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Analysis of Earthquake-Resistant Portal Frame Structures with Ordinary Moment Frames (OMF), Intermediate Moment Frames (IMF), and Special Moment Frames (SMF) based on SNI 1726:2019

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ABSTRACT

Since the 19th century, portal frames have been used to build industrial buildings because the development process is fast, economical, and efficient. This research aims to look at the cross-section of structural elements in BS 5950-1:2000 spans of 15 m and 40 m using SNI 1726:2019 with modal analysis, including horizontal deflection and stress ratio, using LRFD and ASD methods. Then, the structure was revised to be safe against earthquakes researched in three zones, including low (OMF), medium (IMF), and high (SMF) earthquakes. The results of this research show that the horizontal deflection's magnitude still falls within the allowable limit, with the maximum value on Tarutung, the SMF system, and KDS E on a 15-m span of 43.828 mm and a 40-m span of 68.703 mm. However, several of the IMF and SMF systems' cross-sectional structures exceeded the stress ratio capacity. After revision, the percentage ratios of the maximum structural weight using cross-sections on the Indonesian market using two methods and three frame systems with spans of 15 m and 40 m are 16.050% and 17.240%, respectively. The obtained maximum structural weight exceeds the cross-sections of the British standard before revision by 13.935% and 13.187%. It is an SMF system.

Keywords: modal analysis, LRFD, ASD, stress ratio, deflection

1 Introduction

In the current era of globalization, Indonesia has experienced rapid development in terms of construction. One of them is the industrial building form of portal frame structures, both long and short spans. Examples are warehouses and factories. In this case, the structure of the building needs free space because it has access to industrial activities. The truss was introduced first in portal frame construction in Europe. Where in the 19th century, the first metal roof, built in 1786 by Victor Louis (1731-1800) using a pitched roof [1].

Portal frames are generally used for low-rise industrial buildings comprising columns and horizontal or pitched rafters connected by moment-resisting connections. Additionally, the portal frame building's structure also depends on the bending resistance of the interconnections, which is stiffened by a suitable haunch or deepening of the rafter sections *[2]*.

From this description, the authors are interested in researching the portal frames depicted in the British standard, namely, BS 5950-1:2000 (Salter et al., 2004). In cases where the standard only considers wind loads as lateral loads and ignores the possibility of seismic loads. In addition, the UK is a country with high wind speeds, with actual wind speeds that could reach 58.3 m/s (130 mph) based on regional D [3]. In contrast to Indonesia, the wind speed is low. However, seismic loads are one of the lateral loads that work, so the author is interested in researching the cross-section in the British standards in Table A.1 using the Indonesian standard, namely SNI 1726:2019 about earthquakes, to generate work on cross-sectional standards used in planning the structure of portal frames in Indonesia.

The location reviewed in the research is the island of Sumatra. According to the conditions of the moment frame system, the first location of the ordinary moment frame (OMF) conditions is in Bangka Belitung Province, which astronomically is between 106026’26.11″ EL and 2044′27.78″ SL. The second location of the Intermediate Moment Frame (IMF) condition is in Palembang, which is between 104047′0″ EL and -2059′0″ SL. The third Special Moment Frame (SMF) condition in Tarutung is located between 98057′43.2″ EL and 201′18.48″ NL.

In this research, the steel frames analyzed are 15 m spans with a height of 10 m and 40 m spans with a height of 12 m using the load and resistance factor design (LRFD) and the allowable stress design (ASD) methods to obtain a comparison of the behavioral capabilities structure which methods are more efficient and economical to use in planning refers to the standard SNI 1726:2019, SNI 1729:2020, SNI 03-7860-2020, and SNI 7972:2020. Analysis of the portal frame can be done manually, but it is recommended to use the software if the planned structure is in the form of a vast range, which makes it easier for the authors to analyze.

Based on several previous studies related to the planning of the portal frame structure, including analysis of dynamic characteristics of the portal frame with variable sections that analyze the vibration modes of the portal and the effects of lateral loads that occur in seismic loads and wind loads using the Ansys software based on the finite element method [4]. Next in line for research is the ratio of portal steel trusses with portal steel frames for long-span industrial buildings. This study aims to compare the behavior of steel trusses and steel frame portals with the help of SAP2000 software [5]. The study is about planning a special-moment frame system on beam-column components and steel structure connections of the BPJN XI building, which includes evaluating the structure's cross-section to SMF requirements in terms of strength and connectors used with the bolted flange plate moment connection type [6].

The next research topic is the effect of serviceability limits on the optimal design of steel portal frames, which aims to investigate the effect of the deflection service limit using the RC-NGA guideline and the deflection limit using the Steel Construction Institute (SCI) regulations that cover the economics of structures [7]. Subsequent research is on the analysis of gable construction with rafters using honeycomb and truss steel profiles, which aims to create effective, efficient, and economical buildings that can be used as reference material for the industrial sector [8]. Another research is to discuss a comparative study on a two-story car showroom using the pre-engineered building (PEB) concept based on British Standards and the Euro Code. The analysis compares structural behavior in terms of earthquake resistance, structure weight, and building stress ratio [9].

2 Methodology

In this research, a qualitative research methodology was applied. The research begins with finding the problems, then conducting a literature study regarding the portal frame structure using Salter's theory, namely BS 5950-1:2000. Regarding the structural planning stage, the specifications for basic modeling of 3D-shaped buildings, size data, and materials used. The next stage is the creation of a portal frame structure model in ETABS software, where the spans are 15 m and 40 m, followed by structural analysis using the Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) methods to define the loading by following SNI 1727:2020 and SNI 1726:2019. If the structure check is not secure, it should be evaluated until it is safe and meets the permit requirements.

2.1 Research Location

The location reviewed in the research is the island of Sumatra. According to the conditions of the moment frame system, the first location of the ordinary moment frame (OMF) conditions is in Bangka Belitung Province, which astronomically is between $106^{0}26'26.11''$ EL and $2^{0}44'27.78''$ SL. The second location of the Intermediate Moment Frame (IMF) condition is in Palembang, which is between $104^{0}47'0''$ EL and $-2^{0}59'0''$ SL. The third Special Moment Frame (SMF) condition in Tarutung is located between $98^{0}57'43.2''$ EL and $2^{0}1'18.48''$ NL (Figure 1).

Figure 1 Site google earth: Planning location maps of location 1, 2, and 3

2.2 Planning Specifications for Portal Frame Structure

Portal Frame was analyzed in 3D form and the planned building was an industrial building in the shape of a warehouse consisting of 2 building models, namely for model 1 is a short span with a span of 15 m and model 2 is a long span with a span of 40 m (Figure 2 – Figure 3).

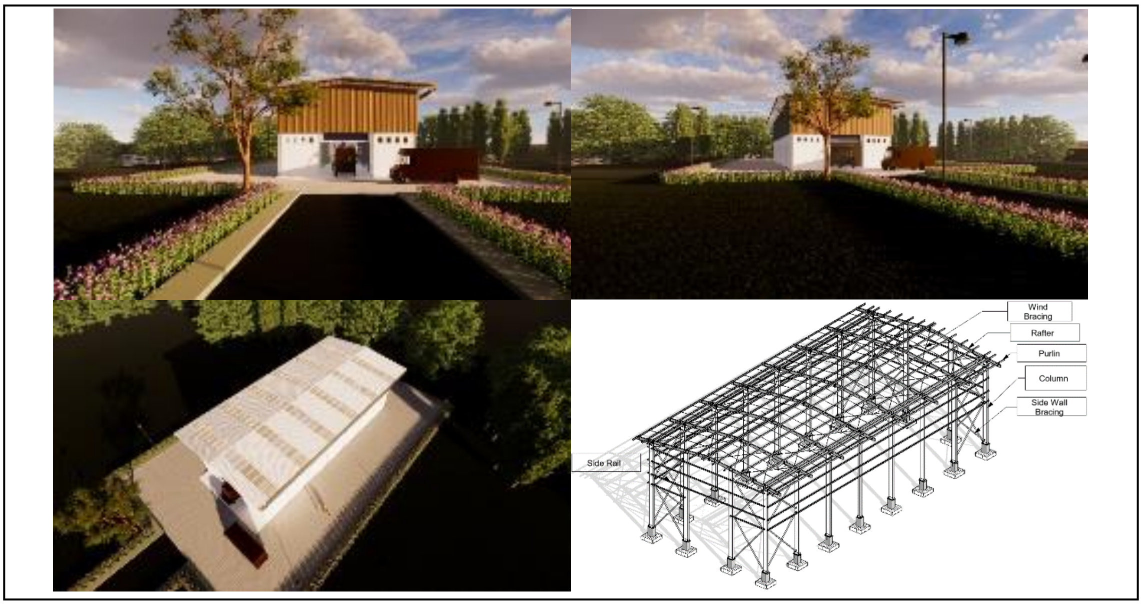


Figure 2 Revit: The 3D shape of a 15 m span industrial building plan (model 1)

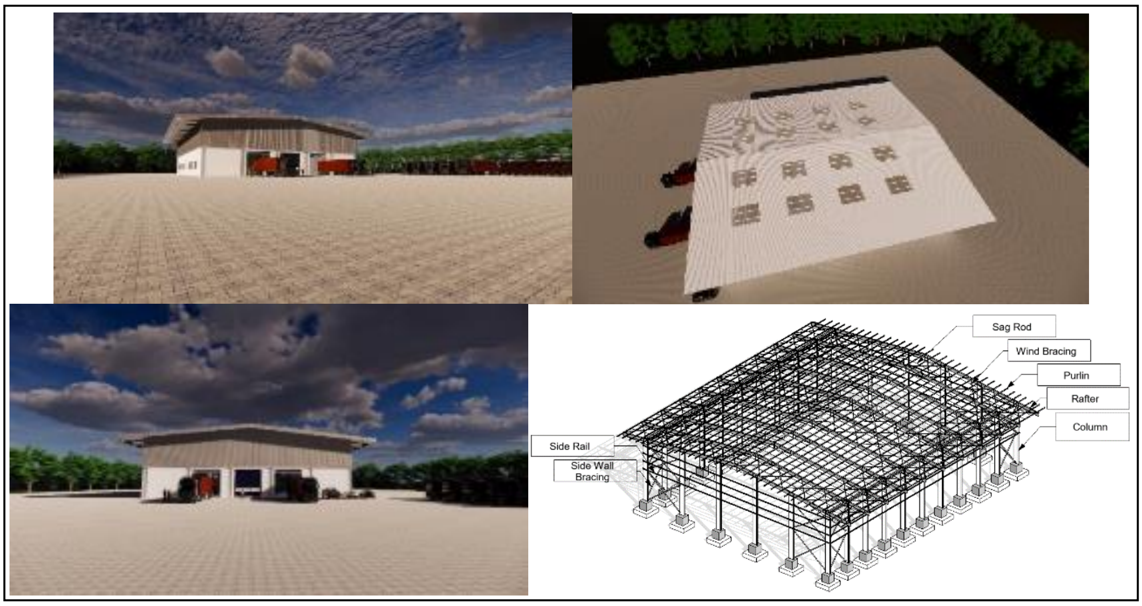


Figure 3 Revit: The 3D shape of a 40 m span industrial building plan (model 2)

The column and rafter material specifications for the portal frame structure are BJ-41, comparable to S275. Z purlin using BJ-55, comparable to S450. Sag rods and wind bracing using BjTP 280 [10].

Check the geometry of the portal frames:

Model 1

$L \quad \leq \quad 5\,h$

$15 \text{ m} \quad \leq \quad 50 \text{ m}$ …ok

$$h_r = \frac{tan\,6^o}{7.5} = 0.7883$$

$h_r \quad \leq \quad 0.25\,L$

$0.788 \text{ m} \quad \leq \quad 3.75 \text{ m}$ …ok

Model 2

$L \quad \leq \quad 5\,h$

$40 \text{ m} \quad \leq \quad 60 \text{ m}$ …ok

$$h_r = \frac{tan\,6^o}{20} = 2.104$$

$h_r \quad \leq \quad 0.25\,L$

$2.104 \text{ m} \quad \leq \quad 10 \text{ m}$ …ok

2.3 Planning for Loads on Portal Frames

The loads working on the portal frame structure include live loads, dead loads, notional loads, wind loads, and earthquake loads. Based on SNI 1727:2020 Section 4.3.1, the living load working on the portal frame structure is a roof live load (L_r) of 0.96 kN/m^2 [11]. Next is the dead load, which is the self-weight of the structure plus any dead extra loads. The self-weight of the structure is calculated automatically by the ETABS software. The dead extra loads include mechanical and electrical, sheeting, and water pipes, for a total of 0.554 kN/m^2.

Next is the wind load (Table 1). The basic wind speed (V_{700}) is 40.9 m/s [12]. Based on SNI 1727:2020, the structure type is SPGAU/MMWFRS. The gust-effect factor (G) and wind directionality factor (K_d) equal 0.85. Surface roughness/exposure categories are C. The topographic factor (K_{zt}) and ground elevation factor (K_e) equal 1. The internal pressure coefficient (GC_{pi}) in the form of windward and leeward directions is -0.18 [11]. A recapitulation of wind load calculations is seen in table 1.

Table 1 A recapitulation of wind load

Location	**q_h (kN/m^2)**		**C_p**		**p (kN/m^2)**		**Wind Load (kN/m)**			
Model	**1**	**2**	**1**	**2**	**1**	**2**	**1**		**2**	
							W_x	W_y	W_x	W_y
Center Wall:										
Windward Wall	1.05	1.08	0.8	0.8	0.91	0.93	4.53	4.53	4.66	7.45
Leeward Wall	0.88	0.92	-0.5	-0.3	-0.22	-0.07	-3.85	-3.85	-3.85	-6.16
Side Wall	0.88	0.92	-0.7	-0.7	-0.36	-0.38	-3.85	-3.85	-6.16	-3.85
Edge Wall:										
Windward Wall	1.05	1.08	0.8	0.8	0.45	0.47	2.26	2.26	2.33	3.72
Leeward Wall	0.88	0.92	-0.5	-0.3	-0.11	-0.03	-1.93	-1.93	-1.93	-3.08
Side Wall	0.88	0.92	-0.7	-0.7	-0.18	-0.19	-1.93	-1.93	-3.08	-1.93
Center Roof:										
Windward Roof	0.88	0.92	-0.48	-0.3	-0.20	-0.07	-0.98	-	-0.35	-
Leeward Roof	0.88	0.92	-0.18	-0.18	0.03	0.03	0.16	-	0.16	-
Edge Roof:										
Windward Roof	0.88	0.92	-0.48	-0.3	-0.10	-0.03	-0.49	-	-0.17	-
Leeward Roof	0.88	0.92	-0.18	-0.18	0.02	0.02	0.08	-	0.08	-

Lastly, there is the earthquake load. Based on SNI 1726:2019 Section 7.2.7.5.1, provided that the steel building is single-story, in seismic design categories D, E, and F, with a height not exceeding 20 m and a roof load not exceeding 0.96 kN/m2 allowed using the intermediate moment frame system (IMF) [13]. The spectral response acceleration for soft clay soil conditions was obtained from Puskim 2021 (Table 2).

Table 2 Earthquake data in Bangka Belitun

	Bangka Belitung	**Palembang**	**Tarutung**
PGA (g)	0.013	0.148	0.838
S_S (g)	0.033	0.291	2.083
S_1 (g)	0.045	0.249	0.929
T_L	12	20	12
S_{DS} (g)	0.050	0.440	1.110
S_{D1} (g)	0.130	0.510	1.240
T_0 (detik)	0.520	0.230	0.220
T_S (detik)	2.600	1.160	1.120
F_a	2.400	1.815	0.800
F_v	4.200	3.043	2.000
S_{MS} (g)	0.079	0.528	1.666
S_{M1} (g)	0.190	0.756	1.857
PGA_M (g)	0.032	0.317	0.921
Seismic Importance Factor (I_e)	1	1	1
Risk Category	II	II	II
Seismic Design Category (KDS)	B	D	E

2.4 Steel Profiles Usage Based on BS 5950-1:2000

The size of elements is planned based on the structure's height and the vertical load carried by the rafter in the form of a combination of ultimate load and serviceability limit states [2].

Model 1 (span 15 m):
$$
\begin{aligned}
W_t &= 1.2\ W_D + 1.6\ W_{Lr} \\
&= (1.2 \text{ x } 6.972) + (1.6 \text{ x } 4.8) \\
&= 16.046 \text{ kN/m} \\
W_t &\approx 16.000 \text{ kN/m}
\end{aligned}
$$

Model 2 (span 40 m):
$$
\begin{aligned}
W_t &= 1.2\ W_D + 1.6\ W_{Lr} \\
&= (1.2 \text{ x } 6.534) + (1.6 \text{ x } 4.8) \\
&= 15.520 \text{ kN/m} \\
W_t &\approx 16.000 \text{ kN/m}
\end{aligned}
$$

Table A.1 in BS 5950-1:2000 was used to determine the element's size for the study (Table 3).

Table 3 Cross-section of the elements based on BS 5950-1:2000

Element	**Model 1**	**Model 2**
Column	UB 610 X 229 X 101	UB 914 X 305 X 289
Rafter	UB 356 X 127 X 33	UB 686 X 254 X 125

*Source: Salter et.al. (2004) [2].

3 Result and Discussion

3.1 Mode Shapes of the structure

According to the requirements described in SNI 1726:2019 Section 7.9.1.1, the mass participation ratio must reach 100% of the structure's mass with periods of less than 0.05 s [13]. The value of the mass participation ratio for modeling at 3 locations using elements on the BS 5950-1:2000 standard was obtained from the ETABS software. The first and second modes are dominant translations. The first mode is dominant X, and the second mode is dominant Y. It also explains that the third mode is dominant rotation (Table 4 & Table 5).

Table 4 Modal mass participant ratio (model 1)

Mode	Period (s)	UX	UY	RZ	SumUX	SumUY	SumRZ
1	0.340	**0.820**	0	3E-06	0.820	0.000	0.000
2	0.217	0	**0.933**	0	0.820	0.933	0.000
3	0.195	0	0	0.261	0.820	0.933	0.261
4	0.129	**6E-06**	**5E-07**	0.5652	0.820	0.933	0.827
5	0.122	**0.141**	0	0.000	0.961	0.933	0.827
6	0.120	**1E-05**	**3E-04**	8E-06	0.961	0.933	0.827
7	0.119	**0.006**	**5E-05**	2E-05	0.967	0.933	0.827
8	0.118	0	**0.001**	3E-05	0.967	0.934	0.827
9	0.118	**1E-04**	**2E-04**	0.0028	0.967	0.935	0.829
10	0.118	**0.002**	**1E-04**	0.0008	0.969	0.935	0.830
11	0.110	0	0	0.0005	0.969	0.935	0.831
12	0.105	0	**0.001**	4E-05	0.969	0.936	0.831
13	0.100	0	**0.059**	0	0.969	0.995	0.831
14	0.090	0	0	0.1221	0.969	0.995	0.953
15	0.076	0	**1E-05**	0.000	0.969	0.995	0.953
16	0.071	**0.012**	0	5E-07	0.981	0.995	0.953
17	0.065	0	**7E-04**	2E-06	0.981	0.996	0.953
18	0.065	**1E-05**	0	0.0033	0.981	0.996	0.956
19	0.056	**2E-05**	**3E-04**	6E-06	0.981	0.996	0.956
20	**0.053**	**4E-04**	**1E-05**	**0.0005**	**0.982**	**0.996**	**0.957**
21	0.047	0.014	2E-06	3E-05	0.995	0.996	0.957

Table 5 Modal mass participant ratio (model 2)

Mode	Period (s)	UX	UY	RZ	SumUX	SumUY	SumRZ
1	0.448	**0.842**	0	0E+00	0.842	0.000	0.000
2	0.441	0	**0.925**	0E+00	0.842	0.925	0.000
3	0.254	0	0	8E-03	0.842	0.925	0.008
4	0.254	0	**0.002**	0E+00	0.842	0.927	0.008
5	0.254	0	0	0E+00	0.842	0.927	0.008
6	0.254	0	0	0E+00	0.842	0.927	0.008
7	0.237	0	0	6E-01	0.842	0.927	0.598
8	0.193	0	0	4E-01	0.842	0.927	0.952
9	0.176	0	**0.07**	0E+00	0.842	0.997	0.952
10	0.156	**0.039**	0	0E+00	0.882	0.997	0.952
11	0.155	0	**3E-05**	0E+00	0.882	0.997	0.952
12	0.153	**0.106**	0	0E+00	0.987	0.997	0.952
13	0.146	0	0	2E-05	0.987	0.997	0.952
14	0.130	**8E-06**	0	0E+00	0.987	0.997	0.952
15	0.130	0	**3E-04**	0E+00	0.987	0.997	0.952
16	0.130	0	0	2E-04	0.987	0.997	0.952
17	0.092	0	0	0E+00	0.987	0.997	0.952
18	0.083	**0.008**	0	0E+00	0.995	0.997	0.952

Mode	Period (s)	UX	UY	RZ	SumUX	SumUY	SumRZ
19	0.077	0	**6E-04**	0E+00	0.995	0.998	0.952
20	0.071	0	**2E-05**	0E+00	0.995	0.998	0.952
21	0.069	0	**1E-06**	8E-07	0.995	0.998	0.952
22	0.065	**2E-04**	0	1E-05	0.995	0.998	0.952
23	0.060	**8E-04**	0	3E-05	0.996	0.998	0.952
24	0.057	0	**2E-05**	0E+00	0.996	0.998	0.952
25	**0.054**	**1E-04**	**0**	**3E-04**	**0.996**	**0.998**	**0.952**
26	0.041	0	4E-05	0E+00	0.996	0.998	0.952

3.2 The behavior of portal frame structure cross-section on BS 5950-1:2000 standard using SNI 1726:2019.

Based on Table 20 of SNI 1726:2019, the planned portal frame falls into risk category II, and the conditions that must be met for checking the permit's horizontal deflection (Δ_a) cannot exceed 0.02 h_s for all other structures [13].

Horizontal deflection requirements,

Model 1 : $\Delta_{a\ limits}$ = 0.02 h_s = 200 mm > Δ_x

Model 2 : $\Delta_{a\ limits}$ = 0.02 h_s = 240 mm > Δ_x

By SNI 1726:2019 Section 7.12.1.1 for seismic design categories D and E,

Model 1 : $\Delta_{a\ limits}$ = 0.02 h_s/ρ = 153.846 mm > Δ_x

Model 2 : $\Delta_{a\ limits}$ = 0.02 h_s/ρ = 184.615 mm > Δ_x

The horizontal deflection requirement is still met using British standard cross-sections. The size of the cross-sections of the OMF, IMF, and SMF systems that used the LRFD and ASD methods increased as the seismic loading increased. Using these cross-sections also shows that the main structure of the portal frame at the Tarutung site, the column and rafter stress ratio capacities, do not meet, so it needs to be looked at more (Table 6).

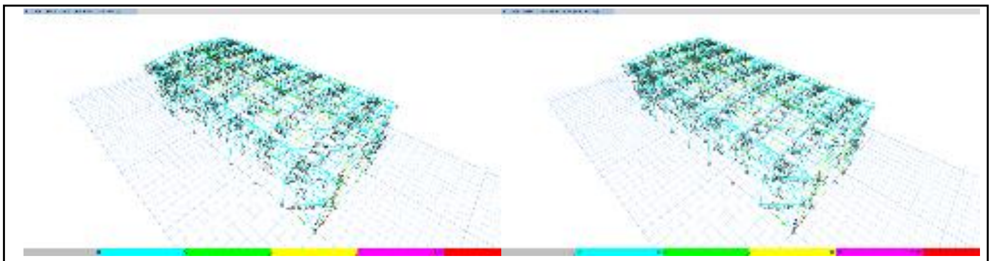
Table 6 Checking portal frame type pitched roof with a slope of 6^o based on BS 5950-1:2000 against SNI 1726:2019

Model 1						
System	**OMF**		**IMF**		**SMF**	
Location	**Bangka Belitung**		**Palembang**		**Tarutung**	
KDS	**B**		**KDS D**		**KDS E**	
Method	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**
Δ_x	1.974	1.974	17.373	17.373	43.828	43.828
Δ_y	0.902	0.902	7.957	7.957	22.634	22.634
Checking Deflection Horizontal	Ok	Ok	Ok	Ok	Ok	Ok
Checking Column Element	Ok	Ok	Ok	Ok	Not Ok	Not Ok
Checking Rafter Element	Ok	Not Ok	Ok	Not Ok	Not Ok	Not Ok
Model 2						
System	**OMF**		**IMF**		**SMF**	
Location	**Bangka Belitung**		**Palembang**		**Tarutung**	
KDS	**B**		**KDS D**		**KDS E**	
Method	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**

Δ_x	3.000	3.000	27.219	27.219	68.703	68.703
Δ_y	2.245	2.245	20.671	20.671	48.548	48.548
Checking Deflection Horizontal	Ok	Ok	Ok	Ok	Ok	Ok
Checking Column Element	Ok	Ok	Ok	Ok	Not Ok	Not Ok
Checking Rafter Element	Ok	Ok	Ok	Ok	Not Ok	Not Ok

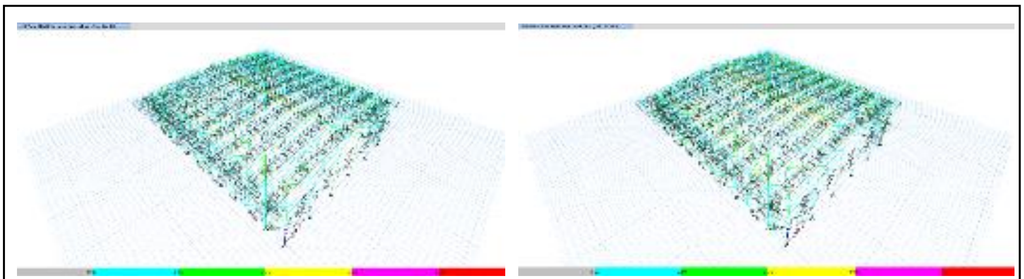
3.3 Improvement of Structural Design Against Earthquakes in Indonesia based on SNI 1726:2019

Evaluation of the design of earthquake-resistant structures in Indonesia based on SNI 1726:2019 using Indonesian cross-sections is within the allowable limits, that is, no more than 1 (Figure 4 – Figure 9).



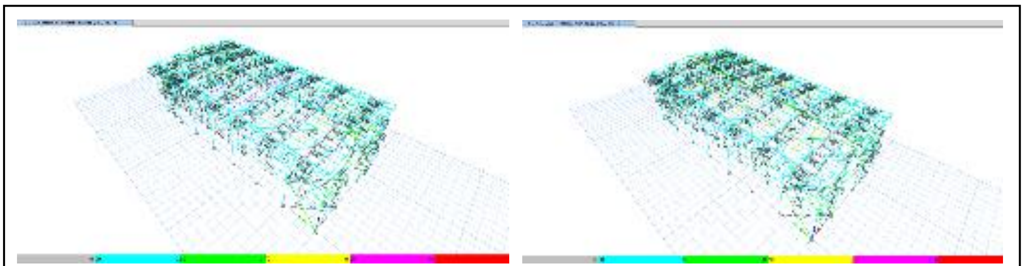
(a) (b)

Figure 4 Evaluation of stress ratio in model 1 at the Bangka Belitung location, OMF system, using methods (a) LRFD, (b) ASD



(a) (b)

Figure 5 Evaluation of stress ratio in model 2 at the Bangka Belitung location, OMF system, using methods (a) LRFD, (b) ASD



(a) (b)

Figure 6 Evaluation of stress ratio in model 1 at the Palembang location, IMF system, using methods (a) LRFD, (b) ASD

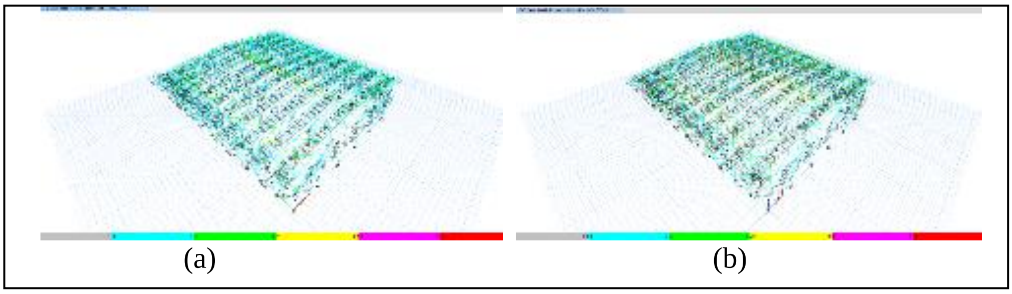


Figure 7 Evaluation of stress ratio in model 2 at the Palembang location, IMF system, using methods (a) LRFD, (b) ASD

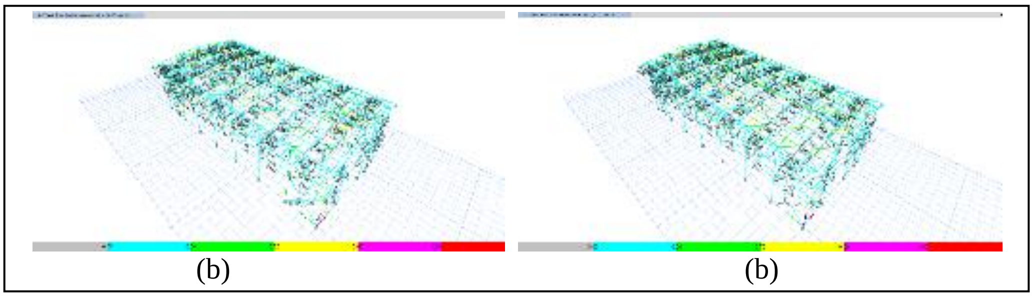


Figure 8 Evaluation of stress ratio in model 1 at the Tarutung location, SMF system, using methods (a) LRFD, (b) ASD

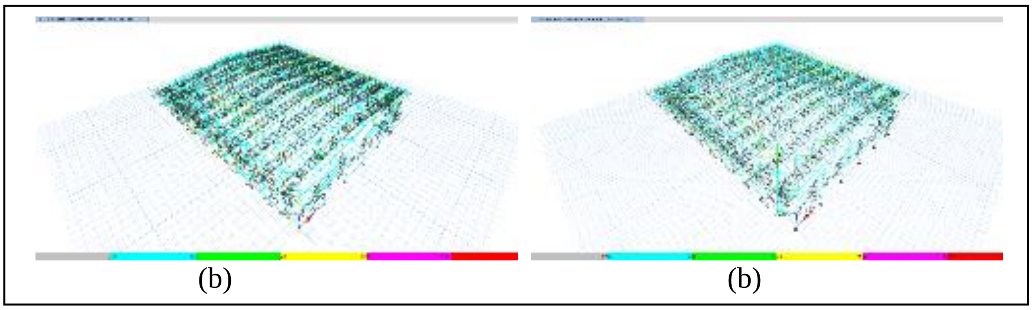


Figure 9 Evaluation of stress ratio in model 2 at the Tarutung location, SMF system, using methods (a) LRFD, (b) ASD

The recapitulation of the cross-sections and column reactions of the portal frame structure researched with a 6° roof pitch using the LRFD and ASD methods based on SNI 1726:2019 varied (Table 7 – Table 8).

Table 7 The recapitulation of the cross-sections and column reactions of the portal frame structure was researched with a 6° roof pitch using the LRFD method based on SNI 1726:2019

Model	Location	System	Element	Material	Cross-section in Indonesia	Column Reaction (kN)
1	Bangka Belitung	OMF	Column	BJ-41	WF 600 X 200 X 6 X 16	227.228
			Rafter & Beam	BJ-41	WF 300 X 150 X 6 X 9	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P25	
			Side Wall Bracing	Bj P 41	O 114.3	
			Sag Rod	BjTP 280	P12	
	Palembang	IMF	Column	BJ-41	WF 600 X 200 X 9 X 19	304.283
			Rafter & Beam	BJ-41	WF 300 X 150 X 6 X 9	

Model	Location	System	Element	Material	Cross-section in Indonesia	Column Reaction (kN)
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P25	
			Side Wall Bracing	Bj P 41	O 139.8	
			Sag Rod	BjTP 280	P12	
	Tarutung	SMF	Column	BJ-41	WF 600 X 300 X 9 X 16	875.682
			Rafter & Beam	BJ-41	WF 300 X 150 X 9 X 12	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P32	
			Side Wall Bracing	Bj P 41	O 216,3	
			Sag Rod	BjTP 280	P12	
2	Bangka Belitung	OMF	Column	BJ-41	WF 900 X 300 X 16 X 25	469.747
			Rafter & Beam	BJ-41	WF 600 X 250 X 12 X 25	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P40	
			Side Wall Bracing	Bj P 41	O 165.2	
			Sag Rod	BjTP 280	P12	
	Palembang	IMF	Column	BJ-41	WF 900 X 300 X 16 X 32	1043.130
			Rafter & Beam	BJ-41	WF 650 X 250 X 12 X 25	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P40	
			Side Wall Bracing	Bj P 41	O 216.3	
			Sag Rod	BjTP 280	P12	
	Tarutung	SMF		BJ-41	WF 950 X 450 X 16 X 36	3071.588
				BJ-41	WF 750 X 300 X 12 X 25	
				BJ-55	Z20019 (203 X 74 X 1.9)	
				BjTS 420	S50	
				Bj P 41	O 355.6	
				BjTP 280	P12	

Table 8 The recapitulation of the cross-sections and column reactions of the portal frame structure was researched with a 6° roof pitch using the ASD method based on SNI 1726:2019

Model	Location	System	Element	Material	Cross-section in Indonesia	Column Reaction (kN)
1	Bangka Belitung	OMF	Column	BJ-41	WF 600 X 200 X 6 X 16	143.170
			Rafter & Beam	BJ-41	WF 300 X 150 X 6 X 12	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P25	
			Side Wall Bracing	Bj P 41	O 114.3	
			Sag Rod	BjTP 280	P12	
	Palembang	IMF	Column	BJ-41	WF 600 X 200 X 9 X 19	231.604
			Rafter & Beam	BJ-41	WF 300 X 150 X 6 X 12	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P25	
			Side Wall Bracing	Bj P 41	O 139.8	
			Sag Rod	BjTP 280	P12	
	Tarutung	SMF	Column	BJ-41	WF 600 X 300 X 12 X 16	616.109
			Rafter & Beam	BJ-41	WF 350 X 175 X 6 X 12	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P32	
			Side Wall Bracing	Bj P 41	O 216.3	
			Sag Rod	BjTP 280	P12	
2	Bangka Belitung	OMF	Column	BJ-41	WF 900 X 300 X 16 X 25	309.541
			Rafter & Beam	BJ-41	WF 650 X 250 X 12 X 28	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P50	
			Side Wall Bracing	Bj P 41	O 165.2	

Model	Location	System	Element	Material	Cross-section in Indonesia	Column Reaction (kN)
			Sag Rod	BjTP 280	P12	
	Palembang	IMF	Column	BJ-41	WF 900 X 300 X 16 X 32	795.452
			Rafter & Beam	BJ-41	WF 650 X 250 X 12 X 28	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTP 280	P50	
			Side Wall Bracing	Bj P 41	O 216.3	
			Sag Rod	BjTP 280	P12	
	Tarutung	SMF	Column	BJ-41	WF 950 X 450 X 16 X 36	2376.439
			Rafter & Beam	BJ-41	WF 750 X 350 X 12 X 32	
			Purlin & Side Rail	BJ-55	Z20019 (203 X 74 X 1.9)	
			Wind Bracing	BjTS 420	S50	
			Side Wall Bracing	Bj P 41	O 355.6	
			Sag Rod	BjTP 280	P12	

Considering the structure's economics, a comparison of the own weight of the portal frame structure taken from BS 5950-1:2000 using the cross-section on the Indonesian market reveals that the load and resistance factor design method is also significantly more economical than the allowable stress design method. There is a percentage reduction for the OMF system, which means some structural cross-sections can be reduced. In contrast, the conditions of the IMF and SMF systems mean that some structural cross-sections must be enlarged in strength capacity compared to the cross-section in the British standard, except in Model 1. The largest self-weight of the structure is also in the SMF system (Figure 10).

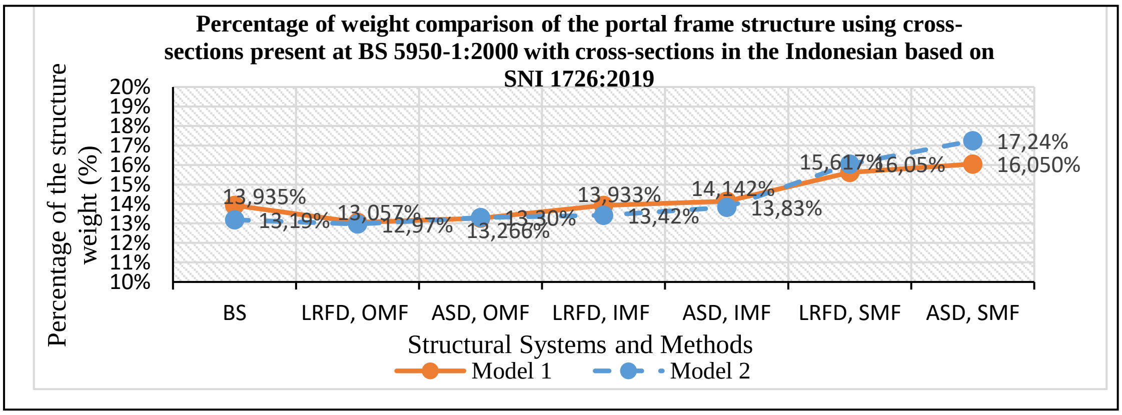


Figure 10 Percentage of weight comparison of portal frame structure using cross-section present at BS 5950-1:2000 with a cross-section in the Indonesian based on SNI 1726:2019

3.4 Horizontal Deflection Check (Δ_z)

At the Bangka Belitung location, which is an OMF system (KDS B), the value of ρ permitted to equal 1.0. While in Palembang and Tarutung locations which are IMF and SMF systems, KDS D and E, the value of ρ permitted to equal 1.3. The portal frame planned to fall into risk category II, then the conditions that must be met for horizontal deflection checking must not exceed 0.02 h_s/ρ (Table 9 and Figure 11 – Figure 12).

Table 9 Check the maximum horizontal deflection in the x and y directions due to changes cross-sectional in size according to SNI 1726:2019

Location	System	Method	Model	Elevation (h) (mm)	Deflection Horizontal Limits ($\Delta_{a\ limits} = 0.02\ h$) (mm)	Deflection Horizontal (Δ) (mm)		Check $\Delta < \Delta_{a\ limits}$	
						x	y	x	y
Bangka	OMF	LRFD	1	10000	200	1.91	0.979	ok	ok

Location	System	Method	Model	Elevation (h) (mm)	Deflection Horizontal Limits ($\Delta_{a\ limits} = 0.02\ h$) (mm)	Deflection Horizontal (Δ) (mm)		Check $\Delta < \Delta_{a\ limits}$	
						x	y	x	y
Belitung			2	12000	240	2.635	2.612	ok	ok
		ASD	1	10000	200	1.879	0.988	ok	ok
			2	12000	240	2.779	2.611	ok	ok
Palembang	IMF	LRFD	1	10000	15.846	17.136	6.520	ok	ok
			2	12000	184.615	21.631	17.97	ok	ok
		ASD	1	10000	15.846	16.676	6.876	ok	ok
			2	12000	184.615	19.050	18.68	ok	ok
Tarutung	SMF	LRFD	1	10000	15.846	31.106	10.49	ok	ok
			2	12000	184.615	41.985	22.60	ok	ok
		ASD	1	10000	15.846	29.625	9.626	ok	ok
			2	12000	184.615	41.487	23.28	ok	ok

Horizontal deflection comparisons for OMF, IMF, and SMF system conditions using both LRFD and ASD methods have successively increased due to higher earthquake loads. The most considerable horizontal deflection is at the Tarutung location, a seismic design category E and SMF system. The analysis results show that the horizontal deflection has met the allowable horizontal deflection (Figure 11 – Figure 12).

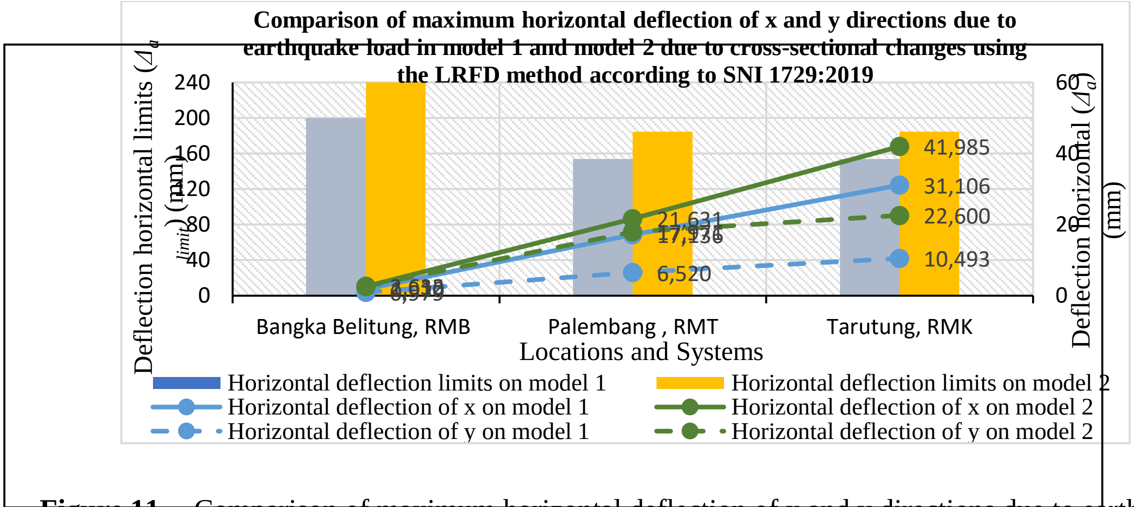


Figure 11 Comparison of maximum horizontal deflection of x and y directions due to earthquake load in model 1 and model 2 due to cross-sectional changes using the LRFD method according to SNI 1729:2019

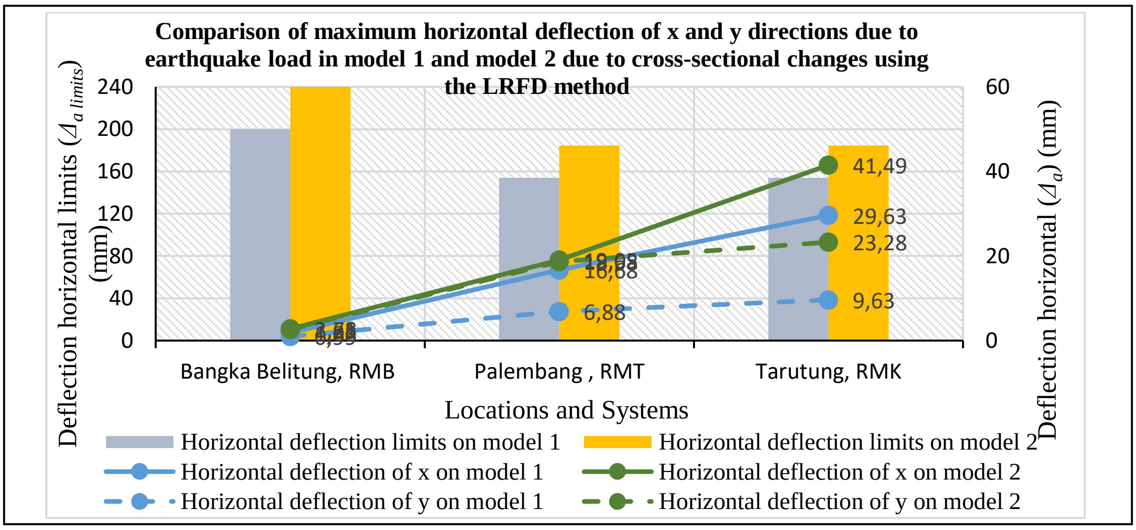


Figure 12 Comparison of maximum horizontal deflection of x and y directions due to earthquake load in model 1 and model 2 due to cross-sectional changes using the LRFD method according to SNI 1729:2019

3.5 Vertical Deflection Check (Δ_z)

Vertical deflection (Δ_z), according to AISC 360-16 Chapter L.2, states that the value of the vertical deflection must satisfy the service limit, which must be less than $L/240$. The service load is the sum of the structure's weight, any additional dead load, and the live roof load [15] (Table 10 and Figure 13).

Table 10 Deflection vertical check (Δ_z) of the rafter due to cross-sectional changes in size

Location	System	Model	Span (L) (mm)	Deflection Vertical Limits ($\Delta_{a\ limits} = L/240$) (mm)	Deflection Vertical (Δ_z) (mm)		Check $\Delta_z < \Delta_{a\ limits}$	
					LRFD	ASD	LRFD	ASD
Bangka Belitung	OMF	1	15000	62.500	8.37	7.129	ok	ok
		2	40000	166.667	30.208	25.95	ok	ok
Palembang	IMF	1	15000	62.500	8.108	6.867	ok	ok
		2	40000	166.667	25.426	24.71	ok	ok
Tarutung	SMF	1	15000	62.500	6.409	4.571	ok	ok
		2	40000	166.667	16.750	14.56	ok	ok

Vertical deflection for OMF, IMF, and SMF system conditions, using the LRFD and ASD methods, successively undermines because the more significant the seismic load, the greater the change in cross-section. The result of the rafter's cross-sectional ability to service conditions has met the permit requirements, which is smaller than the vertical deflection of the permit (Figure 13).

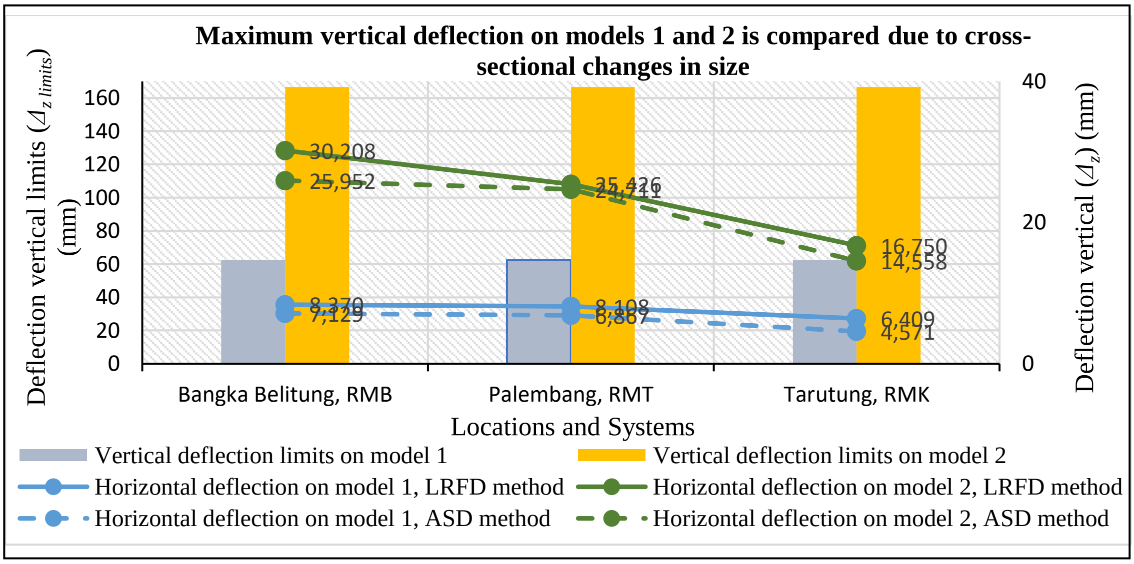


Figure 13 Comparison of the maximum vertical deflection on model 1 and model 2 due to cross-sectional changes in size

3.6 Evaluate the P-delta Effect

P-delta effects are calculated based on SNI 1726:2019, Section 7.8.7. The vertical load (P_x) is the total vertical design load at the level of the floor under consideration, which should not exceed 1.0 [13]. So it uses a combination of gravity loads with a factor of 1.0. From the results obtained, it can be seen that the value of

θ is less than 0.1, θ_{max} less than 0.25, and the column is stable. Therefore, the analysis of the P-delta effects does not need to be examined (Table 11 – Table 12).

$$\theta = \frac{P_x \, \Delta \, I_e}{V \, h \;\; C} \leq 0.1 \quad < \quad \theta_{max} - \frac{0.5}{\;} \quad \leq 0.25 \qquad (1)$$

Table 11 Check the influence of the P-delta effects in the x direction

Location	Method	Model	V_x (kN)	Δ_x	P_x (kN)	θ	β	θ_{max}	Check $\theta < \theta_{max}$
Bangka Belitung, OMF	LRFD	1	27.679	0.002	497.282	0.0011	1	0.167	Stable
		2	102.156	0.003	2097.69	0.0015	1	0.167	Stable
	ASD	1	26.972	0.002	497.282	0.0012	1	0.167	Stable
		2	102.155	0.003	2195.69	0.0017	1	0.167	Stable
Palembang, IMF	LRFD	1	272.192	0.017	497.282	0.0008	1	0.125	Stable
		2	783.133	0.022	2230.64	0.0013	1	0.125	Stable
	ASD	1	269.401	0.017	497.282	0.0008	1	0.125	Stable
		2	835.513	0.019	2347.73	0.0011	1	0.125	Stable
Tarutung, SMF	LRFD	1	760.143	0.031	497.282	0.0004	1	0.091	Stable
		2	2304.264	0.042	2996.066	0.0008	1	0.091	Stable
	ASD	1	750.748	0.030	523.491	0.0004	1	0.091	Stable
		2	2370.015	0.041	3343.241	0.0009	1	0.091	Stable

Table 12 Check the influence of the P-delta effects in the y direction

Location	Method	Model	V_y (kN)	Δ_y	P_x (kN)	θ	β	θ_{max}	Check $\theta < \theta_{max}$
Bangka Belitung, OMF	LRFD	1	53.631	0.001	497.282	0.00030	1	0.167	Stable
		2	142.667	0.003	2097.69	0.00107	1	0.167	Stable
	ASD	1	1305.07	0.001	497.282	0.00001	1	0.167	Stable
		2	140.971	0.003	2195.69	0.00113	1	0.167	Stable
Palembang, IMF	LRFD	1	442.037	0.007	497.282	0.00018	1	0.125	Stable
		2	1305.07	0.018	2230.64	0.00064	1	0.125	Stable
	ASD	1	474.860	0.007	497.282	0.00018	1	0.125	Stable
		2	1351.38	0.019	2347.73	0.00068	1	0.125	Stable
Tarutung, SMF	LRFD	1	1179.45	0.010	497.282	0.00008	1	0.091	Stable
		2	3380.323	0.023	2996.07	0.00030	1	0.091	Stable
	ASD	1	1093.82	0.010	523.491	0.00008	1	0.091	Stable
		2	3688.319	0.023	3343.24	0.00032	1	0.091	Stable

3.7 Connection

The connection is made according to SNI 7860:2020, SNI 7972:2020, and SNI 1729:2020. There are no particular guidelines to follow when planning the connection at the Bangka Belitung location because the frame system there is OMF. Palembang and Tarutung have different rules for how moment connections must be designed in the IMF and SMF systems. These rules can be found in ANSI/AISC 358 or SNI 7972:2020, the current standard [16] (Table 13).

Table 13 Recapitulation of the results of the analysis of the rafter-to-column connections of the portal frame structure

Location	Model	Dimension of Rafter (mm)	Dimensi of Column (mm)	Type	Grade	n	d_b (mm)	t_p (mm)	t_s (mm)
Bangka Belitung, OMF	1	300 x 150	600 x 200		A325	10	16	10	10
	2	600 x 250	900 x 300		A325	16	16	25	16
Palembang, IMF	1	300 x 150	600 x 200	4ES	A490	6	22	22	10
	2	650 x 250	900 x 300	8ES	A490	12	36	38	16
Tarutung, SMF	1	300 x 150	600 x 300	4ES	A490	6	27	25	10
	2	750 x 300	950 x 450	8ES	A490	12	36	40	16

Based on SNI 7860:2020 Section 4.a.(a), there is an additional rule about the moment ratio for the SMF system. This rule explains that the correlation between the beam and column connections cannot be ignored if the structure is in a high-risk earthquake zone and is an SMF system. Therefore, it must be satisfied unless P_{rc} is less than 0.3 P_c [16].

Model 1

Check:
P_{rc} < 0.3 P_c
634.118 < 0.3 x 3677.500
634.118 kN < 1103.250 kN ok

Model 2

Check:
P_{rc} < 0.3 P_c
2302.163 < 0.3 x 10745
2302.163 kN < 3223.500 kN ok

Therefore, the correlation between the beam and column connections can be ignored.

4 Conclusion

According to the research, the horizontal deflection value obtained by using a cross-section on the BS 5950-1:2000 Table A.1 standard and applying SNI 1726:2019, the modal analysis method, and RSA 2021 still satisfies the permit requirements for low, medium, and high-risk earthquakes. In models 1 and 2, which are part of the planned SMF system at Tarutung, the maximum horizontal deflection is 43.828 mm and 68.703 mm, respectively. However, the stress ratio capacity of some cross-sections determined using the ASD method and the BS 5950-1:2000 standard exceeds the permitted limits because the standard only uses the LRFD approach. However, the authors of this research considered both methods. It determines that the Palembang and Tarutung locations, which are IMF and SMF systems using the LRFD method, also exceeded the ratio capacity, requiring additional evaluation. After conducting the evaluation process, the percentage of weight comparison of cross-sectional structures at BS 5950-1:2000 that have been changed according to the market in Indonesia, taking into account the economy obtained, is model 1 of 13.935%, 13.057%, 13.266%, 13.933%, 14.142%, 15.617%, and 16.050%. While in Model 2, it is 13.187%, 12.969%, 13.305%, 13.424%, 13.826%, 15.050%, and 17.240%. Based on these results, the LRFD method is more economical than the ASD method. After the evaluation process, the horizontal deflection result met the permit requirements. The LRFD method produces the largest horizontal deflection magnitudes at the Tarutung location, specifically in models 1 and 2, which are 31.106 mm and 41.985 mm, respectively. Due to its location in a zone with a high risk of earthquakes, Tarutung is an SMF system and falls under design category E. The result of the vertical deflection has also met the permit requirements. The maximum vertical deflection value due to the service condition is at the Bangka Belitung location, the OMF system using the LRFD method, namely in models 1 and 2, is 8.370 mm and 30.208 mm, respectively. An analysis of the P-delta effects does not need to be examined. Because the value of θ is less than 0.1, θ_{max} less than 0.25, and the column is stable. Similarly, with the behavior of moment connections in the portal frame structure, all structural systems met the seismic provisions in the SNI 1860:2020 standard.

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