"International Journal of Applied and Advanced Multidisciplinary Research (IJAAMR)"
Vol.4, No.5, 2026 : 283-300

# Seismic Response of Mid-Rise Steel SMRF with Panel Zone Modeling

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A R T I C L E I N F O

*Keywords: Panel Zone, SMRF Steel Structure, Seismic Response, Response Spectrum Analysis*

*Received : 5 March*
*Revised : 23 April*
*Accepted: 23 Mei*



A B S T R A C T

This study evaluates the effect of panel zone modeling on the seismic response of a six-story Special Moment-Resisting Frame (SMRF) steel structure using Response Spectrum Analysis (RSA) in accordance with SNI 1726:2019 and the Indonesian Design Spectrum for Padang City. Two ETABS models, with and without panel zones, were analyzed under identical conditions. Structural responses evaluated included displacement, inter-story drift, story shear, story stiffness, and P-Delta effects. The results show that panel zones increase displacement, inter-story drift, and stability coefficients while reducing story stiffness, particularly in the X-direction. However, changes in story shear were relatively insignificant. All structural response parameters remained within the allowable limits specified by SNI regulations, indicating satisfactory seismic performance for both models

DOI: https://doi.org/10.59890/ijaamr.v4i5.218
E-ISSN : 3026-2437
https://nvlmultitechpublisher.my.id/index.php/ijaamr/index

## INTRODUCTION

Geographically, Indonesia lies within the Ring of Fire, which results in high seismic activity in the region. This makes the design of earthquake-resistant buildings a critical aspect in the construction industry, particularly for structures located in areas with high earthquake potential (Afiah et al., 2023, 2025). One region with significant earthquake risk is Padang, on Sumatra's western coast near an active subduction zone. Steel structures are widely used in earthquake-resistant design due to their ductility, light weight, and high energy dissipation (Ilham, 2020; Saputra et al., 2020). Among these, the Special Moment-Resisting Frame (SMRF) system is commonly adopted in earthquake-prone areas because it resists deformation and avoids collapse under large seismic loads (Liu et al., 2025).

The behavior of column-beam connections in steel structures has a significant influence on the structure's global response. At these connection points, there is a zone known as the panel zone, located precisely at the intersection of the column and beam in a steel structure. This panel zone resists a combination of shear forces and moments resulting from the interaction between the beam and column during lateral loading (Rasuna, 2025; Satria, 2021). In structural modeling practice, the presence of the panel zone is often ignored under the assumption that the connection is fully rigid. However, an adequately designed panel zone can significantly improve the seismic behavior of Moment-Resisting Frame systems subjected to earthquake loading (Sepasdar et al., 2020).

Studies investigating the effect of panel zone on SMRF steel structures using Indonesian Design Spectrum parameters remains relatively limited, especially for mid-rise steel buildings. Accordingly, this study examines the influence of panel zone modeling on the seismic response of a six-story steel structure. Furthermore, the results of this study are expected to highlight the significance of incorporating panel zone considerations into the analysis of earthquake-resistant steel buildings to achieve more representative structural behavior.

## LITERATURE REVIEW

### *Special Moment-Resisting Frame (SMRF) System*

The Special Moment-Resisting Frame (SMRF) System is a steel structural system that resists seismic lateral forces through beam-column moments (Rasuna, 2025). Its ductility allows stable inelastic deformation under large seismic loads (Joju, 2020). SMRF is extensively implemented in steel-framed structures because of its strength, stiffness, and energy dissipation (Zhai et al., 2022). The SMRF design philosophy follows the *strong column-weak beam* concept, in which yielding is intentionally expected to occur in the beam before column failure (Setiawan et al., 2026). This concept aims to prevent sudden structural collapse and maintain the stability of the structural system during an earthquake.

### *Panel Zone in Steel Structures*

A panel zone is the area on the flanges and web of a column profile located at the junction between a beam and a column (Ramadhan, 2021). When the structure is subjected to lateral loads from an earthquake, this part undergoes shear deformation due to the force transmission between the beam and the column (Rasuna, 2025). Deformation occurring within the panel zone can influence joint behavior and the overall structural response (Sepasdar et al., 2020). In structural modeling, the application of a panel zone at the beam–column connection can help improve the structure's deformation capacity under seismic loads (Lu et al., 2023). The implementation of a panel zone without doubler plates or with relatively small thickness can lead to yielding on the sides of the panel zone and result in relatively high stresses (Rasuna et al., 2024).

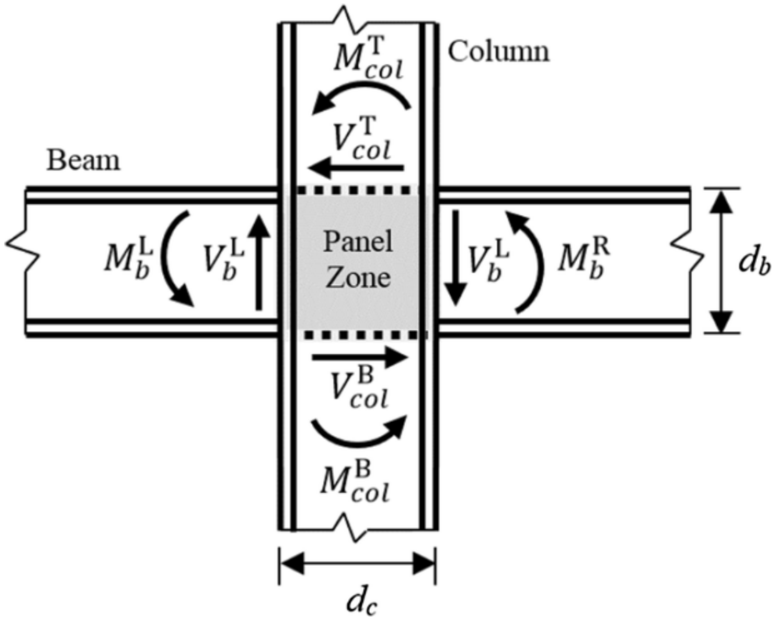



Figure 1. Moment and Shear Forces Occurring at the Beam-Column Junction (Ramadhan, 2021)

***Seismic Response of Structure***

The seismic response of a structure refers to its behavior under lateral loads induced by an earthquake. Structural response parameters are generally assessed based on displacement, inter-story drift, story shear, story stiffness, and the influence of P-Delta secondary effects (Hidayatullah & Walujodjati, 2025). Displacement and inter-story drift are used to evaluate the lateral deformation of a structure due to seismic loading. The greater the deformation that occurs, the higher the structural flexibility will be (Afrida & Trimurtiningrum, 2023). In SMRF steel structures, lateral deformation is influenced by mass distribution, structural stiffness, and beam–column connection behavior (Rasuna, 2025).

Structural stiffness (story stiffness) is directly related to the structural deformation response. A decrease in stiffness can lead to increased displacement and drift due to the structure's reduced ability to resist lateral deformation (Hutubessy, 2022). In steel structures, changes in stiffness can be influenced by joint deformation, including panel zone deformation (Sepasdar et al., 2020). In addition, the structural response is also influenced by the distribution of lateral forces, such as story shear, as well as the secondary P-Delta effect. The P-Delta effect occurs due to the interaction between gravitational forces and the lateral deformation of the structure (Hidayatullah & Walujodjati, 2025). The greater the structural deformation, the greater the contribution of the P-Delta effect to structural stability. Previous studies have shown that modeling the panel zone can affect the displacement, drift, and stiffness values of earthquake-resistant steel structures by increasing lateral deformation and reducing structural stiffness (Isaincu et al., 2018; Kalapodis, 2025; Skiadopoulos & Lignos, 2022; Xu et al., 2024). Therefore, evaluating the seismic response of the structure is crucial to understanding the impact of using the panel zone on the behavior of SMRF steel structures.

***Response Spectrum Analysis (RSA)***

Response Spectrum Analysis (RSA) is a linear dynamic analysis method used to assess a structure's behavior under seismic excitation. The response spectrum is plotted as a graph relating the structural natural period ($T$) versus the maximum acceleration response ($S_a$) (Erick & Susilo, 2022). This method is widely used in seismic-resistant structural design because it can describe the dynamic behavior of a structure more accurately than equivalent static analysis.

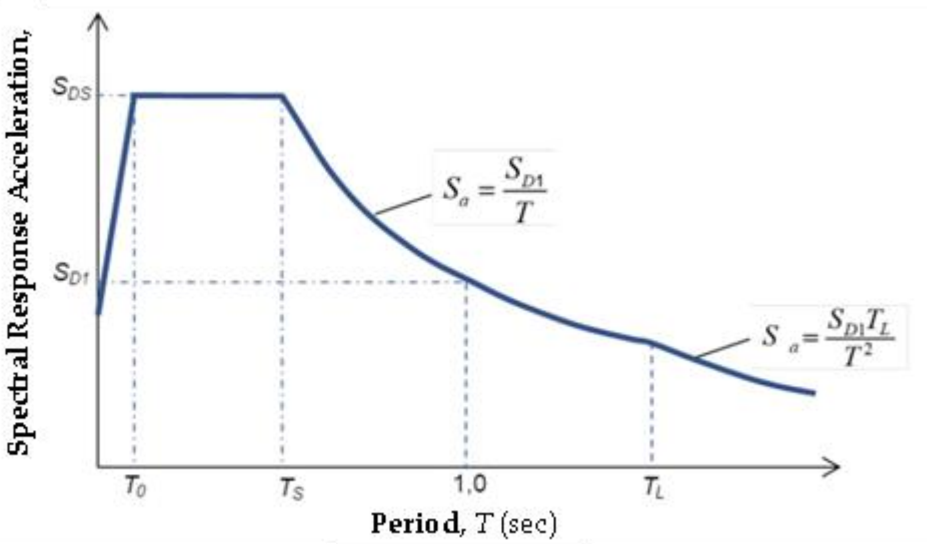
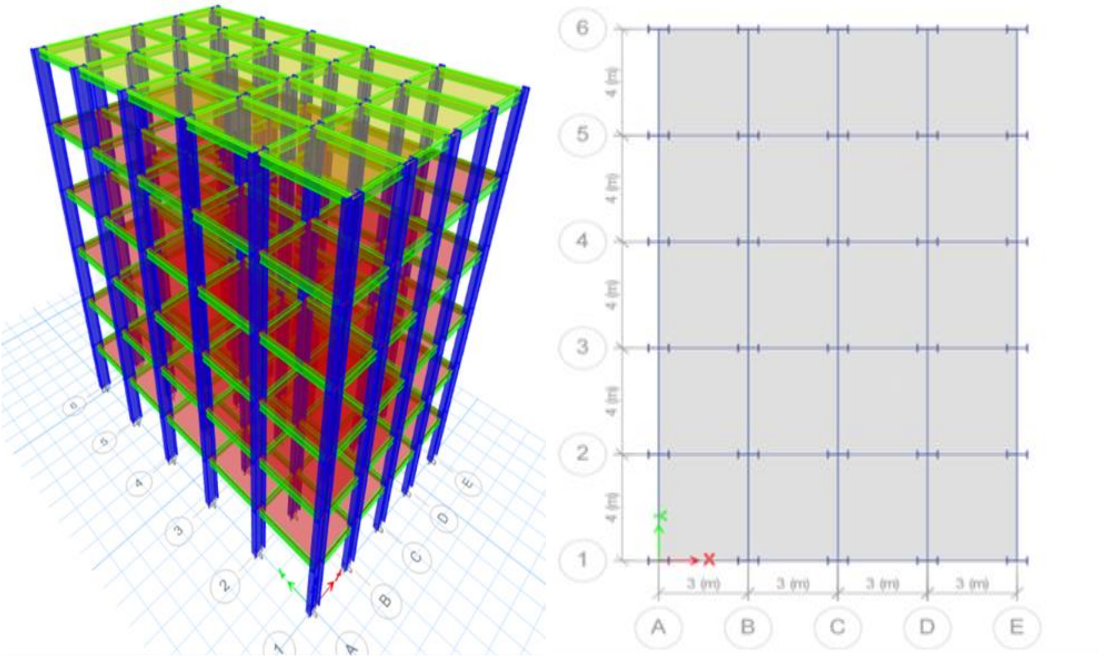



Figure 2. Design Response Spectrum Graph

In response spectrum analysis, verification of the final analysis results is required, particularly for evaluating the magnitude of the base shear force. In accordance with SNI 1726:2019 Section 7.9.1.4.1, if the base shear obtained from the response spectrum analysis ($V_t$) is less than 100% of the base shear determined using the equivalent lateral force procedure ($V_s$), then the result of the base shear must be scaled by a factor of $V_s/V_t$ (Badan Standardisasi Nasional, 2019).

## METHODOLOGY

This study used a numerical approach with ETABS software to analyze the seismic response of the SMRF steel structure. The structure was modeled as a six-story steel building measuring 12 m x 20 m, with a total height of 21 meters and a floor-to-floor height of 3.5 meters.




(a) (b)

Figure 3. Modeling of the SMRF steel structure in the ETABS application:
(a) 3D view, (b) Plan view

The main structural members use IWF 700.300.13.24 steel profiles for columns and IWF 350.175.7.11 profiles for beams, with BJ-41 steel with a yield strength ($f_y$) of 250 MPa and an ultimate tensile strength ($f_u$) of 410 MPa. Meanwhile, for floor and Roof Deck elements, reinforced concrete with a compressive strength of $f'_c$ = 30 MPa is used with thicknesses of 12 cm and 10 cm, respectively. Structural modeling was performed under two conditions: a model without panel zone and a model with panel zone. All structural parameters, loading conditions, and analysis settings were set identically so that the resulting response differences were solely due to the presence of the panel zone.

The seismic parameters were determined in accordance with SNI 1726:2019 and the Indonesian Design Spectrum for the Padang region with site class SE. The parameters used in this study were $S_s$=1.1245 and $S_1$=0.5737, with site coefficients $F_a$=1.0004 and $F_v$=2.0526. Based on these values, the design spectral acceleration parameters obtained were $S_{DS}$=0.75 and $S_{D1}$=0.7851. These parameters were subsequently used to develop the design response spectrum and to perform the dynamic response spectrum analysis of the structure.

In this study, the base shear force obtained from the response spectrum analysis was analyzed to verify whether the dynamic base shear ($V_t$) satisfied the minimum requirement of 100% of the equivalent lateral force base shear ($V_s$) in accordance with SNI 1726:2019.

## RESEARCH RESULT

### *Modal Analysis*

A modal analysis was performed to evaluate the dynamic properties of the structure, including natural periods, vibration frequencies, and modal mass participation. This evaluation is important to confirm that the structural model satisfies the requirements for dynamic analysis in accordance with the applicable seismic design regulations.

Based on ETABS analysis, 18 structural vibration modes were obtained, with the largest fundamental period ($T$) occurring in Mode 1 at 0.926 seconds. The structural period and frequency values are shown in Table 1.

Table 1. Periods and Frequencies

| **Mode** | **Period** (sec) | **Frequency** (cyc/sec) |
|---|---|---|
| 1 | 0.926 | 1.080 |
| 2 | 0.524 | 1.909 |
| 3 | 0.515 | 1.941 |
| 4 | 0.304 | 3.294 |
| 5 | 0.178 | 5.620 |
| 6 | 0.152 | 6.560 |
| 7 | 0.152 | 6.589 |
| 8 | 0.126 | 7.947 |
| 9 | 0.100 | 10.047 |
| 10 | 0.086 | 11.569 |
| 11 | 0.077 | 13.012 |
| 12 | 0.076 | 13.184 |
| 13 | 0.048 | 20.984 |
| 14 | 0.046 | 21.534 |
| 15 | 0.034 | 29.461 |
| 16 | 0.033 | 30.495 |
| 17 | 0.028 | 36.265 |
| 18 | 0.027 | 37.703 |

Before being used in structural response analysis, the fundamental period from the ETABS analysis is first evaluated against the minimum and maximum period limits as specified by earthquake regulations. The calculation results yielded a minimum structural period $T_{a\,min}$=0.82701617 seconds and a maximum period of $T_{a\,max}$ = 1.15782264 seconds.

Since the period values from the analysis satisfy the relationship $T_{a\,min} < T_{ETABS} < T_{max}$, the fundamental period $T$ used in the analysis is the period from the ETABS analysis, which is 0.926 seconds.

Meanwhile, regarding the mass participation of the structural model, the analysis results show that Mode 1 is dominated by Y-direction translation, with a mass participation of 84.22%, while Mode 2 is dominated by X-direction translation, with a mass participation of 78.91%.

In addition, the cumulative mass participation of the structure meets the requirements for dynamic analysis. The cumulative mass participation values in the X-direction (SumUX) and Y-direction (SumUY) each reached 100% at the final mode of the analysis.

Table 2. Modal Participating Mass Ratios

| Mode | Ux | Uy | Sum-UX | Sum-UY |
|---|---|---|---|---|
| 1 | 0 | 0.842 | 0 | 0.842 |
| 2 | 0.781 | 0 | 0.789 | 0.842 |
| 3 | 0 | 0 | 0.789 | 0.842 |
| 4 | 0 | 0.098 | 0.789 | 0.940 |
| 5 | 0 | 0.035 | 0.789 | 0.975 |
| 6 | 0.124 | 0 | 0.905 | 0.975 |
| 7 | 0 | 0 | 0.905 | 0.975 |
| 8 | 0 | 0.016 | 0.905 | 0.991 |
| 9 | 0 | 0.007 | 0.905 | 0.998 |
| 10 | 0 | 0.002 | 0.905 | 1 |
| 11 | 0 | 0 | 0.905 | 1 |
| 12 | 0.052 | 0 | 0.957 | 1 |
| 13 | 0 | 0 | 0.957 | 1 |
| 14 | 0.027 | 0 | 0.983 | 1 |
| 15 | 0 | 0 | 0.983 | 1 |
| 16 | 0.013 | 0 | 0.996 | 1 |
| 17 | 0 | 0 | 0.996 | 1 |
| 18 | 0.004 | 0 | 1 | 1 |

Based on these results, the structural model meets the minimum structural mass participation requirement of 90% as specified in the spectral response analysis. Thus, the structural model is deemed suitable for further dynamic analysis.

***Base Shear Force***

According to SNI 1726:2019, the dynamic base shear force obtained from spectral response analysis ($V_t$) must not be less than 100% of the equivalent static base shear force ($V_s$).

Table 3. Comparison of Dynamic and Equivalent Static Base Shear Force

| Direction | $V_{dynamic}$ (kN) | $V_{static}$ (kN) |
|---|---|---|
| X-Dir | 1084.3920 | 1084.3586 |
| Y-Dir | 1084.3917 | 1084.3586 |

Based on the table, the analysis results show that the dynamic base shear forces in the X- and Y-directions are very close to 100% of the equivalent static base shear force, thus meeting the requirements of SNI 1726:2019. The differences in values are relatively very small and can therefore be ignored.

***Displacement***

A displacement analysis was conducted to evaluate the magnitude of the structure's lateral displacement due to seismic loading based on the Response Spectrum Analysis (RSA) method. This displacement evaluation is important for determining the structure's flexibility and the effect of the panel zone on the building's global deformation.

Based on ETABS analysis, the structural model with a panel zone showed greater displacement than the model without one, particularly in the X-direction. Meanwhile, in the Y-direction, no change in displacement was found between the two structural models.

Table 4. Comparison of Maximum Structural Displacements

| Floor Level | X-Direction | | | Y-Direction | | |
|---|---|---|---|---|---|---|
| | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** |
| Roof Deck | 13.796 | 11.467 | 20.31% | 38.225 | 38.225 | 0% |
| 6th Floor | 12.337 | 10.338 | 19.34% | 36.184 | 36.184 | 0% |
| 5th Floor | 10.238 | 8.650 | 18.36% | 31.785 | 31.785 | 0% |
| 4th Floor | 7.513 | 6.413 | 17.15% | 25.164 | 25.164 | 0% |
| 3rd Floor | 4.438 | 3.845 | 15.42% | 16.779 | 16.779 | 0% |
| 2nd Floor | 1.569 | 1.392 | 12.72% | 7.359 | 7.359 | 0% |

Based on these results, the largest maximum displacement occurred on the roof deck in the Y-direction, amounting to 38.225 mm for both structural models. Meanwhile, in the X-direction, the maximum displacement of the structure with a panel zone reached 13.796 mm, exceeding that of the structure without a panel zone, which was 11.467 mm.

The increase in X-direction displacement due to the use of the panel zone ranged from 12.72% to 20.31% across all building levels. The largest displacement difference occurred on the roof deck, while the smallest difference occurred on the second floor.

***Inter-story Drift***

An analysis of inter-story drift was conducted to evaluate the lateral deformation behavior of the structure under seismic loading in accordance with applicable SNI standards. In this study, the allowable inter-story drift for special moment-resisting steel frame structures with Seismic Design Category D was calculated using the equation:

$$\Delta_{max} = \Delta_a / \rho \text{ .............. (1)}$$

where the value of $\Delta_a$ for all other structures with Risk Category II can be calculated as:

$$\Delta_a = 0.020\, h_{sx} \ \ldots\ldots\ldots\ldots\ (2)$$

With a story-to-story height $h_{sx}$ of 3500 mm and a redundancy factor $\rho$ = 1.3, the allowable story-to-story deflection limit $\Delta_{max}$ is 53.846 mm. Meanwhile, the inelastic displacement is calculated using the formula:

$$\Delta = \delta \times \frac{C_d}{I_e} \ \ldots\ldots\ldots\ldots\ (3)$$

where deflection amplification factor $C_d$ = 5.5 and earthquake importance factor $I_e$ = 1.0.

The analysis results show that both structural models still meet the allowable inter-story drift limit. However, the structure with a panel zone produces larger displacement and drift values compared to the structure without a panel zone, particularly in the X-direction.

Table 5. Inter-Story Drift without Panel Zone

| Floor Level | h (mm) | Displaccement | | Elastic Drift | | Inelastic Drift | | Drift Limit (mm) | Check |
|---|---|---|---|---|---|---|---|---|---|
| | | $D_X$ (mm) | $D_Y$ (mm) | $\delta e_X$ (mm) | $\delta e_Y$ (mm) | $\Delta_X$ (mm) | $\Delta_Y$ (mm) | | |
| Roof Deck | 3500 | 11.467 | 38.225 | 1.129 | 2.041 | 6.210 | 11.226 | 53.846 | OK |
| 6th Floor | 3500 | 10.338 | 36.184 | 1.688 | 4.399 | 9.284 | 24.195 | 53.846 | OK |
| 5th Floor | 3500 | 8.650 | 31.785 | 2.237 | 6.621 | 12.304 | 36.416 | 53.846 | OK |
| 4th Floor | 3500 | 6.413 | 25.164 | 2.568 | 8.385 | 14.124 | 46.118 | 53.846 | OK |
| 3rd Floor | 3500 | 3.845 | 16.779 | 2.453 | 9.420 | 13.492 | 51.810 | 53.846 | OK |
| 2nd Floor | 3500 | 1.392 | 7.359 | 1.392 | 7.359 | 7.656 | 40.475 | 53.846 | OK |

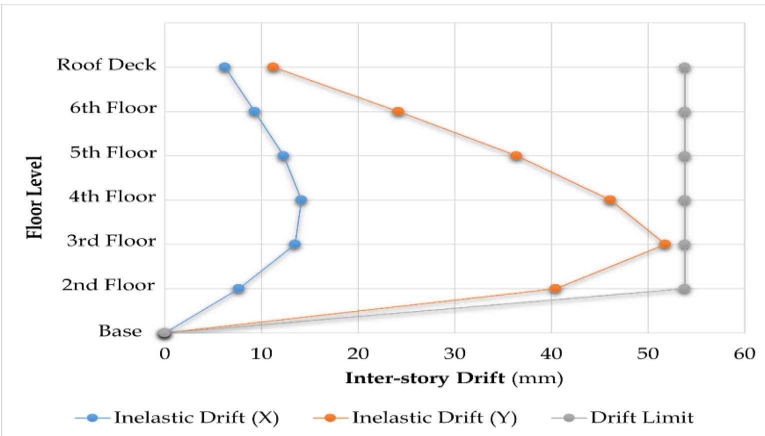



Figure 4. Inter-Story Drift Graph: without Panel Zone

Table 6. Inter-Story Drift with Panel Zone

| Floor Level | | Displaccement | | Elastic Drift | | Inelastic Drift | | | Check |
|---|---|---|---|---|---|---|---|---|---|

| | *h* (mm) | $D_X$ (mm) | $D_Y$ (mm) | $\delta e_X$ (mm) | $\delta e_Y$ (mm) | $\Delta_X$ (mm) | $\Delta_Y$ (mm) | **Drift Limit** (mm) | |
|---|---|---|---|---|---|---|---|---|---|
| Roof Deck | 3500 | 13.796 | 38.225 | 1.459 | 2.041 | 8.025 | 11.226 | 53.846 | OK |
| 6th Floor | 3500 | 12.337 | 36.184 | 2.099 | 4.399 | 11.545 | 24.195 | 53.846 | OK |
| 5th Floor | 3500 | 10.238 | 31.785 | 2.725 | 6.621 | 14.988 | 36.416 | 53.846 | OK |
| 4th Floor | 3500 | 7.513 | 25.164 | 3.075 | 8.385 | 16.913 | 46.118 | 53.846 | OK |
| 3rd Floor | 3500 | 4.438 | 16.779 | 2.869 | 9.420 | 15.780 | 51.810 | 53.846 | OK |
| 2nd Floor | 3500 | 1.569 | 7.359 | 1.569 | 7.359 | 8.630 | 40.475 | 53.846 | OK |

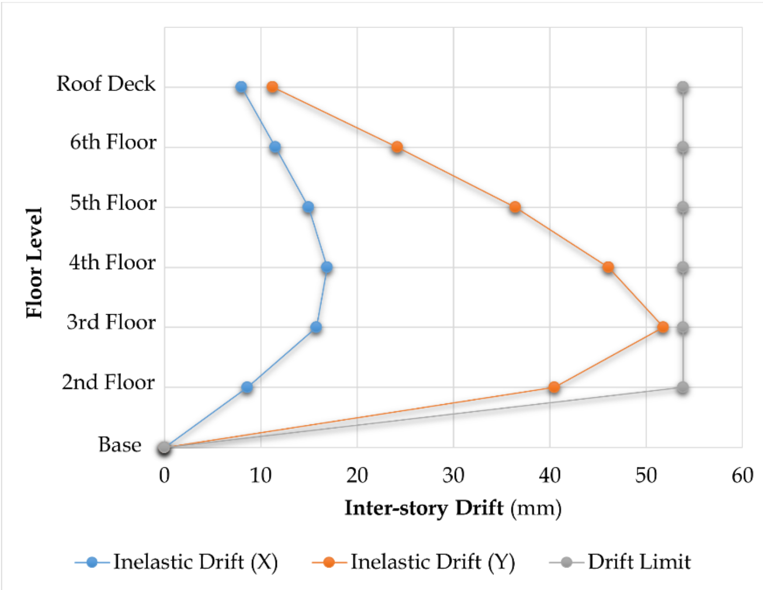


Figure 5. Inter-Story Drift Graph: with Panel Zone

Based on these results, the structural model with the panel zone shows an increase in inter-story deflection in the X-direction across all floors compared to the model without panel zone. The largest increase occurred in the roof deck at 29.23%, while the smallest increase occurred on the second floor at 12.72%.

Meanwhile, in the Y-direction, no change in inter-story drift values was found between the two structural models. The Y-direction drift values across all floors were identical for both the model with panel zone and the model without panel zone.

The maximum inter-story drift value occurred in the Y-direction on the 3rd floor at 51.810 mm. This value remains below the allowable limit of 53.846 mm; therefore, both structural models are deemed to meet the inter-story drift performance requirements based on the applicable SNI standards.

***Story Shear and Stiffness***

Story shear and stiffness analyses were performed to evaluate the distribution of lateral forces and changes in structural stiffness induced by the panel zone. These parameters are important for understanding the effect of the panel zone on the global behavior of the structure under seismic loading.

The analysis results show that the use of panel zone affects the story shear and structural stiffness values, particularly in the X-direction. Meanwhile, the Y-directional responses are relatively small and tend to be insignificant.

Table 7. Comparison of Structural Story Shear

| Floor Level | X-Direction | | | Y-Direction | | |
|---|---|---|---|---|---|---|
| | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** |
| Roof Deck | 217.883 | 214.987 | 1.35% | 194.186 | 194.186 | 0% |
| 6th Floor | 510.430 | 508.368 | 0.41% | 472.784 | 472.784 | 0% |
| 5th Floor | 746.167 | 747.159 | -0.13% | 707.309 | 707.309 | 0% |
| 4th Floor | 921.861 | 926.330 | -0.48% | 892.896 | 892.896 | 0% |
| 3rd Floor | 1032.179 | 1039.626 | -0.72% | 1022.296 | 1022.296 | 0% |
| 2nd Floor | 1075.188 | 1084.392 | -0.85% | 1084.392 | 1084.392 | 0% |

Based on these results, the change in story shear values due to the use of the panel zone is relatively small. In the X-direction, the difference in story shear values ranges from -0.85% to 1.35%. Meanwhile, in the Y-direction, no change in story shear values was found between the two structural models.

The highest story shear value occurs on the 2nd floor, which is 1075.188 kN for the model with panel zone and 1084.392 kN for the model without panel zone.

Table 8. Comparison of Structural Story Stiffness

| Floor Level | X-Direction | | | Y-Direction | | |
|---|---|---|---|---|---|---|
| | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** | **Without Panel Zone (mm)** | **With Panel Zone (mm)** | **Difference (%)** |
| Roof Deck | 147.481 | 187.851 | -21.49% | 91.130 | 91.130 | 0% |
| 6th Floor | 204.970 | 298.334 | -31.30% | 104.669 | 104.669 | 0% |

| | | | | | | |
|---|---|---|---|---|---|---|
| 5th Floor | 272.664 | 332.540 | -18.01% | 105.461 | 105.461 | 0% |
| 4th Floor | 299.066 | 360.119 | -16.95% | 106.001 | 106.001 | 0% |
| 3rd Floor | 359.630 | 423.680 | -15.12% | 108.438 | 108.438 | 0% |
| 2nd Floor | 685.413 | 779.024 | -12.02% | 147.354 | 147.354 | 0% |

The analysis results show that the structure with the panel zone exhibits a decrease in stiffness in the X-direction compared to the model without it. The largest decrease in stiffness occurs on the 6th floor at 31.30%, while the smallest decrease occurs on the 2nd floor at 12.02%.

In the Y-direction, the story stiffness values are identical for both structural models, indicating that the effect of the panel zone on stiffness in the Y-direction is relatively insignificant.

***P-Delta Effect***

A P-Delta analysis is performed to determine the structural stability resulting from lateral deformation under seismic loads. According to SNI, the P-Delta effect can be neglected if the structural stability coefficient ($\theta$) is ≤ 0.10. The value of $\theta$ is calculated using the following equation:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \text{ ............... (4)}$$

The maximum structural stability limit is calculated using the equation:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \text{ ............... (5)}$$

In this study, $\beta$ = 1 and $C_d$ = 5.5 were used, yielding $\theta_{max}$ = 0.0909. For the $\theta$ values obtained from the analysis are presented in Table 9 and 10.

Table 9. Effect of P-Delta on Structure Without Panel Zone

| Floor Level | Inelastic Drift | | Story Forces | | | Stability Coefficient | | P-Delta Limit (mm) | $\theta_{max}$ | Check |
|---|---|---|---|---|---|---|---|---|---|---|
| | $\Delta_X$ (mm) | $\Delta_X$ (mm) | $P$ (kN) | $V_X$ (kN) | $V_Y$ (kN) | $\theta_X$ | $\theta_Y$ | | | |
| Roof Deck | 6.210 | 6.210 | 1083.79 | 214.99 | 194.19 | 0.0016 | 0.0033 | 0.1 | 0.0909 | OK |
| 6th Floor | 9.284 | 9.284 | 3142.46 | 508.37 | 472.78 | 0.0030 | 0.0084 | 0.1 | 0.0909 | OK |
| 5th Floor | 12.304 | 12.304 | 5201.13 | 747.16 | 707.31 | 0.0044 | 0.0139 | 0.1 | 0.0909 | OK |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4th Floor | 14.124 | 14.124 | 7259.80 | 926.33 | 892.90 | 0.0058 | 0.0195 | 0.1 | 0.0909 | OK |
| 3rd Floor | 13.492 | 13.492 | 9318.46 | 1039.63 | 1022.30 | 0.0063 | 0.0245 | 0.1 | 0.0909 | OK |
| 2nd Floor | 7.656 | 7.656 | 11377.13 | 1084.39 | 1084.39 | 0.0042 | 0.0221 | 0.1 | 0.0909 | OK |

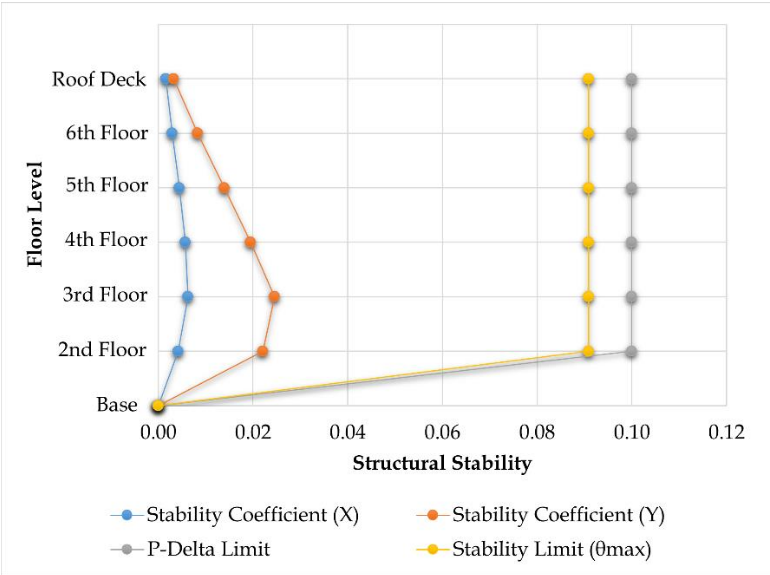



Figure 6. P-Delta Graph for Structure Without Panel Zone

Table 10. Effect of P-Delta on Structure With Panel Zone

| **Floor Level** | **Inelastic Drift** | | **Story Forces** | | | **Stability Coefficient** | | **P-Delta Limit (mm)** | $\theta_{max}$ | **Check** |
|---|---|---|---|---|---|---|---|---|---|---|
| | $\Delta_X$ (mm) | $\Delta_X$ (mm) | $P$ (kN) | $V_X$ (kN) | $V_Y$ (kN) | $\theta_X$ | $\theta_Y$ | | | |
| Roof Deck | 8.025 | 11.226 | 1083.79 | 217.88 | 194.19 | 0.0021 | 0.0033 | 0.1 | 0.0909 | OK |
| 6th Floor | 11.545 | 24.195 | 3142.46 | 510.43 | 472.78 | 0.0037 | 0.0084 | 0.1 | 0.0909 | OK |
| 5th Floor | 14.988 | 36.416 | 5201.13 | 746.17 | 707.31 | 0.0054 | 0.0139 | 0.1 | 0.0909 | OK |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4th Floor | 16.913 | 46.118 | 7259.80 | 921.86 | 892.90 | 0.0069 | 0.0195 | 0.1 | 0.0909 | OK |
| 3rd Floor | 15.780 | 51.810 | 9318.46 | 1032.18 | 1022.30 | 0.0074 | 0.0245 | 0.1 | 0.0909 | OK |
| 2nd Floor | 8.630 | 40.475 | 11377.13 | 1075.19 | 1084.39 | 0.0047 | 0.0221 | 0.1 | 0.0909 | OK |

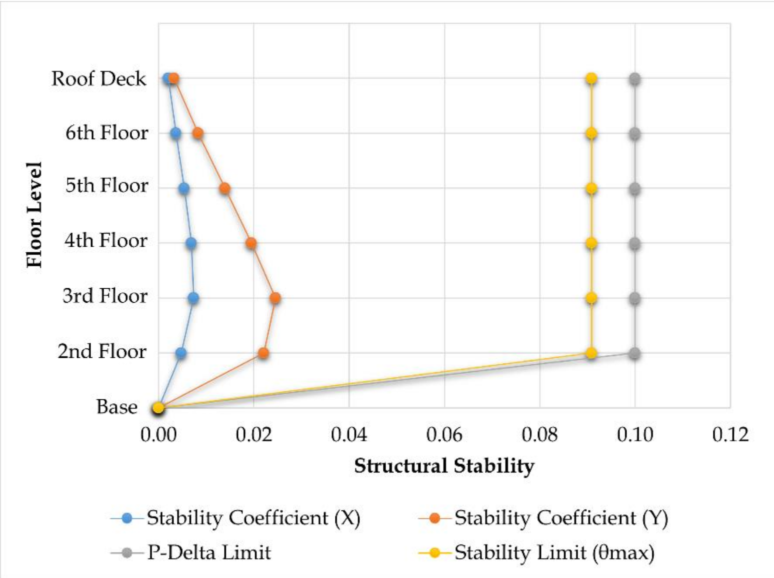


Figure 7. P-Delta Graph for Structure With Panel Zone

Based on these results, the structural model with the panel zone yields a higher stability coefficient than the model without it in the X-direction. The largest increase was in the Roof Deck, at 31.25%, while the smallest was on the second floor, at 11.90%. Meanwhile, in the Y-direction, no difference in stability coefficient values was found between the two structural models. All $\theta_Y$ values showed identical results in both the model with panel zone and the model without panel zone. The maximum stability coefficient was 0.0245 in the Y-direction on the third floor. Nevertheless, all stability coefficient values in both structural models remained below both the P-Delta influence limit and the maximum allowable stability limit; therefore, the structure was deemed to meet stability requirements based on applicable SNI regulations.

## DISCUSSION

The structural vibration modes indicate that Modes 1 and 2 are dominated by translation in the building's principal directions, with no indication of torsional dominance in the initial modes. This indicates that the structure's principal vibration modes exhibit translation characteristics in both principal directions of the structure. The cumulative mass participation in the X and Y directions has reached 100%, so the number of modes used is considered sufficient to represent the structure's dynamic response comprehensively.

Additionally, the fundamental period of 0.926 seconds indicates that the structure falls into the mid-rise building category, with its dynamic response still influenced by the lateral stiffness of the SMRF steel frame system.

The displacement analysis results show that using panel zones increases the structure’s lateral deformation in the X-direction compared to the model without panel zone. The largest increase in displacement occurred at the Roof Deck, amounting to 20.31%, indicating that the additional deformation in the beam–column joint region makes the structure more flexible. Meanwhile, no significant displacement changes were observed in the Y-direction, indicating that the global stiffness of the structure in that direction remains dominant over the contribution of panel zone deformation. The distribution of displacement values shows that the structure's lateral deformation increases with building height. This pattern reflects the typical global deformation behavior observed in multi-story structures under lateral seismic loading.

The inter-story drift results show that the model with the panel zone produces larger drift values compared to the model without the panel zone, particularly in the X-direction. The greatest increase in drift occurred in the Roof Deck, at 29.23%, indicating that the effect of joint flexibility becomes more pronounced at higher building elevations due to the accumulation of lateral structural deformation. Nevertheless, all inter-story drift values remain within the allowable limits per SNI regulations, so the structure is still deemed to meet seismic performance requirements.

The use of panel zone also reduces story stiffness in the X-direction compared to the model without panel zone. The greatest decrease in stiffness occurred on the upper floors of the building, particularly the 6th floor and the roof deck, indicating that joint deformation contributes to the reduction in the structure’s lateral stiffness. Although there was a fairly significant decrease in stiffness, the change in story shear values between the two models was relatively small, suggesting that the influence of the panel zone is more dominant on structural deformation and flexibility compared to the distribution of lateral shear forces.

The results of the P-Delta evaluation show that the structure with panel zone yields a higher stability coefficient than the model without panel zone in the X-direction. This occurs due to increased lateral deformation of the structure caused by reduced stiffness at the beam–column joints. However, all stability coefficient values remain below the P-Delta influence limit and the maximum allowable structural stability limit; therefore, both structural models are still deemed stable against secondary effects caused by seismic loading.

## CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the analysis, it can be concluded that:

1. The use of panel zone affects the seismic response of the SMRF steel structure, particularly in the X-direction, by increasing displacement, inter-story drift, and the structural stability coefficient.
2. Structures with panel zone exhibit a decrease in story stiffness compared to models without panel zone, indicating reduced lateral stiffness due to deformation at the beam–column joints.
3. The change in story shear values between models with and without panel zone is relatively small, so the effect of panel zone is more dominant on structural deformation and flexibility compared to the distribution of lateral forces.
4. In the Y-direction, the use of panel zone does not result in significant changes in structural response because the structure's global stiffness remains more dominant than the contribution of panel zone deformation.
5. All structural response parameters still meet the performance limits based on SNI regulations; therefore, both structural models are deemed safe against the effects of design earthquake loads.
6. Panel zone modeling can be considered in the analysis of earthquake-resistant steel structures to obtain a more realistic structural response regarding lateral deformation and structural stiffness.

## ADVANCED RESEARCH

This study provides an overview of the influence of panel zone on the seismic response of SMRF steel structures through a dynamic analysis approach using the Response Spectrum Analysis (RSA) method. The analysis results indicate that the presence of the panel zone affects structural behavior, particularly lateral deformation, structural stiffness, and structural stability under seismic loading.

Nevertheless, the behavior of earthquake-resistant steel structures still has many aspects that can be further developed, particularly regarding the structure's nonlinear response and the behavior of beam–column connections under more complex seismic loading. Additionally, the influence of panel zone on various building configurations and structural systems remains an area worthy of more in-depth investigation.

Further research is recommended to utilize nonlinear analysis methods, such as Nonlinear Time History Analysis (NLTHA) to observe the structure's inelastic response more realistically. Further research can be conducted by varying the number of stories, structural configurations, joint types, and panel zone modeling methods to gain a more comprehensive understanding of the behavior of SMRF steel structures.

In addition to numerical methods, further research can be conducted through experimental approaches or by validating the results of the analytical modeling. Thus, the research findings are expected to make a broader contribution to the future development of the analysis and design of earthquake-resistant steel structures.

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