



Influence of Semi-Rigid Connection Stiffness on the Seismic Response of a Four-Story Steel Modular Box Building

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Abstract

Steel modular buildings rely on inter-module connections to transfer lateral forces and maintain global stability. However, treating these connections as fully rigid may overestimate structural stiffness and underestimate story drift. This study evaluates the effect of measured connection flexibility on the seismic response of a four-story modular box building. A three-dimensional global model was developed in ETABS, while the rotational stiffness of the existing bolted bracket connection was obtained from a detailed finite-element model and represented as panel-zone rotational springs. The secant rotational stiffness was 2,190.16 kN·m/rad for a continuous-column joint and 1,095.08 kN·m/rad for top and bottom joints. The stiffness classification indicated that the existing connection behaved as a simple-to-partially restrained connection rather than a fully restrained moment connection. When the connection stiffness was included in the global model, the maximum column demand-to-capacity ratio reached 1.048 and the amplified interstory drift ratio reached 0.0269, exceeding the adopted limits of 1.00 and 0.02, respectively. The results demonstrate that realistic connection stiffness should be incorporated in the global analysis of modular steel buildings and that strengthening is required before the connection can be relied upon as part of the seismic-force-resisting system.

Keywords: modular box building; semi-rigid connection; rotational stiffness; interstory drift; ETABS; finite element analysis

1. Introduction

Modular steel construction has been increasingly adopted for residential, commercial, educational, and temporary accommodation buildings because a substantial part of the structural system can be fabricated under controlled factory conditions. Compared with conventional construction, modular construction can shorten on-site activities and improve dimensional consistency. Nevertheless, the structural performance of a modular building is strongly governed by the behavior of its inter-module connections, particularly under lateral loading.

Inter-module connections are frequently idealized as either pinned or fully rigid in global structural models. In practice, bolted brackets, cover plates, contact interfaces, bolt-hole clearances, and local deformation of thin-walled members produce finite rotational stiffness. The fully rigid assumption may therefore overestimate the lateral stiffness of the building and may lead to unconservative estimates of interstory drift and member force redistribution. Previous studies on modular connections have shown that connection details affect moment transfer, initial stiffness, cyclic response, and failure mode [1]-[5].

The investigated building is a four-story steel Modular Box system used for worker accommodation. The building has an approximate plan dimension of 42 m by 15 m and a total structural height of 11.2 m. The modules are connected by three-directional steel brackets and high-strength bolts. Preliminary testing indicated

that the connection stiffness was lower than the stiffness assumed in the original rigid-joint structural model. This discrepancy motivated a global reassessment using the actual rotational stiffness of the connection.

This paper addresses a limited part of the broader strengthening study. The objective is to quantify the influence of the existing connection stiffness on the global seismic response and to determine whether the unstrengthened connection can be represented as a fully restrained moment connection. The study combines a detailed finite-element-derived moment-rotation response with a global ETABS model. The principal outputs are connection classification, maximum member demand-to-capacity ratio, and interstory drift.

2. Method

2.1 Building Description

The case-study structure is a four-story steel modular building without a basement. Its overall dimensions are approximately 42 m in length, 15 m in width, and 11.2 m in height. Each module consists of steel beams and columns connected through prefabricated bracket assemblies. The critical inter-module joint is formed by the intersection of double columns and double beams from adjacent modules. Details of the buildings as set out in Table 1.

Table 1. Building Description

Parameter	Description
Building function	Worker accommodation
Number of stories	4
Plan dimensions	42 m × 15 m
Structural height	11.2 m
Primary structural system	Steel modular box frames with bolted bracket connections

As shown in Figure 1, this is the design of the four-storey modular building that has been constructed. In terms of construction, the foundations utilise a semi-raft concrete system, which is adapted to the local soil conditions and can be seen in Figure 2.



Figure 1. The process of installing the Modular Box



Figure 2. The process of installing the foundation

The Figure 3 shows an existing structure using SS400-grade light steel as the main framework of the modular box, comprising the columns, beams and connections. Meanwhile, the bolts used for reinforcement are 16 mm (M16) High Strength Bolts conforming to ASTM-A325.

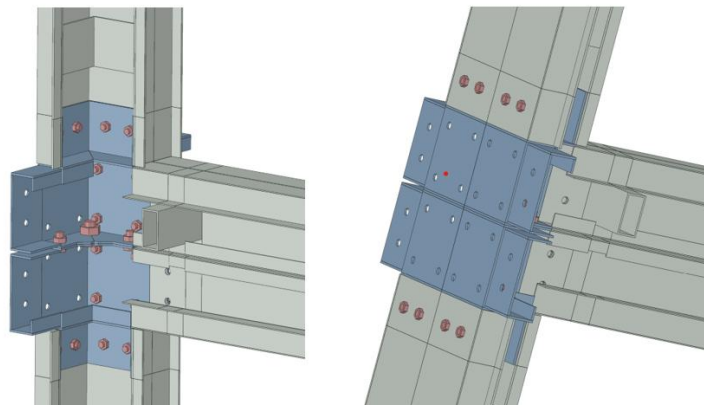


Figure 3. Existing Connection

2.2 Global Structural Model

A three-dimensional ETABS model was developed to evaluate the global behavior of the modular building. The combination of seismic loads and lateral loads is determined in accordance with the design basis used in this thesis. Two connection representations were considered conceptually: a fully restrained model and a model incorporating the finite rotational stiffness of the existing connection. In the latter model, the connection response was represented through panel-zone rotational stiffness.

The present draft intentionally reports the structural response obtained from the adopted design spectrum in this thesis. Before journal submission, the seismic parameters must be recalculated using the actual site coordinates in Sepaku, East Kalimantan, and the corresponding site class. The revised response spectrum should then replace the preliminary spectrum used in the current model as shown in the Figure 4. This building is used to house workers who will be working on projects in the Ibu Kota Nusantara.

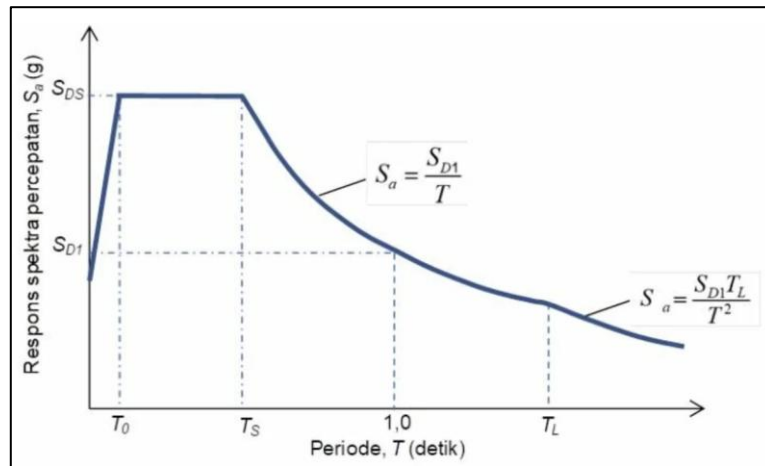


Figure 4. The spectrum response

2.3 Connection Stiffness and Classification

The joints used in this modular system employ steel brackets, with the modular units typically connected using high-strength bolts and plates. The existing bolted connection was represented by a detailed finite-element model. The force-displacement output was transformed into a moment-rotation relationship at the column face. The secant rotational stiffness was determined as

$$K_s = \frac{M_s}{\theta_s} \quad (1)$$

Where K_s is the secant rotational stiffness, M_s is the moment at the selected service-level point, and θ_s is the corresponding joint rotation. The normalized stiffness index was evaluated using $K_s L / EI$, where L , E , and I are the connected beam length, elastic modulus, and second moment of area, respectively. A value of $K_s L / EI \geq 20$ represents a fully restrained connection, while $K_s L / EI \leq 2$ represents a simple connection. Intermediate values indicate partially restrained behavior [6].

The finite-element response provided a secant stiffness of 2,190.16 kN·m/rad for a joint with a continuous column. For joints at the top and bottom of the column line, under the conservative assumption that the stiffness of the non-continuous column is 50 per cent that of a continuous column, giving 1,095.08 kN·m/rad. These figures were obtained from Ansys simulations of the existing joints and the simulation process shown in the Figure 5.

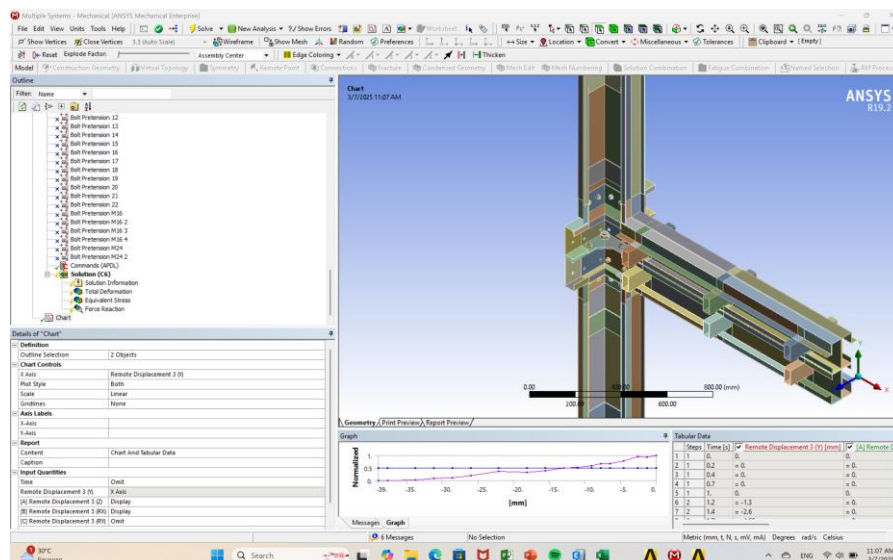


Figure 5. Results of the Ansys model run for the existing joint

2.4 Evaluation Parameters

The global model was evaluated using the maximum member demand-to-capacity ratio and the amplified interstory drift ratio. The drift was calculated from the elastic drift output and amplified using the deflection amplification factor, as expressed by

$$\Delta = \frac{\delta_e c_d}{I_e} \quad (2)$$

Where Δ is the amplified interstory drift ratio, δ_e is the elastic interstory drift ratio obtained from ETABS, C_d is the deflection amplification factor, and I_e is the seismic importance factor. The thesis adopted $C_d = 3.0$ and a drift limit of 0.02. The exact value of $I_e = 1.0$ must be stated explicitly in the revised manuscript.

3. Results and Discussion

3.1 Rotational Stiffness of the Existing Connection

The calculated secant stiffness was 2,190.16 kN·m/rad for the continuous-column joint. The normalized stiffness index fell below the threshold for a fully restrained connection. Consequently, the existing detail should not be modeled as an ideal rigid moment connection. The joint response is more appropriately represented as simple-to-partially restrained, depending on the selected service point and the connected member properties in the Table 2.

Table 2. Member Properties

Joint location	Adopted stiffness (kN·m/rad)	Model representation	Interpretation
Interior/continuous column	2,190.16	Panel-zone rotational spring	Not fully restrained
Top joint	1,095.08	50% of interior stiffness	Conservative end-joint value
Bottom joint	1,095.08	50% of interior stiffness	Conservative end-joint value

This classification has an important implication for structural analysis. A rigid-joint model suppresses connection rotation and transfers larger bending moments through the joints, while a flexible model permits additional rotation and generally increases lateral deformation. Therefore, the idealized rigid model does not accurately represent the existing modular bracket connection. The results of the calculations can be seen in Table 3.

Table 3. Result the calculations stiffness

Time	Disp (mm)	Force (N)	Rad	Moment (kN.m)	Stiffness (kN.m/rad)
-	-		-		
0.20	-	(8.05)	-	(0.01)	
0.40	-	(15.16)	-	(0.02)	
0.70	-	(30.37)	-	(0.04)	
1.00	-	(62.48)	-	(0.08)	

1.20	(1.30)	(1,540.90)	(0.00)	(2.00)	2,003.17
1.40	(2.60)	(1,103.00)	(0.00)	(1.43)	716.95
1.70	(4.55)	(5,767.40)	(0.00)	(7.50)	2,142.18
2.00	(6.50)	(8,423.70)	(0.01)	(10.95)	2,190.16
2.20	(7.80)	(8,441.00)	(0.01)	(10.97)	1,828.88
2.40	(9.10)	(10,896.00)	(0.01)	(14.16)	2,023.54
2.70	(11.05)	(12,796.00)	(0.01)	(16.63)	1,957.04
3.00	(13.00)	(12,862.00)	(0.01)	(16.72)	1,672.06
3.20	(15.60)	(15,899.00)	(0.01)	(20.67)	1,722.39
3.40	(18.20)	(17,524.00)	(0.01)	(22.78)	1,627.23
3.70	(22.10)	(16,455.00)	(0.02)	(21.39)	1,258.32
4.00	(26.00)	(21,113.00)	(0.02)	(27.45)	1,372.35
4.20	(28.60)	(23,335.00)	(0.02)	(30.34)	1,378.89
4.40	(31.20)	(24,076.00)	(0.02)	(31.30)	1,304.12
4.60	(33.80)	(25,355.00)	(0.03)	(32.96)	1,267.75
4.90	(37.70)	(26,068.00)	(0.03)	(33.89)	1,168.57
5.00	(39.00)	(26,195.00)	(0.03)	(34.05)	1,135.12

3.2 Member Demand-to-Capacity Ratio

After the finite rotational stiffness was assigned to the global model, the maximum column demand-to-capacity ratio reached 1.048. This value exceeds the allowable limit of 1.00 and indicates that at least one column does not satisfy the adopted strength criterion. The maximum beam ratio was 0.961 and remained below the limit can be seen in Table 4.

Table 4. Max stress ratio

Member type	Maximum ratio	Limit	Status
Column	1.048	1.00	Not satisfactory
Beam	0.961	1.00	Satisfactory

The column overstress is relatively small in numerical magnitude, but it is structurally significant because it appears only after connection flexibility is represented. This result illustrates that connection modeling affects not only drift but also force redistribution among the modular frame members. The results of the ETABS analysis show that, for the dimensional checks both member checks and displacement checks no

failure was detected; this is shown in Figure 8. And also included is a typical layout in Figure 6 and a side view in Figure 7.



Figure 6. Typical layout

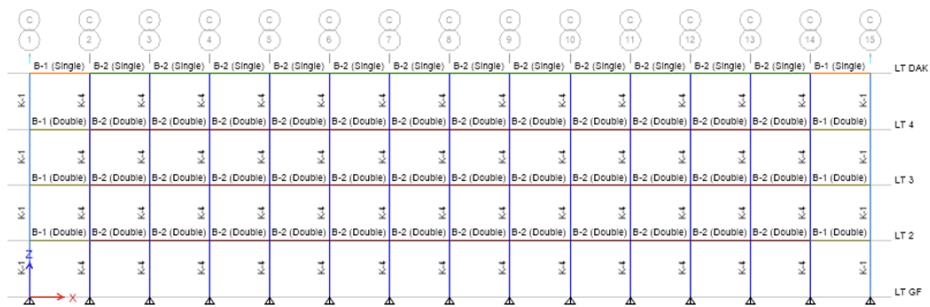


Figure 7. Side view

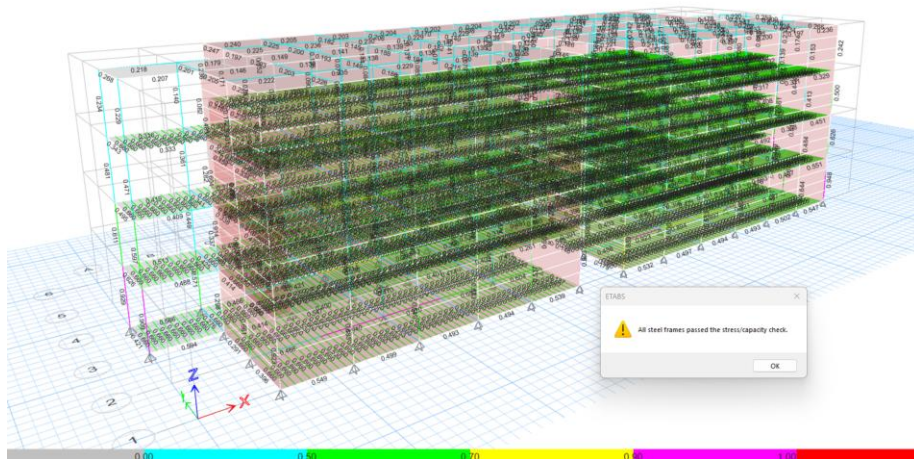


Figure 8. The results of the ETABS analysis

3.3 Interstory Drift

The largest elastic drift ratio reported by the model occurred between the ground floor and the second floor and was 0.008996. Using $C_d = 3.0$ and the importance factor adopted in the thesis, the amplified drift ratio was calculated as 0.0269. This value exceeds the adopted limit of 0.02. The results are shown in the Table 5 and Figure 9.

Table 5. Max drift

Parameter	Value
Critical story	Ground floor to second floor
Elastic drift ratio, δ_e	0.008996
Amplified drift ratio, Δ	0.0269
Adopted limit	0.0200

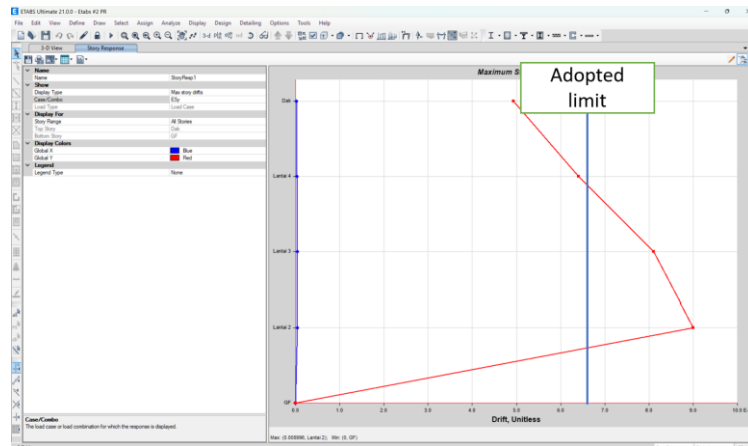


Figure 9. Interstory drift

The amplified drift is approximately 34.5% higher than the adopted limit. This confirms that the existing connection flexibility reduces the effective lateral stiffness of the modular system. The result also supports the need for connection strengthening or for the addition of a separate lateral-force-resisting system.

3.4 Engineering Implications

The findings show that the stiffness of inter-module connections should be treated as an explicit design parameter. For the investigated structure, the unstrengthened connection cannot be assumed to provide fully restrained moment continuity. Its limited stiffness contributes to excessive drift and produces a column demand-to-capacity ratio greater than unity.

From a practical perspective, the result provides a rational basis for selecting the joints that require strengthening. The subsequent stage of the broader research introduces cover plates, angle plates, transverse stiffeners, and additional bolts. However, those strengthening details are outside the scope of the present paper and should be reported separately to avoid overlap with a second publication focused on nonlinear connection behavior.

Several limitations must be recognized. First, the seismic spectrum in the current thesis model must be updated using the actual site parameters. Second, the connection stiffness should be supported by mesh convergence and quantitative validation against experimental data. Third, the global analysis should be repeated after strengthening by inserting the revised moment-rotation stiffness into the ETABS model.

In this study, ETABS was used to perform a global structural analysis and to generate internal forces in the form of moments and shear forces at the joints. These forces were then used as the basis for modelling the joints in ANSYS to obtain the moment–rotation relationship and the rotational stiffness value (K_s). The stiffness values obtained were then re-input into ETABS as rotational springs, so that the behaviour of the joints was no longer assumed to be fully rigid, but rather semi-rigid. Consequently, the structural analysis became more realistic as it took into account the actual stiffness of the joints.

4. Conclusions

1. The existing bolted bracket connection has a finite secant rotational stiffness of 2,190.16 kN·m/rad for a continuous-column joint and 1,095.08 kN·m/rad for top and bottom joints.

2. The stiffness classification indicates that the existing connection is not fully restrained; therefore, an ideal rigid-joint assumption is inappropriate for the investigated modular building.
3. When the measured connection stiffness is included in the global model, the maximum column demand-to-capacity ratio reaches 1.048, while the maximum beam ratio is 0.961.
4. The amplified maximum interstory drift ratio reaches 0.0269, exceeding the adopted limit of 0.02 by approximately 34.5%.
5. The existing connection requires strengthening or supplementary lateral resistance before it can be relied upon as part of the seismic-force-resisting system.

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