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NUMERICAL ANALYSIS FOR PROGRESSIVE COLLAPSE OF REINFORCED CONCRETE CAMPUS BUILDING IN YOGYAKARTA

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Abstract

Indonesia has many populations as well as the buildings to support them. Since Indonesia received its independence in 1945, the number of populations have increased which also impacted the number of building structures. Majorly, the structural elements of buildings in Indonesia are constructed using reinforced concrete because the material is easy to be found. The building facilities are commonly used for residential, school and office, and government and military purposes. With many buildings in this country, a proper continuous and periodical evaluation is needed to make sure the safety of its user. Indonesia is potential to be affected by many threats, from human error to natural disasters. One of the critical natural disasters that take many victims in just seconds is earthquake. In 2004, earthquake and tsunami reached Aceh, one area in northern Indonesia and caused many victims. This event made Indonesia increase the awareness of natural disasters. Since 2002, Indonesia has developed building and earthquake provisions to make sure of people's safety. The current valid reinforced concrete building provision in Indonesia is regulated in SNI 2847:2019 and earthquake provision is 1726:2019, which indicates that Indonesia is aware and updating its provision based on the newest earthquake events. Based on this earthquake awareness, the building is not only necessary to be designed to this load but also evaluated regularly due to the update of earthquake hazard map. This action might not be easy for all building users because it takes time and costs resources to assess the building comprehensively. Thus, the preliminary action can be an easy tool for them to evaluate before taking complete checks. One preliminary check that can be performed is by progressive collapse analysis. This evaluation is based on the guidelines published by General Services Administrations (GSA) in 2016 [1]. In this study, a reinforced concrete building as academic facilities in one of Indonesia's region is assessed using progressive collapse analysis. Numerical analysis was performed by finite element method supported by a program developed by the co-author, namely STERA_3D. Three cases suggested by GSA guidelines were observed by removing the base floor column on the corner, middle of long side plan and short side plan. Conceptual dissipation energy graphs were adopted to assess structural elements state. Non-linear absolute total rotation is the key parameter to judge the condition. From the numerical modelling, it is known that the provided structural elements details are still in the immediate occupancy, life safety, and collapse prevention states and not in the collapse state. Furthermore, the mid-long column removal slightly has critical affects compared to other removal column locations.

Keywords: reinforced concrete campus building, structural earthquake response, progressive collapse analysis

1. Introduction

Indonesia is a country that currently has population more than 260 million. This causes the growth of multi-storey buildings increase and need to be supported by good infrastructure and carried out regular maintenance. These buildings have potential to be damaged due to many occurrences, one of which is due to natural disaster of earthquake. In Indonesia, earthquake loads in buildings have been regulated in Indonesian National Standard SNI 1726:2012, which this provision's scope is mainly on design stage.

Building has potential to collapse due to earthquake initially caused by one important structural member failure, so other surround structural members receive more forces than its capacity. One evaluation method that capable to assess of this potential collapse is progressive collapse analysis. The procedure is popular by removing one axial structural member, column, in three different main places especially in first floor area to see the structural response and compare the demand capacity ratio for linear analysis or element rotational displacement for non-linear analysis to the acceptance criteria. This issue has been widely discussed since the terrorist attack incident caused the collapse of World Trade Center in September 2001 [2]. To conduct the analysis, an independent USA agency called as General Service Administrator suggested analysis procedure by providing a complete guideline to perform the path analysis for progressive collapse [1].

Many studies about progressive collapse previously have been conducted before. This progressive collapse analysis is done from two-dimensional model [3] to three-dimensional model for reinforced concrete buildings [4]. In other publications, the analysis was performed for dynamic response on specific structural concrete elements [5,6] and even also for high-rise buildings [7,8]. The analysis has been tried to be simplified as well [9]. In another study, the analysis was conducted for structural element of precast shear wall under earthquake load [10].

In Indonesia, to evaluate seismic performance of a reinforced concrete building, it is popular to use pushover analysis and time history analysis to check building responses and there are several examples of these analysis [11,12]. However, the usage of progressive collapse analysis in building for earthquake response is still rarely found in Indonesia, so far there was one but only used linear approach by comparing structural elements' demand-capacity ratios [13]. This study gives an example of progressive collapse that is done for Indonesian area, especially Yogyakarta region using a non-linear approach by conducting a time history analysis. It is hoped that from the result discussed below, it can be one of references to conduct preliminary evaluation for buildings due to progressive collapse potential to create harmless concrete buildings against earthquake events.

2. Building Model

Eight floors buildings were modelled as moment-resisting reinforced concrete structures. Each floor has height of 4 m with total building height of 32 m. Modelling was performed using STERA_3D [14] as depicted in Fig. 1. In this study, only two column types were used, namely C1 for every column except the corner and C2 specifically for corner column. The removal column location variation is illustrated in Fig. 2.

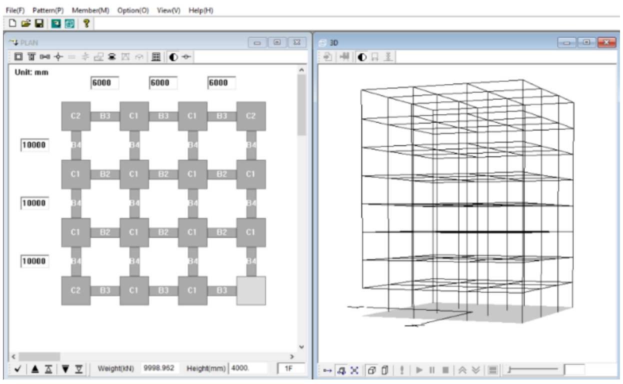
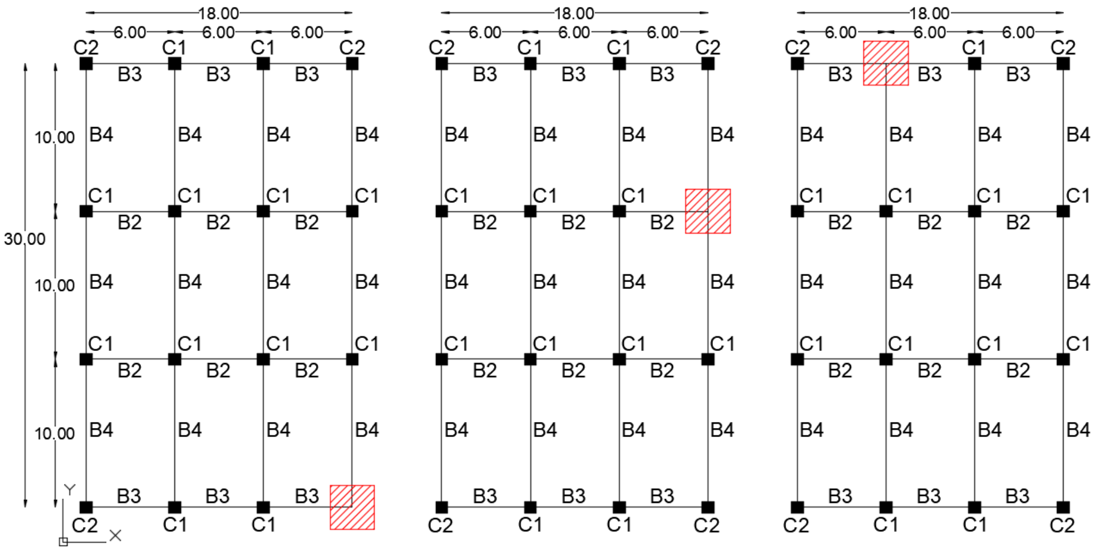


Fig. 1 – Preview of STERA_3D for building modelling software



(a) corner removal (b) mid-long removal (c) mid-short removal

Fig. 2 – Two-dimensional view of three corner removal locations, unit in meter

Structural gravitational loadings were inputted as gravity loadings for analysis which contains of one dead load and half of live load. This live load reduction for academic buildings refers to loading standard for building. The detailed weight for each floor is presented in Table 1.

Table 1 – Inputted weight for building

floor	weight (kN)	floor	weight (kN)
8	9,015.92	4	9,507.44
7	9,507.44	3	9,507.44
6	9,507.44	2	9,507.44
5	9,507.44	1	9,998.96
Total building weight =			76,059.536

Absolute rotational flexural moment and deformation in beam and column from STERA_3D were retrieved after performing response analysis under earthquake motions. In STERA_3D, for reinforced concrete structures, modified Takeda hysteresis loop model is used for the flexural nonlinear model for beams as illustrated in Fig. 3.

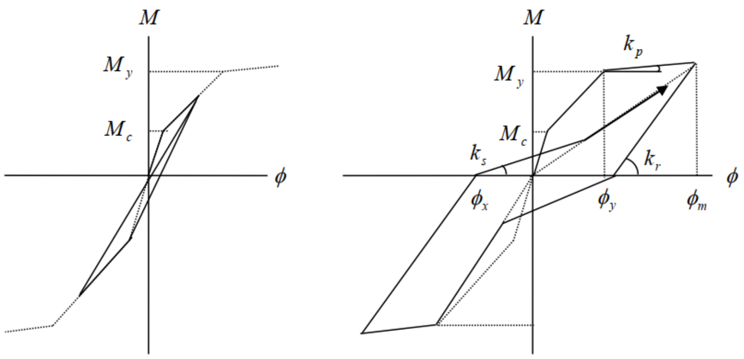


Fig. 3 – Modified Takeda hysteresis model adapted in 3-dimension modelling for concrete structures [14]

Numerical integration method using Newmark-β method with value of β equals to 0.25 (or average acceleration method) was adapted for the earthquake response analysis. The structural dimension and reinforcement detailing for column and beam are presented in Fig. 4 and Table 2.

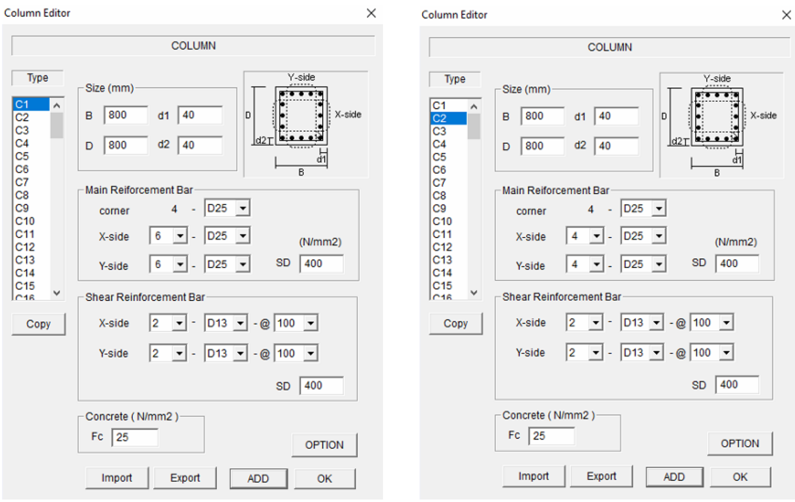


Fig. 4 – Defining Dimension and Reinforcing Detail of Column Elements in STERA_3D

Table 2 – Dimension and Reinforcing Detail of Beam Elements

Beam						Flexural Reinforcement					Shear Reinforcement			
Level	Section	b (mm)	d (mm)	s (mm)	f'_c (MPa)	main r top	⌀ bar	main r bot	⌀ bar	f_y (MPa)	No.	⌀ bar	space (mm)	f_y (MPa)
1,2,3	B2	450	750	150	25	10	D25	5	D25	400	2	D13	100	400
	B3	450	750	150	25	12	D25	6	D25	400	2	D13	100	400
	B4	450	900	150	25	12	D25	6	D25	400	2	D13	100	400
4,5	B5	450	750	150	25	10	D25	5	D25	400	2	D13	100	400
	B6	450	750	150	25	12	D25	6	D25	400	2	D13	100	400
	B7	450	900	150	25	12	D25	6	D25	400	2	D13	100	400
6,7,8	B8	450	750	150	25	10	D25	5	D25	400	2	D13	100	400
	B9	450	900	150	25	10	D25	5	D25	400	2	D13	100	400

3. Applied Earthquake Ground Motion

Three input earthquake ground motions were applied in this study. Since the only open access data retrieved for Indonesian earthquake is the design response spectrum, three artificial time-history waves were generated to match the design response spectrum from Yogyakarta, Indonesia in site class D for stiff soil profile with NSPT between 15 to 50 as stated in Indonesian National Standard (SNI) 1726:2019. The phase of an artificial wave is generated using uniform random numbers and the Jennings' envelope function [15] is used for the amplitude envelope of an artificial wave.

For artificial earthquake waves, it is assumed that the acceleration of E-W direction is 0.3 times of the NS direction. Beside the artificial earthquake waves with random phase, two other earthquake waves were generated with phase information of El-Centro 1940 NS EW and Kobe 1995 NS EW and spectrum information to match the target spectrum. The result of matching process can be seen in Fig. 5 and the earthquake wave motions can be seen in Fig. 6. Detail of the target response spectrum is as follows: S_s = 1.45 g; S_1= 0.65 g; F_a= 1; F_v=1.5; Importance factor (I_e) = 1.5 with building purposes of academic buildings. This study focused on building response due to horizontal earthquakes so vertical earthquake component is neglected.

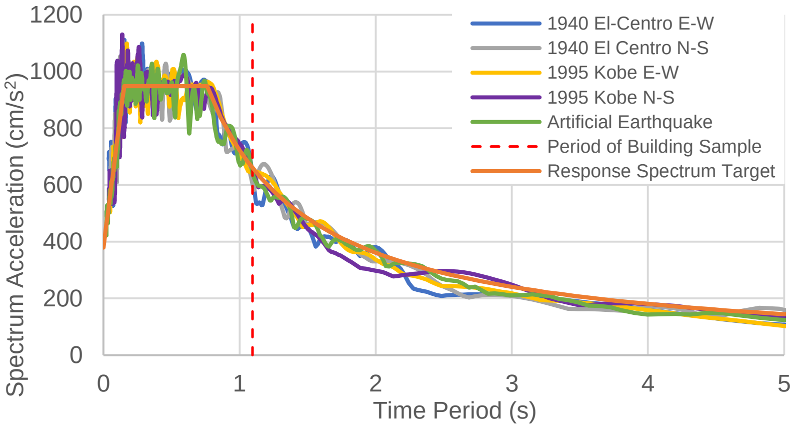


Fig. 5 – Matching result with Yogyakarta response spectrum

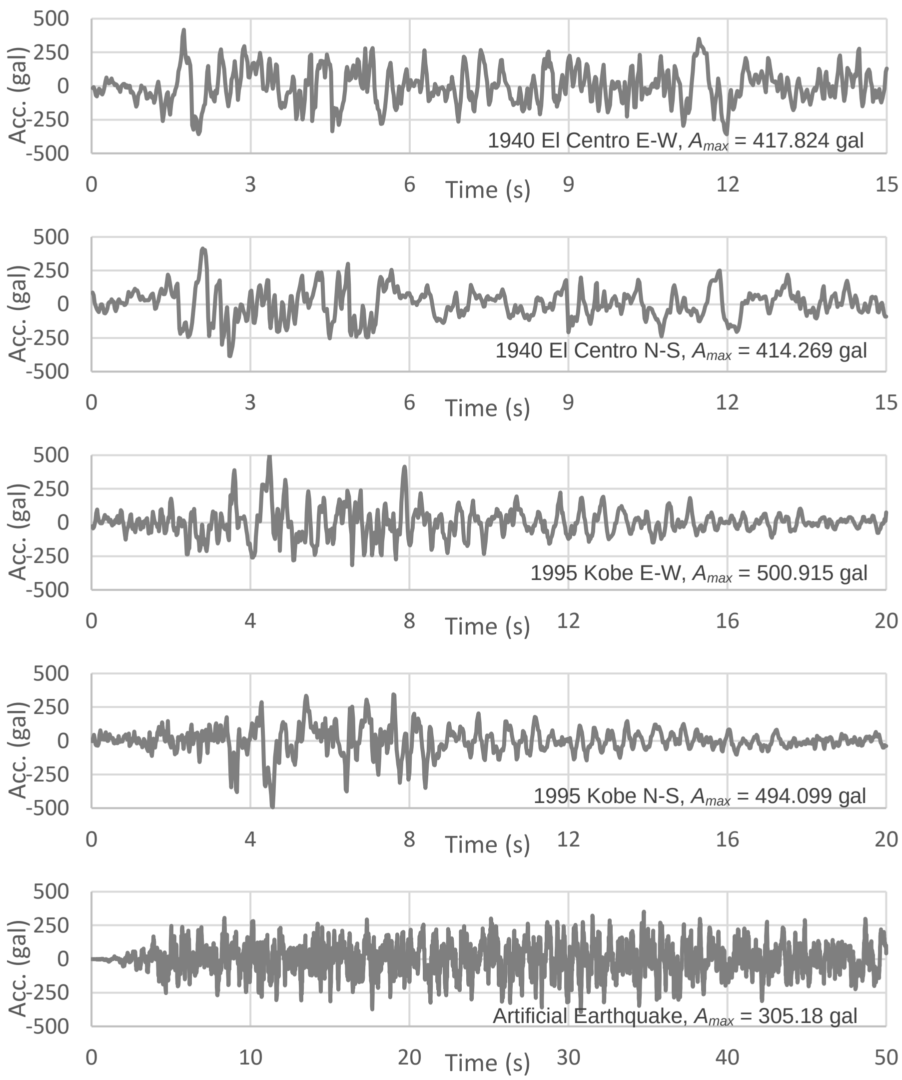


Fig. 6 – Earthquake motions that had been matched with target response spectrum

4. Adopted Acceptance Criteria in Progressive Collapse Analysis

Maeda and Kang in 2009 proposed the post-earthquake damage evaluation of reinforced concrete buildings using two main parameters [16], namely the ratio of dissipated energy, E_d and the residual energy dissipation capacity, E_s as illustrated in Fig. 7. The simplification is furthermore categorized for specify concrete element structures for school buildings in Japan. This study inspired by the same idea to assess the energy to compare the dissipated energy during earthquake while one base column was not working due to damages. Furthermore, the force-deformation relationship used in this research to determine the energy area is based on ASCE 41-17 which is depicted in Fig. 8.

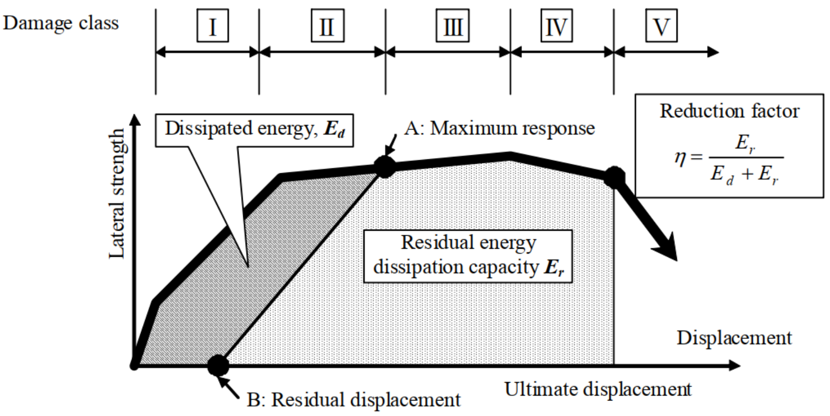


Fig. 7 – Conceptual diagram of seismic capacity reduction factor [16]

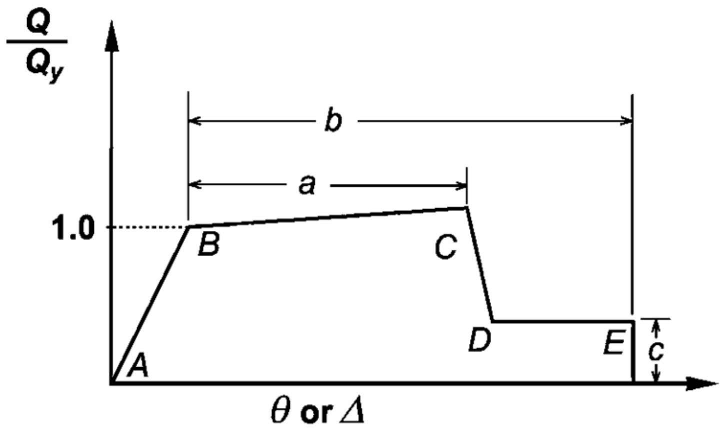
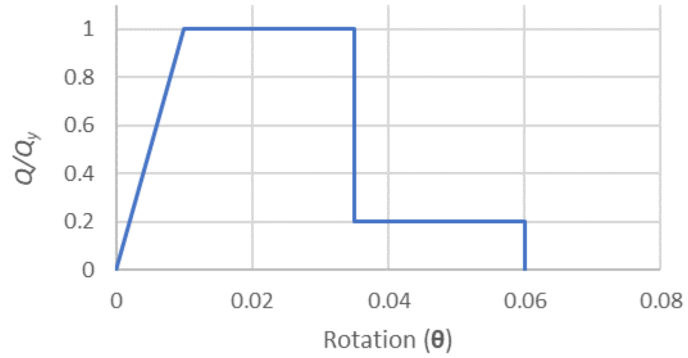
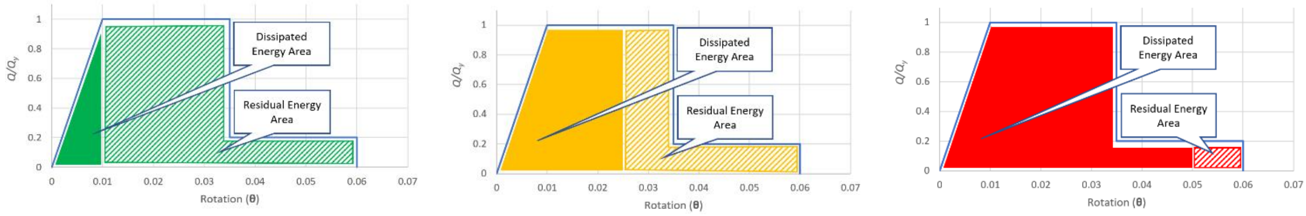


Fig. 8 – Generalized force-deformation relation for concrete elements [17]

Table 10-7 in ASCE 41-17 is referred for rotational limit values of structural rotation (θ) of members for Immediate Occupancy (IO) as 0.01 or 1%, Life Safety (LS) as 0.025 or 2.5%, Collapse Prevention (CP) as 0.05 or 5%. The modelling parameters of reinforced concrete elements in Fig. 8 are selected as *a* value (length from point B to C) to be 0.025, *b* value (length from point B to E) to be 0.05, and *c* value, which is the residual strength, to be 0.2 times of yield strength [17]. In this study, the dissipated energy and residual energy were defined as the Fig. 9 below.





(a) Immediate Occupancy (IO) (b) Life Safety (LS) (c) Collapse Prevention (CP)

Fig. 9 – Adopted justification dissipated and residual energy area for every structural element state

Based on Fig. 9, absolute rotational results of every beam and column elements were inspected and categorized to the justification dissipated and residual energy. For Immediate Occupancy (IO) state, the dissipation energy is defined to be taken 15% total energy represented as solid green triangle area in Fig. 9a, which then this value is referred as $E_{dissipated}$. On the other hand, the residual energy for IO state is 85% referred as $E_{residual}$. This method is also applied for Life Safety (LS) state and Collapse Prevention (CP) which the dissipated energy for LS and CP state is 55% and 95% respectively while the residual energy is 45% and 5%.

5. Result and Discussion

From the modelling with STERA_3D, the natural period of building with corner column removal, mid-long column removal, and mid-short column removal are calculated as 1.094 s, 1.078 s, and 1.083 s respectively. These fundamental periods have slight difference from each other.

In this research, the maximum rotational displacement of the elements under earthquake ground motions is the parameter to decide the level of element's structural performance. After the maximum rotational displacements of the elements were retrieved from non-linear earthquake response analysis using STERA_3D, the corresponding rotation is associated with color and three-dimensional illustration was drawn based on the color as shown in Fig. 10 for every locations of column removal with the detail of defined color is listed in Table 3. The tool used to draw the graph is open-licensed software, FreeCAD, with Python scripting included.

Table 3 – Correlation of structural performance level and defined color

Structural performance level	Defined Color
Immediate Occupancy (IO)	(green)
Life Safety (LS)	(yellow)
Collapse Prevention (CP)	(purple)
Collapse (C)	(red)

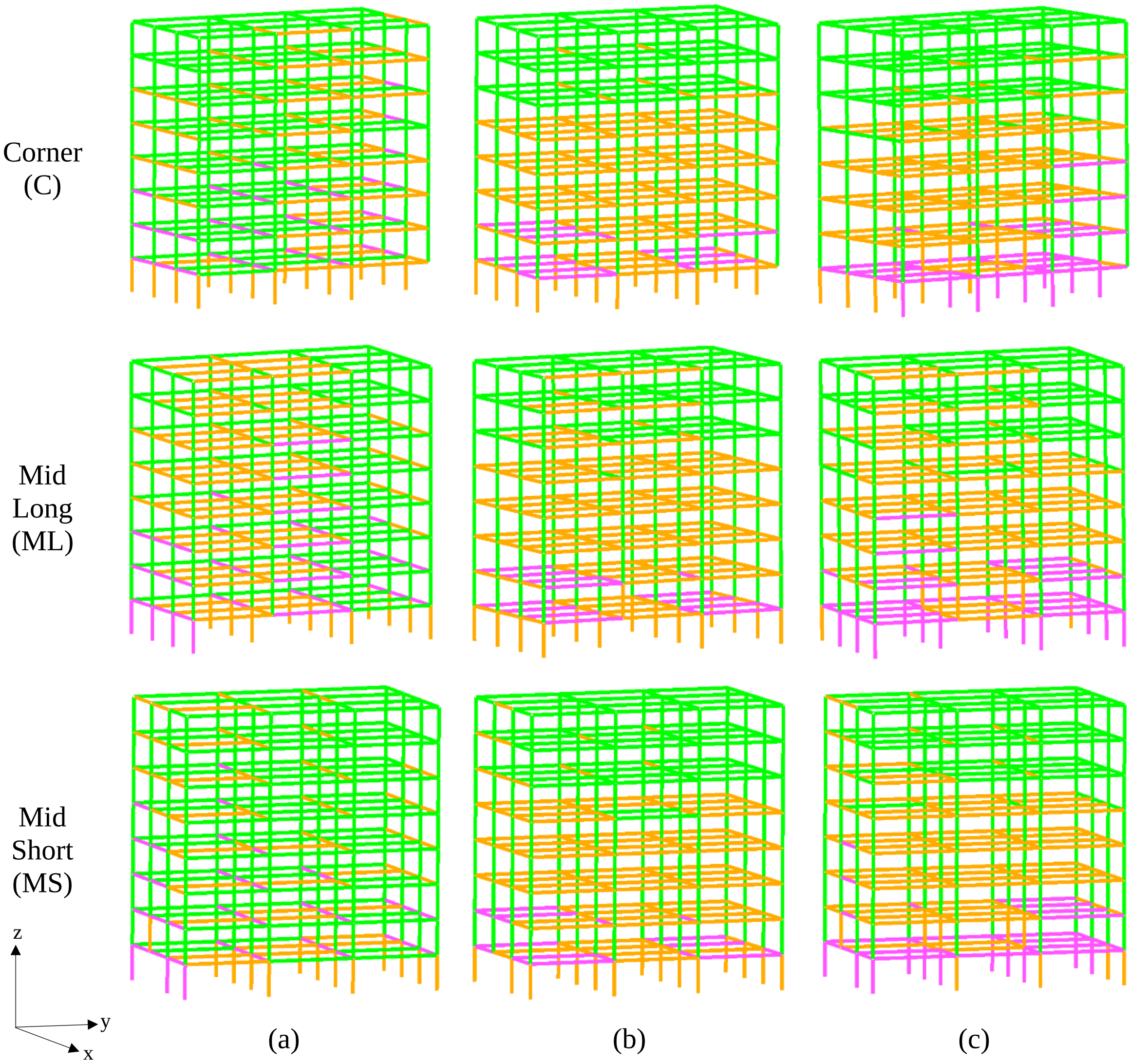


Fig. 10 – Three-dimensional illustration of absolute rotational total for each structural element

In all models, the nearest beams and columns located at above and side of the removal column had higher rotational displacements. There is no rotational displacement of structural element more than 0.05 in the collapse state. The column removal effects local failure especially for beam elements. To decide the detailed effect of column removal, the absolute rotational values were categorized as presented in Table 4 and calculated with ratio of total energy based on its state as delivered in Table 5 and Table 6, which are reflecting the average dissipated and residual energy for inspected case. Then, the dissipated and residual energy were compared to among the column removal location variations. The result of average energy dissipation in each story is presented in Fig. 11. It is seen that the story level one is always the level that dissipate the largest energy compared to other stories. Moreover, Kobe earthquake causes biggest dissipated energy ratio to buildings due to its earthquake phase. The result shows that mid-long case has the highest critical residual energy at storey level one. This confirms the previous research result which used linear analysis by comparing demand-capacity ratio of each structural members [18].

Table 4 – State categorization of beam and column for each level based absolute rotational values for case Mid-Long column removal under Kobe Earthquake

State	**Number of Beam and Column Elements**							
	Storey Level							
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
IO	0	11	16	16	24	32	35	37
LS	8	20	23	23	16	8	5	3
CP	31	9	1	1	0	0	0	0
Total	39	40	40	40	40	40	40	40

Table 5 – Dissipated energy results for case Mid-Long column removal under Kobe Earthquake

State	$E_{dissipated}$ for each state (%)	**Number of Elements × $E_{dissipated}$**							
		Storey Level							
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
IO	15%	0.00	1.65	2.40	2.40	3.60	4.80	5.25	5.55
LS	55%	4.40	11.00	12.65	12.65	8.80	4.40	2.75	1.65
CP	95%	29.45	8.55	0.95	0.95	0.00	0.00	0.00	0.00
$\frac{\text{Total}}{\text{No. of Elements}} =$		0.87	0.53	0.40	0.40	0.31	0.23	0.20	0.18

Table 6 – Residual energy results for case Mid-Long column removal under Kobe Earthquake

State	$E_{residual}$ for each state (%)	**Number of Elements × $E_{residual}$**							
		Storey Level							
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
IO	85%	0.00	9.35	13.60	13.60	20.40	27.20	29.75	31.45
LS	45%	3.60	9.00	10.35	10.35	7.20	3.60	2.25	1.35
CP	5%	1.55	0.45	0.05	0.05	0.00	0.00	0.00	0.00
$\frac{\text{Total}}{\text{No. of Elements}} =$		0.13	0.47	0.60	0.60	0.69	0.77	0.80	0.82

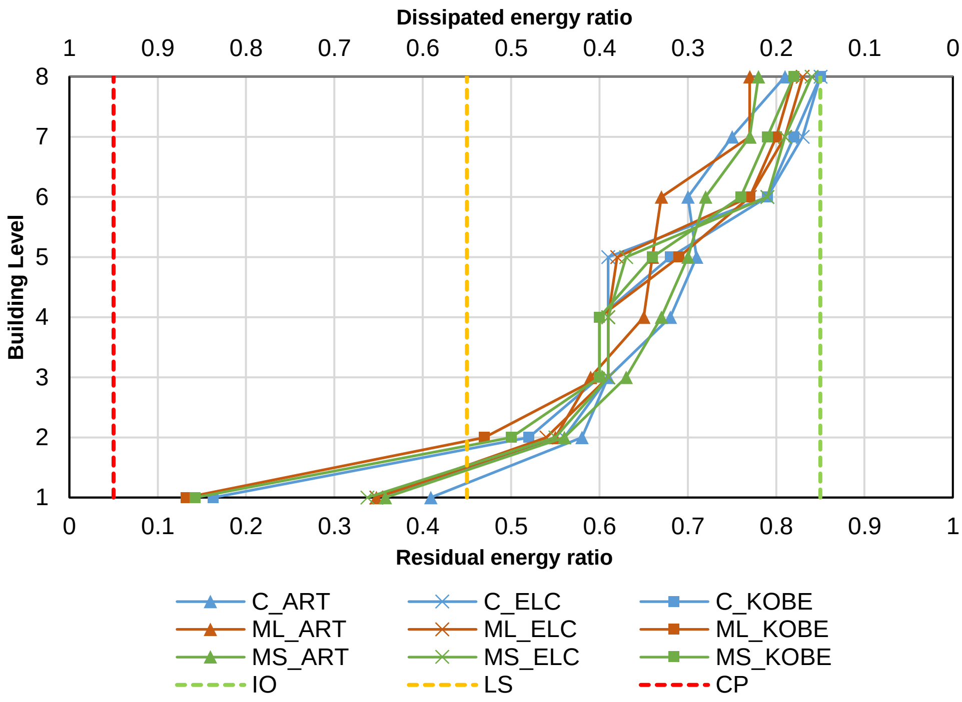


Fig. 11 – Energy ratio based on building level

6. Conclusion

Progressive collapse analysis is one alternative analysis method that can be applied to evaluate building seismic resistance. From the result, it is known that buildings with the structural dimension details designed based on Indonesian code were still in immediate occupancy, life safety, and collapse prevention states under design earthquake ground motions even after removing a column in the first floor, and these structural elements had not been in collapse state. It is found that the mid-long column removal has the worst scenario compared to two other scenarios, namely corner and mid-short column removals, judging by the residual energy ratio left on the first floor.

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