35% of beam components in Model 2 that do not meet the acceptance criteria for FCA, while for Model 3, it's only 23.12%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of FCA beam shear at seismic hazard level BSE-2N are considered as components that have exceeded the Life Safety performance level.

Table 14. *The acceptance criteria ratio of FCA beam shear at seismic hazard level BSE-2N.*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.

1	0.71	8.25	0.80	9.15	0.58	6.15	0.64	6.77
2	0.59	3.64	0.66	4.07	0.41	2.56	0.45	2.79
3	0.29	0.98	0.31	0.96	0.20	0.75	0.22	0.82

Evaluation of beam component performance at the BSE-1N level with IO performance target has an average acceptance criteria that meets the acceptance criteria limits for all models with both RS and LTH methods, but there are some components with maximum acceptance criteria values that do not meet the acceptance criteria limits. Evaluation of beam component performance at the BSE-2N level with LS performance target has an average acceptance criteria that meets the acceptance criteria limits for all models with both RS and LTH methods, but there are some components with maximum acceptance criteria values that do not meet the acceptance criteria limits.

Evaluation of column components

The evaluation of combined column axial force with moment using DCA for the RS and LTH methods in Model 2 and Model 3. The results of the evaluation of combined column axial force with moment at the seismic level BSE-1N (IO) can be seen in Table 15, while for BSE-2N (LS) can be seen in Table 16.

Table 15. *The acceptance criteria ratio of DCA for the combination of column axial force with moment at seismic hazard level BSE-1N.*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.
1	1.24	2.73	1.41	2.94	1.00	2.04	1.26	2.43
2	0.84	1.62	0.95	1.88	0.63	1.35	0.77	1.77
3	0.48	0.97	0.49	0.95	0.33	0.69	0.41	0.83

The number of components that do not meet the acceptance criteria for the combination of column axial force with moment at seismic hazard level BSE-1N is quite significant. With the RS method, there are a total of 55.53% of beam components in Model 2 that do not meet the acceptance criteria for DCA, whereas for Model 3, it's 36.45%. With the LTH method, there are a total of 69.93% of beam components in Model 2 that do not meet the acceptance criteria for DCA, while for Model 3, it's 49.16%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of DCA for the combination of column axial force with moment at seismic hazard level BSE-1N are considered as components that have exceeded the Immediate Occupancy performance level.

Table 16. *The acceptance criteria ratio of DCA for the combination of column axial force with moment at seismic hazard level BSE-2N.*

Floor	Model 2				Model 3			
	RS		LTH		RS		RS	
	Average	Max.	Average	Max.	Average	Max.	Average	Max.
1	1.11	2.52	1.31	2.85	0.91	2.56	1.02	2.83
2	0.76	1.61	0.87	1.94	0.56	1.35	0.68	1.59
3	0.39	0.85	0.46	0.97	0.27	0.57	0.34	0.71

The number of components that do not meet the acceptance criteria for the combination of column axial force with moment at seismic hazard level BSE-2N is quite significant. With the RS method, there are a total of 39.53% of beam components in Model 2 that do not meet the acceptance criteria for DCA, whereas for Model 3, it's 31.25%. With the LTH method, there are a total of 52.80% of beam components in Model 2 that do not meet the acceptance criteria for DCA, while for Model 3, it's 36.45%. Based on ASCE 41-17, components that do not meet the acceptance criteria for the ratio of DCA for the combination of column axial force with moment at seismic hazard level BSE-2N are considered as components that have exceeded the Life Safety performance level.

Evaluation of column shear using FCA for the RS and LTH methods in Model 2 and Model 3. The results of column shear evaluation at the seismic level BSE-1N (IO) are shown in Table 17, while for BSE-2N (LS) can be seen in table 18.

Table 17. *The acceptance criteria ratio of FCA for column shear at seismic hazard level BSE-1N*

Floor	***Model 2***				***Model 3***			
	RS		***LTH***		***RS***		***LTH***	
	Average	***Max.***	***Average***	***Max.***	***Average***	***Max.***	***Average***	***Max.***
1	0.15	0.41	0.21	0.54	0.12	0.29	0.16	0.39
2	0.15	0.60	0.21	0.74	0.10	0.39	0.13	0.48
3	0.13	0.43	0.18	0.64	0.08	0.27	0.10	0.30

There are no components that fail to meet the acceptance criteria for column shear at seismic hazard level BSE-1N.

Table 18. *The acceptance criteria ratio of FCA for column shear at seismic hazard level BSE-2N*

Floor	**Model 2**				**Model 3**			
	RS		**LTH**		**RS**		**RS**	
	Average	**Max.**	**Rerata**	**Average**	**Max.**	**Maks.**	**Average**	**Max.**
1	0.50	1.58	0.58	1.72	0.44	1.10	0.53	1.26
2	0.55	1.80	0.63	2.23	0.37	1.20	0.43	1.41
3	0.44	0.90	0.50	0.95	0.26	0.57	0.31	0.64

Evaluation of combined column axial force with moment performance at the BSE-1N level with IO performance target has an average acceptance criteria that does not meet the acceptance criteria limits on the 1st floor for all models with both RS and LTH methods, but has met the acceptance criteria limits for floors above it. As for the evaluation of column shear performance, there is no average acceptance criteria that do not meet the acceptance criteria limits. In the evaluation of combined column axial force with moment performance at the BSE-1N level, there are some components with maximum acceptance criteria values that do not meet the acceptance criteria limits.

Evaluation of combined column axial force with moment performance at the BSE-2N level with LS performance target has an average acceptance criteria that does not meet the acceptance criteria limits on the 1st floor for all models with both RS and LTH methods, but has met the acceptance criteria limits for floors above it. As for the evaluation of column shear performance, there is no average acceptance criteria that do not meet the acceptance criteria limits. In the evaluation of combined column axial force with moment performance at the BSE-2N level, there are some components with maximum acceptance criteria values that do not meet the acceptance criteria limits.

Comparison of Evaluations in Model 2 and Model 3

Comparison of evaluation results in Model 2 and Model 3 for both RS and LTH methods at BSE-1N and BSE-2N levels indicates that Model 2 yields higher acceptance criteria ratios compared to Model 3. Modeling in Model 2 and Model 3 differs in seismic weight. Loading in Model 2 considers the weight of wet soil, grass, and trees for vegetated roofs, while Model 3 assumes no vegetated roof. This difference results in Model 2 with higher seismic weight yielding higher acceptance criteria ratios compared to Model 3.

Comparison of Evaluations based on RS and LTH Methods

Comparison of evaluation results for RS and LTH methods for all models at BSE-1N and BSE-2N levels indicates that the RS method yields lower acceptance criteria ratios compared to the LTH method. This difference is due to the spectral matching results that are not perfectly identical, resulting in variations in the maximum and minimum values from a number of ground motion data used. Additionally, differences in the simultaneous seismic effects applied in RS and LTH methods also affect the results. The simultaneous seismic effects applied in the RS method use a combination of 100% and an additional 30% in other directions, while the LTH method applies orthogonal pairs of ground motion acceleration histories simultaneously.

CONCLUSION

Based on the conducted research, several conclusions have been drawn as follows: (1) The research results indicate that there is a difference in modeling between Model 1 and Model 2 in terms of seismic weight by 1.20% and period by 9.99%, while the difference between Model 2 and Model 3 in terms of seismic weight is 15.04% and period by 14.98%. (2) Evaluation of component performance at the BSE-1N (IO) and BSE-2N (LS) levels shows that the average acceptance criteria meet the specified limits for all models, except for the performance of the combination of axial force with column moment on the first floor for both BSE-1N and BSE-2N levels. Therefore, the structural performance evaluation based on average demands at BSE-1N and BSE-2N levels varied from Immediate Occupancy (IO) to Life Safety (LS) performance. While based on maximum demands at BSE-1N and BSE-2N levels in all models resulted in performance that exceeds the Immediate Occupancy (IO) and Life Safety (LS) performance limit, except for the performance of column shear at seismic hazard level BSE-1N. (3) Comparison of structural component evaluation results for RS and LTH methods for all models at BSE-1N and BSE-2N levels shows that the RS method produces lower acceptance criteria ratios compared to the LTH method.

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