# Design Method for Strong Columns-Weak Beams for Risk Category IV Buildings with Time History Performance Analysis

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*ABSTRACT*

Collapse pattern that considers the design of strong columns-weak beams in structures located in areas prone to large earthquakes such as Indonesia is an important thing that must be considered. When a large earthquake occurs in buildings with risk category IV based on SNI 1726-2019, the flexible parts of the beam elements are allowed to experience damage, while the columns must not experience soft-story mechanisms. To obtain a more realistic picture of it, in this study Nonlinear Time History Analysis (NLTHA) was chosen because it is considered able to provide a picture of the structural behavior that is closer to reality than Pushover Analysis. The results of the 2D time history performance analysis indicate that a beam-column capacity ratio of 1.2, as specified in SNI 2847-2019, renders the structure unable to achieve the expected collapse performance. By using a trial-and-error process to obtain a safe collapse hierarchy, an increase in column capacity of 2 to 3 times the SNI 2847-2019 column design is obtained. Based on this, in designing structures with risk category IV, apart from having to meet the terms and conditions of SNI the structure also needs to be analyzed for its performance using a more realistic methods such as NLTHA (portal) or 3D to ascertain whether the planned failure pattern can occur or not. This is important considering that the beam column capacity ratio in SNI is not sufficient to reach the Collapse Prevention (CP) condition of 95%.

**Keywords**: Nonlinear Time History, Performance Analysis, Soft Story Mechanism

## 1. INTRODUCTION

In the construction world, structural planning of buildings, especially hospitals, requires a careful and measured approach to ensure optimal building safety and performance. Hospitals are vital and important facilities that are included in risk category IV in the SNI 1726-2019. Category IV is the highest level and provides an indication of the building's very crucial function, where when a large earthquake occurs, this structure is expected to still have a good performance. If it fails, the hierarchy of plastic joint formation and the failure pattern must be designed to be ductile enough so that the failure does not occur suddenly and the energy dissipation that occurs can be maximized. The expected ideal structural performance is to have good energy dissipation, indicated by the plastic joints formation of 95% of the total without any soft stories. Along with developments in technology and understanding of structures behavior, performance-based design is becoming increasingly preferred as an approach to ensuring the reliability and durability of a building, especially in hospital buildings which must be able to function even in emergency conditions.

Performance-based design emphasizes the understanding of structural behavior in a particular situation, such as an earthquake, and then adapting that design to meet a specific goal, such as minimizing damage or maximizing user safety. This is especially important in a hospital context with many residents who need protection and access to continuous medical services. Therefore, this study aims to investigate the performance-based design of a 10-story hospital structure using 2D time history analysis, which is expected to provide in-depth insight into the structure's performance under earthquake conditions.

2D time history analysis is a dynamic analysis method that considers changes in time in the lateral forces exerted on a structure. By utilizing available historical earthquake data, this analysis makes it possible to simulate the response of structures to actual shaking. In the context of a 10-story hospital, this analysis is important due to the complexity of the structure and sensitivity to external shocks that can have a significant impact on building performance. To determine the performance of the structure under consideration, time history analysis is carried out nonlinearly with damping and stiffness values that continue to change when an earthquake is applied. Nonlinear analysis will provide an idea of how ductile the structure is and the energy dissipation that occurs when resisting an earthquake [1].

This research is not only focuses on structural reliability aspects, but also considers other factors that influence its performance in emergency situations. Thus, it is hoped that the results can provide practical guidance for engineers and architects in designing hospitals that are not only structurally sound but also able to maintain continuity of medical services when needed, especially in the face of natural disasters such as earthquakes. The role of performance-based design in the hospital context becomes increasingly significant in ensuring the safety and well-being of its residents.

## 2. THEORY AND METHODS

### 2.1 Nonlinear Structural Analysis

To describe the structural performance, nonlinear analysis needs to be carried out. By utilizing the relationship between moment - curvature or moment - rotation in plastic joints of portal elements such as beams and columns, we can observe the damage pattern that may occur. Whether the pattern of damage that occurs is acceptable or not, this is an interesting point to observe, especially in buildings with risk category IV, where buildings/structures in this category are classified as important facilities such as school buildings, hospitals, energy generating centers and others that are not permitted to experienced sudden collapse. The collapse mechanism that is permitted to occur in buildings of this category is a pattern that still allows the building to survive even if it is hit by an earthquake load that is above the plan so as to minimize casualties in the earthquake disaster.

Nonlinear analysis which is quite popular and often used because of its ease of application is pushover analysis, where the structure is given a push load up to a certain limit to see its performance and failure pattern. However, earthquake loads which are analogous to static thrust loads are certainly not very relevant and less realistic, considering that actual earthquake loads are random dynamic movements that are very difficult to predict. Therefore, nonlinear time history analysis is the most reasonable and realistic approach to apply. In the time history analysis process, the response of the structure is calculated in stages for each time interval using the method of mode superposition or direct integration [2]. The earthquake load applied is in the form of a multi-direction time history, the major direction of the earthquake is applied to the weak axis of the portal, while the minor direction is applied to the stronger axis. This is intended to obtain the critical condition of the building when receiving earthquake loads.

The minimum number of earthquake time series recommended by FEMA 356 [3] is three pairs or three sets of data, where one set consists of two earthquakes in a major horizontal direction, a minor one and one vertical direction. Based on the results of the three types of earthquakes analyzed, the response used as a basis for design or evaluation is the largest value among the three, however if the number of earthquake sets used is greater than or equal to 7 pairs, then in the design and evaluation process it is permissible to use the average response of the seven sets analyzed. It should be remembered that the earthquakes used are a set of earthquakes that have gone through a spectral matching process in accordance with the spectral response data of the site under consideration. The spectral adjustment used in this research is a 2D spectral adjustment, considering that what is being reviewed is only the 2D critical portal of the building. The spectra adjustment procedure adapts the steps developed by Yasa [4] to obtain 3D frequency domain spectra adjustments while maintaining the correlation between the three original earthquake directions.

## 2.2 Failure Mechanism

The response modification factor (R) in the SNI 1726-2019 plays a role in reducing earthquake loads used in designing structures. This R value can describe the level of ductility of the structure that must be met. The greater the R value, it means that the structure is permitted to accept smaller earthquake loads, but has strict ductility requirements to ensure the structure is ductile enough to accept large earthquake loads. When receiving large earthquake loads, some structural elements are permitted to enter the plastic zone or undergo yielding, but of course there are certain parts of the structure that are not permitted.

To ensure that the structure has good ductility, total collapse patterns or sudden collapses need to be avoided. There are two types of collapse patterns, namely beam sidesway mechanism and column sidesway mechanism. Beam sidesway mechanism is a damage/plasticization process that occurs at the end of the beam (Figure 1), while column sidesway mechanism, as shown in Figure 2, has plastic joints/damage located at the top and bottom ends of the column.

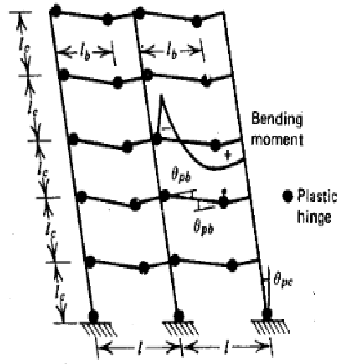



**Figure 1.** Beam sidesway mechanism (R.Park dan T.Paulay, 1975) [5]

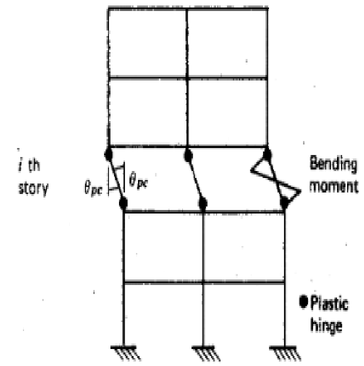



**Figure 2.** Column sidesway mechanism (R.Park dan T.Paulay, 1975) [5]

Column sideways mechanism collapse in structural performance analysis is a failure pattern that is very avoidable because the collapse of the building will occur suddenly. Meanwhile, the recommended failure follows the beam sidesway mechanism, considering that this type of failure has better earthquake energy dissipation or absorption and is described by stable hysteresis with yielding/plastic joints occurring at the ends of the beam and the base of the column only.

To see whether the performance of the structure can achieve the beam sidesway mechanism, NLTHA was carried out whose magnitude was increased gradually to see the distribution of plastic joints and the failure patterns that occurred. Strengthening of columns needs to be done to approach strong column weak beam if a soft-story or column sidesway mechanism occurs, where based on SNI 2847-2019 [6] the minimum ratio of column to beam flexural capacity is 6/5 or 1.2.

### 2.3 Structural Performance Level

The performance level of a structure can be defined by inspecting the level of physical damage to the structure and the threat to the safety of the lives of its occupants. This level describes the condition of structural and non-structural damage at the maximum limit when a large earthquake load occurs. FEMA 356 (2000) [3] divides structural performance into two, namely, Discrete Structural Performance Levels and Intermediate Structural Performance Ranges. Discrete Structural Performance Levels consist of Immediate Occupancy, Life Safety and Collapse Prevention. Intermediate Structural Performance Ranges can be grouped into Damage Control Range and Limited Safety Range. A description of each performance can be seen in the following description.

1. Immediate Occupancy (IO) level: The risk of loss of life at this performance level is very minimal, and physical damage to the structure is not so significant/meaningful that it can be put back into use immediately. Some buildings that must be designed to IO level are buildings with risk category IV, such as hospitals, fire stations and hazardous fuel buildings.
2. Damage Control Level: casualties are still very low, but building structures suffer more severe damage than IO but still lighter than Life Safety. This performance level is generally applied to buildings that have historical value.
3. Life Safety (LS): Damage to structural and non-structural parts is quite significant, but has not experienced partial or complete collapse. Casualties are still relatively low, but the repair process takes quite a long time. Examples of buildings designed at this level are offices, residential warehouses and others.
4. Limited Safety: at this level the level of damage is between Life Safety and Collapse prevention.
5. Collapse Prevention (CP): CP is a level that has a high risk of damage and loss of life, where there is a significant decrease in stiffness and strength in the lateral support system.

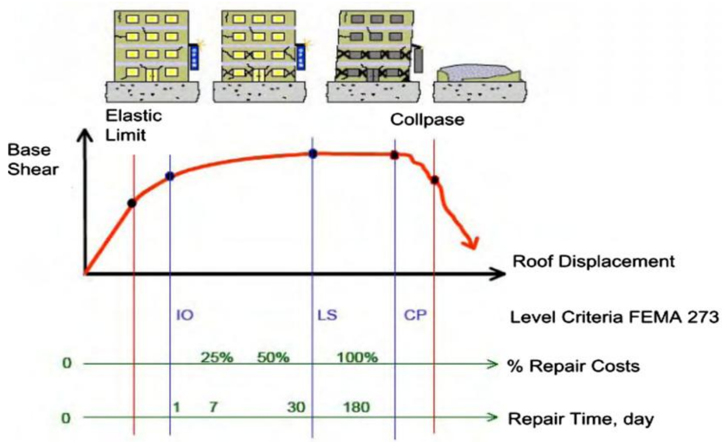



**Figure 3.** Building structure performance level (FEMA 273, 1997) [7]

### 2.4 Plastic Joints

The nonlinear force-displacement behavior is described by modeling plastic joints at both ends of the beam and column elements, where moment type plastic joints (M3) are applied to the beam elements and moment axial combination types (P-M2-M3) to the structural columns. Plastic joints can be defined through the relationship of moment-curvature or moment-rotation from cross-sectional analysis of beams or columns. Apart from cross-sectional analysis, plastic joints can also be obtained through backbone curves from the results of experimental alternating cyclic loads. Meanwhile, in the definition of column plastic joints, it is also necessary to consider the axial variants applied, namely P = 0, P = Pbalance and Pmax.

Apart from the relationship between moment of curvature, moment of rotation or backbone curve from the experimental results of beam-column elements, defining plastic joints also requires the length or zone where plasticization occurs in either the column or beam. There are several methods for determining the length of plastic joints, some of which are: Priestley & Park (1987), Panagiotakos & Fardis (2001) [8] and ACI 318-19 [9]. In this study, the length of the plastic joint follows ACI 318 regulations, where Lp (length of the plastic joint) is taken not less than the cross-sectional height of the beam or column (h) under consideration.

## 2.2 Methods

Collapses that occur in buildings with risk category IV must follow the pattern of collapse of strong columns, weak beams or beam sidesway mechanism, therefore this research discusses specifically how to obtain structures that have ideal performance. The ideal performance in question is that the dominant distribution of plastic joints/plasticization occurs at both ends of the beam, while for columns plastic joints are only permitted to occur at the base. This requirement ensures that there is no soft story mechanism or column sidesway mechanism so that it will provide a sudden collapse. This method is specifically for types of buildings in very important categories, considering that when a large earthquake occurs, buildings with risk category IV are expected to survive and not experience sudden failure.

In this study, the structural design method with category IV was prepared by observing or reviewing a simple building portal model with a hospital function that was included in risk category IV. The structural model is analyzed in two dimensions (2D) to facilitate performance analysis with NLTHA. Generally, to simplify or simplify performance analysis, pushover analysis can be carried out, where the structure is given a static push load which is increased gradually until the beam and column structural elements show a failure pattern. However, this analysis is certainly not as accurate as NLTHA, considering that earthquakes are dynamic loads which are less suitable if represented only as static loads. Therefore, so that the results given are close to the real conditions, the model is simplified to apply NLTHA.

NLTHA requires Time Series data as a reference for applying earthquake loads to the portal/structure being reviewed. The time series used must have characteristics that match the location where the building stands, but often data in the form of time history is not sufficient, so other methods are needed to create artificial time series that match the site location. In this research, 2D earthquake time series adjustments were carried out by applying the method developed by Sindur and Yasa [10], [4], where the simultaneous application of 2D matching will produce an artificial time history with strong correlation between directions. The Kobe earthquake was chosen as the original time history because it was considered to have a large damaging impact by looking at the absorbed power indicator value above 20 hp/ton. Based on Elenas [11] the structural damage index has a good and positive correlation with absorbed power, spectra and earthquake input energy.

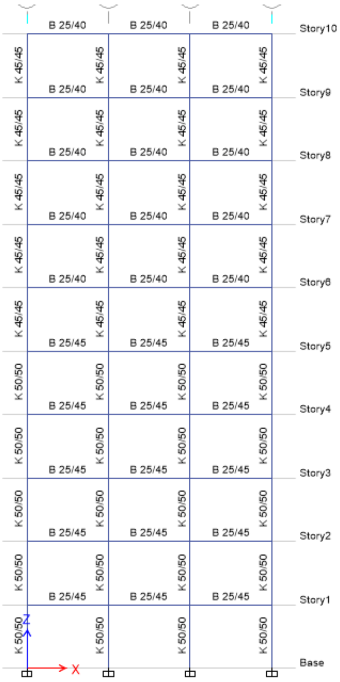



**Figure 4.** 2D Portal Model of the Hospital Structure

## 3. RESULTS AND DISCUSSION

### 3.1 Design of cross-section

In designing the cross section, two analyses are carried out to obtain all the reinforcement in the columns and beams. The first one is carried out to obtain longitudinal reinforcement for the beam and the second run is carried out after the it is defined in the beam section. In this second run, shear reinforcement is obtained in the beam. This is done to ensure that the bending capacity of the beam is weaker than its shear strength, so that shear failure in the beam can be avoided. Apart from ensuring that bending failure occurs before shear, according to SNI 2847-2019 [6] the column to beam capacity ratio must be greater than or equal to 1.2. By considering this, the design of the beam and column reinforcement for the hospital structure model is obtained as follows:

**Table 1**. Beam longitudinal reinforcement

| Beam | D19 | Support reinforcement | | Mid-span reinforcement | |
|---|---|---|---|---|---|
| | | Number | Area (mm$^2$) | Number | Area (mm$^2$) |
| B 25/45 | Top | 3 | 850.59 | 2 | 567.06 |
| | Bottom | 2 | 567.06 | 2 | 567.06 |
| B 25/40 | Top | 3 | 850.59 | 2 | 567.06 |
| | Bottom | 2 | 567.06 | 2 | 567.06 |

**Table 2**. Beam transversal reinforcement

| Beam | Support reinforcement | | Mid-span reinforcement | |
|---|---|---|---|---|
| | Number | Spacing (mm) | Number | Spacing (mm) |
| B 25/45 | 2D10 | 100 | 2D10 | 150 |
| B 25/40 | 2D10 | 100 | 2D10 | 150 |

**Table 3**. Column longitudinal and transversal reinforcement

| Column | Longitudinal reinforcement | Transversal reinforcement | | | |
|---|---|---|---|---|---|
| | | Support reinforcement | | Mid-span reinforcement | |
| | Number | Number | Spacing (mm) | Number | Spacing (mm) |
| K 50/50 | 8D22 | 3D10 | 150 | 3D10 | 150 |
| K 45/45 | 8D19 | 3D10 | 150 | 3D10 | 150 |

### 3.2 Plastic Joints Definition

In carrying out NLTHA, there are several things that need to be done, including: Analysis of cross-sections of beams and columns, adjusting the time series to the target spectral response, and the NLTHA using a time series that has been adjusted to site conditions. The relationship between moment-rotation or moment-curvature is the result of cross-sectional analysis of beams and columns which is applied to define plastic joints in reviewing the condition of each element in the NLTHA. The conditions described through plastic joints will show the pattern of collapse that occurred, and as part of the evaluation to see the performance of the Hospital Building Structure. The following are several moment-rotation relationships in RS structural beams:

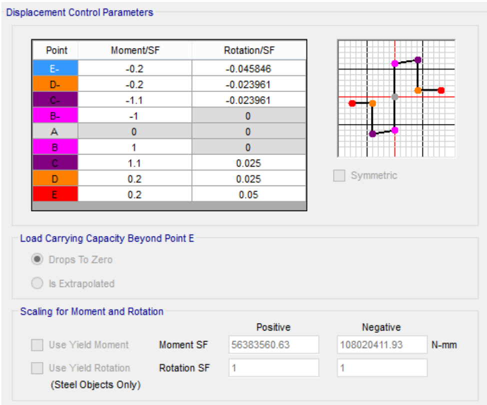



**Figure 5.** Plastic Joint of Beam 25/45 [12]

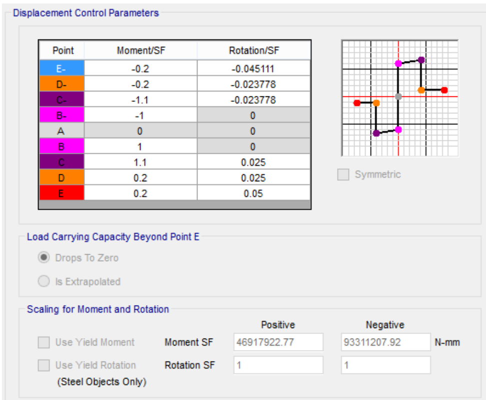
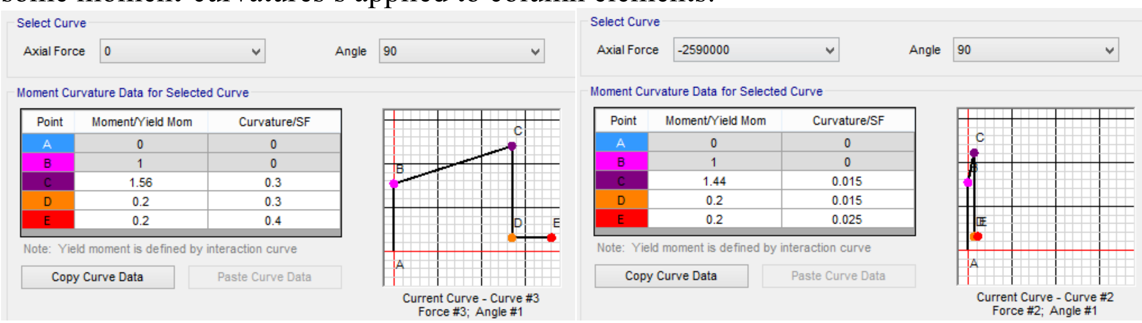



**Figure 6.** Plastic Joint of Beam 25/40 [12]

In designing plastic joints in columns, there are axial force parameters that are needed to determine the moment-curvature relationship. Therefore, it is necessary to calculate the column interaction diagram so that the axial load that will be used can be determined. The axial load used is when it is maximum, in balance conditions, and when there is no axial load. Here are some moment-curvatures s applied to column elements:




(a) (b)

**Figure 7.** (a) Column 45/45 (P = 0) dan (b) Column 45/45 (P = 2590 kN) Balanced Condition [12]

### 3.3 Matching Time Series to Spectra Response Design

Matching time series to the spectral response design is carried out so that the earthquake acceleration motion input used is close to the spectral response design when designing the building, or in accordance with the spectral response that applies to the site where the building stands. Generally, matching is carried out independently for each direction, where the direction of a major earthquake is matched to a major target, a minor earthquake to a minor target, and a vertical earthquake to a vertical target. However, if done independently, the direction cosine and the correlation relationship between the directions will not be maintained, so the connection between the three is not guaranteed. Therefore, 3D or 2D matching is needed to provide direction cosines and correlation relationships in the three directions that are close to the original earthquake. The target used in matching is to combine horizontal and vertical targets using geomean method.

The horizontal spectral response target was taken based on SNI 1726-2019 [13] for Denpasar City, while the vertical spectral response was not set directly in the SNI, so the value was assumed to be 0.5 of the horizontal spectral response. In practice, a more conservative ratio of 2/3 of the horizontal spectrum is recommended, where this value is recommended by ASCE 4-16 [14]. After both responses are obtained, a geomean combination is carried out as shown in Figure 8.

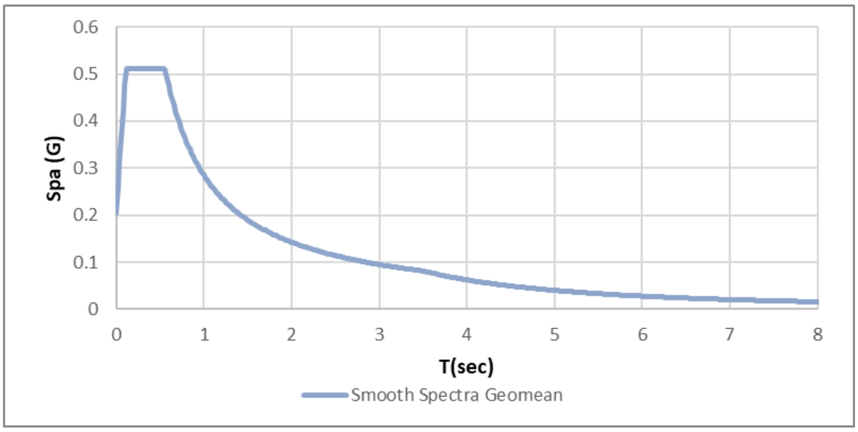



**Figure 8.** Geomean spectra response between horizontal and vertical directions

A comparison of the response spectra of the target, original and after adjusting the 2D spectra in the frequency domain is shown in Figure 9, where it can be seen that the acceleration spectra of the Kobe Earthquake are in accordance with the site targets that apply at the location under consideration. The acceleration time series after adjusting these spectra will later be used as an earthquake load in analyzing the performance of the hospital building portal.

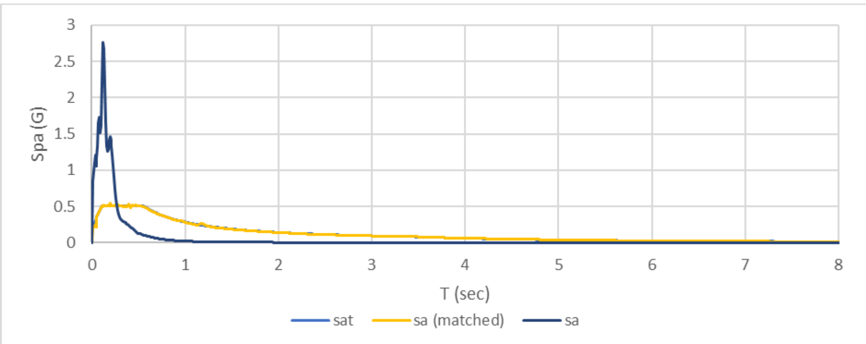



**Figure 9.** Spectral response before (sa) and after adjustment (sa matched)

### 3.4 Non-Linear Time History Analysis (NLTHA)

After the design reinforcement is inserted, the plastic hinges are defined, and matching has been carried out for the earthquake acceleration motion input, the NLTHA is ready to be carried out. In the NLTHA, several checks are carried out, including: the earthquake load given will be multiplied by the scale factor Ie / R, where when the structure is carried out with this load, the previously designed structure must still be in an elastic condition in all structural components. An earthquake load of scale 1 is carried out, where this is an earthquake load based on SNI without reduction due to the response modification factor (R) or amplification due to the priority factor (Ie). Beams may enter the plastic area, but columns may not experience story mechanisms.

The next earthquake load scale continues to be increased until the structure is about to collapse or reaches collapse prevention of 95% of the number of plastic joints in the beam [15]. When the scale of an earthquake increases, if there are columns that experience a story mechanism before the building is about to collapse, then it is necessary to strengthen the columns. In this research, column retrofitting is carried out by increasing the amount of reinforcement.

With the Ie/R earthquake scale factor, it is found that all structural elements are still in an elastic condition. This can happen because the earthquake load with the Ie/R scale is the design strength, where the structure is designed at this level in a linear and elastic manner. If there are structural elements that exceed the elastic zone, it is necessary to evaluate the design that has been carried out. Figure 10.a displays the results of the NLTH analysis on the hospital building model portal for the Ie/R scale.

This earthquake load scale of 1 is equivalent to the earthquake load based on SNI 1726-2019 without any reduction due to the response modification factor (R). The designed structure is permitted to experience plastic in the beams, but is not permitted to experience story mechanisms in the columns. Figure 10.b shows the results of the 2D ETABS hospital structural performance evaluation, with an earthquake load scale of 1.

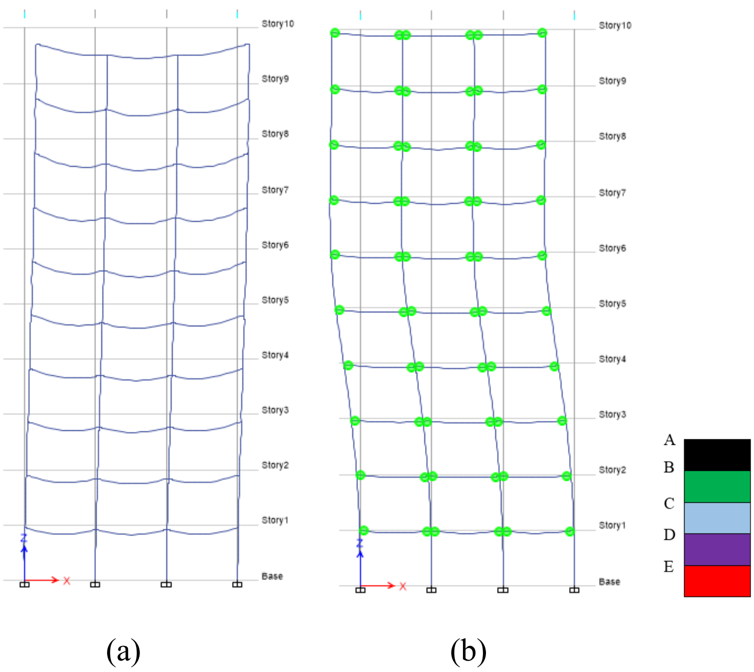



**Figure 10.** (a) distribution of plastic joints for the Ie/R scale and (b) Plastic joints for earthquake scale of 1

The green color in a plastic joint indicates that the part has passed its elastic limit (B-C) but still has the ability to maintain its strength. The light blue color depicts strength degradation (C-D), while the purple and red colors in this task are conditioned on a capacity of 0.2 of the first yield capacity of the beam or column element. The behavior displayed in the structural model is in accordance with what was planned, where the beam elements have experienced plastic while the story mechanism does not occur in the columns. This is because the structure is designed to be elastic, only reaching earthquake loads with a scale factor of Ie/R, the rest of the structure is allowed to experience plastic, but the story mechanism cannot occur.

The next target for increasing the earthquake scale is to reach the building structure before it collapses or achieve collapse prevention of 95% of the number of plastic joints in the beam [15]. Based on FEMA 356 (2000) [3], collapse prevention is represented by deformation at point C (Ultimate point). At earthquake scale 3 (Figure 11), it can be seen that the columns experienced a story mechanism. Apart from that, the beams that reach point C or CP are 5 points or around 8.3% of the total number of plastic joints in the beam. Therefore, to achieve 95% CP, it is necessary to strengthen the structural columns.

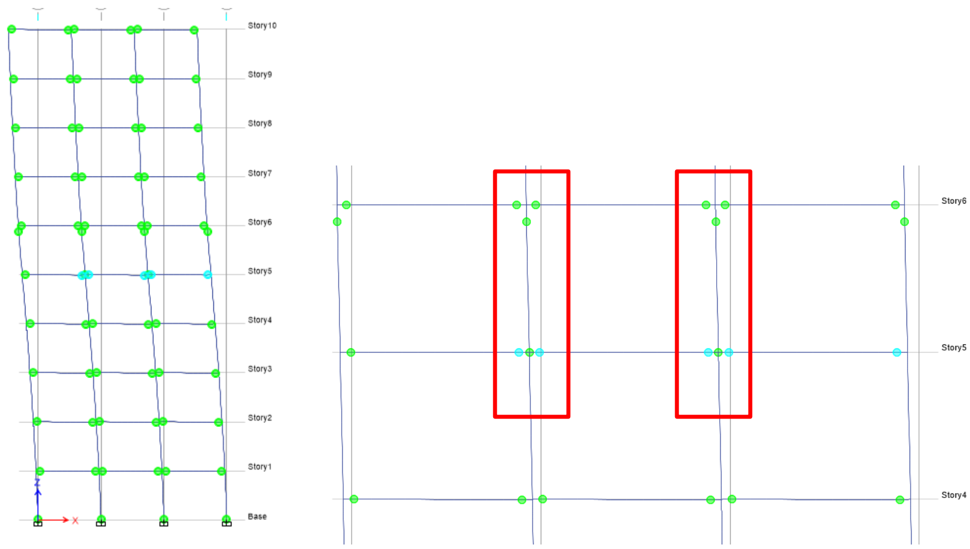



**Figure 11.** Story mechanism occurred at earthquake scale of 3

The appropriate reinforcement and scale factor when the building is about to collapse, or reaches 95% point C (Ultimate point) is sought by trial and error. From the results of this trial and error, it was found that the column reinforcement ratio that could achieve this condition was 4%, and with an earthquake magnification scale of 5.58.

Based on Figure 12 it can be seen that the beams have exceeded point C or CP (Collapse Prevention) by 95%, so it can be said that the building structure is about to collapse under an earthquake load with a scale of 5.58. By comparing the BCC ratio (Beam/Column Capacity Ratio) after and before strengthening, the column enhancement ratio can be determined. Where the increase in columns ranges from approximately 2 to 3 times the initial design strength. Apart from that, this illustrates that the column capacity ratio required by SNI of 1.2 is not able to reach the CP condition of 95% (the structure is about to collapse).

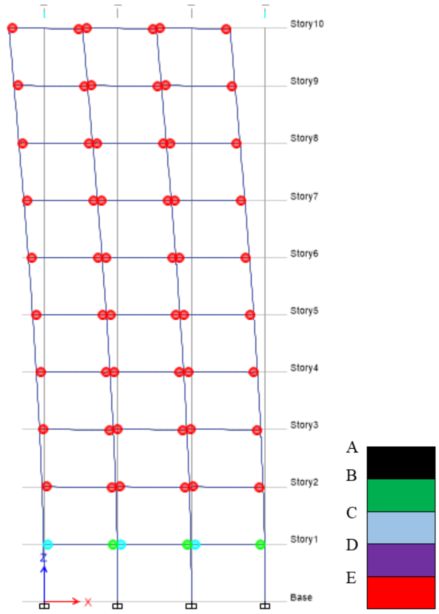



**Figure 12.** Plastic joints distribution of reinforced column cross-section

## 4. CONCLUSIONS

In achieving the expected structural performance, especially those at a very high-risk level or category IV according to the SNI 1726-2019, it is important to consider the ideal collapse. When a large earthquake occurs, beam elements can experience bending damage, but columns must maintain their strength without experiencing a soft story mechanism. This research uses NLTHA because it is considered more capable of representing structural behavior more accurately than the pushover analysis method. The results of the 2D time history performance analysis showed that by applying the beam column capacity ratio according to SNI 2847-2019, the structure is unable to achieve the desired collapse performance. Therefore, re-evaluation of this ratio is necessary, especially in buildings that are very vital or important. Through NLTHA trial and error approach, increasing the column capacity by 2 to 3 times the SNI 2847-2019 column design requirement became necessary to ensure safer collapse mechanism.

Therefore, when designing structures that are classified as risk category IV or vital structures such as hospital buildings, schools, and nuclear reactors, it is not only adequate to comply with SNI standards and provisions, but it is also important to carry out realistic performance analysis such as 2D or 3D Time History. The goal is to ensure that the planned failure pattern will occur. This is important considering the lack of compliance with the beam column capacity ratio according to SNI to achieve a CP condition of 95% (structure nearing collapse).

## REFERENCES

[1] M. Syauqi, R. Suryanita, and Z. Djauhari, "Respon struktur portal baja akibat beban gempa dengan analisis riwayat waktu nonlinier.," *Jom FTEKNIK* , vol. 4, 2017.

[2] N. V. Diredja, Y. A. Pranata, and R. Simatupang, "Analisis dinamik riwayat waktu gedung beton bertulang akibat gempa utama dan gempa susulan," *Dinamika Teknik Sipil*, vol. 12, pp. 70–77, Jan. 2012.

[3] FEMA, *FEMA 356: Prestandard and commentary for the seismic rehabilitation of buildings*. Washington, D.C: Federal Emergency Management Agency, 2000.

[4] P. A. Yasa, “Pengembangan Kumpulan Gerakan Tanah Buatan 3D Kompatibel dengan Desain Spektra,” Institut Teknologi Bandung, Bandung, 2019.

[5] R. Park and T. Paulay, “Reinforced concrete structures,” *John Wiley and Sons*, 1975.

[6] BSN, *SNI 2847:2019 Persyaratan beton struktural untuk bangunan gedung dan penjelasan*. Jakarta: Badan Standardisasi Nasional, 2019.

[7] FEMA, *FEMA 273: Nehrp guidelines for the seismic rehabilitation of buildings*. Washington, D.C: Federal Emergency Management Agency, 1997.

[8] T. B. Panagiotakos and M. N. Fardis, “Seismic response of infilled RC frames structures,” in *Proceedings of 11th World Conference on Earthquake Engineering*, Acapulco, Mexico , 1996.

[9] ACI, *ACI 318-19: Building Code Requirements for Structural Concrete*. Farmington Hills: American Concrete Institute, 2019.

[10] S. P. Mangkoesoebroto, “Seismic performance chart for simple structures,” in *Indonesia Center for Earthquake Engineering*, Bandung: Indonesia Center for Earthquake Engineering, 2007.

[11] A. Elenas, “Correlation between seismic acceleration parameters and overall structural damage indices of buildings,” *Soil Dynamic and Earthquake Engineering*, vol. 20, pp. 93–100, 2001.

[12] Computer and Structures, *CSI Analysis Reference Manuals*. California 94704 USA: University Avenue Berkeley, 2015.

[13] BSN, *SNI 1726:2019 Tata Cara Perencanaan Ketahanan Gempa untuk Struktur Bangunan dan Nongedung*. Jakarta: Badan Standardisasi Nasional, 2019.

[14] ASCE, *ASCE/SEI 4-16 Seismic Analysis of Safety-Related Nuclear Structures*. Reston, Virginia: American Society of Civil Engineers, 2017.

[15] M. H. Prayoga, “Peluang Kegagalan Struktur Rangka Pemikul Momen Kolom Kuat-Balok Lemah Beton Bertulang Akibat Gempa,” Institut Teknologi Bandung, Bandung , 2018.