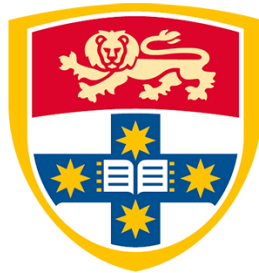


Development of Efficient Structural Systems for Mid-rise Modular Steel Buildings with Pre-loaded Inter-module Connections

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Abstract

The demand for steel-framed modular buildings has recently grown given its many advantages, such as speed of construction, associated cost savings, as well as reduction of waste. In particular, modular buildings have been increasingly considered to be an appropriate solution for addressing severe urban housing shortages in a sustainable way as an alternative to the conventional construction process. Accordingly, the demand for mid- and high-rise construction in urban areas has also rapidly increased, including in seismic areas. From a structural point of view, in order for the many individual modules to behave stably as a compatible building, the connections between modules are most critical. These inter-module connections should be sufficiently rigid, and the installation of the connections also needs to be as easy as possible. In recent years, many inter-module connections have been developed for steel-framed modular buildings to improve the constructability and performance of modular buildings. While each of the inter-module connections developed to date has its own advantages, the connections do not maximise the benefits of modular construction, such as speedy construction and reusability of modules.

This thesis aims to develop an innovative post-tensioned type inter-module connection to maximise construction speed and reusability of modules for mid-rise steel-framed modular buildings as well as to apply the connection in seismic areas. The thesis also aims to provide optimal design principles and structural modelling of the connection to enable engineers to design the connection in practice.

Based on an in-depth analysis of existing inter-module connections from the relevant literature, the post-tensioned type connection (the AJ connection or preload connection) was proposed. This connection connects modules vertically by preload using the developed coupler and high-tensile steel rod bolts within columns. The AJ connection has a simple shape and installation process for connecting modules, which provides an easy and convenient on-site assembly and disassembly process of the modular building. In addition, it allows for the modules to be manufactured as complete finished products, not requiring additional on-site finishing works after connecting modules, unlike existing bolted connections. It also does not require a hydraulic jack to apply preload, unlike the existing post-tensioned connections. Consequently, the AJ connection can maximise the benefits of 3D volumetric modular construction (i.e. fast on-site construction, reusability of modules, safer work environment and automated factory production of modules).

In order to apply the AJ connection to steel-framed modular buildings in practice, the structural performance of the connection was investigated using analytical, experimental and numerical methods. Firstly, the theoretical principles (e.g. preload mechanism, failure modes, determination of demand preload, seismic design application, etc.) of the AJ connection were established using an analytical

approach. Secondly, the applicability of the connection in practice and the established theoretical principles were evaluated and verified through experimental investigations and numerical investigations of the component (i.e. the developed coupler) and the full-scale beam-to-column joint. Thirdly, using the advanced finite element 3D joint models calibrated against the test results, the performance of the AJ connection was additionally investigated at the joint level. Fourthly, by applying the 2D simplified FE model of the AJ connection to a mid-rise modular building frame, the behaviour of the frame was investigated when subjected to wind and seismic loads. Based on these studies, lastly, guidelines for the optimal design of the AJ connection were developed.

The findings of this thesis are that the analytical design methods proposed in this thesis could provide the required amount of preload for preventing the defined failure modes (i.e. gap-opening and slip) at the AJ connection. Consequently, the AJ connection exhibited similar performance to welded connections when subjected to the design wind and seismic loads. In addition, when the AJ connection was designed as a non-dissipative preload connection in accordance with the suggested method in this thesis, the moment-resisting frames consisting of steel-framed modules could exhibit strong column-weak beam behaviour and a ductile response to earthquake loading by both floor and ceiling beams stably reaching their full plastic moment capacities. Therefore, the AJ connection can be simply modelled as fixed connections for structural analysis under the assumption that the demand preload will be induced in the connections. Conclusively, the performance of the AJ connection proposed in this thesis is verified to have sufficient performance for mid-rise steel-framed modular buildings in both seismic and non-seismic areas.

Statement of Originality

This is to certify that to the best of my knowledge, the content of this thesis is my own work. This thesis has not been submitted for any degree or other purposes.

I certify that the intellectual content of this thesis is the product of my own work and that all the assistance received in preparing this thesis and sources have been acknowledged.

I declare that the main concept of the AJ connection came from Professor Bongho Cho. Regarding all the experiments described in Chapter 4, I prepared and conducted these while staying in South Korea from July 2020 to February 2021. I took the lead in the design and manufacture of all the specimens (from the coupler to the beam-to-column joints) and the performing of the experiments, with members in the Prefab-Structure Lab of Ajou University assisting me with the experiments.

For the test of applying preload to the steel rod bolts (Section 4.3), Dami Jung and Yungjun Kim assisted. For the beam-to-column joint test (Section 4.4), Dami Jung, Dongsub Lee, Uijin Seo, and Jahyung Goo assisted.

The theoretical analysis and numerical simulation works of all other chapters were made under the supervision of Professor Kim Rasmussen using the High-Performance Computing (HPC) facility at the University of Sydney.

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Chapter 1

Introduction

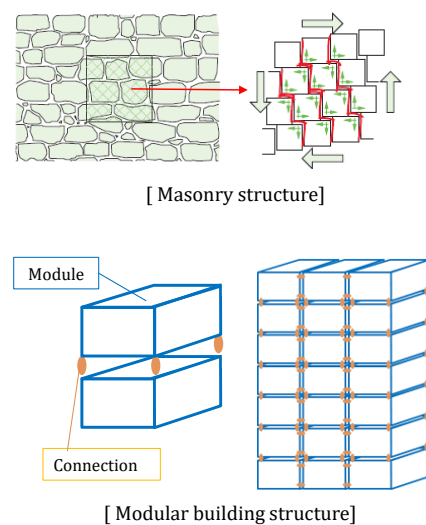
1.1. Background

Modular construction is a type of prefabricated construction system using 3D volumetric modules. The modules are manufactured in a factory as complete products and then transported on-site for assembling to form a building (see Figure 1.1 (a)). Modular construction has many advantages, such as speed of construction, associated cost savings, as well as reduction of waste. In terms of materials, the steel-framed module is most commonly used for modular buildings because of its high strength, durability, and reusability.

In the past, modular buildings were commonly used for low-rise buildings in non-seismic areas. With its many advantages, modular buildings have been increasingly considered to be an appropriate solution for addressing severe urban housing shortages in a sustainable way as an alternative to the conventional construction process. Accordingly, more recently the demand for modular buildings has grown rapidly for mid- and high-rise construction in urban areas, including in seismic areas. However, there are relatively few application cases in seismic areas and limited studies have been conducted.



(a)

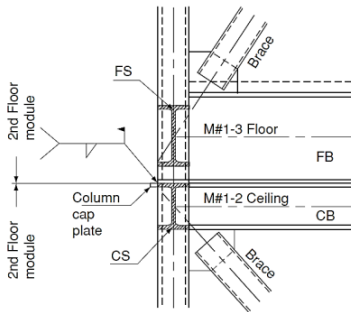
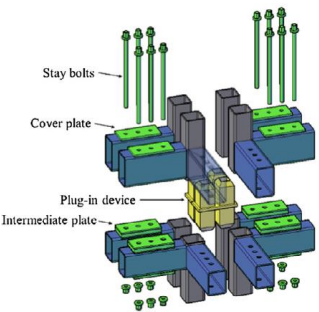
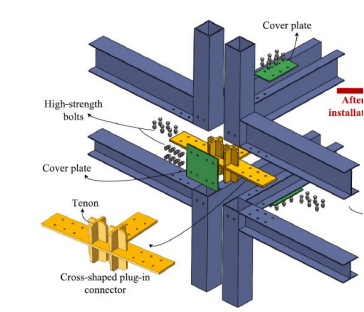
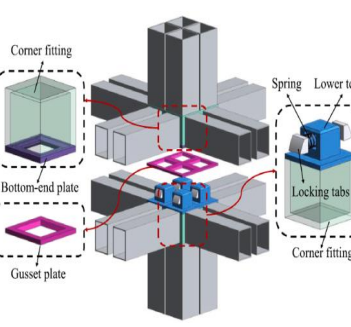
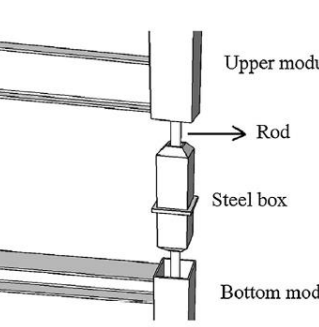
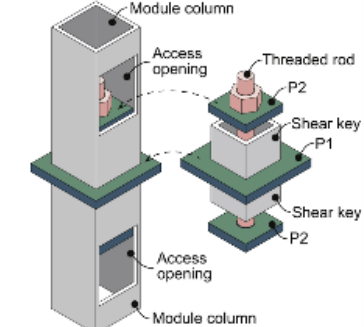


(b)

Figure 1.1: (a) One9 Apartments during construction (Hickory Group, 2014) [1]; and (b) Modular building structure

A modular building can be considered to be similar to a structure in which many bricks are stacked (see Figure 1.1 (b)). From a structural point of view, in order for the many individual modules to behave stably as a compatible building, the connections between modules are most critical. These inter-module connections should be sufficiently rigid both vertically and horizontally, and the installation of the connections also needs to be as easy as possible. In recent years, many inter-module connections have been developed for steel-framed modular buildings to improve the constructability and performance of modular buildings. The inter-module connections can be classified into four main types: (1) welded connection; (2) bolted connection; (3) mechanical connection; and (4) post-tensioned connection (see Table 1.1).

Table 1.1: Representative types of inter-module connections for steel-framed modular buildings

Welded connection	Bolted connection	
 (a) [2]	 (b) [3]	 (c) [4]
Mechanical connection	Post-tensioned connection	
 (d) [5]	 (e) [6]	 (f) [7]

Each type of connection has its advantages and disadvantages. Generally, the welded connection assures the best strength and rigidity; however it requires more time, higher costs, and more skilled workers compared to other types of connections. As an easier and cost-efficient method on site, the bolted connection is preferred for speedy construction. Although the bolted connection is convenient, the modules cannot be manufactured as complete finished products. This is because the installation of the bolts requires external access space, resulting in extra finishing works on site. Also, the shape of the

connection is complex due to the many bolts required (see Table 1.1 (b) and (c)), making it difficult to assemble and disassemble modules on site. In order to overcome these issues of the bolted connection, mechanical and post-tensioned connections have been developed by connecting two modules vertically. In the case of the mechanical connection (see Table 1.1 (d)), self-locking devices are used, making the assembling process of the modules easy and convenient. However, disassembling for reuse of the modules is generally difficult, and the manufacturing cost of the mechanical device is higher than for ordinary bolted connections. In the case of the post-tensioned connection, tensioned tie rods passing through the upper and lower columns are used for connecting the upper and lower modules (see Table 1.1 (e)). Preload is applied to the tied rod by using a hydraulic jack on site. Connections that do not require hydraulic tensioning have also been suggested (see Table 1.1 (f)), which alternatively use a torque wrench through access holes drilled on the surface of the columns. However, the loss of column cross-section is undesirable, particularly for seismic design where the column is an essential member requiring high performance.

Of the inter-module connections that have been researched and developed, each of the suggested inter-module connections has its own advantages, but it seems that the connections do not maximise the benefits of modular construction, such as speedy construction and reusability of modules.

In this thesis, based on an in-depth analysis of the existing inter-module connections, an innovative post-tensioned type connection that maximises construction speed and reusability of modules is proposed for mid-rise steel-framed modular buildings in both seismic and non-seismic areas.

1.2. Aims and Objectives

The main aims of the proposed research are as follows:

- (a) To develop an innovative post-tensioned type inter-module connection (i.e. using couplers and high-tensile steel rod bolts without requiring a hydraulic jack), which can maximise construction speed and reusability of modules, for mid-rise (8-12 storey) steel-framed modular buildings in seismic areas.
- (b) To develop principles of optimal design of a post-tensioned type connection to enable engineers to design and perform structural analysis in practice.

In order to achieve the research aims, the objectives are as follows:

- Develop the details of a new post-tensioned type connection for easy assembly and disassembly of the modules based on an analysis of existing inter-module connections.
- Identify the critical failures of the proposed post-tensioned type connection and analytically

determine the demand preload required to prevent these failures.

- Conduct experiments on beam-to-column joints to verify the validity of the analytically suggested demand preload and evaluate its seismic performance.
- Perform numerical simulations using a 3D FE joint model calibrated against the experiments to investigate the influence of parameters affecting the behaviour of the proposed connection.
- Develop a simplified 2D FE joint model of the proposed connection which can describe important features of the connection, and validate it by comparing it to the 3D joint model for investigating the effect of the connection behaviour on the frame behaviour.
- Investigate the performance of the modular building frame with the proposed connection when subjected to design wind and seismic loads, and conduct performance-based seismic assessments to evaluate structural safety in a medium intensity seismic area.
- Produce design guidelines for the proposed connection for a mid-rise steel-framed modular building in non-seismic and seismic areas based on the experimental and numerical investigation results.

1.3. Research Scope

The thesis focuses on the proposed post-tensioned type inter-module connection for the steel-framed modular building. An evaluation of seismic performance was conducted on a moment-resisting frame composed of corner-supported type modules.

1.4. Research Method

1) Literature

Based on an in-depth analysis of existing inter-module connections from the relevant literature, an innovative post-tensioned type connection is proposed, which connects two modules by preload using the developed coupler and high-tensile steel rod bolts (**SRB**) within columns.

2) Experiments

Coupon tests were performed to determine the actual material properties of the steel used for the beam-to-column joints and for input to the numerical simulations. Also, uniaxial tensile tests were performed on the mechanical splice of the coupler and the SRBs to check if the developed coupler had sufficient strength to connect modules vertically. Tests applying preload to SRBs were carried out to check the feasibility of applying preload to 3m-long SRBs using the developed coupler, as well as to determine the amount of torque corresponding to the demand preload. In addition, full-

scale beam-to-column joint tests were conducted to evaluate the seismic performance of the beam-to-column joint for steel-framed modular buildings featuring preload in the columns.

3) Numerical simulation – 3D joint model

An advanced finite element beam-to-column joint model composed of 3D solid elements was created and used for the design of the full-scale joint test specimens and test set-up. After conducting the joint test, the 3D joint model with material properties obtained from coupon tests was calibrated based on the test results. Parametric studies were conducted using the calibrated joint models to investigate the influence of different parameters on the behaviour of the beam-to-column joints and SRBs in columns.

4) Numerical simulation – 2D frame model

An advanced 2D simplified FE model joint model was developed, which could accurately describe the essential features of the proposed post-tensioned type connection. The 2D model was validated by comparing it to the 3D joint model, and then applied to mid-rise steel framed modular buildings to investigate their behaviour under design wind and seismic loading. The seismic performance was assessed using nonlinear static (Pushover) analysis and the performance-based N2 method.

1.5. Outline of Thesis

Chapter 2 contains an overview of previous experiments and numerical investigations of various inter-module connections for steel-framed modular buildings. Additionally, the backgrounds to modular construction and structural systems are described.

Chapter 3 introduces the proposed innovative post-tensioned type connection (which will be called '**AJ connection**' in this thesis) for a steel-framed modular building, the assembly process, and the preload mechanism of the connection. Also, the failure modes of the connection are defined (i.e. gap-opening and slip), and the determination process for the demand preload required to prevent the modes is suggested. Further, in order to apply the connection in seismic areas, design guidelines for moment-resisting frames with the connections based on the design criteria and detailing rules of Eurocode 1998-1 are developed.

Chapter 4 contains the experimental investigation for the AJ connection, including the test procedure and results. The experimental program consists of coupon tests, the uniaxial tensile tests for the mechanical splice of the developed coupler and the SRBs, the tests on applying preload to 3m-long SRBs using the coupler, and the full-scale beam-to-column joint tests inclusive of the AJ connection (which will be called '**AJ joint**' in this thesis).

Chapter 5 presents the numerical investigation for the AJ beam-to-column joint using advanced finite element 3D models. Unfortunately, an unexpected issue from the test setup affected the joint test results and impeded the evaluation of the joint behaviour. Therefore, the seismic performance of the AJ joint was re-evaluated using the FE models calibrated against the test results. In addition, parametric studies were conducted using the calibrated joint models to investigate the influence of various parameters on the behaviour of the joints and SRBs in columns.

Chapter 6 contains the numerical analysis of a modular building frame with the AJ connection. The 2D simplified FE joint model inclusive of the AJ connection was developed and validated by comparing it to the 3D joint model calibrated in Chapter 5. Then, applying this verified 2D joint model to the moment-resisting frames in a mid-rise modular building, the performance of the AJ connection was investigated for modular buildings subjected to design wind and seismic loads. The performance-based seismic assessment of modular buildings was conducted using nonlinear static analysis and the N2 method.

Chapter 7 provides practicing engineers with guidance on designing the AJ connection for steel-framed modular buildings based on the analytical investigation (Chapter 3), the experimental investigation (Chapter 4) and numerical simulations (Chapters 5 and 6). The design recommendations use the provisions of Eurocode 3 and Eurocode 8.

1.6. Contribution of Thesis

The published conference papers, journal papers, and registered patents based on the work presented in this thesis are as follows.

Patents

- B. Cho, K. Lee, D. Jung, Y. Song and D. Lee. Modular Unit, KR Patent 10-2362146, filed June 7, 2021, and issued Feb 8, 2022

Journals

- D. Jung, B. Cho, Lee, J. Lee, D. Lee and K. Lee. (2022). Experimental Evaluation on Seismic Performance of Exterior Connection in Modular Building System with AJ Connector. Journal of Korean Society of Steel Construction, 34(1), 13-24

Conference papers

- D. Jung, B. Cho, K. Lee and Y. Kwon. (2022). Study on Torque Coefficient of Post-Tension Connection in Unit Module. Proceedings of the Annual Conference of the Architectural Institute of Korea, 42(2), 520-521
- D. Jung, B. Cho, K. Lee, D. Lee and J. Lee. (2022). Seismic Performance Evaluation of Post-tensioned Connection System in Modular Building according to Post-tension Magnitude. Proceedings of the Annual Conference Korean Society of Steel Construction, 33 (1), 235-236
- K. Lee, B. Cho, K. Rasmussen and D. Jung. (2023). Evaluation of Seismic Performance of Innovative Preloaded Connection for Steel-framed Modular Buildings through Numerical Investigations. Proceedings of the 12th International Symposium on Steel Structures by the Korean Society of Steel Construction, Jeju, Korea
- K. Lee, K. Rasmussen and B. Cho. (2023). Evaluation of the Structural Performance of Modular Buildings with Pre-loaded Inter-module Connections. Proceedings of the 9th International Conference on Thin-Walled Structures, Sydney, Australia

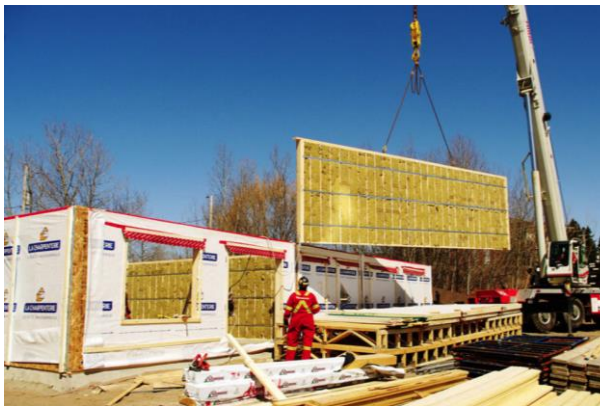
Chapter 2

Literature Review

2.1. Prefabricated Construction

Prefabricated construction (i.e. prefab) is a method in which necessary components are manufactured in a factory and then transported on-site for assembling to build a complete building. Historically, it was used to relieve the massive housing shortages in destroyed areas of European countries during World War II because it enabled the supply of houses quickly at a lower price compared to conventional construction. In recent years, the interest in prefabricated construction has been growing for solving severe urban housing problems in a sustainable way [8] because of its many advantages, such as saving construction time and costs, higher quality control, reduction of waste, etc [9, 10]. Prefabricated construction is currently being applied to low-rise to high-rise buildings worldwide. For example, in China, a 30-storey building was constructed with this system in 15 days [11, 12].

Generally, prefabricated building systems can be categorised into two construction methods: panelised and modular construction [13]. Both have the same construction process from manufacturing components in a factory, being transported to the prepared foundation on-site to being assembled. The main difference between the two methods is the prefabricated unit. In panelised construction, 2D components such as structural panels and line elements (i.e. columns, beams and walls) are used to construct the building (see Figure 2.1 (a)), while modular construction uses 3D volumetric modules, which are usually finished products (see Figure 2.1 (b)).



(a) Panelised construction [14]



(b) Modular construction [15]

Figure 2.1: Offsite-manufactured prefabricated building systems

2.2. Background to Modular Construction

Modular construction is an industrialised construction method in which modules are manufactured in factories, such as how cars or computers are assembled in production lines [16]. The modular construction is a new type of construction that combines methods from the conventional construction industry with that of the manufacturing industry. By manufacturing approximately 85-90% of the construction materials (including the structural frames, the interior and exterior), the equipment, on-site work and construction time are significantly reduced compared to the conventional construction system [17, 18]. It is also recognised as sustainable construction because it minimises the environmental impact by reducing waste with the construction process [19, 20]. For these reasons, the interest in modular construction has been increasing in recent years. Modular construction has various benefits to clients, suppliers, and the wider society as follows:

- **Saving time and costs:** While modules are manufactured in a factory, on-site work for the foundation proceeds simultaneously (see Figure 2.2). Also, producing modules is independent of weather conditions because all work is conducted indoors. Consequently, construction time can be reduced by 50% compared to conventional construction [21, 22]. This shorter construction period saves overall construction costs and gives clients the opportunity to utilise their building earlier (e.g. being able to rent out the units earlier).
- **Eco-friendly construction:** While on-site work in conventional construction typically produces large amounts of waste, modular construction reduces waste and greenhouse emissions by maximising the efficiency of raw materials and energy in the construction process [23-25]. This is because the consistent work process indoors allows a manufacturer to know the necessary amount of materials and to control the energy in the production of modules. It has been found that up to a 52% reduction in construction waste can be achieved in comparison with conventional construction [26]. In addition, it minimises negative effects from on-site work, such as site disturbance and noise to residents around the construction site since modular construction requires significantly less days of on-site works.
- **Reuse and relocation of modular buildings:** The modular buildings can be disassembled and transported to other locations for reuse. From an environmental and economic point of view, it means that a lot of the required new raw materials, energy, labour force, and construction time for new buildings can be saved. Moreover, for clients, this gives them more mobility and flexibility of their buildings.
- **Efficient construction method in unfavourable conditions:** Modular construction can be used for unfavourable construction sites or where there are severe weather conditions for building construction (e.g. small islands, mountainous areas, and cold areas where there is a prolonged

snow season) because it is possible to transport modules by road, rail, sea and even by air. Also, modular construction can be a time-efficient solution to supply temporary houses in areas affected by natural disasters such as cyclones, earthquakes and tsunamis [27].

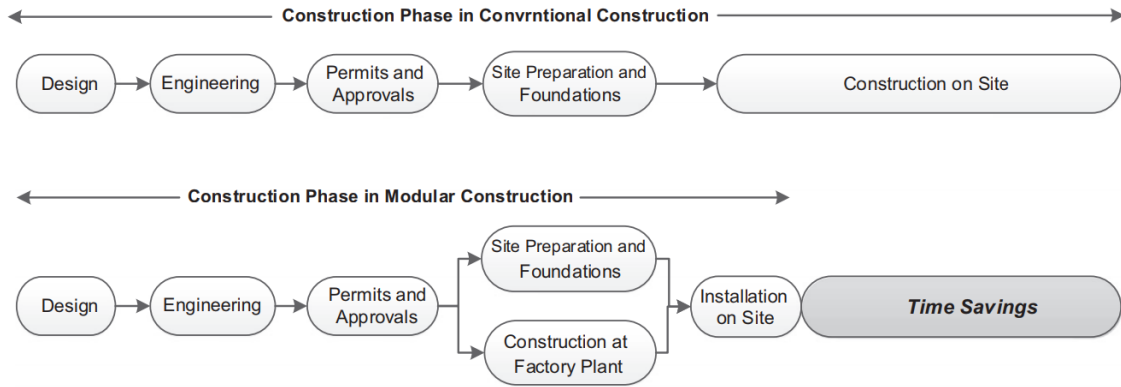


Figure 2.2: Comparison between conventional construction and modular construction [28]

In the 1990s, modular construction was mainly employed for low-rise residential buildings. With the development of diverse high-rise building technologies and market needs in the 2000s, modular construction is currently being applied to residential and commercial buildings of 10-40 storeys worldwide. Figure 2.3 shows modular buildings ranging from low-rise to high-rise buildings.



(a) 6-storey apartment, New York, USA [29]



(b) 10-storey apartment, Melbourne, Australia [1]



(c) 29-storey student accommodation, London, UK [30]

Figure 2.3: Current use of modular construction in low- to high-rise buildings

2.3. Steel-framed Modular Buildings

2.3.1. Steel-framed Modules

Steel, concrete, and timber are the most commonly used materials for modular buildings. Figure 2.4 shows modules using different materials. In recent years, glass fibre-reinforced polymers have been studied to be used as structural members for modular buildings [31, 32]. Among them, steel is the most widely used material because of its high strength, durability, and lightweight. These material properties are suitable for low- to high-rise modular buildings. In addition, steel has higher ductility than other materials, so it is advantageous to apply to modular buildings, even in earthquake-prone areas. As a recyclable material, steel is appropriate for sustainable buildings as well.

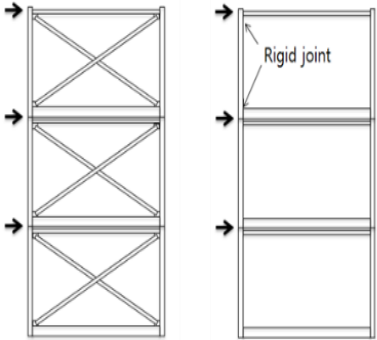
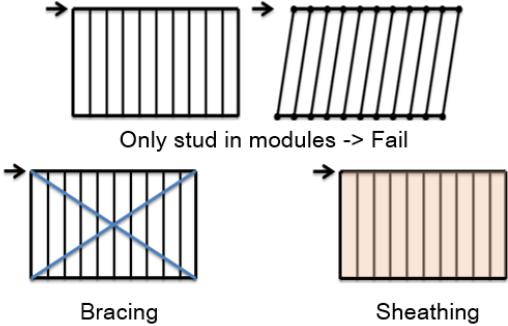


Figure 2.4: Modules in different materials

The steel-framed module is mainly classified into three basic types based on the load transfer mechanism [21], as shown in Table 2.1:

- Corner-supported module: Vertical loads are transferred through edge beams and corner columns in the module. Since the corner columns have to resist large compressive force, square or rectangular hollow sections are usually used. Lateral loads are resisted by bracing or rigid joints between beams and columns.
- Load-bearing module: Vertical loads are transferred through its side-walls, being typically steel C-studs in the module. Lateral loads are resisted by bracing [36-39] or sheathing [40-43] installed on the side-walls because having only studs in the side-walls is insufficient.
- Non-load bearing module (i.e. pods): Instead of modules resisting vertical and lateral loads, other exterior structural frames resist the loads. The modules are inserted and installed in the frames.

Table 2.1: Three basic types of steel-framed modules based on the load transfer mechanism

Type	Vertical load-resisting element	Lateral load-resisting element
Corner supported module	 (a) Columns [44]	 (b) Bracing or rigid joint in a module [45]
Load-bearing module	 (a) Side walls [33]	 (b) Sheathing or bracing in a module
Non-load bearing module (i.e. Pods)	 (a) Non-load bearing module [46]	 (b) Exterior frame [47]

2.3.2. Structural System for Steel-framed Modular Buildings

Steel-framed modular buildings can be categorised into three main structural systems:

- Modules self-supported system:

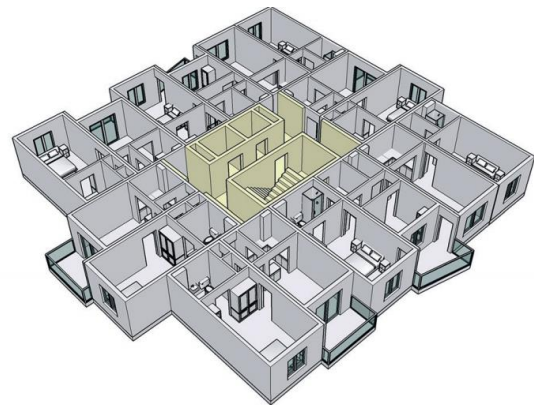
Modules resist vertical and lateral loads by themselves. The vertical loads are transferred through side-walls or corner columns in the modules. Bracing or sheathing in the modules resists the lateral loads. Since the load-resisting capacity of the module itself is not large, this structural system is generally used for low-rise buildings.

- Modules with a central core:

The structural system using central core and modules is applied to mid- or high-rise modular buildings. The vertical loads are transferred through side-walls or corner columns in the modules. The lateral loads are resisted by a reinforced concrete (RC) or steel core, as shown in Figure 2.5. The RC core is normally located at the centre of the building. The lateral force is transferred to the core through secure diaphragm action between the modules and the core [21].



(a) Apex house, Wembley, London [48]



(b) Modules clustered around a concrete core [49]

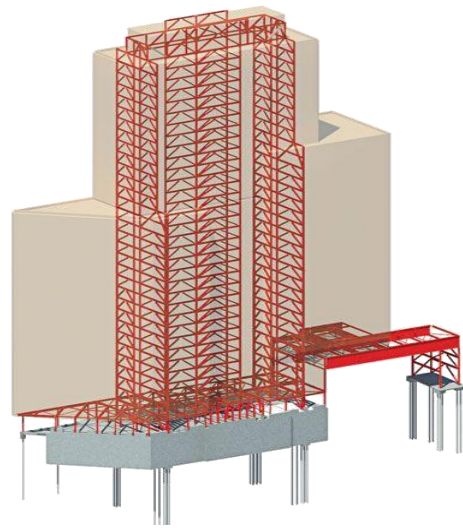
Figure 2.5: Structural system of modules with a central core for mid- and high-rise modular buildings

- Modules with steel frames:

Steel frames and modules are applied to mid- or high-rise modular buildings as well. The vertical loads are transferred through side-walls or corner columns in the modules. The lateral loads are resisted by the steel frames (see Figure 2.6 (b)), which play a major role in the whole structure instead of the centre core. The steel frames are installed with the modules simultaneously on-site. In this system, secure connections between the modules and the steel frames are important for transferring the lateral loads to the steel frames.



(a) Atlantic Yards B2 in Brooklyn, USA [50]



(b) Structural system of Atlantic Yards B2 [44]

Figure 2.6: Structural system of modules with steel frames for mid- and high-rise modular buildings

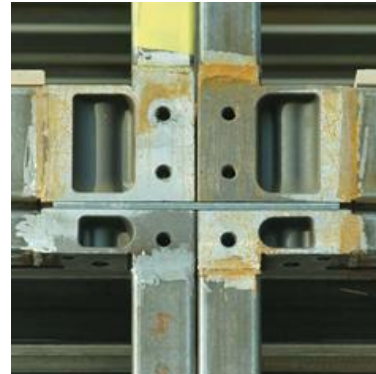
2.4. Inter-module Connections for Steel-framed Modular Buildings

The inter-module connection is most critical for the structural stability of a modular building. It should provide secure rigidity to make the many individual modules behave stably as a compatible building. In addition, there has to be easy installation of the connection to maximise its advantages, such as speedy construction and ease of disassembly to reuse modules. Recently, the demand for mid- and high-rise steel-framed modular buildings has grown, so the vertical and lateral loads that these modular buildings should resist are much larger than those for low-rise buildings. The structural performance and easy installation of inter-module connections play a more important role.

The connections in steel-framed modular buildings have more complex shapes than those in conventional steel structures (see Figure 2.7). For instance, the interior connection in conventional steel structures consists of one column and two beams, while the connection in modular buildings consists of four columns and four beams. In the case where eight modules are installed, a total of eight columns and sixteen beams needs to be placed, and so the joint is not only very complex, there is also difficulty of access to connect all modules securely.



(a) Typical steel structure [51]



(b) Modular building structure (VectorBloc Corp) [52]

Figure 2.7: Beam-to-column joint shapes

In recent years, many inter-module connections have been developed to improve the constructability and performance of steel-framed modular buildings. The inter-module connections can be classified into four main types: (1) welded connection; (2) bolted connection; (3) mechanical connection; and (4) post-tensioned connection [5]. Each type of connection has advantages and disadvantages. Generally, the welded connection assures the highest strength and rigidity of the inter-module connection. It also provides excellent flexibility in positioning the modules during the assembly process on-site. However, the welding process on-site requires more time, higher costs, and more skilled workers compared to other types of connections. Accordingly, the other types of connections have been preferred for their ease of installation, cost-effectiveness, and speedy construction. This section reviews previous experimental and numerical investigations of various types of inter-module connections (excluding

welded connections) for steel-framed modular buildings.

2.4.1. Bolted connection

Gunawardena et al. [53, 54] (2016) investigated a bolted connection between corner-supported modules for a high-rise modular building (see Figure 2.8). Upper and lower modules are connected at columns vertically by bolts, and plates extended from the top and bottom of columns overlap each other to connect the modules horizontally. The connecting process is conducted from inside the building using access space through the ceiling or floor. This bolted connection was tested under shear loading with different bolt and bolt-hole sizes. The test results showed that slip from the column-to-column connection is critical, and large slot holes must be avoided because accumulated slips in each storey causes large top displacement in the building. Based on the test results, the 2D spring connection model was developed and applied to the building frame for advanced seismic analysis. In addition, in order to construct high-rise modular buildings without RC core for using the full benefits of modular construction, the corner-supported module reinforced by stiff concrete walls was suggested to increase the lateral force resistance of the module itself. The feasibility of the structure system was verified by performing numerical simulation studies using 10- and 20-storey 3D modular building models subjected to seismic and wind loads.

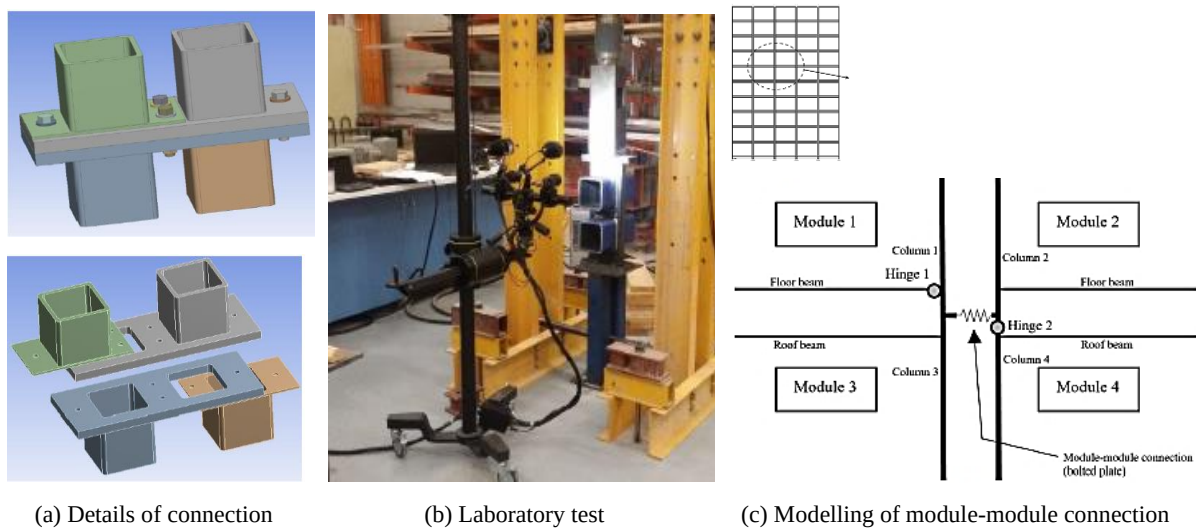


Figure 2.8: Bolted column-to-column connection by Gunawardena (2016)

Lee et al. [55, 56] (2017) proposed a ceiling-bracket-type modular joint with corner-supported modules for application to mid- and high-rise modular buildings in seismic areas. To prevent the loss of an RHS column sectional area caused by an access hole at the column for installation of a bolted connection, upper and lower modules are connected at beam flanges using bolts and a developed steel plate (see

Figure 2.9 (b)). The installation of the bolted connection is executed inside modules, so the ceiling in the module should be partially opened when the module is manufactured in a factory. The exterior joints (C-shaped channel beam and RHS column) were tested under cyclic loading to investigate their seismic performance using the bracket thickness, depth, and stiffener as parameters. The capacity design was applied to avoid the plastic hinge formation in the column and rather, have it formed in the beams to dissipate energy. The proposed connection had the equivalent performance to a fully restrained moment connection and satisfied the criteria set by AISC 341-16 for the special moment-resisting frame.

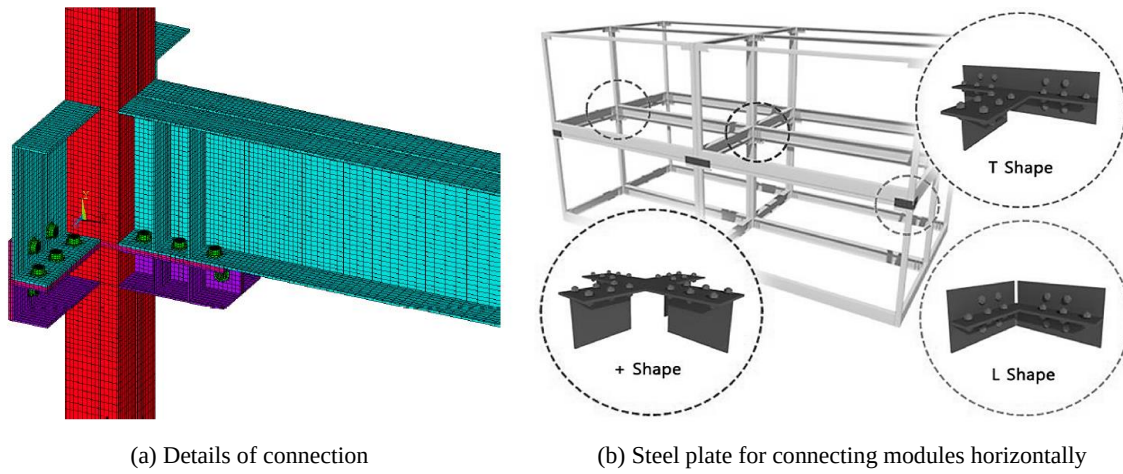


Figure 2.9: Ceiling-bracket-type modular joint by Lee et al. (2017)

Cho et al. [57] (2019) suggested an inter-module connection with blind bolts, which allows free access to the structural members of the closed cross-section, such as an RHS column (see Figure 2.10). Four modules are connected by a steel connection plate and blind bolts. In order to avoid the weak column-strong beam behaviour, which might be caused by ceiling and floor beams, the suggested beam-to-column joint inclusive of the blind-bolted connection was designed with the strength of its beam members considerably lower than that of its column members. Also, knee brace members were installed at the joint to increase its bending stiffness. The seismic performance of the interior beam-to-column joint (C-shaped channel beam and RHS column) was investigated under cyclic loading. The test results indicate that the joint inclusive of the blind-bolted connection has a sufficient level of strength. The specimen with the knee braces retained approximately 2 times higher stiffness and 1.6 times higher strength than the one without them. In addition, the simplified connection model utilising rotational springs was proposed, which could accurately predict the structural behaviour of the proposed blind-bolted connection.

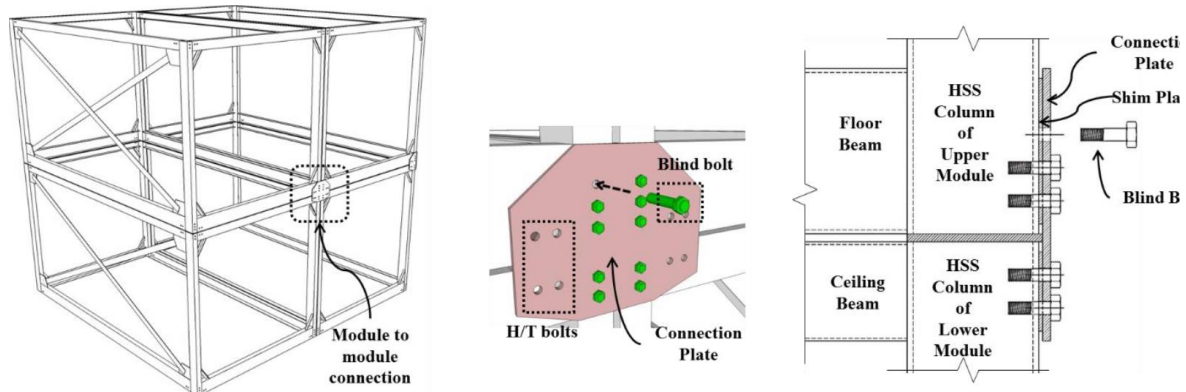


Figure 2.10: Blind-bolted connection by Cho et al. (2019)

Deng et al. [58, 59] (2018) proposed a bolted connection with a welded cover plate to meet the architectural design demand and form a moment-resisting frame with better seismic behaviour (see Figure 2.11). The connecting process is conducted from outside of the building through access space on the surface of columns. The modules are connected vertically and horizontally using the cruciform gusset plate and bolts, and then the cover plate is welded to replace the cut portions of the columns. A series of full-scale exterior and interior beam-to-column joints (HSS beams and HSS columns) were tested under monotonic and cyclic loading to investigate its load-transferring mechanism and seismic performance. The cyclic loading test results show that the exterior joint specimens experienced the strengthening stage and developed the full strength of the two beams, but due to weld fracture at the beam-column welded connection, the ductility was reduced by 20-40% compared to the one under monotonic loading. Since they had earlier fracture than expected due to the stress concentration at the corners of the beam-column welded connection, the connection was suggested to use for the ordinary moment-resisting frame. The connection was classified as a semi-rigid connection, according to Eurocode 3.

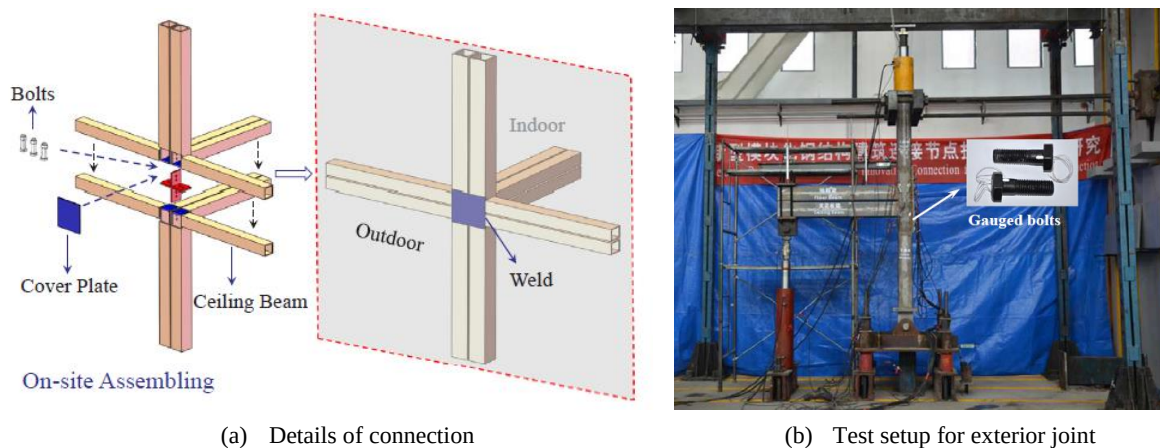


Figure 2.11: Bolted connection with welded cover plate by Deng et al. (2018)

Chen et al. [3, 60] (2017) suggested the inter-module connection without cross-sectional loss at columns, which comprises beam-to-beam long-stay high-tensile strength bolts and a cast plug-in device for horizontal connections (see Figure 2.12). The column in an upper module is inserted into the corresponding head at the plug-in device for alignment and the long-stay bolts are inserted into the holes in the beam from the inside of the modules and fastened to connect the upper floor beam and bottom ceiling beam. Monotonic and cyclic loading tests were performed for interior and exterior beam-to-column joints to study the load-bearing mechanism and seismic behaviour. The diagonal stiffeners could increase the bending stiffness and lateral bearing strength but resulted in a reduction of energy dissipation and there was relatively rapid deterioration of the joint once the stiffeners fractured. The tests and FE simulations revealed that the floor and ceiling beams had individual bending behaviours instead of ideally working together as a large composite beam. The tests results showed that cracking of the weld at the beam-column connections was the critical failure, indicating that further attention is required to ensure weld quality and connection capability. Subsequently, Peng et al. [61] (2020) studied the cracking and fracture of the weld at the beam-column connections for the optimal design of this inter-module connection. In addition, Peng et al. [62] (2021) developed a simplified joint model for this inter-module connection using spring elements and investigated its lateral performance in multi-storey modular buildings using pushover analysis. The effect of the strong column and weak beam ratio and boundary conditions of columns on the inelastic behaviour was investigated. The seismic performance of the modular frames was assessed using the displacement method, as specified in FEMA-356.

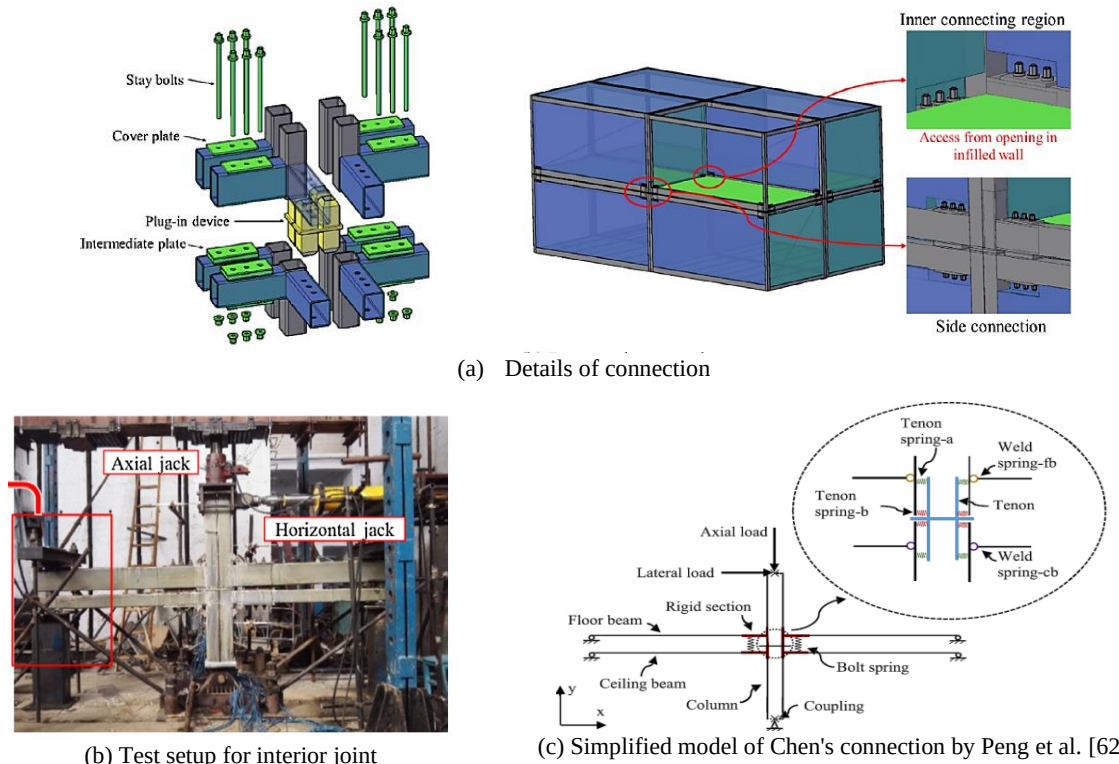


Figure 2.12: Inter-module connection using cast plug-in device long-stay high-tensile strength bolts [3] by Chen et al. (2017)

Dhanapal et al. [63] (2019) investigated the performance of the VectorBloc connector (see Figure 2.13), which was developed by VectorBloc Corp., through axial tension and compression tests. The modules are connected at the corners using VectorBloc connectors consisting of upper and lower blocs. The upper bloc connects the ceiling beams to the column in a lower module whereas the lower bloc connects the floor beams to the column in an upper module. All the columns and beams are made of HSS sections. When the modules are installed on-site, a gusset plate and registration pin are installed on the upper bloc. Then, the upper modules are connected to the lower modules by the threaded end of an SHCS. The connecting process is conducted from either inside or outside of the building on the floor in the upper modules. Dhanapal et al. [64] (2020) conducted both experimental and numerical studies on the bending behaviour, with and without axial load, of a typical corner connection made of the VectorBloc connector and thin-walled HSS beams and HSS columns. According to this study, the connection can be classified as a rigid connection. Subsequently, Hajimohammadi et al. [65] (2022) studied the registration pin and suggested an empirical method for determining the thread stripping capacities of the pin when a non-zero loading angle is present.

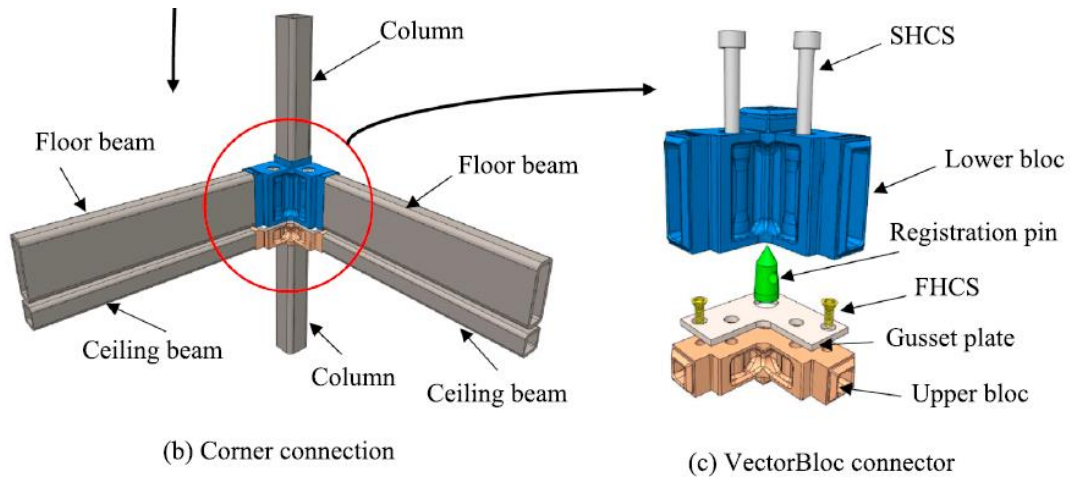


Figure 2.13: VectorBloc connector by VectorBloc Corp (2021) [63]

Chen et al. [66] (2019) also suggested a rotary inter-module connection at the end of modular columns with a corner fitting that can be used for module hoisting and transportation. The connector is composed of connecting plate, nut, upper and lower rotating parts (see Figure 2.14). This connection has a simple configuration and convenient assembly and disassembly. In addition, it is not required to open holes in beams or columns to install the connections, so it did not compromise the internal architecture decoration of the modules. The connecting process is conducted from outside of the building through an access hole on the surface of the corner fitting. A bending resistance test was conducted to understand the working mechanism of the rotary connection, and FE analysis was conducted to investigate the initial rotational stiffness obtained through the preceding test [66].

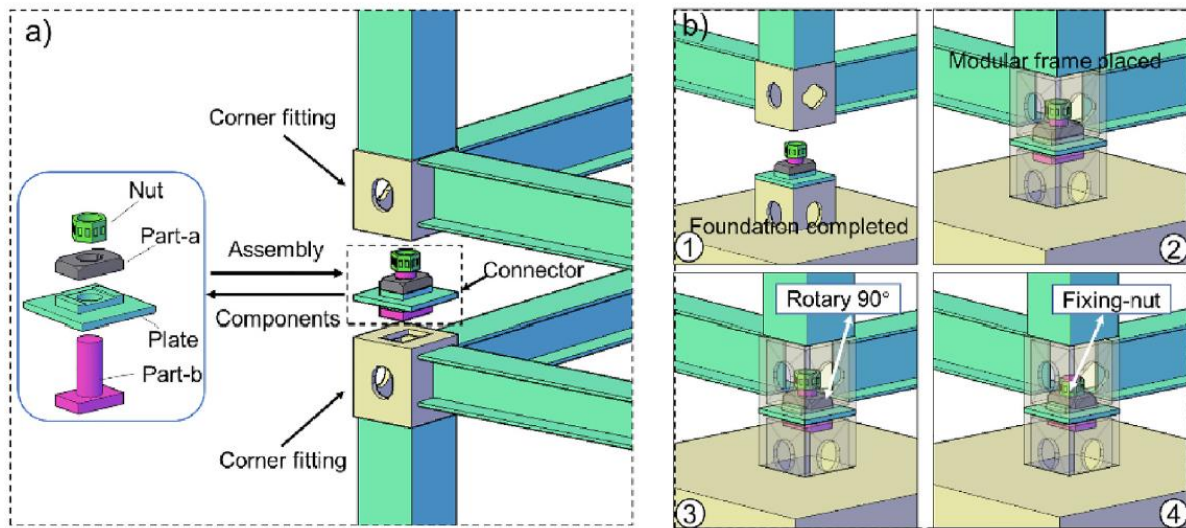


Figure 2.14: Installation procedure of rotary inter-module connection by Chen et al. (2019)

Then, Chen et al. [67] (2020) conducted tensile and shear resistance tests for the rotary inter-module connection, FE analysis, and parametric analysis for the connection to investigate the tensile and shear performance and influencing factors affecting the bearing capacity of the connection (see Figure 2.15). Liu et al. [68] (2020) conducted monotonic tests for two-storey full-scale modular frames with the rotary inter-module connections to clarify the understanding of their lateral-force resistance mechanism and then developed theoretical modelling of the initial lateral stiffness for the modular structures inclusive of the connections based on the test results. Liu et al. [69, 70] (2022) also performed cyclic tests with two-storey full-scale modular frames to investigate the seismic performance of the connection, considering different reinforcing intra-module connection details.

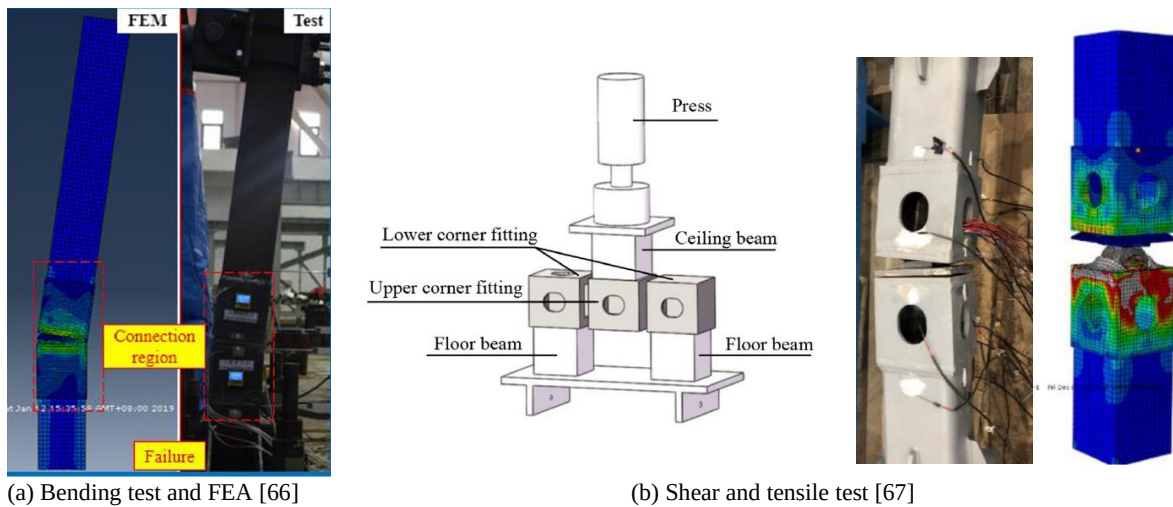


Figure 2.15: Rotary inter-module connection by Chen et al. (2019)

2.4.2. Mechanical connection

Dai et al. [71] (2019) suggested a plug-in self-lock connector to connect columns vertically (see Figure 2.16). A single plug-in self-lock joint consists of two joint boxes and a set of plug-in self-lock connectors. The stud is the main element for resisting axial and shear forces. The joint boxes are installed on the top and bottom of the upper and lower HSS columns, and the RHS or H-section beams are welded to the joint boxes. When an upper module is placed on a lower module during assembly, it is connected automatically by the plug-in self-lock connector. As a result, the on-site activity is minimised, and construction speed is increased. Since all connection devices are set in the joint box, there are no conflicts with the other installed structure components. The performance of the connection was tested by a push-out test, and then its performance was tested using full-scale exterior beam-to-column joint specimens under monotonic and cyclic loading. The test results showed that the joints have excellent seismic behaviour with failure of the beam, showing good ductility of 2.7-3.1 and energy dissipation capability. They were classified into semi-rigid joints according to Eurocode 3. In addition, although the joints of the H-section beams were smaller section areas and similar section inertia moments, they presented significantly higher strength and energy dissipation capability than the joints of the square hollow section beams. Also, the strength of the joint with a cover plate was 8-12% higher than that of the joint with rib stiffeners.

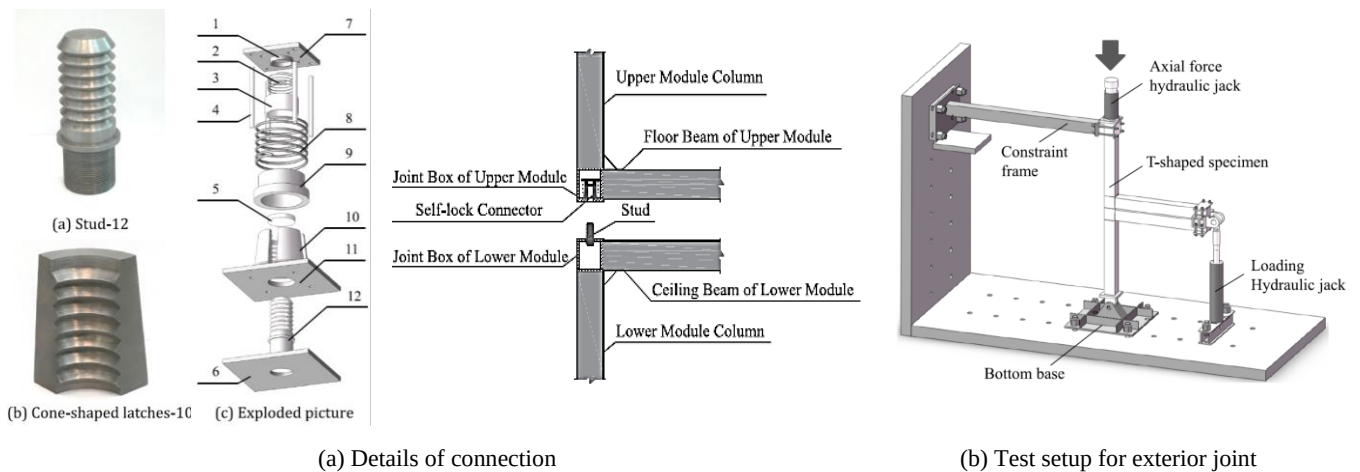


Figure 2.16: Plug-in self-lock joint by Dai et al. (2019)

Chen et al. [5] (2021) proposed a self-locking inter-module connection that has a simple configuration (see Figure 2.17). The locking tabs are installed on the top and bottom of the upper and lower HSS columns. When an upper module is placed on a lower module, they are connected automatically by the locking device. The exterior beam-to-column joint tests were conducted under cyclic loading to assess the seismic behaviour of the connection using the connection type (i.e. welded and self-locking), gap

between floor and ceiling beams and beam section (i.e. H-section and RHS) as parameters. The test results showed the following: (1) that the self-locking connection had the same reliable inter-module connection as welding; (2) the average ductility coefficients of the joints with self-locking connections were 2.50–3.20; (3) the gap between twin-beams did not affect the joint behaviour; and (4) the ductility and energy dissipation capacity of the joint with H-section beam was greater than the joint with the RHS beam. Further, the detailed FE 3D model was developed and validated with the test results. The reinforcement of the cover plate on the beams prevented the failure of the beam-column welded connection by forming the plastic hinges away from the connection, which guaranteed the ductile behaviour of the joint.

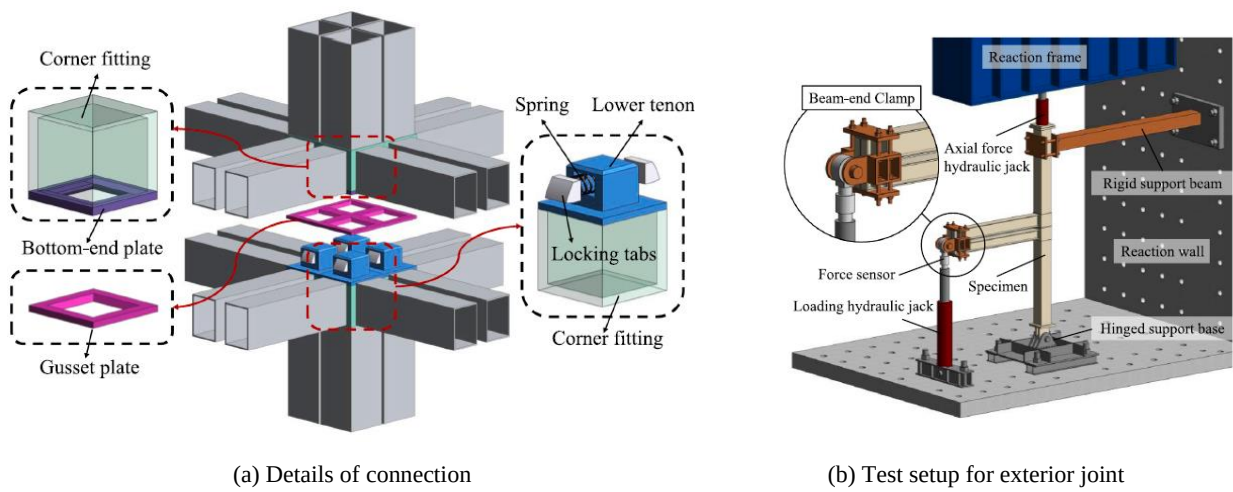


Figure 2.17: Plug-in self-lock joint by Chen et al. (2021)

2.4.3. Post-tensioned connection

Chen et al. (2017) [72] proposed a pre-stressed modular connection with corner-supported modules with rectangular concrete-filled steel tubes as columns for high-rise buildings (see Figure 2.18). The modular columns are connected vertically by the pre-stressed strands. The strands are stretched first to the design stress level, and then the concrete is poured into the columns. Infill concrete increases the compression capacity of the column and prevents the column from local buckling. The moment-transferring performance of this connection was studied through monotonic and cyclic loading tests first [73], and then a full-scale two-storey pretension assembled modular frame was tested to investigate its seismic performance under cyclic loading. Dimensions of the tested framed modules are 3.6m X 4.5m X 3.0m with HSS column and H-section shaped floor and ceiling beams. The test results presented good seismic performance without strength reduction and much structural failure at the rarely occurring earthquake level. When contact slipping between concrete and plugin bars or strands at the column-to-column connections occurred, the gap opened between the columns, causing strength and stiffness degradation on the lateral bearing of the overall modular frame. Further, a simplified modelling method

was suggested for the connection to understand the internal stress state of the frame and working mechanism under lateral loads, which shows that the anchoring column ends and the column base region were subjected to high stress.

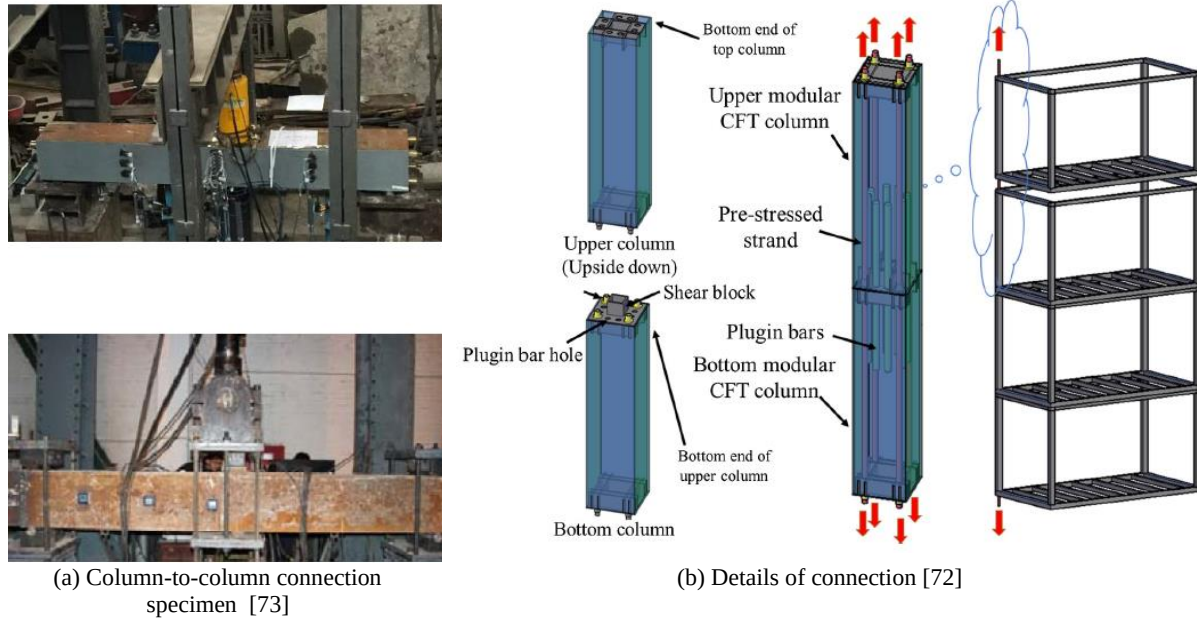
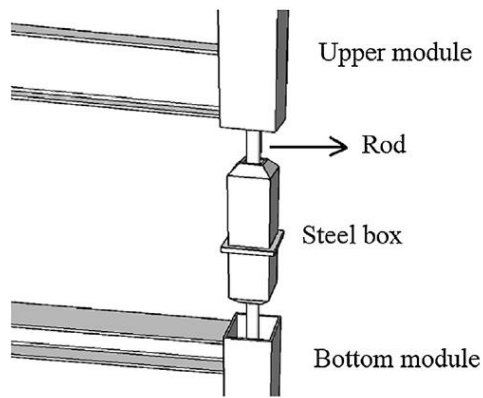
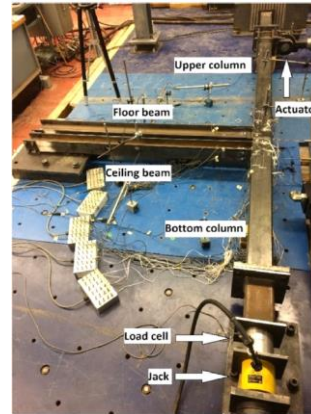


Figure 2.18: Pre-stressed connection using strands and concrete by Chen et al. (2017)

Sanches et al. [6] (2018) suggested a post-tensioned connection for the vertical inter-module connection, using a post-tensioned threaded rod inside the HSS columns and a steel box (see Figure 2.19). Post-tensioning load is applied using a hydraulic jack. The modules are connected horizontally by the steel box. The lateral loads are transferred by the steel box from the upper column to the lower column. Also, this box serves as a guide for the upper module and the rod during the assembly process. The rod can be connected by couplers to cover longer heights and enable several modules to be tied together. The exterior beam-to-column joints (H-shaped beam and HSS column) were tested under cyclic loading to investigate their seismic performance using the magnitude of post-tensioning load, thickness of steel box, connecting the rod by the coupler, as parameters. The test results indicate that, although this post-tensioned connection presented higher stiffness degradation ratios than welded specimens, the proposed post-tensioned connection behaves similarly to a typical welded connection. Additionally, a limitation of the thickness of the steel box was suggested because it affects lateral bearing capacity, energy dissipation, stiffness degradation, etc.



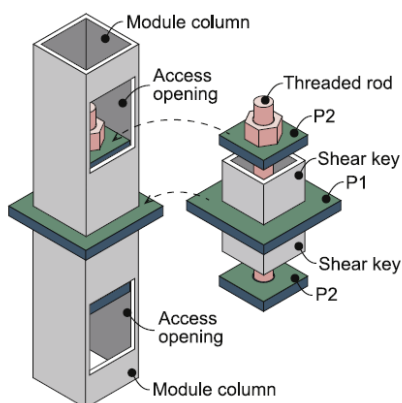
(a) Details of connection



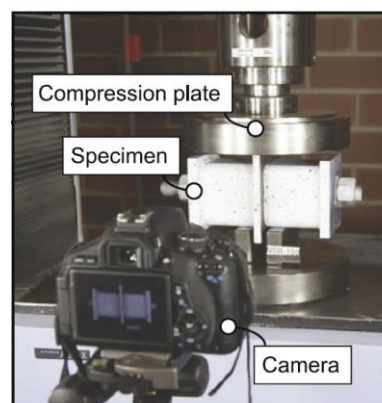
(b) Test setup for exterior joint

Figure 2.19: Post-tensioned connection using rod with steel box by Sanches et al. (2018)

Lacey et al. [74] (2019) developed a new connection consisting of a shear key combined with a post-tensioned tie rod in HSS columns, which connects modules vertically (see Figure 2.20). The tie rod is post-tensioned using a torque wrench through the access opening provided in the columns from inside the module such that the requirement for working at heights during site installation is reduced. The shear force-displacement and load-slip behaviours of the connection were investigated experimentally. In addition, numerical simulation was conducted to investigate the influence of the preload, slip factor and contact area on the load-slip behaviour. The study proposed an empirical model, which has two parameters (i.e. slip load and controls the initial load-slip stiffness), for the initial shear load slip behaviour of the proposed connection. Further, using a calibrated numerical model based on the test, the axial force-displacement and moment-rotation behaviour of the connection were studied [7]. Bending moments acting on the columns had a negligible effect on the shear behaviour of the connection, while axial forces affected the effective clamp force and, hence, the slip resistance.



(a) Details of connection



(b) Test setup

Figure 2.20: Post-tensioned connection using tie rod with shear key by Lacey et al. (2019)

Chua et al. [75] (2020) investigated the effects of inter-module connection modelling and floor diaphragm discontinuity on the global sway behaviour of high-rise modular buildings with different lateral force-resisting systems. The study was conducted using the connection where a tie rod in HSS columns is used to connect modules vertically, and a tie plate is used for the horizontal connection between modules (see Figure 2.21). Simplified horizontal and vertical connection models were proposed by analysing existing simplified connection models suggested by several researchers and capturing the actual stiffness of the inter-module connection. Detailed studies were conducted on a 10-storey modular building frame and a 40-storey steel modular building with a reinforced concrete core. One of the findings in the study was that adding steel bracing to modules does not help to increase the lateral resistance of a modular building if the column-to-column connection has a relatively low tensile capacity and stiffness. Also, although the steel bracings improve the lateral stiffness of the building, it amplifies the toppling effects of the stacked-up modules. Therefore, the tension rod in the column must have sufficient axial stiffness to prevent vertical separation of the columns for effectively transferring the lateral load to the braced modules.

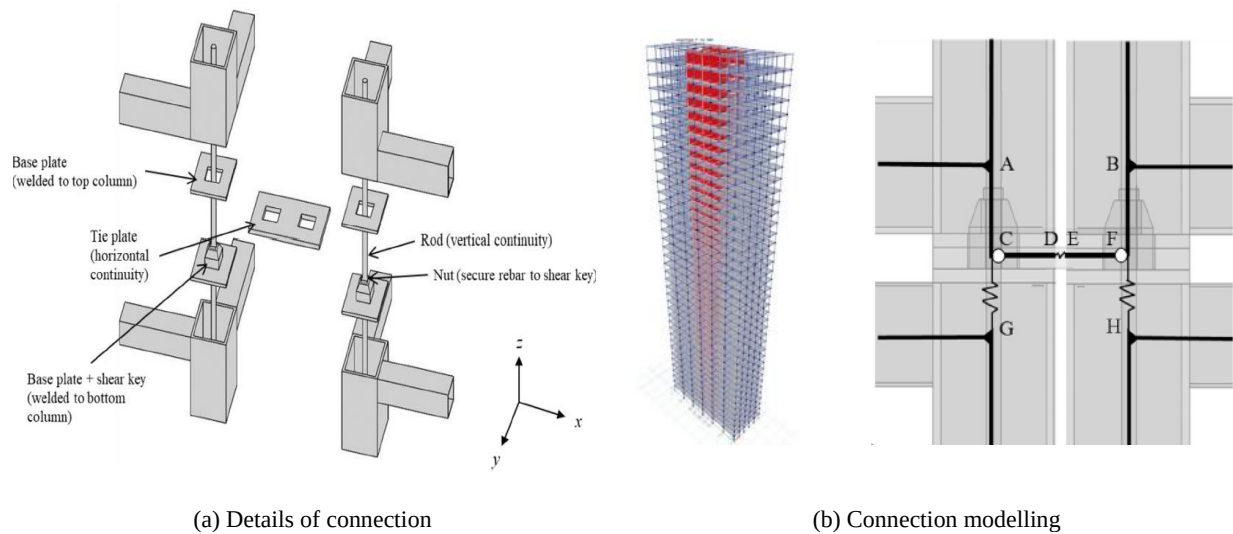
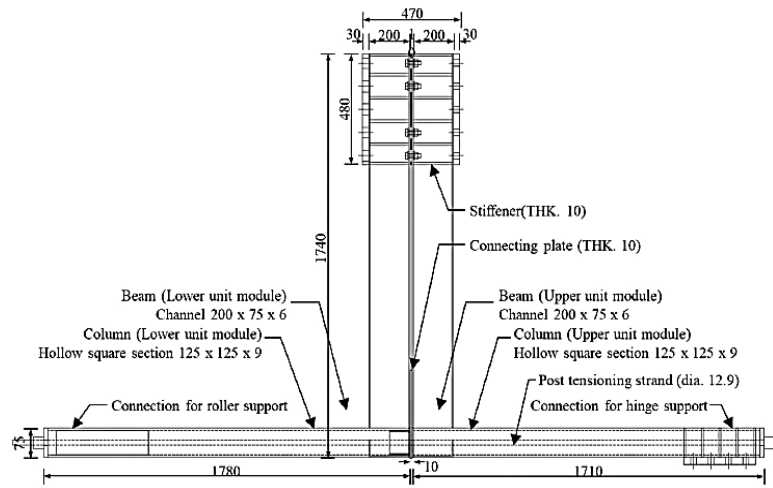


Figure 2.21: Post-tensioned connection using rod with base plate + shear key by Chua et al. (2020)

Shin et al. [76] (2022) suggested another post-tensioned inter-module connection consisting of a strand and a shear key (see Figure 2.22). The strand is positioned inside an HSS column and is anchored against end plates located at column ends. When an upper module is stacked on a lower module, the strand is installed, and it goes through the column full height. Then, the strand is post-tensioned using a hydraulic jack to connect the modules vertically. Several modules may be connected vertically at once by establishing anchorages at the roof and base plate of modular buildings, resulting in an efficient and faster assembly process. The full-scale exterior C beam-HSS column joints inclusive the connections were tested to evaluate their seismic performance and were compared with the bolted joint specimens.

The test results indicate that the post-tensioned joints performed similarly to the bolted joints, and they were classified as semi-rigid and partial-strength joints according to Eurocode 3.



(a) Details of connection



(b) Test setup for exterior joint

Figure 2.22: Post-tensioned connection using strand by Shin et al. (2022)

Chapter 3

Innovative Preloaded Inter-module Connection for Steel-framed Modular Buildings

3.1. Introduction

In recent years, many types of inter-module connections (e.g. welded, bolted, post-tensioned and mechanical connections) have been developed to improve the constructability and performance of steel-framed modular buildings. In this chapter, based on an in-depth analysis of the advantages and disadvantages of existing inter-module connections, an innovative post-tensioned type connection (referred to as the 'AJ' connection) is proposed. This connection enables easy assembly and disassembly between modules on site, maximising the benefits of modular construction. The shape of the connection, the assembly process of the steel-framed modular building using the connection, and the preload mechanism on the connection are described in this chapter.

When the multi-storey modular building assembled by the AJ connection is subjected to lateral force, it has two possible types of failure at the connection: gap-opening (the column separates by uplift at the connection) and slip. A minor gap-opening and slip would affect the serviceability of the building. However, if the deformations are large (particularly the gap-opening), they can significantly affect the overall stability of the building by reducing the stiffness of the entire frame, which causes additional second-order moments, often referred to as P- Δ effect. Therefore, they must be prevented by selecting an appropriate amount of preload at the connection. This chapter describes the impact of the gap-opening on the beam-to-column joint behaviour and the determination process for the demand preload required to prevent the gap-opening and slip at the connection.

In addition, in order to apply the AJ connection to multi-storey steel-framed modular buildings in seismic areas, this chapter presents design guidelines for moment-resisting frames to exhibit strong column-weak beam type behaviour for the global plastic failure mode under seismic loading, based on the design criteria and detailing rules of Eurocode 1998-1.

3.2. AJ Connection for Steel-framed Modular Buildings

3.2.1. Background of Development of the Innovative Preloaded Connection

In recent years, as inter-module connections for steel-framed modular buildings, bolted connections are preferred because it is an easier and cheaper on-site method than welded connections and provides speedy construction. As presented in Table 3.1 (2), various bolted connections have been developed. The bolts are installed at columns (see Table 3.1 (b)-(i)) or beams (see Table 3.1 (j)-(n)) to connecting modules. Bolted connections generally consist of a connecting plate or a cast plug-in device with multiple bolts. Although the bolted connection is convenient, the modules cannot be manufactured as complete finished products because external access space is required to install the bolts. Since the external access space is usually made in the floor slab or ceiling of the modules, extra finishing work is required on site after installing the bolts, increasing construction time and costs. In particular, connections joining eight modules are challenging to access for installing the bolts (see Figure 3.3). In order to resolve this difficulty, connections have been proposed to use a plug-in device for the columns and install the bolts through the beams (see Table 3.1 (j)-(l)). However, the shape of the connections is complex due to the many bolts installed, making it difficult to assemble and disassemble modules on site.

In order to overcome the challenges of using bolted connections, two types of connections have been developed as alternatives: the mechanical connection and the post-tensioned connection. In the case of the mechanical connection (see Table 3.1 (3)), two modules are vertically connected by simply inserting columns in the upper module into a locking device mounted on the columns of the lower module, making the assembling process of the modules fast and convenient. However, disassembling for reusing the modules is difficult, and the manufacturing cost of the mechanical device for the inter-module connection is higher than in ordinary bolted connections. In the case of post-tensioned connections (see Table 3.1 (4)), two modules are vertically connected by tensioned tie rods passing through the columns from the upper to lower columns in the modules (see Table 3.1 (r), (t) and (v)). In this type of connection, a plug-in device such as the steel box or the plate welded with a shear key is inserted into the columns to resist shear force between columns, and preload is applied to the tied rod fitted inside the columns using a hydraulic jack on site. The preload connection without tie rods have also been suggested, as shown in Table 3.1 (u). In this case, preload is applied to the rod within the columns by using a torque wrench or an impact wrench through access holes in the surface of the columns. However, the loss of column cross-section is undesirable particularly for seismic design because the column is an essential member requiring high performance. As presented in Table 3.1 (s), a composite connection has been proposed featuring a high-strength compact size column and connector modules securely fastened in the vertical direction. It is made by pouring concrete into the column where prestressed steel bars are

installed. However, this composite connection slows down the construction time due to site work for concrete, and is impossible to disassemble for reuse.

Overall, a number of inter-module connections have been researched and developed over the last several years. Each of the suggested inter-module connections has its own advantages, but it is evident that improved inter-module connections are necessary. In this study, an innovative connection of the post-tensioned type is proposed based on an in-depth analysis of the advantages and disadvantages of the existing inter-module connections. This connection enables easy assembly and disassembly between modules on site, maximising the benefits of modular construction (i.e. fast construction and reuse of modules). Also, the connection can be applied to mid- and high-rise steel-framed modular buildings in seismic areas. Section 3.2.2 describes the proposed connection in detail.

Table 3.1: Various types of inter-module connections for steel-framed modular buildings

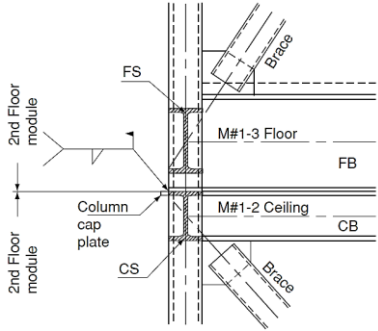
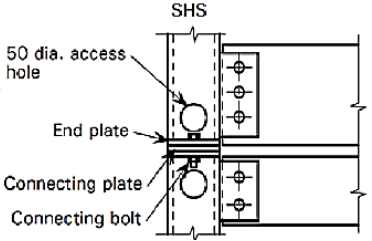
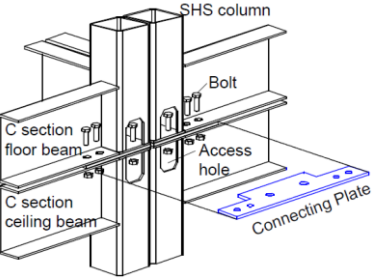
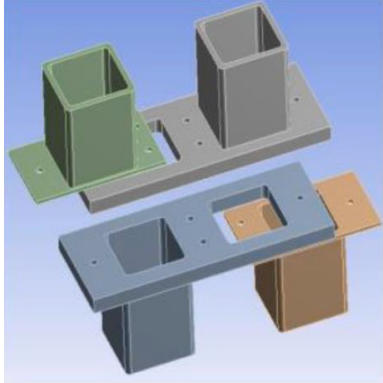
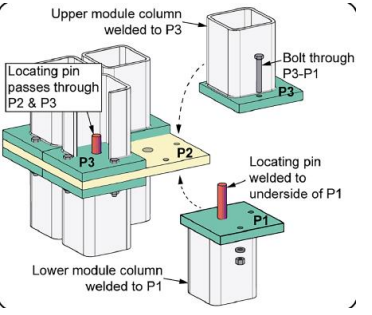
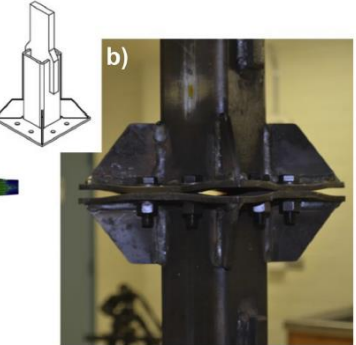
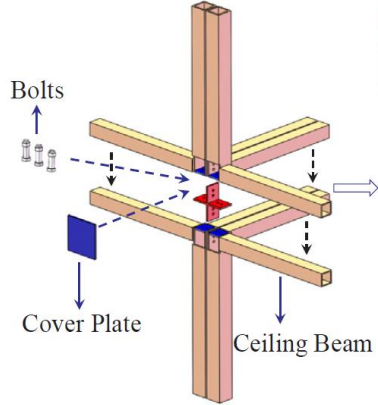
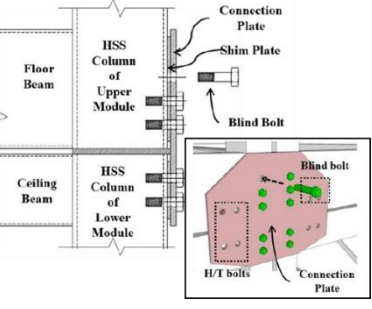
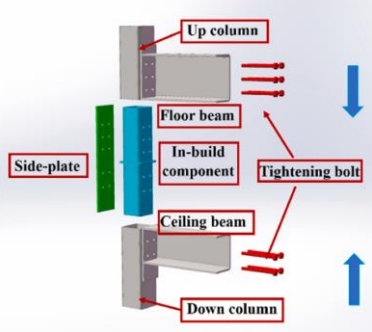
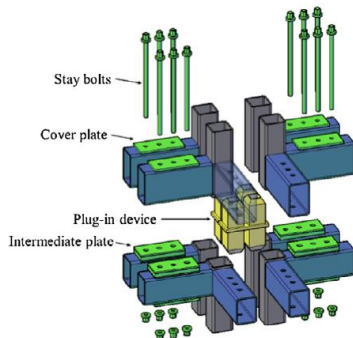
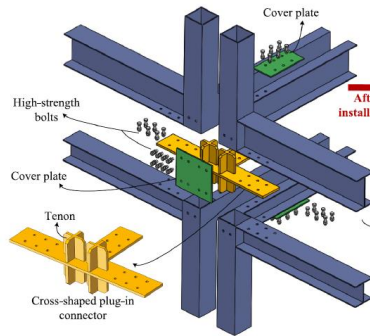
(1) Welded connection	(2) Bolted connection	
 (a) Fully welded connection (Annan et al. [2, 36, 77])	 (b) Bolted connection with access hole (Lawson et al. [78])	 (c) Bolted connection with access hole (Lee et al. [79])
(2) Bolted connection		
 (d) Bolted connection with endplate (Gunawardena [54])	 (e) New interlocking inter-module connection (Lacey et al. [80, 81])	 (f) Steel bolted flanged connection (Ding et al. [82])
 (g) Bolted connection with cruciform gusset plate (Deng et al. [58, 59])	 (h) Blind-bolted connection (Cho et al. [57])	 (i) Column bolted connection (Ma et al. [83])

Table 3.1 (cont.): Various types of inter-module connections for steel-framed modular buildings

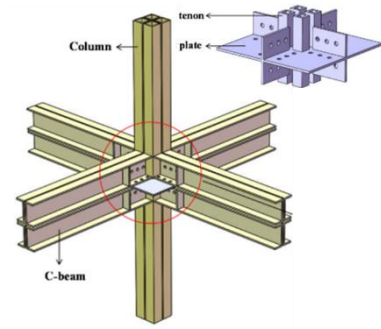
(2) Bolted connection



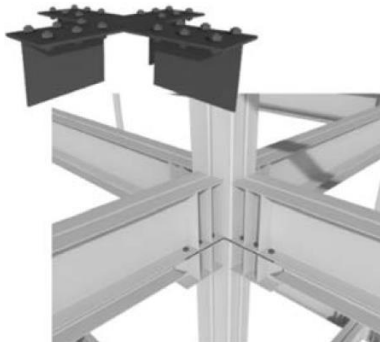
(j) Beam-to-beam inter-module connection with plug-in device (Chen et al. [3, 60])



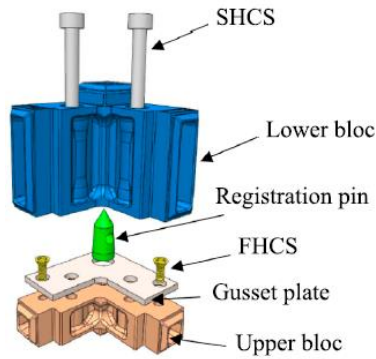
(k) Inter-module connection with bolted cover plates (Zhang et al. [4])



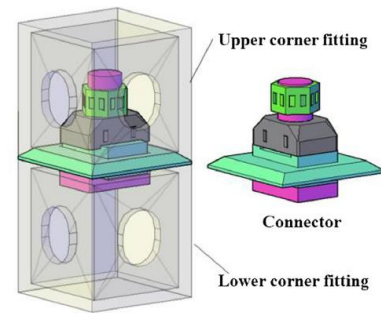
(l) Bolted connection with socket-shaped tenons and cruciform plate (Deng et al. [84])



(m) Bolted steel bracket joint (Lee et al. [55, 56])

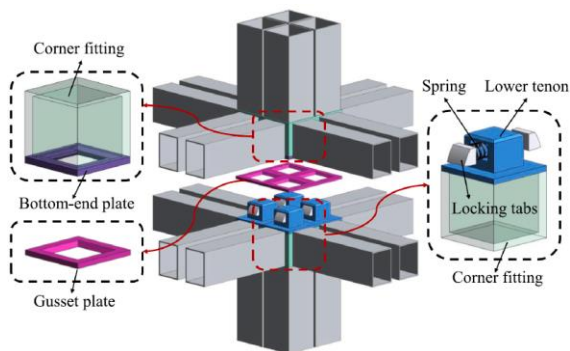


(n) VectorBloc connection (Dhanapal et al. [63-65])

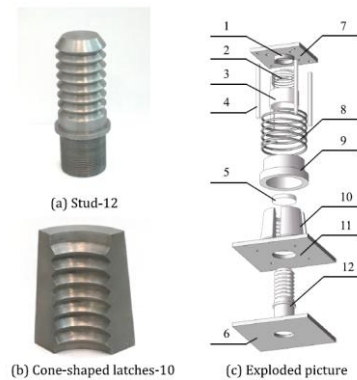


(o) Rotary inter-module connection (Chen et al. [66-70])

(3) Mechanical connection



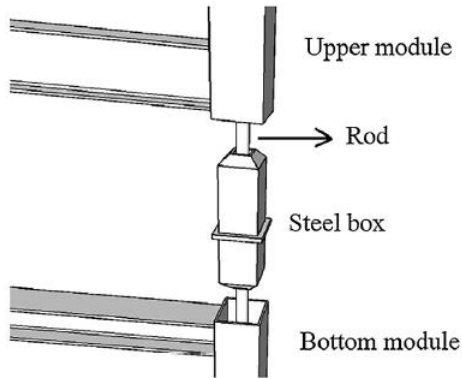
(p) Self-locking inter-module connection (Chen et al. [5])



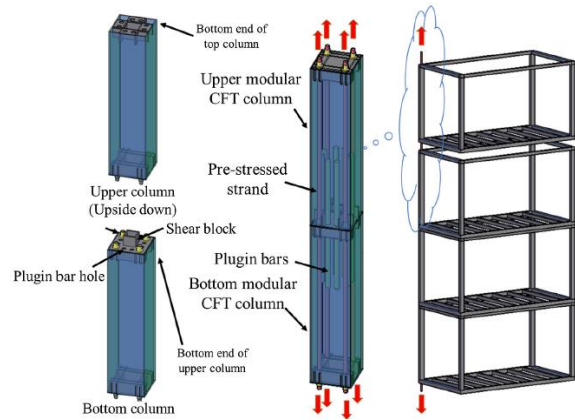
(q) Plug-in self-lock connection (Dai et al. [71])

Table 3.1 (cont.): Various types of inter-module connections for steel-framed modular buildings

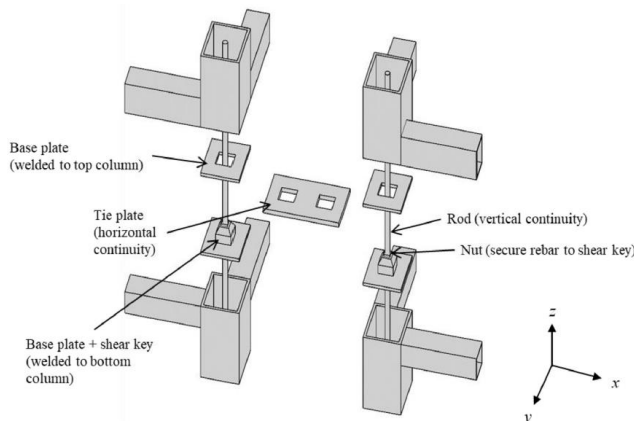
(4) Post-tensioned connection



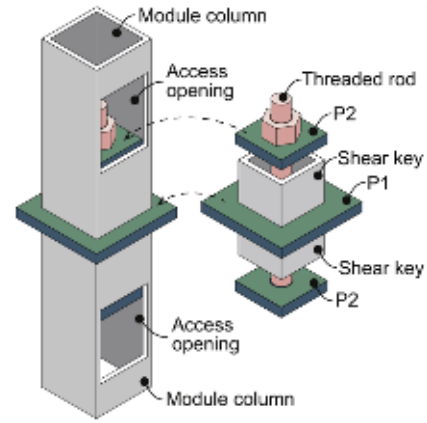
(r) Vertical post-tensioned connection
(Sanches et al. [6])



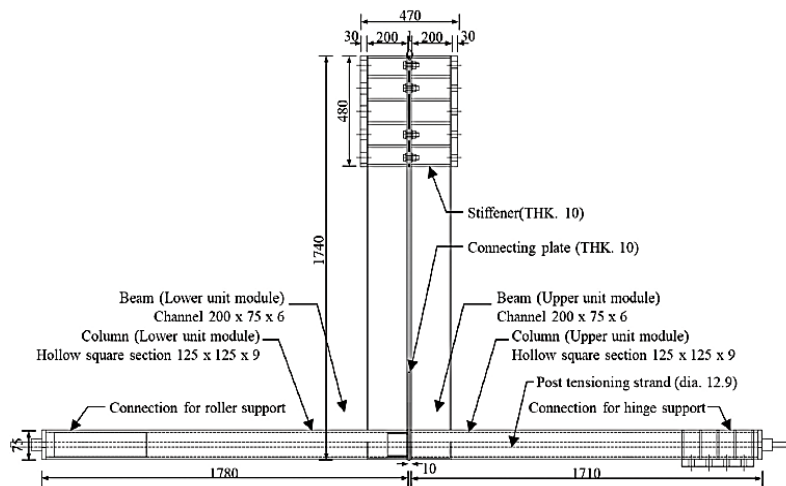
(s) Pretension connection
(Chen et al. [72, 73])



(t) Rod connection with shear key
(Chua et al. [75])



(u) Post-tensioned inter-module connection
(Lacey et al. [7, 74])



(v) Post-tensioned connection using strand
(Shin et al. [76])

3.2.2. Innovative Preloaded Inter-module Connection (AJ Connection)

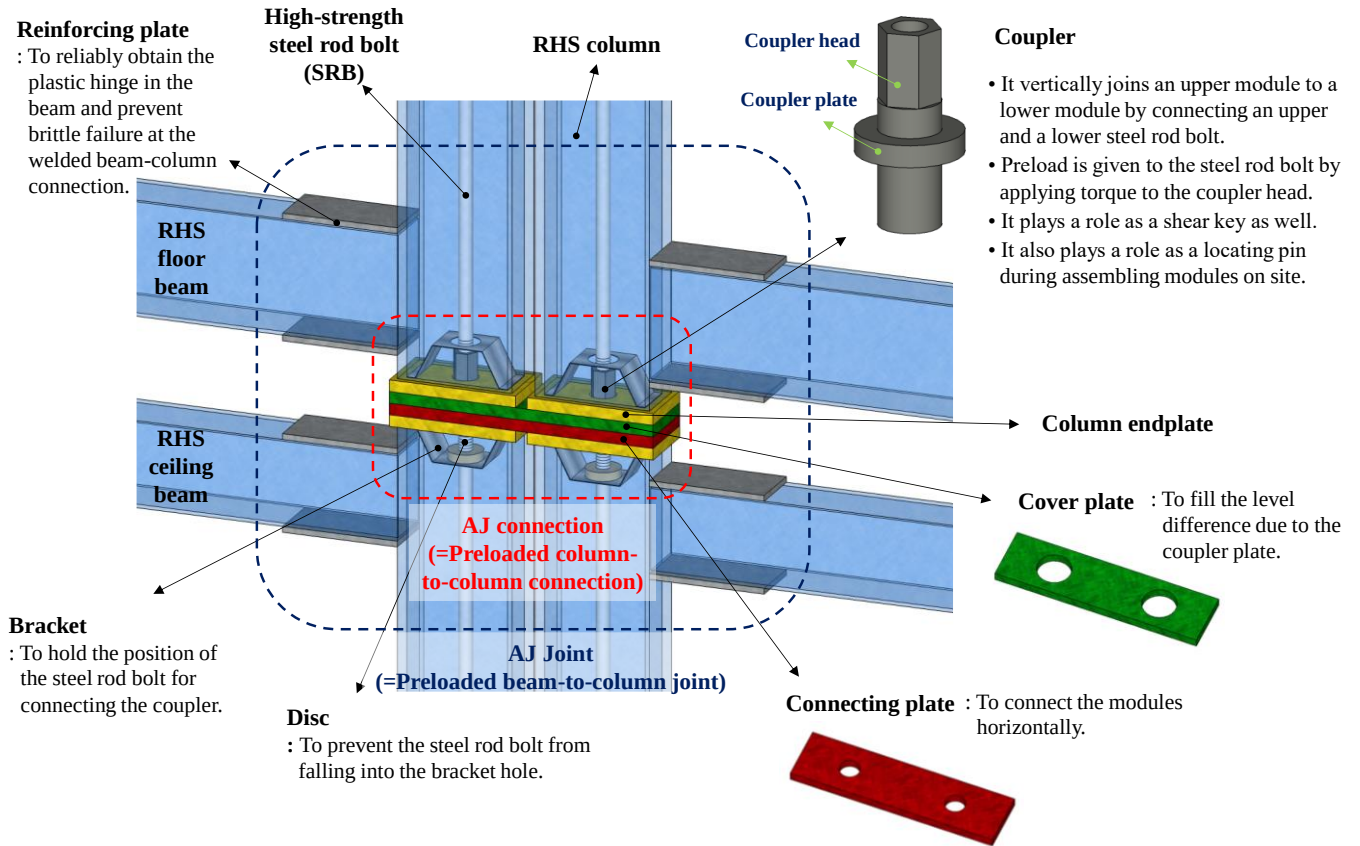


Figure 3.1: Configuration of the interior preloaded beam-to-column joint with the AJ connection

The innovative post-tensioned type connection proposed in this study is a vertical preload column-to-column connection between modules for a steel framed-modular building (referred to as the '**AJ connection** or **preload connection**'). The AJ connection was jointly researched and developed by the University of Sydney in Australia and the Prefab-Structure Lab of Ajou University in South Korea. Currently, this connection has been patented in South Korea. Figure 3.1 presents the configuration of the interior joint with the AJ connection. It connects the columns in the upper and lower modules vertically by preload. The preload is applied using the developed coupler and the high-tensile steel rod bolts (SRB) fitted inside the columns. The left and right modules are connected horizontally through the connecting plate. As shown in Figure 3.2, the SRBs are inserted into the columns in the module during the manufacturing process in a factory. The shape of the connection varies according to the location in the modular building and number of connected columns, as presented in Figure 3.3. The details of the assembly process for the modular building using the AJ connection are shown in Figure 3.4. The AJ connection enables easy assembly and disassembly between modules on site, maximising the benefits of 3D volumetric modular construction (e.g. fast construction and reuse of modules). The advantages of the AJ connection are described in detail below in comparison to the existing connections.

■ Faster construction on site

Bolted connections require external access space (e.g. making a hole in a floor slab or ceiling adjoining modules for installing bolts to connect modules on site), which does not allow the modules to be produced as complete finished products in a factory. Therefore, after the connecting work is completed, extra finishing works must be carried out on site. However, the AJ connection does not need any such external access space because its installation is conducted on the top of the modules. So, the modules can be manufactured as complete finished products in the factory, and no additional finishing works is required. As a result, the quality of modules and construction speed on site can be improved.

■ Improved constructability

Compared to the existing bolted connections having complex shapes due to multiple bolts and requiring a cast plug-in device, the AJ connection has a much simpler shape and installation process for connecting the modules by using one or two couplers and the SRBs per column. In addition, preload can be applied by a torque wrench or an impact wrench widely used for ordinary high-tensile strength bolts, unlike other existing post-tensioned connections requiring a hydraulic jack on site for applying preload.

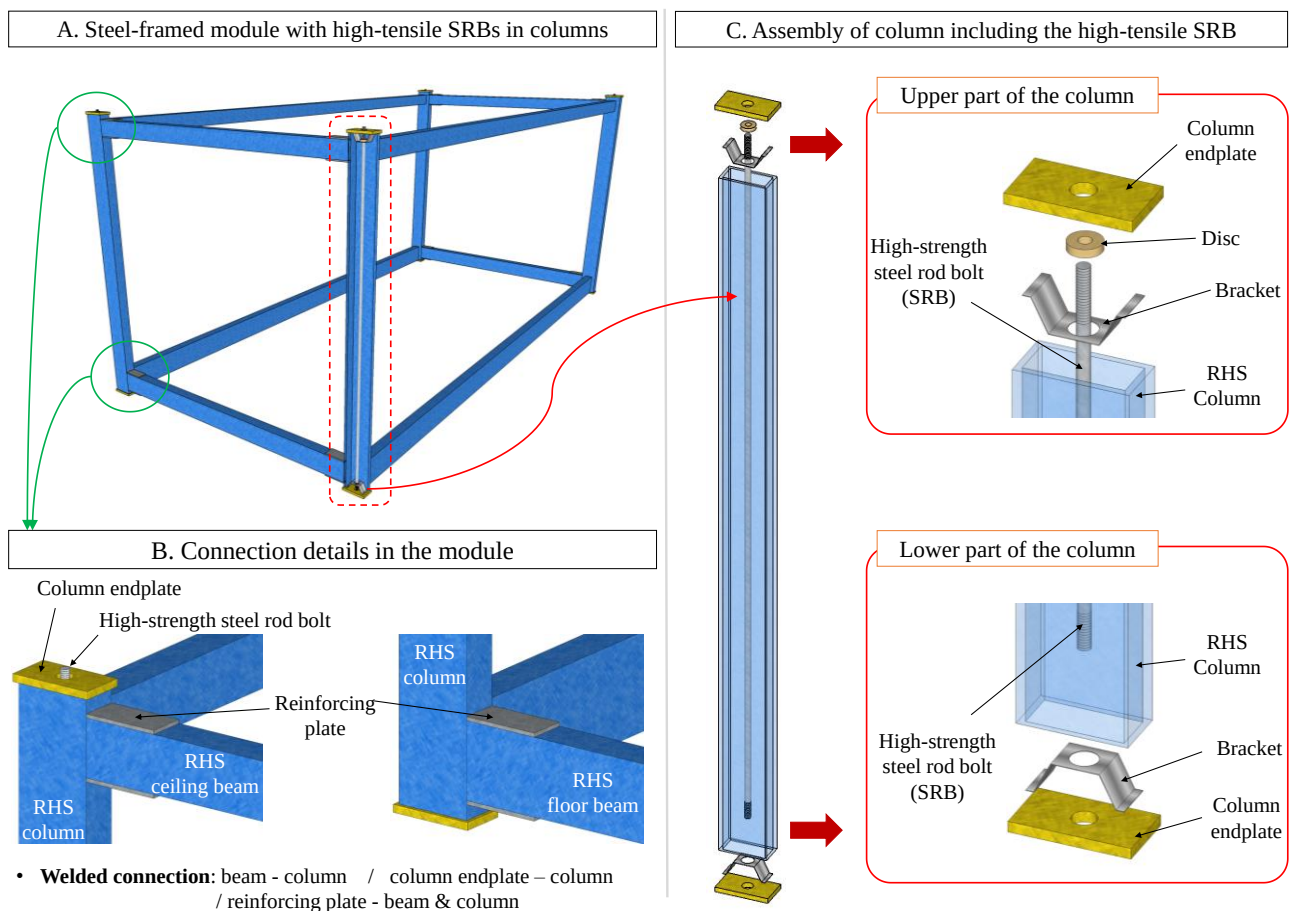


Figure 3.2: Single steel-framed module with high-tensile steel rod bolts installed within columns

■ **Improved structural performance**

Some bolted connections require an access hole (e.g. mouse hole), which is made by drilling through the column surface, to install the bolts (see Table 3.1 (b), (c), and (u)). It means that the overall resistance of the column is reduced due to the loss of cross-sectional area. In particular, columns with access holes are undesirable in seismic structural design because the access holes are usually located near the panel zone, rendering the columns vulnerable to shear failure in the panel zone under earthquake loading. On the other hand, the AJ connection requires no access holes, which allows the full cross-sectional capacity of the column to be utilised.

■ **Improved safety for workers on site**

For some bolted connections, when connecting the outermost modules, bolts are installed from the exterior of the building. In such cases, temporary scaffolding needs to be installed for workers. Hence, in the case of multi-storey modular buildings, workers are exposed to the danger of a falling accident when installing bolts. Conversely, the AJ connection provides workers with a safer on-site working environment because all works for connecting modules are conducted on the top of the modules. Added safety is derived from applying preload by a torque wrench or an impact wrench, as opposed to maneuvering a heavy hydraulic jack.

■ **Improved reusability of modules by easy disassembly**

One of the significant advantages of modular construction is that the modular building can be reused by disassembling and reassembling modules. However, in the case of many existing bolted or mechanical connections, their complex shapes make it difficult or impossible to disassemble the modules. On the other hand, the AJ connection facilitates a convenient disassembling process of the modular building on site, increasing the reusability of the modules.

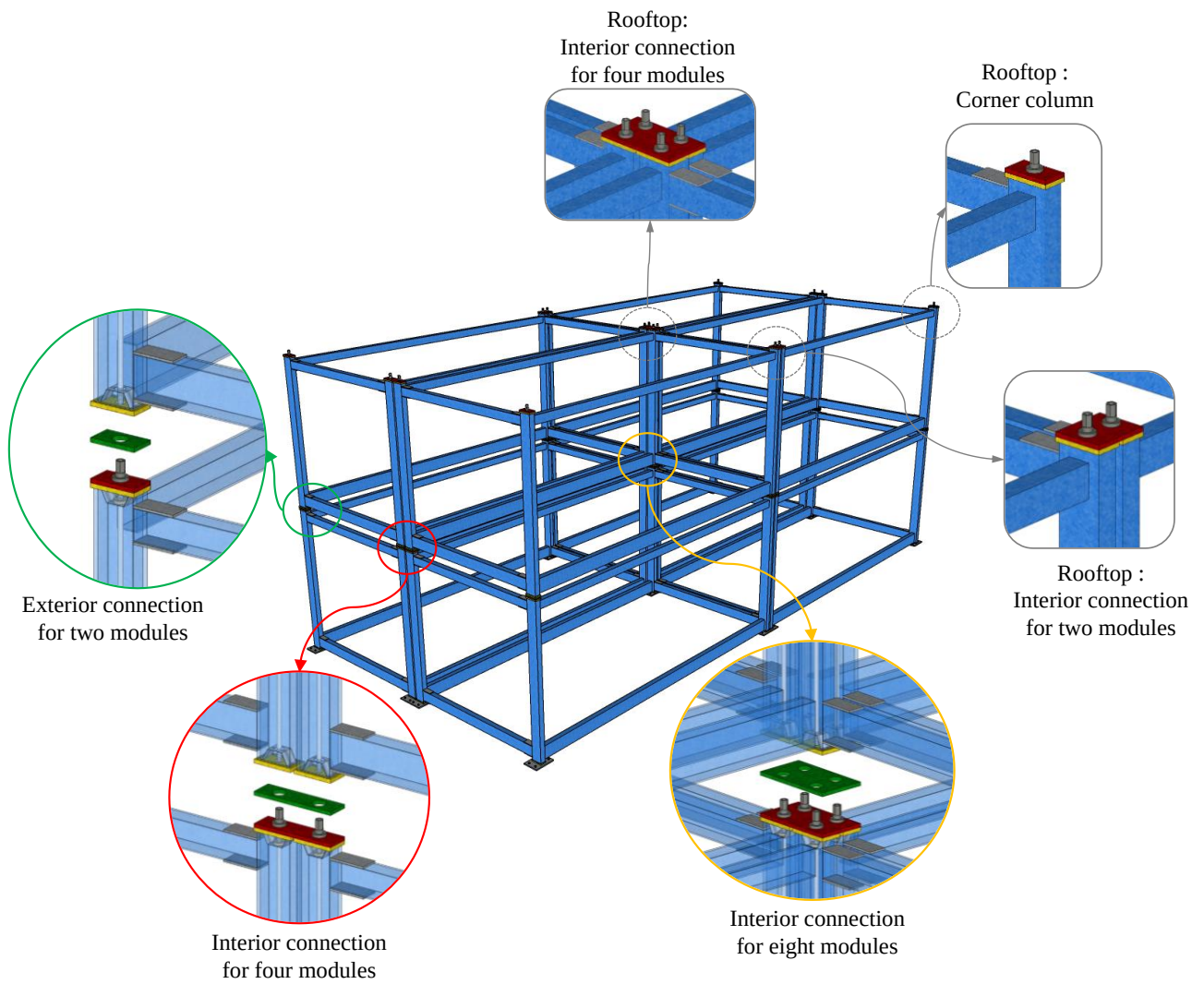


Figure 3.3: Shape of AJ connection in accordance with the locations in steel-framed modular building

3.2.3. Assembly Process of Steel-framed Modular Buildings with the AJ Connection

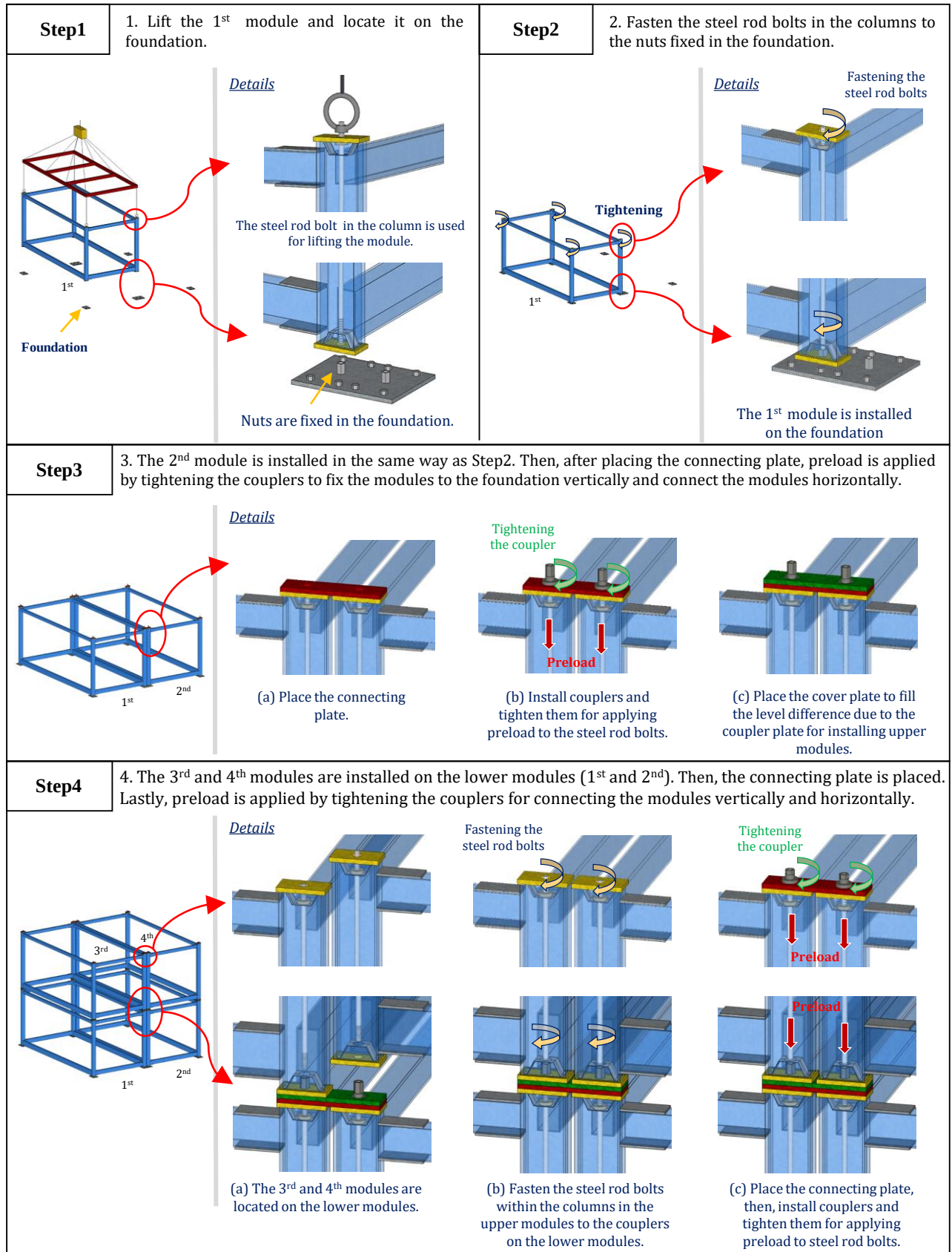


Figure 3.4: Assembly process of the steel-framed modular building with the AJ connection

3.3. Preload Mechanism and Clamping Force at the AJ connection

Figure 3.5 shows the preload (P) in the SRBs and the clamping forces (C) between the surfaces of plates in the AJ connection when an upper column and a lower column are assembled. The connection has three contact surfaces (S):

S1: Interface between Connecting plate and Endplate-1F

S2: Interface between Cover plate and Connecting plate

S3: Interface between Endplate-2F and Cover plate

The contact surfaces marked in red ($S1$) have a clamping force of C_1 induced by the preload P_1 applied to SRB-1F in column-1F. The contact surfaces marked in green ($S2$ and $S3$) have clamping forces of C_2 and C_3 , respectively, induced by the preload P_2 applied to SRB-2F in column-2F.

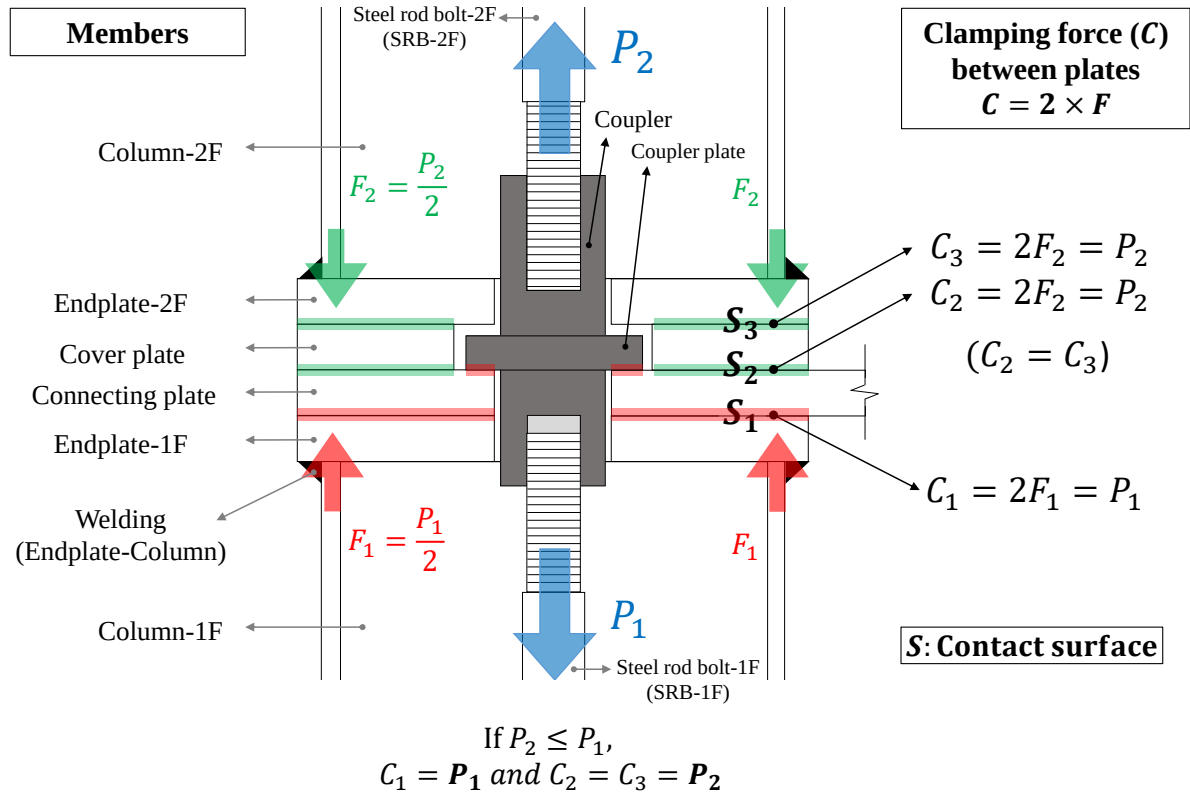
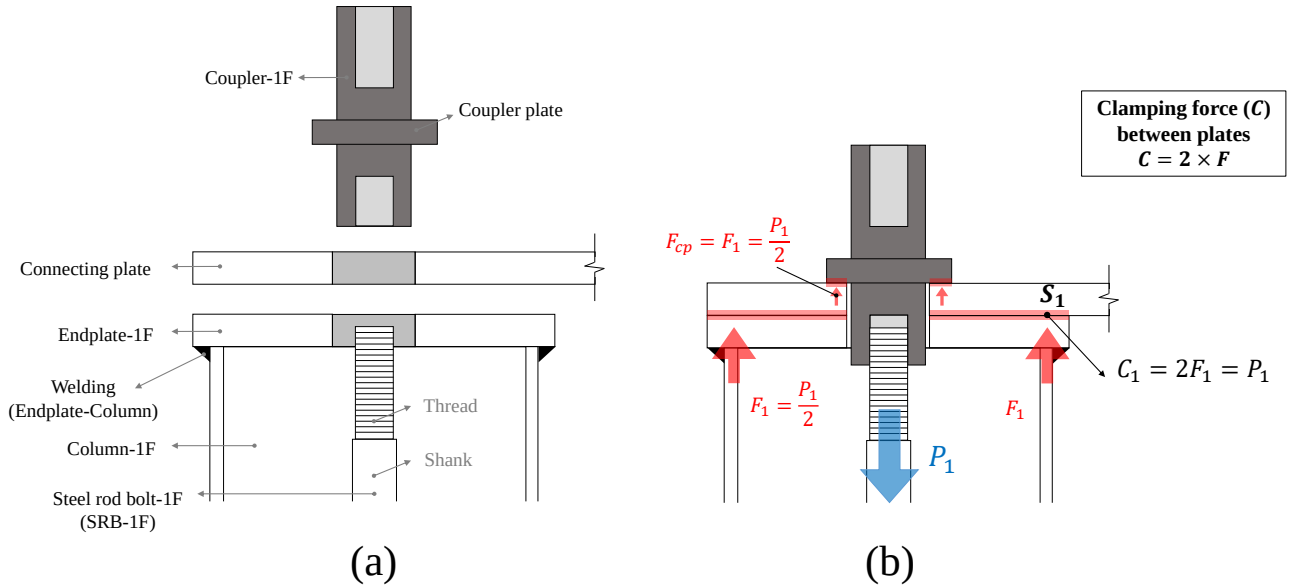


Figure 3.5: Cross-sectional view of preload column-to-column connection (= AJ connection):
 Preload (P) in the SRBs and the clamping forces (C) at contact surfaces (S)

Figure 3.6 presents the detailed process of generating clamping forces at the connection by applying preload to the SRBs, according to the assembling process of column-1F and column-2F. (As presented in Figure 3.7, since the preload is applied at the centreline of the column, it is relatively evenly distributed towards the outside of each plate.)

Step 1: For column-1F



Step 2: For column-2F

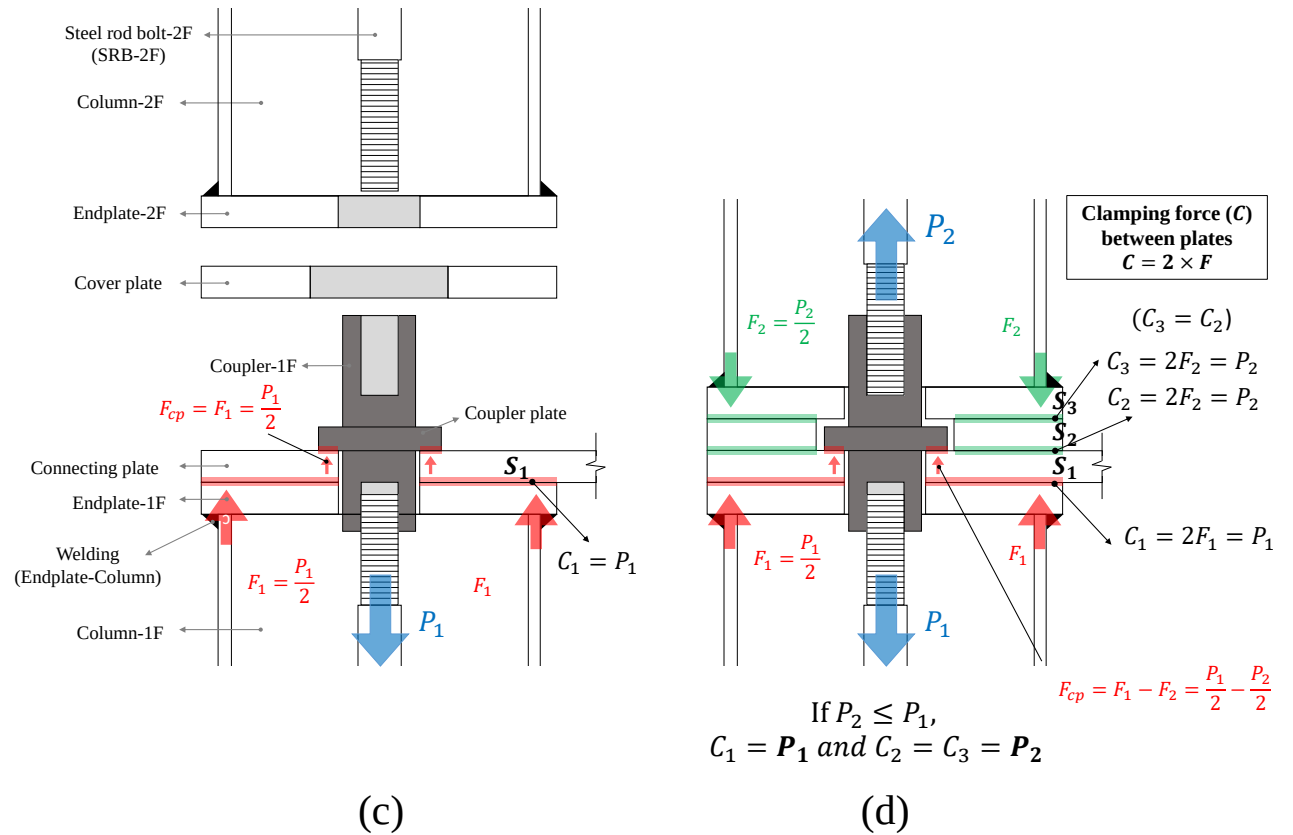


Figure 3.6: Detailed process of generating clamping forces (C) at the connection by preload (P) according to the assembling process of column-1F and column-2F (cross-sectional view of the AJ connection)

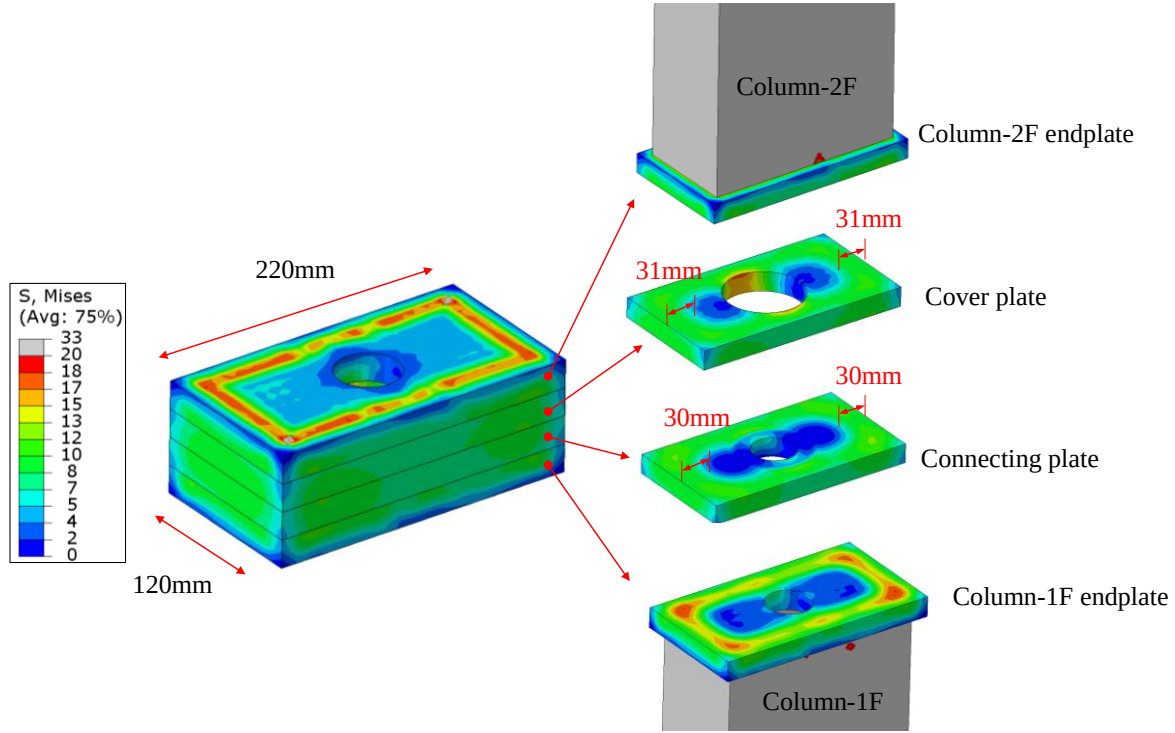


Figure 3.7: Von Mises stress distribution at the AJ connection after applying preload of 188kN in FE model (P-DB) in Section 4.4.8. (Note: The coupler and the SRBs are not shown in this Figure.)

As shown in Figure 3.6 (a) and (b), the connecting plate is placed on endplate-1F, and then the preload P_1 is applied to SRB-1F by tightening coupler-1F. As a result, the contact surface $S1$ has the clamping force of C_1 equal to P_1 . Then, as shown in Figure 3.6 (c) and (d), the cover plate is placed on the connecting plate to fill the level difference due to the coupler plate, and column-2F is placed on the cover plate. After SRB-2F is fastened to coupler-1F, coupler-2F is tightened to give preload to SRB-2F (refer to Figure 3.8 (b)), resulting in the contact surfaces $S2$ and $S3$ having the clamping forces of C_2 and C_3 , respectively, both equal to P_2 . While the preload P_2 is being applied to SRB-2F, coupler-1F is subjected to the pulling force of P_2 . Thereby, the clamping force ($2F_{cp}=P_1$) between the coupler plate and the connecting plate is reduced by P_2 , which means that the clamping force of C_1 ($=P_1$) at the contact surface $S1$ is also reduced to P_1-P_2 . However, a force of the same magnitude P_2 is applied simultaneously to the same surface from column-2F. As a result, the contact surface $S1$ maintains the initial clamping force of C_1 as follows: $C_1 = (P_1 - P_2) + P_2 = P_1$. (For more details, refer to Figure 3.9 showing free-body diagrams of the clamping forces of each contact surface in the connection.)

According to the assembling process of the upper and lower modules on-site, Figure 3.8 presents the location of the clamping forces generated by applying preload to the SRBs. As shown in Figure 3.8 (b), the preload P_1 in SRB-1F determines the clamping forces of C_1 and C_B . The preload P_2 in SRB-2F determines the clamping forces of C_2 , C_3 , and C_4 ,

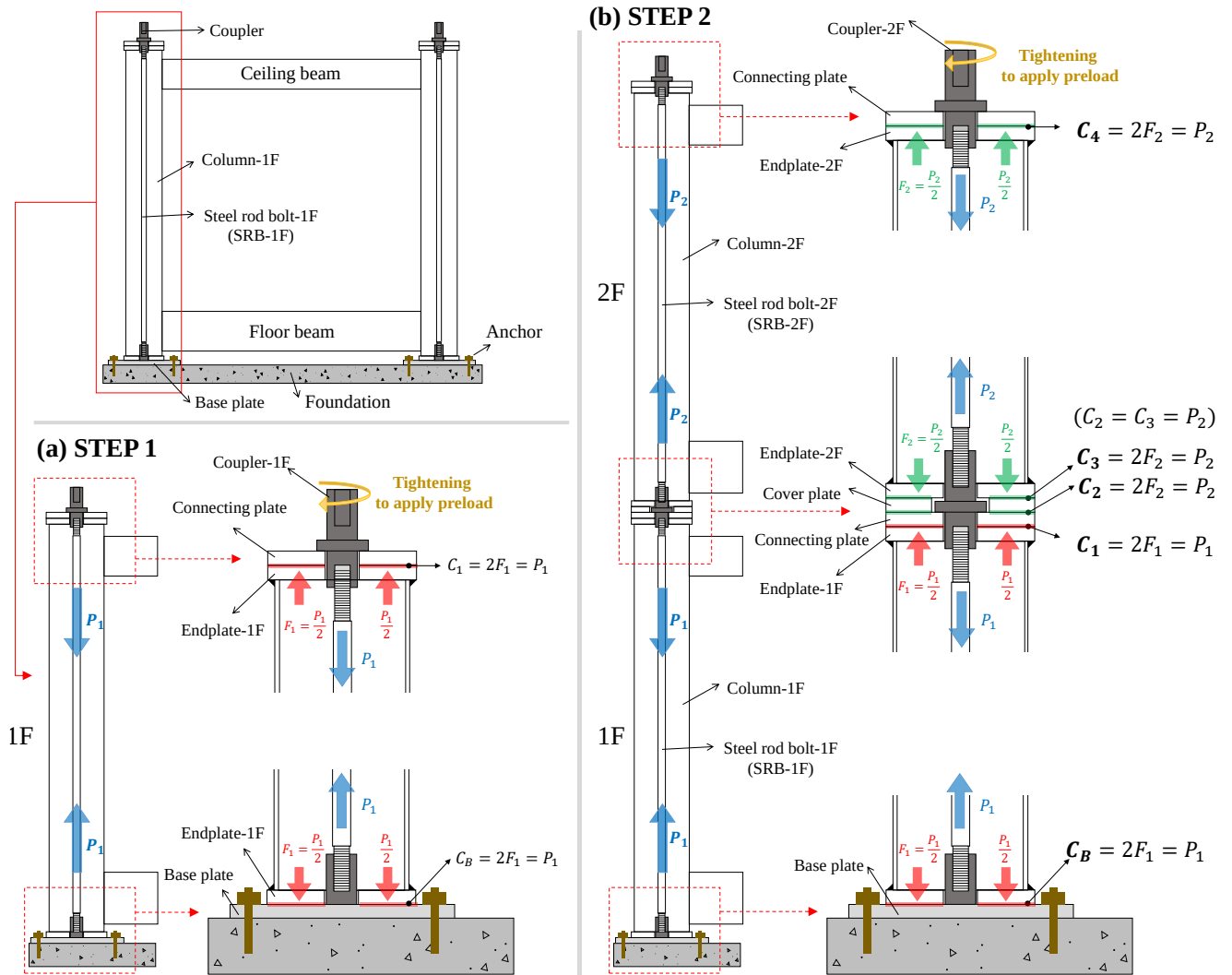


Figure 3.8: Clamping forces (C) at connection by preload (P) according to the assembling process of the upper and lower modules

The clamping forces (C_1 , C_2 , and C_3) at the contact surfaces can be changed depending on the difference in magnitude between preload P_1 in SRB-1F and preload P_2 in SRB-2F. Figure 3.9 and Figure 3.10 present free body diagrams for the clamping forces (C_1 , C_2 , and C_3) at the contact surfaces for the two possible cases of P_1 relative to P_2 :

- In case $P_1 \geq P_2$ (see Figure 3.9): $C_1 = P_1$, and $C_2 = C_3 = P_2$. In other words, when P_2 has been applied to SRB-2F, coupler-1F still remains in contact with the connecting plate (see Figure 3.9

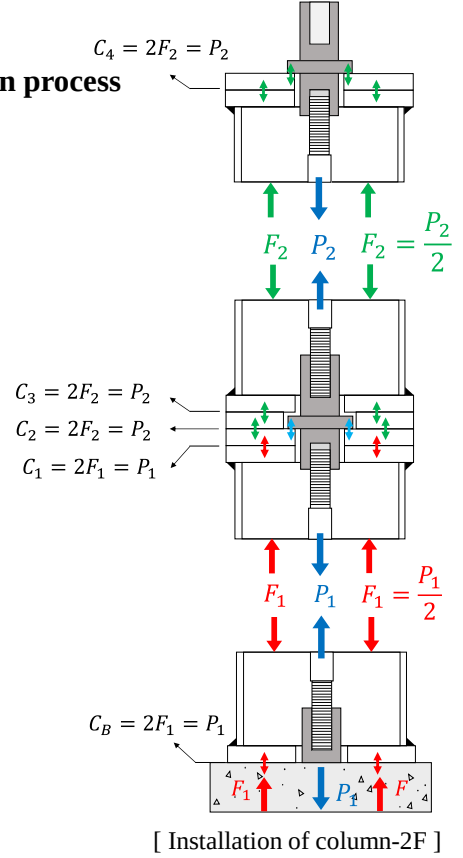
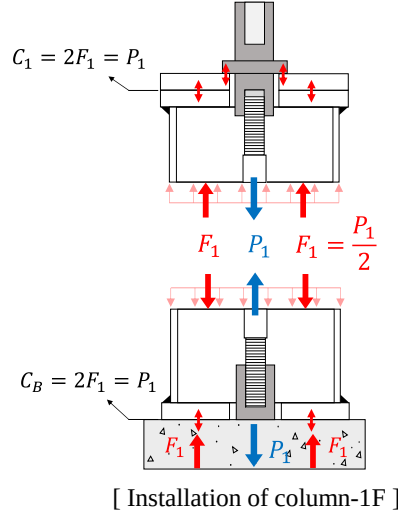
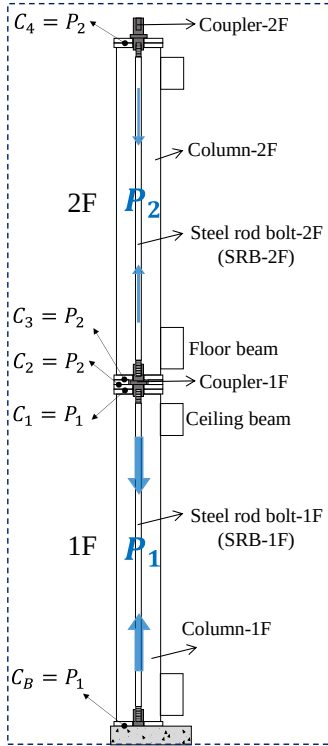
(2)). Conclusively, the preload P_1 in SRB-1F determines the clamping force of C_1 for column-1F at the connection. The preload P_2 in SRB-2F determines the clamping forces of C_2 and C_3 for column-2F at the connection. If $P_1 = P_2$, the clamping force at the contact surfaces are the same ($C_1 = C_2 = C_3$).

- In case $P_1 < P_2$ (see Figure 3.10): Until P_2 reaches P_1 , $C_1 = P_1$ and $C_2 = C_3 = P_2$, as mentioned above. However, when P_2 exceeds P_1 , coupler-1F is subjected to a larger upward pulling force of P_2 than downward pulling force P_1 , resulting in coupler plate-1F becoming unattached from the connecting plate (see Figure 3.10 (2)). From that moment, SRB-1F, coupler-1F and SRB-2F begin to act as one long bolt. When tightening coupler-2F, they are elongated altogether (see Figure 3.10 (1)). Therefore, all contact surfaces (C_B , C_1 , C_2 , C_3 and C_4) in the columns from the base plate to the top of column-2F have the same clamping force, P_2 . Note that it is here assumed the clearance between the top surface of the coupler plate and the surface of endplate-2F is sufficiently large that contact between the surfaces does not develop.

It is desirable to make the connection have a uniform clamping force for all the contact surfaces by applying the same amount of preload to SRB-1F (P_1) and SRB-2F (P_2). If preload in the lower column is applied smaller than expected, it is theoretically possible to make the connection have a uniform clamping force by applying a large preload to the upper column (as described in the case $P_1 < P_2$). However, this becomes the situation of applying preload to a 6m SRB consisting of SRB-1F, coupler-1F and SRB-2F (if SRB-1F and SRB-2F are each 3m long, respectively), for which accurate data is not available. It is unknown whether the preload applied to the 6m SRB will be as large as desired and what torque would be required. Therefore, it is recommended to apply the same amount of required preload to the upper and lower columns as precisely as possible to ensure a uniform clamping force is achieved.

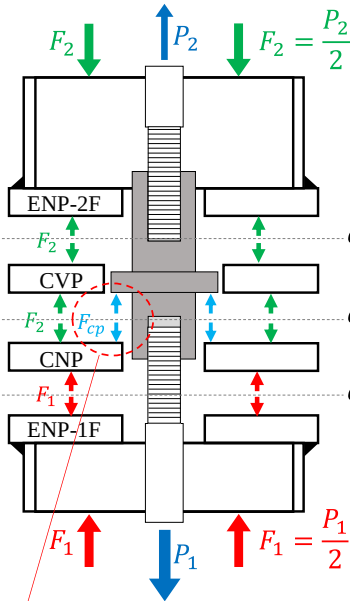
■ In case that preload $P_1 \geq P_2$

(1) Preload and clamping force according to the installation process



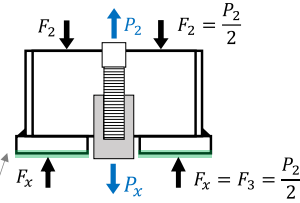
(2) Clamping force (C) at the connection

ENP-2F : Endplate-2F
CVP : Cover plate
CNP : Connecting plate
ENP-1F : Endplate-1F

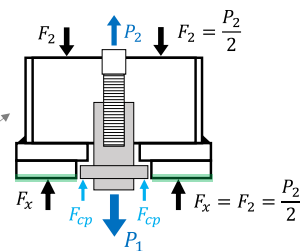


* The coupler and the connecting plate (CNP) are in contact until the preload, P_2 has been applied.

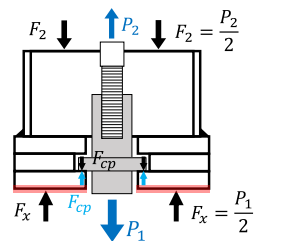
(1) At contact surface-3: C_3



(2) At contact surface-2: C_2



(3) At contact surface-1: C_1



$$\sum F_y = 0,$$

For a coupler and steel rod bolts

$$P_2 - P_x = 0 \rightarrow P_x = P_2$$

For a column and a plate

$$2F_x - 2F_2 = 2F_x - 2 \times \frac{P_2}{2} = 0 \rightarrow F_x = \frac{P_2}{2}$$

$$C_3 = 2F_x = 2 \times \frac{P_2}{2} = P_2$$

For a coupler and steel rod bolts

$$P_2 - P_1 + 2F_{cp} = 0 \quad F_{cp} = \frac{(P_1 - P_2)}{2}$$

For a column and a plate

$$2F_x - 2F_2 = 2F_x - 2 \times \frac{P_2}{2} = 0 \rightarrow F_x = \frac{P_2}{2}$$

$$C_2 = 2F_x = 2 \times \frac{P_2}{2} = P_2$$

For a coupler and steel rod bolts

$$P_2 - P_1 + 2F_{cp} = 0 \quad F_{cp} = \frac{(P_1 - P_2)}{2}$$

For a column and a plate

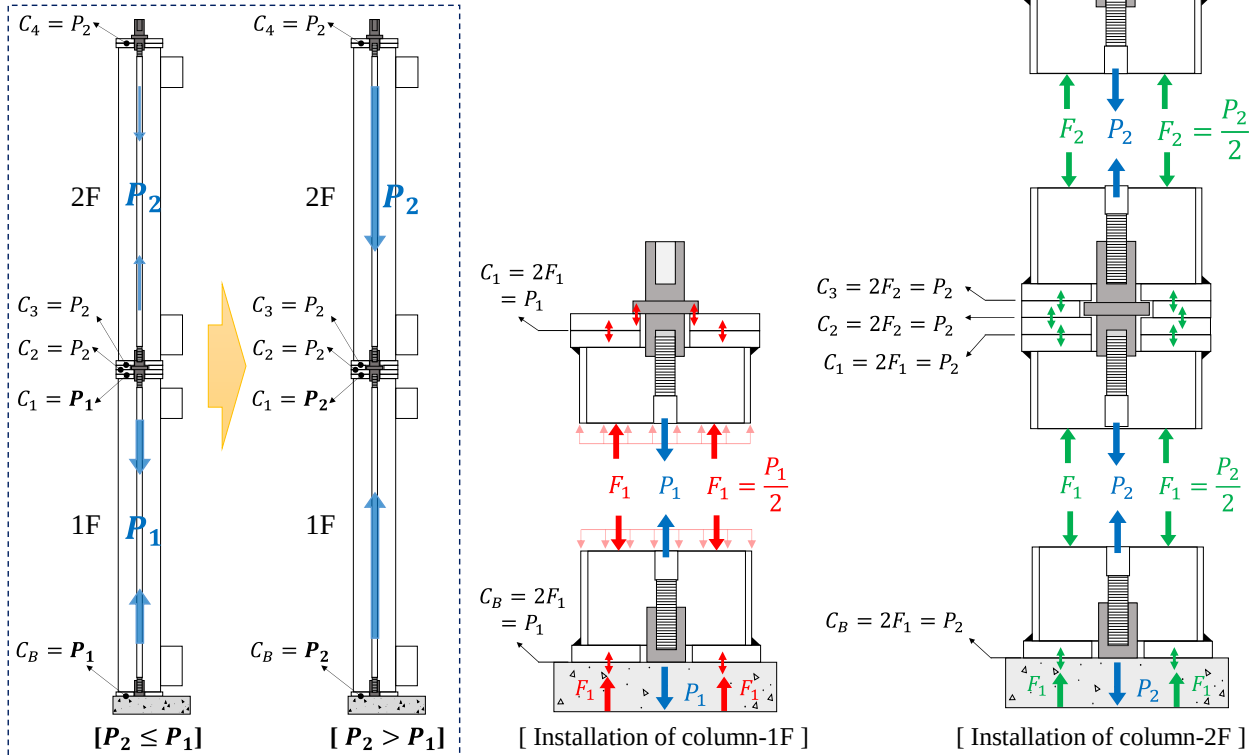
$$2F_x - 2F_2 - 2F_{cp} = 2 \left(F_x - \frac{P_2}{2} - \frac{(P_1 - P_2)}{2} \right) = 0 \rightarrow F_x = \frac{P_1}{2}$$

$$C_1 = 2F_x = 2 \times \frac{P_1}{2} = P_1$$

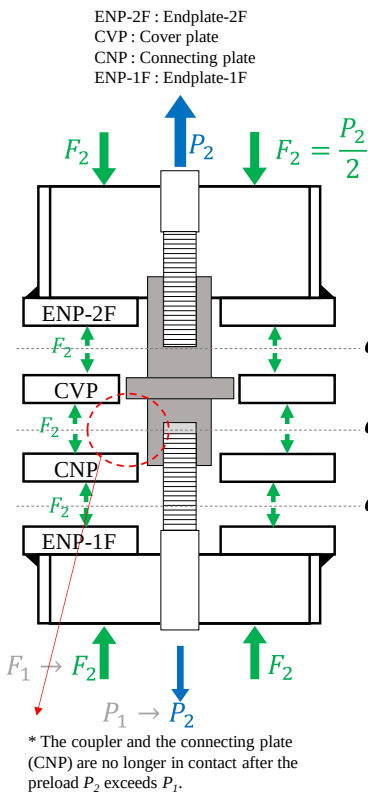
Figure 3.9: Free body diagrams for clamping forces (C) at the connection for the preload case of $P_1 \geq P_2$

■ In case that preload $P_1 < P_2$

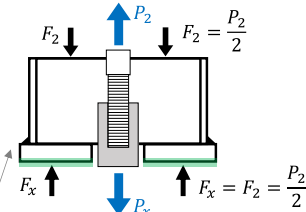
(1) Preload and clamping force according to the installation process



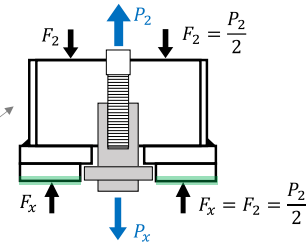
(2) Clamping force (C) at the connection



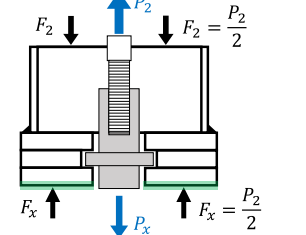
(1) At contact surface-3: C_3



(2) At contact surface-2: C_2



(3) At contact surface-1: C_1



$$\sum F_y = 0,$$

For a coupler and steel rod bolts

$$P_2 - P_x = 0 \rightarrow P_x = P_2$$

For a column and a plate

$$2F_x - 2F_2 = 2F_x - 2 \times \frac{P_2}{2} = 0 \rightarrow F_x = \frac{P_2}{2}$$

$$C_3 = 2F_x = 2 \times \frac{P_2}{2} = P_2$$

For a coupler and steel rod bolts

$$P_2 - P_x = 0 \rightarrow P_x = P_2$$

For a column and a plate

$$2F_x - 2F_2 = 2F_x - 2 \times \frac{P_2}{2} = 0 \rightarrow F_x = \frac{P_2}{2}$$

$$C_2 = 2F_x = 2 \times \frac{P_2}{2} = P_2$$

For a coupler and steel rod bolts

$$P_2 - P_x = 0 \rightarrow P_x = P_2$$

For a column and a plate

$$2F_x - 2F_2 = 2 \left(F_x - \frac{P_2}{2} \right) = 0 \rightarrow F_x = \frac{P_2}{2}$$

$$C_1 = 2F_x = 2 \times \frac{P_2}{2} = P_2$$

Figure 3.10: Free body diagrams for clamping forces (C) at the connection for the preload case of $P_1 < P_2$

3.4. Determination of Demand Preload for the AJ Connection

3.4.1. Failure Mode at the AJ Connection

The AJ connection aims to be a rigid column-to-column connection between modules, making the entire building frame composed of modules behave as a moment-resisting frame. Also, in the event of an earthquake, the connection should have stable behaviour while the moment-resisting frames develop ductile behaviour by forming plastic hinges in the beams of each module, as shown in Figure 3.11 (a). However, if preload is insufficient at the AJ connection, two types of failure are possible: slip and gap-opening. They affect the overall stability of the modular building, as shown in Figure 3.11 (b) and (c), respectively. The associated deformations result in a significant reduction of the stiffness of the entire frame and instability may occur. The effects of slip and gap-opening at the connection are described in detail in this section.

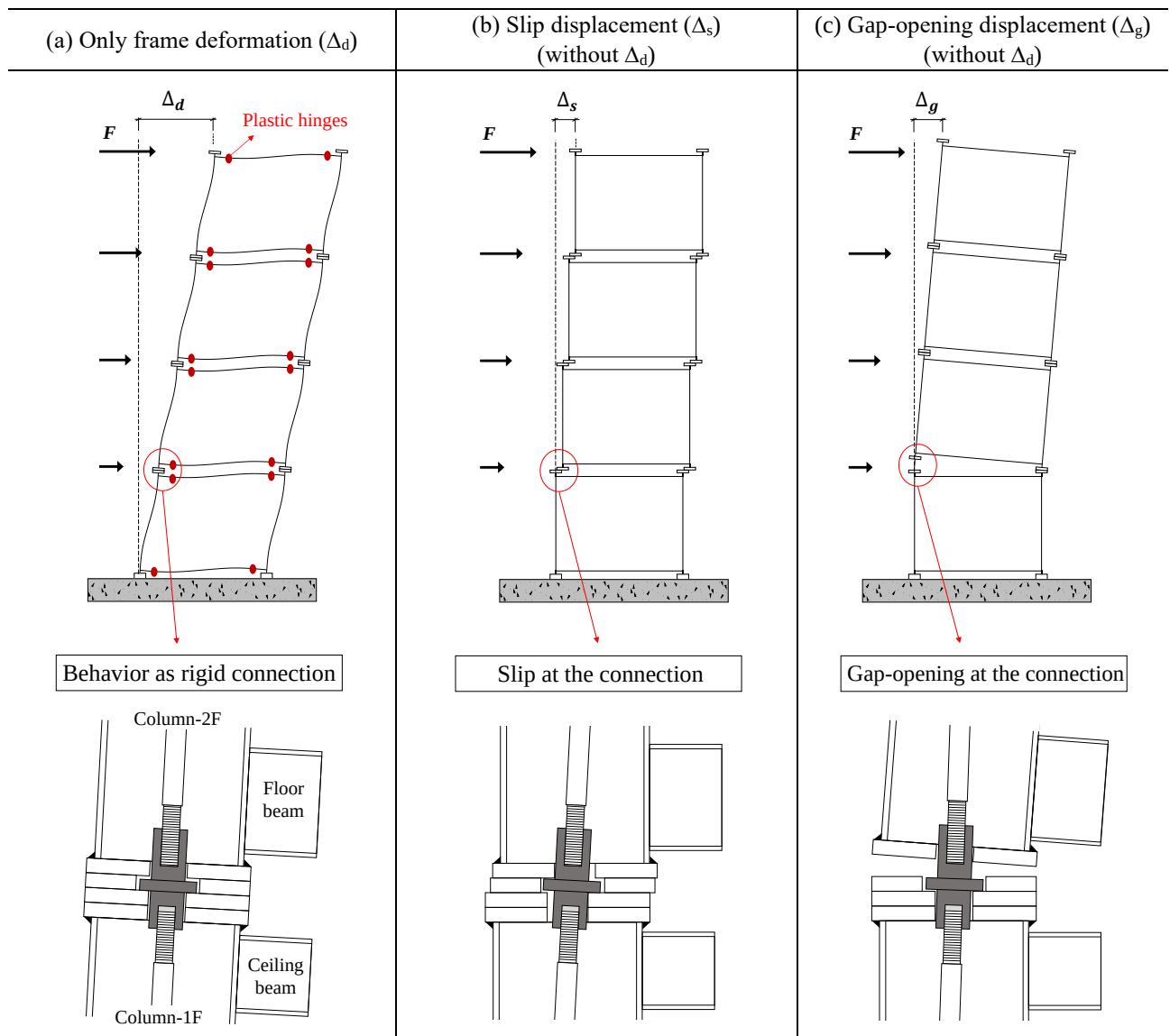


Figure 3.11: Deformed shapes of a multi-storey modular building subjected to a lateral load depending on the AJ connection behaviour

▪ Gap-opening

In case the tensile force acting on the column-to-column connection is larger than the clamping force induced by preloading, uplift of the column occurs, as shown in Figure 3.11 (c). In this study, this phenomenon is called gap-opening at the connection.

As the lateral force is applied to the modular building in Figure 3.12 (a), the columns and beams are simultaneously subjected to axial and shear forces. Depending on the dominant force between them, the gap-opening can be classified into two types. When the axial tensile force A_T acting on the column becomes dominant, the column separates as shown in Figure 3.12 (b). On the other hand, if the shear forces V acting on the columns and beams are the dominant forces, contact between connecting plates is retained as the gap-opening develops, as shown in Figure 3.12 (c).

As presented in Figure 3.11 (c), the gap-opening at the connection causes a large displacement at the top of the building, reducing the stiffness of an overall building significantly and severely affecting the overall stability of the modular building. In addition, SRBs within columns will be subjected to large tensile forces to resist the gap-opening at the connection. That makes the SRBs, which are the most critical members in the inter-module connection, exposed to potential yielding and fracture. Therefore, the gap-opening must be prevented at the connection.

▪ Slip

When installing a bolted connection to connect columns for assembling two modules on-site, the positions of bolt holes between the modules frequently do not fit correctly due to manufacturing tolerances from producing modules in a factory. Therefore, sufficient bolt-hole tolerance is generally required for easy installation of the bolted connection on-site. This also applies to the AJ connection in which a coupler is used at the connection instead of bolts. If slip occurs at the connection due to a lateral force, the large coupler-hole tolerance would cause a large slip displacement at the connection. The slip displacements at the connections of each storey would accumulate and result in a significantly large displacement at the top of the building with associated P- Δ effect, as shown in Figure 3.11 (b). Therefore, slip should be prevented or minimised as much as possible in multi-storey modular buildings.

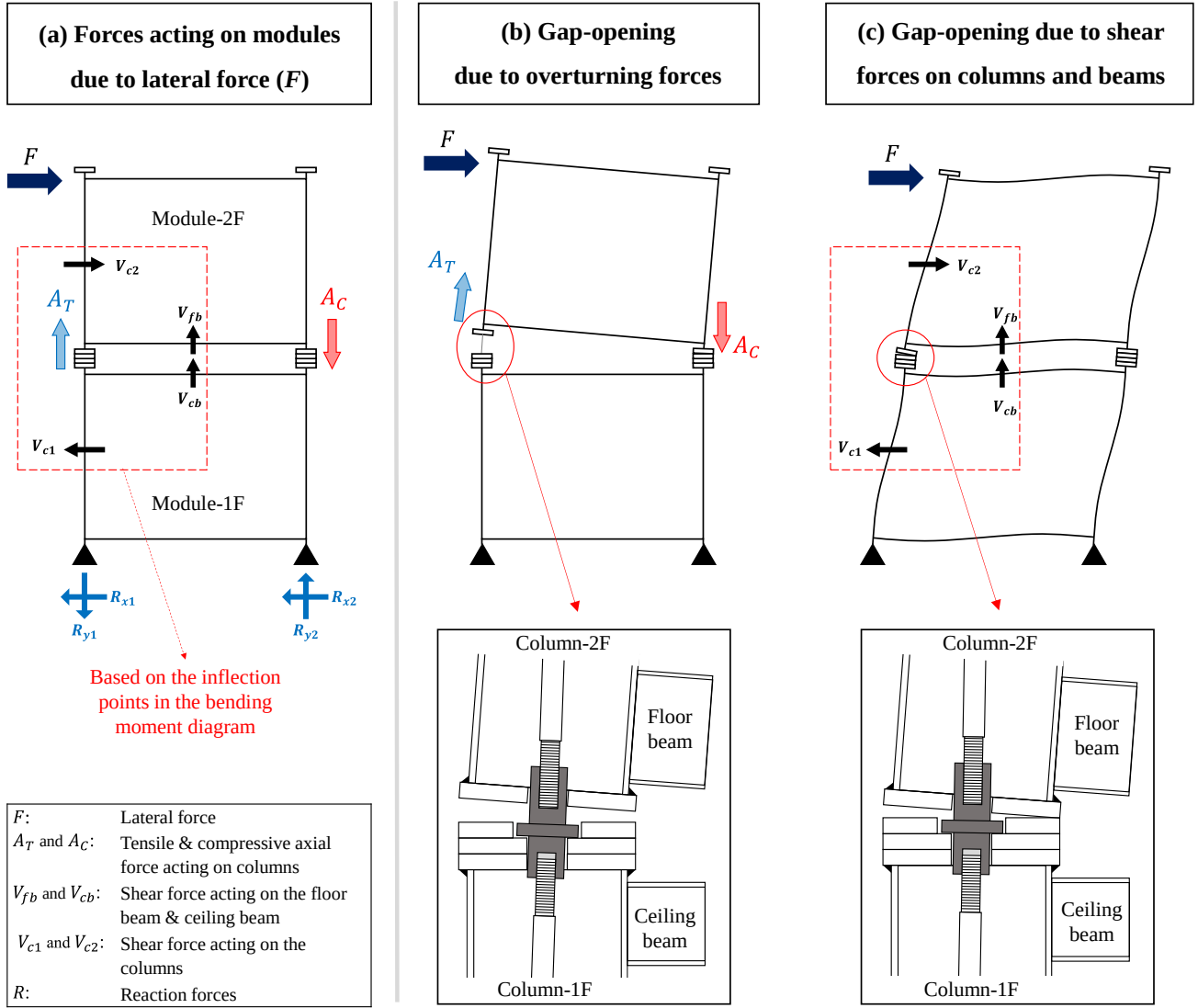


Figure 3.12: Two types of gap-opening at the AJ connection due to lateral force

The following sections describe the process for determining the demand preload to prevent gap-opening and slip at the AJ connection. The demand preload differs depending on the type of connection, i.e. bearing or slip-resistant connection. If the connection is designed as a bearing connection, the demand preload becomes the force P_g required to prevent gap-opening at the connection. On the other hand, for a slip-resistant connection, the larger preload of P_g and slip force P_s becomes the demand preload because both gap-opening and slip must be prevented. If the connection is applied to a mid- or high-rise modular building, the AJ connection is likely to be designed as a slip-resistant connection because the accumulated slip at the connection may have a large effect on the modular building. In contrast, for a low-rise building, the connection is likely to be designed as a bearing connection.

3.4.2. Study of Overturning and Stabilising Forces

3.4.2.1. Overturning Effect

The overturning of an object occurs when the overturning moment M_{ov} acting on the object is greater than the stabilising moment M_{st} . As presented in Figure 3.13, when the column that is not fixed on the foundation is subjected to the lateral force F , the force tries to rotate the column with a moment M_{ov} ($=F \cdot h$) at the edge of the column endplate, point O in Figure 3.13 (a). M_{ov} can be represented by a force-couple comprising an uplifting force T_F and a contact force C_F at the two ends of the column endplate. On the other hand, the self-weight W , which can be expressed as the resultant gravitational force G ($=Wb_e$) in Figure 3.13 (b), generates the stabilising moment in the opposite direction at point O . In this case, overturning occurs when:

$$M_{ov} = F \cdot h > M_{st} = \frac{Gb_e}{2} \quad (G = Wb_e) \quad (\text{Eq. 3.1})$$

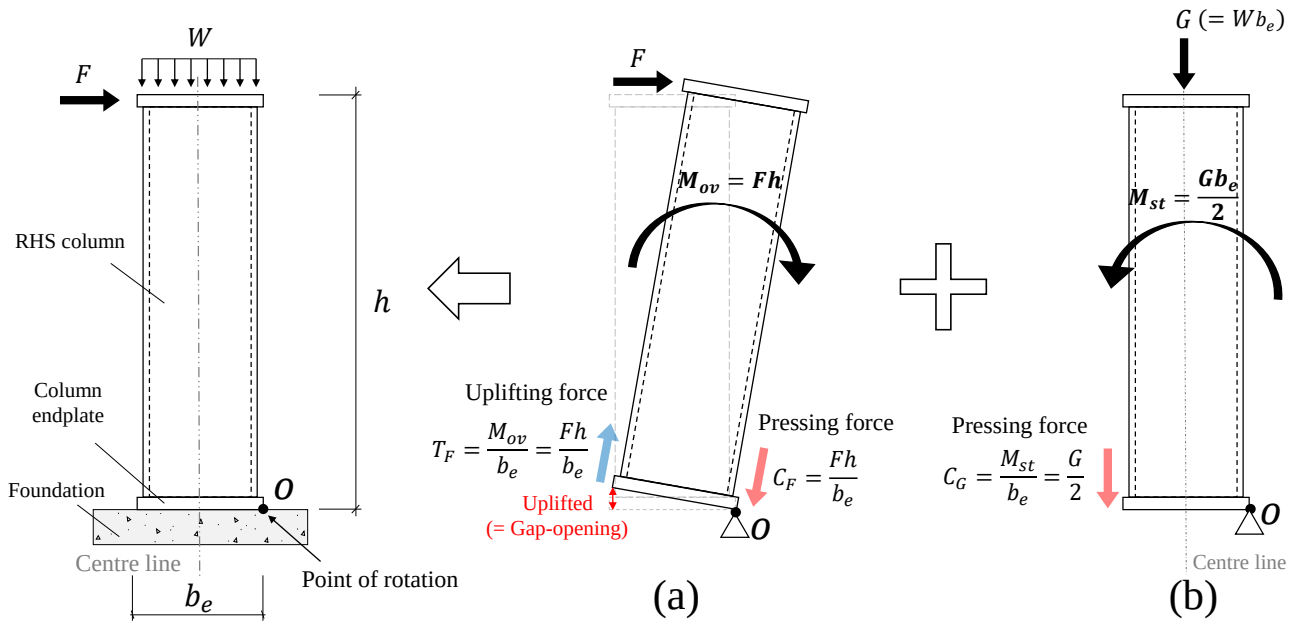


Figure 3.13: Overturning and stabilising moment

If the equilibrium of moments is satisfied at point O , the column stands upright without overturning. In other words, using the condition of moment equilibrium, the minimum self-weight W_{min} , which can resist the overturning effect due to the lateral force, can be found as follows:

$$\sum M_o = 0, \quad (-F \cdot h) + G \cdot \frac{b_e}{2} = 0 \quad (\text{Eq. 3.2})$$

$$G = \frac{2(F \cdot h)}{b_e} \rightarrow W_{min} = \frac{2(F \cdot h)}{b_e^2}$$

3.4.2.2. Preloaded Column and Gap-opening

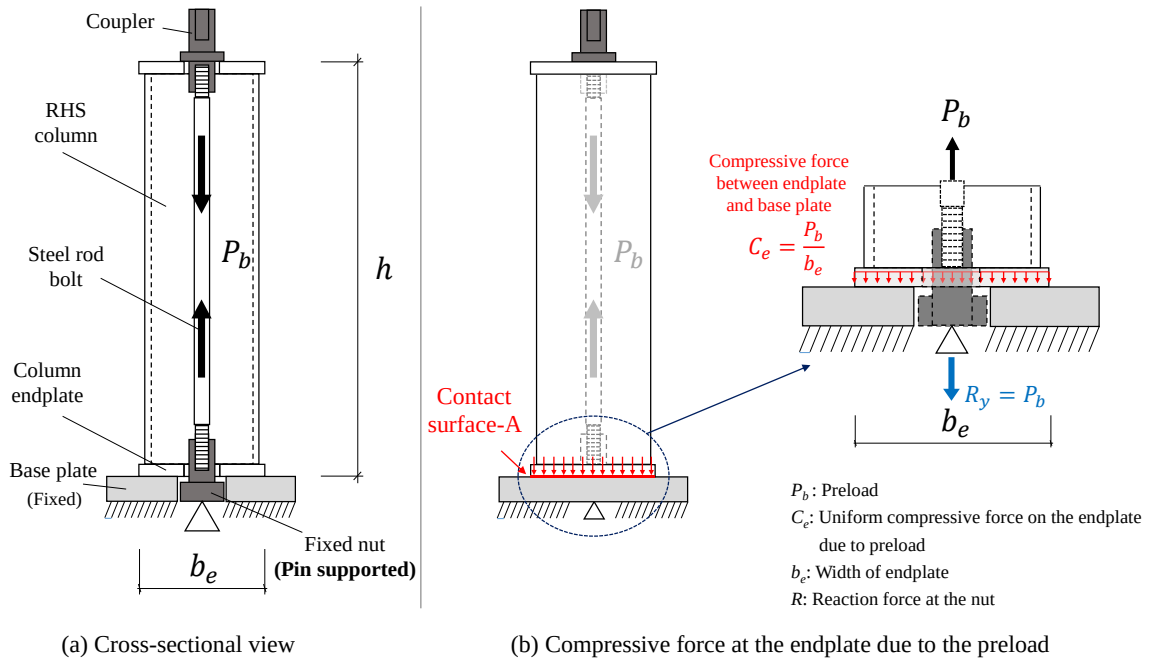


Figure 3.14: Preloaded column

The concept of the overturning and stabilising moment forces can be applied to the preloaded connection to determine the demand preload for preventing gap-opening. As shown in Figure 3.14 (b), if a preload P_b is applied to the SRB at the centre of the RHS column by tightening the coupler, the preload is uniformly distributed on the column endplate and may be conservatively modelled as a distributed compressive force $C_e = P_b/b_e$. (In Figure 3.14, the self-weight of the column is omitted.) Then, if the lateral force F is applied to the top of the column in Figure 3.15 (a), the force F tries to rotate the column at point O (the edge of the column endplate) with the overturning moment $M_{ov}(=Fh)$. On the other hand, the uniformly distributed compressive force on the column endplate C_e due to the preload generates a stabilising moment $M_{st}(= (C_e b_e) b_e/2 = P_b b_e/2)$. Figure 3.15 (b) shows the net forces acting on the column about its overturning point O . Using the equilibrium of moments at point O by (Eq. 3.3), the minimum demand preload $P_{b,min}$ can be found by requiring the column to stand without overturning against the lateral force F . In addition, it is possible to obtain the critical lateral load F_{cr} , which is the maximum lateral load that the preload $P_{b,min}$ can sustain before gap-opening occurs at the bottom of the column.

$$\sum M_o = 0, \quad M_{ov} + M_{st} = (-F \times h) + (C_e \cdot b_e) \times \frac{b_e}{2} = 0 \quad (C_e = \frac{P_b}{b_e}) \quad (\text{Eq. 3.3})$$

$$P_b = \frac{2(F \cdot h)}{b_e} = P_{b,min} \quad (\text{Eq. 3.4})$$

$$F_{cr} = \frac{P_{b,min} \cdot b_e}{2h} \quad (\text{Eq. 3.5})$$

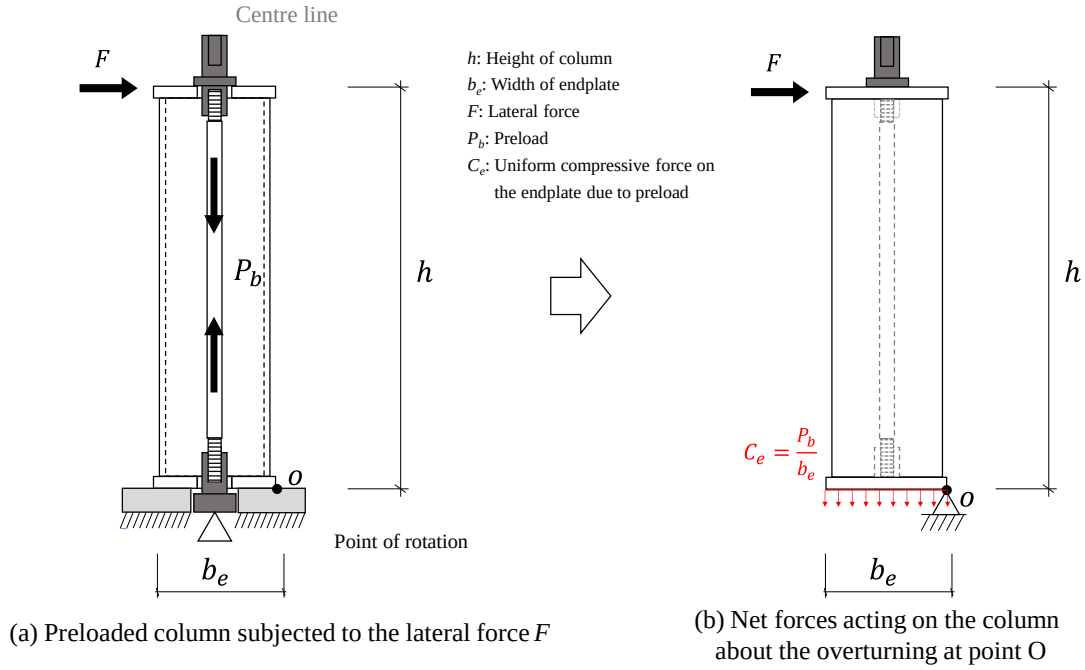


Figure 3.15: Preloaded column subjected to lateral force F

If the preload P_b in the SRB is equal to or greater than $P_{b,min}$, the column develops no gap-opening at its bottom against the lateral force F , and the SRB maintains only the initial preload P_b . In other words, the SRB is subjected to the tensile force of P_b due to the preload.

$$P_b \geq P_{b,min} \rightarrow \text{No gap opening occurs,}$$

$$\text{Tensile force in the SRB is } P_b$$

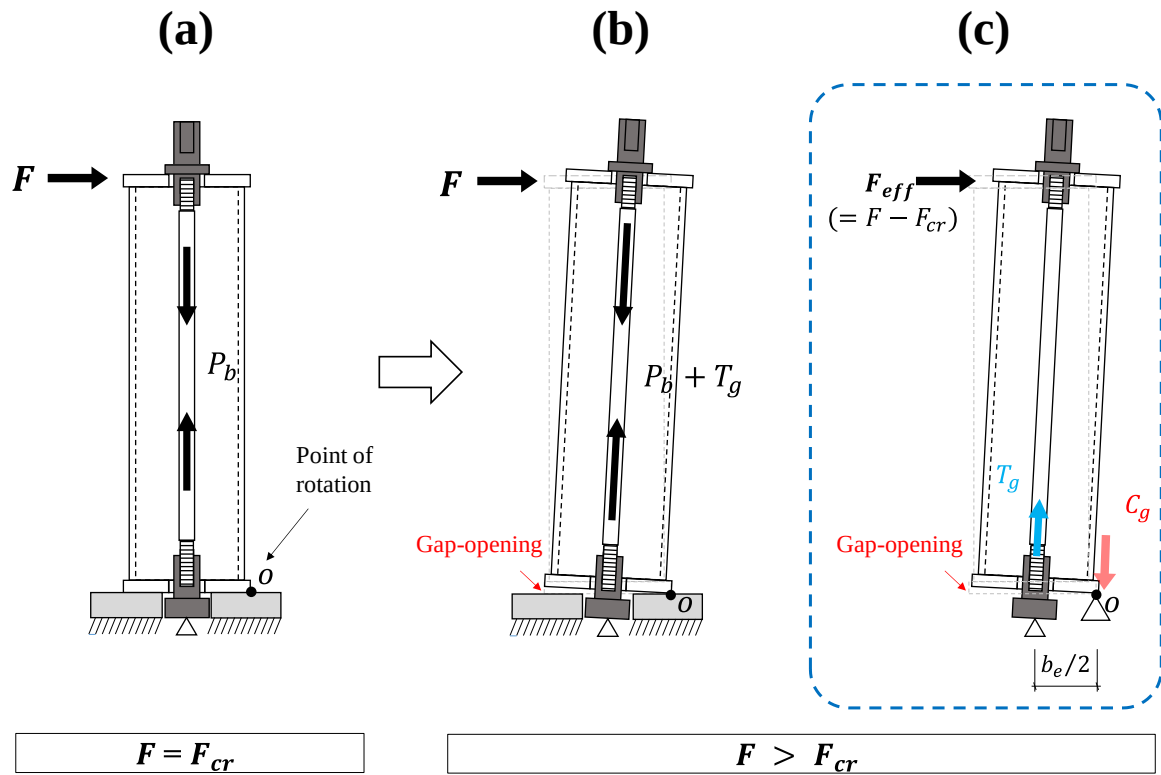
On the other hand, if the preload P_b is less than $P_{b,min}$, the column is affected by the overturning effect. The column stands upright without gap-opening until the lateral force reaches F_{cr} (see Figure 3.16 (a)). When F exceeds F_{cr} , gap-opening occurs at the bottom (see Figure 3.16 (b)) and a compression force C_g is applied at point O , which by equilibrium introduces a tensile force T_g in the SRB (see Figure 3.16 (c)). As a result, the total tensile force in the SRB becomes $P_b + T_g$. The effective lateral force causing T_g and C_g is defined as $F_{eff} = F - F_{cr}$. T_g and C_g can be calculated by (Eq. 3.6).

$$P_b < P_{b,min} \rightarrow \text{Gap opening occurs,}$$

$$\text{Tensile force in the SRB is } P_b + T_g$$

$$T_g = C_g = \frac{F_{eff} \cdot h}{b_e/2} \quad (\text{Eq. 3.6})$$

$$F_{eff} = F - F_{cr} = F - \frac{P_{b,min} \cdot b_e}{2h} \quad (\text{Eq. 3.7})$$



$$F_{cr} = \frac{P_{b,min} \times b_e}{2h}$$

- After F exceeds F_{cr} , the column loses its stability, and gap-opening occurs. The steel rod bolt is subjected to tensile load T_g by resisting the overturning effect in addition to P_b .

Figure 3.16: Behaviour of preloaded column with gap-opening

3.4.2.3. Preloaded Column-beam and Gap-opening

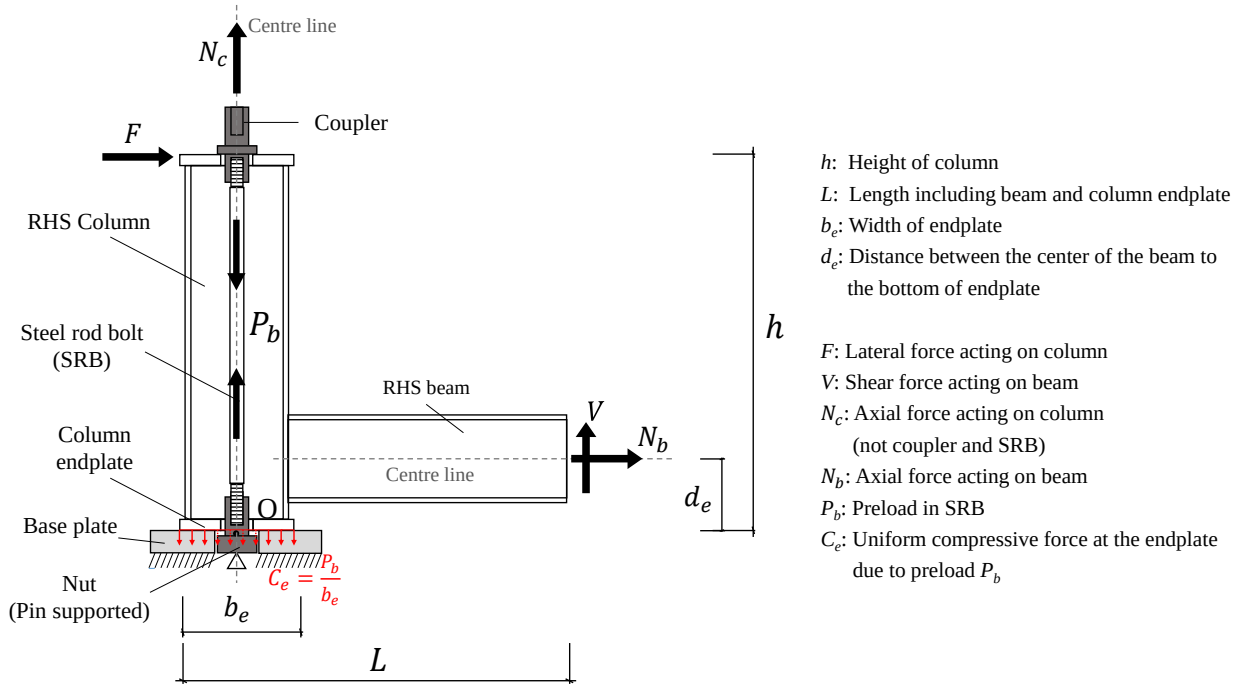


Figure 3.17: L-shaped preloaded column-beam with acting forces

In the case of the L-shaped column-beam in Figure 3.17, the forces acting on the column (F and N_c) and the beam (V and N_b) produce the overturning moments at the bottom of the column. As shown in Figure 3.18, gap-opening will occur at points A or B at the column bottom depending on the magnitudes of the overturning moments $M_{A,O}$ (causing gap-opening at Point A) and $M_{B,O}$ (causing gap-opening at Point B) with respect to the centre of the column (Point O). On the other hand, the compressive force C_e at the bottom of the column endplate due to the preload P_b in Figure 3.17 generates the stabilising moment to prevent gap-opening at both points A and B.

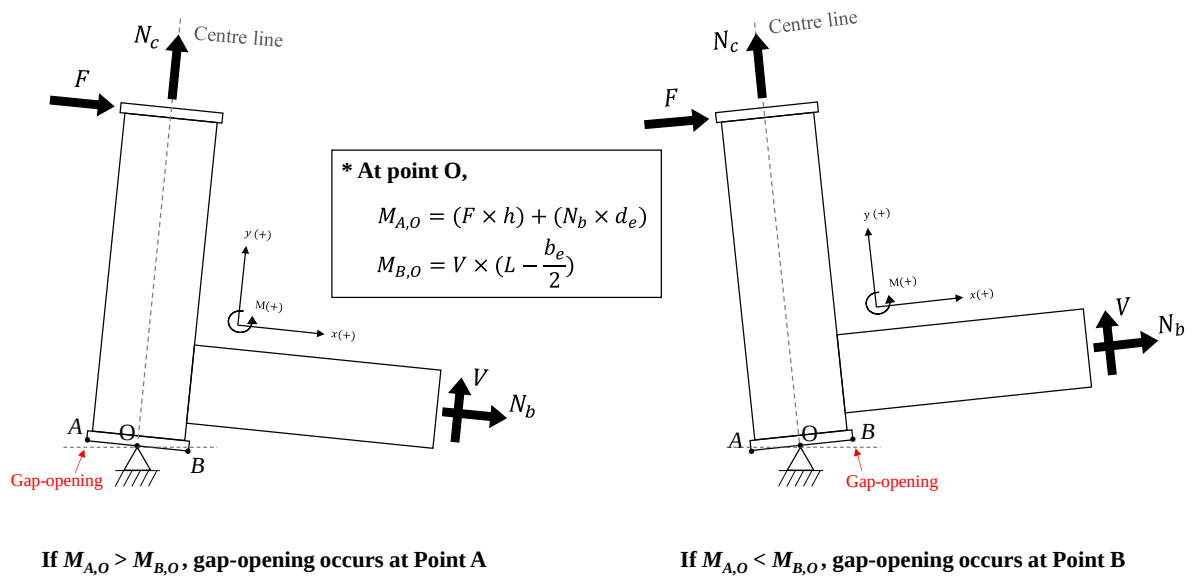


Figure 3.18: Determination of gap-opening location at L-shaped column-beam

Figure 3.19 presents free body diagrams of the L-shaped column-beam with all acting forces. The minimum required preload for preventing gap-opening can be obtained by requiring the equilibrium of moments at points A and B.

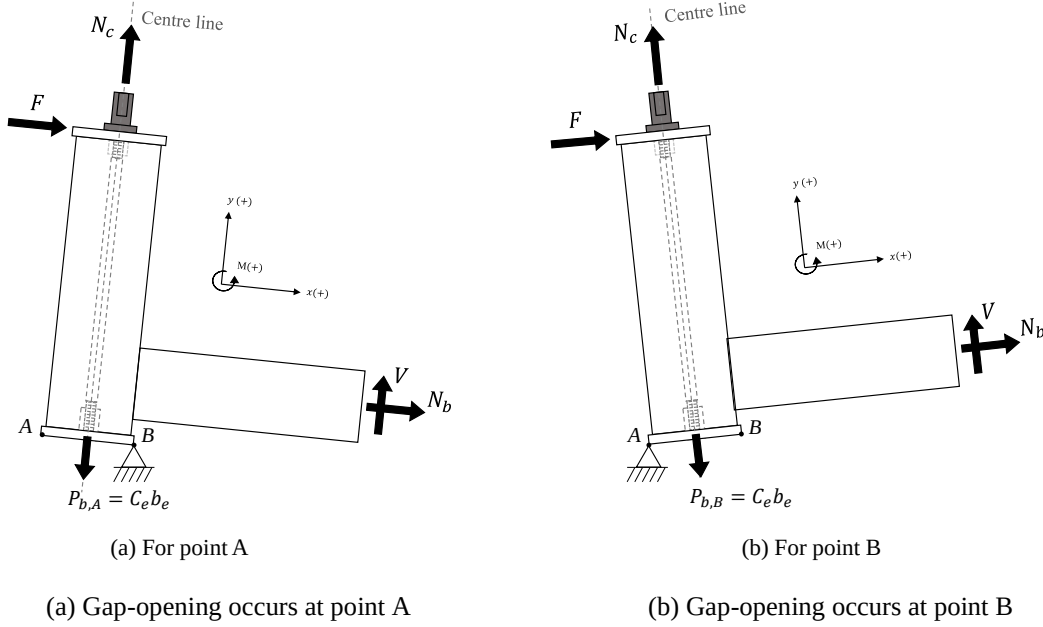


Figure 3.19: Free body diagram to determine the demand preload (P_b) for preventing gap-opening at points A and B

(1) Gap-opening at point A

i) The minimum demand preload to prevent gap-opening at point A ($P_{b,A}$)

$P_{b,A}$ can be determined using the equilibrium of moments at point B in Figure 3.19 (a), (refer to Figure 3.17 for nomenclature and dimensions):

$$\sum M_B = 0, \quad -Fh + V(L - b_e) + P_{b,A} \frac{b_e}{2} - N_c \frac{b_e}{2} - N_b d_e = 0 \quad (\text{Eq. 3.8})$$

$$P_{b,A} = N_c + \frac{2}{b_e} (Fh - V(L - b_e) + N_b d_e) \geq 0 \quad (\text{Eq. 3.9})$$

If $P_{b,A}$ is less than 0, no gap-opening occurs at point A and no preload is required.

ii) Estimating the tensile force $T_{g,A}$ applied to the SRB due to gap-opening at point A

The minimum demand preload required to prevent gap-opening at Point A is denoted by $P_{b,A}$, as shown in Figure 3.20 (a). After the occurrence of gap-opening, the SRB is subjected to an additional tensile force $T_{g,A}$. In other words, only the additional “effective” external force after gap-opening $F_{eff} (= F - F_{cr})$ contributes to $T_{g,A}$. For the preloaded column-beam, the occurrence of

gap-opening involves the forces F , N_c , V and N_b . Since the forces are all caused by the lateral force F , they increase and decrease simultaneously depending on the lateral force. Therefore, the effective forces contributing to $T_{g,A}$ in the preloaded column-beam may collectively be defined as follows:

$$E_{eff} = E - E_{cr} \quad (\text{Eq. 3.10})$$

where :

- E_{eff} The effective forces acting on the column and the beam contributing to the tensile force $T_{g,A}$ in the SRB after gap-opening;
- E The forces acting on the column and beams;
- E_{cr} As the critical forces acting on the column and the beam at incipient gap-opening, they are the greatest forces in which the preload P_b prevents gap-opening.

As shown in Figure 3.20 (a), if the minimum demand preload $P_{b,A}$ is applied to the column, no gap-opening occurs at point A, and equilibrium of moments at point B is maintained between $P_{b,A}$ and other acting forces. The forces acting on the column-beam at incipient gap-opening are the critical forces E_{cr} . When F exceeds $F_{cr,A}$ (causing the other forces N_c , V , and N_b acting on the column-beam to increase as well), gap-opening occurs at point A, as presented in Figure 3.20 (b), and the SRB is subjected to the additional tensile force $T_{g,A}$. The effective forces E_{eff} contributing to $T_{g,A}$ are presented in Figure 3.20 (b).

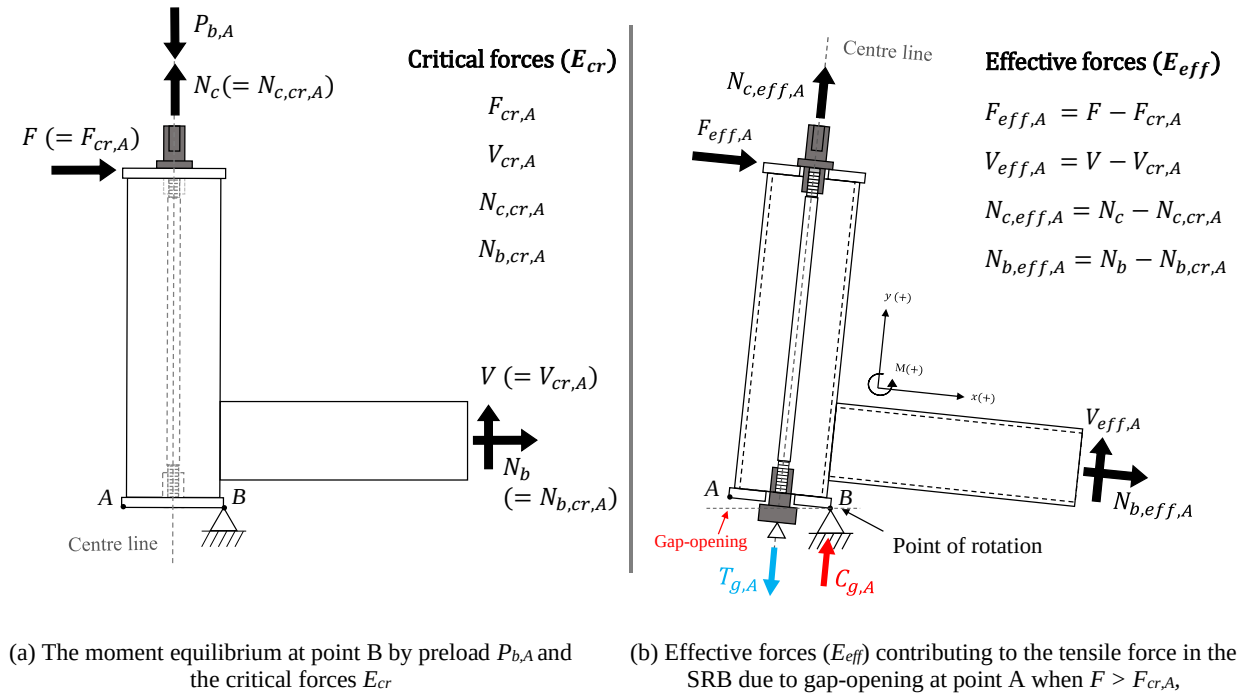


Figure 3.20: Forces when gap-opening occurs at point A

$T_{g,A}$ can be found by requiring the equilibrium of moments at point B (see Figure 3.20 (b) and Figure 3.17).

$$\sum M_B = 0, \quad -F_{eff,A} \cdot h + V_{eff,A} \cdot (L - b_e) - N_{c,eff,A} \cdot \frac{b_e}{2} - N_{b,eff,A} \cdot d_e + T_{g,A} \cdot \frac{b_e}{2} = 0 \quad (\text{Eq. 3.11})$$

$$T_{g,A} = N_{c,eff,A} + \frac{2}{b_e} (F_{eff,A} \cdot h - V_{eff,A} \cdot (L - b_e) + N_{b,eff,A} \cdot d_e) \quad (\text{Eq. 3.12})$$

The final tensile force which the SRB must resist becomes:

$$P_b = P_{b,A} + T_{g,A} \quad (\text{Eq. 3.13})$$

where :

$P_{b,A}$ The initial tensile force due to preload;

$T_{g,A}$ The tensile force due to gap-opening at Point A, (Figure 3.20 (b)).

(2) Gap-opening at point B

i) The minimum demand preload to prevent the gap-opening at point B ($P_{b,B}$)

$P_{b,B}$ can be determined using the equilibrium of moments at point A in Figure 3.19 (b), (refer to Figure 3.17 for nomenclature and dimensions):

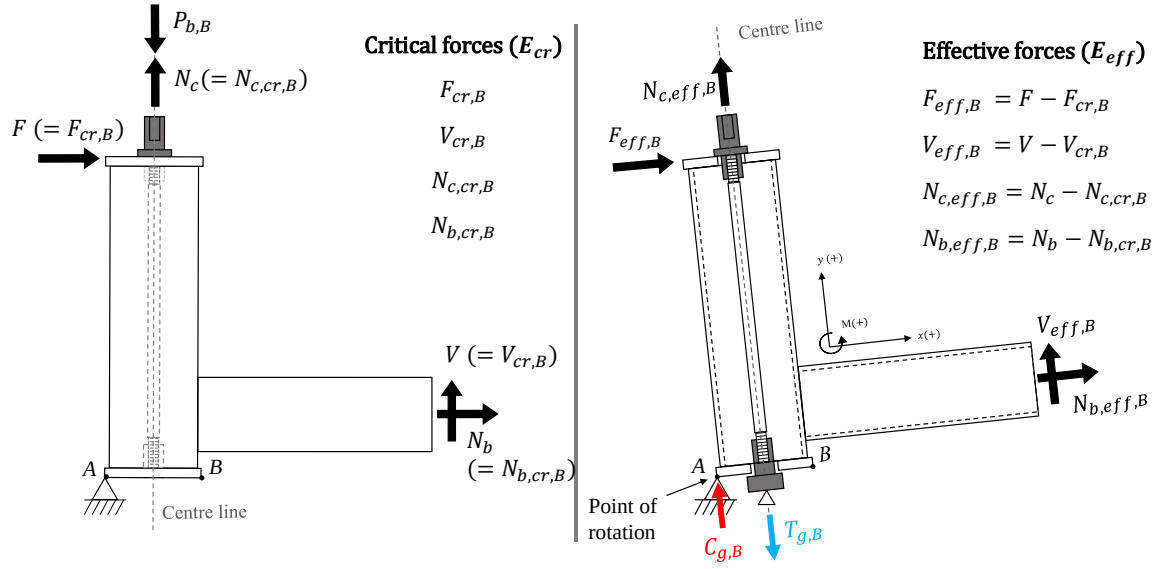
$$\sum M_A = 0, \quad -Fh + VL - P_{b,B} \frac{b_e}{2} + N_c \frac{b_e}{2} - N_b d_e = 0 \quad (\text{Eq. 3.14})$$

$$P_{b,B} = N_c + \frac{2}{b_e} (VL - Fh - N_b d_e) \geq 0 \quad (\text{Eq. 3.15})$$

If $P_{b,B}$ is less than 0, no gap-opening occurs at point B, and no preload is required.

ii) Estimating the tensile force $T_{g,B}$ applied to the SRB due to gap-opening at point B

The forces acting on the column-beam at incipient gap-opening and after gap-opening at point B are shown in Figure 3.21 (a) and Figure 3.21 (b), respectively, defined in a similar way to when gap-opening occurs at point A. $P_{b,B}$ is the minimum demand preload required to prevent gap-opening and $T_{g,B}$ is the additional force developing in the SRB as a result of the effective forces as shown in Figure 3.21 (b).



(a) The moment equilibrium at point A by preload $P_{b,B}$, and the critical forces E_{cr}

(b) Effective forces (E_{eff}) contributing to the tensile force in the SRB due to gap-opening at point B when $F > F_{cr,B}$

Figure 3.21: Forces when gap-opening occurs at point B

$T_{g,B}$ can be found by requiring the equilibrium of moments at point A (see Figure 3.21 (b) and Figure 3.17).

$$\sum M_A = 0, \quad -F_{eff,B} \cdot h + V_{eff,B} \cdot L + N_{c,eff,B} \cdot \frac{b_e}{2} - N_{b,eff,B} \cdot d_e - T_{g,B} \cdot \frac{b_e}{2} = 0 \quad (\text{Eq. 3.16})$$

$$T_{g,B} = N_{c,eff,B} + \frac{2}{b_e} (V_{eff,B} \cdot L - F_{eff,B} \cdot h - N_{b,eff,B} \cdot d_e) \quad (\text{Eq. 3.17})$$

The final tensile force which the SRB must resist becomes:

$$P_b = P_{b,B} + T_{g,B} \quad (\text{Eq. 3.18})$$

where :

$P_{b,B}$ The initial tensile force due to preload;

$T_{g,B}$ The tensile force due to gap-opening at Point B, (Figure 3.21 (b)).

3.4.3. Demand Preload for Preventing Gap-opening (P_g) at the AJ Connection

The behaviour of a multi-storey steel-framed modular building subjected to lateral forces depends on the rigidity of inter-module connections. As presented in Figure 3.22 (a), if the modules are connected vertically and horizontally with appropriate rigidity (i.e. axial, shear, and bending rigidity), the modular building will ideally behave as a continuous structure. In contrast, if the rigidity of the inter-module connections is insufficient, particularly the horizontal inter-module connections, the modular building will behave similarly to multiple cantilever columns, as shown in Figure 3.22 (b). Specifically, the axial tensile (T) and compressive forces (C) acting on the columns in the modules due to the lateral force depend on the rigidity of the horizontal inter-module connections.

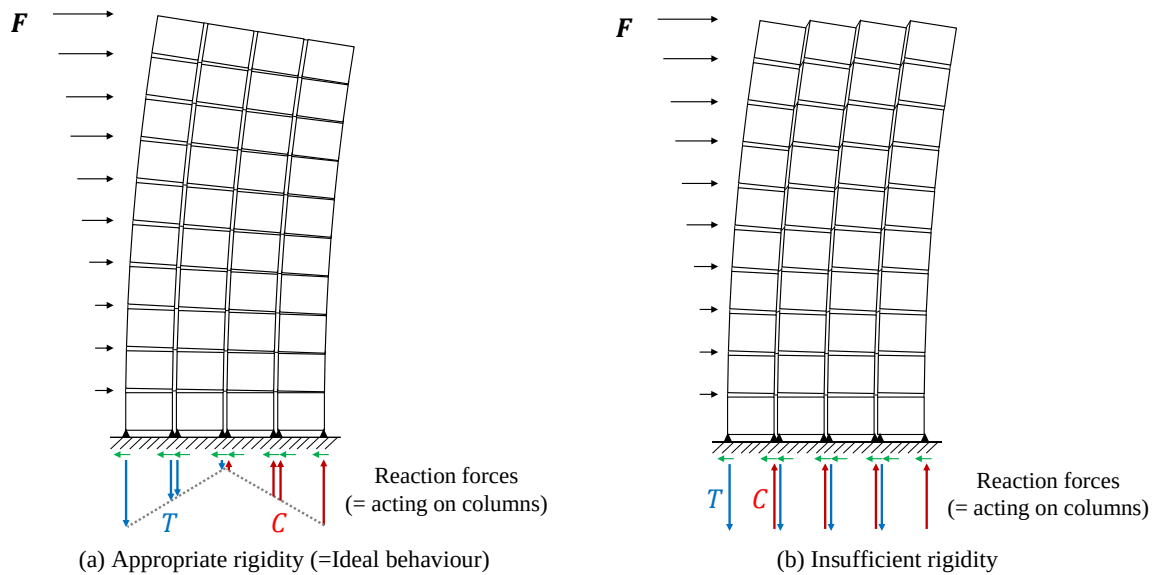


Figure 3.22: Behaviour of the modular building depending on horizontal rigidity

For post-tensioned type connections, such as the AJ connection, the axial tensile and compressive forces acting on the columns affect the demand preload of the column-to-column connections. Therefore, when performing structural analysis of the modular building to obtain the demand preloads of the connections, the rigidity of the connections between modules must be known to model the horizontal connection accurately, (e.g. as pinned or rigid). In the case of the AJ connection, the modules are horizontally connected by the connecting plate of thickness 20mm. According to Section 6.2.3, the AJ connection is considered to have sufficient vertical and horizontal rigidity for the modules to behave as a continuous structure.

In case gap-opening occurs at the AJ connections in multi-storey modular buildings due to lateral force, the deformed shapes are presented in Figure 3.23 according to the location of the connection. The further out the module is located from the centre, the greater tensile or compressive forces are induced in the columns. This implies that AJ connections located near the outermost modules will have gap-

opening caused by axial tensile forces dominantly (see Figure 3.23 (a)). On the other hand, the closer the module is located to the centre, the more the axial tensile forces in the columns decrease and the shear forces acting on the column and beam become the dominant forces causing gap-opening (see Figure 3.23 (b)). In any case, gap-opening should be prevented at all connections by applying preload.

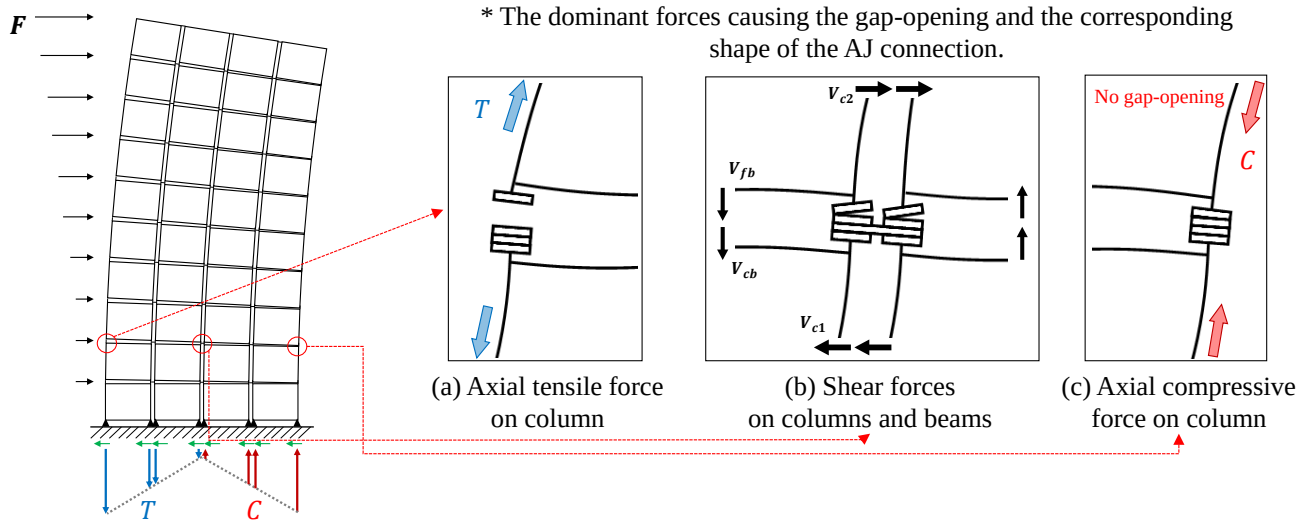
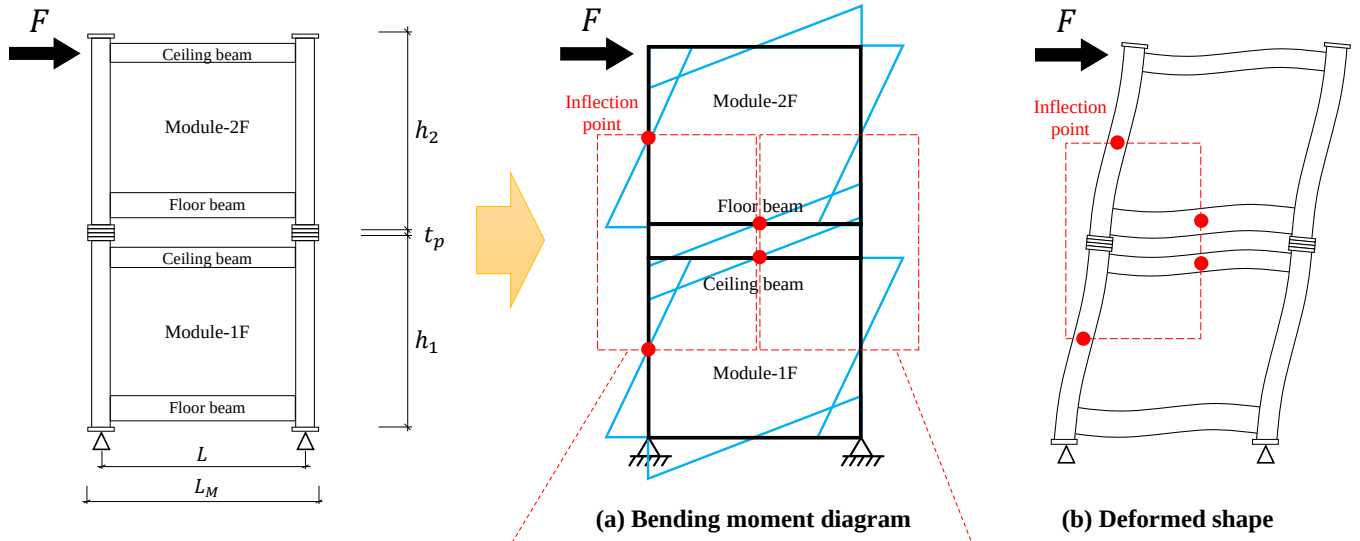


Figure 3.23: Deformed shapes according to the location of AJ connections in a multi-storey modular building when gap-opening occurs due to lateral force

This section presents the process of determining the demand preload (P_g) to prevent gap-opening at the AJ connection, using a two-storey module frame as an example. Figure 3.24 (a) presents the bending moment diagram (BMD) when the lateral force is applied to the frame. Based on the assumed inflection points of columns and beams in the BMD, (implying zero moment), the deformed shape of a beam-to-column joint with the external forces applied is presented in Figure 3.24 (c) and (d). The demand preload is determined at the connection where the tensile axial force A_T acts due to the lateral force (see Figure 3.24 (c)). The connection where the compressive axial force A_C acts (see Figure 3.24 (d)), is unlikely to have gap-opening or slip because of the closing effect of the compressive force A_C and gravity load G . Figure 3.25 (a) presents forces acting on the preloaded beam-to-column joint. The occurrence of the gap-opening at the connection is related to the following forces:

- 1) Gravity load due to the self-weight of modules (G_1 and G_2);
- 2) Axial tensile load (A_T) or compressive load (A_C) due to the lateral force;
- 3) Shear forces acting on the column (V_{c1} and V_{c2}) and the beam (V_{b1} and V_{b2});
- 4) Axial loads (N_{b1} and N_{b2}) on beams;
- 5) Preloads (P_{b1} and P_{b2}) in the SRBs.

(The gravity load due to the self-weight of modules helps to prevent the gap-opening at the connection. The effect of self-weight is included when calculating the demand preload in this section. Otherwise, the demand preload becomes significantly large.)



F : The lateral load
 t_p : Thickness of plates at connection
 h_1 : Height of module-1F
 h_2 : Height of module-2F
 L : Width of module (column centre to centre)
 L_M : Width of module (including column-endplate)

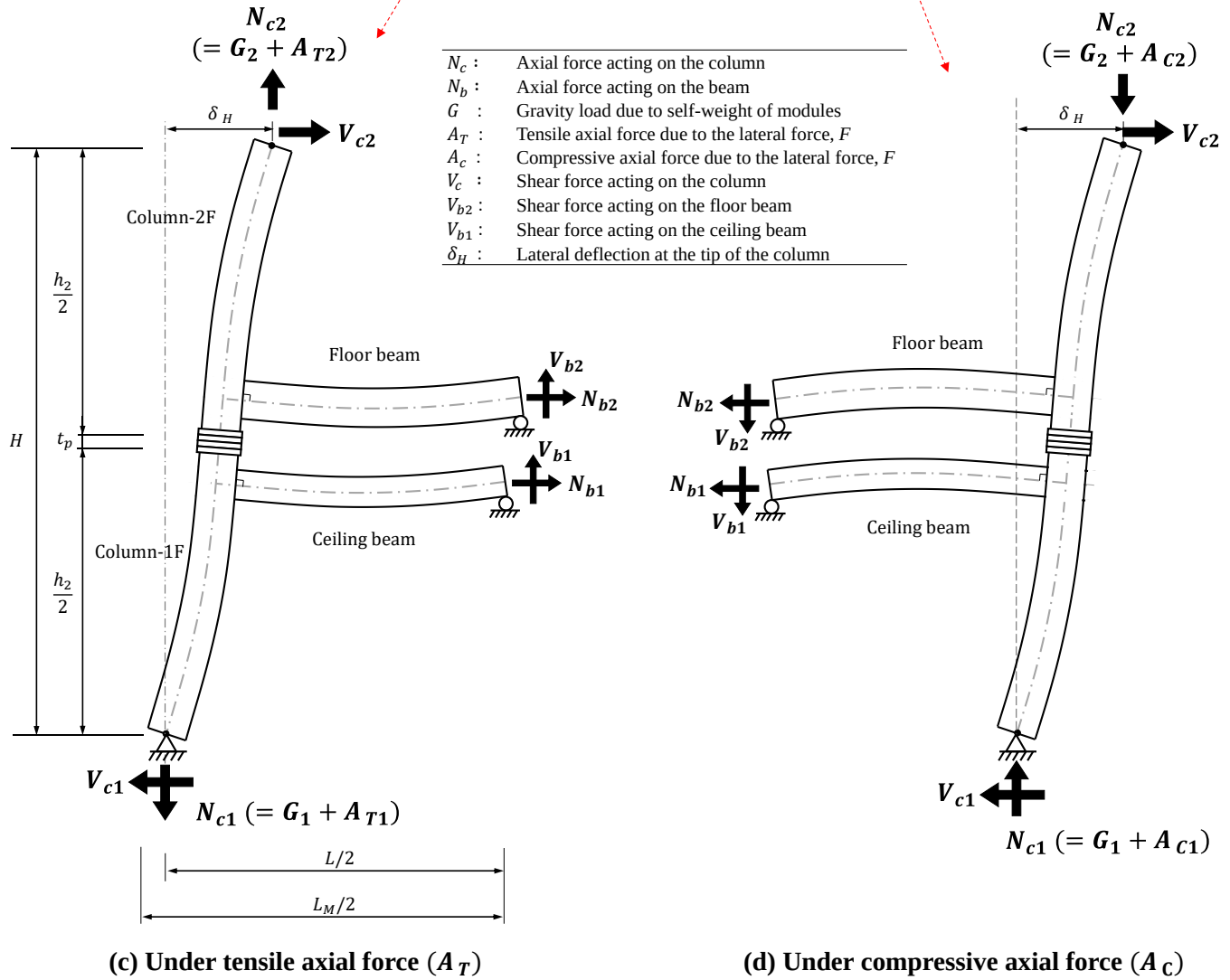


Figure 3.24: Deformed shape of the subassembly (beam-to-column joint) with the external forces in the modular building

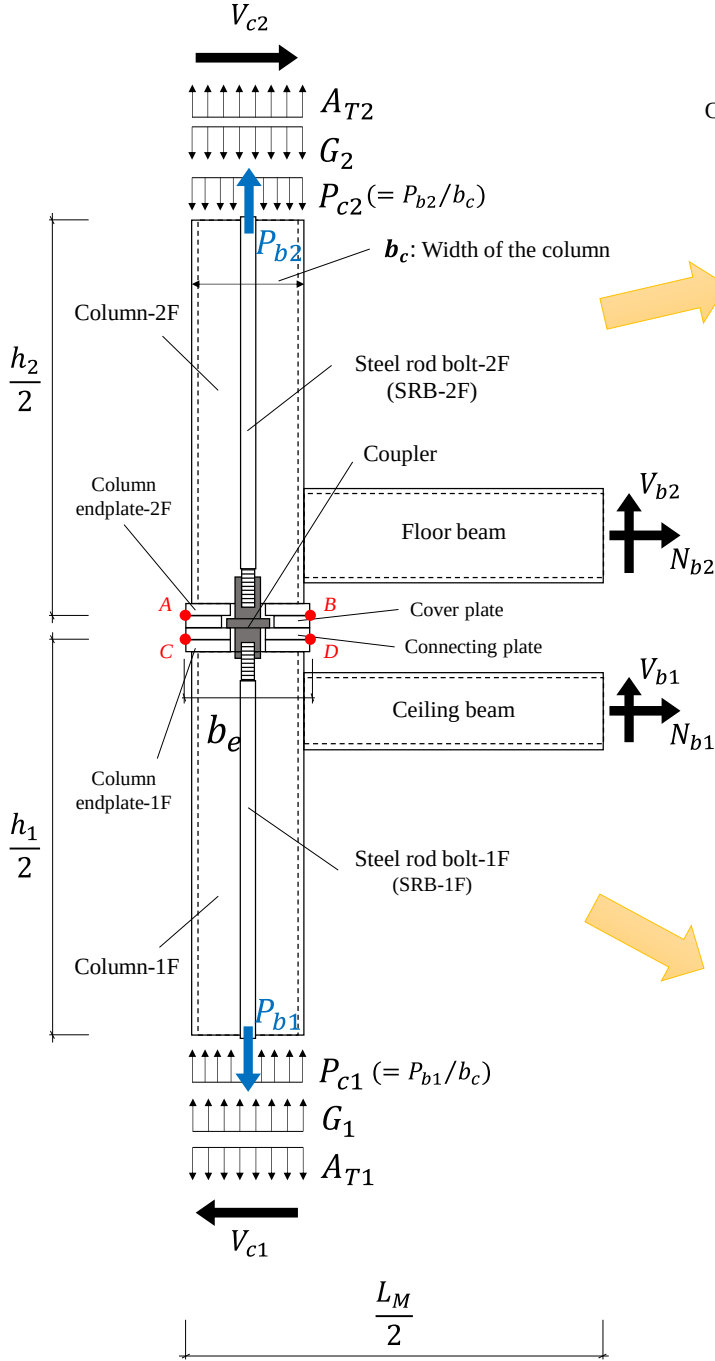
The gap-opening will occur at the contact surfaces -1 and -2 (see Figure 3.25 (d)). The upper column-beam (see Figure 3.25 (b)) will develop a gap-opening at point A or B on contact surface-2. The clamping force on contact surface-2 is determined by the preload P_{b2} , which is applied through SRB-2F within column-2F. On the other hand, the lower-column beam (see Figure 3.25 (c)) will develop a gap-opening at point C or D on contact surface-1. The clamping force on contact surface-1 is determined by the preload P_{b1} , which is applied through SRB-1F within column-1F. The demand preloads to prevent gap-openings at the contact surfaces-1 and -2 need to be calculated separately because of the different design forces acting on the upper column-beam and the lower column-beam.

Since the coupler at the connection is stationary vertically if $P_{b1} \geq P_{b2}$ (refer to Figure 3.9) and is able to resist the shear forces acting on the columns by friction on the column endplates provided by preload, the coupler can be modelled as a pinned support when calculating the demand preload, as presented in Figure 3.25 (b) and (c).

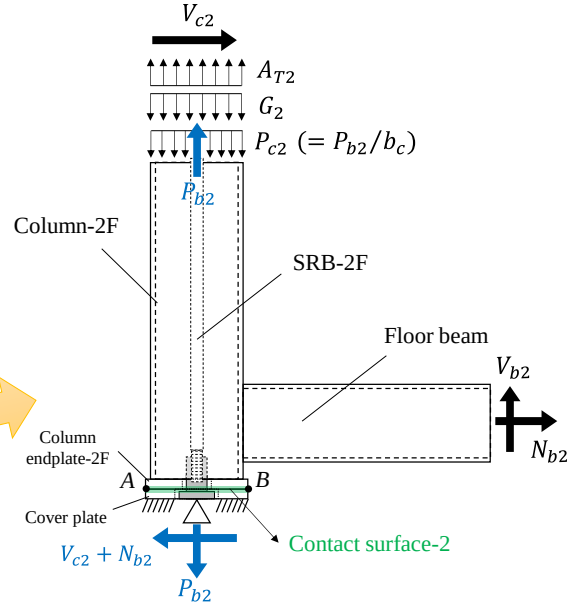
Theoretically, it is possible to connect the lower column and upper column by applying the demand preloads P_{b1} and P_{b2} to contact surfaces-1 and -2, respectively. In that case, the clamping forces between the contact surfaces at the connection will be different. However, as mentioned in Section 3.3, it is preferable to make all contact surfaces have uniform clamping forces at the connection. Therefore, it is recommended to choose the greater of the demand preloads P_{b1} and P_{b2} , and then apply this preload to both the upper and lower columns.

(a) Connection for the upper and lower columns

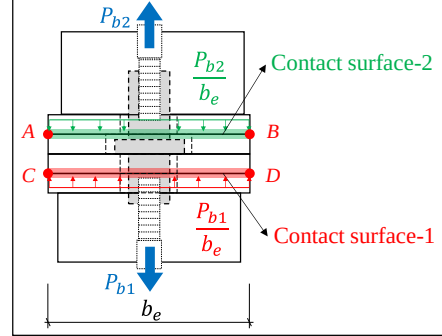
- Forces acting on column & beam
- Forces acting on steel rod bolt & coupler



(b) Upper column-beam



(d) Possible location of gap-opening



(c) Lower column-beam

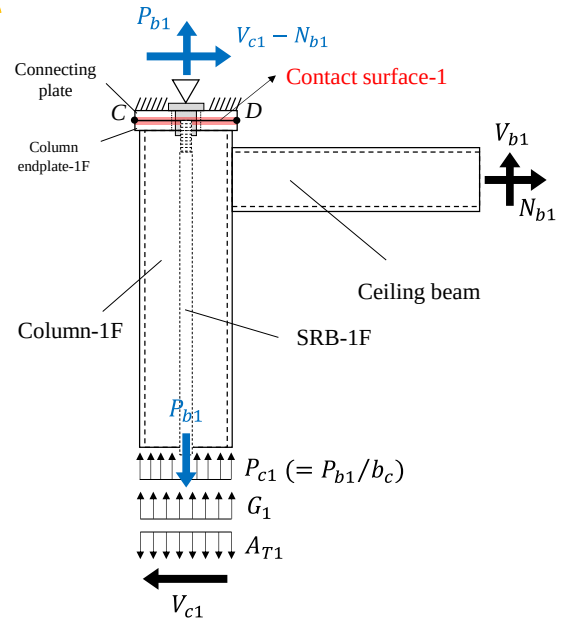


Figure 3.25: Design forces for calculating the demand preload (P_g) to prevent gap-opening at AJ connection (refer to Figure 3.24 for nomenclature and dimensions)

Step 1: Demand preload (P_{b2}) for the upper column-beam

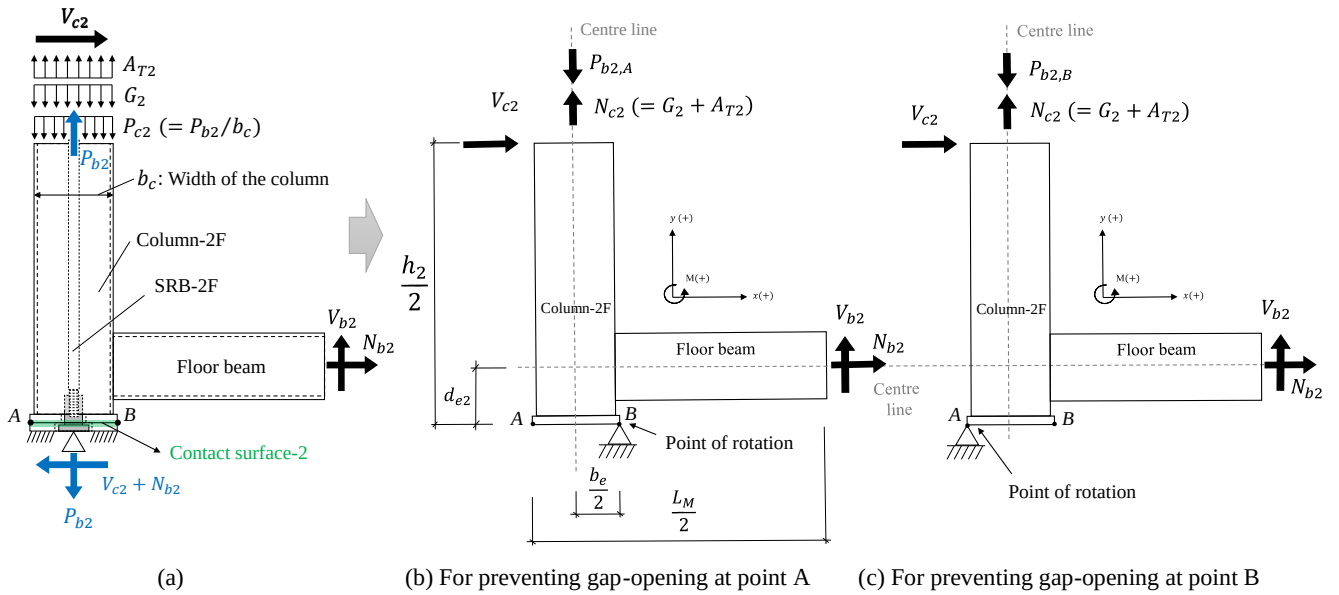


Figure 3.26: Free body diagrams of the upper column-beam for the demand preload (P_{b2}) to prevent the gap-opening at contact surface-2 (refer to Figure 3.24 for nomenclature and dimensions)

Figure 3.26 (a) shows all forces acting on column-2F, floor beam and SRB-2F. Figure 3.26 (b) and (c) present free body diagrams of the upper column-beam to obtain the demand preload (P_{b2}) for preventing gap-opening at points A and B on contact surface-2. The gap-opening is prevented when the stabilising moment M_{st} is equal to or greater than the overturning moment M_{ov} . Using the moment equilibrium equations at points A and B, the minimum demand preload can be determined as follows (refer to Section 3.4.2.3 for a further detailed explanation of the process):

- 1) The minimum demand preload ($P_{b2,A}$) to prevent the gap-opening at point A (see Figure 3.26 (b)):

$$\sum M_B = 0, \quad M_{ov,B} + M_{st,B} = 0$$

$$\left(M_{ov,B} = -V_{c2} \cdot \frac{h_2}{2} - N_{c2} \cdot \frac{b_e}{2} - N_{b2} \cdot d_{e2} \quad \text{and} \quad M_{st,B} = V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + P_{b2,A} \cdot \frac{b_e}{2} \right) \quad (\text{Eq. 3.19})$$

$$-V_{c2} \cdot \frac{h_2}{2} - N_{c2} \cdot \frac{b_e}{2} - N_{b2} \cdot d_{e2} + V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + P_{b2,A} \cdot \frac{b_e}{2} = 0$$

$$P_{b2,A} = N_{c2} + \frac{2}{b_e} \left(V_{c2} \cdot \frac{h_2}{2} - V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{b2} \cdot d_{e2} \right) \quad (\text{Eq. 3.20})$$

- 2) The minimum demand preload ($P_{b2,B}$) to prevent the gap-opening at point B (see Figure 3.26 (c)):

$$\sum M_A = 0, \quad M_{ov,A} + M_{st,A} = 0$$

$$\left(M_{ov,A} = V_{b2} \cdot \frac{L_M}{2} + N_{c2} \cdot \frac{b_e}{2} \quad \text{and} \quad M_{st,A} = -N_{b2} \cdot d_{e2} - V_{c2} \cdot \frac{h_2}{2} - P_{b2,B} \cdot \frac{b_e}{2} \right) \quad (\text{Eq. 3.21})$$

$$V_{b2} \cdot \frac{L_M}{2} + N_{c2} \cdot \frac{b_e}{2} - N_{b2} \cdot d_{e2} - V_{c2} \cdot \frac{h_2}{2} - P_{b2,B} \cdot \frac{b_e}{2} = 0$$

$$P_{b2,B} = N_{c2} + \frac{2}{b_e} \left(V_{b2} \cdot \frac{L_M}{2} - V_{c2} \cdot \frac{h_2}{2} - N_{b2} \cdot d_{e2} \right) \quad (\text{Eq. 3.22})$$

Then, the larger preload of $P_{b2,A}$ and $P_{b2,B}$ becomes the demand preload (P_{b2}) for preventing gap-opening at contact surface-2 (between column endplate-2F and the cover plate at the connection).

$$P_{b2} \geq \text{Max} (P_{b2,A}, P_{b2,B}) \quad (\text{Eq. 3.23})$$

Step 2: Demand preload (P_{b1}) for the lower column-beam

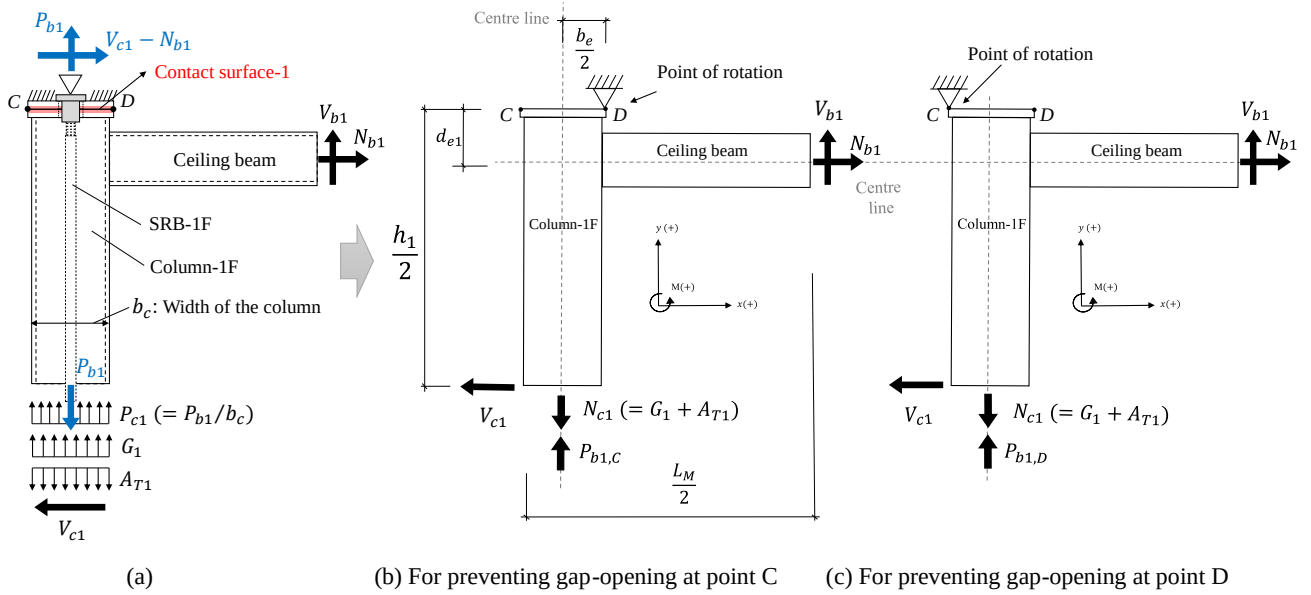


Figure 3.27: Free body diagram of the lower column-beam for the demand preload (P_{b1}) to prevent gap-opening at contact surface-1. (refer to Figure 3.24 for nomenclature and dimensions)

Figure 3.27 (a) shows all forces acting on column-1F, ceiling beam and SRB-1F. Figure 3.27 (b) and (c) present the free body diagrams of the lower column-beam to obtain the preload force (P_{b1}) for preventing the gap-opening at points C and D on contact surface-1. Using the moment equilibrium equations at points C and D, the minimum demand preload can be determined as follows:

1) The minimum demand preload ($P_{b1,C}$) to prevent the gap-opening at point C (see Figure 3.27 (b)):

$$\sum M_D = 0, \quad M_{ov,D} + M_{st,D} = 0$$

$$\left(M_{ov,D} = V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{c1} \cdot \frac{b_e}{2} + N_{b1} \cdot d_{e1} \quad \text{and} \quad M_{st,D} = -V_{c1} \cdot \frac{h_1}{2} - P_{b1,C} \cdot \frac{b_e}{2} \right) \quad (\text{Eq. 3.24})$$

$$V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{c1} \cdot \frac{b_e}{2} + N_{b1} \cdot d_{e1} - V_{c1} \cdot \frac{h_1}{2} - P_{b1,C} \cdot \frac{b_e}{2} = 0$$

$$P_{b1,C} = N_{c1} + \frac{2}{b_e} \left(V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) - V_{c1} \cdot \frac{h_1}{2} + N_{b1} \cdot d_{e1} \right) \quad (\text{Eq. 3.25})$$

2) The minimum demand preload ($P_{b1,D}$) to prevent the gap-opening at point D (see Figure 3.27 (c)):

$$\sum M_C = 0, \quad M_{ov,C} + M_{st,C} = 0$$

$$\left(M_{ov,C} = -V_{c1} \cdot \frac{h_1}{2} - N_{c1} \cdot \frac{b_e}{2} \quad \text{and} \quad M_{st,C} = N_{b1} \cdot d_{e1} + V_{b1} \cdot \frac{L_M}{2} + P_{b1,D} \cdot \frac{b_e}{2} \right) \quad (\text{Eq. 3.26})$$

$$-V_{c1} \cdot \frac{h_1}{2} - N_{c1} \cdot \frac{b_e}{2} + N_{b1} \cdot d_{e1} + V_{b1} \cdot \frac{L_M}{2} + P_{b1,D} \cdot \frac{b_e}{2} = 0$$

$$P_{b1,D} = N_{c1} + \frac{2}{b_e} \left(V_{c1} \cdot \frac{h_1}{2} - V_{b1} \cdot \frac{L_M}{2} - N_{b1} \cdot d_{e1} \right) \quad (\text{Eq. 3.27})$$

Then, the larger preload of $P_{b1,C}$ and $P_{b1,D}$ becomes the demand preload (P_{b1}) for preventing the gap-opening at contact surface-1 (between column endplate-1F and the connecting plate at the connection).

$$P_{b1} \geq \text{Max} (P_{b1,C}, P_{b1,D}) \quad (\text{Eq. 3.28})$$

Step 3: Determination of the demand preload (P_g) for the AJ connection

The demand preload (P_g) for preventing the gap-opening at the connection is determined as the greater of P_{b1} (for the lower column) or P_{b2} (for the upper column). This preload (P_g) is applied to both the lower and upper columns.

$$P_g = \text{Max} (P_{b1}, P_{b2}) \quad (\text{Eq. 3.29})$$

Finding the location of gap-opening at the AJ connection

The potential locations of a gap-opening are points A, B, C, and D of the AJ connection shown in Figure 3.25 (d). The gap-opening occurs at the load where the overturning moment M_{ov} exceeds the stabilising M_{st} the earliest among the four points. Therefore, the gap-opening position can be predicted by comparing the ratio of M_{ov} to M_{st} at each point, as presented in Table 3.2, and selecting the point with the largest $\frac{M_{ov}}{M_{st}}$ -ratio.

Table 3.2: Prediction of the location of the gap-opening at the AJ connection of the beam-to-column joint

Position of gap-opening at the AJ connection		Ratio of overturning moment (M_{ov}) to stabilising moment (M_{st})	
Upper column-floor beam	Point A	$\frac{ M_{ov,B} }{ M_{st,B} } = \frac{\left -V_{c2} \cdot \frac{h_2}{2} - N_{c2} \cdot \frac{b_e}{2} - N_{b2} \cdot d_{e2} \right }{\left V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + P_{b2} \cdot \frac{b_e}{2} \right }$	*Refer to (Eq. 3.19) and Figure 3.26 (b) for dimension and forces
	Point B	$\frac{ M_{ov,A} }{ M_{st,A} } = \frac{\left V_{b2} \cdot \frac{L_M}{2} + N_{c2} \cdot \frac{b_e}{2} \right }{\left -N_{b2} \cdot d_{e2} - V_{c2} \cdot \frac{h_2}{2} - P_{b2} \cdot \frac{b_e}{2} \right }$	*Refer to (Eq. 3.21) and Figure 3.26 (c) for dimension and forces
Lower column-ceiling beam	Point C	$\frac{ M_{ov,D} }{ M_{st,D} } = \frac{\left V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{c1} \cdot \frac{b_e}{2} + N_{b1} \cdot d_{e1} \right }{\left -V_{c1} \cdot \frac{h_1}{2} - P_{b1} \cdot \frac{b_e}{2} \right }$	*Refer to (Eq. 3.24) and Figure 3.27 (b) for dimension and forces
	Point D	$\frac{ M_{ov,C} }{ M_{st,C} } = \frac{\left -V_{c1} \cdot \frac{h_1}{2} - N_{c1} \cdot \frac{b_e}{2} \right }{\left V_{b1} \cdot \frac{L_M}{2} + N_{b1} \cdot d_{e1} + P_{b1} \cdot \frac{b_e}{2} \right }$	*Refer to (Eq. 3.26) and Figure 3.27 (c) for dimension and forces
➔ The point having the maximum $\left \frac{M_{ov}}{M_{st}} \right $ -value will be the location of the first gap-opening			

3.4.4. Demand Preload for Preventing Slip (P_s) at the AJ Connection

When slip occurs at the AJ connection in a multi-storey modular building due to a lateral force such as wind and earthquake, the accumulated slip displacement from the AJ connection can affect the stability of the entire structure (refer to Figure 3.11 (b)). Therefore, for the multi-storey modular building, it is recommended that the AJ connection be designed as a slip-resistant connection according to the connection category C in EN 1993-1-8 [85], which does not allow the occurrence of slip at the ultimate limit state. This means the slip resistance $F_{s,Rd}$ should be greater than the design ultimate shear load $F_{v,Ed}$ transferred between the clamped plates, i.e.

$$F_{s,Rd} \geq F_{v,Ed}. \quad (\text{Eq. 3.30})$$

The slip resistance $F_{s,Rd}$ at the ultimate limit state is calculated by (Eq. 3.31). It is dependent on the friction coefficient μ of the clamped plates and the normal force $F_{N,c}$ on the contact surface.

$$F_{s,Rd} = \mu \cdot F_{N,c} \quad (\text{Eq. 3.31})$$

In the AJ connection, the normal force $F_{N,c}$ for the slip resistance is decided not only by preload but also by the following forces:

- 1) Gravity load due to the self-weight of modules (G);
- 2) Axial tensile load (A_T) or compressive load (A_C) due to the lateral force;
- 3) Shear forces acting on the beam (V_{b1} and V_{b2});
- 5) Preloads ($P_{b1,s}$ and $P_{b2,s}$) in the SRBs.

Figure 3.28 (a) presents forces acting on the preloaded beam-to-column joint (based on the assumed inflection points in the bending moment diagram of columns and beams under the lateral loading shown in Figure 3.24 (a)). The slip will occur at the contact surface-1 and -2 at the connection, as illustrated in Figure 3.28 (d). For the upper column, the slip will occur on contact surface-2 (see Figure 3.28 (b)), for which the normal force for calculating the slip resistance $F_{s,Rd2}$ is determined by the preload $P_{b2,s}$ applied through SRB-2F within column-2F. On the other hand, the lower column will have slip occurring on contact surface-1 (see Figure 3.28 (c)), for which the normal force for calculating the slip resistance $F_{s,Rd1}$ is determined by the preload $P_{b1,s}$ applied through SRB-1F within column-1F. The demand preloads to prevent slip at contact surfaces-1 and -2 should be calculated separately due to the different design forces acting on the upper column-beam and the lower column-beam. The demand preload (P_s) for preventing slip becomes the greater of $P_{b1,s}$ and $P_{b2,s}$. This preload (P_s) should be applied to both the upper and lower columns to ensure uniform clamping forces on all contact surfaces of the connection.

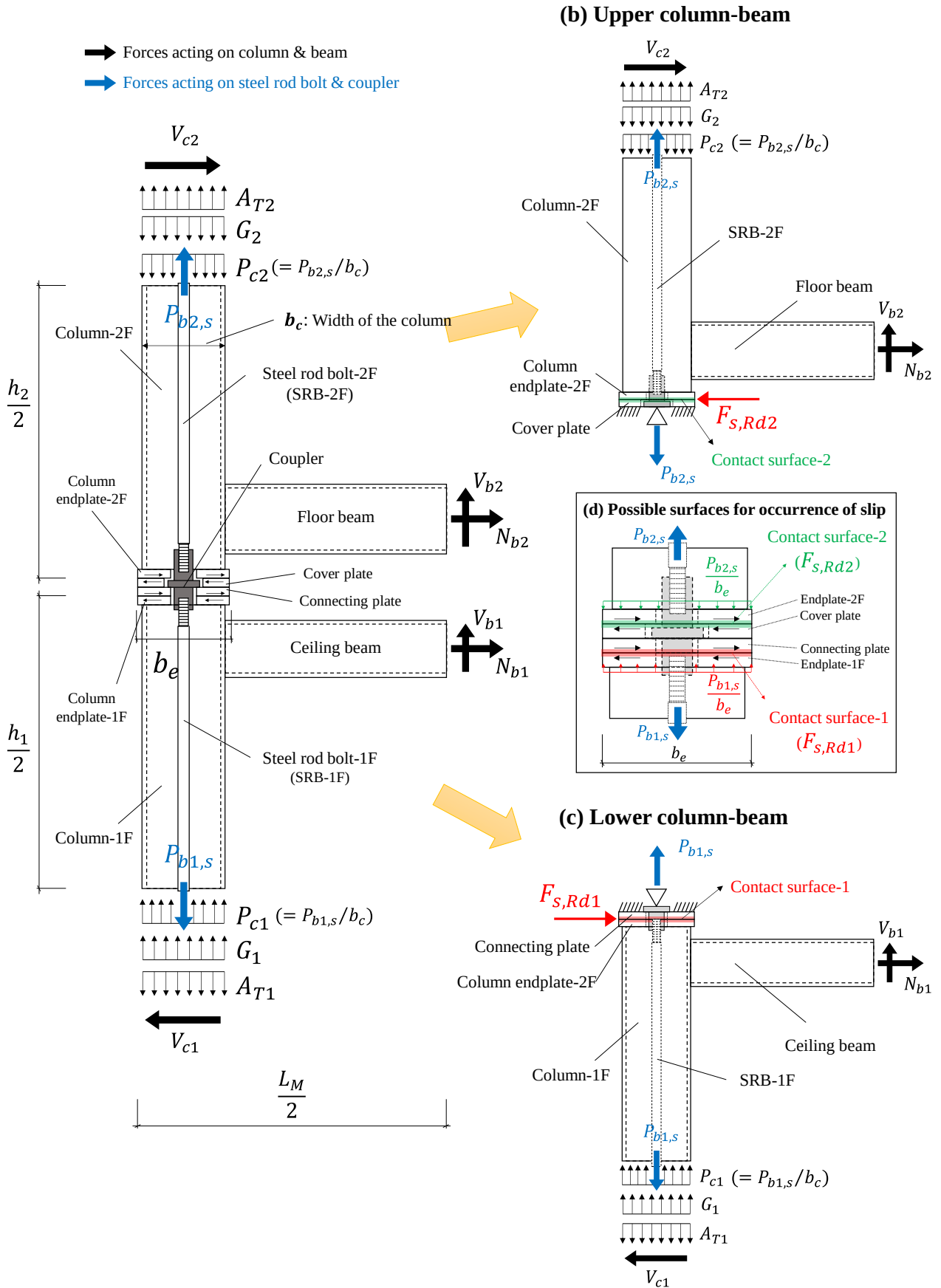


Figure 3.28: Design forces for calculating the demand preload (P_s) to prevent slip at AJ connection (refer to Figure 3.24 for nomenclature and dimensions)

Step 1: Demand preload ($P_{b2,s}$) for the upper column-beam

Figure 3.29 (a) shows all forces acting on column-2F, floor beam, and SRB-2F. Figure 3.29 (b) presents the free body diagram of the upper column-beam to obtain the demand preload $P_{b2,s}$ for preventing slip on contact surface-2. The slip resistance $F_{s,Rd2}$ at contact surface-2 should be greater than the design ultimate shear load $F_{v,Ed2}$ as follows:

$$F_{s,Rd2} \geq F_{v,Ed2} = V_{c2} + N_{b2} \quad (\text{Eq. 3.32})$$

For the slip resistance $F_{s,Rd2}$, the friction coefficient μ_2 is determined by the level of roughness of the surfaces between the column endplate-2F and the cover plate. The compressive normal force $F_{N,c2}$ applied to the surfaces is given by (Eq. 3.34). It should be noted that the shear force on the floor beam V_{b2} reduces the normal force.

$$F_{s,Rd2} = \mu_2 \cdot F_{N,c2} \quad (\text{Eq. 3.33})$$

$$F_{N,c2} = P_{b2,s} - N_{c2} - V_{b2} \quad (\text{Eq. 3.34})$$

$$F_{s,Rd2} = \mu_2 \cdot (P_{b2,s} - N_{c2} - V_{b2}) \quad (\text{Eq. 3.35})$$

By substituting (Eq. 3.35) for $F_{s,Rd2}$ in (Eq. 3.32), the demand preload $P_{b2,s}$ for preventing slip at contact surface-2 becomes:

$$P_{b2,s} \geq \frac{V_{c2} + N_{b2}}{\mu_2} + N_{c2} + V_{b2} \quad (\text{Eq. 3.36})$$

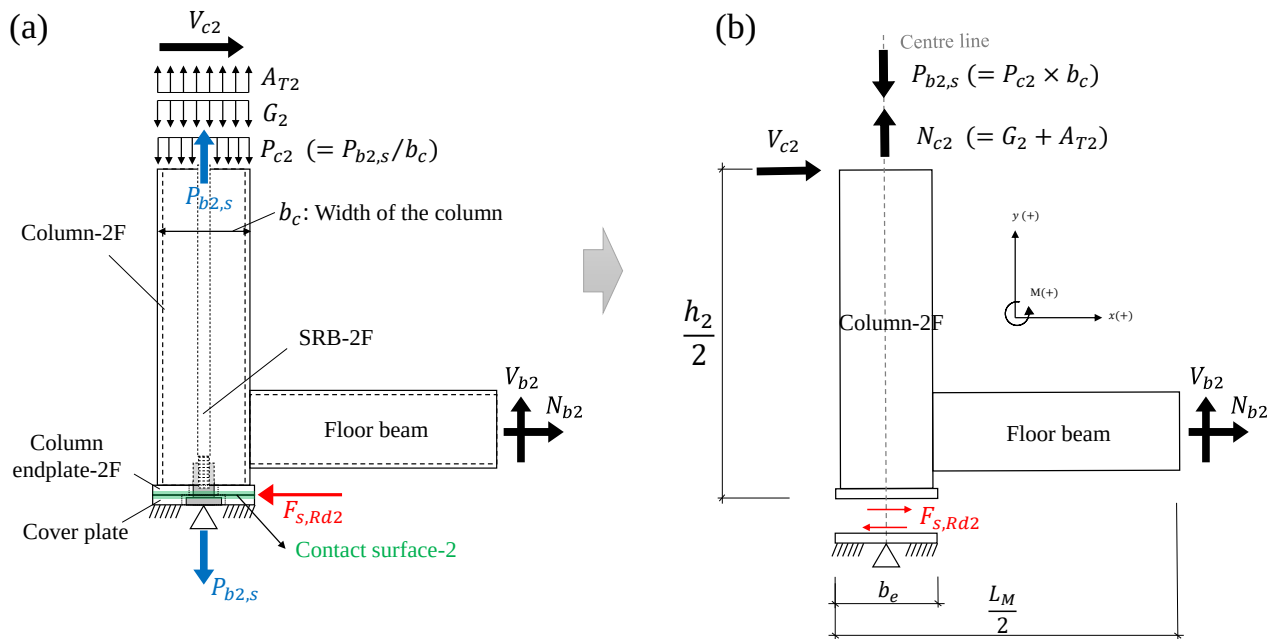


Figure 3.29: Free body diagram of the upper column-beam for demand preload ($P_{b2,s}$) to prevent slip on contact surface-2 (refer to Figure 3.24 for nomenclature and dimensions)

Step 2: Demand preload ($P_{b1,s}$) for the lower column-beam

Figure 3.30 (a) shows all forces acting on column-1F, ceiling beam, and SRB-1F. Figure 3.30 (b) presents the free body diagram of the upper column-beam to obtain the demand preload $P_{b1,s}$ for preventing the slip on contact surface-1 at the connection. The slip resistance $F_{s,Rd1}$ at contact surface-1 should be greater than the design ultimate shear load $F_{v,Ed1}$:

$$F_{s,Rd1} \geq F_{v,Ed1} = V_{c1} - N_{b1} \quad (\text{Eq. 3.37})$$

For the slip resistance $F_{s,Rd1}$, the friction coefficient μ_1 is determined by the level of roughness of the surfaces between the column endplate-1F and the connecting plate. The compressive normal force $F_{N,c1}$ applied to the surfaces is given by (Eq. 3.39). It should be noted that the shear force on the ceiling beam V_{b1} increases the normal force.

$$F_{s,Rd1} = \mu_1 \cdot F_{N,c1} \quad (\text{Eq. 3.38})$$

$$F_{N,c1} = P_{b1,s} - N_{c1} + V_{b1} \quad (\text{Eq. 3.39})$$

$$F_{s,Rd1} = \mu_1 \cdot (P_{b1,s} - N_{c1} + V_{b1}) \quad (\text{Eq. 3.40})$$

By substituting (Eq. 3.40) for $F_{s,Rd1}$ into (Eq. 3.37), the demand preload $P_{b1,s}$ for preventing the slip on contact surface-1 becomes:

$$P_{b1,s} \geq \frac{V_{c1} - N_{b1}}{\mu_1} + N_{c1} - V_{b1} \quad (\text{Eq. 3.41})$$

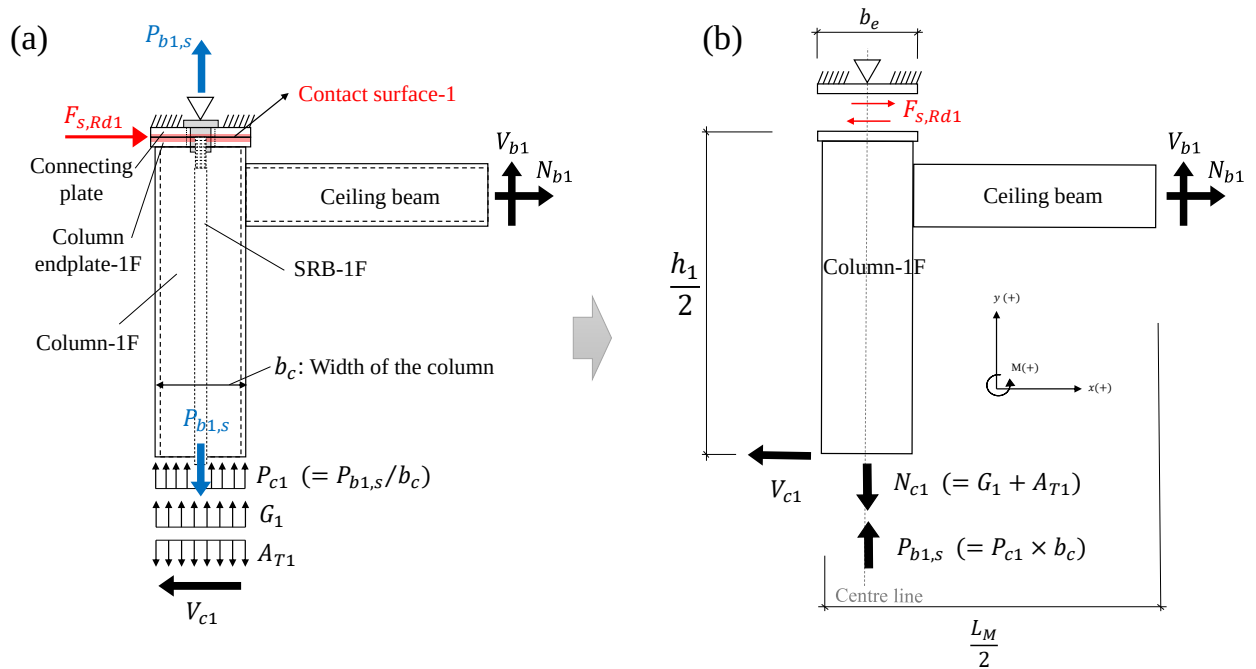


Figure 3.30: Free body diagram of the lower column-beam for demand preload ($P_{b1,s}$) to prevent slip on the contact surface-1 (refer to Figure 3.24 for nomenclature and dimensions)

Step 3: Determination of the demand preload (P_s) for the connection

The demand preload (P_s) for preventing slip at the connection is determined as the greater of $P_{b1,s}$ (for the lower column) or $P_{b2,s}$ (for the upper column). This preload (P_s) is applied to both the lower and upper columns.

$$P_s = \text{Max} (P_{b1,s}, P_{b2,s}) \quad (\text{Eq. 3.42})$$

3.4.5. Weight Effect of Upper Modules on Preload in the Lower Column

3.4.5.1. Reduction in Preload Due to the Weight of Upper Modules

In the previous Sections 3.4.3 and 3.4.4, the demand preloads to prevent gap-opening (P_g) and slip (P_s) at the AJ connection were determined without accounting for the effect of the weight from the upper modules. If a module is stacked on another module, the columns in the lower module incur compressive axial deformation due to the weight of the upper module. It causes the stretched length of the SRB within the lower column to decrease, resulting in a reduction of preload in the lower module. In the case of the AJ connection located on the lower floor of a multi-storey modular building, the reduction in preload is further increased because of the accumulated weight of the upper modules. Therefore, in order to maintain the demand preload (P_g or P_s) at the connection after completing the construction of the building, the reduction in preload due to the weight of the upper modules must be accounted for. This section explains the process of reducing the preload in the lower column due to the weight of the upper modules and determines the final reduced preload.

Figure 3.31 (a) shows one of the columns in a module. The column with the SRB can be represented by two springs: (i) bolt part consisting of the SRB and the coupler; and (ii) column part consisting of the column and connected plates. The axial stiffness of the bolt part k_B and the column part k_M are determined by (Eq. 3.44) and (Eq. 3.45), respectively, based on the axial stiffness of each member through (Eq. 3.43). For the bolt part, k_B is approximately the same as the axial stiffness of the bolt shank $k_{b,sh}$ because the bolt shank is much longer than the length of the bolt thread and coupler. So, the axial stiffness of the bolt part can be assumed as $k_{b,sh}$. For the same reason, the axial stiffness of the column part k_M can be approximated by the stiffness of the column k_{Col} .

$$k = \frac{EA}{L} \quad (\text{Eq. 3.43})$$

where: k The axial stiffness of a member; E Young's modulus;
 A The cross-sectional area of a member; L Length of a member.

$$\frac{1}{k_B} = \frac{1}{k_{b,th}} + \frac{1}{k_{b,sh}} + \frac{1}{k_{Coup}} \approx \frac{1}{k_{b,sh}} \quad (\text{For bolt part}) \quad (\text{Eq. 3.44})$$

$$\frac{1}{k_M} = \frac{1}{k_{Cp}} + \frac{1}{k_{Ep}} + \frac{1}{k_{Col}} + \frac{1}{k_{Ep}} \approx \frac{1}{k_{Col}} \quad (\text{For column part}) \quad (\text{Eq. 3.45})$$

k : axial stiffness k_B for bolt part; $k_{b,th}$ for SRB thread; $k_{b,sh}$ for SRB shank; k_{coup} for coupler
 k_M for column part; k_{cp} for connecting plate; k_{Ep} for endplate, k_{col} for column

When a preload $P_{i,1F}$ has been applied to SRB-1F (see Figure 3.31 (b)), the bolt part is elongated by $\delta_{B,p}$ while column-1F is shortened by $\delta_{M,p}$. The magnitude of the preload $P_{i,1F}$ is determined by the elongated length of the bolt $\delta_{B,p}$, and thus it can be calculated by (Eq. 3.46). The shortened length of the column part $\delta_{M,p}$ due to $P_{i,1F}$ is obtained by (Eq. 3.47).

$$P_{i,1F} = k_B \times \delta_{B,p} \quad (\text{Eq. 3.46})$$

$$\delta_{M,p} = \frac{P_{i,1F}}{k_M} \quad (\delta_{M,p} \neq \delta_{B,p}) \quad (\text{Eq. 3.47})$$

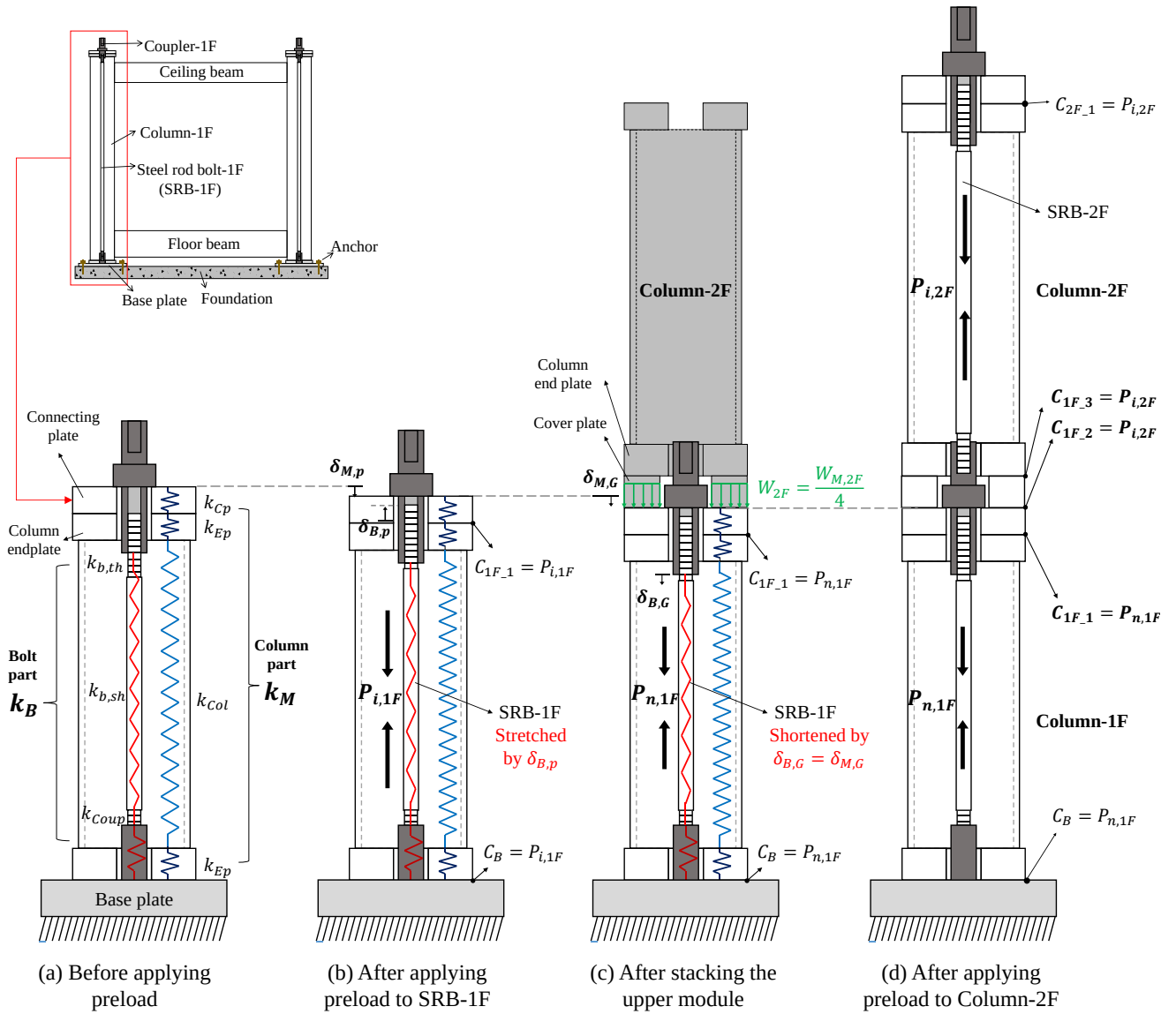


Figure 3.31: Change of the preload in the lower column due to the weight of the upper module

Then, as shown in Figure 3.31 (c), if the column of an upper module is stacked on column-1F having the preload $P_{i,1F}$, one-fourth of the weight of the upper module $W_{M,2F}$ (if a module has four columns) is transferred through only the column part because column-2F is placed on column-1F without transferring load to coupler-1F. So, column-1F undergoes compressive axial deformation of $\delta_{M,G}$ as follows,

$$\delta_{M,G} = \frac{W_{2F}}{k_M} \quad (W_{2F} = 1/4W_{M,2F}) \quad (\text{Eq. 3.48})$$

which causes the stretched length of the bolt part to be reduced by the same amount ($\delta_{B,G}$):

$$\delta_{B,G} = \delta_{M,G} \quad (\text{Eq. 3.49})$$

In turn, this reduction ($\delta_{B,G}$) of the initial elongation of $\delta_{B,p}$ causes a reduction of the initial preload $P_{i,1F}$ in SRB-1F, which implies a decrease in the clamping force (C_{1F_1}) on the contact surfaces between the connecting plate and the endplate at the connection. The reduction of preload $P_{R,1F}$ in SRB-1F can be calculated as:

$$P_{R,1F} = k_B \times \delta_{B,G} \quad (\text{Eq. 3.50})$$

Using (Eq. 3.48) and (Eq. 3.49), the reduction in preload $P_{R,1F}$ can be expressed in terms of the ratio of the axial stiffness of the bolt part to that of the column part and the weight acting on the column:

$$P_{R,1F} = \frac{k_B}{k_M} \times W_{2F} \quad (\text{Eq. 3.51})$$

Consequently, after the upper module is stacked on the lower module, the remaining preload $P_{n,1F}$ (or net preload) in column-1F is obtained by subtracting the reduced preload $P_{R,1F}$ from the initial preload $P_{i,1F}$:

$$\begin{aligned} P_{n,1F} &= P_{i,1F} - P_{R,1F} \\ &= P_{i,1F} - \left(\frac{k_B}{k_M} \times W_{2F} \right) \end{aligned} \quad (\text{Eq. 3.52})$$

Also, the clamping force C_{1F_1} at the connection is reduced from $P_{i,1F}$ to $P_{n,1F}$ due to the weight of the upper module.

Figure 3.31 (d) presents the clamping forces at the connection after applying preload $P_{i,2F}$ to SRB-2F

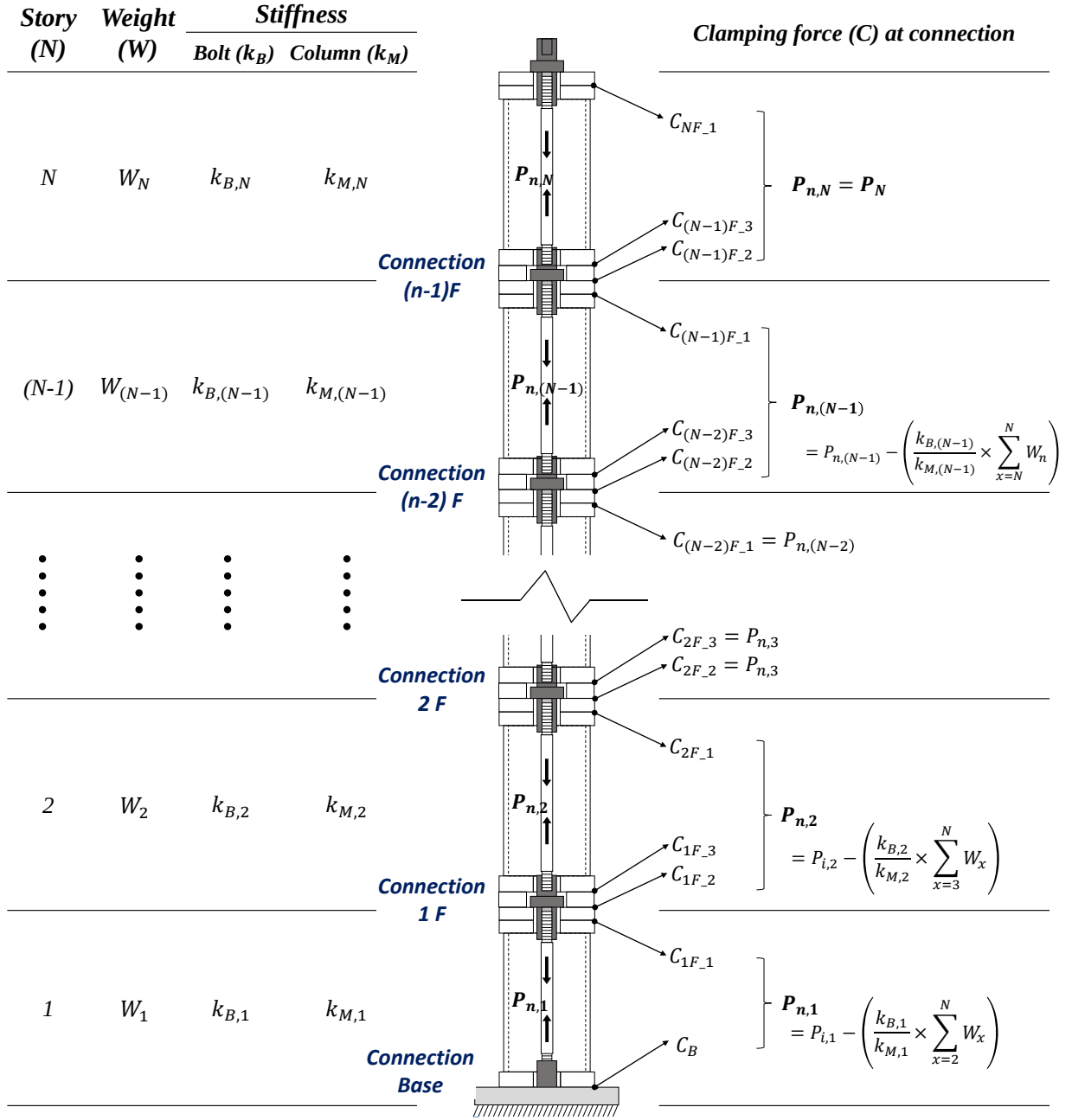
for connecting the upper column to the lower column. Unless another module is stacked on the top of column-2F, the clamping forces of C_{1F_2} (between the connecting plate and cover plate) and C_{1F_3} (between the cover plate and endplate) at the connection become $P_{i,2F}$ without any reduction.

In summary, the reduction in preload occurs as the initially stretched length of the SRB by preload decreases. When the column has compressive axial deformation due to the weight of the upper modules, the SRB within the column also becomes shortened by the same amount of the deformation of the column. In other words, the reduced preload is dependent on the amount of compressive axial deformation of the column.

In general, if the axial stiffness of the column is large, its compressive deformation is small and hence the reduction in preload is small. Conversely, if the axial stiffness of the bolt is large, the reduction in preload becomes large. The influence of the axial stiffness of the column and bolt parts on the reduction in preload is determined by (Eq. 3.51). It follows that since the column and the SRB have almost the same Young's modulus E and length, the ratio of their cross-sectional areas determines the reduced preload amount. It should be noted that if two SRBs are used in the column, which would typically be the case if a H-column is used, the reduction in preload is doubled under the same weight of the upper modules. In this case, in order to avoid significant loss of preload, the axial stiffness of the SRBs should be reduced by using a smaller diameter of the bolts.

Figure 3.32 presents one of the columns in a module of an N -storey modular building. The reduction in preload at each storey due to the weight of the upper modules can be obtained through (Eq. 3.53). The remaining preload in the SRB at each storey, which is the net preload, is expressed as (Eq. 3.54). Once the net preloads in the upper and lower columns have been obtained, the net clamping forces C_{NF} at the connection by preload can be found (e.g. Connection 1F in Figure 3.32: $C_{1F_1}=P_{n,1}$ and $C_{1F_2}=C_{1F_3}=P_{n,2}$). It is noted that the connection located on the lowest floor has the most significant reduction in preload.

To better understand the weight effect of upper modules in the multi-storey modular building, a sample calculation is provided in Appendix A, using the 3.3m (W) X 3m (H) X 6.6m (L) module frequently used in South Korea. Typically, the weight of this module is around 12-14 tons. In case a 12-storey modular building is constructed using this module, it is expected that an initially applied preload of 150kN in the connection located on the lowest floor will be reduced by about 20% to 121kN. Further details are provided in the sample calculation. Note that compared to Australian modules, Korean modules are generally smaller but heavier because of the thicker concrete floor slab of over 150mm.



$$P_{R,x} = \left(\frac{k_{B,x}}{k_{M,x}} \times \sum_{n=x+1}^N W_n \right) \quad (\text{Eq. 3.53})$$

$$P_{n,x} = P_{i,x} - P_{R,x} = P_{i,x} - \left(\frac{k_{B,x}}{k_{M,x}} \times \sum_{n=x+1}^N W_n \right) \quad (\text{Eq. 3.54})$$

where: $P_{n,x}$ Remaining preload (= net preload) at x^{th} storey due to the weight of the upper modules;
 $P_{R,x}$ Reduced preload at x^{th} storey due to the weight of the upper modules;
 $P_{i,x}$ Initial preload at x^{th} storey;
 $k_{B,x}$ The axial stiffness of bolt part at x^{th} storey by (Eq. 3.44);
 $k_{M,x}$ The axial stiffness of column part at x^{th} storey by (Eq. 3.45);
 W_n Weight of the x^{th} storey module delivered to the column below;
 N The highest storey of the modular building.

Figure 3.32: Calculation of remaining preload at connections in multi-storey buildings due to the weight of upper modules

3.4.5.2. Final Demand Preload Considering the Weight Effect of Upper Modules

The demand preloads required to prevent gap-opening P_g and slip P_s are determined by (Eq. 3.29) and (Eq. 3.42), respectively. In order to maintain these preloads after completing the construction of the modular building, the reduction in preload (P_R) due to the weight of the upper modules must be added to P_g and P_s . Therefore, the final demand preloads to prevent gap-opening ($P_{F,g}$) and slip ($P_{F,s}$) at the connections are as follows:

- Final demand preload ($P_{F,g}$) for preventing gap-opening:

$$\begin{aligned} P_{F,g} &= P_g + P_R \\ &= P_g + \left(\frac{k_{B,x}}{k_{M,x}} \times \sum_{n=x+1}^N W_n \right) \end{aligned} \quad (\text{Eq. 3.55})$$

- Final demand preload ($P_{F,s}$) for preventing slip:

$$\begin{aligned} P_{F,s} &= P_s + P_R \\ &= P_s + \left(\frac{k_{B,x}}{k_{M,x}} \times \sum_{n=x+1}^N W_n \right) \end{aligned} \quad (\text{Eq. 3.56})$$

where:	P_g	Demand preload for preventing gap-opening at the AJ connection by (Eq. 3.29);
	P_s	Demand preload for preventing slip at the AJ connection by (Eq. 3.42);
	P_R	Reduced preload at x^{th} storey due to the weight of the upper modules;
	$k_{B,x}$	The axial stiffness of bolt part at x^{th} storey by (Eq. 3.44);
	$k_{M,x}$	The axial stiffness of column part at x^{th} storey by (Eq. 3.45);
	W_n	Weight of the x^{th} storey module delivered to the column below;
	N	Number of storeys of the modular building.

3.5. Effect of Gap-opening on the Behaviour of the AJ Joint

This section focuses on the potential gap-opening caused by shear forces acting on columns and beams at the AJ connection. The following is investigated:

- The effect of module geometry (i.e. difference in the sizes of floor and ceiling beams, height and width of the module) on the gap-opening and demand preload.
- The location where gap-opening will occur at the connection and calculating the critical lateral force at which the induced preload can prevent gap-opening ($F_{cr,g}$). $F_{cr,g}$ is used as the analytically estimated strength of the occurrence of gap-opening to compare it with the test and simulation results.
- The behaviour of the preload beam-to-column joint (i.e. load bearing and failure mechanism) in case a gap-opening occurs at the connection.

3.5.1. Shear Forces on the Floor Beam (V_{fb}) and Ceiling Beam (V_{cb})

If gap-opening occurs at the AJ connection, the behaviour of the beam-to-column joint differs depending on the location of the gap-opening at the connection. Therefore, in order to analytically predict the effect of the gap-opening on the preload beam-to-column joint, where the gap-opening occurs needs to be first determined.

When modules are assembled on site, a floor beam of an upper module and a ceiling beam of a lower module form a double beam. Figure 3.24 (a) presents the bending moment diagram of a two-storey steel-framed modular building as a lateral force is applied. Figure 3.33 shows the deformed shape of the subassembly based on the assumption that the inflection points are located at beam midspans and column mid-heights. The potential positions for the gap-opening are points A, B, C and D of the connection. To find the critical location, the shear forces acting on the floor beam V_{fb} and the ceiling beam V_{cb} due to the lateral force V_{c2} must be known. The shear forces (V_{fb} , V_{cb}) cannot be statically determined because the presence of two beams makes the joint subassembly statically indeterminate, but can be determined through the following reasonable assumption:

- The floor and ceiling beams have the same deflection if the connection is rigid (see Figure 3.33).

With this assumption, the total shear force acting on the two beams V_B can be written as follows:

$$V_B = V_{fb} + V_{cb} \quad (\text{Eq. 3.57})$$

$$V_B = \frac{V_{c2} \cdot H}{(L/2)} \quad (\text{Eq. 3.58})$$

The deflection of the floor beam Δ_{fb} and ceiling beam Δ_{cb} caused by the shear forces (V_{fb} , V_{cb}) can be computed based on linear theory using (Eq. 3.59). According to the assumption that the deflections of the two beams are the same, the relation between the shear forces (V_{fb} , V_{cb}) acting on the beams can be expressed as (Eq. 3.60).

$$\Delta_{fb} = \frac{V_{fb} \cdot L_b^3}{3 \cdot E_{fb} \cdot I_{fb}} \quad \Delta_{cb} = \frac{V_{cb} \cdot L_b^3}{3 \cdot E_{cb} \cdot I_{cb}} \quad (\text{cantilever deflection formula}) \quad (\text{Eq. 3.59})$$

$$\Delta_{fb} = \Delta_{cb} \rightarrow \frac{V_{fb} \cdot L_b^3}{3 \cdot E_{fb} \cdot I_{fb}} = \frac{V_{cb} \cdot L_b^3}{3 \cdot E_{cb} \cdot I_{cb}} \rightarrow \frac{V_{fb}}{E_{fb} \cdot I_{fb}} = \frac{V_{cb}}{E_{cb} \cdot I_{cb}} \quad (\text{Eq. 3.60})$$

* E_{fb} and E_{cb} : Young's modulus for the floor beam and the ceiling beam, respectively

* I_{fb} and I_{cb} : Second moment of area for the floor beam and the ceiling beam, respectively

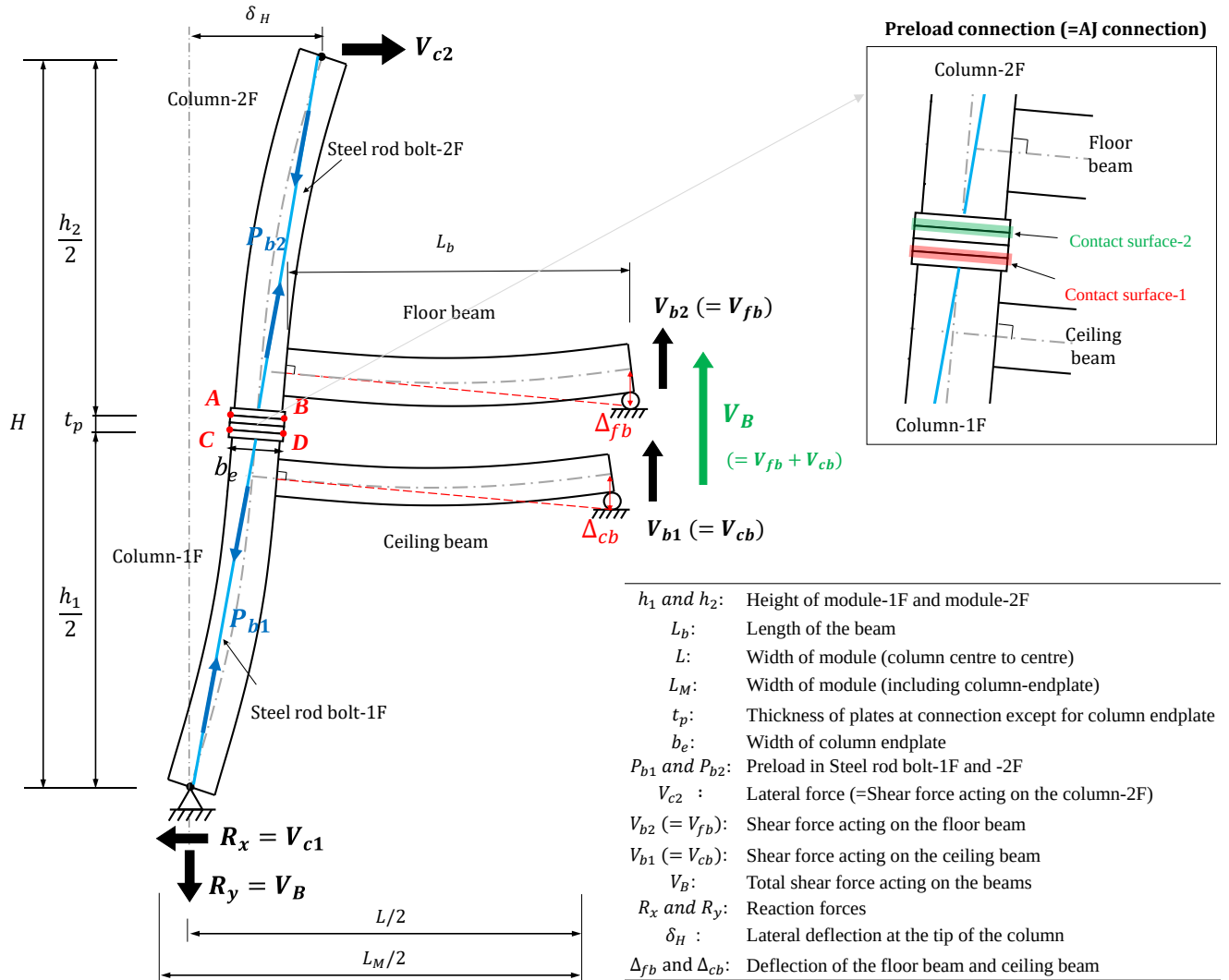


Figure 3.33: Preload beam-to-column joint with potential locations for gap-opening (A, B, C, D) and slip

Using (Eq. 3.57) and (Eq. 3.60), the expressions for the shear forces (V_{fb} , V_{cb}) acting on the floor and ceiling beams can be obtained from (Eq. 3.61) and (Eq. 3.62). These equations show that the shear forces acting on the two beams are determined by the Young's modulus (E) and second moment of area (I) of the beams. In case both beams are made of the same material, the shear forces acting on the beams are determined solely by their second moments of area.

$$V_{fb} = \left(\frac{E_{fb} \cdot I_{fb}}{E_{fb} \cdot I_{fb} + E_{cb} \cdot I_{cb}} \right) \cdot V_B \rightarrow V_{fb} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot V_B \quad (\text{if } E_{fb} = E_{cb}) \quad (\text{Eq. 3.61})$$

$$V_{cb} = \left(\frac{E_{cb} \cdot I_{cb}}{E_{fb} \cdot I_{fb} + E_{cb} \cdot I_{cb}} \right) \cdot V_B \rightarrow V_{cb} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot V_B \quad (\text{if } E_{fb} = E_{cb}) \quad (\text{Eq. 3.62})$$

Consequently, of the two beams, the one with the larger cross-sectional size or deeper depth (i.e. the beam with the greater flexural stiffness EI) will be subjected to a relatively larger shear force.

3.5.2. Effect of Difference in Size Between the Floor and Ceiling Beams on Gap-opening

In general, since the floor beam serves as a more important structural member than the ceiling beam in a modular building, the cross-sectional size of the floor beam is larger than that of the ceiling beam. It means that the floor beam has greater flexural stiffness EI and hence is subjected to a relatively larger shear force than the ceiling beam, according to Section 3.5.1. Those unbalanced shear forces acting on the two beams adversely affect the gap-opening at the AJ connection.

With the shear forces acting on the floor beam V_{fb} and the ceiling beam V_{cb} in accordance with (Eq. 3.61) and (Eq. 3.62), it is possible to predict the location of the gap-opening and determine the effect of the difference in the cross-sectional size between the two beams on the gap-opening. Figure 3.35 shows three cases of the beam-to-column joint subjected to lateral force V_{c2} ($=35\text{kN}$) on the top of the column and the expected gap-opening at the AJ connection, depending on the difference in cross-sectional size between the floor and ceiling beams when the two modules-2F and -1F have the same height h and width L_M . The demand preload for preventing the gap-opening for each case is also presented (calculated according to Section 3.4.3) to better understand the effect of the difference in the size of the two beams.

As an example, Figure 3.34 presents the detailed calculation for Case 1 shown in Figure 3.35, for which the floor beam has a larger cross-section than the ceiling beam. There are four potential locations A, B, C, and D for the gap-opening at the AJ connection. The location of the gap-opening can be approximately predicted using the moment equilibrium at point O for the column 2F-floor beam and

point O' for the column 1F-ceiling beam. (The precise prediction is described in Section 3.5.3). If the moments $M_{c2,o}$ and $M_{fb,o}$ caused by the shear forces acting on the column 2F-floor beam do not satisfy moment equilibrium at point O, gap-opening will occur at points A or B depending on the rotational direction of the larger moment between them. In the same way, for the column 1F-ceiling beam, gap-opening will occur at points C or D if the moments $M_{c1,o'}$ and $M_{cb,o'}$ do not satisfy moment equilibrium at point O'. The gap-opening occurs finally where the moment imbalance is more severe between the column 2F-floor beam and the column 1F-ceiling beam.

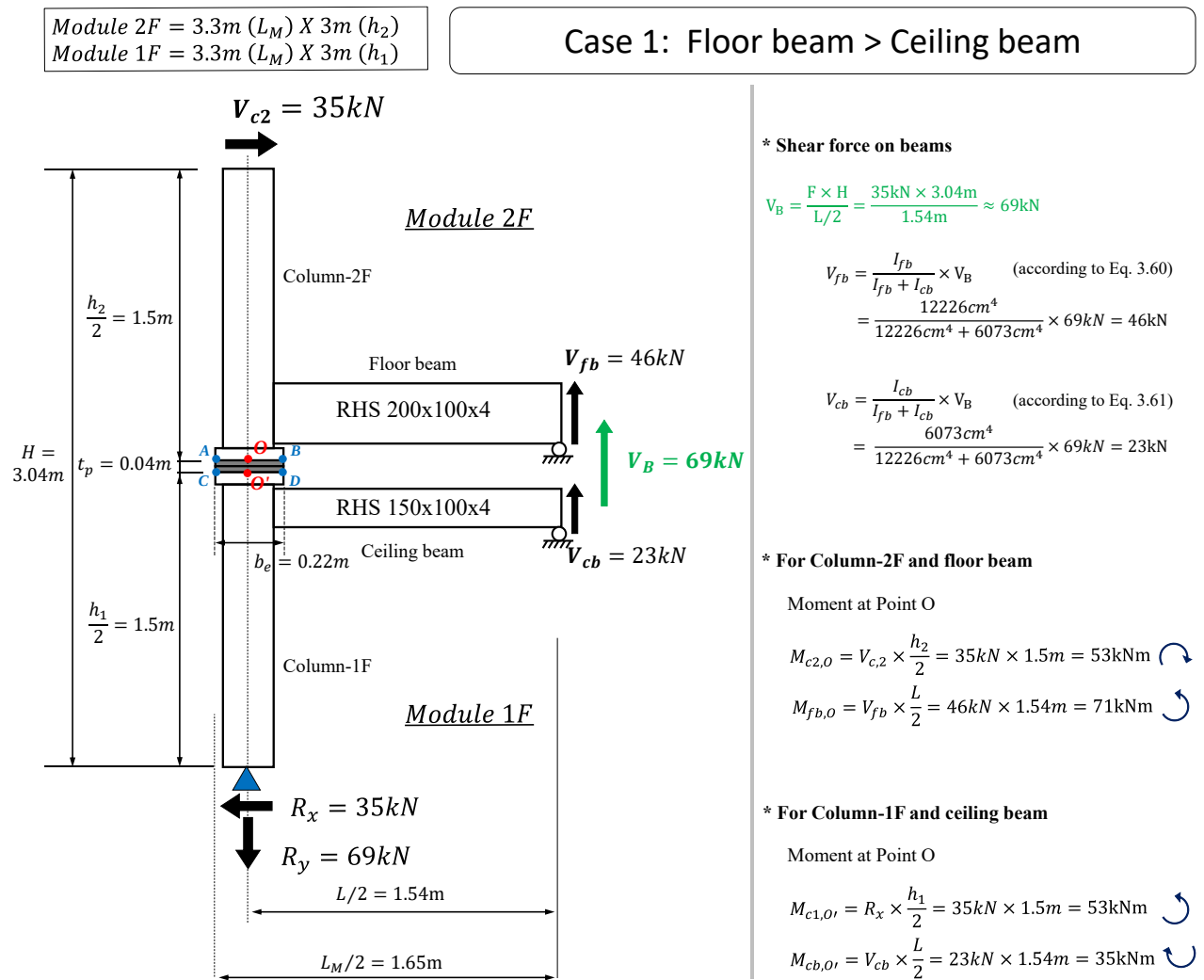


Figure 3.34: Detailed calculation to find the gap-opening location for Case 1 of Figure 3.35 using moments at O and O'

In the three cases shown in Figure 3.35 (a), (b), and (c), the same lateral force V_{c2} ($=35kN$) is applied but the resulting shear forces acting on the two beams are different. As a result, each case has a different gap-opening location, as shown in Figure 3.35 (d), (e), and (f). In case the cross-sectional size of the floor beam is greater than the ceiling beam (see Case 1 in Figure 3.35), the gap-opening occurs at Point B because the column 2F-floor beam has a more severe moment imbalance than the column 1F-ceiling beam. The demand preload for preventing the gap-opening at the connection becomes 215kN according

to (Eq. 3.29). On the other hand, if the cross-sectional size of the ceiling beam is larger than that of the floor beam (see Case 2 in Figure 3.35), the gap-opening will occur at Point C. Lastly, if both beams have the same cross-sectional size (see Case 3 in Figure 3.35), the shear forces acting on the two beams are equal. As a result, both column 2F-floor beam and column 1F-ceiling beam are in moment equilibrium at Point O and O', respectively, by the shear forces acting on the columns and the beams (see Figure 3.35 (f)). It means that the propensity of gap-opening is significantly reduced. The demand preload is 41kN and so is reduced by 80% compared to Case 1 (215kN).

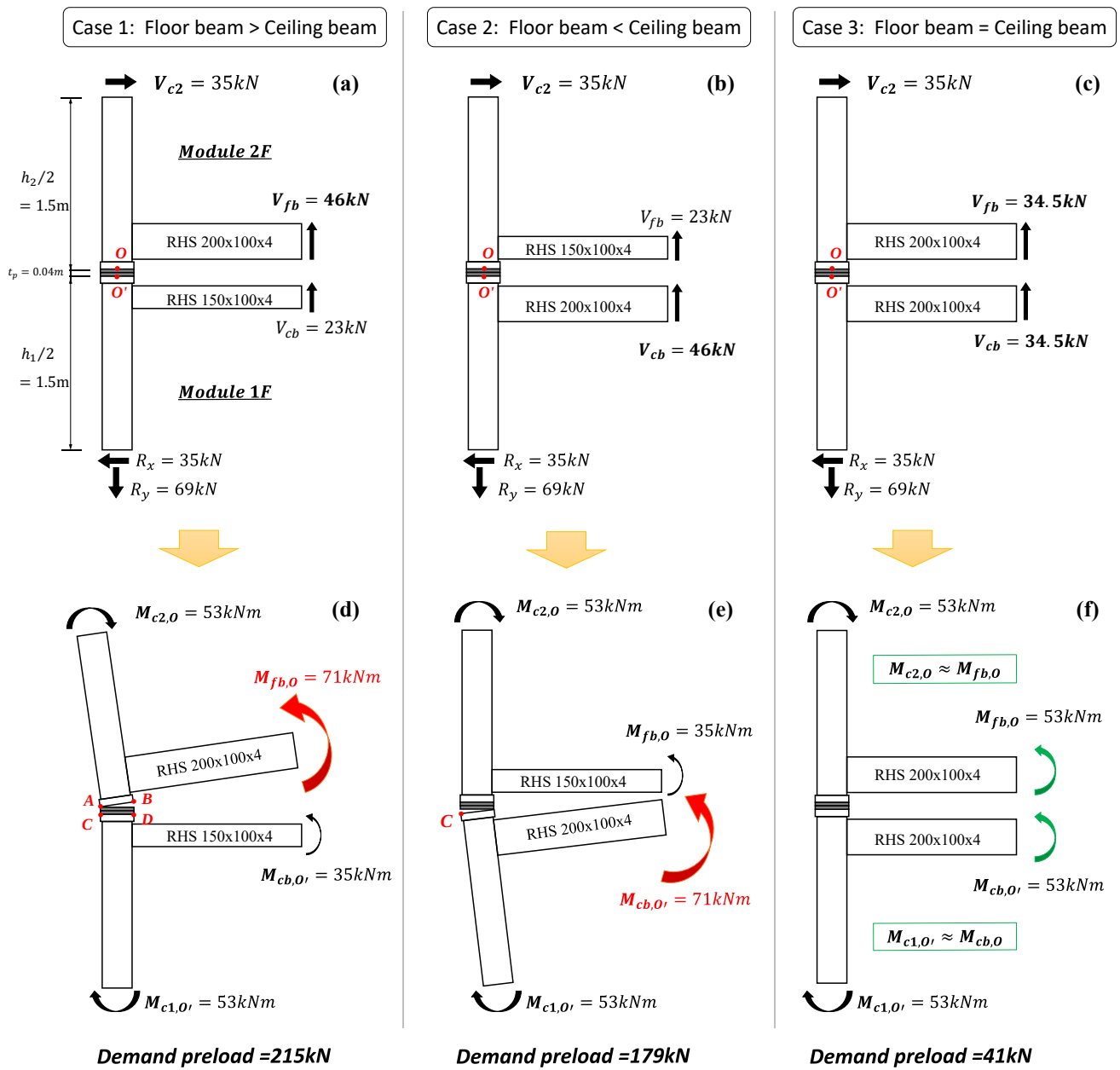


Figure 3.35: Gap-opening depending on the relative cross-sectional sizes of the floor and ceiling beams
(In case module-2F and module-1F have the same size: 3.3m (L_M) X 3m ($h_1=h_2$))

In case different heights are used for module-2F ($h_2=3\text{m}$) and module-1F ($h_1=4.5\text{m}$), the AJ connection is much more vulnerable to gap-opening because the moment imbalance in the column 2F-floor beam is more severe (see Figure 3.36 (b)), compared to when the two modules have the same height ($h_1=h_2=3\text{m}$), (Case 1 in Figure 3.35 (d)). The demand preload becomes 386kN which is 1.8 times greater than when equal heights are used as in Case 1. If the same cross-sectional size is used for the two beams (see Figure 3.36 (c)), this alleviates the moment imbalance and reduces the demand preload by over 50% to 169kN.

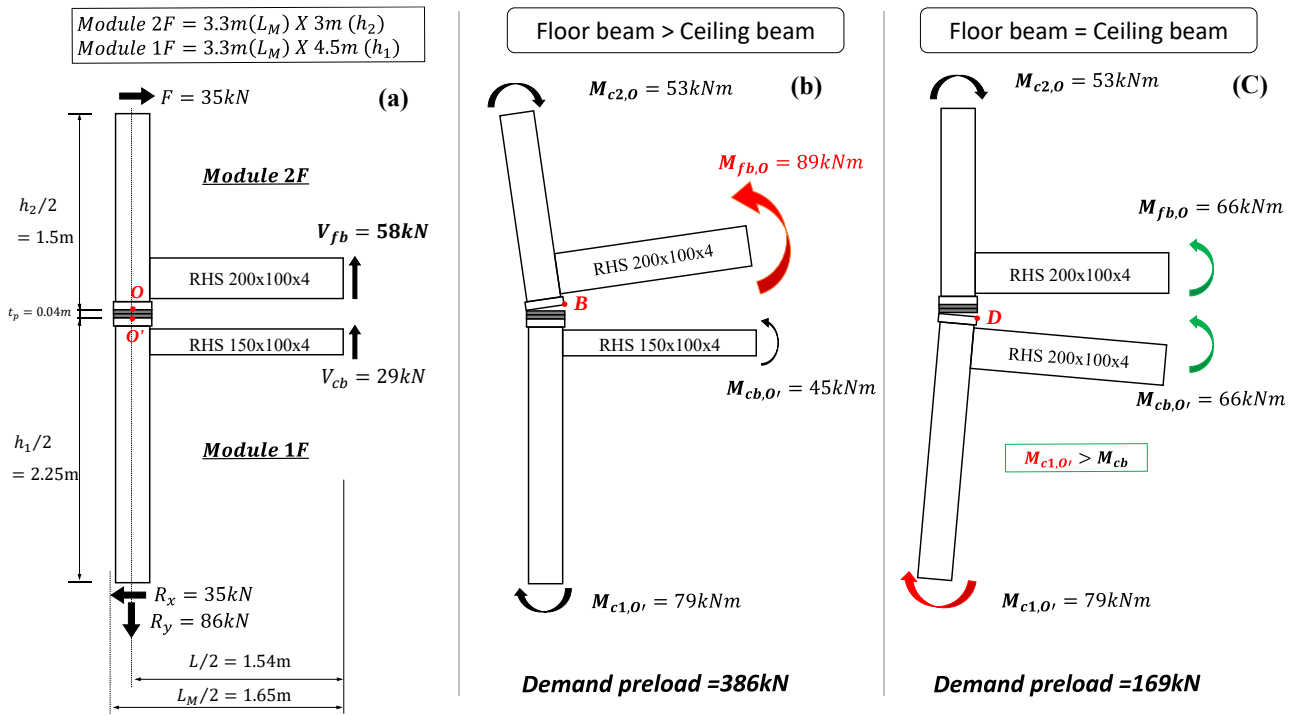


Figure 3.36: Gap-opening depending on the difference in the cross-sectional size of floor and ceiling beams (In case the module-2F ($h_2=3\text{m}$) and the module-1F ($h_1=4.5\text{m}$) have different height).

In case the width of the modules is increased to $L_M=4.5\text{m}$ (see Figure 3.37 (a)), the AJ connection becomes slightly less vulnerable to gap-opening because the shear forces acting on the beams are reduced under the same lateral force V_{c2} ($=35\text{kN}$). In case the cross-sectional size of the floor beam is greater than that of the ceiling beam (see Figure 3.37 (b)), the demand preload becomes 202kN which is a 7~8% reduced magnitude compared to Case 1 with $L_M=3.3\text{m}$ (see Figure 3.35 (d)). If the same cross-section is used for the two beams (see Figure 3.37 (c)), the demand preload becomes 31kN, which is a reduction of 85% compared to using different beam sizes.

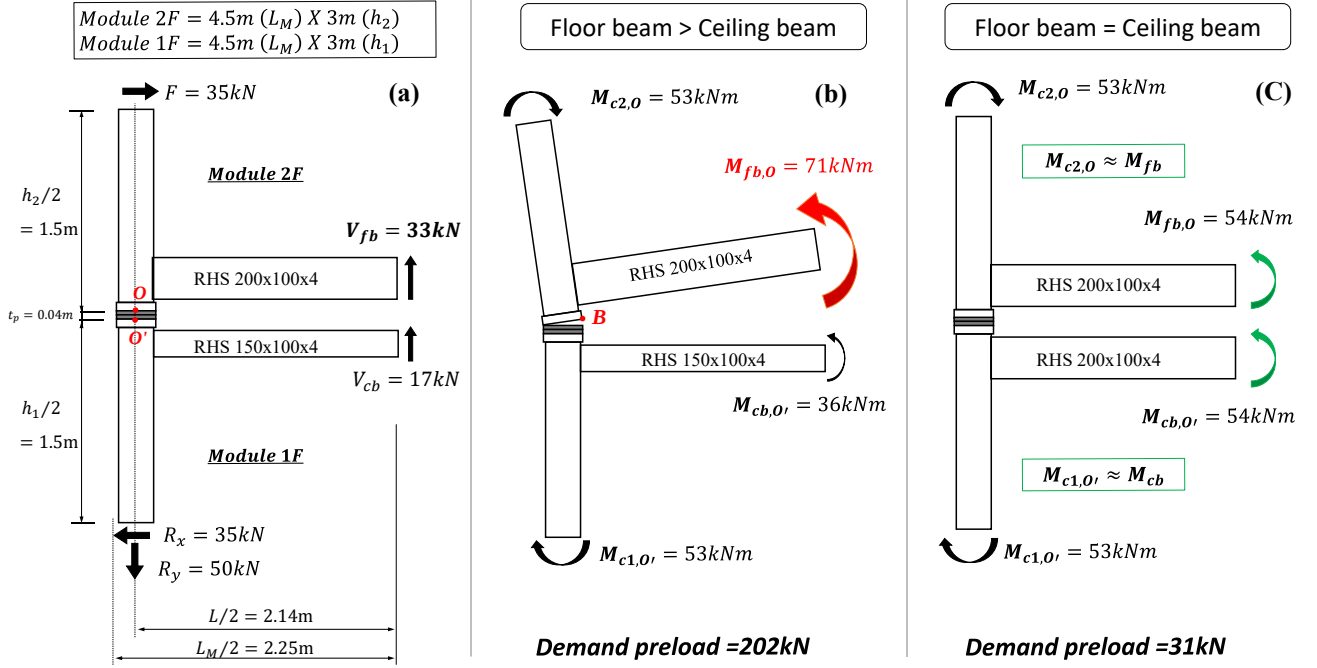


Figure 3.37: Gap-opening depending on the cross-sectional size of the floor and ceiling beams
 (In case the module-2F and the module-1F have the same size: 4.5m (L_M) X 3m ($h_1=h_2$)).

In closing, by reducing the difference in the cross-sectional size (i.e. flexural stiffness EI) of the floor and ceiling beams, the impact of gap-opening is minimised and the demand preload to prevent the gap-opening is reduced. In the event of an earthquake, the lower storeys in a multi-storey modular building are subjected to large lateral forces, which means that AJ connections are vulnerable to gap-opening and require very large demand preloads. In that case, it is a good strategy to use the same cross-sectional size of the floor and ceiling beams in the lower storeys. This is also an effective strategy for AJ connections between modules of different heights.

3.5.3. Critical Lateral Force for Gap-opening and Slip at the AJ Connection

Sections 3.4.3 and 3.4.4 present methods to determine the preload required to prevent gap-opening and slip in the column-to-column AJ connection. This section determines the critical strengths (i.e. lateral force or moment) preventing gap-opening and slip by induced preload for the beam-to-column joint. In other words, they are the analytically estimated strengths of the occurrence of gap-opening ($F_{cr,g}$) and slip ($F_{cr,s}$) of the beam-to-column joint. These critical strengths are compared with the test results and numerical simulation results in order to validate the suggested design method for determining the demand preload for the AJ connection, as described in Sections 3.4.3 and 3.4.4.

(It is noted that the beam-to-column joint test was conducted by applying a lateral force to the tip of the beams, as shown in Figure 4.40. Therefore, for comparison with test and simulation results, the critical strengths are expressed as the lateral force V_B applied to the beams.)

■ Critical lateral force ($F_{cr,g}$) for gap-opening at the AJ connection

The critical strength $F_{cr,g}$ is defined as the minimum lateral force at the tip of the beams V_B to cause the gap-opening at one of the four points A, B, C, and D at the AJ connection, as shown in Figure 3.33. In other words, it is the minimum lateral force to make the magnitude of the overturning moment M_{ov} equal to the stabilising moment M_{st} at one of the four points (refer to Section 3.4.2).

For example, suppose the gap-opening occurs at Point B (see Figure 3.33), implying that the overturning moment ($M_{ov,A}$) exceeds the stabilising moment ($M_{st,A}$) at Point A. Therefore, $F_{cr,g}$ becomes the lateral force to make $M_{ov,A}$ equal to $M_{st,A}$. These two moments have opposite rotational directions, so $F_{cr,g}$ is determined from the equation:

$$M_{ov,A} = -M_{st,A} \quad (\text{Eq. 3.63})$$

According to (Eq. 3.21), (Eq. 3.63) becomes:

$$\left(V_{b2} \cdot \frac{L_M}{2} + N_{c2} \cdot \frac{b_e}{2} \right) + \left(-N_{b2} \cdot d_{e2} - V_{c2} \cdot \frac{h_2}{2} - P_{b2} \cdot \frac{b_e}{2} \right) = 0 \quad (\text{Eq. 3.64})$$

V_{c2} and V_{b2} are determined as follows:

$$V_{c2} = V_B \cdot \frac{(L/2)}{H} \quad (\text{Eq. 3.65})$$

$$V_{b2} = V_{fb} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot V_B \quad \text{according to (Eq. 3.61)}$$

Substituting for V_{b2} and V_{c2} in (Eq. 3.64),

$$\left(\frac{I_{fb}}{I_{fb} + I_{cb}}\right) \cdot V_B \cdot \frac{L_M}{2} + N_{c2} \cdot \frac{b_e}{2} - N_{b2} \cdot d_{e2} - V_B \cdot \frac{(L/2)}{H} \cdot \frac{h_2}{2} - P_{b2} \cdot \frac{b_e}{2} = 0 \quad (\text{Eq. 3.66})$$

Then, solve (Eq. 3.66) for V_B , which becomes the critical lateral force $F_{cr,g,B}$ for the gap-opening at point B.

$$V_B = F_{cr,g} = \frac{N_{b2} \cdot d_{e2} + P_{b2} \cdot \frac{b_e}{2} - N_{c2} \cdot \frac{b_e}{2}}{\left(\frac{I_{fb}}{I_{fb} + I_{cb}}\right) \cdot \frac{L_M}{2} - \frac{(L/2)}{H} \cdot \frac{h_2}{2}} \quad (\text{Eq. 3.67})$$

In case axial forces are not applied to the columns and beams, as shown in Figure 3.33, $F_{cr,g,B}$ simplifies to:

$$V_B = F_{cr,g,B} = \frac{P_{b2} \cdot \frac{b_e}{2}}{\left(\frac{I_{fb}}{I_{fb} + I_{cb}}\right) \cdot \frac{L_M}{2} - \frac{(L/2)}{H} \cdot \frac{h_2}{2}} \quad (\text{if } N_{b2} = N_{c2} = 0) \quad (\text{Eq. 3.68})$$

Table 3.3 presents the critical lateral forces acting on the beams ($V_B = F_{cr,g}$), causing gap-opening at each of the possible points (A,B,C,D) of the AJ connection (see Figure 3.33). The minimum positive force among them becomes the critical lateral force $F_{cr,g}$.

The corresponding critical moment strength $M_{cr,g}$ is calculated as follows:

$$M_{cr,g} = F_{cr,g} \times \frac{L}{2} \quad (\text{Eq. 3.69})$$

Table 3.3: Critical lateral force ($V_B = F_{cr,g}$) to cause gap-opening at the preload beam-to-column joint
(Refer to Figure 3.33 for dimensions and the directions of the forces acting on the beam-to-column joint)

Position of gap-opening at the connection		Critical lateral force for the gap-opening ($F_{cr,g} = V_B$ in Figure 3.33)
Upper column-floor beam	Point A	$F_{cr,g,A} = -\frac{P_{b2} \cdot \frac{b_e}{2}}{\left(\frac{I_{fb}}{I_{fb} + I_{cb}}\right) \cdot \left(\frac{L_M}{2} - b_e\right) - \frac{(L/2)}{H} \cdot \frac{h_2}{2}}$
	Point B	$F_{cr,g,B} = \frac{P_{b2} \cdot \frac{b_e}{2}}{\left(\frac{I_{fb}}{I_{fb} + I_{cb}}\right) \cdot \frac{L_M}{2} - \frac{(L/2)}{H} \cdot \frac{h_2}{2}}$
Lower column- ceiling beam	Point C	$F_{cr,g,C} = \frac{P_{b1} \cdot \frac{b_e}{2}}{\left(\frac{I_{cb}}{I_{fb} + I_{cb}}\right) \cdot \left(\frac{L_M}{2} - b_e\right) - \frac{(L/2)}{H} \cdot \frac{h_1}{2}}$
	Point D	$F_{cr,g,D} = -\frac{P_{b1} \cdot \frac{b_e}{2}}{\left(\frac{I_{cb}}{I_{fb} + I_{cb}}\right) \cdot \frac{L_M}{2} - \frac{(L/2)}{H} \cdot \frac{h_1}{2}}$

In case the axial forces acting on the columns N_c and the beams N_b are '0'

■ Critical lateral force ($F_{cr,s}$) for slip at the AJ connection

As shown in Figure 3.33 (refer also to Figure 3.28 (d)), the connection can have slip at contact surface-1 and surface-2 where the preloads P_{b1} and P_{b2} are applied, respectively. The critical strength for the slip $F_{cr,s}$ is defined as the minimum lateral force V_B at the tip of the beams to cause slip at one of the contact surfaces of the AJ connection. It can be obtained as follows:

(i) Lateral force ($F_{cr,s1}$) to cause slip at contact surface-1

As shown in Figure 3.30, slip occurs on contact surface-1 when the ultimate shear load $F_{v,Ed1}$ produced by the forces acting on column-1F and the ceiling beam is greater than the slip resistance $F_{s,Rd1}$. Thus, $F_{cr,s1}$ becomes the lateral force to make $F_{v,Ed1}$ equal to $F_{s,Rd1}$.

$$F_{v,Ed1} = F_{s,Rd1} \quad (\text{Eq. 3.70})$$

According to (Eq. 3.37) and (Eq. 3.40), (Eq. 3.70) can be written as:

$$V_{c1} - N_{b1} = \mu_1 \cdot (P_{b1,s} - N_{c1} + V_{b1}) \quad (\text{Eq. 3.71})$$

V_{c1} and V_{b1} are as follows :

$$\begin{aligned} V_{c1} &= V_{c2} = V_B \cdot \frac{(L/2)}{H} && \text{as per Figure 3.30} && (\text{Eq. 3.72}) \\ V_{b1} &= V_{cb} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot V_B && \text{according to (Eq. 3.62)} \end{aligned}$$

Substitute for V_{c1} and V_{b1} in (Eq. 3.71).

$$V_B \cdot \frac{(L/2)}{H} - N_{b1} = \mu_1 \cdot \left(P_{b1,s} - N_{c1} + \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot V_B \right) \quad (\text{Eq. 3.73})$$

Then, solve (Eq. 3.73) for V_B , which becomes the critical lateral force $F_{cr,s1}$ for the slip on contact surface-1.

$$F_{cr,s1} = V_B = \frac{\mu_1 \cdot (P_{b1,s} - N_{c1}) + N_{b1}}{\frac{(L/2)}{H} - \mu_1 \cdot \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right)} \quad (\text{Eq. 3.74})$$

In case axial forces are not applied to the columns and beams, as shown in Figure 3.33, $F_{cr,s1}$ simplifies to:

$$F_{cr,s1} = V_B = \frac{\mu_1 \cdot P_{b1,s}}{\frac{(L/2)}{H} - \mu_1 \cdot \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right)} \quad (\text{if } N_{b1} = N_{c1} = 0) \quad (\text{Eq. 3.75})$$

(ii) Lateral force ($F_{cr,s2}$) to cause slip at contact surface-2

As shown in Figure 3.30, slip occurs on contact surface-2 when the ultimate shear load $F_{v,Ed2}$ produced by the forces acting on column-1F and the ceiling beam becomes greater than the slip resistance $F_{s,Rd2}$. Thus, $F_{cr,s2}$ becomes the lateral force to make $F_{v,Ed2}$ equal to $F_{s,Rd2}$.

$$F_{v,Ed2} = F_{s,Rd2} \quad (\text{Eq. 3.76})$$

According to (Eq. 3.32) and (Eq. 3.35), (Eq. 3.76) can be written as:

$$V_{c2} + N_{b2} = \mu_2 \cdot (P_{b2,s} - N_{c2} - V_{b2}) \quad (\text{Eq. 3.77})$$

By (Eq. 3.72) and (Eq. 3.61), (Eq. 3.77) becomes:

$$V_B \cdot \frac{(L/2)}{H} + N_{b2} = \mu_2 \cdot \left(P_{b2,s} - N_{c2} - \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot V_B \right) \quad (\text{Eq. 3.78})$$

Then, solve (Eq. 3.78) for V_B , which becomes the critical lateral force $F_{cr,s2}$ to cause slip at contact surface-2.

$$F_{cr,s2} = V_B = \frac{\mu_2 \cdot (P_{b2,s} - N_{c2}) - N_{b2}}{\frac{(L/2)}{H} + \mu_2 \cdot \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right)} \quad (\text{Eq. 3.79})$$

In case axial forces are not applied to the columns and beams, as shown in Figure 3.33, $F_{cr,s1}$ becomes simplified to:

$$F_{cr,s2} = V_B = \frac{\mu_2 \cdot P_{b2,s}}{\frac{(L/2)}{H} + \mu_2 \cdot \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right)} \quad (\text{if } N_{b2} = N_{c2} = 0) \quad (\text{Eq. 3.80})$$

(iii) The critical lateral force for the slip $F_{cr,s}$ is the smaller of $F_{cr,s1}$ or $F_{cr,s2}$:

$$F_{cr,s} = \text{Min} (F_{cr,s1}, F_{cr,s2}) \quad (\text{Eq. 3.81})$$

The corresponding critical moment strength $M_{cr,s}$ is calculated as follows:

$$M_{cr,s} = F_{cr,s} \times \frac{L}{2} \quad (\text{Eq. 3.82})$$

3.5.4. Behaviour of Joint with Gap-opening

Based on the 'strong column-weak beam' concept for moment-resisting frames consisting of the steel-framed modules, the AJ connection proposed in this research is designed to form plastic hinges in the floor and ceiling beams when subjected to earthquake loading. However, a gap-opening at the AJ connection may change the intended failure mechanism.

Figure 3.38 shows different failure modes depending on the occurrence of gap-opening at the AJ connection when a lateral force is applied to the tip of the beams. (In general, the cross-sectional size of the floor beam is larger than that of the ceiling beam in the modular building, and the same steel grade is used for the two beams.) Their failure mode can be predicted on the basis of the studies presented in Sections 3.5.1 through 3.5.3.

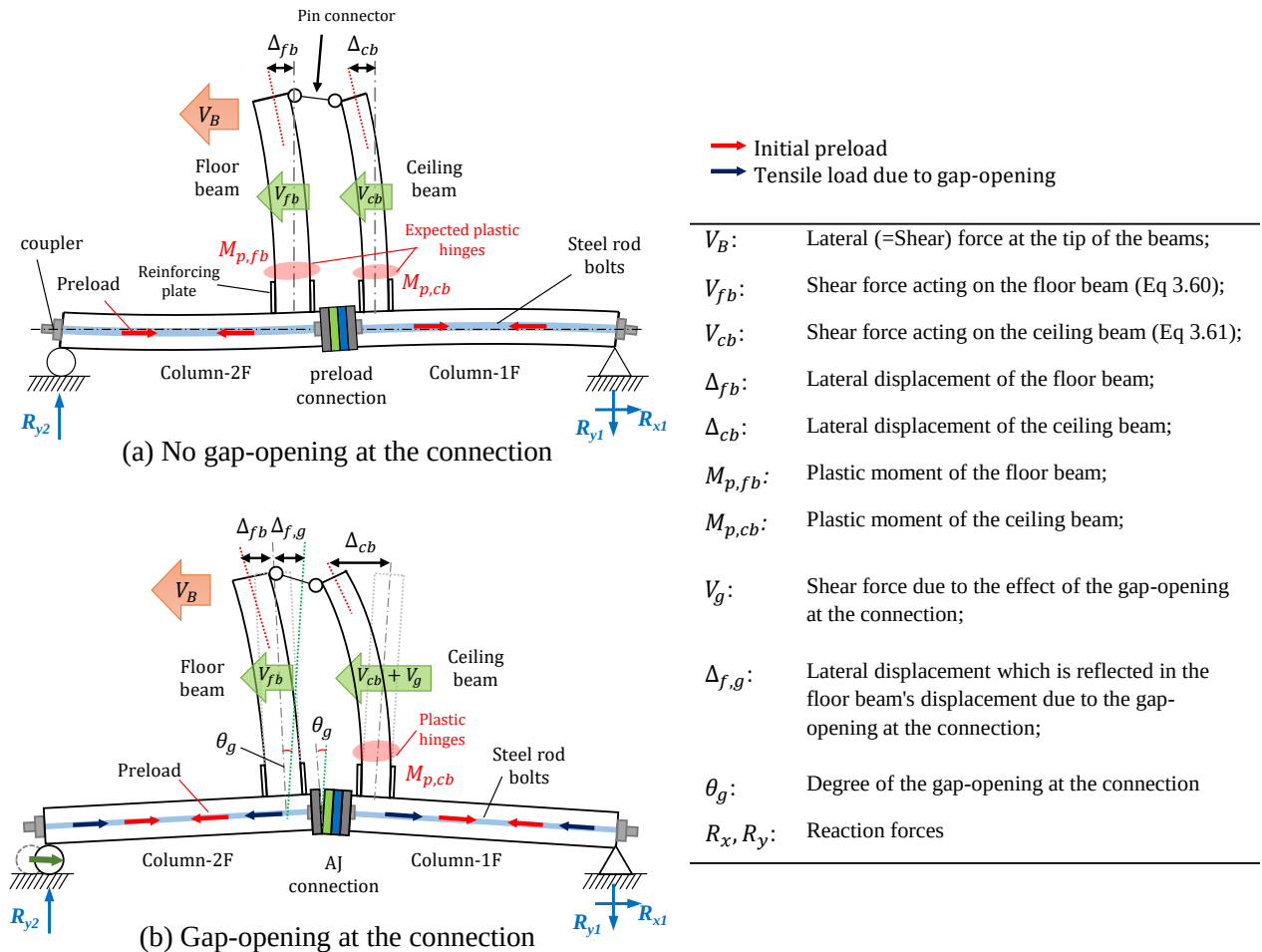


Figure 3.38: Different failure modes depending on the occurrence of gap-opening at the AJ connection

If the gap-opening can be prevented at the AJ connection by the applied preload (see Figure 3.38 (a)), the lateral forces V_B acting at the tip of the beams are distributed to the floor and ceiling beams based

on their relative bending stiffness EI (refer to Section 3.5.1), causing the floor beam to yield earlier than the ceiling beam. The ceiling beam yields shortly after because it attracts more of the additional applied force once the floor beam yields. Eventually, both the floor and ceiling beams can fully reach and sustain their plastic moment capacities at the expected location of the hinges (near the reinforcing plates) and hence, the final failure will occur in the beams. (In actuality, the results of the pre-test numerical simulations in Section 4.4.8 show that the floor beam yields and fails earlier than the ceiling beam (see Figure 4.51 (a)).) While loading is applied, the SRBs in the columns remain stable, maintaining the tensile load applied by the preloading of the connection.

On the other hand, if gap-opening occurs at the connection (the gap-opening location is found using Table 3.2), as shown in Figure 3.38 (b), the lateral displacement caused by the gap-opening $\Delta_{f,g}$ causes further displacement of the floor beam. It follows that the total lateral displacement of the floor beam consists of the displacement produced by the flexural deformation of the floor beam Δ_{fb} and the displacement caused by the gap-opening $\Delta_{f,g}$. As a result, the ceiling beam, which has the same lateral displacement as the floor beam, is required to undergo additional flexural deformation which implies that an additional lateral load is applied to the ceiling beam. If gap-opening does not occur at the AJ connection, the lateral (shear) forces acting on the floor and ceiling beams are determined based on the bending stiffness (EI) of the beams through (Eq. 3.61) and (Eq. 3.62), as described in Section 3.5.1. However, after the gap-opening starts to occur, these equations no longer apply. Rather, the ceiling beam is subjected to a higher percentage of the total lateral force, while the floor beam is subjected to a smaller percentage, compared to before the gap-opening occurs. Consequently, the gap-opening causes the ceiling beam to yield and fail first, and also prevents the floor beam from reaching its plastic moment capacity. In other words, the gap-opening at the connection may not allow the total of the plastic moment capacities of both the floor and ceiling beams to be utilised. Also, the gap-opening causes the lateral stiffness of the joint to decrease. As a result, gap-opening at the AJ connection diminishes the seismic performance of the joint.

In addition, when the gap-opening begins to occur, the SRBs, which only maintain the tensile load applied by the preload, are subjected to an additional tensile force as shown in Figure 3.38 (b). When the ceiling beam fails first for the reason explained above, the SRBs serve to prevent the joint from sudden strength degradation by partially resisting the lateral force. However, this would expose the SRBs to the risk of yielding and rupture, and potentially compromise their essential function of preserving the integrity of the connection.

■ Prediction of the gap-opening type

There are two possible types of gap-opening at the AJ connection depending on the direction of drift deformation, as illustrated in Figure 3.39. They are named as the interior gap-opening and the exterior

gap-opening in this study, depending on their location at the connection.

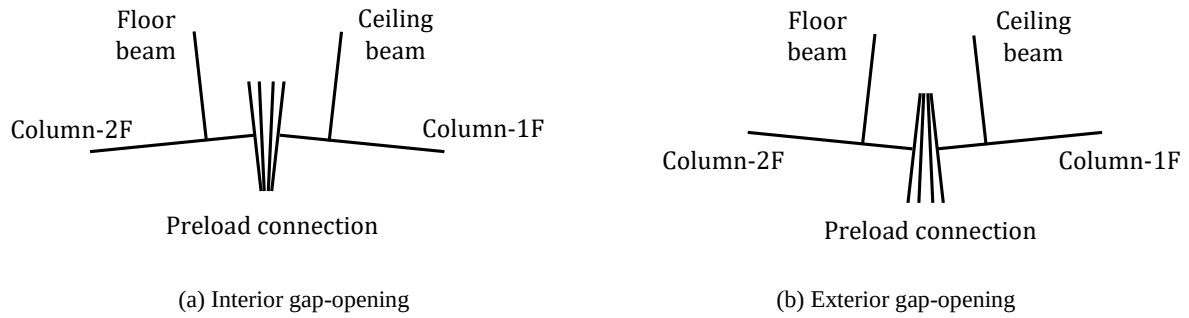


Figure 3.39: Types of gap-opening at the AJ connection depending on the gap location

Using Table 3.2, it is possible to predict the type and location of the gap-opening at the connection and the resulting failure mode of the beam-to-column joint. Figure 3.40 and Table 3.3 show the failure modes due to gap-opening according to the direction of the lateral force for three cases (see below).

Case 1 - Bending stiffness (EI) : Floor beam > Ceiling beam (see Figure 3.40 (a))

- In both directions of the lateral force, the gap-opening occurs on the side of the connection closest to the floor beam, which has a larger EI , so the lateral displacement due to the gap-opening $\Delta_{f,g}$ leads to increased lateral displacement of the tip of the ceiling beam, which makes it yield first and fail earlier than the floor beam. Regarding the SRBs maintaining the tensile load applied by the preload, they are subjected to an additional tensile force due to the gap-opening.

Case 2 - Bending stiffness (EI) : Floor beam < Ceiling beam (see Figure 3.40 (b))

- In both directions of the lateral force, the gap-opening occurs on the side of the connection closest to the ceiling beam, which has a larger EI , so the lateral displacement due to the gap-opening $\Delta_{c,g}$ leads to increased lateral displacement of the tip of the floor beam, which makes it yield first and fail earlier than the ceiling beam. Regarding the SRBs maintaining the tensile load applied by the preload, they are subjected to an additional tensile force due to the gap-opening.

Case 3 - Bending stiffness (EI) : Floor beam = Ceiling beam (see Figure 3.40 (c))

- In both directions of the lateral force, although the gap-opening occurs, its impact is minor. The amount of gap-opening is too small to change the failure mechanism. So, the floor and ceiling beams yield and reach their plastic moment simultaneously. For the SRBs maintaining initial tensile load by the preload, they are subjected to an additional tensile force, but the magnitude is minor.

Table 3.4: Different failure modes due to gap-opening at the AJ connection depending on the difference in the bending stiffness of the floor and ceiling beams

Name	Bending stiffness (EI)	Gap-opening		Tensile force acting on the SRBs	Failed mode
		Pulling force (+ V_B)	Pushing force (- V_B)		
Case1	Floor beam > Ceiling beam	Interior (Col 2F - FB)	Exterior	Initial preload + large tensile force due to gap-opening	Ceiling beam → Floor beam
Case2	Floor beam < Ceiling beam	Exterior (Col 1F - CB)	Interior	Initial preload + large tensile force due to gap-opening	Floor beam → Ceiling beam
Case3	Floor beam = Ceiling beam	Interior (Col 2F-FB) but minor gap-opening	Interior (Col 1F-CB)	Initial preload + minor tensile force due to gap-opening	Both beams

In the case of joints with different beam sizes (i.e. different bending stiffness EI), when gap-opening occurs at the AJ connection, the type (interior or exterior) and location (near the floor beam or near the ceiling beam) of the gap-opening are different depending on the direction of the lateral force and relative bending stiffness. However, regardless of the type and location of the gap-opening, it is highly likely that the beam having the smaller bending stiffness (EI) will yield and fail first, while the other beam will not yield or yield only slightly.

When a gap-opening occurs at the AJ connection, the results are as follows:

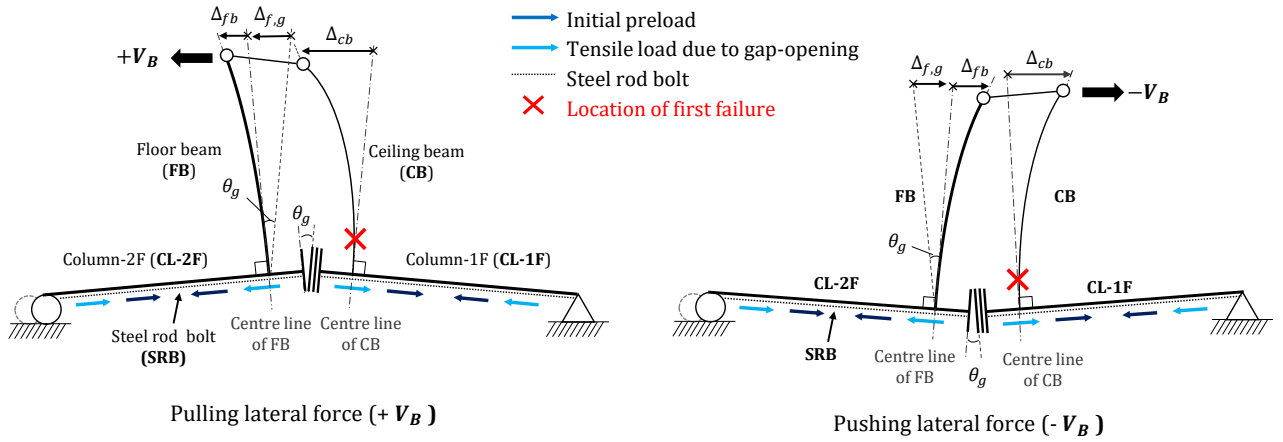
- 1) Reduction of the lateral stiffness of the beam-to-column joint.
- 2) Reduction of the ductility of the beam-to-column joint since it prevents the floor and ceiling beams from utilising their total plastic moment capacities (the smaller beam may reach its plastic moment capacity, but the larger beam will not).
- 3) A large additional tensile force is applied to the SRBs inside the columns, exposing them to the risk of yielding and rupture.

(The severity of these impacts is different depending on the timing of the gap-opening. The earlier it occurs, the greater the impact.)

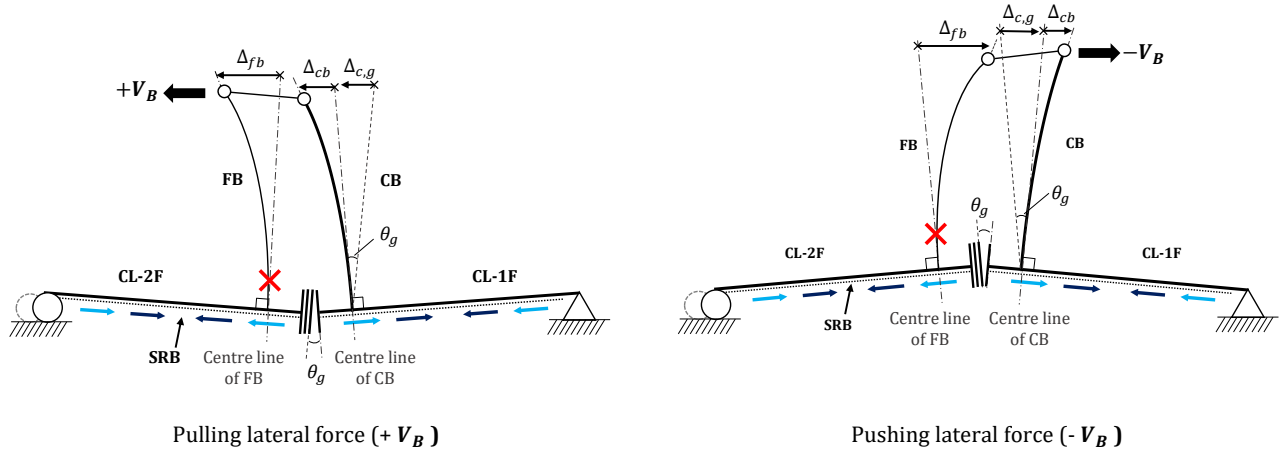
On the other hand, if the same size is used for both beams, the impacts can be reduced significantly by minimising the gap-opening at the AJ connection. More generally, the smaller the difference in bending stiffness between the two beams, the greater the capacity to exploit full plastic bending capacity of the beams and reduce additional tensile forces in the SRBs. The demand preload to prevent gap-opening is reduced as well.

Irrespective of the choice of beam size, gap-opening at the AJ connection should be prevented by selecting a sufficient amount of preload. If this is not possible, the impact should be minimised as much as possible.

Case 1 – Bending stiffness (EI): Floor beam > Ceiling beam



Case 2 – Bending stiffness (EI): Floor beam < Ceiling beam



Case 3 – Bending stiffness (EI): Floor beam = Ceiling beam

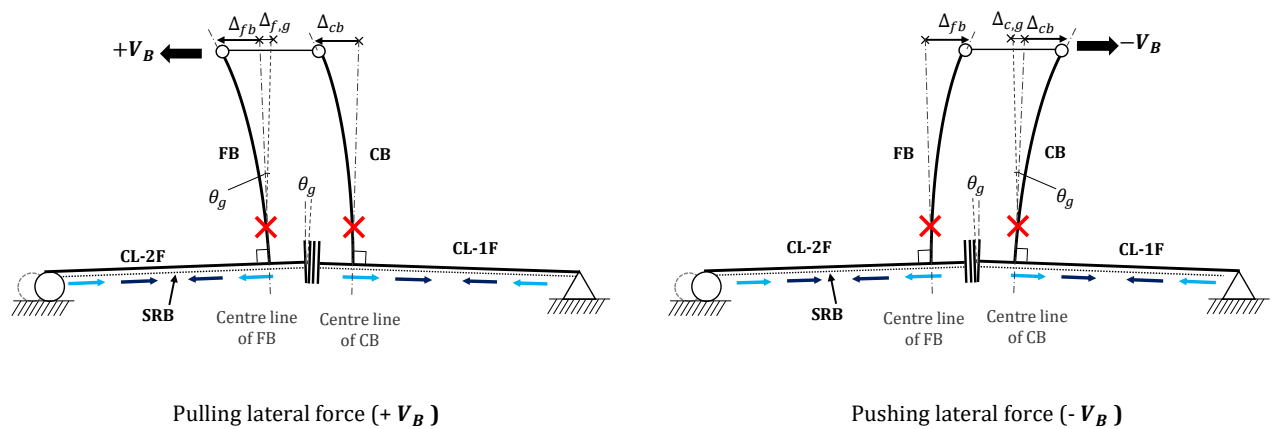


Figure 3.40: Different failure modes due to gap-opening at the AJ connection depending on the difference in bending stiffness of the floor and ceiling beams (refer to Figure 3.38 for nomenclature)

3.6. Application of the AJ Joint for Seismic Design

In contemporary seismic design, a building is designed using reduced seismic design loads depending on its inelastic deformation capacity, which is determined by the ductility of the building. In the case of a multi-storey building, the ductility is obtained by the deformation capacities of members and the distribution of plastic hinges over the height of the structure. However, excessive inelastic deformation of the columns or panel zones (see Figure 3.41 (a) and (b)) can cause a soft storey collapse mechanism (see Figure 3.41 (2)) or premature brittle fracture of a beam-column connection. In order to obtain a ductile behaviour of the structure, the capacity design principle is generally used in seismic design. In the case of moment-resisting frames (**MRFs**), the strong column-weak beam concept is recommended as the primary design strategy to guarantee overall ductile behaviour. This ensures that a multi-storey MRF building has a global plastic failure mode with plastic hinges forming only at the ends of the beams and the base of the frame, while the columns and panel zones remain in the elastic range, as shown in Figure 3.41 (1).

The same methodology is applied in this study to ensure the MRFs consisting of steel-framed modules assembled by AJ connections will exhibit ductile behaviour with plastic hinges forming only at the ends of the beams in the event of earthquakes. This section presents design details of the beam-to-column joint with the AJ connection, based on the design criteria and detailing rules of EN 1998-1 [86].

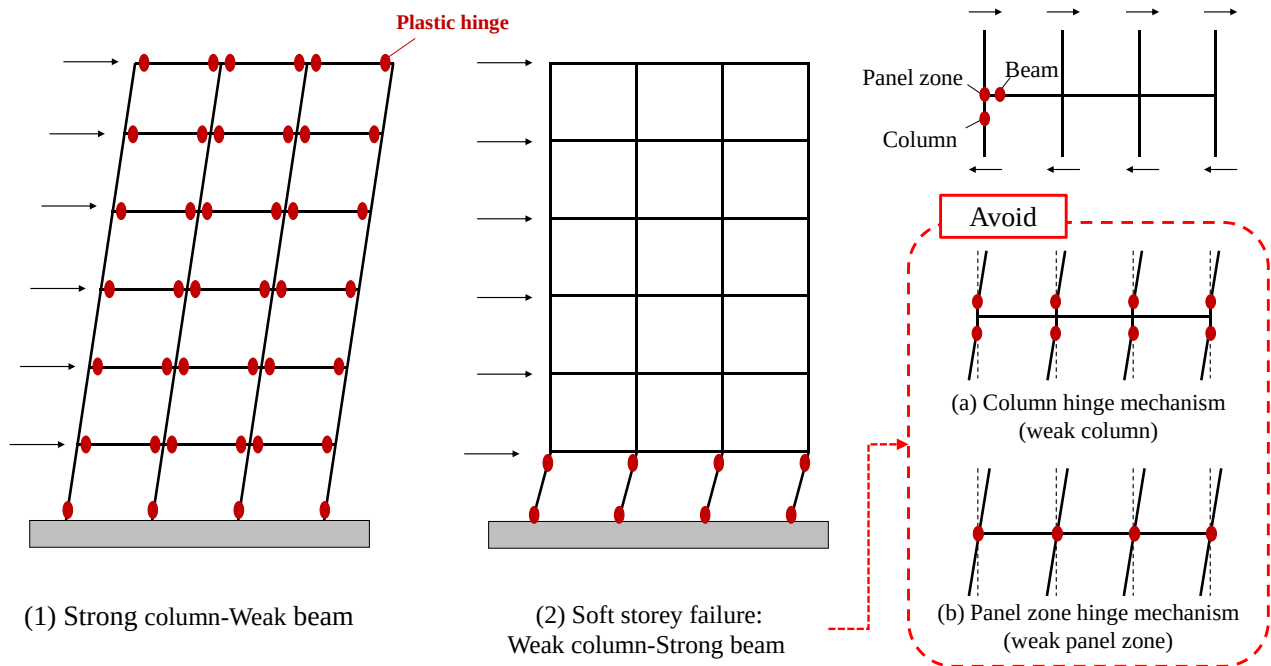


Figure 3.41: Collapse mechanisms of strong column-weak beam and soft storey failure [87]

3.6.1. Dissipative and Non-Dissipative Members in the AJ Joint

Figure 3.42 shows the dissipative and non-dissipative members in the exterior beam-to-column joint with the AJ connection. In order to form plastic hinges securely in the beams and avoid brittle failure in the welded connections between the columns and the beams, reinforcing plates are attached to the beam flanges by welding. In accordance with the capacity design principle, to obtain the strong column-weak beam behaviour of the MRF, the floor and ceiling beams are designed as dissipative members to have plastic hinges for energy dissipation. In contrast, the columns and the panel zones are designed as non-dissipative members to remain in the elastic range until the beams fail. There are two types of connections in the beam-to-column joint: (1) welded beam-column connection within a module and (2) the inter-module preload column-column connection (the AJ connection). Both connections must be designed as non-dissipative connections. The demand preload for the AJ connection also must be designed as a non-dissipative member to prevent the gap-opening and slip at the connection until the two beams reach their plastic moments and fail at the location of the expected plastic hinges, as presented in Figure 3.42.

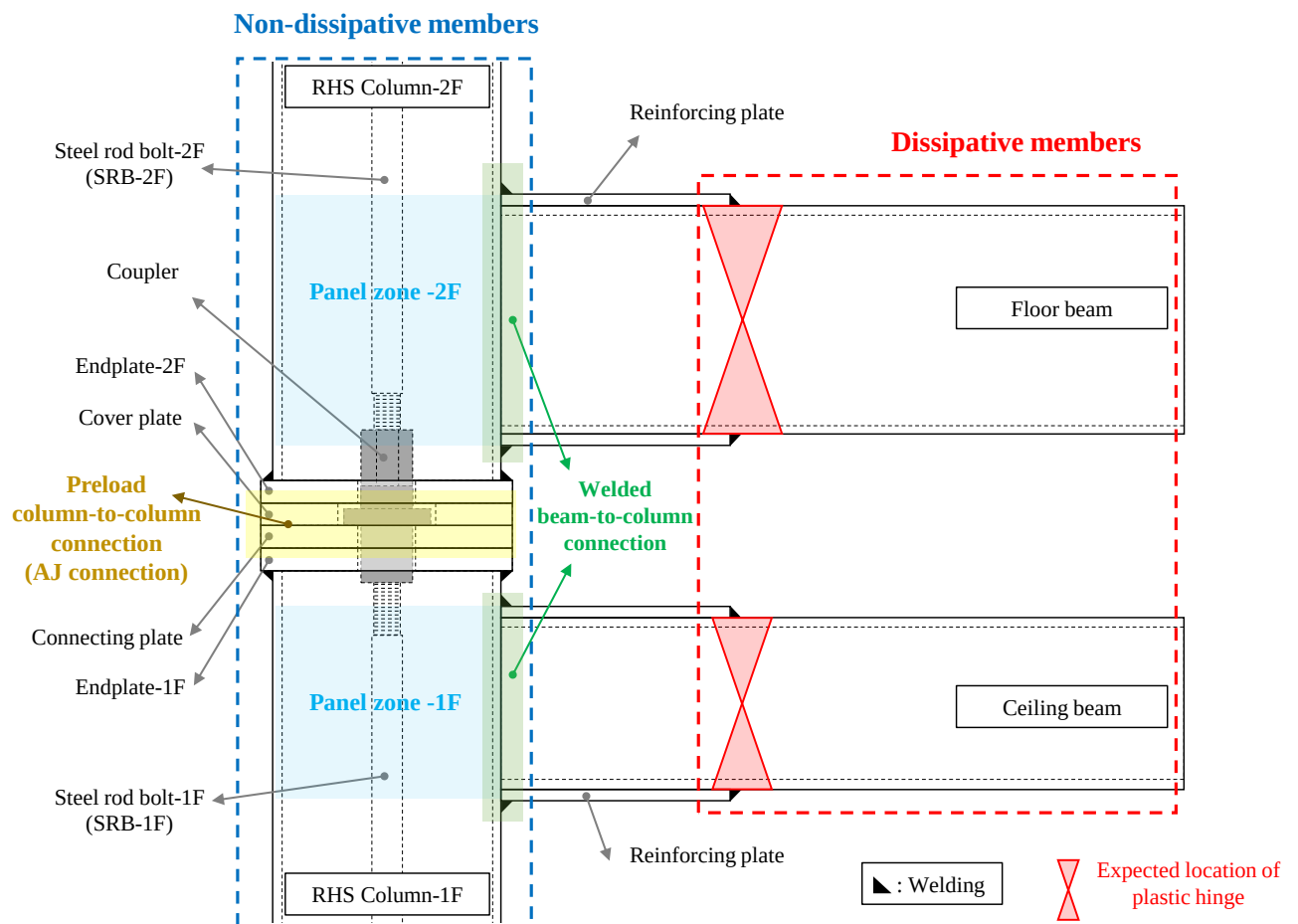


Figure 3.42: Dissipative and non-dissipative members in the exterior preload beam-to-column joint

3.6.2. Code Requirements for the Floor and Ceiling Beams

The floor and ceiling beams (see Figure 3.42) are subject to the same design process of a general dissipative beam for MRFs set out in EN 1998-1 [86]. It is noted that the two beams should be checked individually.

First, sufficient local ductility of members which dissipate energy in compression or bending should be ensured by restricting the width-thickness ratio b/t , according to the cross-sectional classes specified in EN 1993-1-1 [88]. Namely, in order to form plastic hinges in the beams without local buckling, the cross-sections of the floor and ceiling beams should belong to cross-sectional classes 1 or 2.

In addition, for plastic hinges to develop in both beams, the full plastic moment resistance and rotation capacity should not be decreased by compression and shear forces. Therefore, the following inequalities must be verified for the floor and ceiling beams at the locations where the formation of hinges are expected:

$$\frac{M_{Ed}}{M_{pl,Rd}} < 1.0 \quad (\text{Eq. 3.83})$$

$$\frac{N_{Ed}}{N_{pl,Rd}} < 0.15 \quad (\text{Eq. 3.84})$$

$$\frac{V_{Ed}}{V_{pl,Rd}} < 0.5 \quad (\text{Eq. 3.85})$$

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \quad (\text{Eq. 3.86})$$

where:

M_{Ed}, N_{Ed}, V_{Ed}	Design moment, axial force, and shear forces;
$M_{pl,Rd}, N_{pl,Rd}, V_{pl,Rd}$	Design resistances in accordance with EN 1993-1-1;
$V_{Ed,G}$	Design value of the shear force due to non-seismic actions;
$V_{Ed,M}$	Design value of the shear force due to the introduction of plastic moments: $V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B})/L$;
$M_{pl,Rd,A}$ and $M_{pl,Rd,B}$	Beam plastic moments with opposite signs at the end sections A and B of the beam.

Both beams should also be verified as having sufficient resistance against lateral and lateral-torsional buckling in accordance with EN 1993-1-1, assuming the formation of a plastic hinge at one end of the beam. The beam end to be considered is the most stressed end in the seismic design situation.

3.6.3. Code Requirements for Columns

The columns (Columns-1F and -2F in Figure 3.42) are designed in almost the same way as designing a general non-dissipative column for MRFs as per EN 1998-1 [86]. It is noted that the two columns should be checked individually. EN 1998-1 imposes the strong column-weak beam concept for MRFs to guarantee overall ductile behaviour. In order to achieve this, the forces acting on the columns due to the seismic action calculated by an elastic model should be amplified by the magnification factor Ω , which represents the minimum design overstrength of the dissipative elements, as follows:

$$\Omega = \min \left(\frac{M_{pl,Rd,i}}{M_{Ed,i}} \right) \quad (\text{Eq. 3.87})$$

where:

$M_{Ed,i}$ Design value of the bending moment in beam "i" in the seismic design situation;
 $M_{pl,Rd,i}$ The corresponding plastic moment.

The magnification factor must be calculated for all beams where dissipative zones are located. Then, in accordance with EN 1993-1-1 [88], the resistance and stability must be checked for the columns under the most unfavourable combination of bending moment M_{Ed} , shear force V_{Ed} and axial force N_{Ed} . In the case of the column with the AJ connection, it is noted that the column is additionally subjected to the compressive force ($P_{Ed,C}$) due to the preload applied for the column-to-column connection. Therefore, $P_{Ed,C}$ must be added to the design axial force (N_{Ed}) acting on the column when verifying the column resistance and stability.

$$M_{Ed} = M_{Ed,G} + \gamma_{sh} \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \quad (\text{Eq. 3.88})$$

$$V_{Ed} = V_{Ed,G} + \gamma_{sh} \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \quad (\text{Eq. 3.89})$$

$$N_{Ed} = N_{Ed,G} + \gamma_{sh} \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} + P_{Ed,C} \quad (\text{Eq. 3.90})$$

where:

$M_{Ed,G}, N_{Ed,G}, V_{Ed,G}$ The forces (moment, axial, and shear) in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;
 $M_{Ed,E}, N_{Ed,E}, V_{Ed,E}$ The forces (moment, axial, and shear) in the column due to the design seismic action;
 γ_{sh} Hardening overstrength factor. The recommended value is $\gamma_{sh} = 1.1$ according to EN 1998-1;
 γ_{ov} Material overstrength factor. The recommended value is $\gamma_{ov} = 1.25$ according to EN 1998-1;
 $P_{Ed,C}$ Compression force in the column due to preload ($P_{Ed,C}$) for the column-to-column connection.

In addition, in order to enable the development of a number of hinges evenly across the building consistent with the global collapse mechanism (by preventing soft storey failure due to the formation

of plastic hinges in columns, as presented in Figure 3.41 (2) or partial global plastic failure mechanisms due to buckling of columns), EN 1998-1 requires that the local hierarchy criterion is satisfied at all exterior and interior beam-to-column joints as follows:

$$\sum M_{Rc} \geq 1.3 * \sum M_{Rb} \quad (\text{Eq. 3.91})$$

where:

$\sum M_{Rc}$ The sum of the design values of the moment resistance of the columns framing the joint. The minimum value of column moment resistance accounting for the presence of axial forces produced by the seismic design situation;

$\sum M_{Rb}$ The sum of the design values of the moment resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{Rb}$.

As shown in Figure 3.43, in the case of the interior joint with the AJ connection for the steel framed-modular building, each column is connected to one beam, unlike the interior beam-to-column joint in the conventional steel structure where one column is connected to two beams. Therefore, the terms $\sum M_{Rc}$ and $\sum M_{Rb}$ for the joint with the AJ connection are calculated as follows:

$$\sum M_{Rc} = M_{ct,n} + M_{cb,n} \quad (\text{Eq. 3.92})$$

$$\sum M_{Rb} = M_{b,n} \quad (\text{Eq. 3.93})$$

where:

$M_{ct,n}$ and $M_{cb,n}$ The minimum value of the moment resistance of the column at the top ($M_{ct,n}$) and bottom ($M_{cb,n}$) of the "n" th module considering the presence of axial forces produced by the seismic design situation (see Figure 3.43);

$M_{b,n}$ The moment resistance of the beam in the "n" th module.

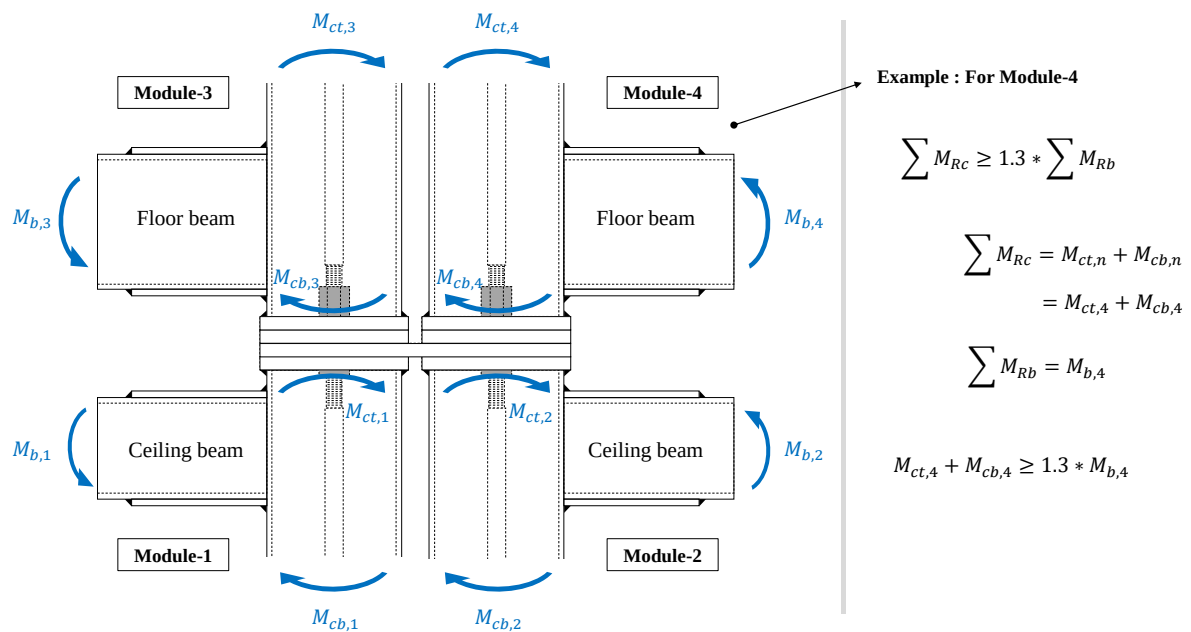


Figure 3.43: Interior beam-to-column joint with the AJ connection

3.6.4. Code Requirements for Non-Dissipative Connections

If only beams are designed as dissipative members in MRFs, a beam-to-column joint has to be designed as a rigid or semi-rigid full strength joint. Thus, a beam-column connection should be designed as a non-dissipative connection that ensures sufficient overstrength to remain elastic and avoid localisation of plastic strains while the beams are dissipating energy by forming plastic hinges. Therefore, the connection must satisfy the following overstrength criterion in accordance with section 6.5.5 (3) of EN 1998-1:

$$R_d \geq \gamma_{sh} \cdot \gamma_{ov} \cdot R_{fy} \quad (\text{Eq. 3.94})$$

where :

R_d	Resistance of the connection in accordance with EN 1993;
R_{fy}	Plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993;
γ_{sh}	Hardening overstrength factor. The recommended value is $\gamma_{sh} = 1.1$ according to EN 1998-1;
γ_{ov}	Material overstrength factor. The recommended value is $\gamma_{ov} = 1.25$ according to EN 1998-1.

Figure 3.42 and Figure 3.43 show the exterior and interior beam-to-column joints with the AJ connection for the steel-framed modular building, respectively. For the modular building, the beam-to-column joint is formed by assembling two or four modules, and it includes two types of connections, depending on the location (see Figure 3.42): (1) Welded beam-column connection in a module and (2) preload column-column connection between modules. Both connections should be designed as non-dissipative connections. Therefore, according to the overstrength criterion (Eq. 3.94), the beam-column connection should be designed to have sufficient overstrength. For the preload column-column connection, the demand preload to prevent gap-opening and slip at the connection should be designed considering the overstrength of dissipative members. These design requirements are explained in detail in Sections 3.6.4.1 and 3.6.4.2.

3.6.4.1. Beam-column Connection in a Module

For a rigid or semi-rigid full strength joint, the beam-column connection is generally designed as a rigid (e.g. fully welded or welded/bolted configuration) or semi-rigid connection (e.g. bolted end plated configuration). No matter which one of the two is selected as the beam-column connection in a module, the connection should satisfy the overstrength criterion (Eq. 3.94). For the beam-to-column joint with the AJ connection, the beams are fully welded to the columns, and reinforcing plates are attached to flanges of the beams in order to obtain the plastic hinges securely in the beams and prevent brittle failure

in the welded beam-column connection, as shown in Figure 3.44. Since the location of the expected plastic hinges are near the reinforcing plate, the design moments ($M_{c,Ed}$) for the beam-column connections are as follows (see Figure 3.44):

$$M_{c,Rd,B2} \geq M_{c,Ed,B2} = \gamma_{sh} \cdot \gamma_{ov} \cdot (M_{pl,Rd,B2} + V_{Ed,B2} \cdot l_h) \text{ for the floor beam-column connection} \quad (\text{Eq. 3.95})$$

$$M_{c,Rd,B1} \geq M_{c,Ed,B1} = \gamma_{sh} \cdot \gamma_{ov} \cdot (M_{pl,Rd,B1} + V_{Ed,B1} \cdot l_h) \text{ for the ceiling beam-column connection} \quad (\text{Eq. 3.96})$$

where :

- $M_{c,Rd,B}$ The resistance of the connection;
- $M_{c,Ed,B}$ The design bending moment at the column face;
- $M_{pl,Rd,B}$ The plastic flexural strength of the connected beam;
- $V_{Ed,B}$ The shear force corresponding to the formation of a plastic hinge in the connected beam, calculated according to (Eq. 3.86);
- l_h The distance between the column face and the expected location of the plastic hinge;
- γ_{sh} Hardening overstrength factor. The recommended value is $\gamma_{sh} = 1.1$ according to EN 1998-1;
- γ_{ov} Material overstrength factor. The recommended value is $\gamma_{ov} = 1.25$ according to EN 1998-1.

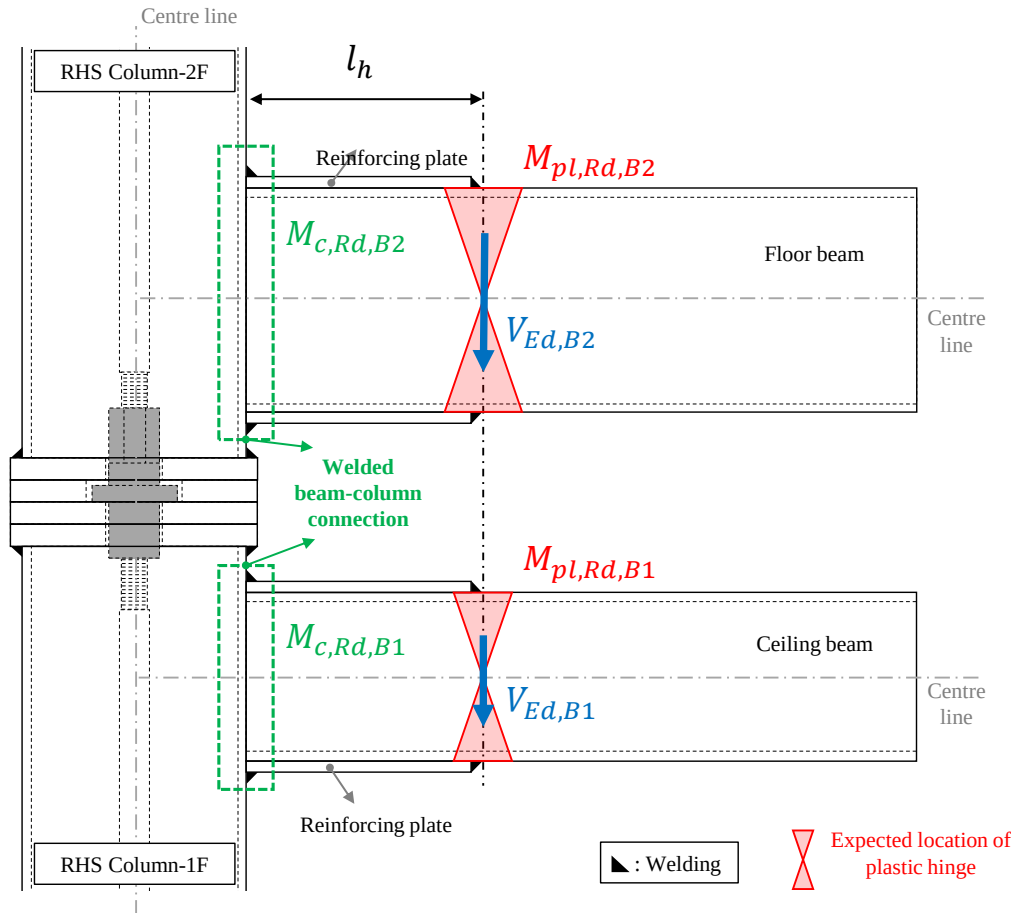


Figure 3.44: Details of beam-column connection (expected location of plastic hinges and corresponding design lever arm l_h)

3.6.4.2. Preload Column-column Connection Between Modules (AJ Connection)

This section determines the design forces required to calculate the demand preload for the non-dissipative preload connection. The demand preload is calculated using the design methods for gap-opening and slip set out in Sections 3.4.3 and 3.4.4, respectively.

The moment capacity of a beam-to-column joint in a modular building is defined as the sum of the plastic moment resistances of the floor and ceiling beams as follows [58, 59]:

$$M_{pl,Rd,Beam} = M_{pl,Rd,B1} + M_{pl,Rd,B2} \quad (\text{Eq. 3.97})$$

where :

$M_{pl,Rd,Beam}$ Total plastic moment resistance of both the floor and ceiling beams;

$M_{pl,Rd,B1}$ Plastic moment resistance of the ceiling beam;

$M_{pl,Rd,B2}$ Plastic moment resistance of the floor beam.

The demand preload is defined as the preload that can prevent gap-opening (and slip) until both floor and ceiling beams achieve their plastic moment strength, accounting for material overstrength ($\gamma_{ov} = 1.25$) and hardening overstrength ($\gamma_{sh} = 1.1$). So, the demand preload for an upper column and a lower column are calculated based on the plastic hinge strengths of the floor beam and the ceiling beam, respectively.

In determining the design forces for the demand preload, in addition to the shear forces acting on the floor and ceiling beams which are the direct forces causing the plastic hinges in the beams, there are axial and shear forces acting on the columns (refer to Figure 3.45 (b)). Therefore, the design forces should be determined separately for the upper column-floor beam and the lower column-ceiling beam. Figure 3.45 shows the design forces acting on the beams and columns at the inflection points. The demand preload for the upper column is determined by the design forces ($N_{Ed,C2}$, $V_{Ed,C2}$, $N_{Ed,B2}$ and $V_{Ed,B2}$) based on the plastic hinge in the floor beam ($\gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,B2}$). For the lower column, the demand preload is determined by the design forces ($N_{Ed,C1}$, $V_{Ed,C1}$, $N_{Ed,B1}$ and $V_{Ed,B1}$) based on the plastic hinge in the ceiling beam ($\gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,B1}$).

Based on the detailed rules in EN 1998-1 [86], the determination process of the demand preload for the non-dissipative preload connection is as follows:

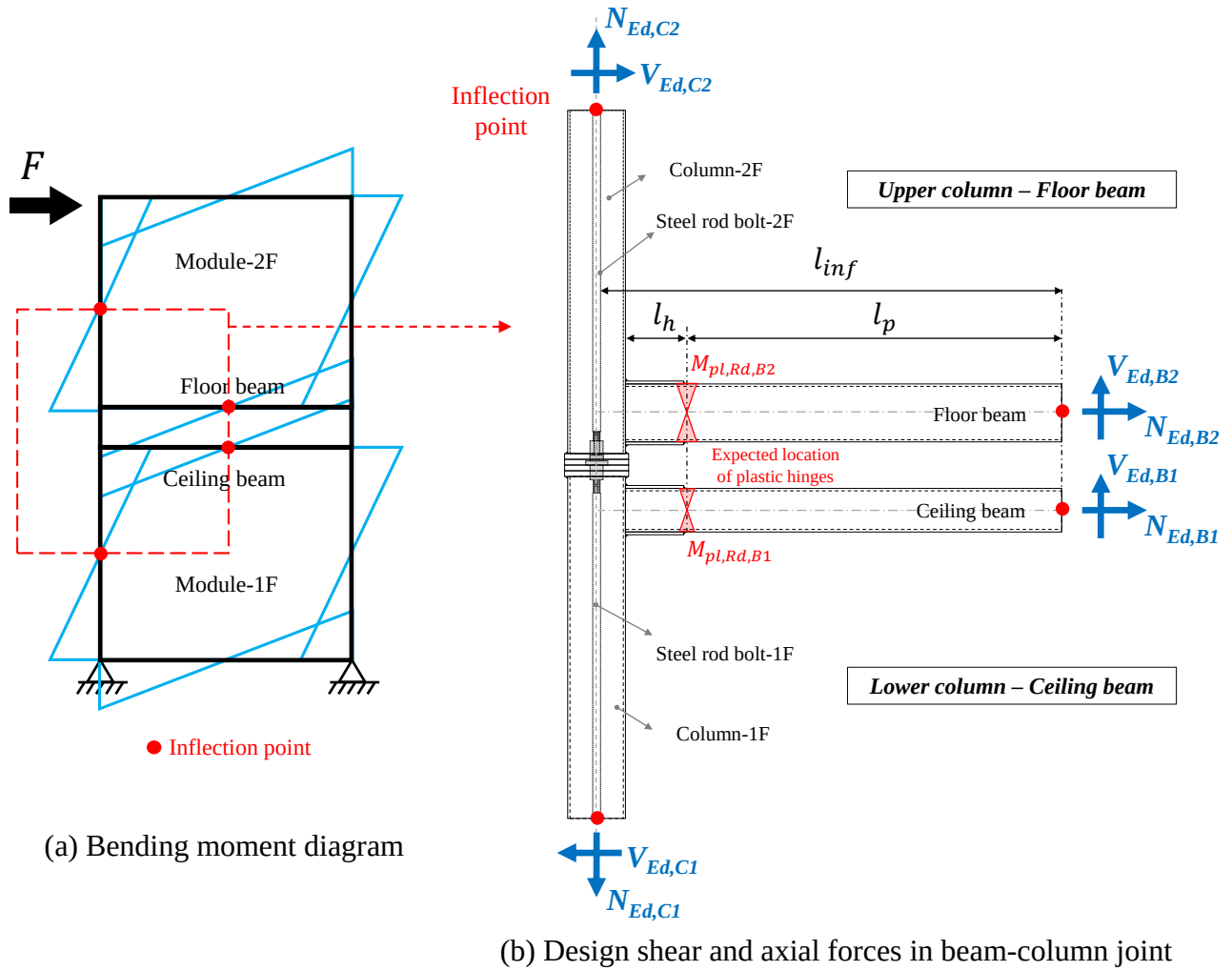


Figure 3.45: Design shear and axial forces for calculating the demand preload for the AJ connection in seismic design

Step 1: Determine amplification factor α

Find the forces acting on the columns and the beams around the preload connection due to the seismic action calculated by an elastic model. Then, find the factors α , which are used as the amplification factor for the design overstrength of the beams (α_{B2} for the floor beam, α_{B1} for the ceiling beam), as follows:

$$\alpha_{B2} = \left(\frac{M_{pl,Rd,B2}}{M_{Ed,B2}} \right) \quad \alpha_{B1} = \left(\frac{M_{pl,Rd,B1}}{M_{Ed,B1}} \right) \quad (\text{Eq. 3.98})$$

where:

- $M_{pl,Rd,B}$ The plastic moment resistance at the expected location for the plastic hinge in the ceiling beam ($M_{pl,Rd,B1}$) and the floor beam ($M_{pl,Rd,B2}$);
- $M_{Ed,B}$ The design moment at the expected location for the plastic hinge in the ceiling beam ($M_{Ed,B1}$) and the floor beam ($M_{Ed,B2}$) in the seismic design situation.

Step 2: Determine design forces for calculating demand preload for the preload connection

In order to obtain the design forces for the demand preload, the shear and axial forces acting on the columns and beams (at the inflection points in the bending moment diagram) due to the seismic action calculated by an elastic model should be amplified by the factor α_B . The demand preload for the preload connection shall be determined under the most unfavourable combination of shear force V_{Ed} and axial force N_{Ed} (see Figure 3.45 (b)):

▪ Design forces to obtain the demand preload for the upper column - floor beam

$$V_{Ed,C2} = V_{Ed,G,C2} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B2} \cdot V_{Ed,E,C2} \quad (\text{on the upper column}) \quad (\text{Eq. 3.99})$$

$$N_{Ed,C2} = N_{Ed,G,C2} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B2} \cdot N_{Ed,E,C2} \quad (\text{on the upper column}) \quad (\text{Eq. 3.100})$$

$$V_{Ed,B2} = V_{Ed,G,B2} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B2} \cdot V_{Ed,E,B2} \quad (\text{on the floor beam}) \quad (\text{Eq. 3.101})$$

$$N_{Ed,B2} = N_{Ed,G,B2} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B2} \cdot N_{Ed,E,B2} \quad (\text{on the floor beam}) \quad (\text{Eq. 3.102})$$

where:

$N_{Ed,G,C2}, V_{Ed,G,C2}$	The forces (axial and shear) in the upper column due to the non-seismic actions included in the combination of actions for the seismic design situation;
$N_{Ed,E,C2}, V_{Ed,E,C2}$	The forces (axial and shear) in the upper column due to the design seismic action;
$N_{Ed,G,B2}, V_{Ed,G,B2}$	The forces (axial and shear) in the floor beam due to the non-seismic actions included in the combination of actions for the seismic design situation;
$N_{Ed,E,B2}, V_{Ed,E,B2}$	The forces (axial and shear) in the floor beam due to the design seismic action;
γ_{sh}	Hardening overstrength factor. The recommended value is $\gamma_{sh} = 1.1$ according to EN 1998-1;
γ_{ov}	Material overstrength factor. The recommended value is $\gamma_{ov} = 1.25$ according to EN 1998-1;
α_{B2}	Amplification factor for the floor beam in Step1.

▪ Design forces to obtain the demand preload for the lower column - ceiling beam

$$V_{Ed,C1} = V_{Ed,G,C1} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B1} \cdot V_{Ed,E,C1} \quad (\text{on the lower column}) \quad (\text{Eq. 3.103})$$

$$N_{Ed,C1} = N_{Ed,G,C1} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B1} \cdot N_{Ed,E,C1} \quad (\text{on the lower column}) \quad (\text{Eq. 3.104})$$

$$V_{Ed,B1} = V_{Ed,G,B1} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B1} \cdot V_{Ed,E,B1} \quad (\text{on the ceiling beam}) \quad (\text{Eq. 3.105})$$

$$N_{Ed,B1} = N_{Ed,G,B1} + 1.1 \cdot \gamma_{ov} \cdot \alpha_{B1} \cdot N_{Ed,E,B1} \quad (\text{on the ceiling beam}) \quad (\text{Eq. 3.106})$$

where:

$N_{Ed,G,C1}, V_{Ed,G,C1}$	The forces (axial and shear) in the lower column due to the non-seismic actions included in the combination of actions for the seismic design situation;
$N_{Ed,E,C1}, V_{Ed,E,C1}$	The forces (axial and shear) in the lower column due to the design seismic action;
$N_{Ed,G,B1}, V_{Ed,G,B1}$	The forces (axial and shear) in the ceiling beam due to the non-seismic actions included in the combination of actions for the seismic design situation;
$N_{Ed,E,B1}, V_{Ed,E,B1}$	The forces (axial and shear) in the ceiling beam due to the design seismic action;
γ_{sh}	Hardening overstrength factor. The recommended value is $\gamma_{sh} = 1.1$ according to EN 1998-1;
γ_{ov}	Material overstrength factor. The recommended value is $\gamma_{ov} = 1.25$ according to EN 1998-1;
α_{B1}	Amplification factor for the ceiling beam.

Step 3: Find the demand preload for preventing gap-opening (P_g) and slip (P_s)

Using the design forces determined by (Eq. 3.99) - (Eq. 3.106), calculate the demand preload P_g to prevent the gap-opening in accordance with the calculation process in Section 3.4.3 (Steps 1-3). In the same way, determine the demand preload P_s to prevent the slip using the calculation process in Section 3.4.4 (Steps 1-3).

Step 4: Determine the final preload for the connection ($P_{F,g}$ and $P_{F,s}$)

The weight of upper modules reduces applied preload (refer to Section 3.4.5). Therefore, the final demand preload for preventing the gap-opening ($P_{F,g}$) and slip ($P_{F,s}$) are determined according to (Eq. 3.55) and (Eq. 3.56), respectively.

Step 5: Design the SRB and the coupler

Using the final demand preload for preventing gap-opening ($P_{F,g}$) and slip ($P_{F,s}$), design the SRB and the coupler, which is described in Section 7.6.1.

In the outlined elastic design method, the demand preload is calculated for the upper and lower columns based on the plastic moments of the floor and ceiling beams, respectively. This approach can be applied to any case of determining the demand preload of the AJ connection, regardless of differences in the lengths of the upper and lower columns, differences in the cross-sectional sizes of the floor and ceiling beams, and differences in the steel grades of the floor and ceiling beams. However, if the floor and ceiling beams have different cross-sectional sizes, the design method produces conservative design forces, resulting in more demand preload than necessary. In case the floor beam has a deeper cross-section than the ceiling beam, it is expected that the floor beam yields earlier than the ceiling beam because it is subjected to relatively more shear force (refer to Section 3.5.1). The FE beam-to-column joint simulation results confirmed that once the floor beam yields, the ceiling beam yields immediately, which is at a lower lateral force than expected by the elastic approach. This is because the ceiling beam is subjected to a larger shear force as the floor beam gradually loses bending stiffness after it yields. However, when the demand preload for the lower column-ceiling beam is calculated using the elastic design method suggested, the above phenomena cannot be taken into account, resulting in a larger demand preload than necessary. Therefore, if the floor and ceiling beams have different cross-sectional sizes, it is recommended to determine the design forces for the demand preload based on the beam that yields first (i.e. the beam having larger bending stiffness) for an economical design. That is, using the amplification factor α_B (Eq. 3.98) for the beam which yields first, determine the design forces to obtain the demand preload for both the upper column-floor beam (Eq. 3.99) - (Eq. 3.102) and lower column-ceiling beam (Eq. 3.103) - (Eq. 3.106).

3.7. Conclusion

Based on an in-depth analysis of existing inter-module connections, the innovative post-tensioned type connection (referred to as the 'AJ' connection) is proposed for mid- and high-rise steel framed modular buildings. This column-to-column connection connects modules vertically by preload, which is applied using the developed coupler and 3m high-tensile SRBs within columns. The AJ connection was jointly researched and developed by the University of Sydney in Australia and the Prefab-Structure Lab of Ajou University in South Korea. The following conclusions were drawn from the investigation of the AJ preload connection:

- (1) The AJ connection has a simple shape and installation process for connecting modules, which provides an easy and convenient on-site assembling and disassembling process of the modular building. In addition, it allows for the modules to be manufactured as complete finished products not requiring additional on-site finishing works after connecting modules, unlike existing bolted connections. Consequently, the AJ connection can maximise the benefits of 3D volumetric modular construction (i.e. fast construction on site, reusability of modules, and automated factory production of modules).
- (2) According to the assembling process of an upper column and a lower column, the preloading mechanism of the AJ connection was explained in detail in this chapter, (refer to Figure 3.5). The connection has three contact surfaces between constituent plates. Depending on the magnitude of preload applied from the upper column and the lower column, the clamping forces on the contact surfaces can differ in the connection. It is desirable to make the connection have a uniform clamping force for all the contact surfaces by applying the same amount of preload to the upper column and lower column.
- (3) When a modular building is subjected to a lateral force such as wind or earthquake, the AJ connection can have two types of failure: gap-opening and slip. These failure modes can significantly reduce the stiffness of the entire frame and affect the overall stability of the building, by causing an additional P- Δ effect. This chapter presents the process for determining the demand preloads (P_g and P_s) for preventing gap-opening and slip at the connection, from defining design forces to calculating the demand preload. If the connection is used as the bearing connection, P_g becomes the demand preload. On the other hand, for the slip-resistant connection, the greater preload of P_g and P_s becomes the demand preload.
- (4) When a module is stacked on another module, the weight of the upper module causes the preload in the lower module to decrease. Therefore, in order to maintain the demand preload (P_g or P_s) at the AJ connection after completing the construction of a modular building, an enhanced preload to

be subsequently reduced due to the weight of upper modules must be applied to the connection to achieve the demand preload in the finished building. This chapter presents the calculation process for the reduction in preload caused by the weight of upper modules.

- (5) Depending on the occurrence of gap-opening at the AJ connection, the behaviour of the beam-to-column joint is significantly different. In case no gap-opening occurs at the connection, both the floor and ceiling beams (dissipative members) reach their full plastic moment capacities M_p . Until then, the SRBs within the columns stably maintain the initial preload. On the other hand, if gap-opening occurs at the connection, it causes various negative effects as follows: (i) reducing the initial lateral stiffness, (ii) decreasing ductility by delaying or preventing the formation of plastic hinges in both beams (i.e. only the smaller beam reaches its full M_p , while the larger beam may or may not reach its M_p), (iii) the SRBs within columns are subjected to large tensile force in addition to the initial preload, exposing them to the risk of yielding and rupture. As a result, the gap-opening at the preload connection significantly reduces the seismic performance of the joint. Therefore, the gap-opening at the connection must be prevented by preload.
- (6) The gap-opening at the AJ connection is affected by differences in the flexural stiffness (EI) of the floor and ceiling beams. The impact of gap-opening increases with increasing difference between the flexural stiffness of the two beams. In case the same cross-sectional size of beam (same EI) is used for the floor and ceiling beams, the impact of gap-opening becomes minimised, and the demand preload to prevent gap-opening is significantly reduced as well. Therefore, in the case of AJ connections subjected to large lateral forces (e.g. on the lower floor in a multi-storey building), it is a good strategy to use the same beam size for the floor and ceiling beams.
- (7) This study aims to produce moment-resisting frames consisting of steel-framed modules connected by AJ connections that exhibit strong column-weak beam behaviour and a ductile response to earthquake loading. Based on the design criteria and detailing rules of EN 1998-1, design details for the beam-to-column joint with the AJ connection are presented, considering the characteristics of the modular building. The AJ connection should be designed as a non-dissipative preload connection, which means gap-opening and slip at the connections should be prevented until both the floor and ceiling beams (dissipative members) fully reach their plastic moment capacities M_p . The design forces to obtain the demand preload that satisfies this condition are derived in this chapter. For the column, the compressive force due to the preload must be added to the design axial force when verifying the column resistance and stability.

Chapter 4

Experimental Investigation of the AJ Connection

4.1. Introduction

This chapter outlines both the experimental procedure undertaken and the experimental results for the AJ connection proposed in this study. The experiments were carried out in conjunction with Ajou University, South Korea. The experimental investigation consisted of three types of tests: material tests, tests of applying preload to SRBs, and full-scale beam-to-column joint tests. The material tests include the uniaxial tensile tests for the mechanical splice of the coupler and the SRBs to check if the developed coupler had sufficient strength to connect modules vertically. An overview of the experimental program is provided in Table 4.1. This chapter also covers the design of the test set-up, instrumentation, and loading procedures (see Section 4.4.7).

The material tests were performed to determine the actual material properties of the steel used for the beam-to-column joints and for input to the numerical simulation described in Chapter 5. The material tests were conducted at the Steel Structure Laboratory of POSCO Global R&D Center, South Korea.

Before testing the full-scale beam-to-column joints, the tests on applying preload to steel rod bolts (SRBs) were carried out to check the feasibility of applying the calculated demand preload (see Section 4.4.7) to 3m-long SRBs using the proposed coupler, as well as to determine the amount of torque corresponding to the demand preload. The preloading of SRB tests were performed at the laboratory of PETRA Test, Evaluation & Consulting (PETRATEC), South Korea.

Finally, the full-scale beam-to-column joint tests were conducted to evaluate the seismic performance of the beam-to-column joint for steel-framed modular buildings including a preload in the columns. The parameters for the joint tests consisted of loading (monotonic, quasi-static cyclic), connection type (welded, preloaded), and the combination of the cross-sectional size of the floor beam and ceiling beam, chosen to investigate the level of the gap-opening at the AJ connection. The test results, which include observations made during the tests and the response of the joints subjected to monotonic and quasi-static cyclic loading, are shown for each specimen. This full-scale beam-to-column joint tests were completed at the Steel Structure Laboratory of POSCO Global R&D Center, South Korea.

Table 4.1: Summary of experimental investigation

Test Type	Conceptual illustration	Test characteristics	No. of specimens
Material tests		Tensile tests • RHS Beam (SRT 275) • RHS Column (SRT275) • Connecting Plate / Column endplate / Cover plate (S275) • Shim plate (S275) • Steel rod bolt (AISI 4140) • Coupler (SM 45C)	24
Applying preload tests		Preload tests for high-tensile steel rod bolts • Checking the feasibility of applying preload to 3m steel rod bolts, using the developed coupler Tools for applying preload: • Torque wrench • Impact wrench	5
Beam-column joint tests		Monotonic loading tests • P-DB-M • P-SB-M Cyclic loading tests • P-DB-Q • P-SB-Q • W-SB-Q Intended failure mode: Formation of the plastic hinges in the beams P – AJ preload connection W – Welded connection DB – Different beam size for a floor beam and a ceiling beam SB – Same beam size for a floor beam and a ceiling beam M – Monotonic loading test Q – Quasi static loading test	5

4.2. Material Tests

Coupon tests were conducted to determine the material properties of each component of the beam-to-column joints. The steel grade and section size of the components are summarised in Table 4.2 and their locations in the joint specimens are illustrated in Figure 4.1. Uniaxial tensile strength tests for a coupler were also performed.

Table 4.2: List of joint components for the material test

No.	Component	Steel Grade	Section size
1	Column	SRT355	RHS 200X100X12
2	Beam-1	SRT275	RHS 150X100X4
3	Beam-2	SRT275	RHS 200X100X4
4	Column endplate / Connecting plate / Cover plate	SS275	Plate 220X120X20(Φ 45) Plate 220X120X20(Φ 45) Plate 220X120X20(Φ 78)
5	Reinforcing plate	SS275	Plate 220X120X10
6	Steel rod bolt	AISI 4140 (=ASTM A193 Grade B7)	Φ 24
7	Coupler	SM45C (=F10T Bolt)	See Figure 4.16

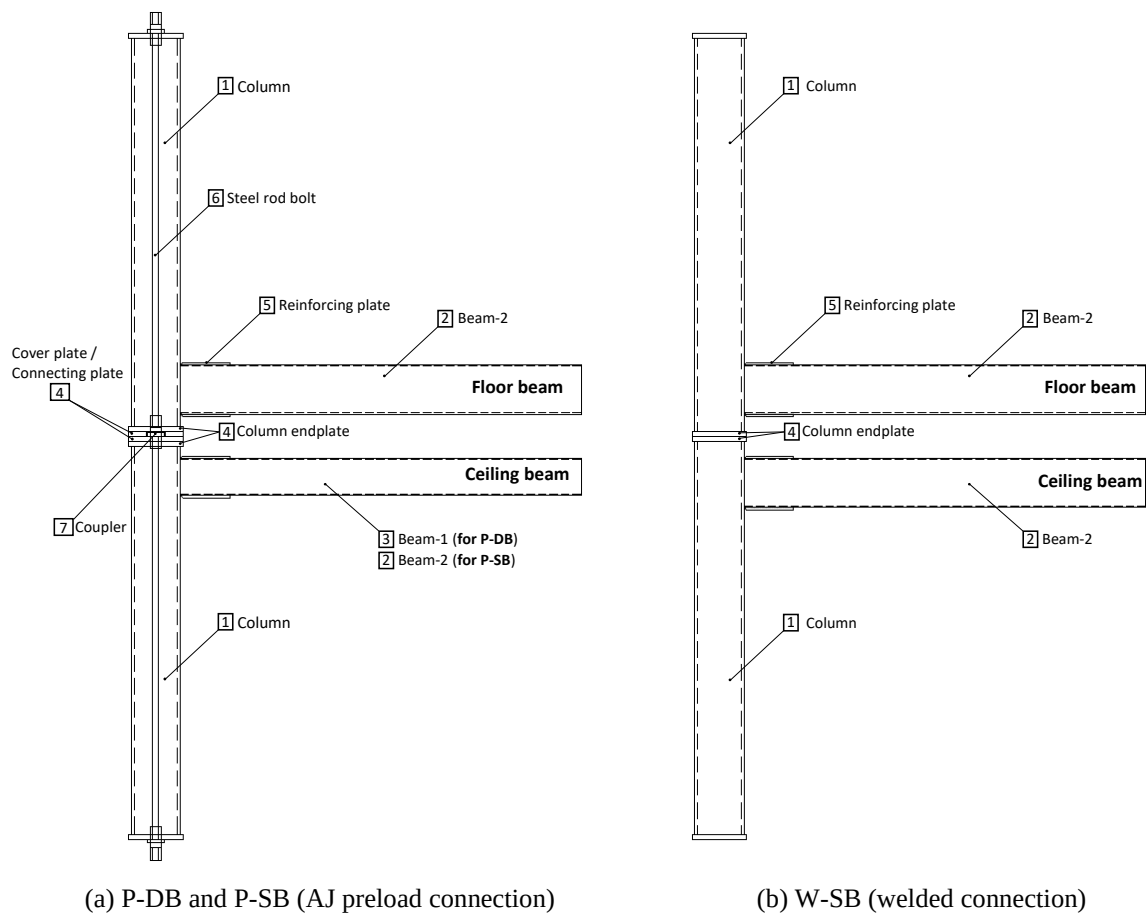


Figure 4.1: Beam-column joint components for material tests

4.2.1. Coupon Test

Three coupons were prepared for each component, according to the Korean standard [89]. The coupons were cut from the same batch of materials used for the full-scale beam-to-column joint tests described in Section 4.4, and tested in accordance with the Korean standard [90]. A displacement loading rate of 0.2mm/min. was selected. A gauge extensometer was attached to the coupons during the tests. The tensile load and extensometer displacement were recorded at a rate of 5 readings per second. A strain gauge was placed at the centre of the coupons to accurately determine the value of Young's modulus and to check the accuracy of the extensometer, as shown in Figure 4.2. Detailed dimensions of the coupons and the test results are presented below. The 0.2% proof stress was used to determine the yield stress, based on the measured stress-strain curves.

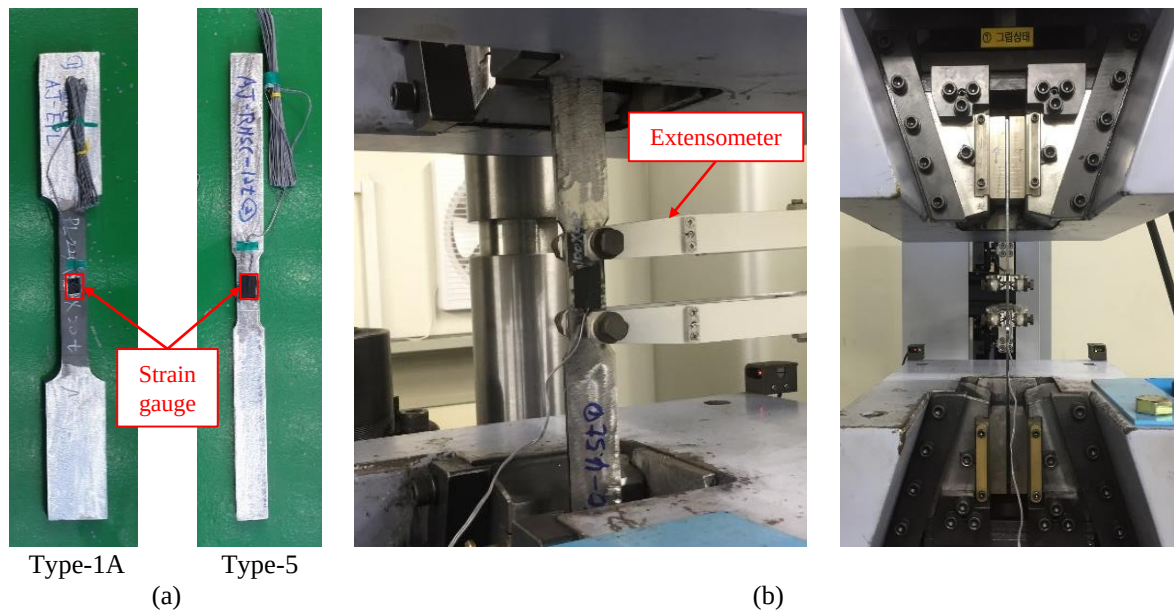


Figure 4.2: Coupon test under monotonic tensile loading: (a) Two types of coupons according to KS B 0801:2007 [89]; (b) Steel coupon placed in UTM with an extensometer

(1) Column: RHS 200X100X12 (SRT355)

Three coupons from the column were tested to determine the actual material properties. Table 4.3 and Figure 4.3 show the detailed dimensions of the tested coupons. The test results and the stress-strain curves are shown in Table 4.4 and Figure 4.4, respectively. The test results satisfied the minimum requirements of KS D 3568:2018 [91] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.3: Measured value of the tested coupons – Column

No.	Measured value		Gauge Length (mm)
	Width (mm)	Thickness (mm)	
SP1	24.5	11.8	50
SP2	25.0	12.0	50
SP3	25.0	12.0	50
Average	24.9	11.9	-

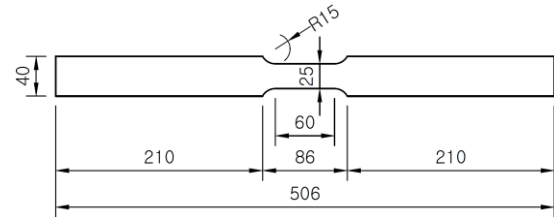


Figure 4.3: Dimensions of the tested coupons – Column

Table 4.4: Coupon test results – Column

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Column	SRT355	SP1	211763	490.6	542.8	28.8	1.11
		SP2	213693	503.3	550.8	27.7	1.09
		SP3	197307	493.3	536.1	28.7	1.09
		Average	207588	495.8	543.2	28.4	1.10
		Nominal value*	-	355	500	23.0	-

* Minimum requirements of KS D 3568:2018

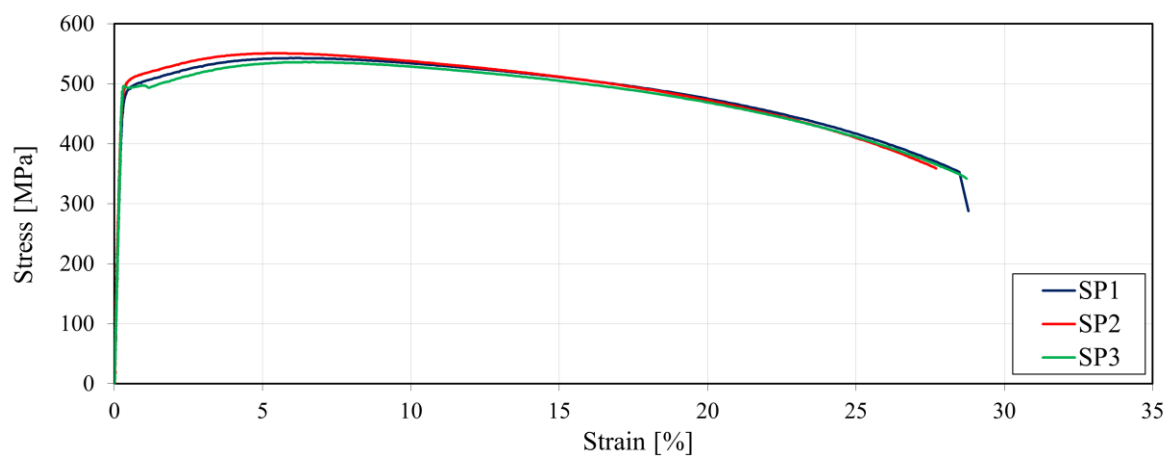


Figure 4.4: Stress-strain curves – Column

(2) Beam-1: RHS 150X100X4 (SRT275)

Three coupons from the ceiling beam were tested to determine the actual material properties. Table 4.5 and Figure 4.5 show the detailed dimensions of the coupons. The test results and the stress-strain curves are shown in Table 4.6 and Figure 4.6, respectively. The test results satisfied the minimum requirements of KS D 3568:2018 [91] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.5: Measured dimensions of the tested coupons – Beam-1

No.	Measured value		Gauge Length (mm)
	Width (mm)	Thickness (mm)	
SP1	25.1	3.90	50
SP2	24.7	3.96	50
SP3	24.1	3.93	50
Average	24.7	3.93	-

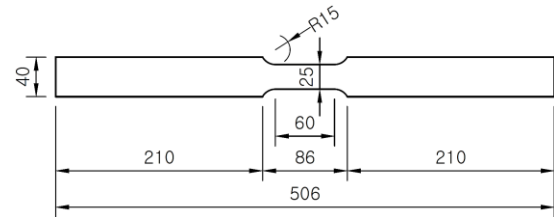


Figure 4.5: Dimensions of the tested coupons – Beam-1

Table 4.6: Coupon test results – Beam-1

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Beam-1	SRT275	SP1	205480	335.2	467.6	32.0	1.39
		SP2	200550	312.9	455.7	33.3	1.46
		SP3	202408	324.0	427.2	34.5	1.32
		Average	202813	324.1	450.1	33.3	1.39
		Nominal value*	-	275	410	23.0	-

* The minimum requirements of KS D 3568:2018

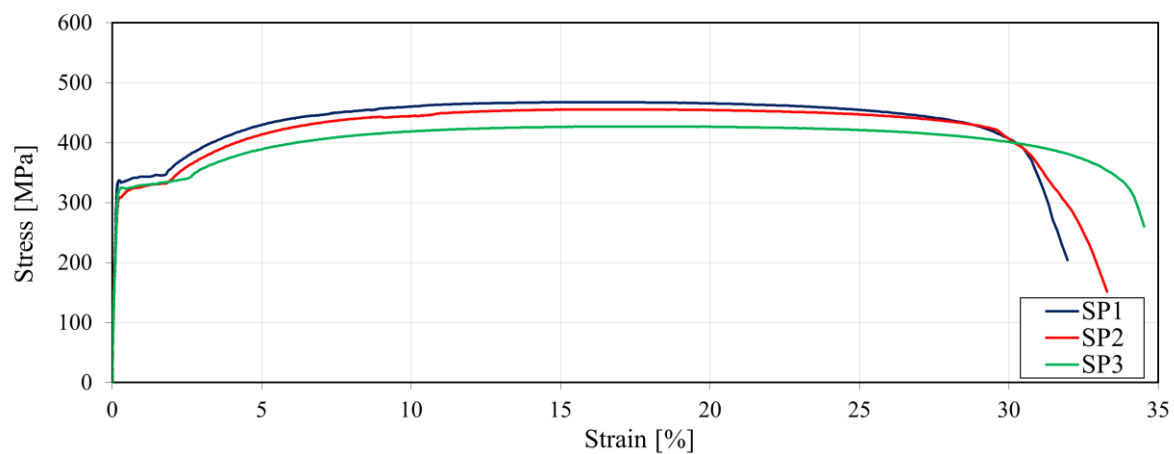


Figure 4.6: Stress-strain curves – Beam-1

(3) Beam-2: RHS 200X100X4 (SRT275)

Three coupons from the floor beam were tested to determine the actual material properties. Table 4.7 and Figure 4.7 show the detailed dimensions of the tested coupons. The test results and the stress-strain curves are shown in Table 4.8 and Figure 4.8, respectively. The test results satisfied the minimum requirements of KS D 3568:2018 [91] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.7: Measured dimensions of the tested coupons – Beam-2

No.	Measured value		Gauge Length (mm)
	Width (mm)	Thickness (mm)	
SP1	24.5	3.90	50
SP2	24.7	3.91	50
SP3	22.8	3.91	50
Average	24.0	3.91	-

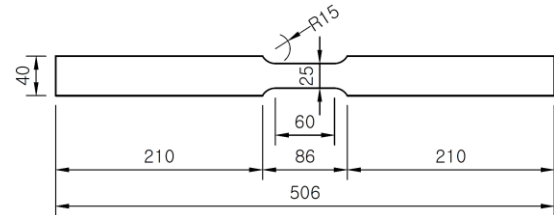


Figure 4.7: Dimensions of the tested coupons – Beam-2

Table 4.8: Coupon test results – Beam-2

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Beam-2	SRT275	SP1	210452	316.8	464.2	32.6	1.47
		SP2	215348	328.3	464.1	34.1	1.41
		SP3	216307	332.7	474.3	29.9	1.43
		Average	214036	325.9	467.5	32.2	1.43
		Nominal value*	-	275	410	23.0	-

* Minimum requirements of KS D 3568:2018

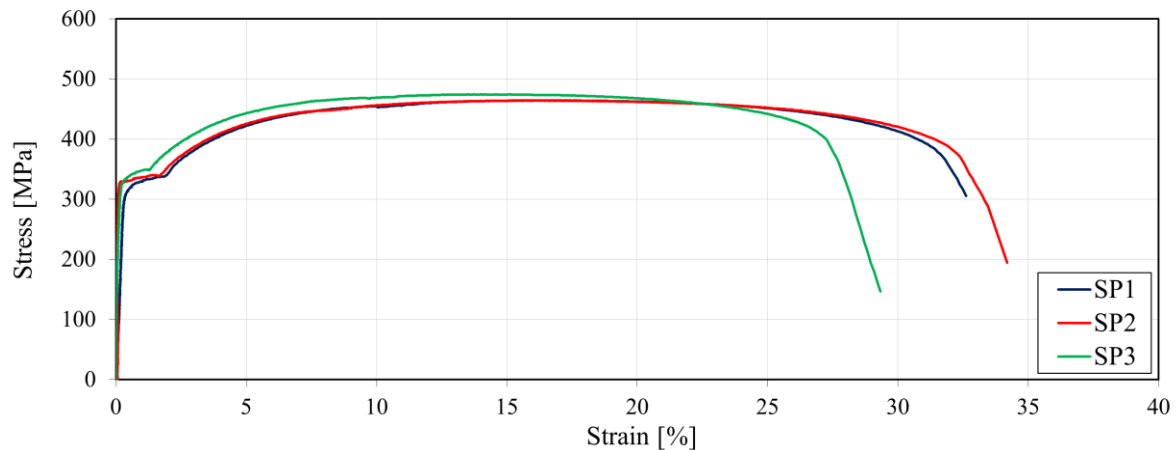


Figure 4.8: Stress-strain curves – Beam-2

(4) Column endplate / Connecting plate / Cover plate: Thickness 20mm (SS275)

A column endplate, a connecting plate, and a cover plate were manufactured from the same parent steel plate. Three coupons cut from the parent plate were tested to determine the actual material properties. Table 4.9 and Figure 4.9 show the detailed dimensions of the tested coupons. The test results and the stress-strain curves are shown in Table 4.10 and Figure 4.10, respectively. The test results satisfied the minimum requirements of KS D 3503:2018 [92] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.9: Measured dimensions of the tested coupons – Plates

No.	Measured value		Gauge Length (mm)
	Width (mm)	Thickness (mm)	
SP1	39.9	20.1	200
SP2	39.5	20.1	200
SP3	39.7	19.9	200
Average	39.7	20.0	-

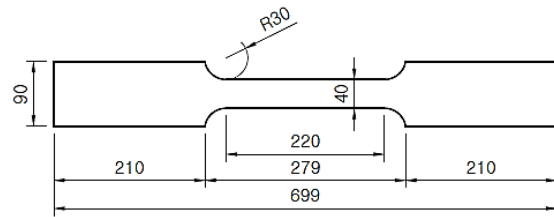


Figure 4.9: Dimensions of the tested coupon – Plates

Table 4.10: Coupon test results – Plates

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Column Endplate Connecting plate Cover plate	SS275	SP1	187175	321.1	456.4	26.8	1.42
		SP2	186211	325.1	456.4	28.4	1.40
		SP3	190204	333.4	466.8	25.5	1.40
		Average	187863	326.5	459.9	26.9	1.41
		Nominal value*		265	410	21.0	-

* Minimum requirements of KS D 3503:2018

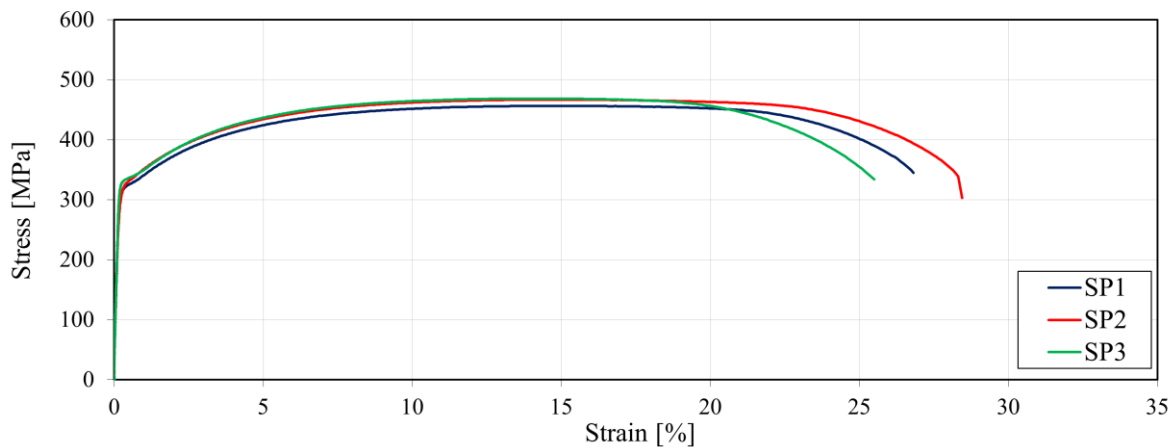


Figure 4.10: Stress-strain curves – Plates

(5) Reinforcing plate: Thickness 10mm (SS275)

Three coupons cut from the reinforcing plate were tested to determine the actual material properties. Table 4.11 and Figure 4.11 show the detailed dimensions of the tested coupons. The test results and the stress-strain curves are shown in Table 4.12 and Figure 4.12, respectively. The test results satisfied the minimum requirements of KS D 3503:2018 [92] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.11: Measured dimensions of the tested coupons – Reinforcing plate

No.	Measured value		Gauge Length (mm)
	Width (mm)	Thickness (mm)	
SP1	39.9	9.8	200
SP2	40.0	9.8	200
SP3	39.9	9.7	200
Average	39.9	9.8	-

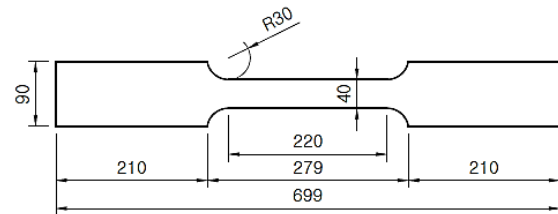


Figure 4.11: Dimensions of the tested coupons – Reinforcing plate

Table 4.12: Coupon test results – Reinforcing plate

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Reinforcing plate	SRT275	SP1	191604	298.6	470.3	27.2	1.58
		SP2	187500	297.0	470.3	29.8	1.58
		SP3	191216	299.7	467.4	25.8	1.56
		Average	190107	298.4	469.3	27.6	1.57
		Nominal value*	-	275	410	21.0	-

* Minimum requirements of KS D 3503:2018

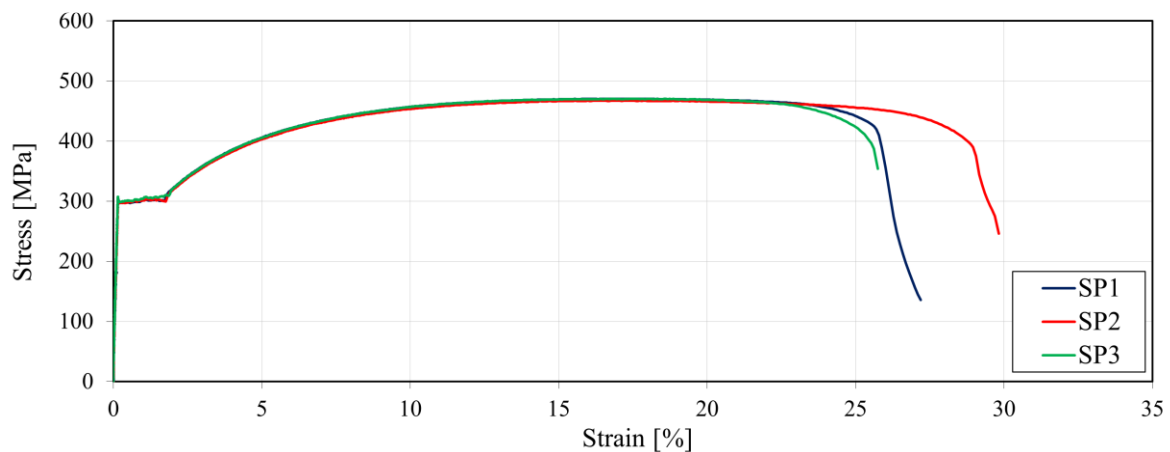


Figure 4.12: Stress-strain curves – Reinforcing plate

(6) Steel rod bolt (AISI4140)

ASTM A193 Grade B7 bolts are manufactured from chromium-molybdenum alloy steel (AISI 4140) through quenching and tempering to obtain the desired mechanical properties. The SRB is made of the same material as the Grade B7 bolt. Three coupons from the SRB were tested to determine the actual material properties. Table 4.13 and Figure 4.13 show the detailed dimensions of the tested coupons. The test results and the stress-strain curves are shown in Table 4.14 and Figure 4.14, respectively. The test results satisfied the minimum requirements of ASTM A193/A193M–20 [93] in terms of yield strength, ultimate tensile strength and elongation at fracture.

Table 4.13: Measured dimensions of the tested coupons – SRB

No.	Measured value	Gauge Length (mm)
	Diameter (mm)	
SP1	12.5	50
SP2	12.5	50
SP3	12.5	50
Average	12.5	-

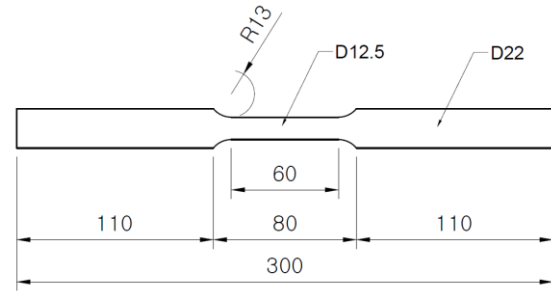


Figure 4.13: Dimensions of the tested coupons – SRB

Table 4.14: Coupon test results – SRB

Member	Steel type	Sample	Elastic Modulus	Yield Stress (f_y)	Ultimate strength (f_u)	Elongation at fracture	f_u / f_y
			[MPa]	[MPa]	[MPa]	[%]	-
Steel rod bolt (SRB)	AISI 4140	SP1	197000	865.8	953.2	20.2	1.10
		SP2	190040	862.3	948.3	19.9	1.10
		SP3	198090	859.6	946.9	19.3	1.10
		Average	195040	862.6	949.5	19.8	1.10
		Nominal value*	-	725	862	16	-

* Minimum requirements of ASTM A193/A193M–20

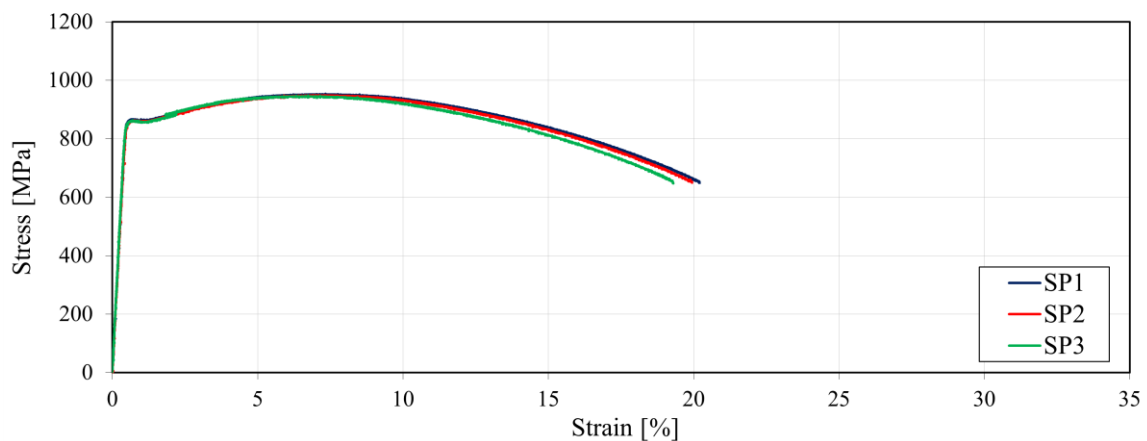


Figure 4.14: Stress-strain curves coupons – Steel rod bolt

4.2.2. Uniaxial Tensile Test for the Coupler

The proposed AJ connection in this research was designed based on the concept of strong-column and weak-beam for resisting earthquakes, which means that non-dissipative members such as columns and connections should remain in the elastic range while the dissipative members like beams undergo plastic deformation when an earthquake occurs. As the coupler and the SRB are located in the column, they are designed to be non-dissipative members. The coupler plays a variety of roles in the AJ connection, (refer to Figure 3.1). One of the main functions of the coupler is to vertically connect the upper and lower modules by connecting the SRB within the columns of the upper and lower modules. Additionally, the coupler also serves to fix the connecting plate that connects the modules horizontally. If the coupler is destroyed, the structural integrity of the building is significantly affected. Therefore, as an essential structural member, the coupler is designed to remain elastic until the SRB is fractured. To investigate this design criterion, uniaxial tensile tests were conducted of the coupler and SRB assembly, using the experimental set-up commonly used for couplers of steel bars in reinforced concrete structures.

(1) Description of specimens

Three specimens were prepared for the uniaxial tensile test (see Figure 4.15). The details of the coupler and its material properties are shown in Figure 4.16 and Table 4.15. The coupler was made of SM45C carbon steel through heat treatment [94], as also commonly used for high tensile bolts. The coupler was manufactured to have similar material properties to the F10T bolts in accordance with the KS B 1010 [95], JIS B1186 [96], and ASTM F3125 [97] standards.

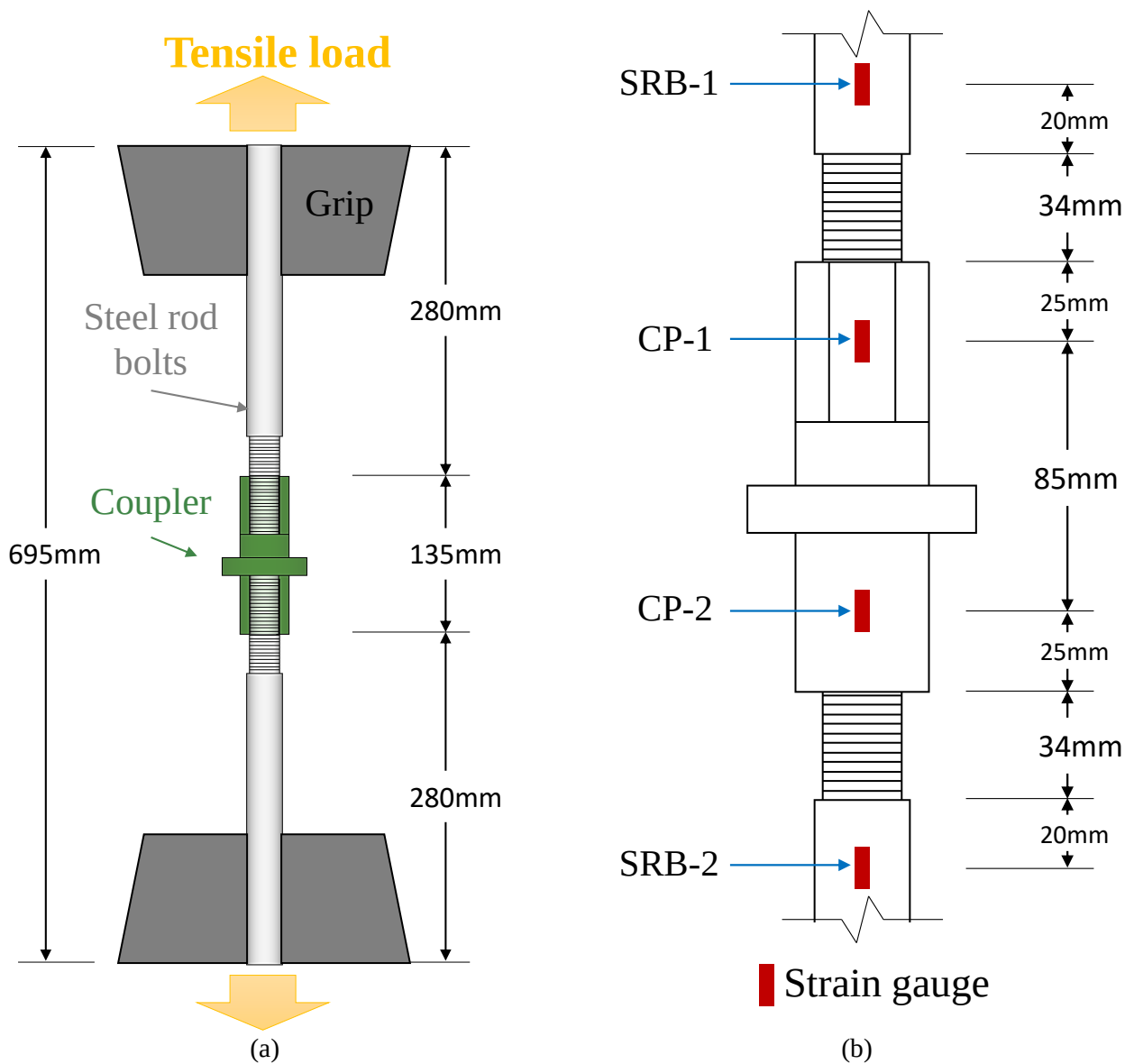


Figure 4.15: (a) Test setup of uniaxial tensile test for the coupler (cross-sectional view);
(b) Location of strain gauges on the coupler and SRBs

Table 4.15: Nominal material properties and dimensions of the coupler

Material properties					Dimension and cross-sectional area						
Steel type	Elastic modulus	Yield Stress	Ultimate strength	Hardness	Upper part (A-A')		Coupler plate (B-B')		Lower part (C-C')		
	[MPa]	[MPa]	[MPa]	[HRC]	Hexagonal nut	Area [mm ²]	Diameter [mm]	Area [mm ²]	Inner Diameter [mm]	Outer Diameter (mm)	Area [mm ²]
SM45C	210000	900	1000	33	M24	670	72	4072	42	24	933

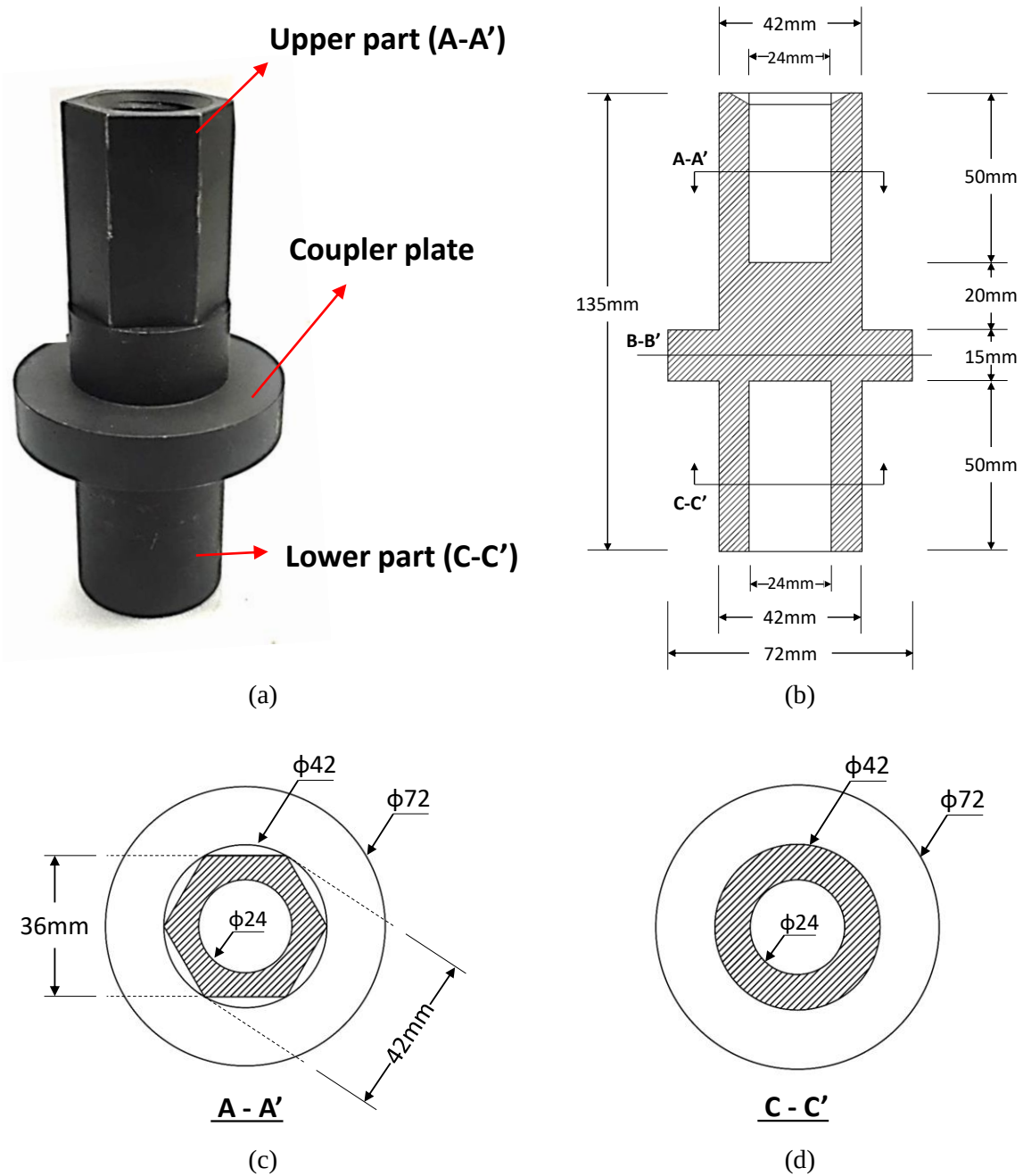


Figure 4.16: (a) 3D image of coupler; (b) (c) and (d) Cross-sectional views of coupler

The material properties and detailed dimensions of the SRB used for the uniaxial tests are shown in Table 4.16 and Figure 4.17. The nominal diameter of the shank is 24.95mm, and the diameter of the thread is the same size for the M24 bolts.

Table 4.16: Nominal material properties and dimensions of SRB

Material properties					Dimension and cross-sectional area							
Steel type	Elastic modulus	Yield Stress	Ultimate strength	Hardness	Length	Shank section		Thread				
						Diameter	Area	Diameter	Area	Length	Pitch	Chamfer length
	[MPa]	[MPa]	[MPa]	[HRC]		[mm]	[mm]	[mm ²]	[mm]	[mm2]	[mm]	[mm]
AISI4140												
(ASTM A193 B7)	210000	725	862	28	330	24.95	489	20.75	339	85	3	5

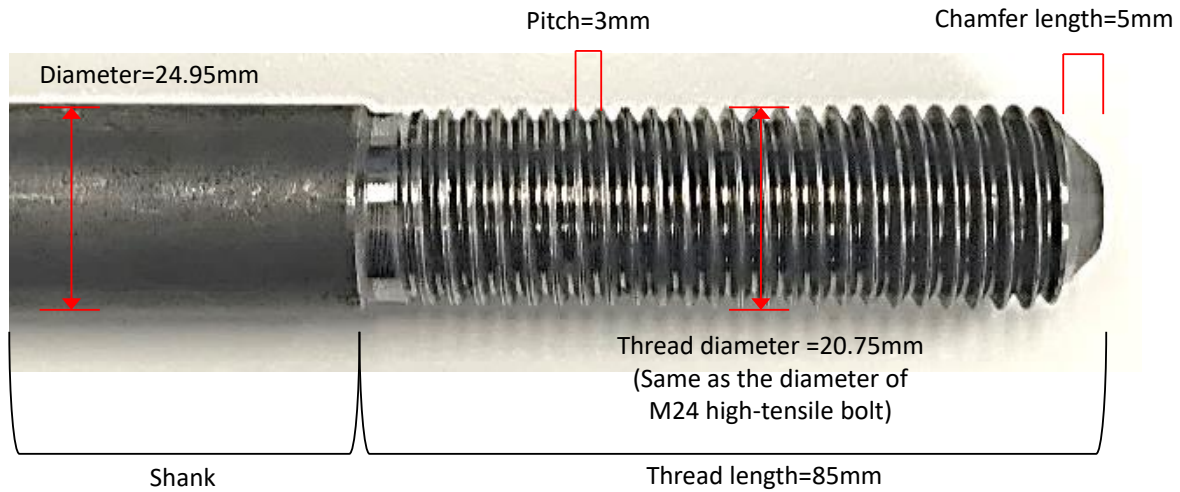


Figure 4.17: Dimensions of the SRB

The coupler has different cross-sections along the length, unlike couplers used for reinforcing bars in reinforced concrete construction. The minimum cross-sectional area of the coupler is in the shape of the hexagonal nut used in the upper part (see Figure 4.16 (a) and (c)), which was designed to have a yield strength ($F_{y,CP}$) greater than the tensile strength of the SRBs ($F_{u,SRB}$). Therefore, the fracture of the specimen is expected to be at the thread of the SRB which has the smallest cross-sectional area. Table 4.17 shows the estimated yield and tensile strengths of the coupler and the SRB, based on the material properties and dimensions of the coupler and SRB shown in Table 4.15 and Table 4.16.

Table 4.17: Nominal yield and ultimate strengths of coupler and SRB (M24)

Section	Coupler		SRB	
	Upper part (A-A')	Lower part (C-C')	Shank	Thread
Yield strength [kN]	603	840	354	246
Ultimate strength [kN]	670	933	421	292
Minimum strength	Yield strength ($F_{y,CP}$) = 603kN Ultimate strength ($F_{u,CP}$) = 670kN		Yield strength ($F_{y,SRB}$) = 246kN Ultimate strength ($F_{u,SRB}$) = 292kN	

(3) The uniaxial test procedure for coupler

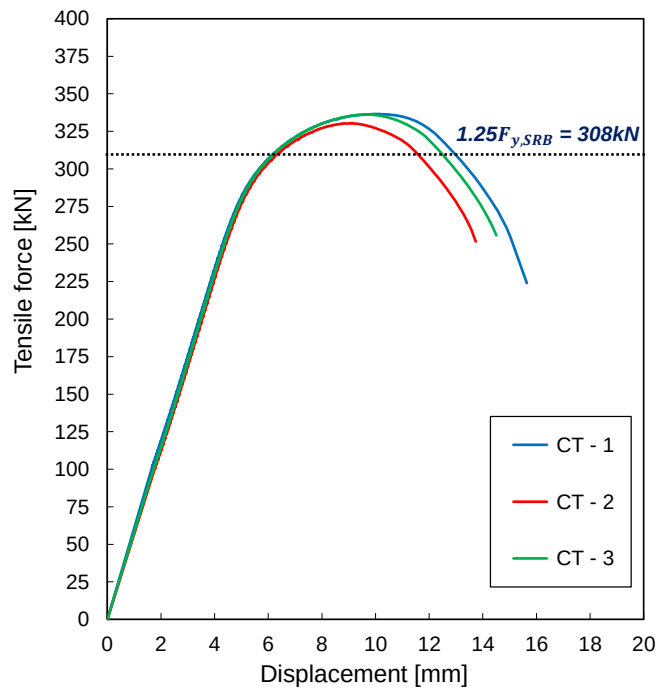
The ISO 15835-1&2 [98] and KS D0249 [99] standards provide the performance requirements and testing protocols for mechanical splices (including a coupler) for reinforced concrete construction. The proposed coupler in this research is different from the couplers used for concrete reinforcing bars, but their related standards nevertheless contain appropriate experimental and performance requirements. Therefore, the experiments were conducted with reference to the standards mentioned above, emphasising the evaluation of strength criteria. Figure 4.15 shows the test setup. A displacement loading rate of 0.2mm/min. was selected. The tensile load was applied to the specimens until fracture occurred. Two strain gauges were attached at the lower and upper parts of the SRBs, and two strain gauges were attached on the surfaces of the coupler for all specimens to measure the strain variation during the test, as shown in Figure 4.15 (b).

(3) Test results

Uniaxial tensile tests were carried out on three specimens, and the results are summarised in Table 4.18. The load-displacement curves, which were collected from the UTM directly, are shown in Figure 4.18. All specimens yielded at loads above the nominal ultimate strength ($F_{u,SRB}=292\text{kN}$) of the SRB, and the final failure occurred in the threads of the SRBs, rather than the coupler. The average yield and ultimate strength of the test assemblies were 296MPa and 334MPa, respectively. Figure 4.19 shows the strain of the coupler and the SRBs measured during the test for each specimen, according to Figure 4.15 (b). The linearity of the CP curves suggests the coupler remained within the elastic range until the threaded part of SRBs failed. In the case of SRBs, recalling that the strains were measured at the bolt shank, rather than the bolt thread, the measured values indicate that the material almost remained in the elastic range until fracture through the bolt thread occurred. However, compared to the strain rate of the coupler, it was confirmed that most of the strain occurred in the shank of the SRBs and, by implication, in the bolt thread area.

Table 4.18: Uniaxial tensile test results for specimens – the coupler and the SRB (M24)

Specimens	Yield load (P_y) [kN]	Ultimate load (P_u) [kN]	Yield displacement (D_y) [mm]	Ultimate displacement (D_u) [mm]	Ductility ratio ($\mu = D_u/D_y$)	Failure mode
CT-1	298	337	5.6	9.9	1.77	Fracture of the SRB
CT-2	293	331	5.5	9.1	1.65	Fracture of the SRB
CT-3	297	336	5.6	9.8	1.75	Fracture of the SRB
Average	296	334	5.6	9.6	1.72	-



(a)



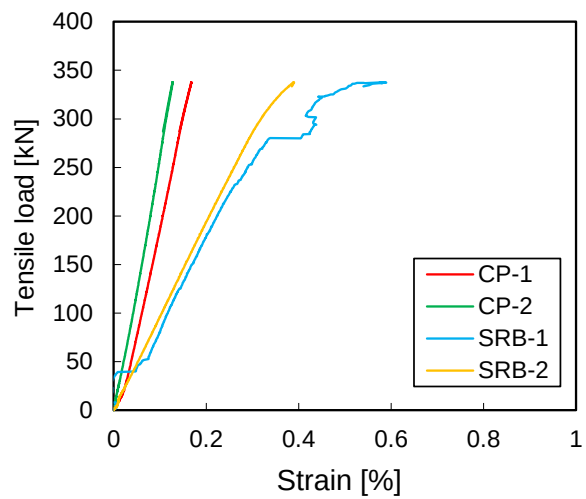
(b)

Figure 4.18: (a) Load-displacement curves from the uniaxial test results for the coupler;
(b) Failure modes: Fracture at the thread of SRBs

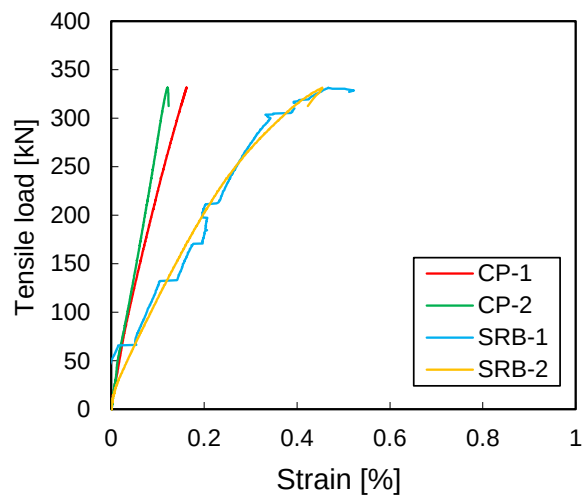
The requirements for mechanical joints in KS D0249 [99] are as follows:

- i) The ultimate strength of the coupler must be at least 125% of the nominal yield strength of the reinforcing bar or at least the tensile strength of the reinforcing bar.
- ii) The fracture location must be on the reinforcing bar, not on the coupler.

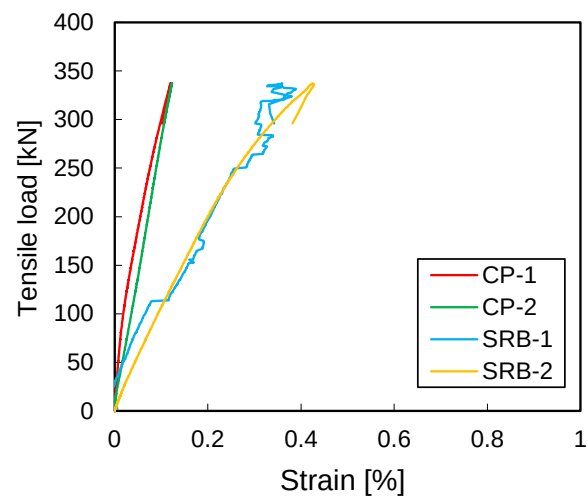
The proposed coupler was evaluated with reference to the above criteria. Based on the test results, the ultimate strength of all specimens was over 1.25 times the nominal yield strength of the SRB ($1.25F_{y,SRB} = 308\text{kN}$), and all failures occurred in the SRB, as shown in Figure 4.18 (a) and (b). Therefore, the test results satisfied the minimum requirements of KS D0249 [99].



(a) CT-1



(b) CT-2



(c) CT-3

Figure 4.19: Load-strain curves of coupler & SRB and image of failure from uniaxial test

4.3. Test of Applying Preload to Steel Rod Bolts

This research proposes the AJ preload connection for a steel-framed modular building, using a long steel rod bolt (approx. 3m in length) inside a rectangular hollow section (**RHS**) column and a newly developed coupler. Applying preload to the SRB with the coupler is the same mechanism of applying preload to a high tensile bolt, as commonly used for connections in steel structures. However, since this long bolt has not previously been used as a preloaded bolt, there is no experimental data for the tensioning of the bolt to a predefined load level. Therefore, it was necessary to confirm through experiments whether it is possible to apply a preload to the long SRB for the proposed AJ connection for steel-framed modular buildings.

Generally, a torque wrench or an impact wrench is used to apply a specific torque to a nut or a bolt for obtaining the demand preload. Currently, the torque method, which controls bolt-tightening by a specific torque value to obtain the preload, is the most widely used method. EN 1090-2:2018 [100] and KBC 2016 [101] provides the torque reference values for the torque method, depending on the magnitude of the preload and the diameter of the bolt. The standards allow calculation of the demand torque required to reach the demand preload and to select the appropriate torque wrench or impact wrench settings for tensioning the bolts. However, it was uncertain whether these torque reference values in the current standards can be applied to the long SRB with the coupler in the proposed connection. Therefore, it was necessary to determine the relationship between the torque applied to the coupler and the resulting preload in the long SRB through a test. The test on applying preload by a manual torque wrench was conducted first to determine the torque coefficient for the 3m-SRB. Then, based on the torque wrench test results, the torque capacity of an impact wrench was selected, and then the tests on applying preload to the SRB with the impact wrench were performed.

One of the important conditions of the proposed connection is that when the upper module frame is disassembled from the lower module frame for reuse, the amount of preload applied to columns in the lower module frame must be constant and not significantly decrease. That is, when removing the preload by loosening the coupler at the upper column while disassembling, it was necessary to check whether the amount of preload in the lower column decreases due to the coupler at the lower column being loosened. This was also assessed in the tests.

4.3.1. Test Procedure and Specimens

Figure 4.20 (a) shows the location of the test assembly in the modular structure. The test for applying preload to the SRB was designed using the full length of a column in the upper module frame (referred to as the **upper column**) and half the length of a column in the lower module frame (referred to as the **lower column**), as shown in Figure 4.20 (c). The lower column was needed in order to measure the change of its applied preload when the upper column was assembled or disassembled. The test was carried out according to the test procedure shown in Figure 4.21. The SRBs, couplers and washers were replaced for each test, but the other components such as the columns and plates were reused.

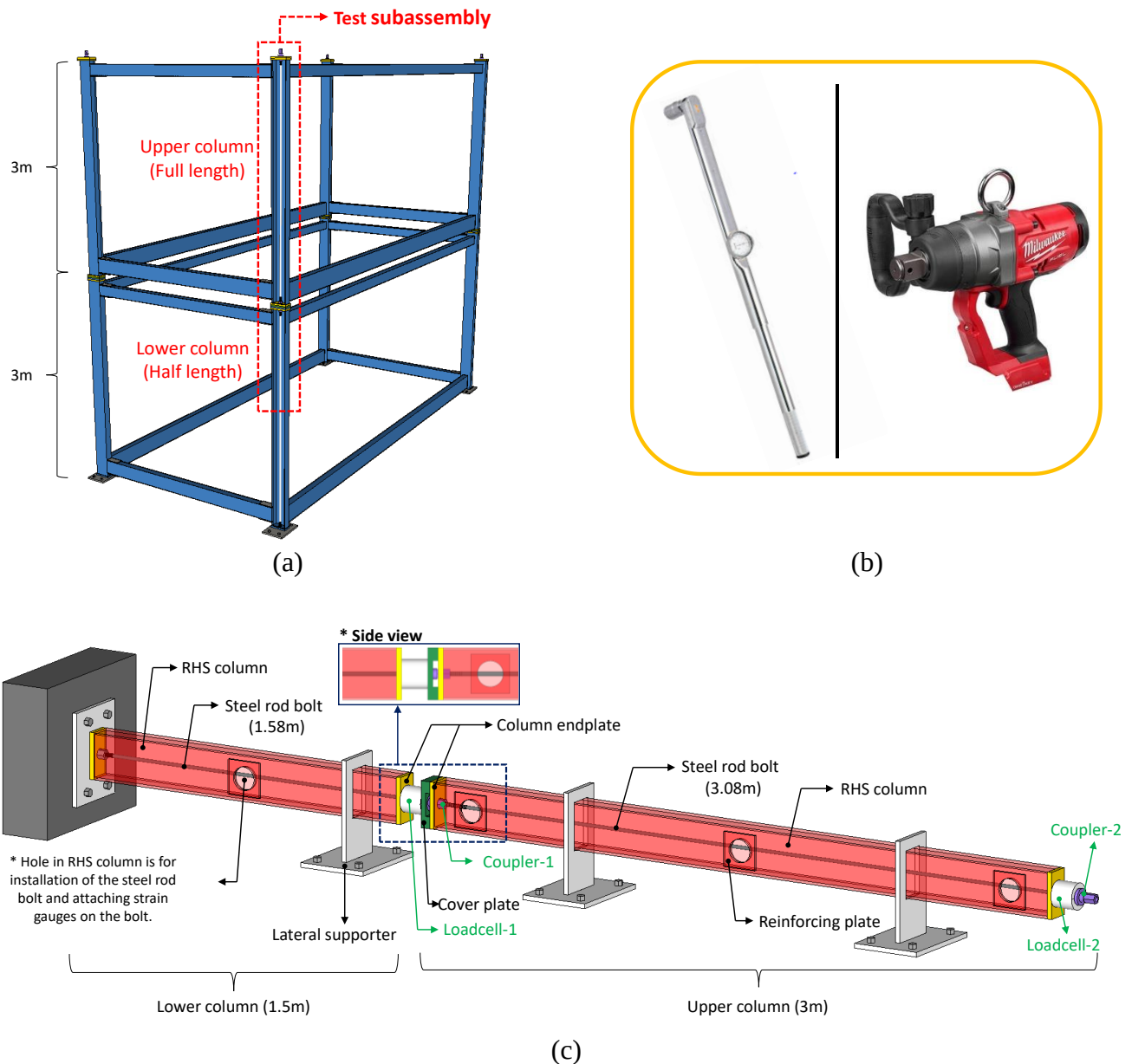


Figure 4.20: (a) Location of the test subassembly; (b) Torque tool: Torque wrench & impact wrench; and (c) Description of test subassembly for the tests on applying preload to long SRBs

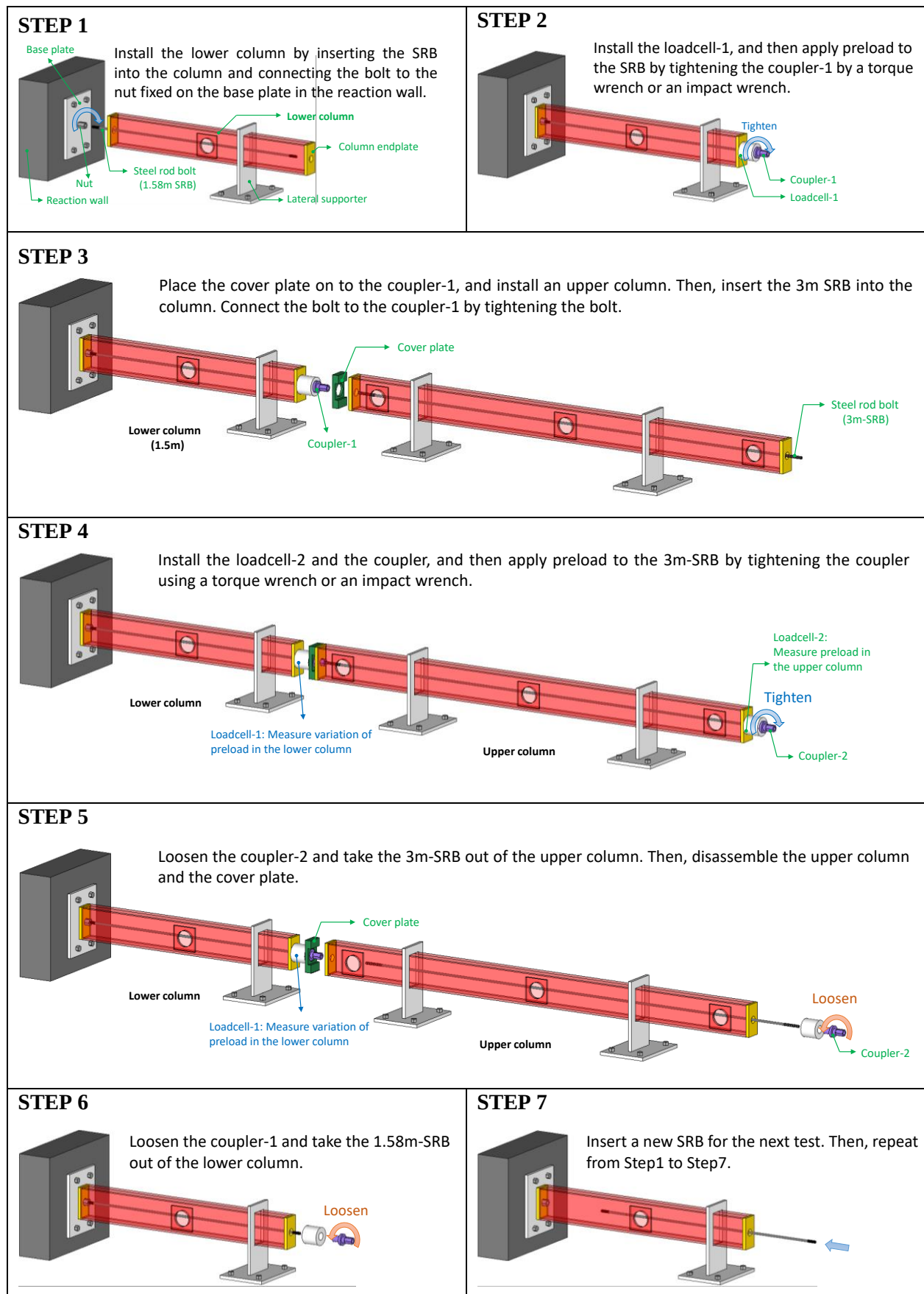


Figure 4.21: Test procedure of applying preload to SRBs

The tests were divided into two types, depending on the tightening tools: a torque wrench or an impact wrench (see Figure 4.20 (b)). The manual torque wrench tests were conducted to investigate the relationship between the tightening torque and the preload of the SRB. Then, the cordless impact wrench test was performed, which is commonly used on construction sites as the most convenient tool for tightening bolts.

A total of 5 specimens were prepared for the tests. Each specimen consisted of 3.08m-SRB and 1.58m-SRB and two couplers. A summary of the test specimens and conditions for the tests is shown in Table 4.19. All torque wrench tests were conducted under the same conditions without surface lubrication. For the cordless impact wrench tests, its battery condition and surface lubrication were considered in the IMW-2-HB and IMW-2-HB-L specimens. All specimens were tested using the procedure shown in Figure 4.21, except for the last two specimens. For these specimens (IMW-2-HB and IMW-2-HB-L), after completing the test of IMW-2, only the coupler installed on the 3.08m-SRB was loosened and replaced with a new coupler, and then preload was applied by tightening the new coupler with different conditions. The material properties and member sizes of the specimens are summarised in Table 4.20.

Table 4.19: Summary of test specimens and conditions for applying preload to high-tensile SRBs

Specimens	Types of wrench	Surface condition	Battery condition	Quantity		
				SRB		Coupler
				1.58m	3.08m	
TW-1	Torque wrench	Non-lubrication	N/A	1	1	2
TW-2	Torque wrench	Non-lubrication	N/A	1	1	2
TW-3	Torque wrench	Non-lubrication	N/A	1	1	2
IMW-1	Impact wrench (Cordless)	Non-lubrication	Fully charged	1	1	2
IMW-2	Impact wrench (Cordless)	Non-lubrication	Fully charged	1	1	2
IMW-2-HB*	Impact wrench (Cordless)	Non-lubrication	Half charged	-	-	1
IMW-2-HB-L*	Impact wrench (Cordless)	Lubrication	Half charged	-	-	1

*After completing the test of IMW-2, only the coupler tightened to the 3.08m-SRB was loosened and replaced by a new coupler, and then preload was applied by tightening the new coupler.

Table 4.20: Material properties and member size

Type	Steel grade	E (GPa)	F_y (MPa)	F_u (MPa)	Member size
Upper column	SRT275	210	275	410	RHS 200 x 100 x 8.8
Lower column	SRT275	210	275	410	RHS 200 x 100 x 8.8
Column endplate	SM355	210	355	490	200 x 100 x 20 ($\Phi 45$)
Cover plate	SM355	210	355	490	200 x 100 x 20 ($\Phi 78$)
Reinforcing plate	SM355	210	355	490	150 x 150 x 10 ($\Phi 78$)
Steel rod	AISI 4140 (=ASTM A193 Grade B7)	210	725	862	$\Phi 24.9$
Coupler	SM45C (=F10T Bolt)	210	900	1000	See Figure 4.16

4.3.2. Test Setup

The test setup is shown in Figure 4.22 and Figure 4.23. The base plate was fixed to the reaction wall using a bolted connection. The lateral supporters were installed at intervals to prevent lateral torsion of the columns, each fixed to the floor using bolted connections. In order to install the SRBs and to attach strain gauges on them, holes were drilled in the webs of the columns, and reinforcing plates were installed by welding around the holes of the columns.

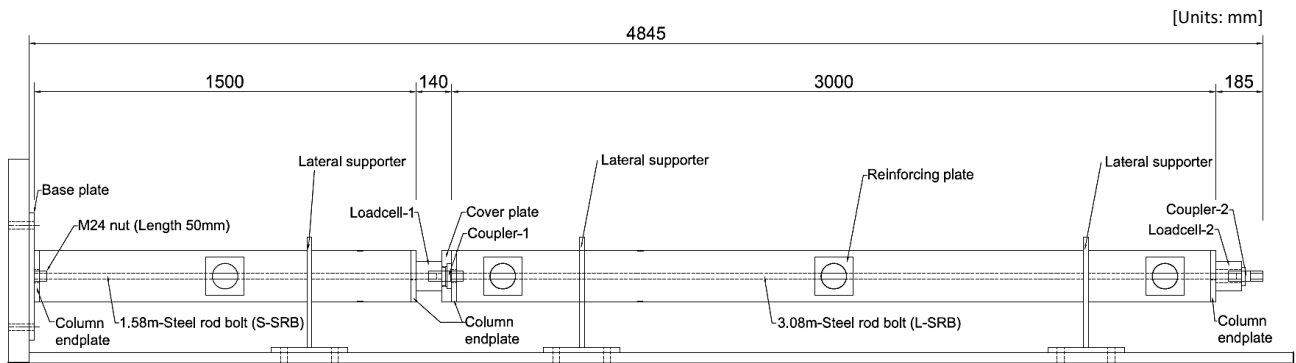


Figure 4.22: Side view drawing of the test assembly

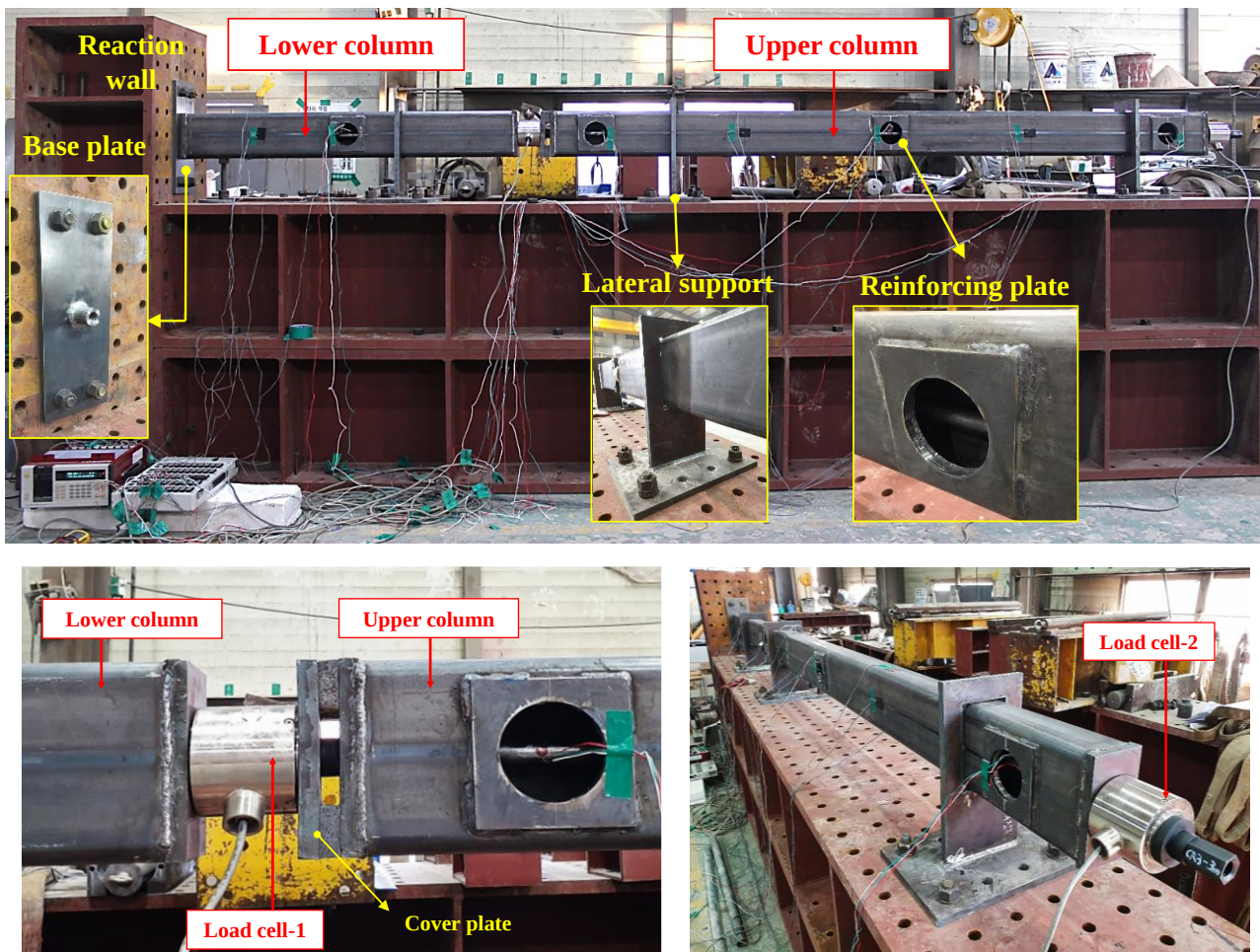
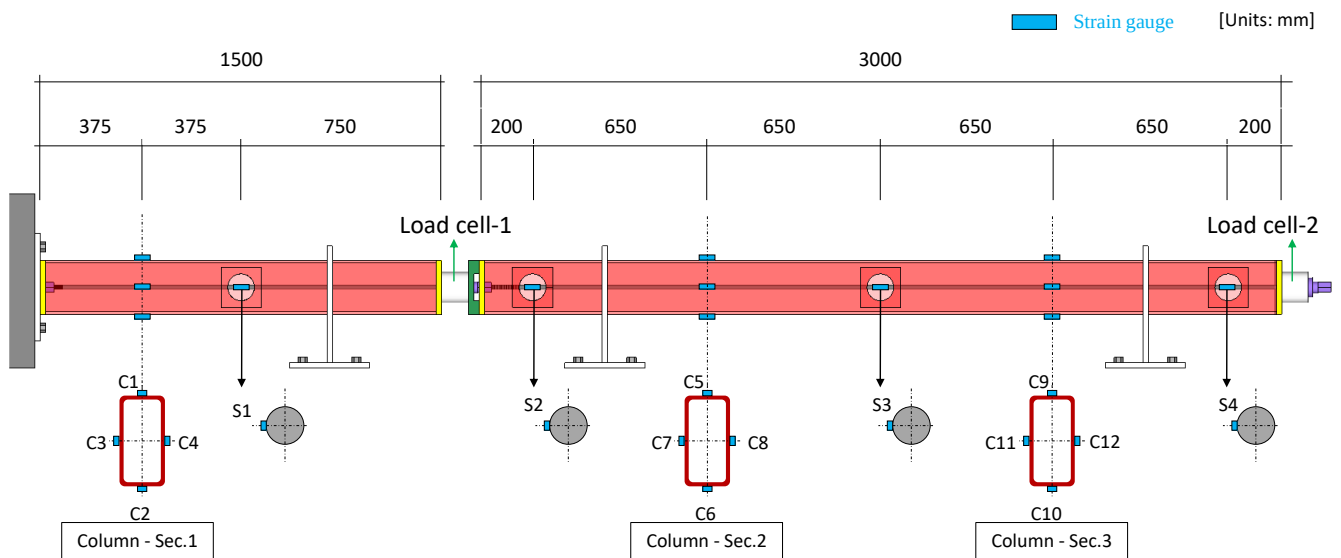


Figure 4.23: Test setup for applying preload to SRBs

4.3.3. Instrumentation

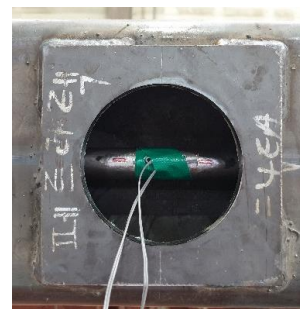
Load cells and strain gauges were used to measure the induced preload, as shown in Figure 4.24. The load cells (each with a capacity of 588kN) were installed on the upper column and the lower column. To measure the distribution of strain in the cross-section of the RHS column, strain gauges were attached at centre of the top and bottom of the flange, and the left and right sides of the web at three locations along the columns (see Column Sec.1, Sec.2, and Sec.3 in Figure 4.24). Strain gauges were also attached on the surface of the 1.58m-SRB at one location in the lower column and on the surface of the 3.08m-SRB at three locations in the upper column (see Figure 4.24).



(a) Load cells and strain gauges: 'C' for columns 'S' for SRBs



(b) Installation of load cell-1 & load cell-2



(c) Strain gauges on the SRB and the column



Figure 4.24: Instrumentation arrangement for the test of applying preload to SRBs

4.3.4. Test Results of the Torque Wrench

To determine the relationship between the torque and preload for the 3.08m-SRB, three torque wrench tests were conducted. A torque wrench with the capability of 1373Nm (140kgf.m) was used for applying the torque.

In the case of the lower column and the 1.58m-SRB (referred to as **S-SRB**), the preload was increased in units of 40kN, and the corresponding torque was measured. For the upper column and the 3.08m-SRB (referred to as **L-SRB**), the preload was increased in units of 20kN, and the corresponding torque was measured. The preload was measured by the load cells (see Figure 4.25 (c)), and the magnitude of the applied torque was measured using a dial torque wrench (see Figure 4.25 (d)). While the upper column was being assembled to and disassembled from the lower column by tightening and loosening the coupler from the L-SRB, the amount of preload in the lower column was measured.

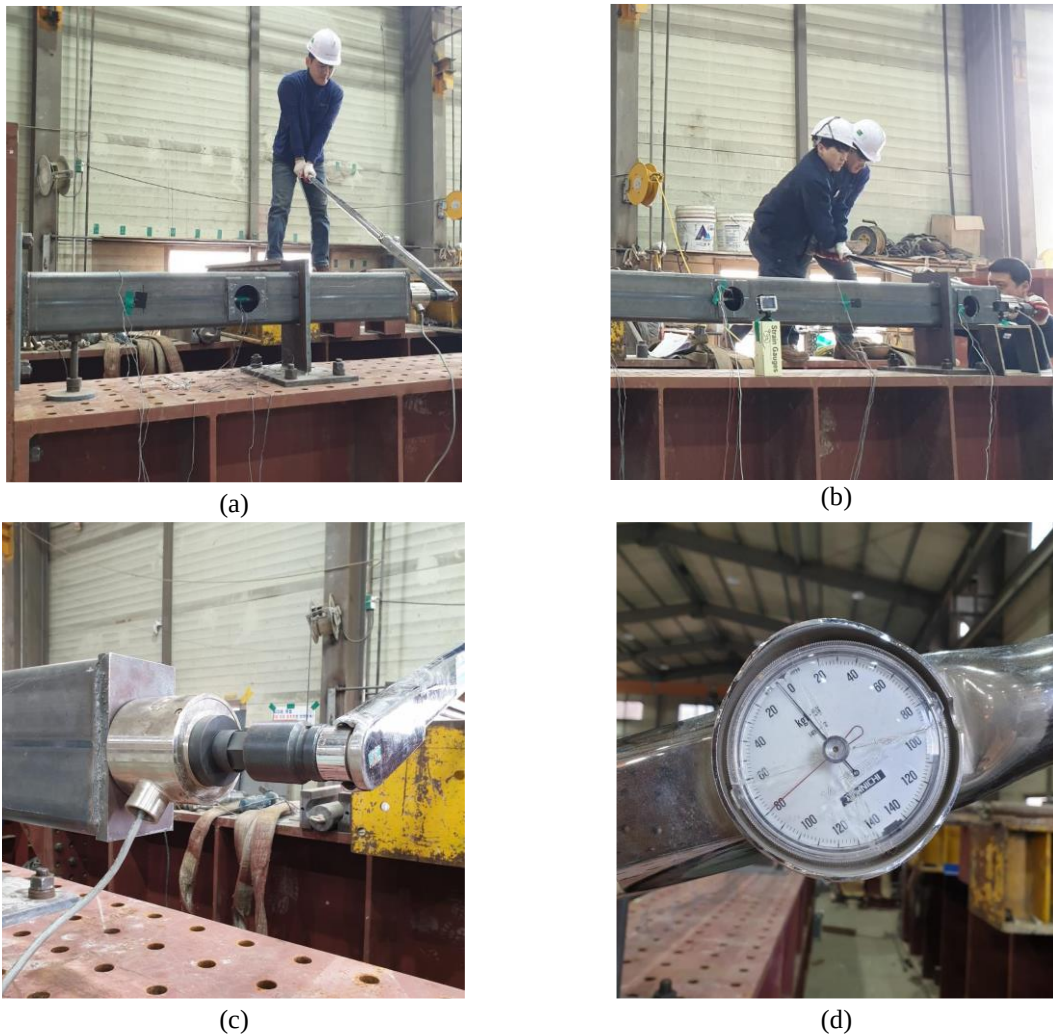


Figure 4.25: Torque wrench test: (a) Applying preload to the lower column, (b) Applying preload to the upper column, (c) Tightening the coupler by a torque wrench, and (d) Measuring applied torque using a dial torque wrench

4.3.4.1. Relationship Between Torque and Preload, and Torque Coefficient

The relationships between the applied torque and preload for each of the specimens TW-1, TW-2, and TW-3 are drawn in Figure 4.26 (a), (b), and (c). The test results are approximately linear, whereas theoretically, the relationship is perfectly linear [102]. However, it is impossible to obtain a perfectly linear response from an experiment due to dispersion, uncertainty, and measurement error in the tightening process [103].

In the case of specimen TW-1, until reaching the preload of 100kN, the demand torque was similar for the S-SRB and the L-SRB. From the preload of 100kN upwards, in order to reach the preload of 160kN, a torque 1.15 times higher was required for the L-SRB than for the S-SRB. For the TW-2 and TW-3 specimens, the demand torque for the L-SRB was 1.8 and 1.3 times higher, respectively, than for the S-SRB to obtain the same magnitude of preload.

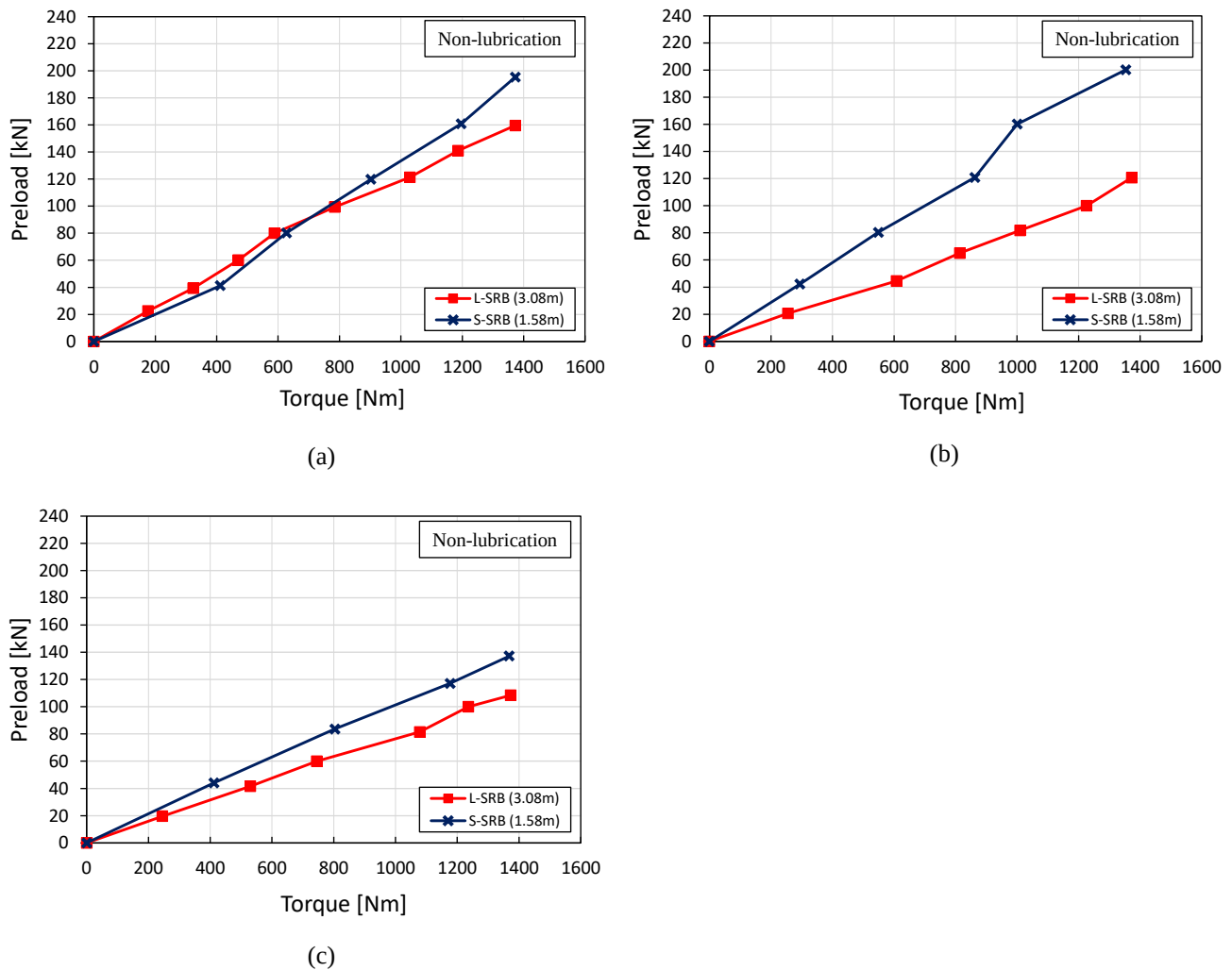


Figure 4.26: Torque-preload curves from torque wrench test: (a) TW-1, (b) TW-2, and (c) TW-3

The torque-preload relationship is often simplified by a linear curve defined by constant K , which is the torque coefficient or nut factor. With the torque coefficient (K), the widely used short-form torque-preload equation is expressed in (Eq. 4.1) [104]. Some authors [105-108] suggest the approximate or mean value of 0.2 as the torque coefficient for steel fasteners, such as screws and nuts made of steel. The Korean standards KS B 1010: 2009 [95] provides $K=0.2$ for the non-lubricated surface of a nut and $K=0.15$ for the lubricated surface of a nut for a high tensile bolt.

$$T = K \cdot F \cdot D \quad (\text{Eq. 4.1})$$

T (N mm): The input tightening torque applied to the fastener head or nut

F (N): The achieved preload

D (mm): The nominal thread diameter

Table 4.21 shows the measured preload and torque for the L-SRBs and the S-SRBs during the test. Based on the data, the torque coefficient K could be derived through (Eq. 4.1) using the nominal thread diameter of the SRB in Table 4.16. The average torque coefficients of the L-SRB and the S-SRB are 0.53 and 0.39, respectively. The fitted linear torque-preload curves for the L-SRBs and the S-SRBs are illustrated in Figure 4.27 (a) and (b), as are the measured curves. Additionally, the theoretical torque-preload relationship for an M24 bolt (having the same bolt size as the SRBs) with a non-lubricated surface of a nut ($K=0.2$) is also plotted to provide a comparison with the L-SRB and the S-SRB. The torque coefficient for the L-SRB and the S-SRB are approximately 2.7 and 2.0 times higher than that of the M24 bolt (see Figure 4.27 and Table 4.22).

In other words, to obtain the same magnitude of preload, a 2.65 times higher torque is required for the L-SRB, compared to the M24 bolt. From the test results, it was found that the demand torque tends to increase as the length of the bolt increases for obtaining the same magnitude of preload.

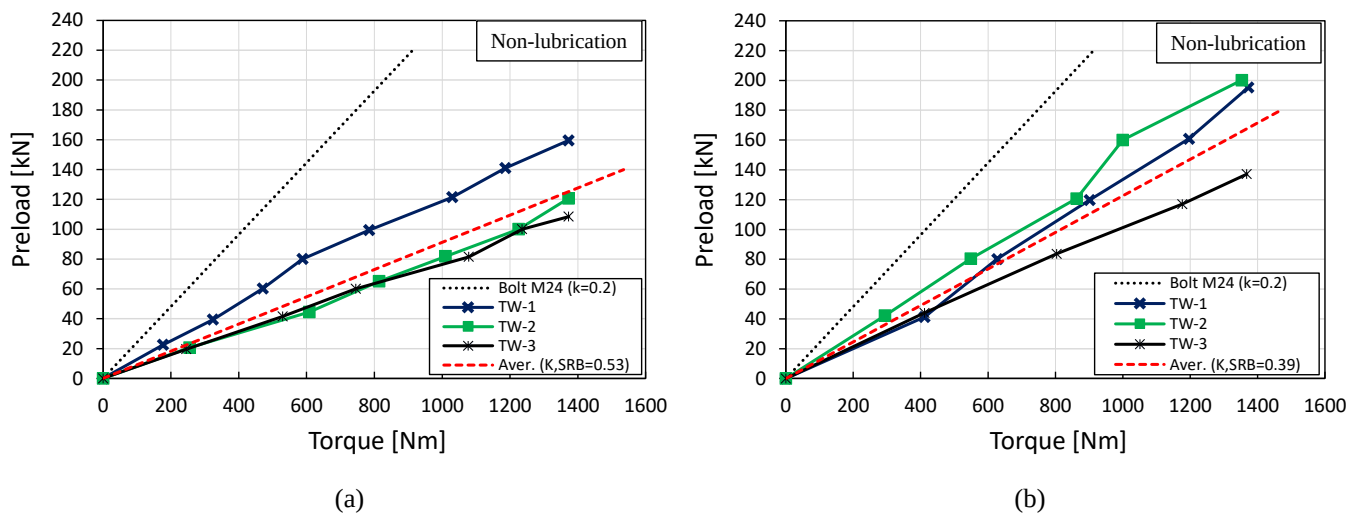


Figure 4.27: Torque-preload curves from torque wrench test: (a) L-SRB (3.08m) and (b) S-SRB (1.58m)

Table 4.21: Experimental results for L-SRB (3.08m) and S-SRB (1.58m) - Torque wrench test

L-SRB: 3.08m-steel rod bolt

TW-1			TW-2			TW-3		
Preload [kN]	Torque [Nm]	Torque coefficient (K)	Preload [kN]	Torque [Nm]	Torque coefficient (K)	Preload [kN]	Torque [Nm]	Torque coefficient (K)
23	177	0.38	21	255	0.60	20	245	0.60
39	324	0.40	45	608	0.66	42	530	0.61
60	471	0.38	65	814	0.60	60	745	0.60
80	588	0.35	82	1010	0.60	81	1079	0.64
99	785	0.38	100	1226	0.59	100	1236	0.60
121	1030	0.41	121	1373	0.55	108	1373	0.61
141	1187	0.41				170	Over*	-
160	1373	0.41						
184	Over*	-						
Mean value of torque coefficient		0.39	Mean value of torque coefficient		0.58	Mean value of torque coefficient		0.61

S-SRB: 1.58m-steel rod bolt

TW-1			TW-2			TW-3		
Preload [kN]	Torque [Nm]	Torque coefficient (K)	Preload [kN]	Torque [Nm]	Torque coefficient (K)	Preload [kN]	Torque [Nm]	Torque coefficient (K)
41	412	0.48	42	294	0.34	44	412	0.45
80	628	0.38	80	549	0.33	84	804	0.46
120	902	0.36	121	863	0.34	117	1177	0.48
161	1196	0.36	160	1000	0.30	137	1368	0.48
195	1373	0.34	200	1353	0.33	180	Over*	-
			211	Over*	-			
Mean value of torque coefficient		0.38	Mean value of torque coefficient		0.33	Mean value of torque coefficient		0.47

* Since the capacity of the torque wrench was exceeded, the amount of torque could not be measured.

Table 4.22: Average torque coefficient of L-SRB (3.08m) and S-SRB (1.58m)

Type	Torque coefficient (K)	Ratio (K_{SRB}/K_{M24})
Steel rod bolt - 3.08m ($K_{L,SRB}$)	0.53	2.65
Steel rod bolt - 1.58m ($K_{S,SRB}$)	0.39	1.98
M24 high tensile bolt (K_{M24})	0.20	1.00

4.3.4.2. Impact on Preload of the Lower Column During Assembly and Disassembly of the Upper Column

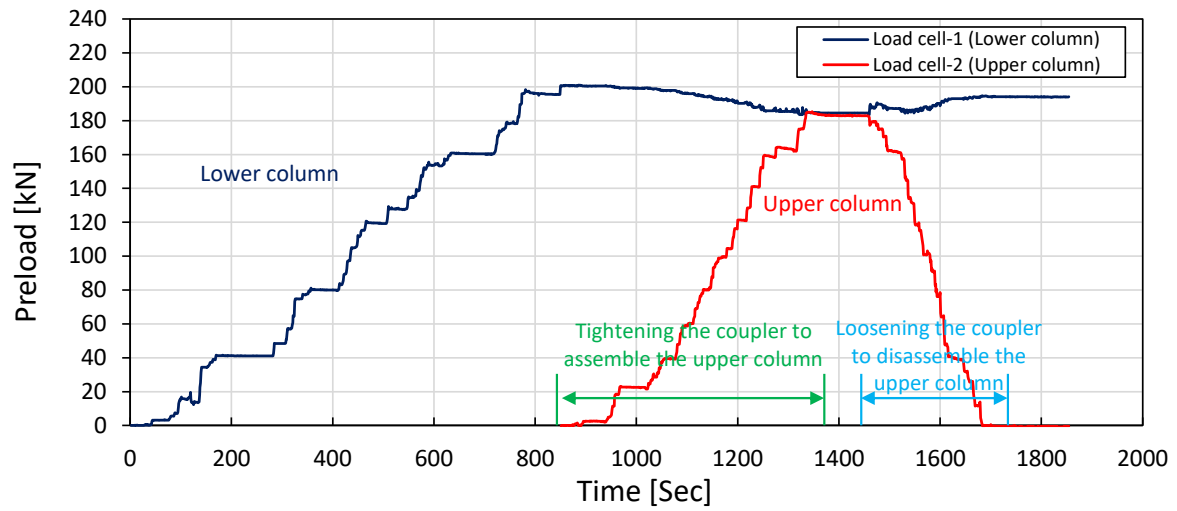
After the upper module is disassembled from the lower module for reuse, the preload in the columns in the lower module should be constant and not significantly decrease. To assess this, the change of the preload in the lower column was measured while the upper column was being assembled to and disassembled from the lower column.

In the tests, the upper column was assembled and disassembled in the condition where the amount of the preload in the lower column was higher than in the upper column. The torque required for L-SRB in the upper column to apply preload was much higher than the torque required for S-SRB in the lower column. This caused the maximum applied preload in the upper column to be lower than the one in the lower column. For the TW-1 and TW-3 specimens, the differences between the preloads applied to the lower and the upper columns was 10kN and 15kN, respectively. On the other hand, in the case of TW-2, there was a difference of about 60kN.

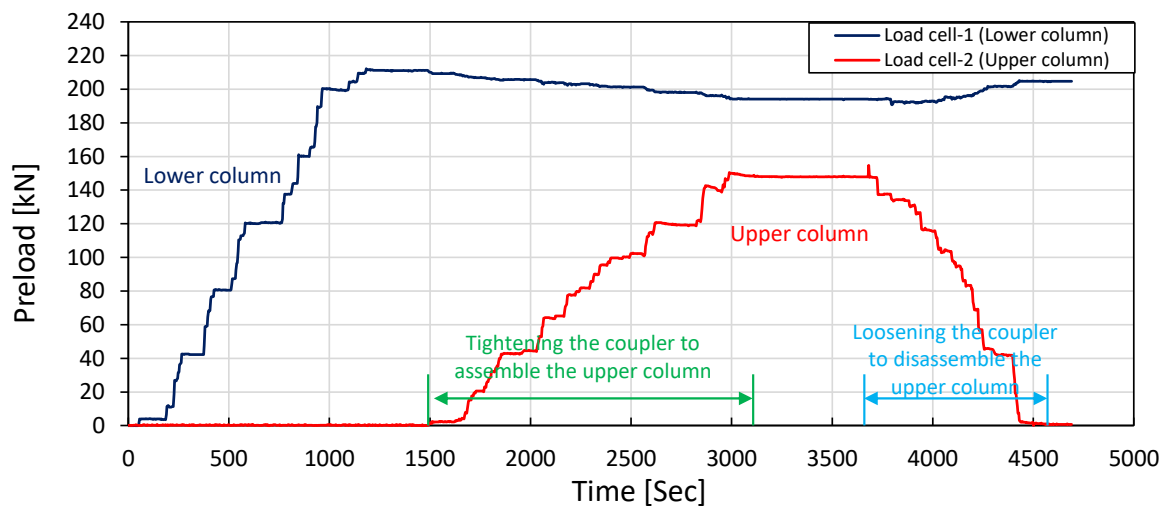
Table 4.23 and Figure 4.28 show the test results for the TW-1, TW-2, and TW-3 specimen. For all three specimens, when the preload in the upper column reached the maximum value, indicating the upper column was completely assembled to the lower column, the preload in the lower column tended to decrease by 7-8%, compared to the initial preload. Subsequently, when the upper column was disassembled by loosening the coupler, the preload in the lower column returned to approximately the same level as the initial preload (see Figure 4.28). The test confirmed that there was little impact on the preload in the lower column when the upper column was assembled to and disassembled from the lower column.

Table 4.23: Change of preload in the lower column during the assembling of and disassembling from the upper column - Torque wrench test

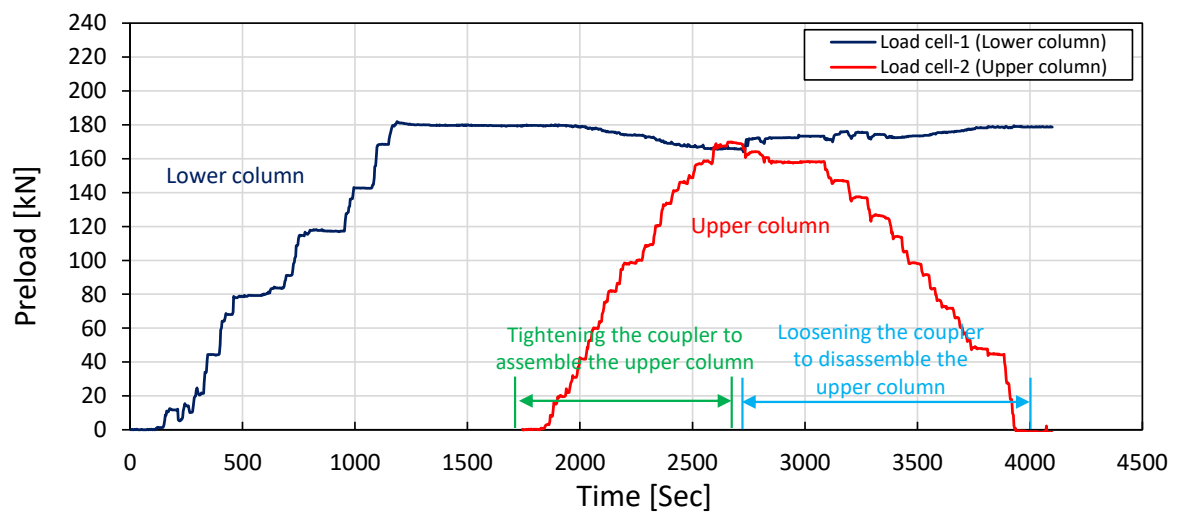
Specimens	Lower column (1.5m)			Upper column (3m)
	Initial preload	Preload [kN] After assembling of the upper column	After disassembling of the upper column	Applied preload [kN]
TW-1	200	185	195	185
TW-2	210	194	205	148
TW-3	180	168	179	169



(a)



(b)



(c)

Figure 4.28: Change of preload in the lower column during the assembling of and disassembling from the upper column:
(a) TW-1, (b) TW-2 and (c) TW-3

4.3.5. Test Result of the Impact Wrench

When modules are assembled on-site using the AJ connection, the preload is likely to be applied to the SRB by an impact wrench. Therefore, it is necessary to conduct tests with an impact wrench to check whether the demand preload is achieved.

Based on the torque wrench test results, the torque capacity of the impact wrench was selected. The test results (refer to Figure 4.27 (a)) showed that the average demand torque required for applying a preload of 130kN to the 3.08m steel rod bolt (L-SRB) was approximately 1400Nm. In order to apply 200kN of preload, the required torque capacity of the impact wrench was 2000Nm or greater.

To conduct the impact wrench tests, a cordless impact wrench, which had a torque capacity of 2033Nm was selected. The reason for choosing a cordless impact wrench was to increase the convenience of working on the construction site. The chosen impact wrench had four different settings of torque capacity: 530 Nm, 1020Nm, 1800Nm, and 2033Nm. During the tests, the torque capacity of the impact wrench was increased, and the corresponding preload was measured. The change of the preload in the lower column was also measured while the upper column was assembled to and disassembled from the lower column.

Two specimens (IMW-1 and IMW-2) were prepared for the test. The tests were carried out using the same procedure as for the torque wrench tests (see Figure 4.21). The battery of the impact wrench was fully charged for these tests. In the case of IMW-2-HB and IMW-2-HB-L, only the coupler tightened to the 3.08m-steel rod bolt (L-SRB) in the IMW-2 specimen was loosened and replaced by a new coupler, and then preload was applied by tightening the new coupler. For these tests, the battery of the impact wrench was half-charged to check the effect of the battery condition. For the IMW-2-HB-L test, the coupler surfaces and the thread of the L-SRB were lubricated (refer to Table 4.19).

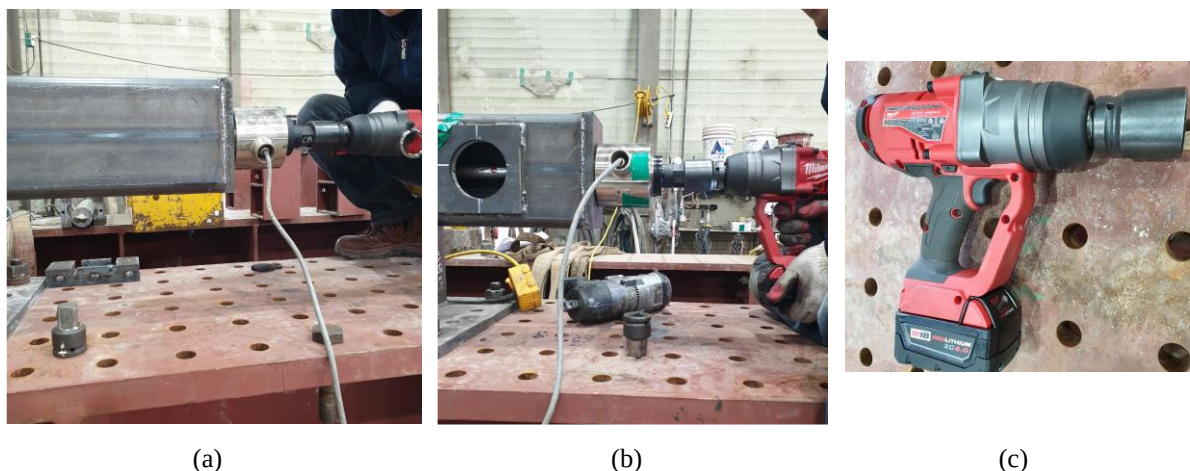


Figure 4.29: Impact wrench test: (a) Applying preload to the lower column; (b) Applying preload to the upper column; and (c) Cordless impact wrench used for the test

4.3.5.1. Relationship Between Torque and Preload

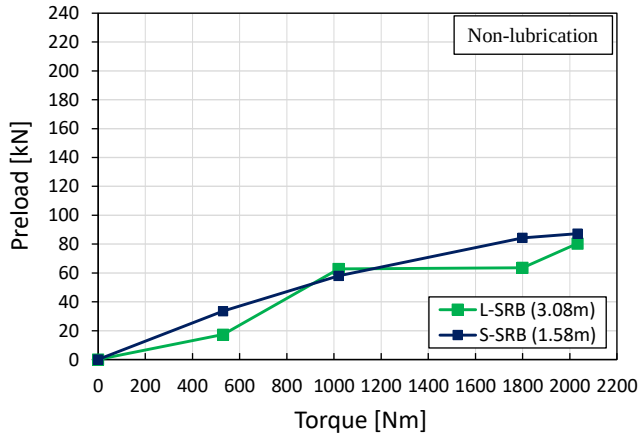
The measured relationships between the torque and preload for each of the IMW-1 and IMW-2 specimens are shown in Figure 4.30 and Table 4.24. The test results of the L-SRBs and the S-SRBs from specimens IMW-1 and IMW-2 are presented in Figure 4.31. Compared to the torque wrench tests (refer to Figure 4.26), a much higher torque was applied to the L-SRB and the S-SRB but contrary to what was expected, the applied preload was lower than the average preload from the torque wrench test, as shown in Figure 4.30, and even though the magnitude of the torque capacity of the impact wrench increased from 1000Nm to 1800Nm, the amount of preload increased only very slightly in both the L-SRB and the S-SRB (see Figure 4.30 (a) and (b)).

In the case of the L-SRB, the IMW-1 and IMW-2 tests showed similar patterns of the torque-preload relationship as shown in Figure 4.31 (a), but overall, the IMW-1 test induced about 20kN lower preload for the same amount of the torque than the IMW-2 test. In the case of the S-SRB, the IMW-1 and IMW-2 tests showed similar results, but compared to the average preload from the torque wrench tests, the applied preload was significantly lower, typically about a half, for the same amount of the torque (see Figure 4.31 (b)).

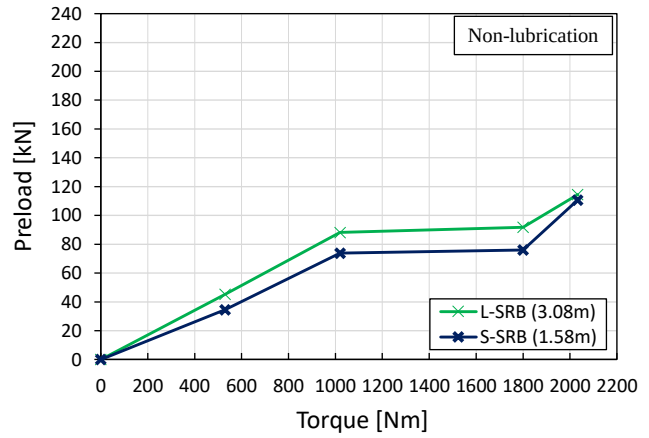
While the torque wrench tests showed that the demand torque to obtain the same magnitude of the preload increases as the length of the bolt increases, the results of the impact wrench tests showed almost no difference between the L-SRB and the S-SRB. In fact, a higher preload was applied to the L-SRB than the S-SRB in the IMW-2 test for the same magnitude of torque (see Figure 4.30 (b)).

Table 4.24: Amount of preload applied according to fastening torque by the cordless impact wrench [unit: kN]

Specimens	Steel rod bolt	Fastening torque			
		530 Nm	1020 Nm	1800 Nm	2033 Nm
IMW-1	1.58m (S-SRB)	34	58	84	87
	3.08m (L-SRB)	17	63	64	80
IMW-2	1.58m (S-SRB)	35	74	76	111
	3.08m (L-SRB)	45	88	92	115

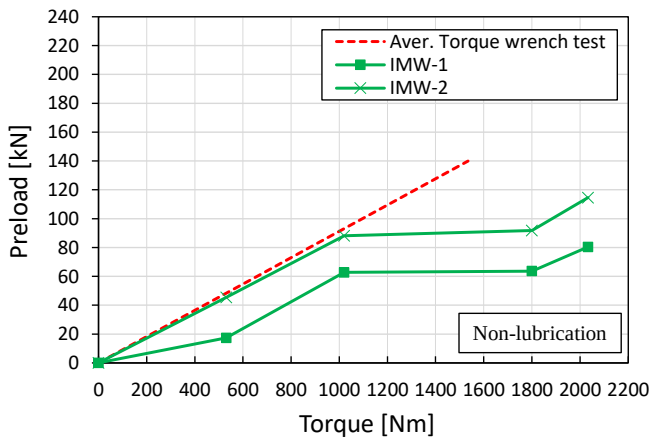


(a)

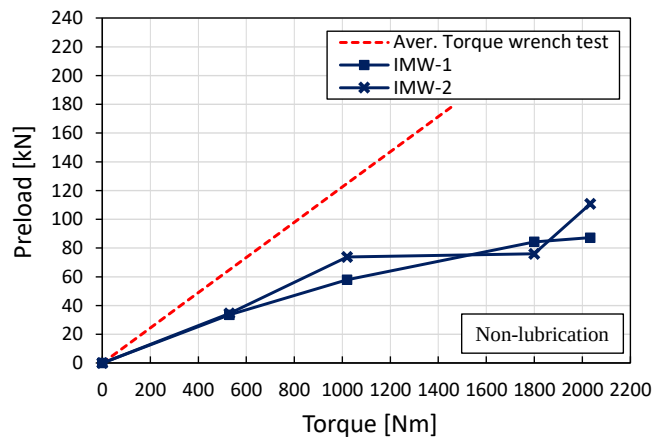


(b)

Figure 4.30: Torque-preload curves from the impact wrench tests: (a) IMW-1 and (b) IMW-2



(a)



(b)

Figure 4.31: Comparison of the impact wrench test results and the torque wrench test result in the torque-preload curve: (a) L-SRB (3.08m) and (b) S-SRB (1.58m)

4.3.5.2. Effect of Battery Charge of the Impact Wrench and Lubrication on Preload

The performance of the cordless impact wrench may depend on the battery state of charge. Therefore, additional tests were conducted using the IMW-2 test specimens. After completing the test for IMW-2, only the used coupler for the L-SRB was replaced with a new coupler, and then a preload was induced by tightening the new coupler. The impact wrench with a half-charged battery was used for the IMW-2-HB and IMW-2-HB-L tests. For the IMP-2-HB-L test, a lubricant was applied to the coupler surface, the washer surface in contact with load cell-2 and the bolt thread of the L-SRB. The maximum torque of 2033 Nm was applied to the new couplers in the IMW-2-HB and IMP-2-HB-L test specimens.

Figure 4.32 shows the preload induced in the L-SRB by the applied torque of 2033Nm. The preload in the IMW-2-HB test was reduced by 13% and 40%, respectively, compared to the preload induced by the impact wrench with a fully charged battery (IMW-1 and IMW-2). This means that the battery state of charge of the cordless impact wrench affects the preload. In the case of the lubricated IMW-2-HB-L test specimen, the test results showed that the preload applied to the L-SRB was increased by 1.73 times compared to the non-lubricated IMW-2-HB specimen.

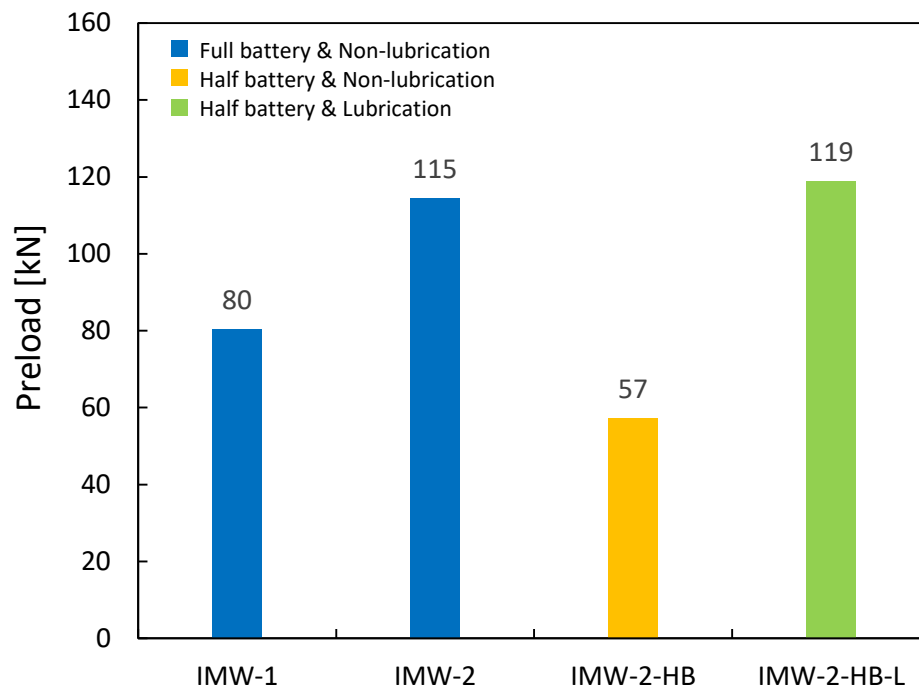


Figure 4.32: Amount of preload induced in the 3.08m-steel rod bolt (L-SRB) by a torque of 2033Nm using the impact wrench

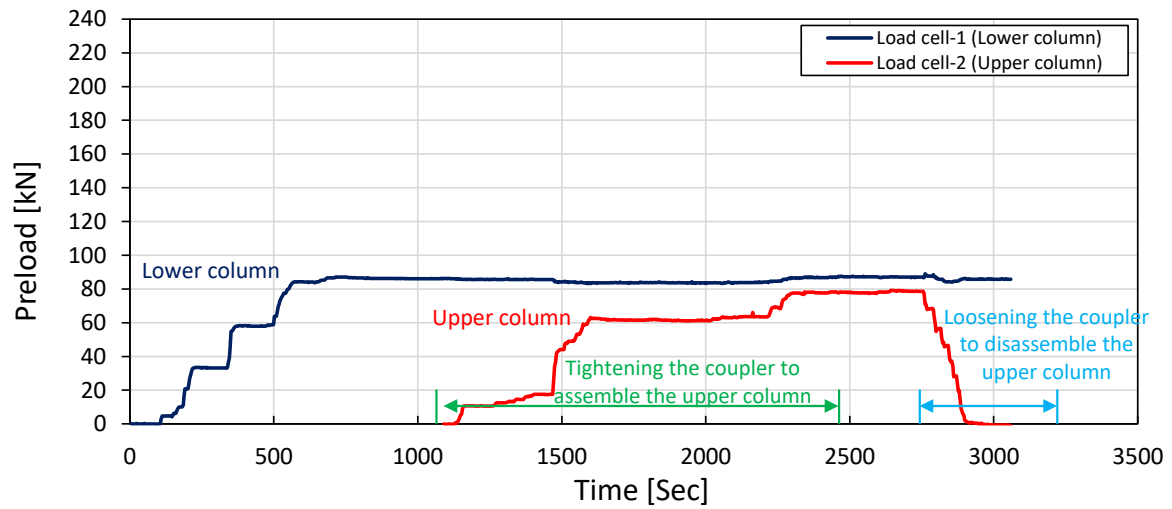
4.3.5.3. Impact on Preload of the Lower Column During Assembly and Disassembly of the Upper Column

The change of the preload in the lower column was measured while the upper column was being assembled to and disassembled from the lower column. Table 4.25 and Figure 4.33 show the test results for each specimen of the IMW-1 and IMW-2 tests. The results of the IMW-2-HB and IMW-2-HB-L tests are presented together with the IMW-2 test results in Figure 4.33 (b) because the IMW-2-HB and IMW-2-HB-L tests were conducted by replacing only the coupler tightened to L-SRB after completing the IMW-2 test.

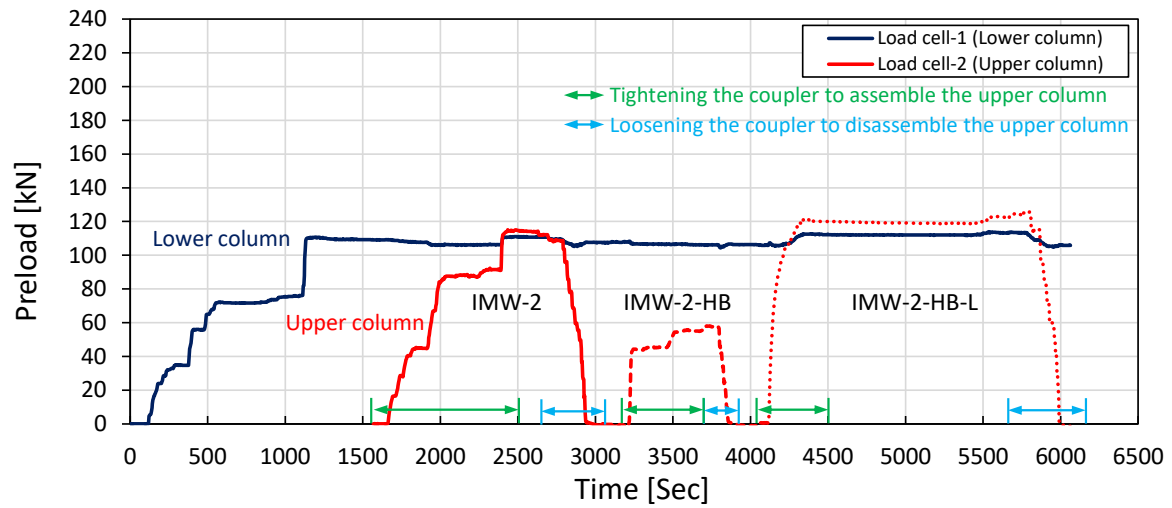
For all specimens, when the preload in the upper column reached the maximum value, the change of preload in the lower column was less than 5%, compared to the initial preload. When the upper column was disassembled by loosening the coupler, the preload in the lower column for all specimens returned to approximately the same level as the initial preload (see Table 4.25 and Figure 4.33). The test confirmed that there was little impact on the preload in the lower column when the upper column was assembled to and disassembled from the lower column.

Table 4.25: Change of preload in the lower column during the assembling of and disassembling from the upper column - Impact wrench test

Specimens	Lower column (1.5m)			Upper column (3m)
	Preload [kN]			Applied preload [kN]
	Initial preload	After assembling of the upper column	After disassembling of the upper column	
IMW-1	87	87	86	80
IMW-2	111	110	109	115
IMW-2-HB	108	106	106	57
IMW-2-HB-L	106	111	106	119



(a)



(b)

Figure 4.33: Change of preload in the lower column during the assembling of and disassembling from the upper column: (a) IMW-1 and (b) IMW-2, IMW-2-HB, and IMW-2-HB-L

4.3.6. Comparison of Preloads Measured by Strain Gauges and Load Cell

The strains measured on SRBs and columns by strain gauges were converted to stresses. They were then compared to the stress obtained by the load cell in order to check whether it is possible to measure the preload accurately without installing load cells in a beam-to-column joint test (or a future full-scale test of a module frame).

Figure 4.24 shows the locations of strain gauges on the test specimens. The strain gauges were attached symmetrically to both sides of the flange and web of the columns to be able to eliminate bending effects. In the case of the SRB, the strain gauges were attached to only one side. Table 4.27 - Table 4.36 present the stress variation measured by strain gauges on the column and the SRB for each test specimen until the preload was completely applied. The axial strain measured on the column section and the SRBs during the tests were converted to the stress by multiplying the measured values of Young's modulus (E), and the axial forces measured by the load cells were divided by the respective cross-sectional areas of the column and the SRBs to convert to stress.

Figure 4.34 shows the comparison of stress measured by the load cells (σ_{LC}) and the strain gauges (σ_{st}) on columns and SRBs, as obtained from the torque wrench and impact wrench tests. The y-axis is the ratio of stress obtained from the strain gauge readings to stress obtained from the load cell readings. Figure 4.34 shows in separate figures, the average stress measured on the column section based on the stresses at the centre of the top and bottom flanges, the centre of the left and right sides of the web, and all stresses on the flanges and webs. Overall, the stress obtained at the column section was much closer to the stress measured from the load cell, compared to the stress obtained on the SRB. In the case of the column section, the difference between the stresses measured by the strain gauges and the load cell ranged between 0% and 20%. Most of them had a difference of less than 10% (over half of them were within 5%). On the other hand, in the case of the stress measured on the SRB, the difference ranged between 9% and 35%. Due to the rounded surface and the small diameter of the SRB, it seems that the strain gauges may not have been accurately attached in the horizontal direction, causing the measured strain to be not purely axial.

Regarding the measuring locations on the column section, there was no significant difference between the flange and the web. In addition, the test results showed that the average stress measured on the flange or the web (using two strain gauges) were almost the same as the average stress measured on the flange and the web (using four strain gauges).

In conclusion, from the test results, it was found that it is more efficient to measure the preload by attaching strain gauges symmetrically to the flange or web on the column section than to the SRBs.

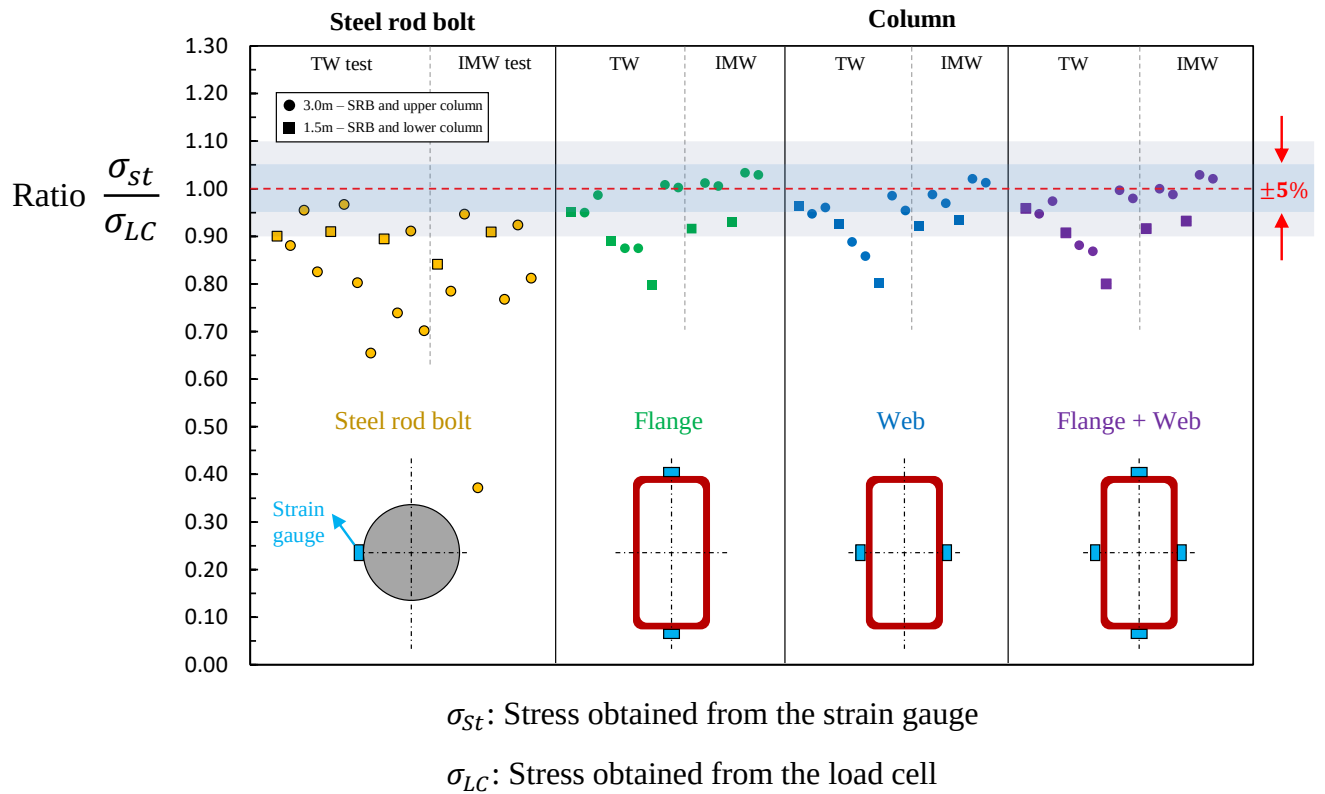


Figure 4.34: Comparison of stress measured by load cells and strain gauges on columns and SRBs

4.3.7. Distribution of Preload on Column Section

The amount of the demand preload in the proposed connection is determined to prevent the gap-opening of the connections between columns that may otherwise occur when a lateral force is applied. The distribution of preload stress on the cross-section of the column can be related to the gap-opening. Ideally, the preload would be distributed uniformly on the cross-section. However, when the module units are assembled on-site, it is impossible to apply preload uniformly on the column section, partly because it is difficult to locate the coupler exactly at centre of the cross-section of the column when applying the preload, and partly because the columns are not manufactured perfectly straight. These eccentricities will produce a bending stress on the column cross-section, and hence a non-uniformly distributed preload stress.

During the tests, the distributions of the stresses on the cross-sections of the upper and lower columns were measured by strain gauges, and the measured stresses were compared to the uniformly distributed stress. Table 4.27 - Table 4.36 show the stress measured on the columns and SRBs for each specimen up to the time the preload was applied completely to the upper and lower columns. The left side in the tables shows the stress measured by each strain gauge on the column sections (see Column - Sec.1, Sec.2 and Sec.3). The stress measured by the load cell ($\sigma_{LC,Col}$) was obtained as shown by the horizontal blue dotted line in the graphs, representing the uniformly distributed stress.

Table 4.26 summarises the ratio of the stress measured on the column by each strain gauge to the uniformly distributed stress for the upper and lower columns. The distributions of stress measured on the upper and lower columns demonstrate that the preload was not uniformly applied to the cross-section of the columns. Compared to the uniformly distributed stress on the cross-section, the largest difference was 62% for the upper column and 34% for the lower column.

It is predicted that this non-uniformity of the distributed preload stress will cause the areas with relatively low stress to develop premature gap-opening. Therefore, it is necessary to consider the effect of the non-uniformly distributed preload stress during the design of the connection.

Table 4.26: Ratio of the stress measured on the column section by strain gauges to the uniformly distributed stress

Upper column (3m)

Specimens	Load cell data		Ratio (Measured stress / Uniform stress)							
	Applied preload [kN]	Uniform compressive stress* [MPa]	Column - Sec.2				Column - Sec.3			
		Column $\sigma_{LC,Col}$								
			(C5)	(C6)	(C7)	(C8)	(C9)	(C10)	(C11)	(C12)
TW-1	185	-38.1	1.12	0.77	0.59	1.29	1.07	0.92	0.85	1.07
TW-2	154	-30.4	1.34	0.41	0.69	1.08	1.01	0.75	0.82	0.89
TW-3	170	-35.0	1.24	0.78	0.38	1.59	1.11	0.89	0.78	1.13
IMW-1	80	-16.5	1.35	0.67	0.42	1.55	1.14	0.87	0.74	1.20
IMW-2	115	-23.7	1.17	0.90	0.78	1.26	1.08	0.97	0.95	1.07

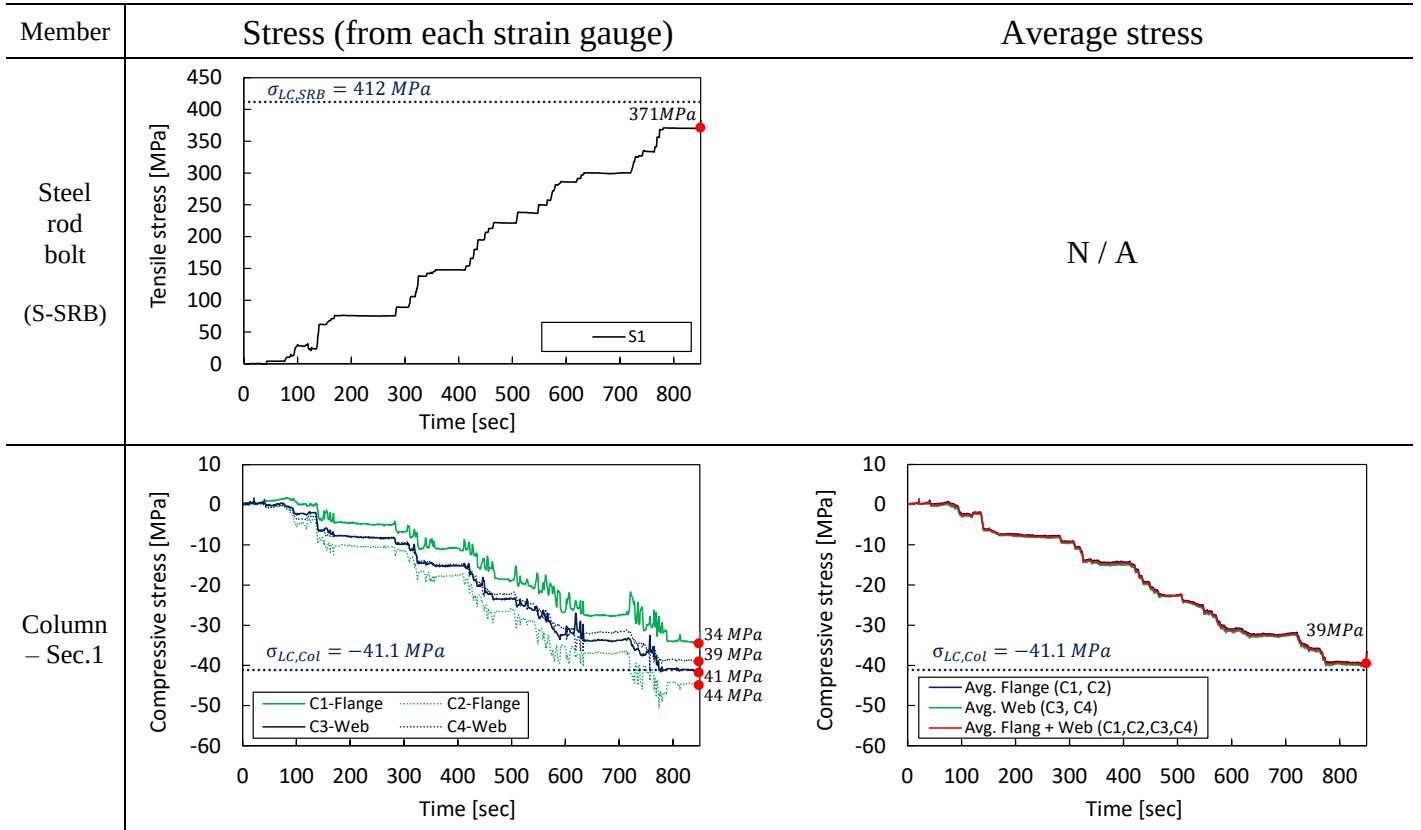
Lower column (1.5m)

Specimens	Load cell data		Ratio (Measured stress / Uniform stress)							
	Applied preload [kN]	Uniform compressive stress* [MPa]	Column - Sec.1							
		Column $\sigma_{LC,Col}$								
			(C1)	(C2)	(C3)	(C4)				
TW-1	200	-41.1	0.83	1.08	0.99	0.94				
TW-2	210	-43.2	0.76	1.02	0.89	0.96				
TW-3	180	-37.0	0.66	0.94	0.84	0.76				
IMW-1	87	-17.9	0.76	1.07	0.97	0.87				
IMW-2	111	-22.8	0.77	1.09	0.99	0.88				

* Preload measured by the load cell was divided by the nominal cross-sectional area of the column in Table 4.20

(): Strain gauge number in Figure 4.24

Table 4.27: Stress variation measured by strain gauges - 1.5m lower column and SRB in TW-1



A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-1**	200	412	-41.1

**B. Strain gauge
– SRB –**

Location	Stress [MPa] S1	Ratio*** S1
S1**	371	0.90

**C. Strain gauge
– Column –**

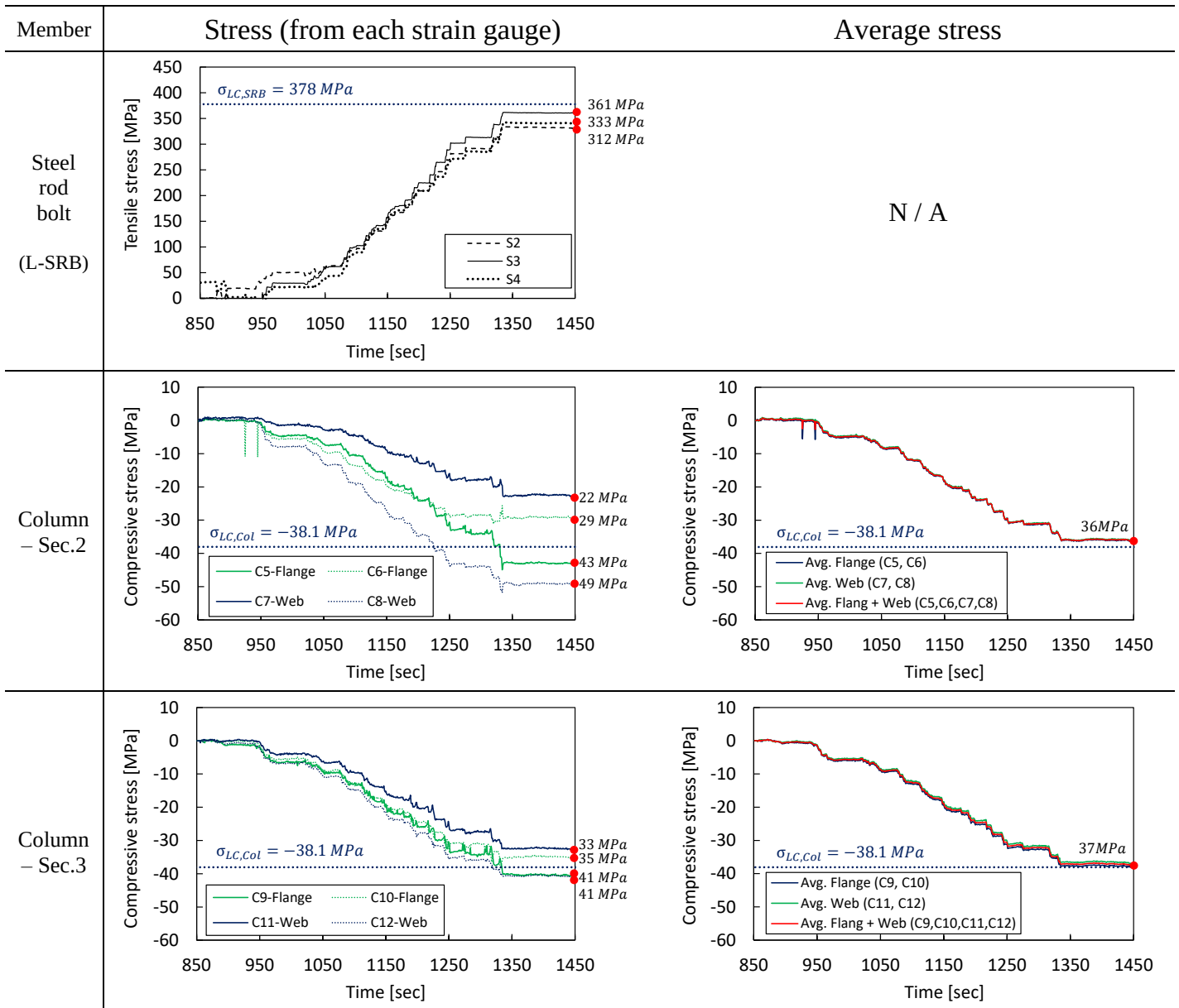
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec.1 (C1, C2, C3, C4)**	-39.1	-39.6	-39.4	0.95	0.96	0.96

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.28: Stress variation measured by strain gauges - 3m upper column and SRB in TW-1

**A. Load cell**

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-2**	185	378	-38.1

**B. Strain gauge
– SRB –**

Location	Stress [MPa]			Ratio***		
	S2	S3	S4	S2	S3	S4
S2, S3, S4**	333	361	312	0.88	0.96	0.83

**C. Strain gauge
– Column –**

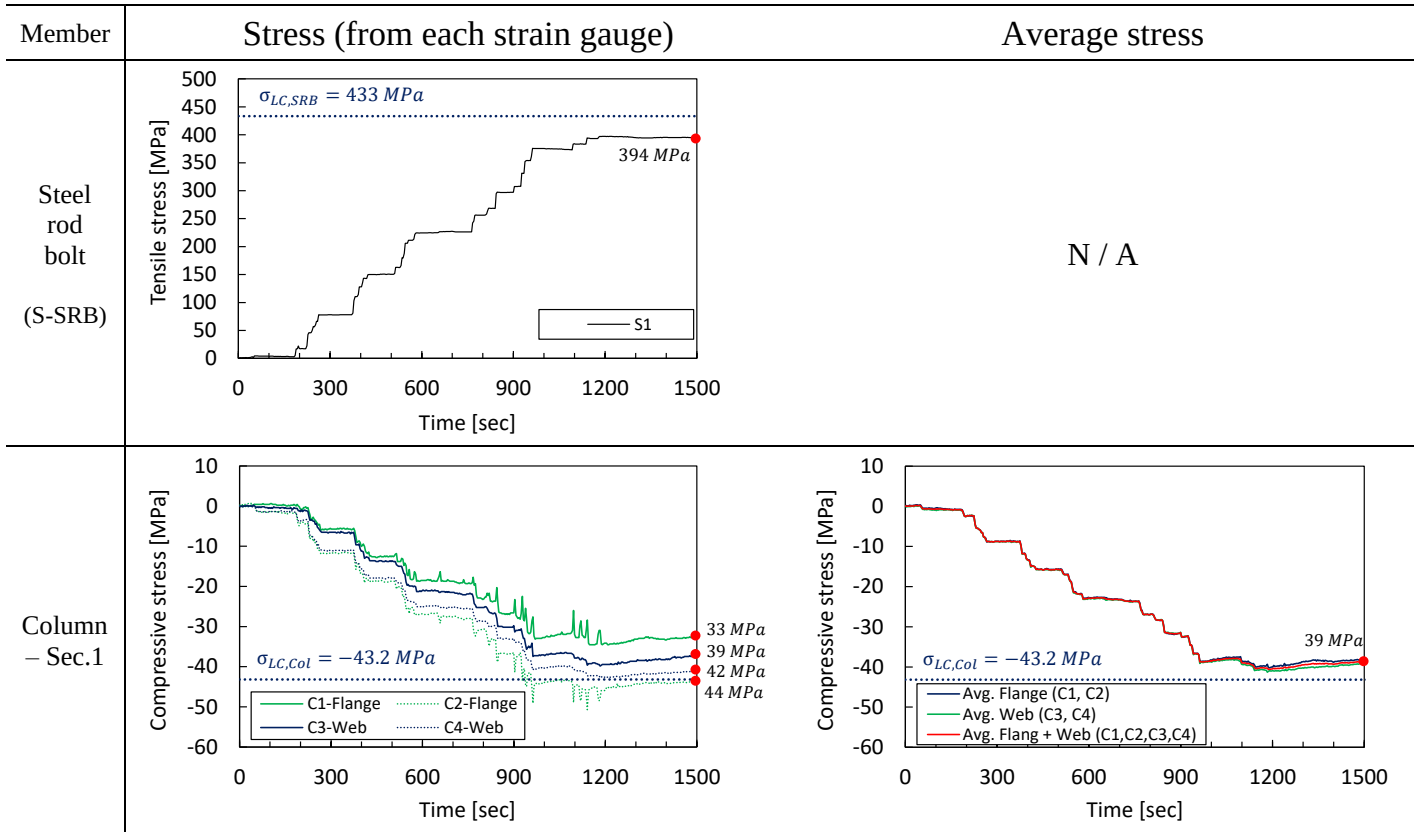
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec2 (C5, C6, C7, C8)**	-36.2	-36.1	-36.1	0.95	0.95	0.95
Column Sec3 (C9, C10, C11, C12)**	-37.6	-36.6	-37.1	0.99	0.96	0.97

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.29: Stress variation measured by strain gauges - 1.5m lower column and SRB in TW-2



A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-1**	210	433	-43.2

**B. Strain gauge
– SRB –**

Location	Stress [MPa] S1	Ratio*** S1
S1**	394	0.91

**C. Strain gauge
– Column –**

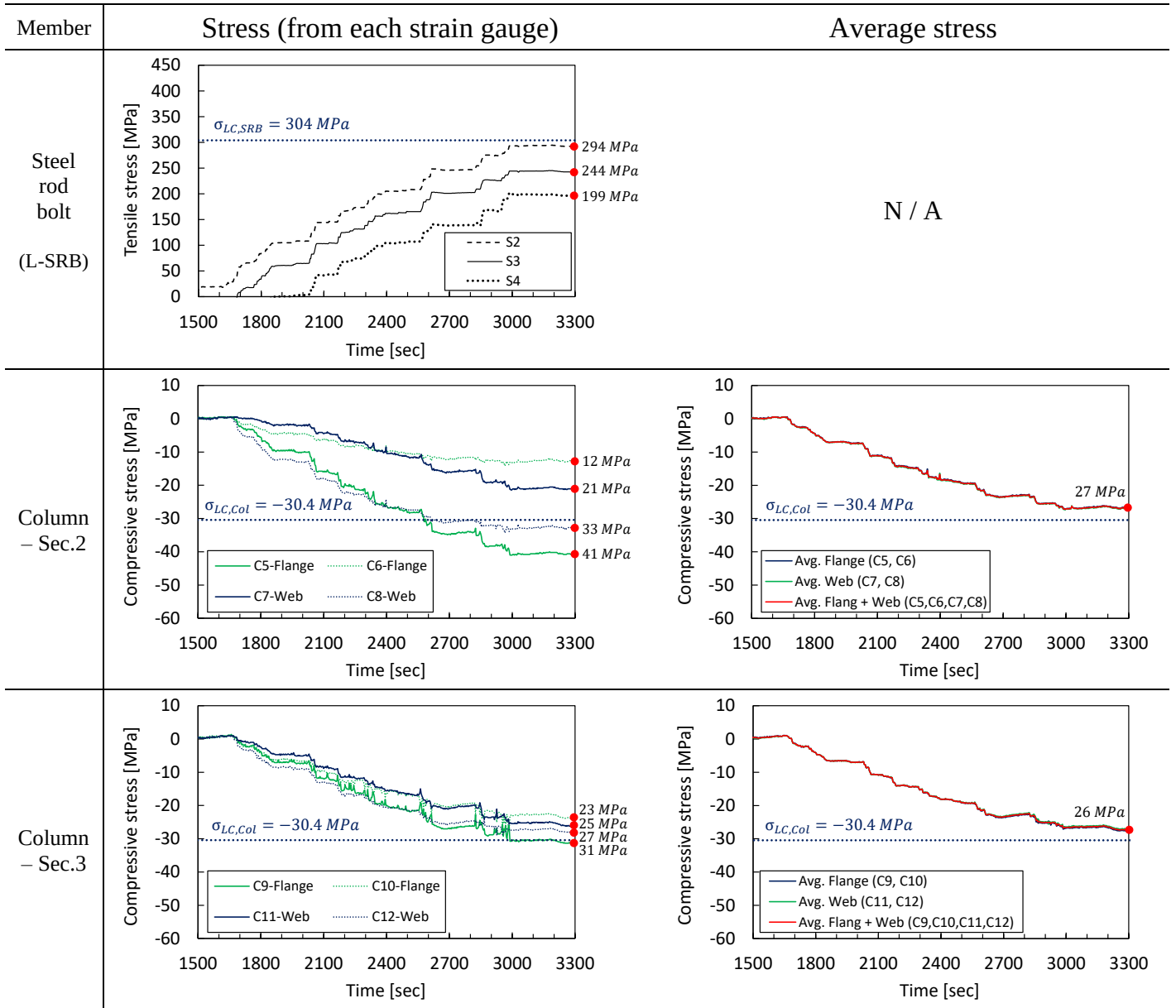
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec.1 (C1, C2, C3, C4)**	-38.5	-40	-39.2	0.89	0.93	0.91

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges was divided by the stress measured by the load cell.

Table 4.30: Stress variation measured by strain gauges - 3m upper column and SRB in TW-2


A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-2**	154	304	-30.4

**B. Strain gauge
– SRB –**

Location	Stress [MPa]			Ratio***		
	S2	S3	S4	S2	S3	S4
S2, S3, S4**	294	244	199	0.97	0.80	0.65

**C. Strain gauge
– Column –**

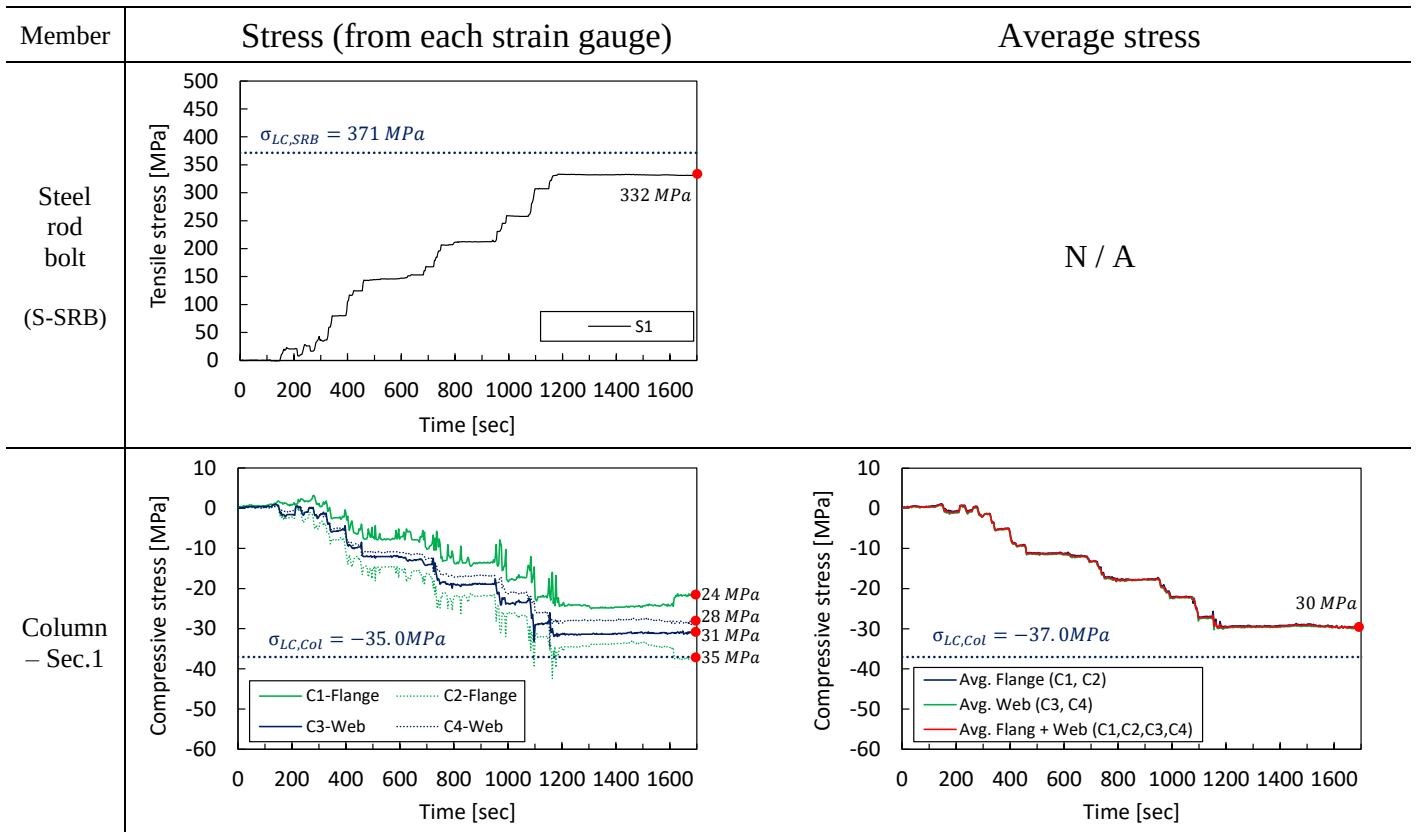
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec2 (C5, C6, C7, C8)**	-26.6	-27.0	-26.8	0.88	0.89	0.88
Column Sec3 (C9, C10, C11, C12)**	-26.6	-26.1	-26.4	0.88	0.86	0.87

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges was divided by the stress measured by the load cell.

Table 4.31: Stress variation measured by strain gauges - 1.5m lower column and SRB in TW-3


A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-1**	180	371	-37

**B. Strain gauge
– SRB –**

Location	Stress [MPa] S1	Ratio*** S1
S1**	332	0.89

**C. Strain gauge
– Column –**

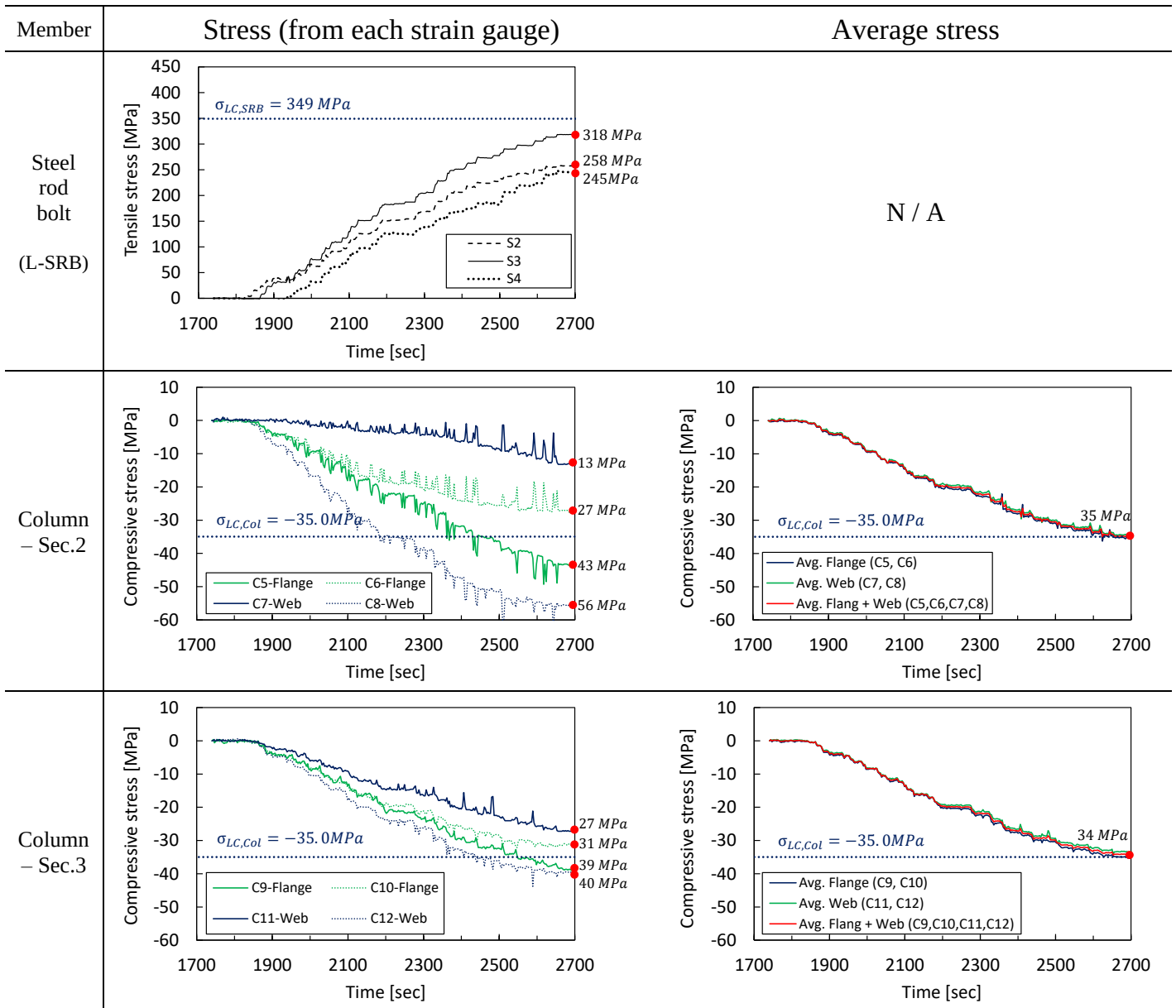
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec.1 (C1, C2, C3, C4)**	-29.5	-29.7	-29.6	0.80	0.80	0.80

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.32: Stress variation measured by strain gauges - 3m upper column and SRB in TW-3


A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-2**	170	349	-35.0

**B. Strain gauge
– SRB –**

Location	Stress [MPa]			Ratio***		
	S2	S3	S4	S2	S3	S4
S2, S3, S4**	258	318	245	0.74	0.91	0.70

**C. Strain gauge
– Column –**

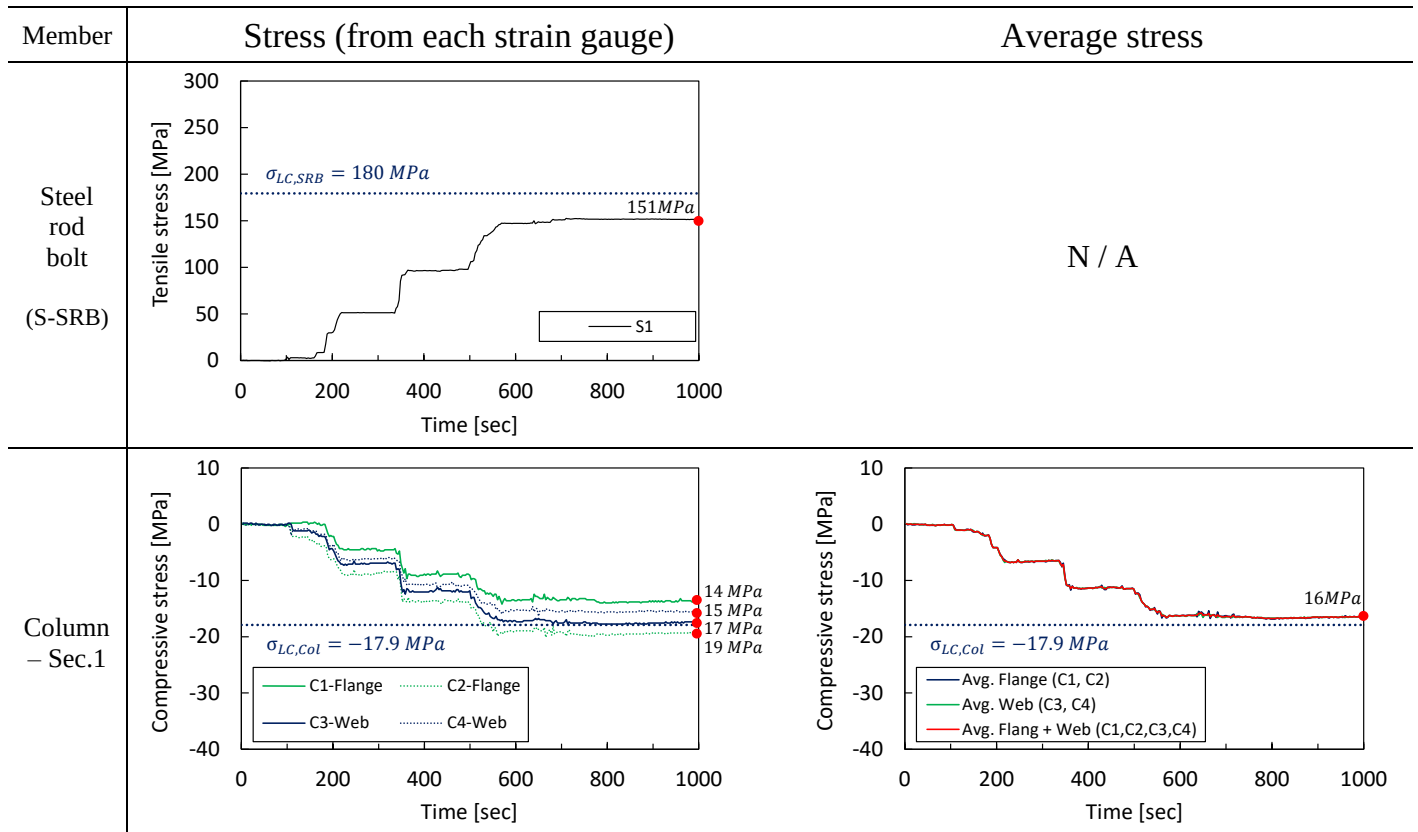
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec2 (C5, C6, C7, C8)**	-35.3	-34.5	-34.9	1.01	0.99	1.00
Column Sec3 (C9, C10, C11, C12)**	-35.1	-33.4	-34.3	1.00	0.95	0.98

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.33: Stress variation measured by strain gauges - 1.5m lower column and SRB in IMW-1


A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-1**	87.0	180	-17.9

**B. Strain gauge
– SRB –**

Location	Stress [MPa] S1	Ratio***
S1**	151	0.84

**C. Strain gauge
– Column –**

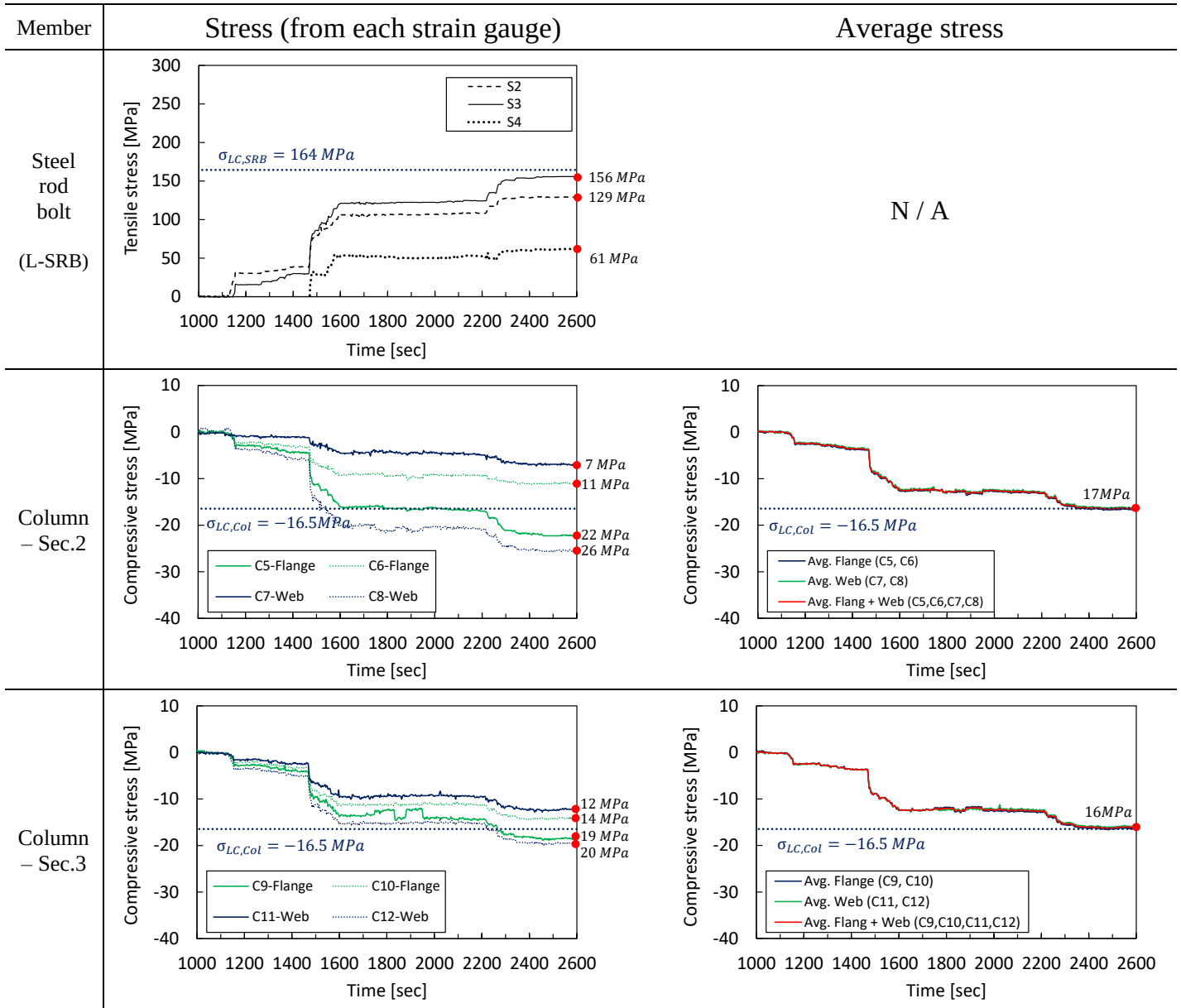
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec.1 (C1, C2, C3, C4)**	-16.4	-16.5	-16.4	0.92	0.92	0.92

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.34: Stress variation measured by strain gauges – 3m upper column and SRB in IMW-1


A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-2**	80	164	-16.5

**B. Strain gauge
– SRB –**

Location	Stress [MPa]			Ratio***		
	S2	S3	S4	S2	S3	S4
S2, S3, S4**	129	156	61	0.78	0.95	0.37

**C. Strain gauge
– Column –**

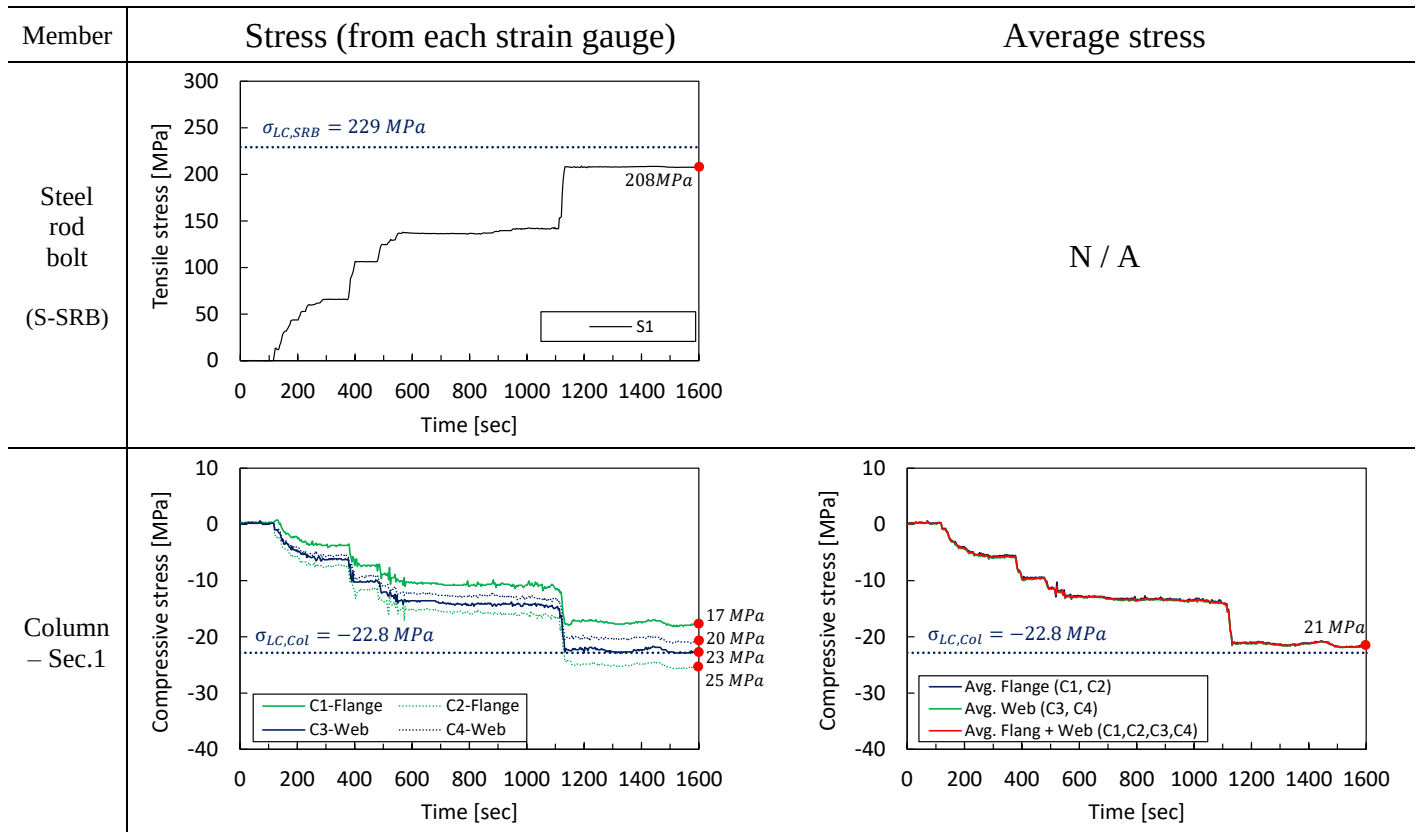
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec2 (C5, C6, C7, C8)**	-16.7	-16.3	-16.5	1.01	0.99	1.00
Column Sec3 (C9, C10, C11, C12)**	-16.6	-16	-16.3	1.01	0.97	0.99

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.35: Stress variation measured by strain gauges - 1.5m lower column and SRB in IMW-2



A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,Col}^*$
Load cell-1**	111	229	-22.8

B. Strain gauge – SRB –

Location	Stress [MPa] S1	Ratio*** S1
S1**	208	0.91

C. Strain gauge – Column –

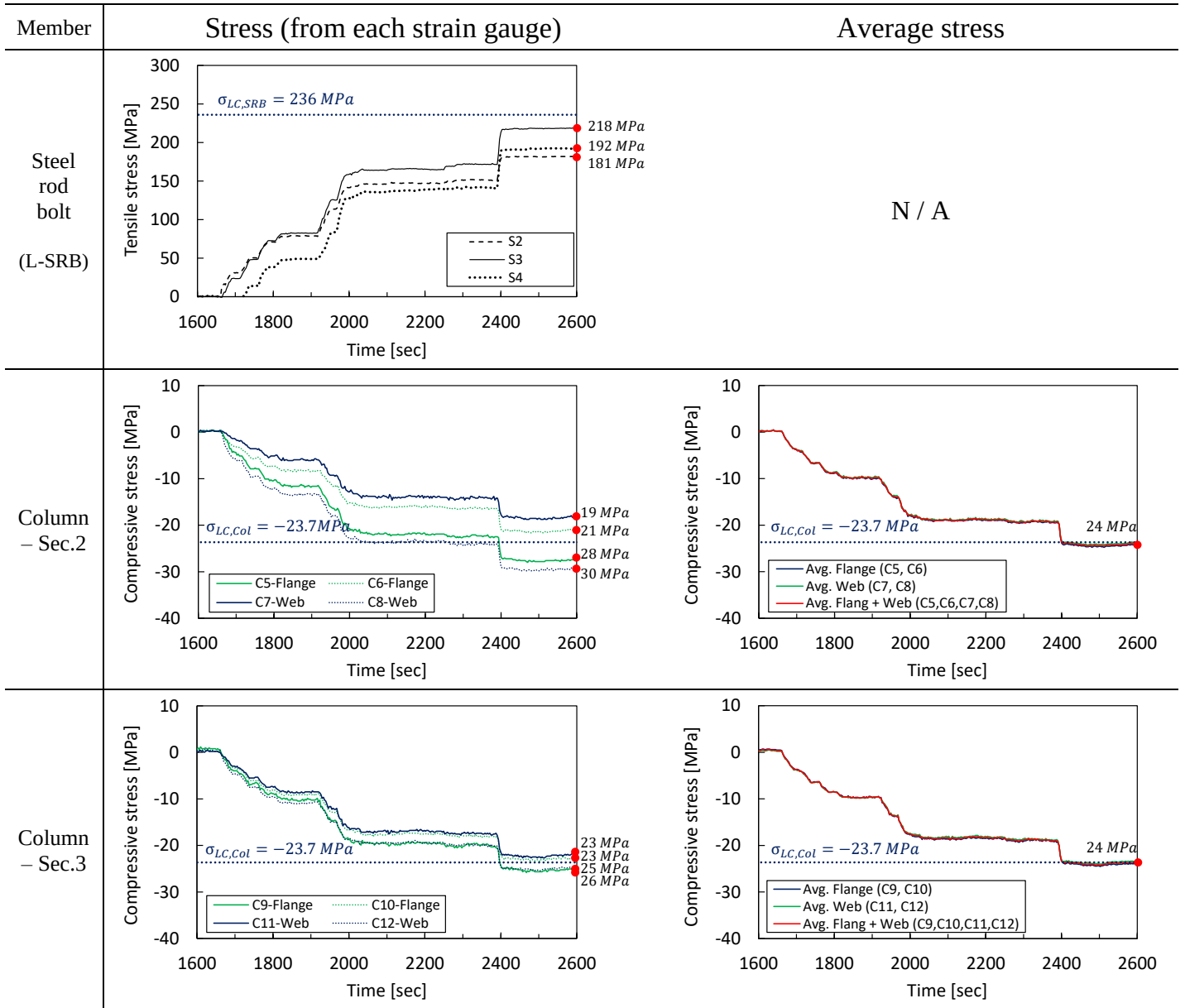
Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec.1 (C1, C2, C3, C4)**	-21.2	-21.3	-21.2	0.93	0.93	0.93

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

Table 4.36: Stress variation measured by strain gauges – 3m upper column and SRB in IMW-2



A. Load cell

Location	Preload [kN]	SRB [MPa] $\sigma_{LC,SRB}^*$	Column [MPa] $\sigma_{LC,col}^*$
Load cell-2**	115	236	-23.7

**B. Strain gauge
– SRB –**

Location	Stress [MPa]			Ratio***		
	S2	S3	S4	S2	S3	S4
S2, S3, S4**	181	218	192	0.77	0.92	0.81

**C. Strain gauge
– Column –**

Location	Average Stress [MPa]			Ratio***		
	Flange	Web	All	Flange	Web	All
Column Sec2 (C5, C6, C7, C8)**	-24.5	-24.2	-24.4	1.03	1.02	1.03
Column Sec3 (C9, C10, C11, C12)**	-24.4	-24	-24.2	1.03	1.01	1.02

* The stress was obtained by dividing the preload by the respective nominal cross-sectional area of the SRB and column shown in Table 4.20.

** The locations of strain gauges are shown in Figure 4.24.

*** The stress measured by the strain gauges divided by the stress measured by the load cell.

4.3.8. Discussion and Further Research

The torque wrench test confirmed that applying preload over 150kN (max 185kN) to the 3.08m-SRB was feasible. Also, the (non-lubrication) torque coefficients K for the 3.08m-SRB and the 1.58m-SRB could be determined, which were 0.53 and 0.39, respectively. These values were approximately 2.7 and 2.0 times greater than the theoretical torque coefficient for the general M24 high-tensile bolt (same bolt size as the SRBs). The torque wrench test results showed that if the length of the bolt increased, the demand torque also increased for obtaining the same amount of the preload.

Based on the torque coefficient K from the torque wrench tests, the capacity of the cordless impact wrench with the maximum fastening torque capacity of 2023 Nm was selected. According to (Eq. 4.1), the corresponding preload with $K=0.53$ became 184kN. However, test results showed that the amount of preload induced by the impact wrench was 80 kN and 115kN, which were much lower than expected. Compared to the torque wrench test results, the preload applied was approximately 20-35% lower than the average preload from the torque wrench test at the same amount of the torque. In the case of the performance of the impact wrench, the fastening torque in the impact wrench is affected by a wide variety of factors such as the bolt diameter, bolt length, impact socket size, etc. Thus, the cordless impact wrench might not deliver the maximum fastening torque value provided by the manufacturer during the tests.

The torque coefficient obtained from the test is affected by numerous factors such as the friction coefficient of the surfaces, tightening speed, tightening sequence, the type of tools used, lubrication, coating, washers and plastic deformation of threads, etc. [102, 109] Since the influence of these many factors is expressed as merely the torque coefficient one constant K , it is generally necessary to test a large number of specimens in order to determine the mean and standard deviation of the appropriate torque coefficient.

Consequently, further tests of a larger number of specimens would be required to determine more accurate torque coefficients for the 3m-long SRB and to evaluate whether the corresponding preload to the torque coefficient can be obtained stably using the impact wrench.

Regarding the impact on the preload of the lower column when the upper column is disassembled from the lower column, both torque wrench tests and impact wrench tests confirmed that there was little impact on the preload in the lower column.

4.4. Preloaded Beam-to-column Joint Test (AJ Joint)

The main objective of the beam-to-column joint test was to evaluate the seismic performance of the AJ connection proposed in this study for steel-framed modular buildings. The details of the test assemblies were selected from the exterior frame between the 1st floor and 2nd floor in the 8-storey prototype building (refer to Section 6.4.1), as shown in Figure 4.35. Assuming that the inflection points of columns and beams occur at the midpoint of the members under earthquake loading, as presented in Figure 4.35 (b), the lengths of the beams and columns for the test assemblies were determined as the half-length of a module. Generally, there are two types of test setup for an exterior beam-to-column joint test, as shown in Figure 4.35 (c). In Type A, the lateral force is applied at the top of the column, and hinges are installed at the bottom of the column and at the end of the beams. In Type B, the lateral force is applied at the tip of the beams, and hinges are installed at the ends of the columns. In this research, Type B was selected considering the laboratory conditions available to conduct the tests for the proposed AJ connection. It was also relatively easier to assemble the test specimens by applying preload to columns placed horizontally. The details of the test setup are shown in Section. 4.4.2.

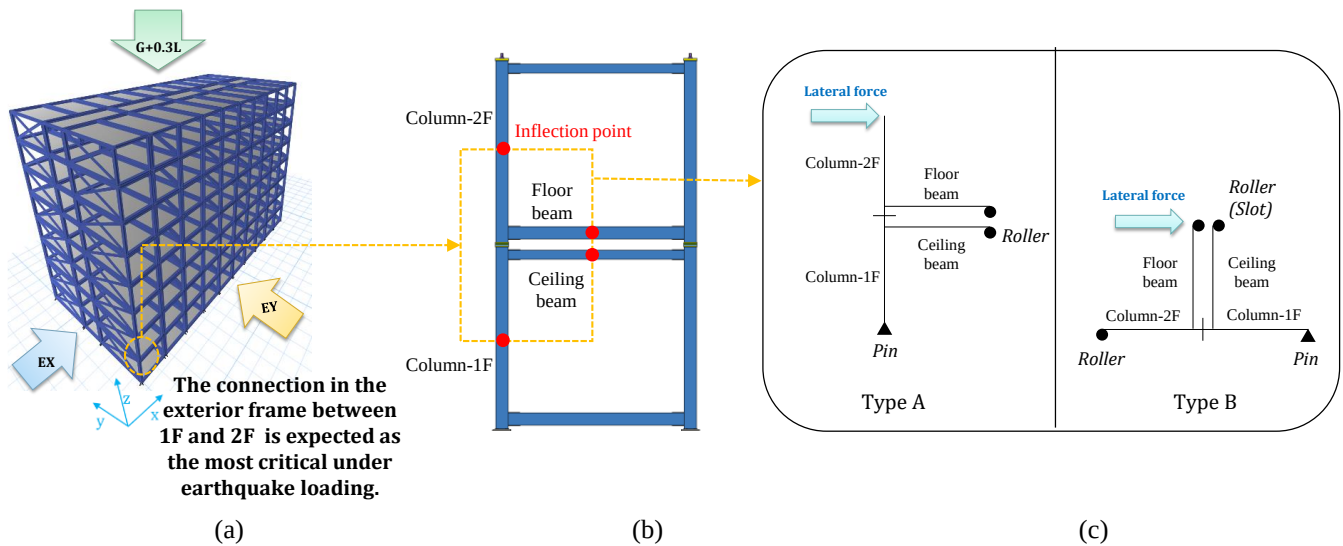


Figure 4.35: Conception of the test setups: (a) Prototype 8-storey building; (b) Moment-resisting frame in the short span direction of the modules; and (c) Types of the test setup for an exterior beam-to-column joint

4.4.1. Description of Test Specimens

The beam-to-column joint with the AJ connection proposed in this research is designed based on the strong column-weak beam concept, using the seismic design capacity principles [86]. Accordingly, the floor and ceiling beams were designed as dissipative members with capacity to sustain plastic deformation for energy dissipation, while the columns and panel zones were designed as non-dissipative members to remain in the elastic range until the beams fail. Since the preload column-to-column connection is located in a non-dissipative zone, the coupler and the SRB are designed as non-dissipative members. The demand preload to prevent gap-opening at the connection is calculated using the same concept, treating the connection as a non-dissipative member. Consequently, the failure of the joint was intended to occur in the beams for all specimens. The floor beam and ceiling beam were welded to the columns. In order to avoid brittle failure from welding between the beam and the column and to ensure plastic hinges would develop in the beams, reinforcing plates were welded to the beams near the columns for all specimens.

Five full-scale exterior beam-to-column joint specimens were tested under monotonic loading and quasi-static cyclic loading. Table 4.37 presents a summary of the test specimens. The P-DB and P-SB specimens are AJ connections. They are identical specimens, except for the ceiling beam size. In the case of P-DB, the floor beam has a larger cross-section than the ceiling beam, whereas P-SB has the same size floor and ceiling beams.

Generally, in modular building structures, the floor beam is larger than the ceiling beam, like P-DB. However, according to the analytical approach described in Section 3.5.2 for calculating the relationship between demand preload and gap-opening at the connection, it was found that the demand preload to prevent the gap-opening can be reduced if the difference in bending stiffness (EI) between the floor and ceiling beams is small. So, the P-SB test was included to investigate the effect on the gap-opening of the difference in bending stiffness between the two beams. The magnitude of the applied preload required to prevent the gap-opening was calculated based on the analysis described in Section 4.4.7. The P-DB and P-SB specimens were tested under monotonic loading and quasi-static cyclic loading. The details of the specimens are presented in Figure 4.36 - Figure 4.39. The W-SB-Q specimen is the column-to-column welded connection. It was tested under quasi-static cyclic loading as the reference model to which the seismic performance of the AJ connection can be compared.

The monotonic tests were performed to verify the design method for calculating the demand preload for preventing gap-opening and to study the actual lateral load resisting mechanism with or without gap-opening, the effect of the difference in the depth of the floor and ceiling beams on the gap-opening, the practical load-bearing capacity, the rotation capacity of the joint, and the failure mode. The cyclic loading test was performed to investigate the seismic performance and failure mode of the AJ

connection.

To ensure the reliable behaviour of the joint without gap-opening at the connection, the applied preload was increased by 10% from the demand preload obtained by the analytical approach (see Table 4.41). For the P-DB-M/Q tests, the target preload was 207kN, but a preload greater than 155kN/185kN could not be applied with an impact wrench while assembling the P-DB-M and P-DB-Q specimens.

Table 4.37: Summary of beam-to-column joint test specimens (the bold marks indicate varying parameters)

Specimen	Connection type	Floor beam [mm]	Ceiling beam [mm]	Column [mm]	SRB [mm]	Preload [kN]	Loading	Intended failure mode
P-DB-M	Preload	□200 x 100 x 4	□150 x 100 x 4	□200 x 100 x 12	Φ24.95	155	Monotonic	Beam
P-SB-M	Preload	□200 x 100 x 4	□ 200 x 100 x 4	□200 x 100 x 12	Φ24.95	145	Monotonic	Beam
P-DB-Q	Preload	□200 x 100 x 4	□150 x 100 x 4	□200 x 100 x 12	Φ24.95	185	Cyclic	Beam
P-SB-Q	Preload	□200 x 100 x 4	□ 200 x 100 x 4	□200 x 100 x 12	Φ24.95	145	Cyclic	Beam
W-SB-Q	Welded	□200 x 100 x 4	□ 200 x 100 x 4	□200 x 100 x 12	N/A	N/A	Cyclic	Beam

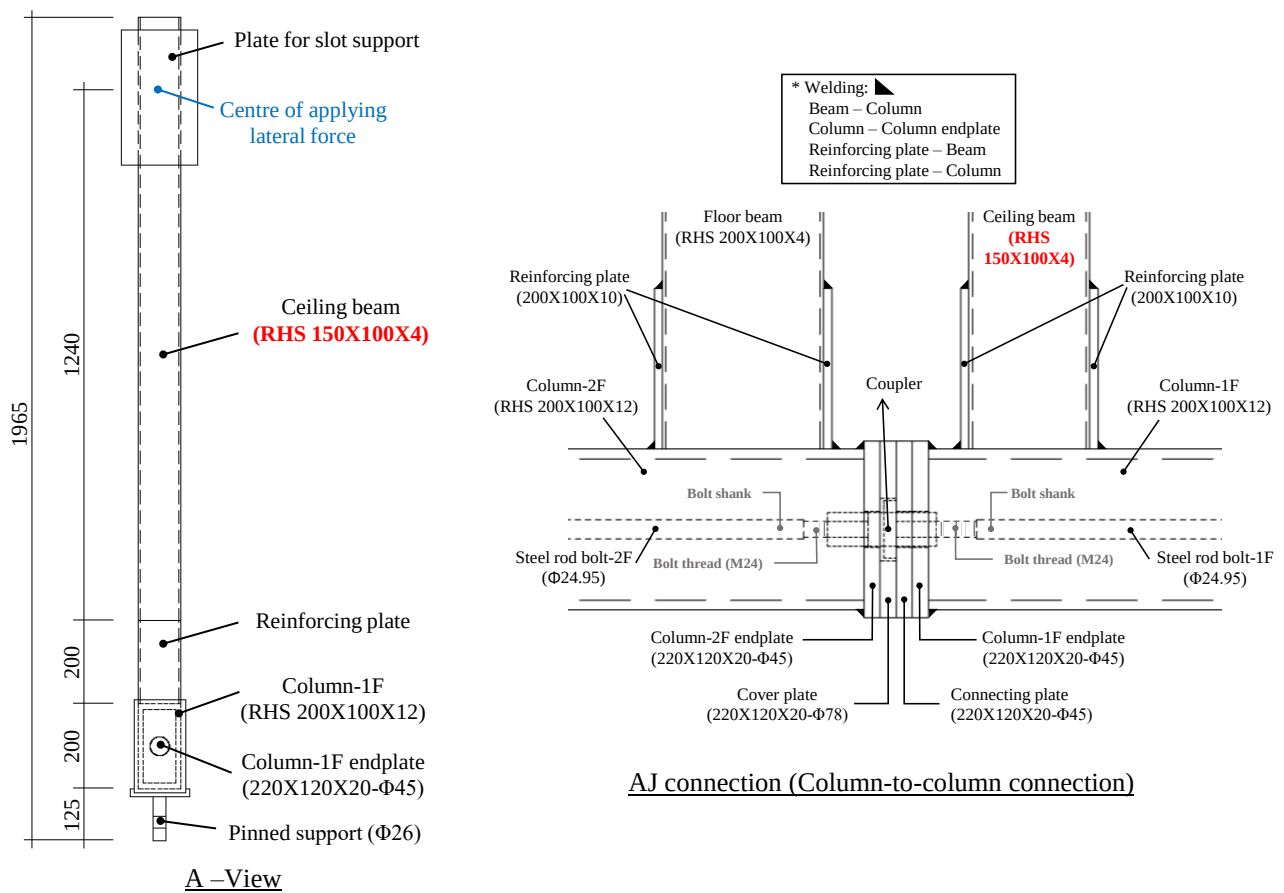
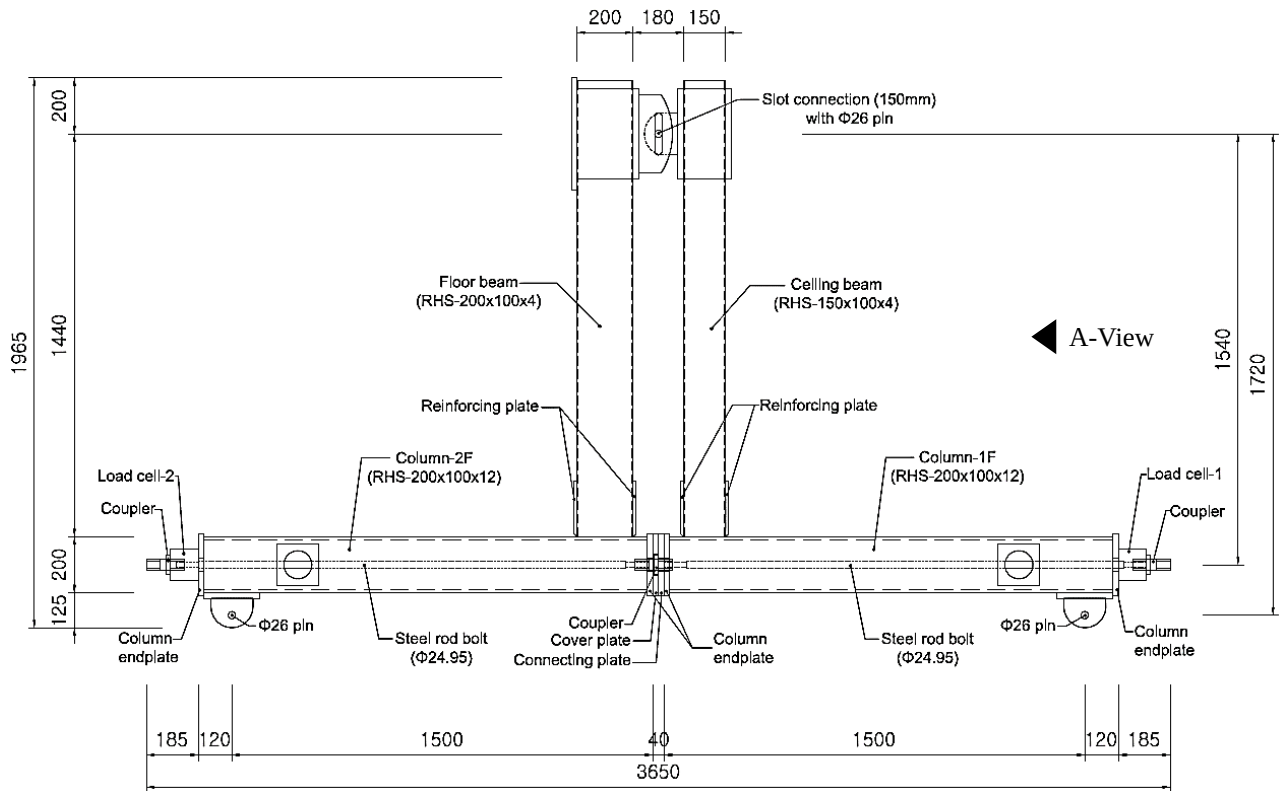


Figure 4.36: Details of the test specimen: P-DB-M and P-DB-Q (units: mm)

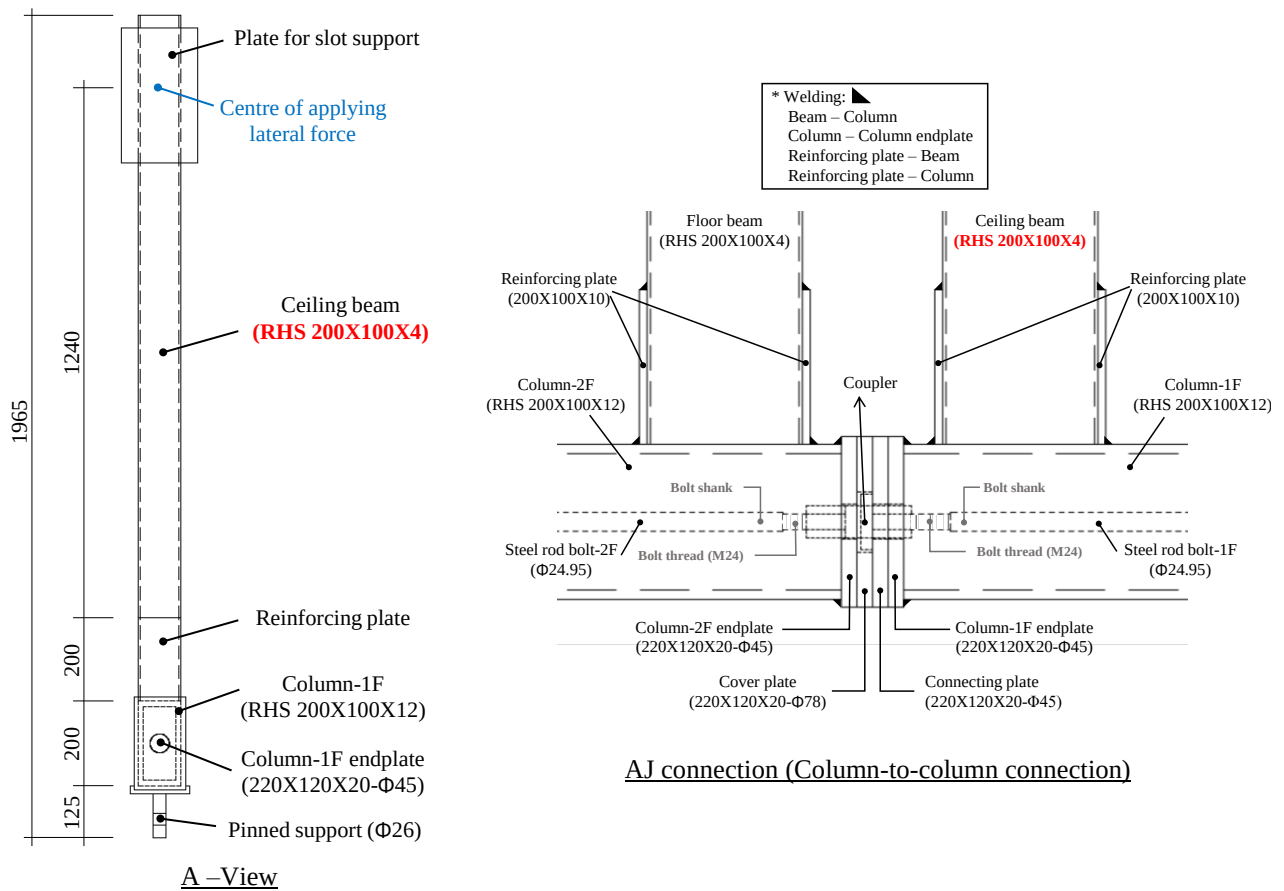
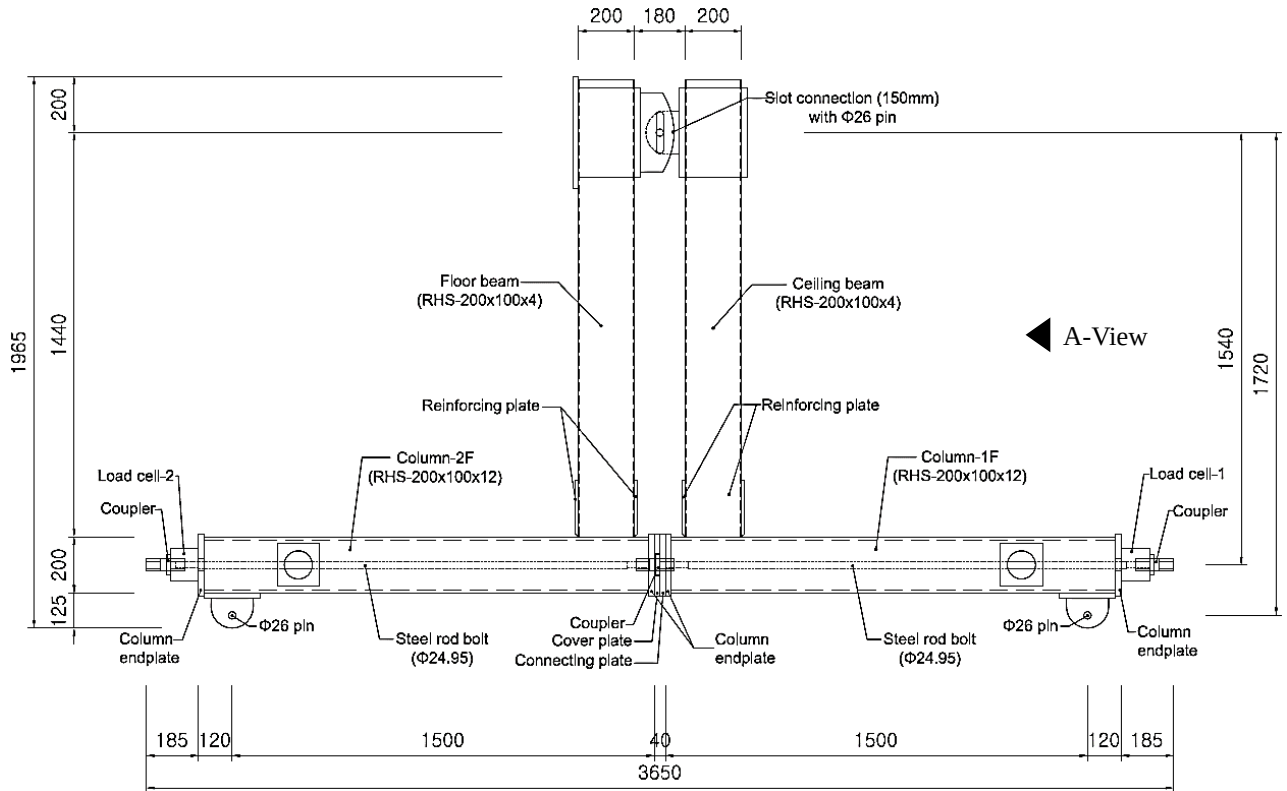


Figure 4.37: Details of the test specimen: P-SB-M and P-SB-Q (units: mm)

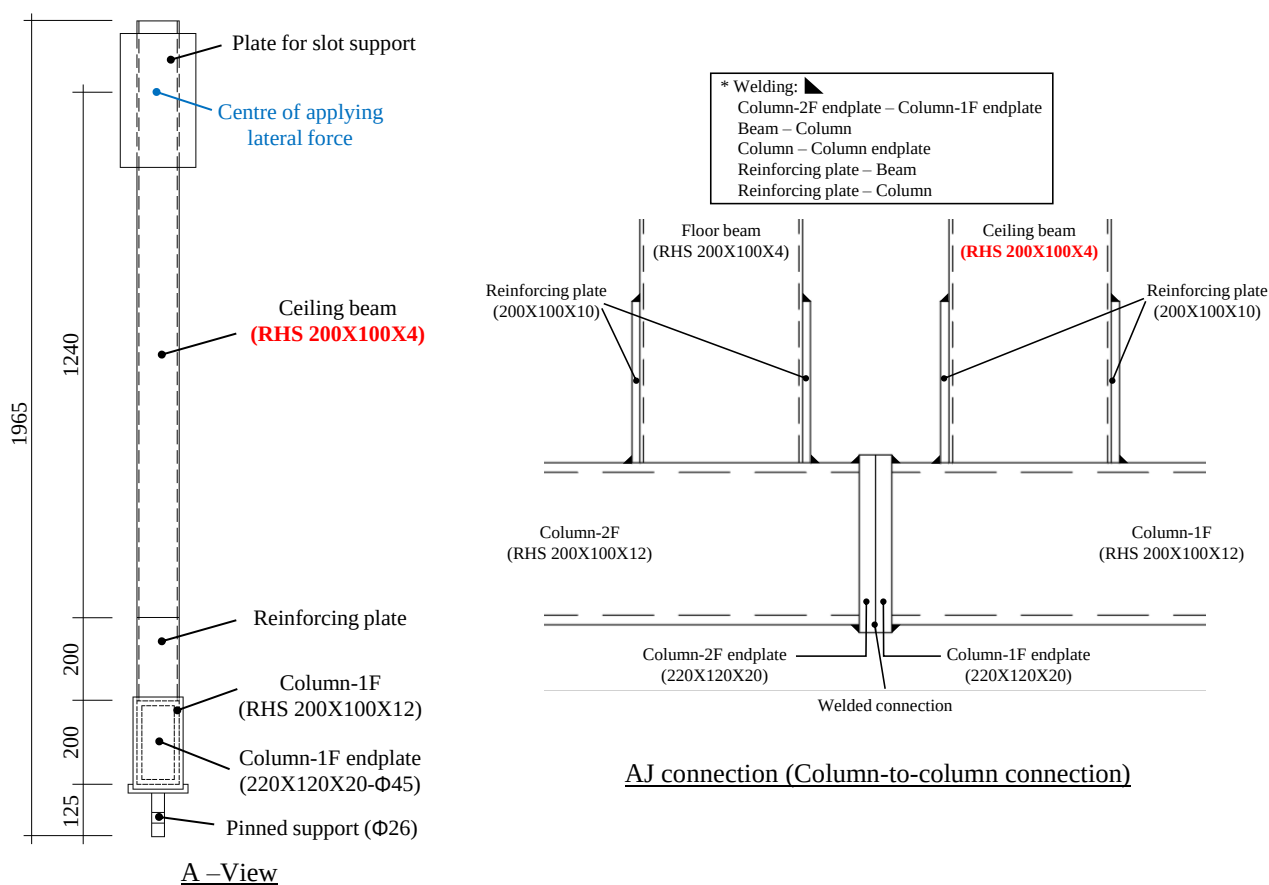
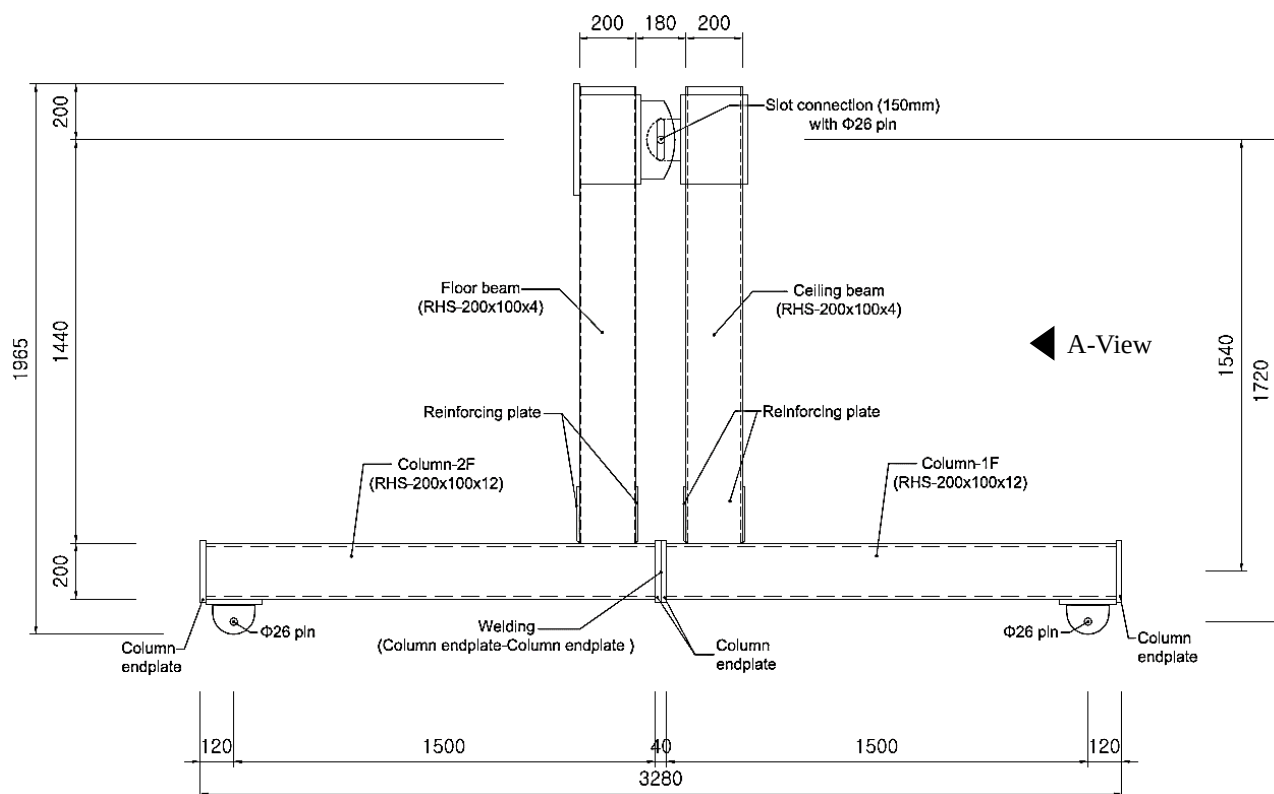
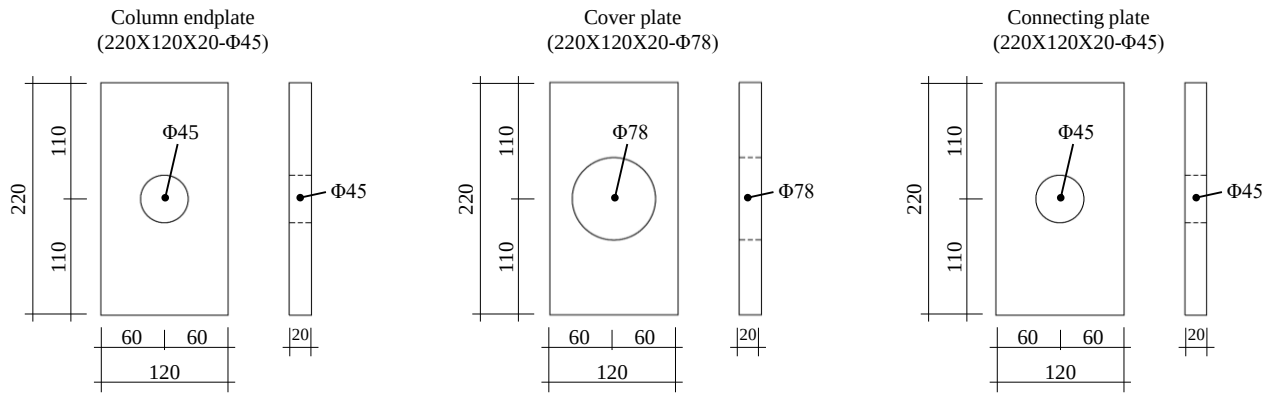
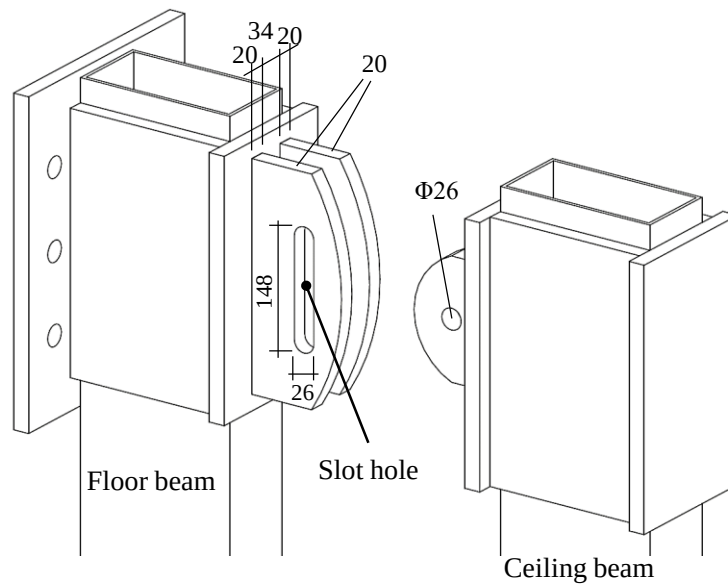
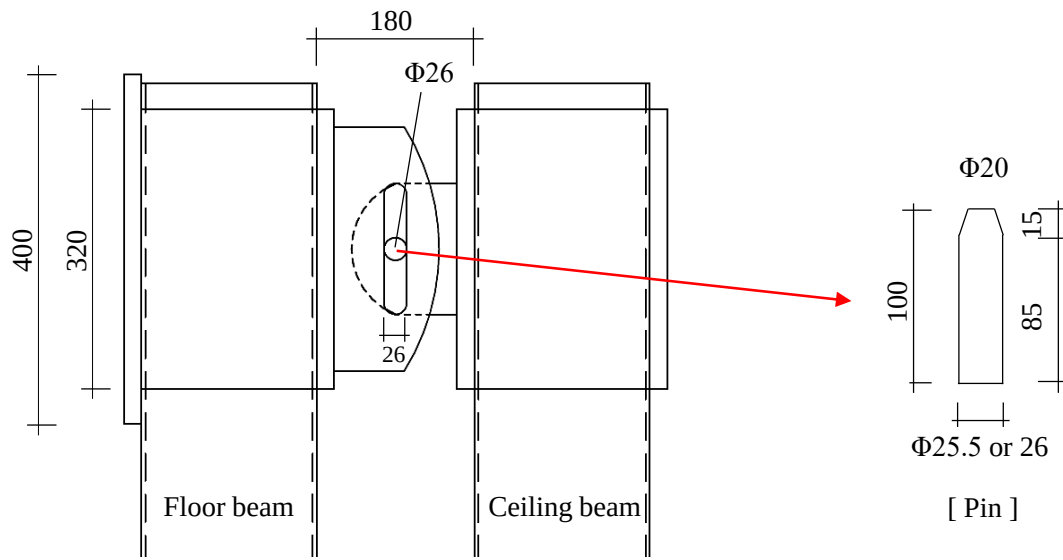


Figure 4.38: Details of the test specimen: W-SB-Q (units: mm)



(a) Plates in AJ connection



(b) Slot connection for the floor beam and the ceiling beam

Figure 4.39: Details of the test specimens: Plates in AJ connection and the slot connection for the beams (units: mm)

In order to assess the characteristics of all parts of the beam-to-column test assemblies, material tests were performed. Three tensile coupon specimens were prepared for each component. The details of the test results are provided in Section 4.2. Table 4.38 presents a summary of the average measured material properties in terms of: the Young's modulus (E), the nominal yield stress (f_n), the measured yield stress (f_y), the measured ultimate tensile strength (f_u), the ratio between measured yield strength and nominal strength (f_y/f_n), the ratio between measured ultimate tensile strength and measured yield strength (f_u/f_y), the elongation at fracture (Elo), and the uniform elongation at the ultimate tensile strength (ϵ_u).

Table 4.38: Summary of the average measured material properties

Component	Steel Grade	Section size	E [GPa]	f_n [MPa]	f_y [MPa]	f_u [MPa]	f_y/f_n	f_u/f_y	Elo . [%]	ϵ_u [%]
Column	SRT355	□200 x 100 x 12	208	355	496	543	1.40	1.10	28.4	6.1
Ceiling beam	SRT275	□150 x 100 x 4	203	275	324	450	1.18	1.39	33.3	16.4
Floor beam	SRT275	□200 x 100 x 4	214	275	326	468	1.19	1.43	32.2	15.3
Column endplate / Connecting plate / Cover plate	SS275	220X120X20	188	275	327	460	1.23	1.41	26.9	14.4
Reinforcing plate	SS275	200X100X10	190	275	298	469	1.09	1.57	27.6	16.8
Steel rod bolt	ASTM A193 Grade B7	Φ24.95	195	725	863	950	1.19	1.10	19.8	7.1
Coupler*	SM45C (=F10T Bolt)	See Figure 4.16	210*	900*	-	1000*	-	-	-	-

* The coupon test was not performed, so the material properties are its nominal value.

The section properties of the beam and column members in the test assemblies were determined based on the measured material properties in accordance with EN 1993-1-1 [88], as shown in Table 4.39. The column is a Class 1 cross-section for compression and major axis bending. The ceiling and floor beams are both classified as Class 1 cross-sections for major axis bending, i.e. they can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance [88]. A Class 1 cross-section is a compact section according to the AISC-360 Specification for structural steel buildings [110]. Therefore, the bending design resistances ($M_{c,Rd}$) of the ceiling and floor beams are calculated as below:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} \quad (\text{for Class 1 or 2 cross sections}) \quad (\text{Eq. 4.2})$$

where:

- $M_{c,Rd}$ Design resistance for bending;
- $M_{pl,Rd}$ Plastic moment capacity;
- W_{pl} Plastic section bending modulus;
- f_y Yield strength of the material;
- γ_{M0} Partial factor for resistance of cross-sections.

Table 4.39: Section properties of beam and column members for major axis y-y bending

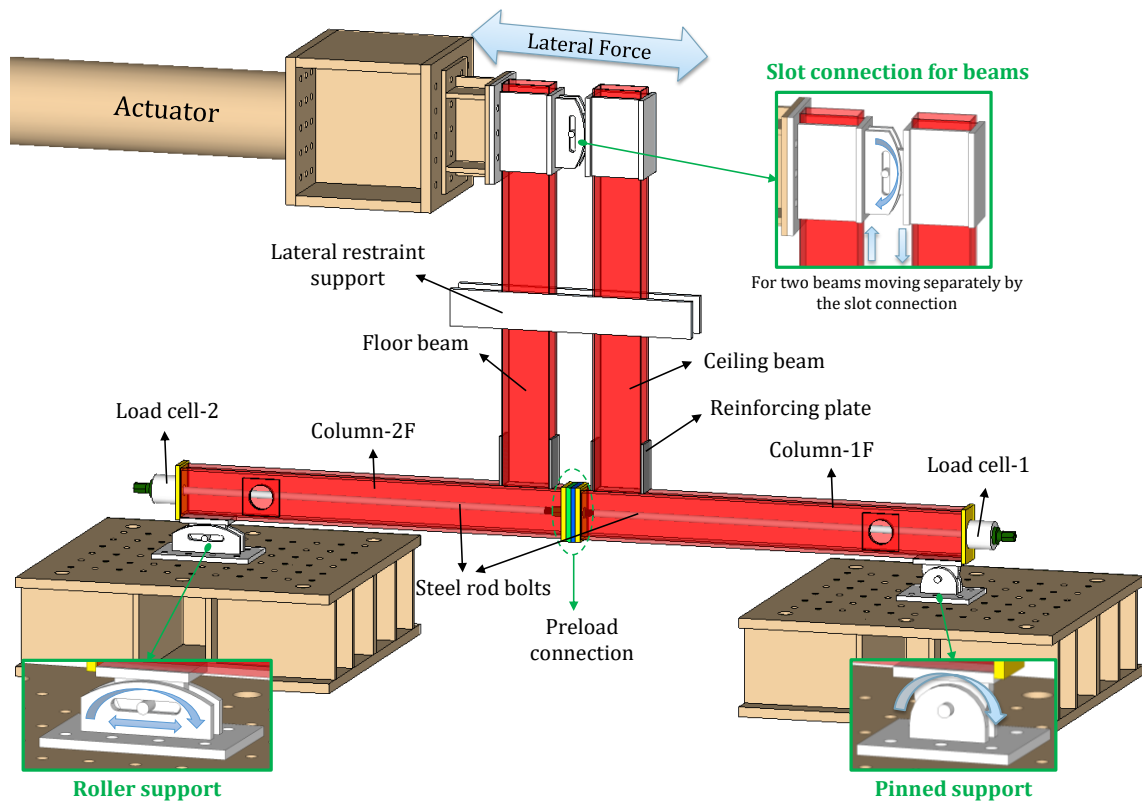
Type	Cross section [mm]	t [mm]	Cross-section classification	Section properties			Elastic moment		Plastic moment	
				I_y [cm ⁴]	$W_{el,y}$ [cm ³]	$W_{pl,y}$ [cm ³]	$M_{el,n}$ [kNm]	M_{el} [kNm]	$M_{pl,n}$ [kNm]	M_{pl} [kNm]
Ceiling beam	□ 150 x 100 x 4	3.93	Class 1	598	79.7	95.8	22.3	25.8	26.8	31.1
Floor beam	□ 200 x 100 x 4	3.91	Class 1	1198	119.7	147.2	33.6	39.0	41.4	48.0
Column	□ 200 x 100 x 12	11.9	Class 1	3029	302.9	392.6	108.2	150.3	140.3	194.8

where:

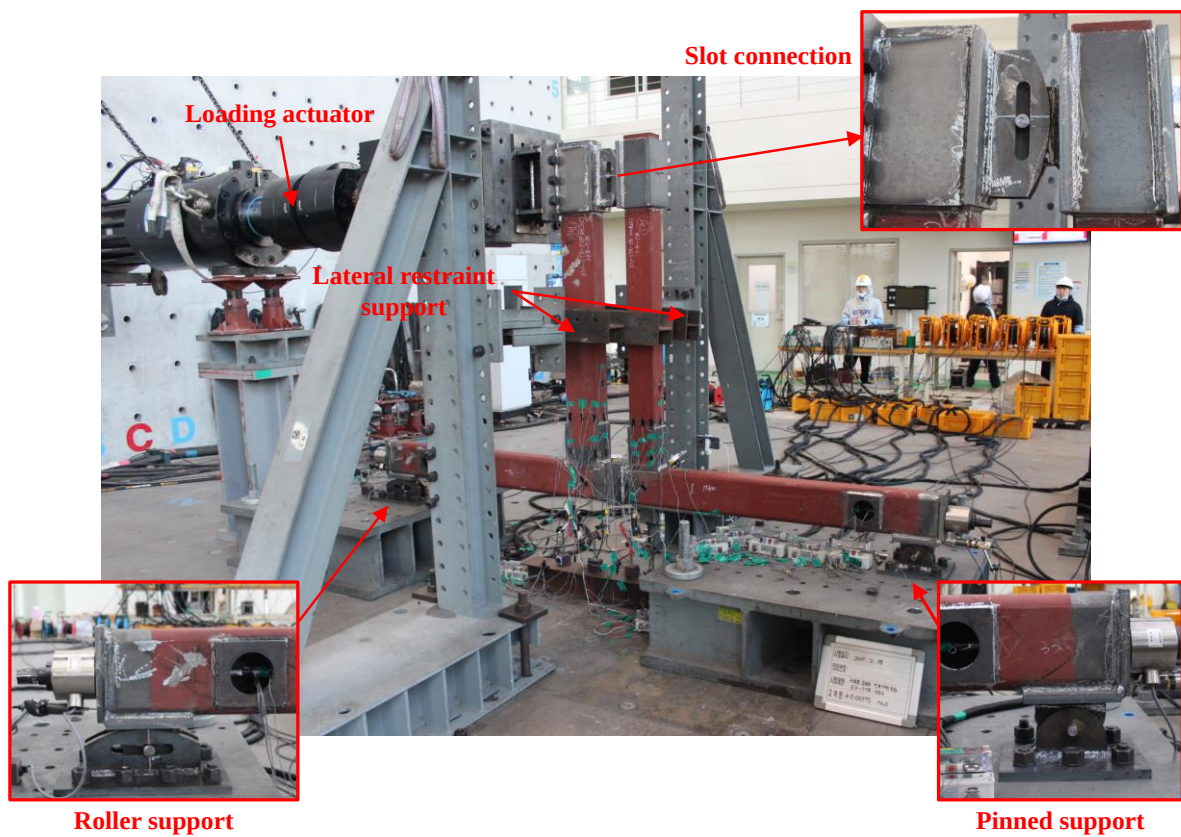
- t Measured thickness of the rectangular hollow section;
- I_y Second moment of area about the major axis;
- $W_{el,y}$ Elastic section modulus;
- $W_{pl,y}$ Plastic section bending modulus;
- $M_{el,n}$ Nominal elastic bending moment resistance;
- $M_{pl,n}$ Nominal plastic bending moment resistance;
- M_{el} Elastic bending moment resistance with the measured thickness and yield stress (refer to Table 4.38);
- M_{pl} Plastic bending moment resistance with the measured thickness and yield stress (refer to Table 4.38).

4.4.2. Test Setup

The test setup is illustrated in Figure 4.40 (AJ joint) and Figure 4.41 (welded joint). Two hinged supports were installed at the ends of the columns to simulate the assumed mid-length inflection points. In order to make the floor and ceiling beams behave separately, a slot connection was applied at the tips of the two beams. Lateral supports were installed to restrain the out-of-plane movement of the specimen during the test.



(a)



(b)

Figure 4.40: Experimental test setup for P-DB and P-SB: (a) Conceptual scheme; (b) Test assembly

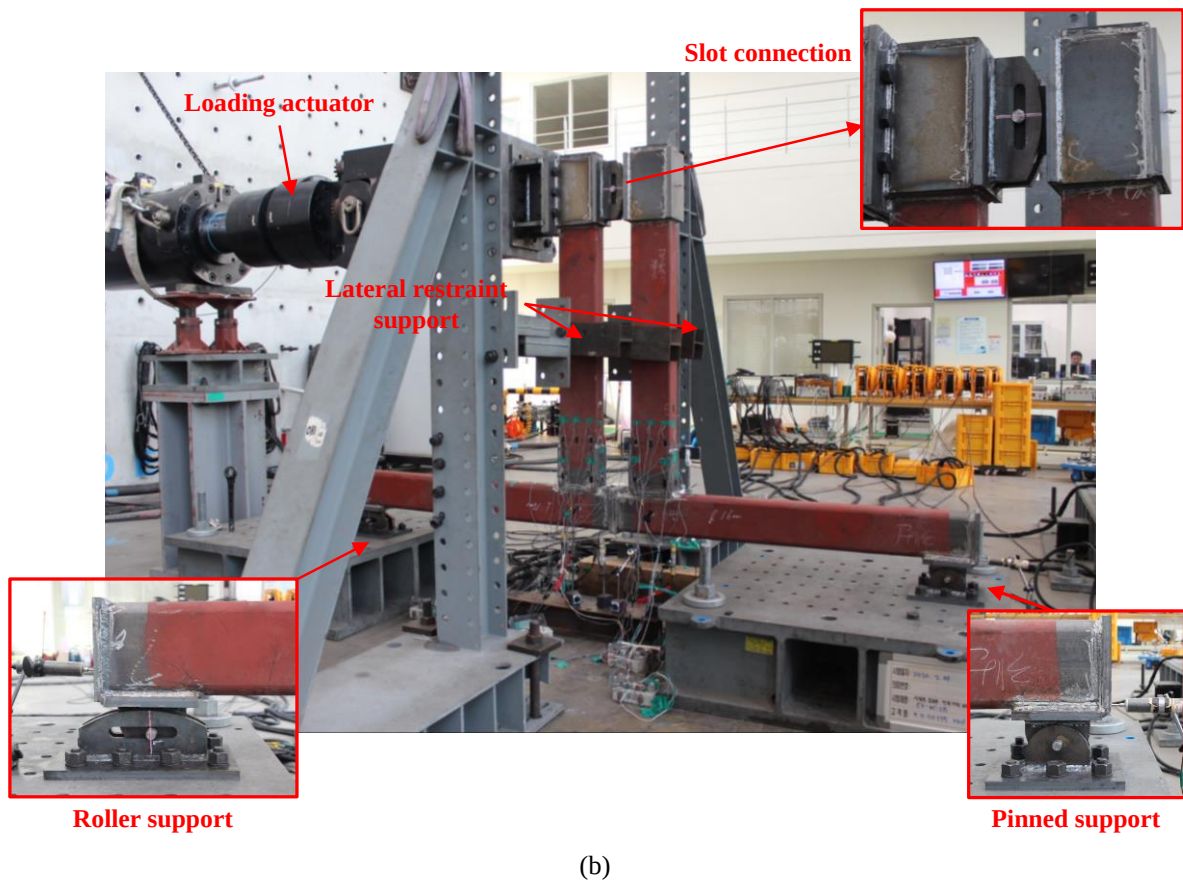
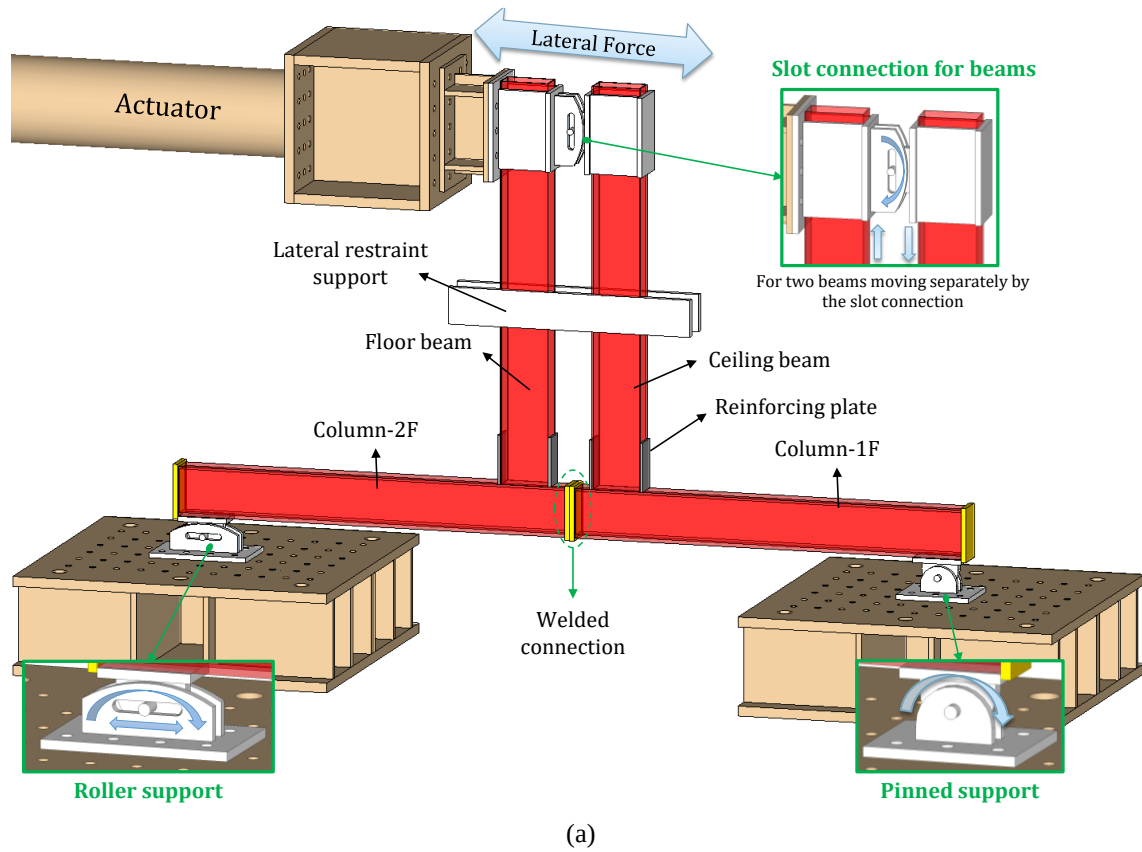
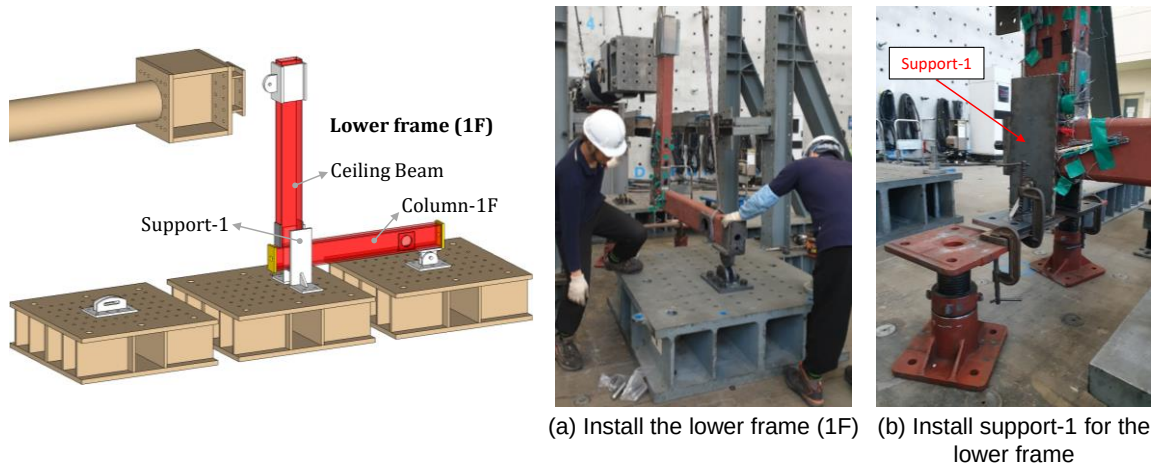


Figure 4.41: Experimental test setup for W-SB: (a) Conceptual scheme; (b) Test assembly

4.4.3. Test Procedure

The test procedure for the beam-to-column joint with AJ connection is presented in Figure 4.42 and Figure 4.43.

STEP 1: Installation of the lower frame (1F)



STEP 2 – Applying preload to column-1F

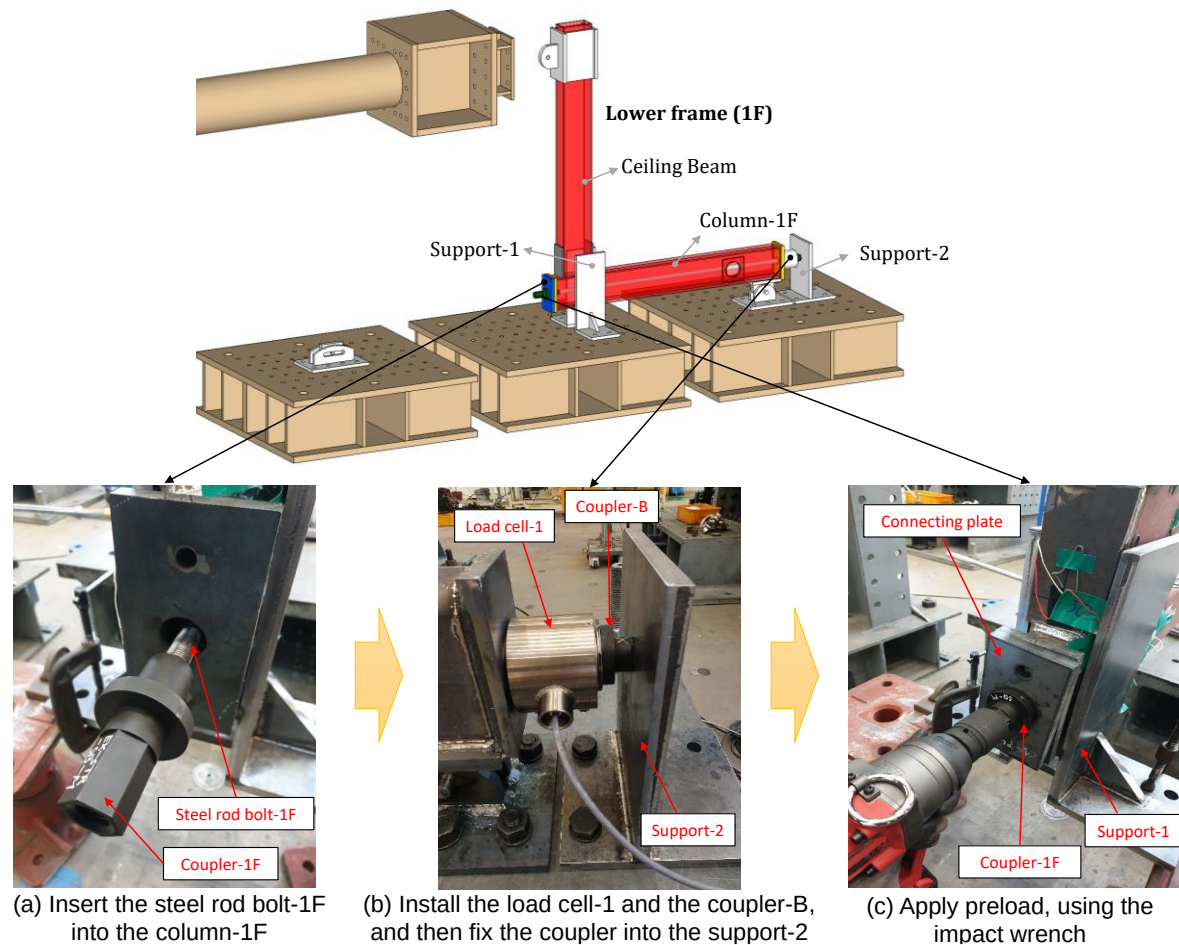
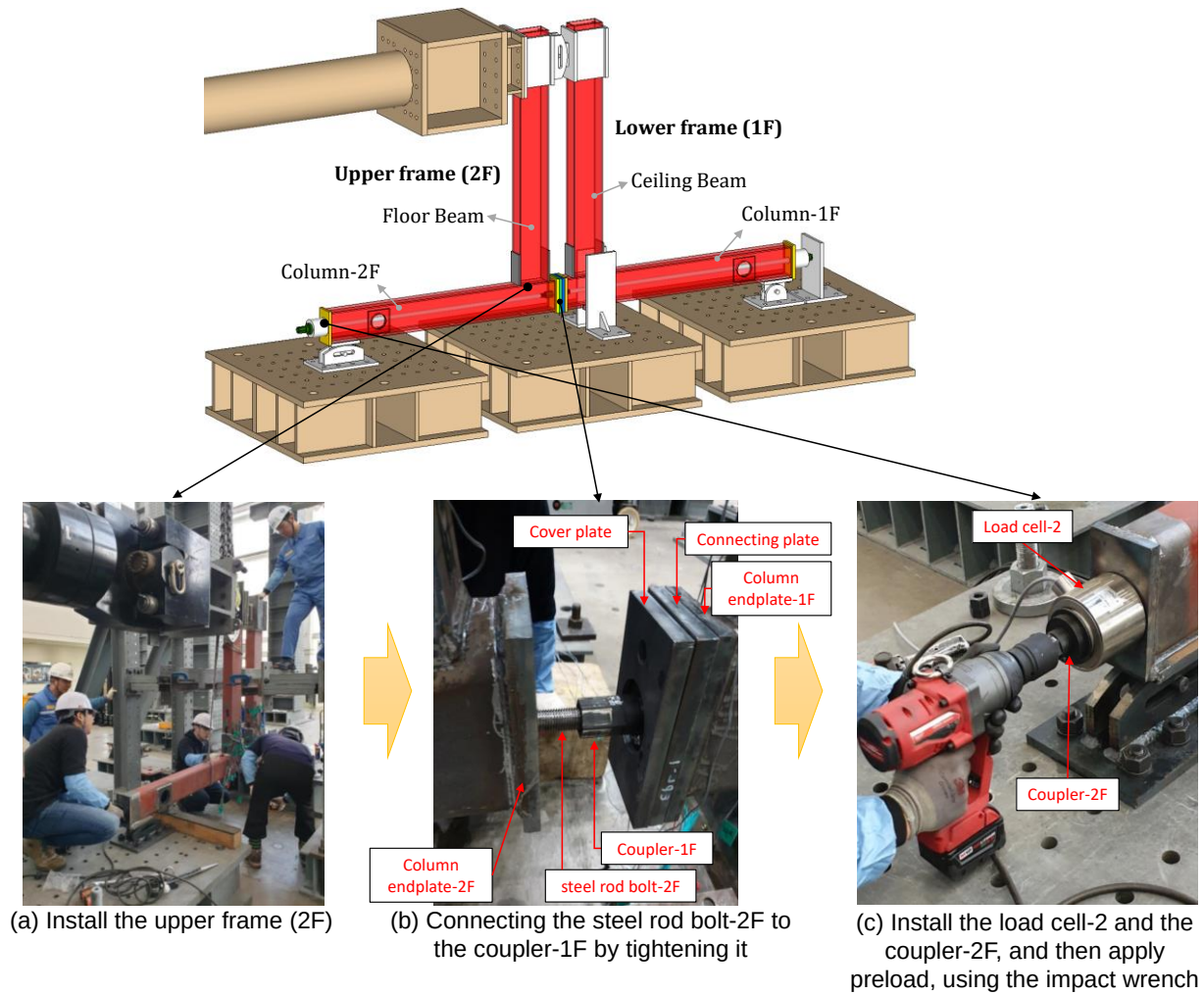


Figure 4.42: Test procedure for the beam-to-column joint with the AJ connection: Step 1 and Step 2

STEP 3: Installation of the upper frame (2F) & applying preload to column-2F



STEP 4: Installation of the lateral supports, and then applying a lateral force

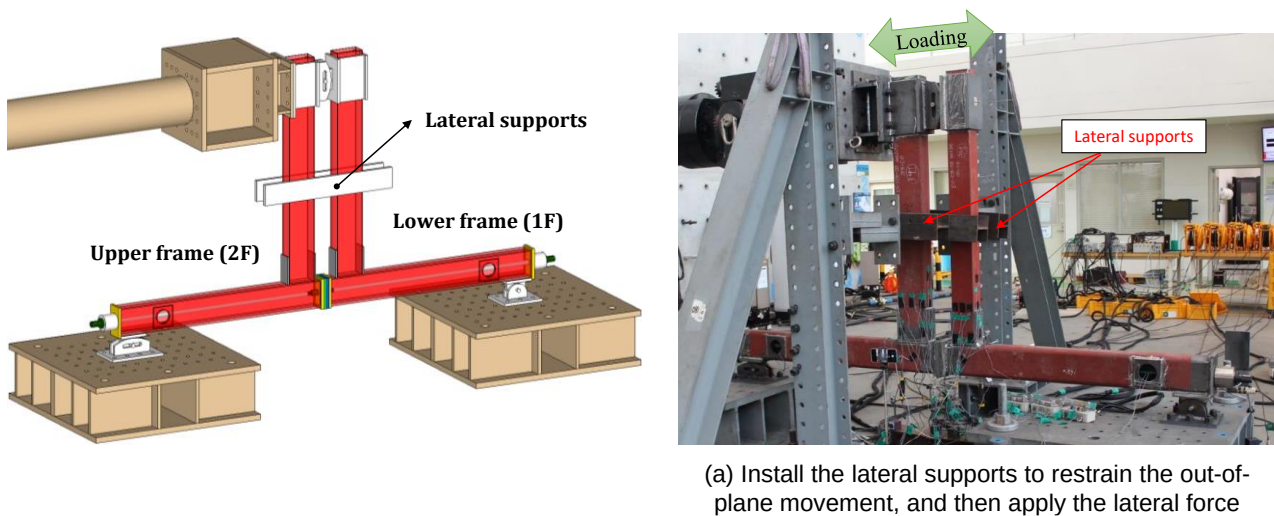


Figure 4.43: Test procedure for the beam-to-column joint with the AJ connection: Step 3 and Step 4

4.4.4. Instrumentation

All specimens were instrumented to characterise the mechanical behaviour of the joint. Linear variable differential transformers (LVDTs) were placed to measure the displacement of the specimens, as shown in Figure 4.44. Transducer D1 was placed at the tip of beams to measure the horizontal displacement, whereas transducers D2 and D3 were set to monitor the horizontal displacements of columns. Transducers D4, D5, D6, and D7 were placed to measure the vertical displacements of the columns, and transducers D8, D9, D10, and D11 were placed in the diagonal direction on the columns to monitor the shear deformation of the panel zone. Transducers D12 and D13 were placed horizontally to measure the gap-opening at the connection for only the P-DB and P-SB specimens.

Strain gauges were attached to the columns, beams, panel zones, and SRBs to investigate the stress distribution of the specimens. The arrangement of strain gauges is shown in Figure 4.45. The strain gauges were attached in two locations for the SRBs, as indicated in Figure 4.45 (G-view and H-view). Since it was difficult to attach the strain gauges on the bolt thread, strain gauges No. 31 – No. 34 were attached to the bolt shank just above the bolt thread near the connection to indirectly measure the stress variation (refer to Figure 4.62 (d)). Additional strain gauges (No. 29 – No. 30) were attached to the bolt shank far away from the connection to measure stress variation without the possible influence of localised bending. The load cells were installed to measure the applied preload in the columns while assembling and testing the joint specimens.

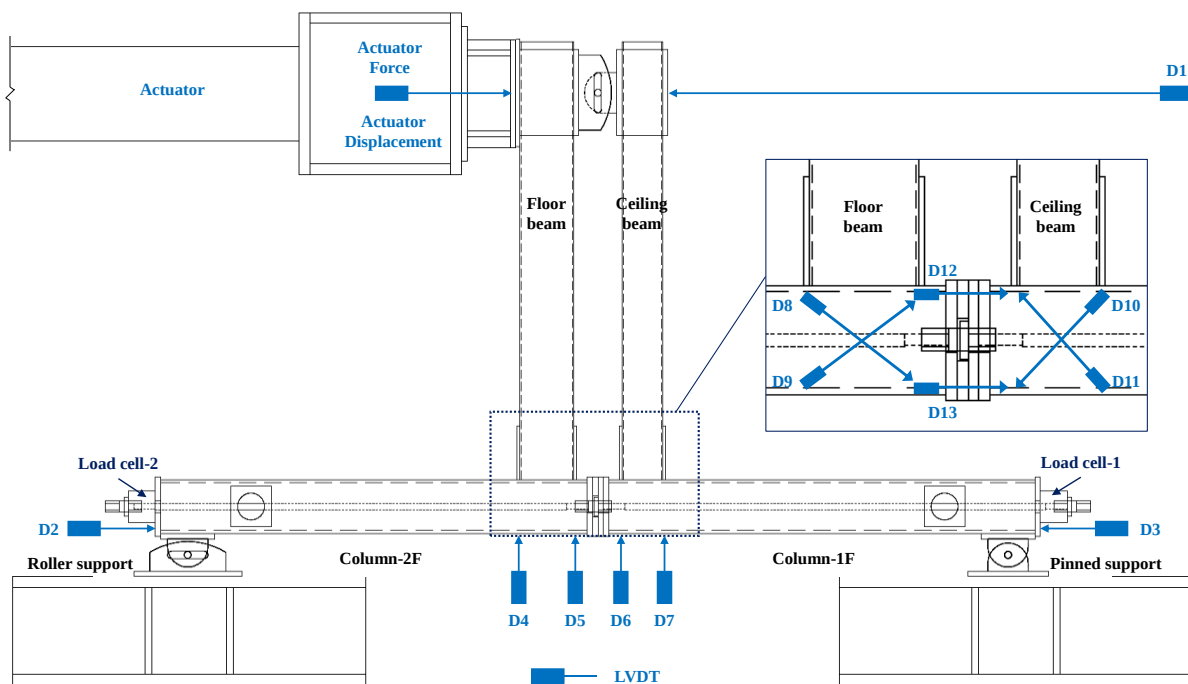


Figure 4.44: Instrumentation arrangement for the beam-to-column joint test – Displacement transducers

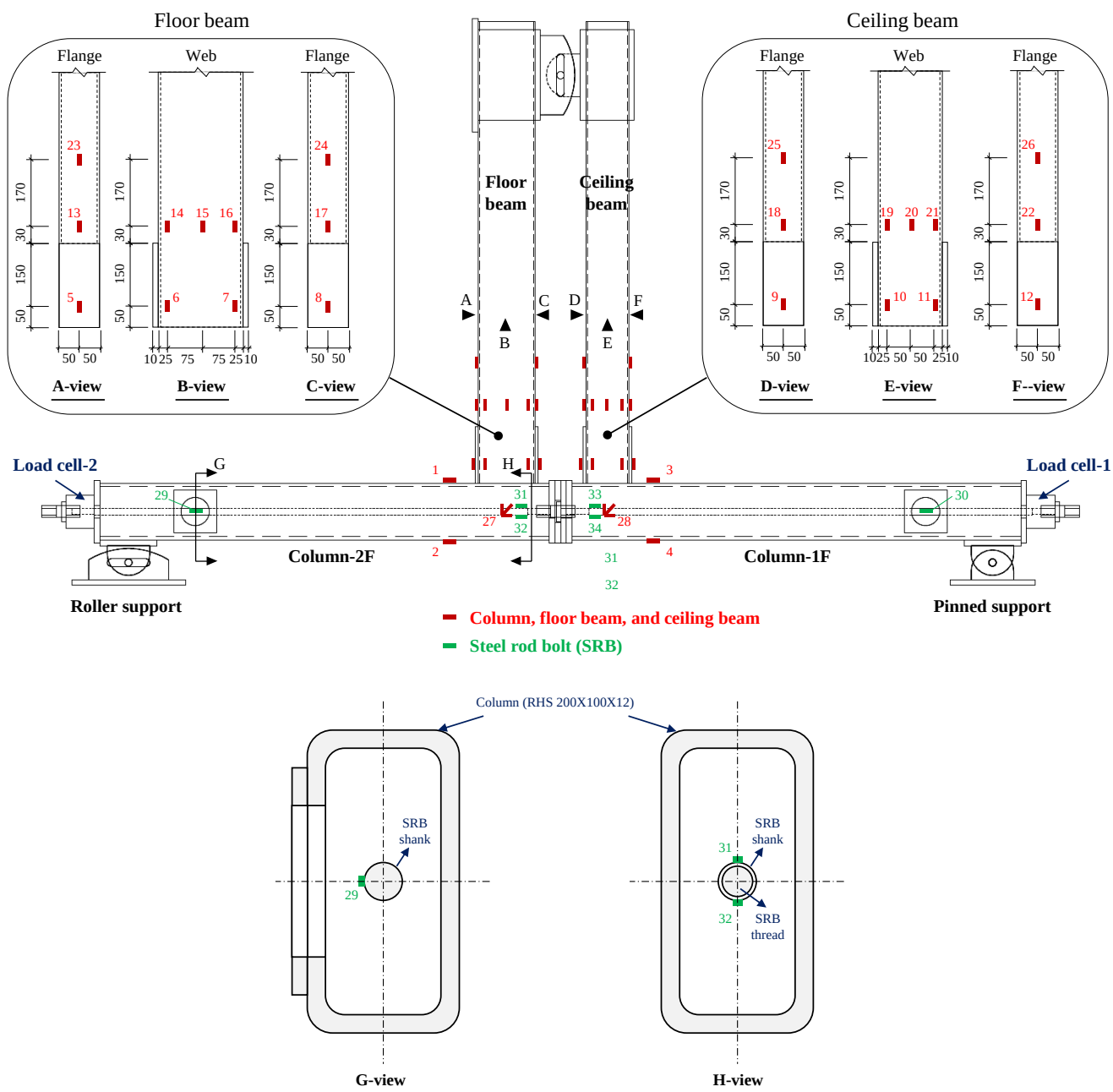


Figure 4.45: Instrumentation arrangement for the beam-to-column joint test – Strain gauges (units: mm)

4.4.5. Test Control Parameter

Parameters to evaluate rotation capacity of a beam-to-column joint are the interstorey drift θ [rad] of the test assembly and the bending moment M [kNm] at the column centreline produced by the lateral force. These parameters are calculated using (Eq. 4.3) and (Eq. 4.4) with symbols defined in Figure 4.46 and used to draw the moment-interstorey drift relationship for the test assembly. Specifically, the moment-interstorey drift (M - θ) curve is drawn with reference to the column centreline. In the test assembly, the locations of plastic hinges are in the beams near the reinforcing plates welded to the beams. Therefore, the expected plastic moment capacity (M_p) of the test assembly is determined by (Eq. 4.5).

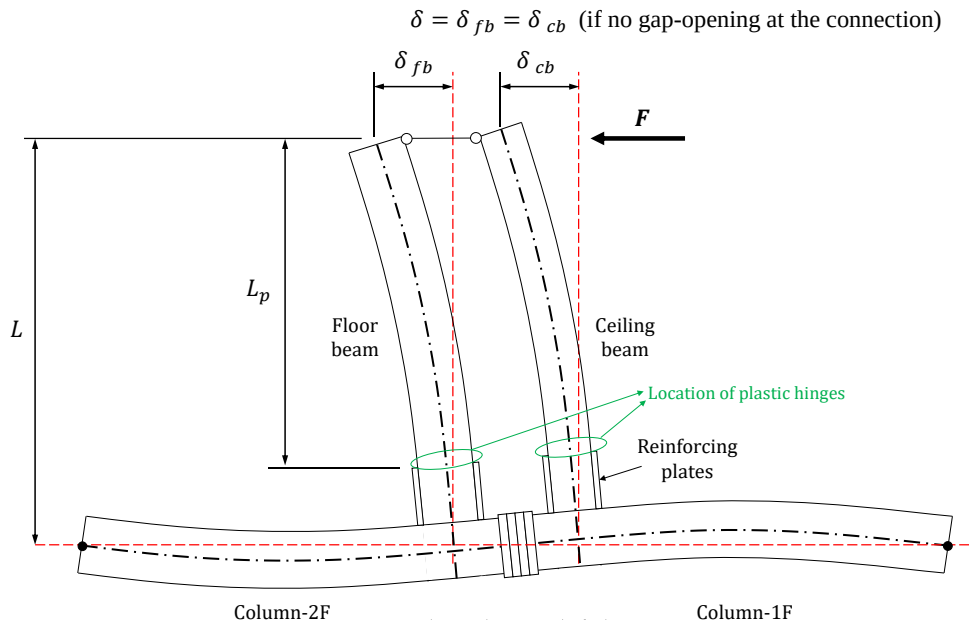


Figure 4.46: Definition of parameters for tests of beam-to-column joints

$$\theta = \frac{\delta}{L} \quad (\text{Eq. 4.3})$$

$$M = F \cdot L \quad (\text{Eq. 4.4})$$

$$M_p = \frac{M_{pl,Rb,Beam}}{L_p} \cdot L \quad (\text{Eq. 4.5})$$

where:

F	Lateral force applied at the tip of the beam;
θ	Interstorey drift of the test assembly;
M	Bending moment at the column centreline;
M_p	Plastic moment strength of the joint at the column centreline;
$M_{pl,Rb,Beam}$	Plastic bending moment resistance of the floor beam and ceiling beam (refer to Eq. 3.97);
L	Beam length to column centreline;
L_p	Beam length to the end of the reinforcing plate;
δ	Lateral displacement at the tip of the beam;
δ_{fb}	Lateral displacement at the tip of the floor beam;
δ_{cb}	Lateral displacement at the tip of the ceiling beam.

4.4.6. Loading Protocol

In the loading stage, the preload was applied first to the columns by tightening the coupler onto the SRBs. Then, monotonic loading or quasi-static cyclic loading was applied by a hydraulic actuator (3000kN) at the tip of the beams.

For the monotonic loading, a pulling force was applied by gradually increasing the displacement at the tip of the beams until the specimen failed. To estimate the initial stiffness, one unloading-reloading cycle was introduced. For the quasi-static cyclic loading, the ANSI/AISC 341-10 [111] loading protocol was used, which is shown in Table 4.40 and Figure 4.47. In the tests, the interstorey drift ratio of 0.01 corresponded to a displacement of 15.4mm.

Table 4.40: ANSI/AISC 341-10 (2010) cyclic loading protocol

Step	Drift (rad)	Number of cycles
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2
8	0.04	2
9	0.05	2

The loading is continued with increments of 0.01 rad, with two cycles per step.

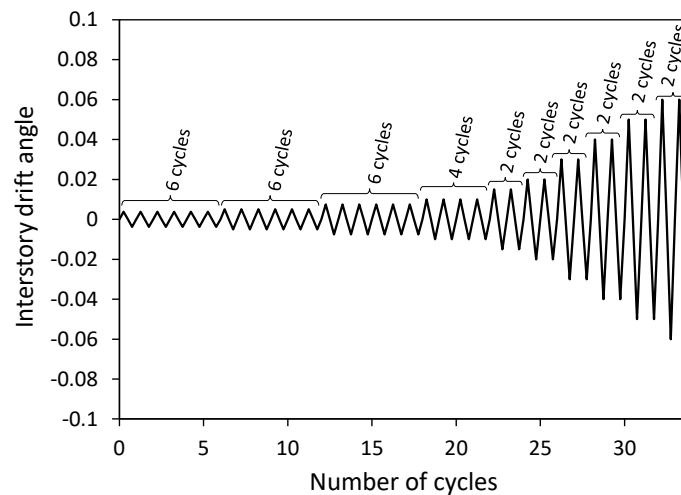


Figure 4.47: ANSI/AISC 341-10 (2010) cyclic loading protocol

4.4.7. Determination of the Demand Preload

As discussed in Section 3.5.4, the impact of a gap-opening at the AJ connection disturbs utilising the total plastic moment capacities of the floor and ceiling beams by causing only one beam to form a plastic hinge instead of both beams, which would reduce the ductility of the beam-to-column joint, and consequently, the magnitude of preload should be selected so as to prevent gap-opening.

Recalling that as per the capacity design principle, the preload column-to-column connection is a non-dissipative component and hence, the demand preload for the connection shall be calculated in the same way as designing a non-dissipative member. That is, the preload shall prevent the gap-opening at the connection until both the floor and ceiling beams yield, reach their plastic moment capacities, undergo hardening and fail at the positions intended to incur plastic deformations. Figure 4.48 shows a test specimen with the location of expected plastic hinges and forces acting on the columns and beams when the lateral force is applied at the tip of the beams. Based on Section 3.6.4.2, design forces for the demand preload are determined first, and then the demand preload is calculated by the following steps:

1. Define a design bending moment of the floor beam and ceiling beam for demand preload

First, a design bending moment for the non-dissipative connection needs to be defined in accordance with section 6.5.5 of Eurocode 1998-1 [86]. The design bending moment for the demand preload is fully strain hardened plastic hinges on the floor beam ($M_{g,Ed,fb}$) and the ceiling beam ($M_{g,Ed,cb}$) as per (Eq. 4.6) and (Eq. 4.7), respectively. Since the actual yield stress of the beam material is generally higher than the nominal yield stress, the plastic moment is multiplied by the material overstrength factor γ_{sh} . Also, the strain hardening effect is taken into account by multiplying the plastic moment by the hardening overstrength factor γ_{ov} .

$$M_{g,Ed,fb} = \gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,fb} \quad (\text{for floor beam}) \quad (\text{Eq. 4.6})$$

$$M_{g,Ed,cb} = \gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,cb} \quad (\text{for ceiling beam}) \quad (\text{Eq. 4.7})$$

where :

$M_{g,Ed,fb}$	Design bending moment for the floor beam;
$M_{g,Ed,cb}$	Design bending moment for the ceiling beam;
$M_{pl,Rd,fb}$	Nominal plastic moment resistance of the floor beam;
$M_{pl,Rd,cb}$	Nominal plastic moment resistance of the ceiling beam;
γ_{sh}	Hardening overstrength factor, $\gamma_{sh} = 1.1$ according to EN 1998-1;
γ_{ov}	Material overstrength factor, $\gamma_{ov} = 1.25$ according to EN 1998-1.

2. Calculate the lateral force $F_{p,fb}$ and $F_{p,cb}$ to make each beam reach the design bending moment at the expected location

As the lateral force F is applied on the tip of the beams in Figure 4.48, the shear forces acting on the floor beam V_{fb} and the ceiling beam V_{cb} are determined according to (Eq. 3.60) and (Eq. 3.61) as below:

$$V_{fb} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F \quad (\text{for floor beam}) \quad (\text{Eq. 3.60})$$

$$V_{cb} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F \quad (\text{for ceiling beam}) \quad (\text{Eq. 3.61})$$

The shear forces at the tip of the floor and ceiling beams to cause each beam to reach their design bending moment ($M_{g,Ed,fb}$ and $M_{g,Ed,cb}$) can be determined, respectively, as follows:

$$V_{p,fb} = \frac{M_{g,Ed,fb}}{L_p} = \frac{\gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,fb}}{L_p} \quad (\text{for floor beam}) \quad (\text{Eq. 4.8})$$

$$V_{p,cb} = \frac{M_{g,Ed,cb}}{L_p} = \frac{\gamma_{sh} \cdot \gamma_{ov} \cdot M_{pl,Rd,cb}}{L_p} \quad (\text{for ceiling beam}) \quad (\text{Eq. 4.9})$$

Where :

- $V_{p,fb}$ The shear force at the tip of the floor beam to cause the design bending moment $M_{g,Ed,fb}$;
- $V_{p,cb}$ The shear force at the tip of the ceiling beam to cause the design bending moment $M_{g,Ed,cb}$;
- L_p Distance between the loading point and the location of the expected plastic hinge.

Using (Eq. 3.60) and (Eq. 4.8), the lateral force $F_{p,fb}$ making the floor beam reach the design bending moment ($M_{g,Ed,fb}$) at the expected location of plastic hinge is expressed as (Eq. 4.10). In the same way, the lateral force $F_{p,cb}$ making the ceiling beam reach its plastic moment is presented as (Eq. 4.11) through (Eq. 3.61) and (Eq. 4.9).

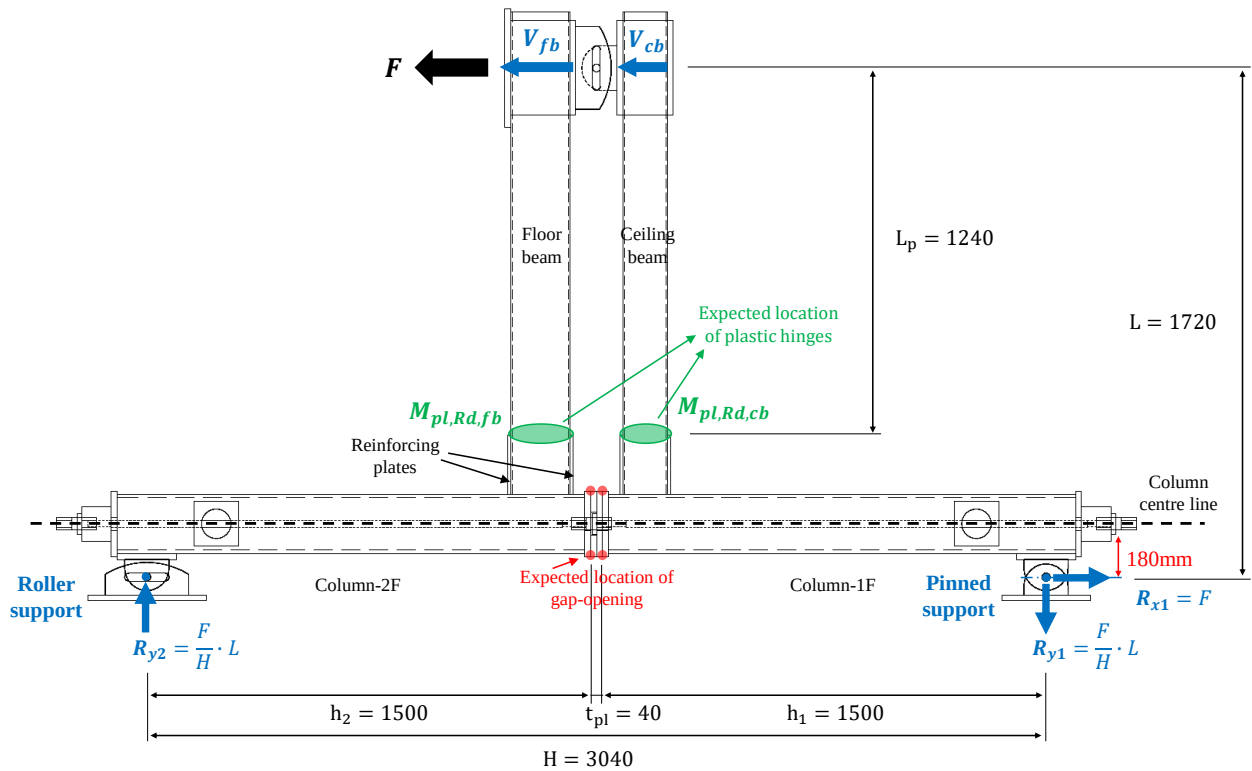


Figure 4.48: Location of plastic hinges and forces acting on the columns and beams due to the lateral force F in the test specimen

$$F_{p,fb} = \left(\frac{M_{g,Ed,fb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{fb}} \right) \quad (\text{for floor beam}) \quad (\text{Eq. 4.10})$$

$$F_{p,cb} = \left(\frac{M_{g,Ed,cb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{cb}} \right) \quad (\text{for ceiling beam}) \quad (\text{Eq. 4.11})$$

3. Determine the design forces acting on beams and columns for demand preload P_g

For column-2F

As shown in Figure 4.49 (a), the design forces to calculate the demand preload for column-2F becomes the shear forces acting on column-2F $V_{Ed,C2}$ and the floor beam $V_{Ed,B2}$ when the lateral force $F_{p,fb}$ is applied at the tip of the beams.

$$V_{Ed,B2} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = V_{p,fb} \quad (\text{acting on floor beam}) \quad (\text{Eq. 4.12})$$

$$V_{Ed,C2} = \frac{F_{p,fb}}{H} \cdot L \quad (\text{acting on column} - 2F) \quad (\text{Eq. 4.13})$$

For column-1F

As presented in Figure 4.49 (b), the design forces to calculate the demand preload for column-1F becomes the shear forces acting on column-1F $V_{Ed,C1}$ and ceiling beam $V_{Ed,B1}$ when the lateral force $F_{p,cb}$ is applied at the tip of the beam.

$$V_{Ed,B1} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,cb} \quad (\text{acting on ceiling beam}) \quad (\text{Eq. 4.14})$$

$$V_{Ed,C1} = R_{y1} = \frac{F_{p,cb}}{H} \cdot L \quad (\text{acting on column} - 1F) \quad (\text{Eq. 4.15})$$

$$R_{x1} = F_{p,cb} \quad (\text{acting on column} - 1F) \quad (\text{Eq. 4.16})$$

It is noted that the beam-to-column tests were carried out by placing the columns horizontally because it would have been difficult to apply preload to the columns if the columns were arranged vertically. For this reason, the supports at the end of the columns were installed under the columns (refer to Figure 4.48), and consequently, the reaction force generated by the applied lateral force was not located on the column centreline, but on the centreline of the roller and pinned supports, which was 180mm lower than the column centreline. It was expected that the eccentricity of the horizontal reaction force R_{x1} generated at the pinned support would affect the gap-opening at the connection, and the eccentricity was therefore taken into account when calculating the demand preloads.

4. Determine the demand preload

Using the process Steps 1, 2 and 3 in Section 3.4.3, the demand preload to prevent the gap-opening P_g at the AJ connection is determined. Also, using the process Steps 1, 2 and 3 in Section 3.4.4, the demand preload to prevent the slip P_s at the connection can be calculated. The detailed calculations of the demand preloads for the P-DB and P-SB specimens are shown in Appendix B.

Table 4.41 presents the demand preloads P_g and P_s calculated for the P-DB and P-SB specimens. For the test specimens, the demand preload P_g is selected to see if the gap-opening is prevented. In the pre-test numerical simulation presented in Section 4.4.8, these demand preloads P_g are applied to the test specimens to investigate the behaviour of the joint under the monotonic loading. The simulation results show that the gap-opening is prevented at the connection by the demand preload in the P-DB specimen, but the P-SB specimen has the gap-opening although it is very minor (refer to Section 4.4.8).

Table 4.41: Demand preloads P_g and P_s for the P-DB and P-SB specimens

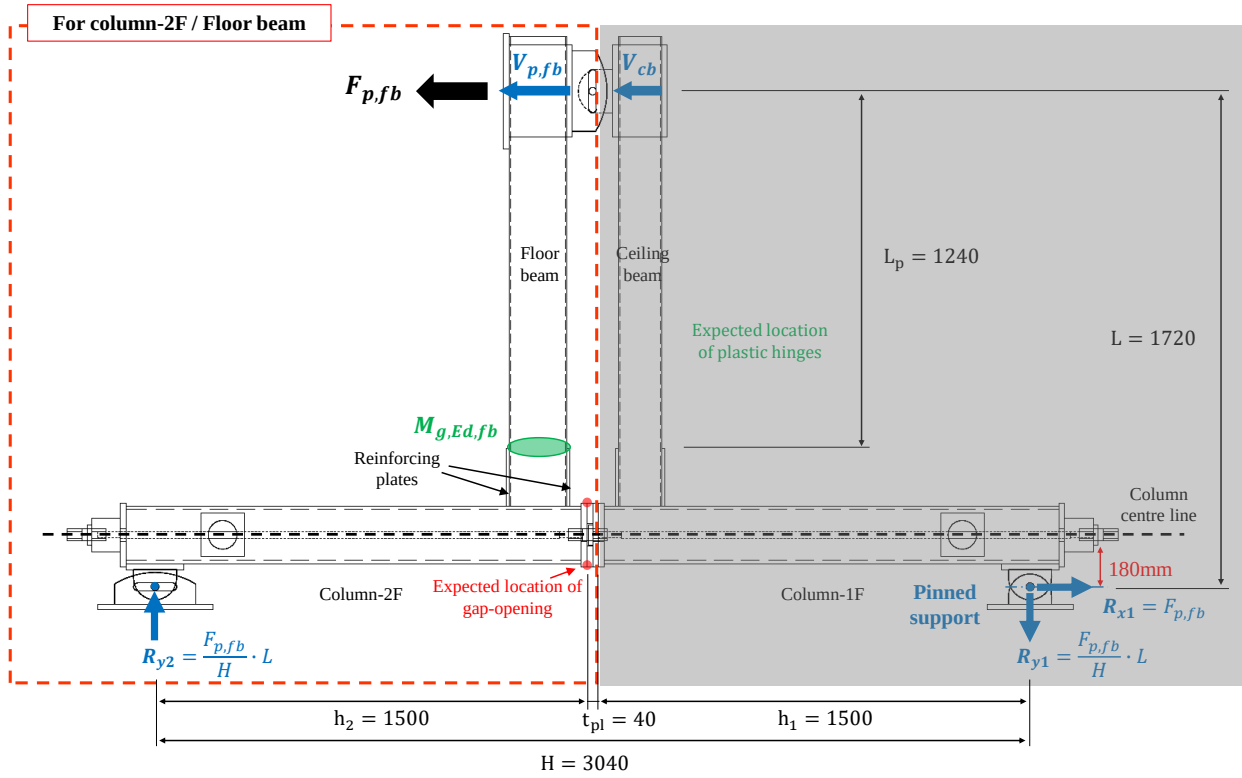
Preload	P-DB-M/Q	P-SB-M/Q
Demand preload for gap-opening (P_g)	188 kN	131 kN
Demand preload for slip (P_s) *	229 kN	221 kN

* Assuming the contact surfaces between plates in the AJ connection are cleaned surfaces by wire-brushing with loose rust removed [100], the friction coefficient (μ) of 0.3 is used.

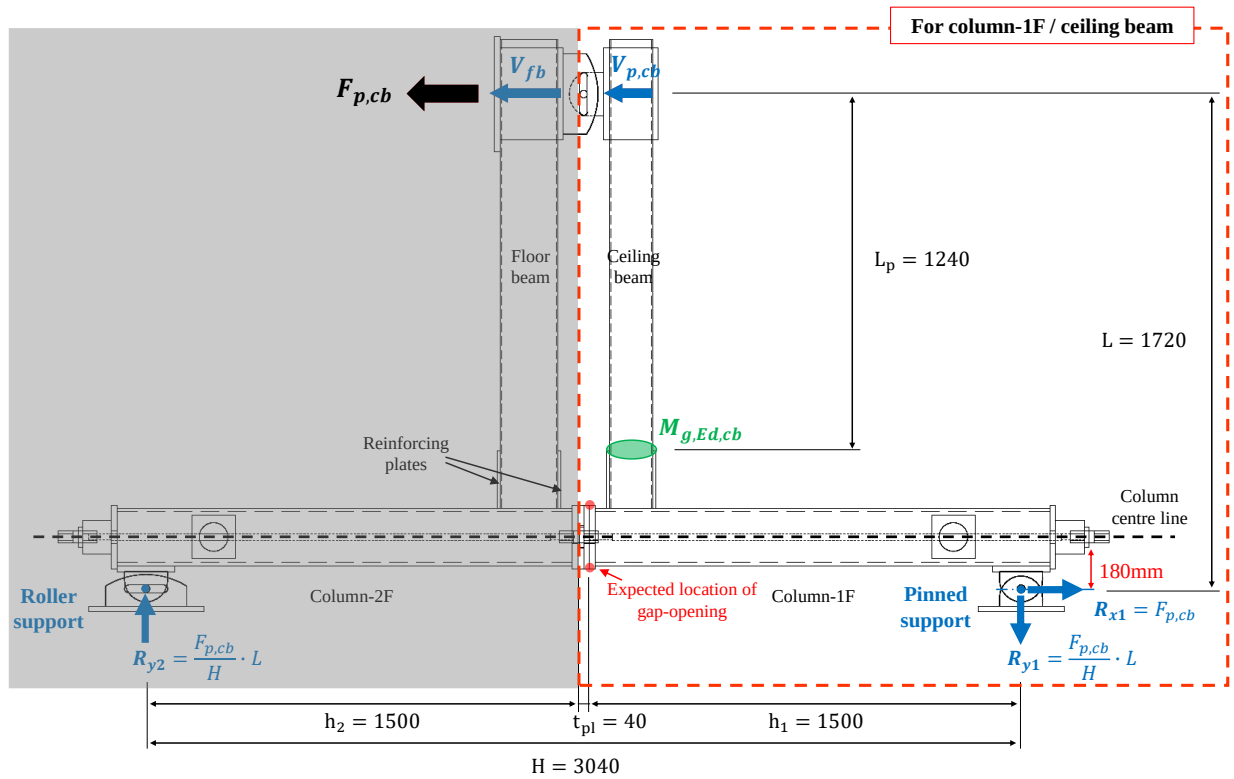
For the beam-to-column tests, it was decided to apply preloads increased by 10% from the demand preloads to ensure the reliable behaviour of the joint without gap-opening occurring at the connection. These enhanced demand preloads are shown in Table 4.42.

Table 4.42: Final applied preloads P_g for the P-DB and P-SB specimens for the beam-to-column joint tests

Preload	P-DB-M/Q	P-SB-M/Q
Applied preload for test ($1.1 \times P_g$)	207 kN	145 kN



(a) For column-2F – Floor beam



(b) For column-1F – Ceiling beam

Figure 4.49: Design forces to determine the demand preload for the beam-to-column joint in test setup

4.4.8. Pre-Test Numerical Investigation of the Beam-to-column Joints

Before conducting the beam-to-column joint test, pre-test numerical simulations were performed under monotonic loading to predict the behaviour of the joint and avoid unexpected failure during the tests. The simulations were performed using the finite element modelling software Abaqus [112].

Since the numerical simulations were carried out before conducting a material sample test, the elastic-perfectly plastic material model was used for the simulations based on the nominal material properties of the steel grades, as presented in Figure 4.50. For the floor and ceiling beams, which are the dissipative members, the nominal yield stress was multiplied by both the material overstrength factor ($\gamma_{ov} = 1.25$) and the hardening overstrength factor ($\gamma_{sh} = 1.1$) to estimate the actual yield stress of the steel grade and consider the effect of strain, respectively. Thereby, it was possible to investigate the avoidance of gap-opening at the AJ connection and the behaviour of the non-dissipative members, including the column, panel zone and SRBs, until the beams with their estimated actual maximum strength had developed plastic deformation and reached their ultimate capacities. For all other components, the nominal material properties were used.

The FE modelling of the AJ connection consisted of two steps: Firstly, the demand preload calculated in the previous Section 4.4.7 (P_g in Table 4.41) was applied to the column-to-column connection using a Static General analysis. Secondly, a lateral force was applied by controlling a displacement at the tip of the beams. The latter analysis was conducted as a Dynamic Explicit analysis due to the large number of contact surfaces in the FE model and the development of inelastic deformation. All components of the beam-to-column joint were modelled using solid elements. It is to be noted that initial imperfections were not considered in the FE models for the pre-test numerical simulations, and the preload was applied at the centre of the column section. The friction coefficient (μ) of 0.3 for surfaces cleaned by wire-brushing with loose rust removed [100] was used for contact surfaces between the plates at the AJ connection, which was the same condition as the beam-to-column joint test. Detailed information about the FE model is provided in Section 5.3.1.

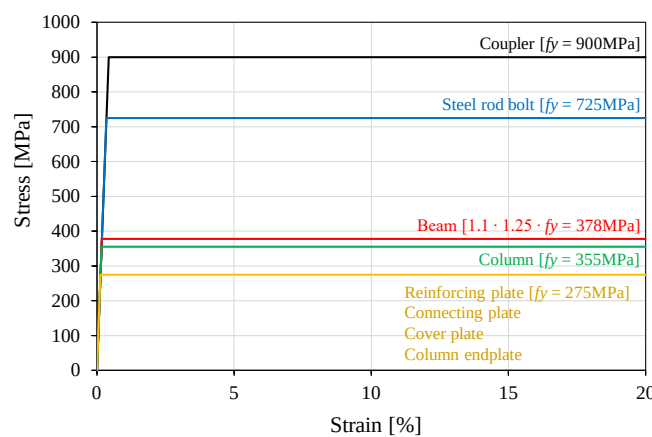


Figure 4.50: Elastic-perfectly plastic material properties of the steel grades used for the pre-test numerical simulations of the joint

The numerical simulations allowed investigation of the failure mode of beam-to-column joint with the AJ connection, the possible gap-opening at the connection, the monotonic response curve, and distributions of stress and plastic strain.

(1) Failure mode

Figure 4.52 presents the moment versus interstorey drift curve (computed at the column centreline) for the P-DB, P-SB, and W-SB connections. While gap-opening did not occur in the test of the P-DB preload connection, gap-opening did occur late in the test of the P-SB connection. Since the gap-opening occurred after the floor and ceiling beams yielded, it was very minor and had little impact on the overall behaviour of the P-SB joint. All specimens showed ductile behaviour by plastic hinges forming at the expected locations in both floor and ceiling beams near the reinforcing plates. Figure 4.51 shows the failure modes of all specimens, which are the intended beam failure modes. In the case of P-DB, in which the floor beam has the larger cross-sectional size than the ceiling beam, the floor beam yielded first and the ceiling beam yielded shortly after as predicted in Section 3.5.4. Both beams reached their plastic moment capacities, and then the floor beam failed first by local buckling (see Figure 4.52 (a)) followed by the ceiling beam. For P-SB and W-SB having the same cross-sectional size for the floor and ceiling beams, the floor and ceiling beams yielded and reached their plastic moment capacities simultaneously. They also failed at the same time (see Figure 4.52 (b)). In all simulations, regardless of the difference in the cross-sectional size of the floor and ceiling beams, both beams eventually reached their plastic moment. For all specimens, until the beams failed, the non-dissipative components (columns, couplers and SRBs) remained in the elastic range, as shown in Figure 4.54 (b) and Figure 4.55 (b). It was observed that the centre of the plastic hinges was located at a distance approximately 50mm away from the end of the reinforcing plates on the floor and ceiling beams. The failure shapes and stress distributions of P-DB and P-SB are shown in Figure 4.54 (a) and Figure 4.55 (a), respectively.

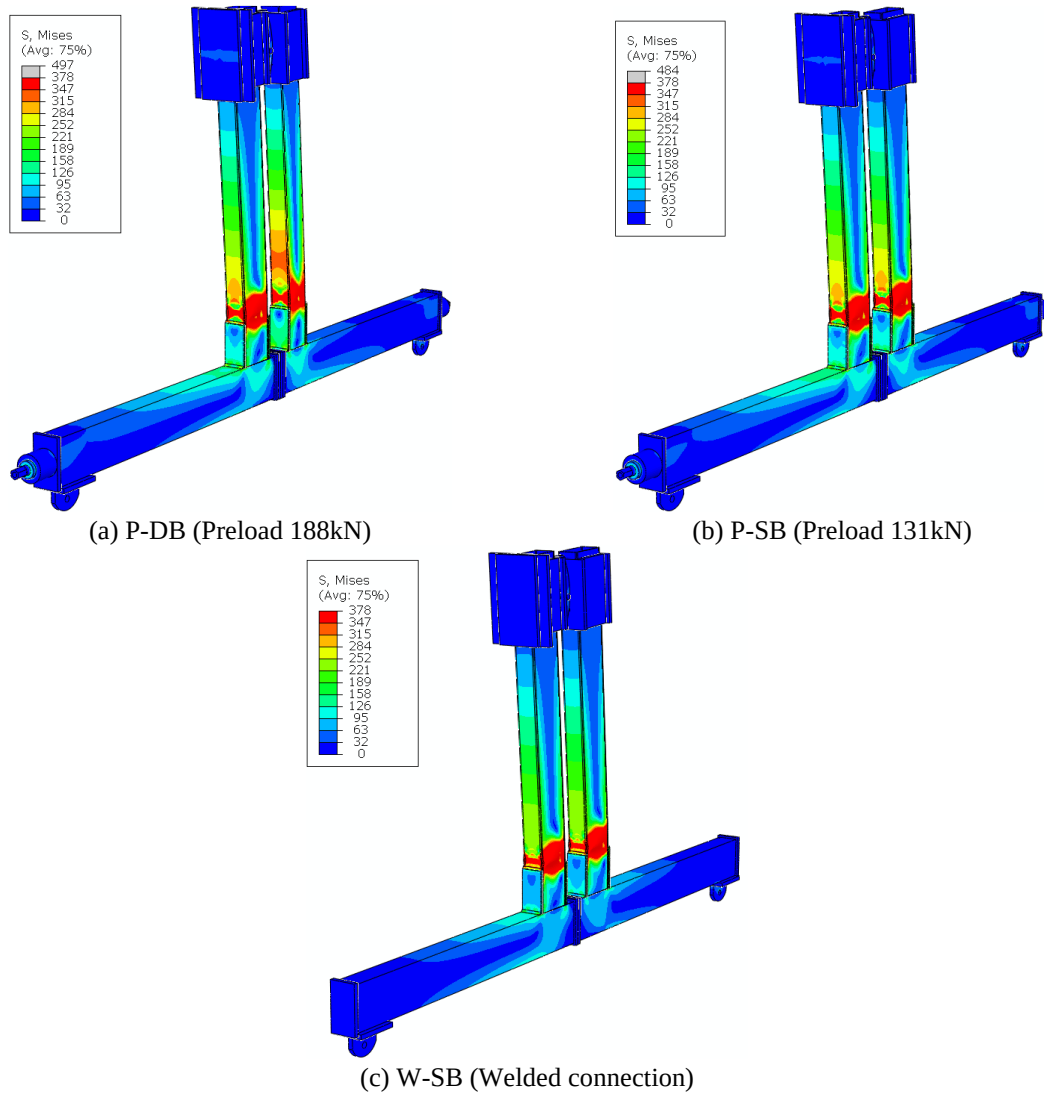
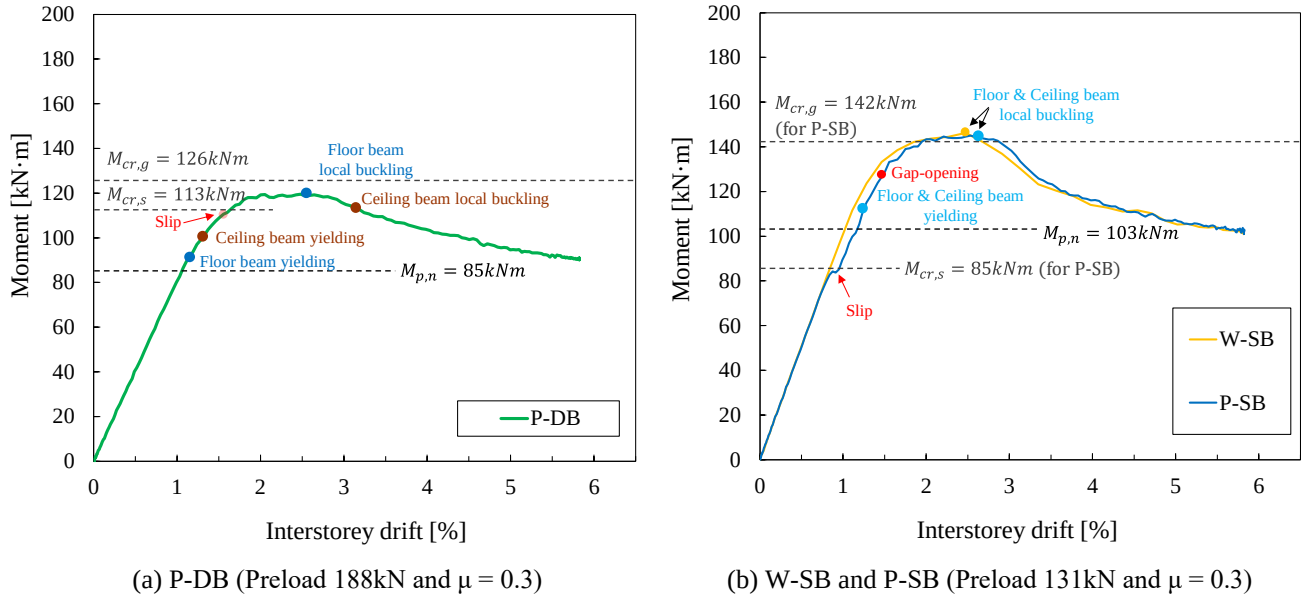


Figure 4.51: Failure mode from pre-test numerical simulations (Mises stress)

(2) Strength and stiffness

As shown in Figure 4.52, all specimens reached much higher moment strength than the nominal value ($M_{p,n}$) given by (Eq. 4.5). This is because the yield stress of the beam was multiplied by the material overstrength factor (γ_{ov}) and the hardening overstrength factor (γ_{sh}). P-SB had higher strength and stiffness than P-DB because of its larger ceiling beam cross-section. Comparing P-SB to W-SB, which is identical to the P-SB specimen except for the connection type, P-SB had the same initial stiffness and ultimate strength as W-SB and would have the same overall performance as the welded connection in case no gap-opening or slip occurred. Although a gap-opening occurred in P-SB, it occurred at a moment approximately 10% lower than the expected strength ($M_{cr,g}$) and had no noticeable effect on the stiffness and strength.

Unexpectedly, the demand preload (P_g in Table 4.41) did not prevent the gap-opening of the P-SB specimen, even though the effect of the gap was quite minor and could be ignored. Later analysis revealed that in P-SB, the slip, which occurred prior to the gap-opening, led to earlier gap-opening at a lower strength than the expected strength implied by the demand preload P_g assuming no gap-opening occurs (refer to Section 5.5.1.2(3)). Specifically, after slip occurs, a complex bearing between the coupler and the plates at the AJ connection affects the applied preload.



$M_{p,n}$: Nominal plastic moment strength of the beam-to-column joint by (Eq. 4.5).

$M_{cr,g}$: Critical moment strength to prevent the gap-opening by the applied preload at the connection by Table 3.3 and (Eq. 3.69).

$M_{cr,s}$: Critical moment strength to prevent the slip by the applied preload at the connection by (Eq. 3.81) and (Eq. 3.82).

Figure 4.52: Moment-interstorey drift curves for P-DB, P-SB, and W-SB obtained from pre-test numerical simulations

(3) Gap-opening and slip at the AJ connection

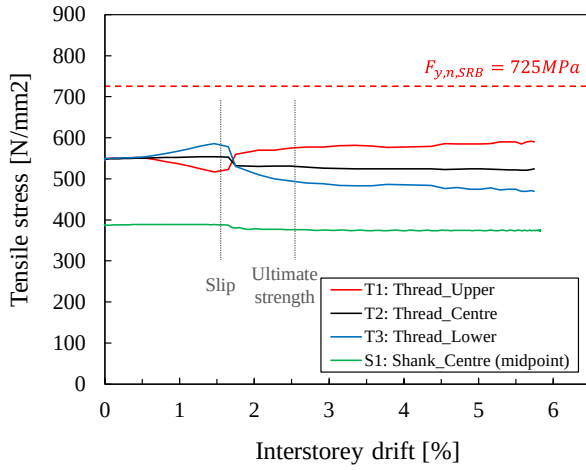
Figure 4.54 (c) and Figure 4.55 (c) show the variation of stress distribution at the P-DB and P-SB preload connections, respectively, for increasing interstorey drift levels. In the case of P-DB, the initial stress induced by the preload decreased gradually in the interior part of the connection as the lateral load increased, while the stress in the exterior part increased. Although the gap-opening did not occur at the P-DB connection, the result shows that if a gap-opening was to occur, it would occur at the interior part of the connection. Conversely, in the case of P-SB, the initial stress induced by the preload increased in the interior part of the connection when the lateral load increased, while the stress in the exterior part decreased, resulting in the abovementioned minor exterior gap-opening. Notwithstanding this gap-opening, the FE simulations confirmed that the demand preload calculated by the analytical approach (Table 4.41) can generally prevent gap-opening at the connection. As to slip in AJ connections,

both P-DB and P-SB experienced slip near the expected strength ($M_{cr,s}$), as shown in Figure 4.52. The slip in the AJ connection reduces the initial stiffness of the joint, and induces a bending moment in the SRBs near the coupler, as described in the following paragraph.

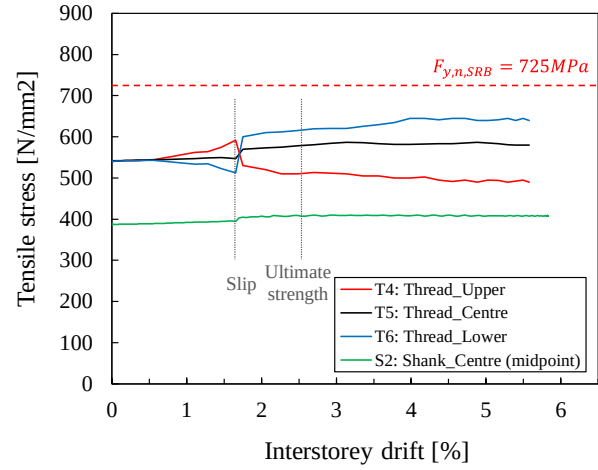
(4) Behaviour of steel rod bolts and couplers

Figure 4.53 presents the stress variations in the SRBs as functions of the interstorey drift for the P-DB and P-SB specimens. The locations of the measured stresses are shown in Figure 4.54 (a) and Figure 4.55 (a). The FE simulation results show that stresses in the SRBs in P-DB and P-SB remained below their nominal yield stress ($f_{y,n,SRB}$) prior to the floor and ceiling beams failing. The simulations also show that a gap-opening and slip change the stresses in the SRBs. The effect of gap-opening and slip on the axial force in the SRBs (see stresses S1 and S2 for bolt shank, and T2 and T5 for bolt thread in Figure 4.53) is relatively minor and hardly discernible for P-DB. However, slip induces bending of the SRBs with significant subsequent change to the stresses near the thread of SRBs (T1 and T3 for column-2F and T4 and T6 for column-1F). Prior to slip occurring, a small degree of bending is apparent (see difference between the blue and red curves in Figure 4.53 (a) and (b)), which resulted from a rotation of the coupler caused by the flexural deformation of the columns. The separation between the red and blue curves accelerates after slip occurred, implying significant additional bending and associated bending stresses, especially for P-SB, for which slip occurred well before reaching the ultimate strength. The simulation also showed that the SRB in column-1F was more affected by bending than the SRB in column-2F. That is, overall, the bolt thread in column-1F was more affected by the effect of bending than the bolt thread in column-2F.

The simulation results demonstrated the effect of slip at the AJ connection on the SRBs, which is important because it cannot be predicted analytically, notably the magnitude of the bending moment acting on the SRBs due to slip. The FE simulation confirmed that even if gap-opening is prevented, there is a possibility of damage to the SRB resulting from localised bending in case slip occurs under lateral loading. A more in-depth study of slip-induced bending is presented in Section 5.5.1.2.

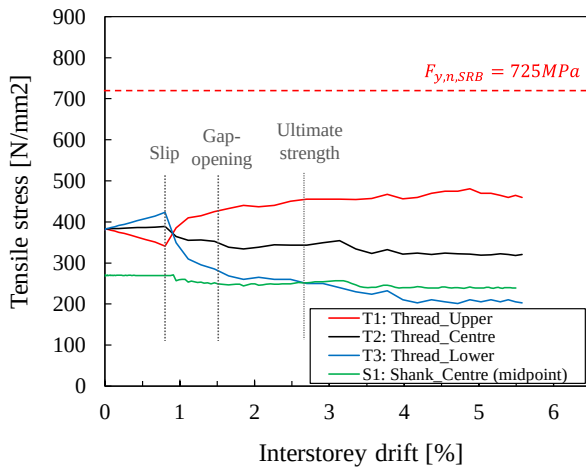


SRB in column-2F

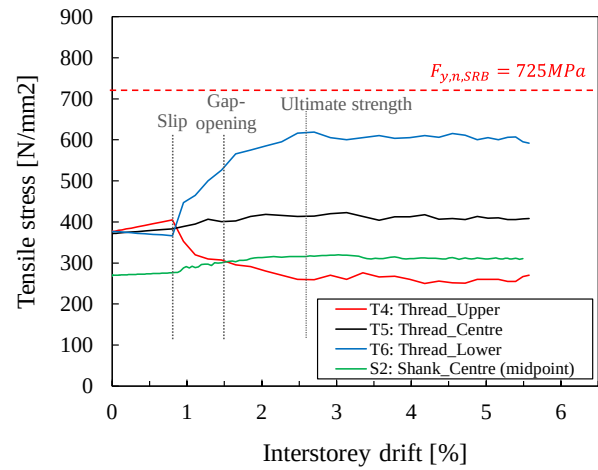


SRB in column-1F

(a) P-DB (Preload 188kN)



SRB in column-2F



SRB in column-1F

(b) P-SB (Preload 131kN)

Figure 4.53: Stress variation of SRBs in P-DB and P-SB

(5) Conclusion

The FE numerical simulation results showed that the P-DB and P-SB beam-to-column joints with AJ connections developed the intended beam failure modes with ductile behaviour, as required for a strong column-weak beam seismic moment-resisting frame. It was confirmed that the analytical preload calculation in Section 4.4.7 can be employed to prevent gap-opening from developing prior to the failure of the floor and ceiling beams. Additionally, it was confirmed that the AJ connection performs similarly to the welded connection provided the proper amount of preload is applied, and that slip may induce high unintended bending stresses in the SRBs.

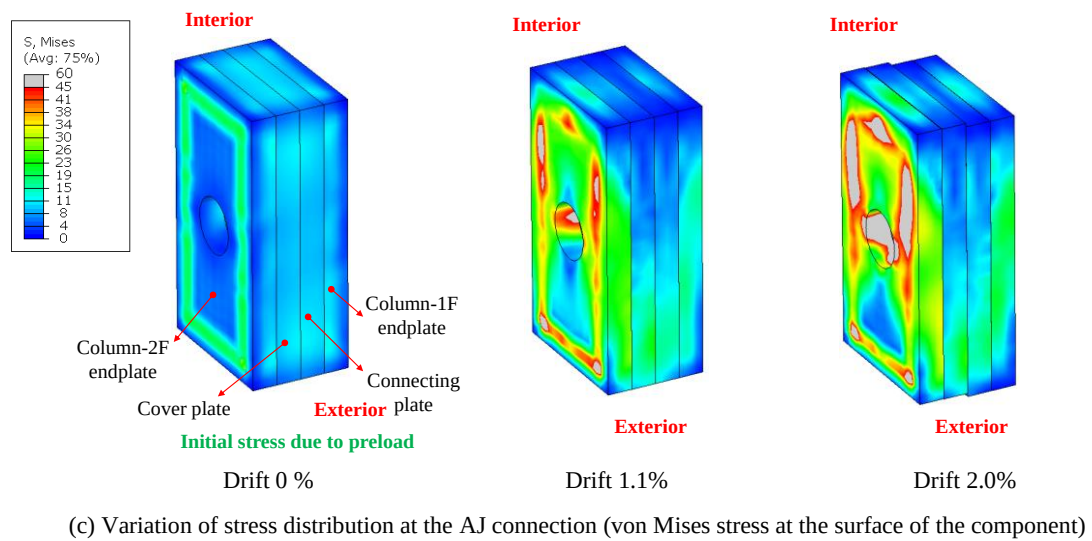
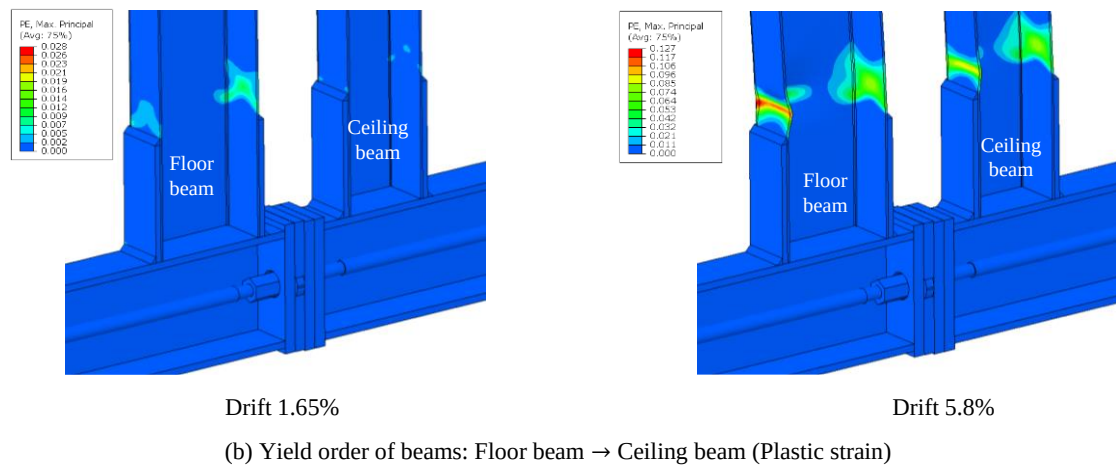
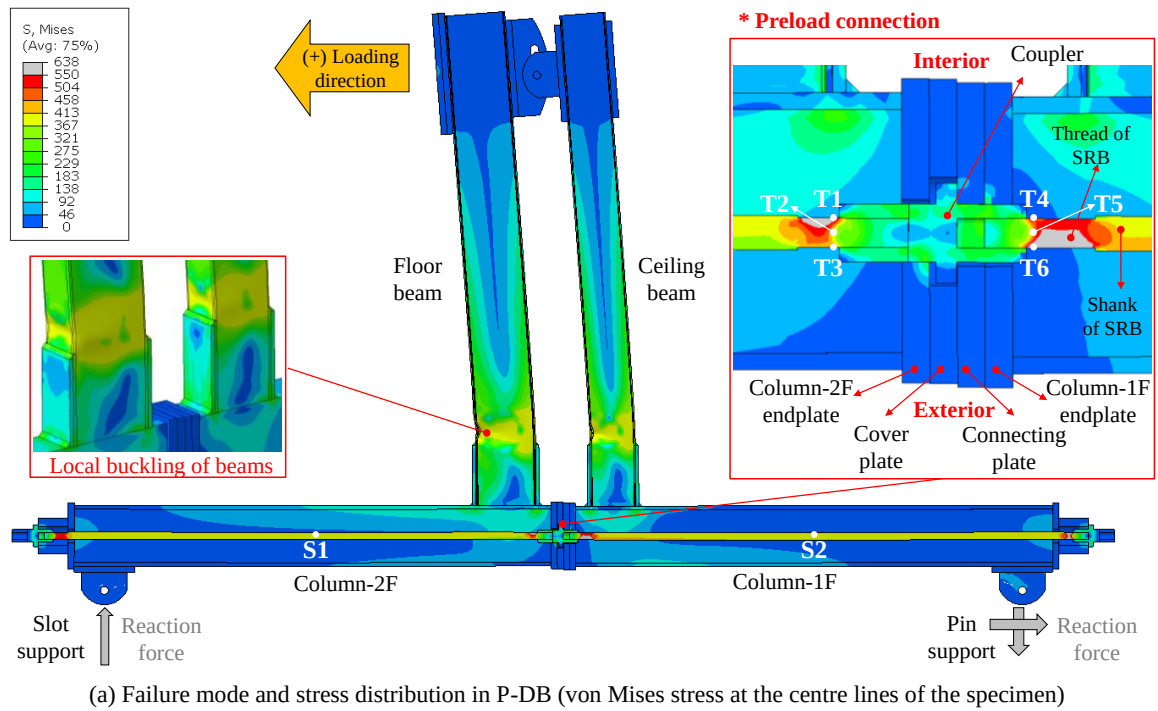


Figure 4.54: Results of pre-test numerical simulation for P-DB (Preload 188kN)

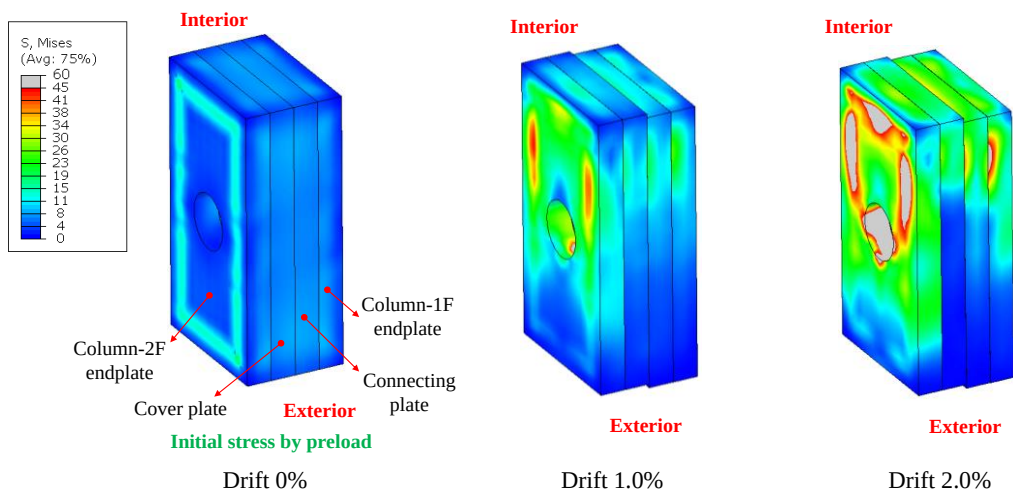
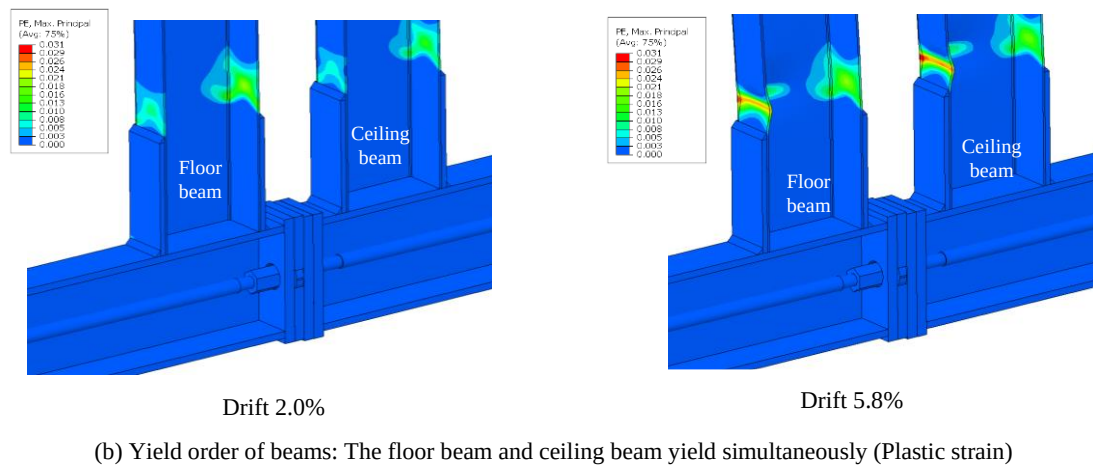
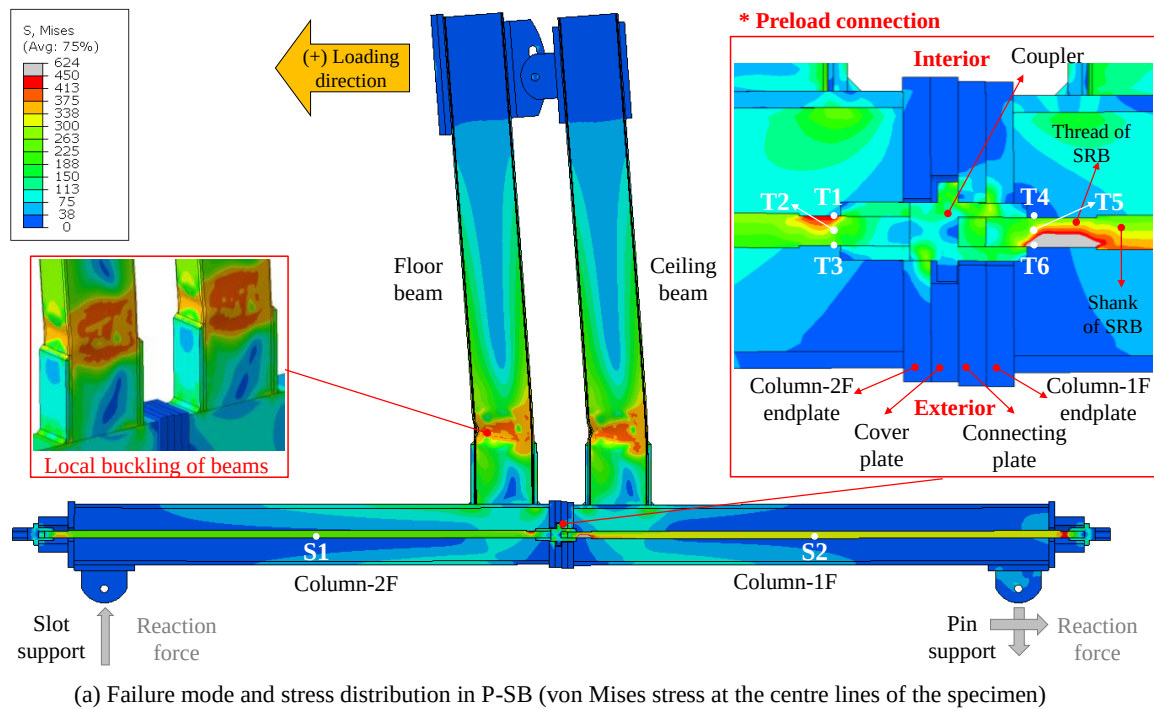


Figure 4.55: Results of pre-test numerical simulation for P-SB (Preload 131kN)

4.4.9. Experimental Observation

To ensure reliable behaviour of the joint without a gap-opening developing at the connection, the preload applied to all specimens was increased by 10% from the demand preload obtained by the analytical approach (refer to Table 4.42). The possible onset of gap-opening at the connection was determined based on two variations: the directly measured gap-opening displacement at the AJ connection (see Figure 4.44: LVDTs D12 and D13) and the increase of tensile load/stress in the SRBs measured by the load cells and strain gauges.

4.4.9.1. Monotonic Test

1) P-DB-M

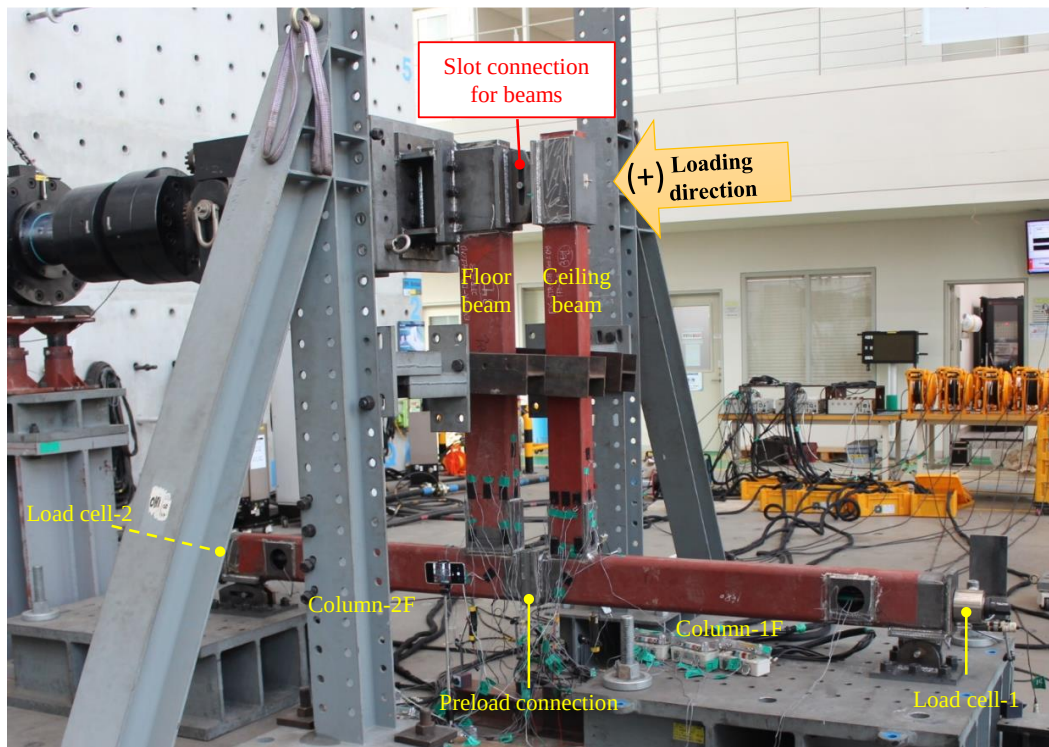


Figure 4.56: P-DB-M test assembly - before conducting the test

The demand preload was 207kN for the P-DB-M specimen. However, a preload greater than 155kN could not be applied with an impact wrench in the process of assembling the P-DB-M specimen. Hence, the test was carried out with the preload of 155kN, which is also the preload required to prevent gap-opening prior to the floor and ceiling beams reaching the actual plastic moment capacities (without considering strain hardening effects).

Figure 4.57 presents the lateral load versus displacement curve and the moment versus interstorey drift curve (computed at the column centreline) under monotonic loading. When the lateral load increased to 55kN at the displacement of 10mm (+0.64% drift), a slip occurred between the plates at the AJ

connection with a slightly loud sound from the slot connection at the tip of the beams, and the lateral load decreased suddenly from 55kN to 38kN. When the lateral load reached approximately 50kN at the displacement of 12.6mm (+0.82% drift), although the gap-opening was invisible at the AJ connection, this load was considered to be the onset of the gap-opening. This is because the displacement of the interior part of the connection began to increase as shown in Figure 4.58, as did the tensile force in the SRBs as shown in Figure 4.60 (a). As the lateral load reached 78kN at the displacement of 20mm (+1.3% drift), the ceiling beam yielded just above the reinforcing plates. After the ceiling beam yielded, the gap-opening at the interior part of the connection widened faster and the tensile stress in the SRBs also increased. At the displacement of 22mm (+1.5% drift), one unloading-reloading cycle was applied to estimate the stiffness under reloading.

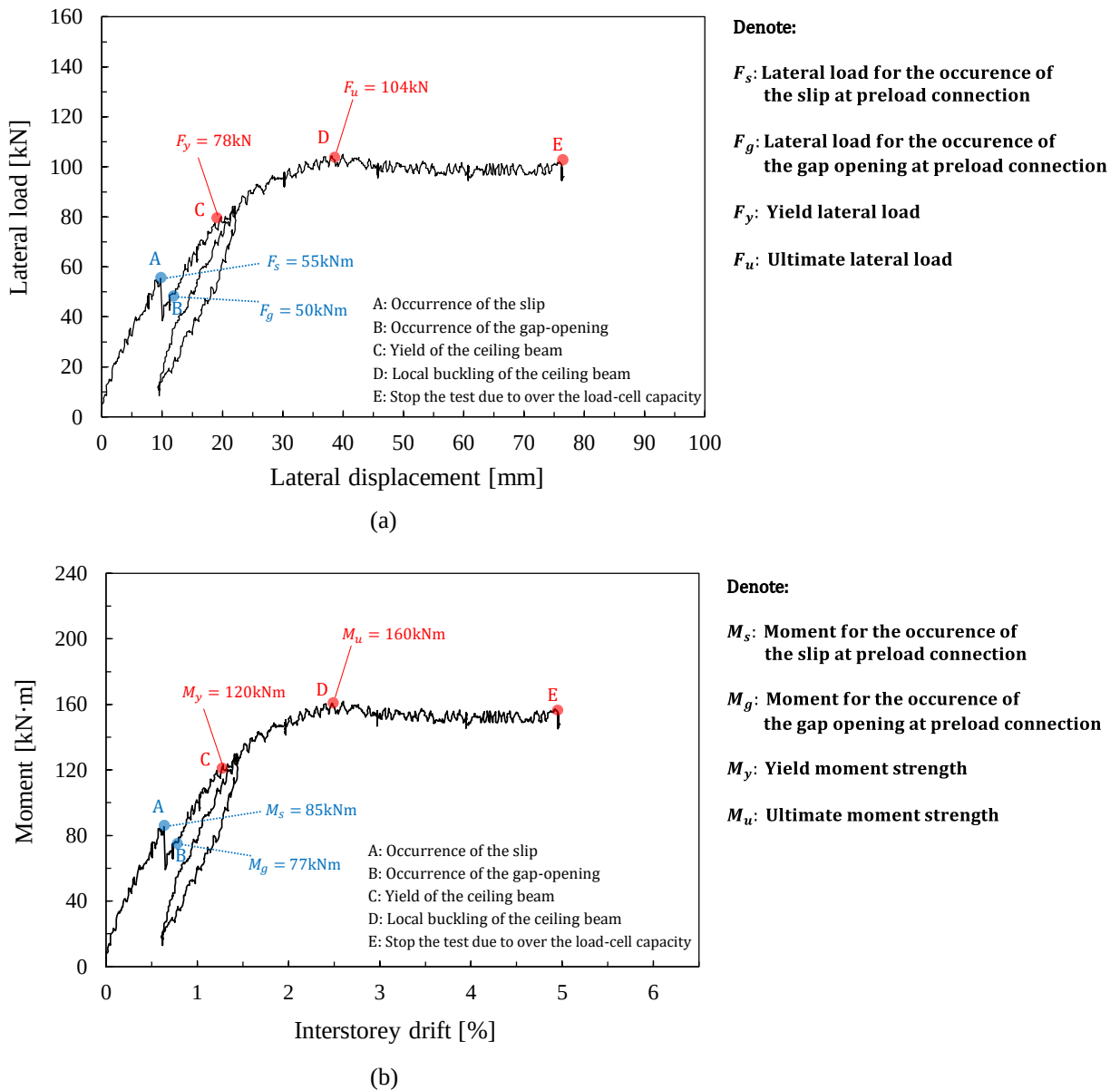


Figure 4.57: Monotonic response curves of P-DB-M: (a) Lateral load-displacement curve; (b) Moment-drift curve

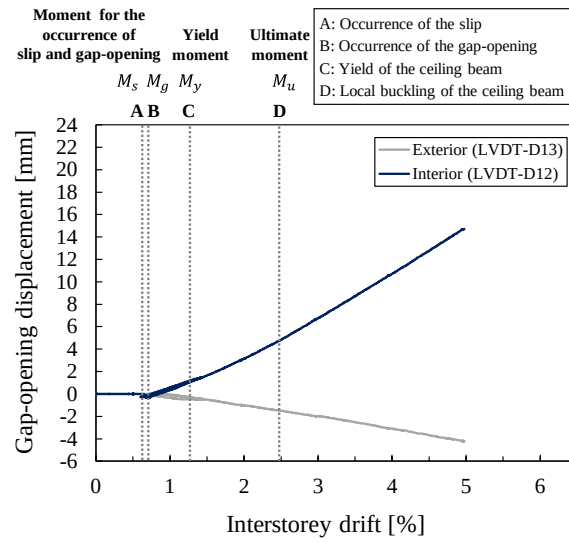


Figure 4.58: Displacement for the gap-opening at the AJ connection in P-DB-M

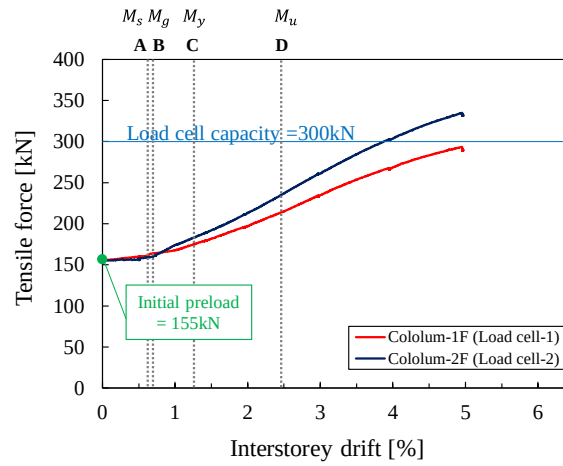
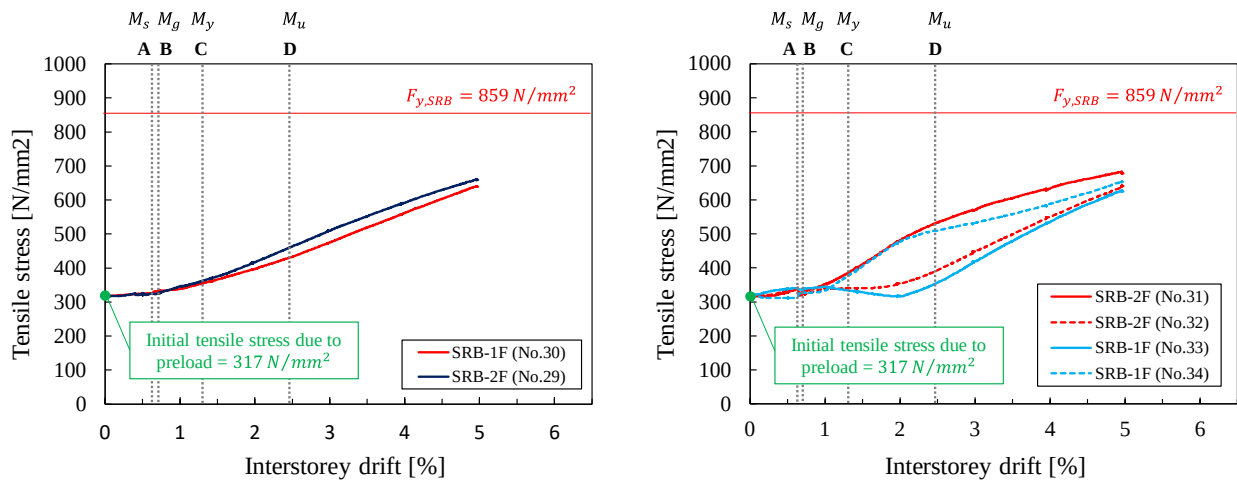


Figure 4.59: Tensile force variation for the SRBs in the columns measured by load cells in P-DB-M



(a) Away from the connection

(b) Near the connection

*The location of strain gauges is shown in Figure 4.45.

Figure 4.60: Tensile stress variation at the shank of SRBs measured by strain gauges in P-DB-M

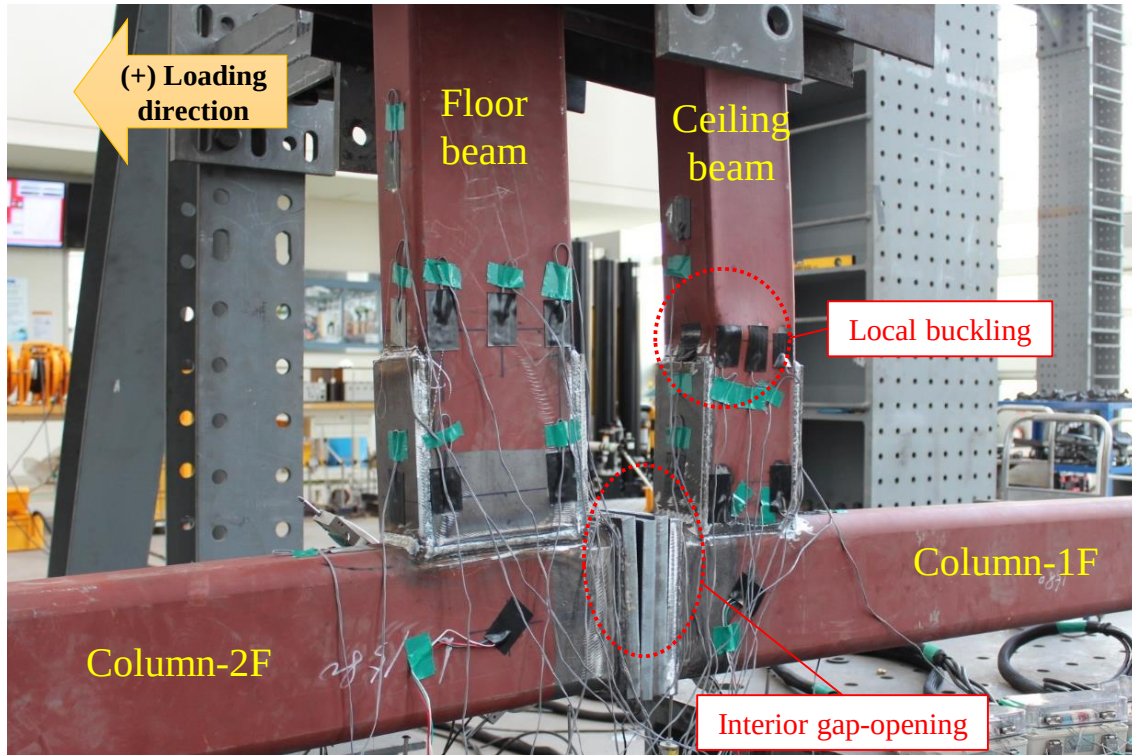


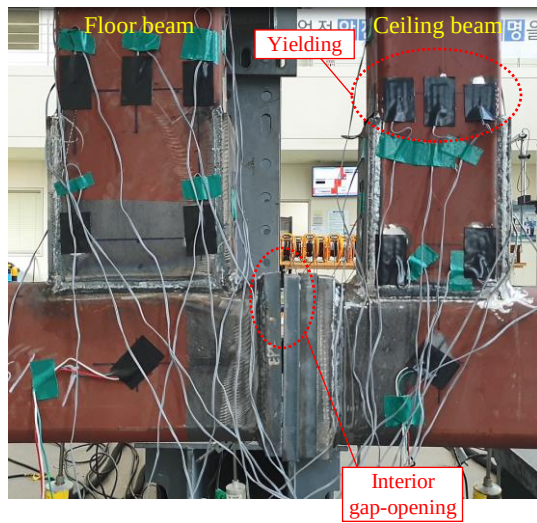
Figure 4.61: Failure mode of P-DB-M under monotonic load (drift 4.9%)

When the lateral load increased to 104kN at the displacement of 38mm (+ 2.47% drift) (see Figure 4.62 (a)), local buckling started to occur in the ceiling beam, triggering a more rapid increase of the gap-opening. Despite the local buckling of the ceiling beam, there was only a very slight strength reduction because the SRBs in the columns resisted the lateral load. The floor beam was not yielded prior to failure of the ceiling beam. The test was stopped at the displacement of 76mm (+4.9% drift) because the tensile force in the SRBs exceeded the capacity of load-cells (300kN). This implies that a tensile force of about 170kN was applied to the SRBs due to gap-opening in addition to the initial preload of 155kN (see Figure 4.59). Figure 4.61 and Figure 4.62 (b) present the final failure shape.

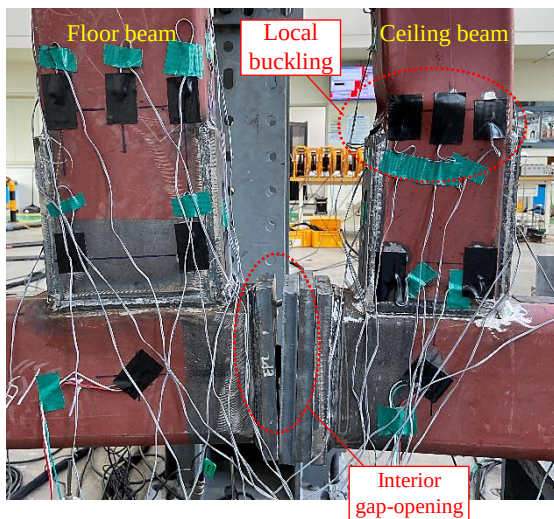
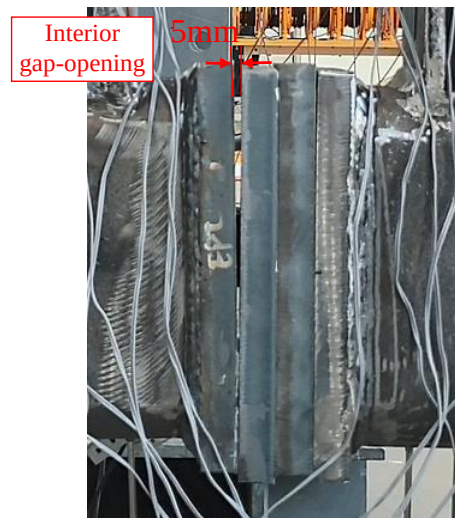
Figure 4.60 presents the stress variations in the SRBs as functions of the interstorey drift. The stress measured at a distance from the connection (see Figure 4.60 (a)) shows a similar tendency to the tensile load variation measured by the load cells in Figure 4.59, i.e. the tensile stress increase was due to the effect of gap-opening. Similarly, it is observed that the stress measured on the bolt shank near the connection (see Figure 4.60 (b)) was subjected to an additional tensile stress due to gap-opening and bending stress caused by a rotation of the coupler when bearing against the plates after the occurrence of slip at the connection. Although the bolt shank stress remained below the yield stress ($f_{y,SRB}$) until the test was finished, it was found that the bolt thread in column-2F at the connection was bent by the effect of gap-opening and the associated bending action, as shown in Figure 4.62 (d).

There was no visible damage to the coupler, and no signs of the coupler being loosened from the SRBs after finishing the test. As mentioned previously, the coupler was not instrumented because of the difficulty of attaching strain gauges. Nevertheless, given that its yield strength was much higher than the tensile strength of the SRBs (refer to Table 4.17), which was confirmed by the uniaxial tensile test of the coupler and SRBs as described in Section 4.2.2, it is likely that the coupler behaved within the elastic range, as desired.

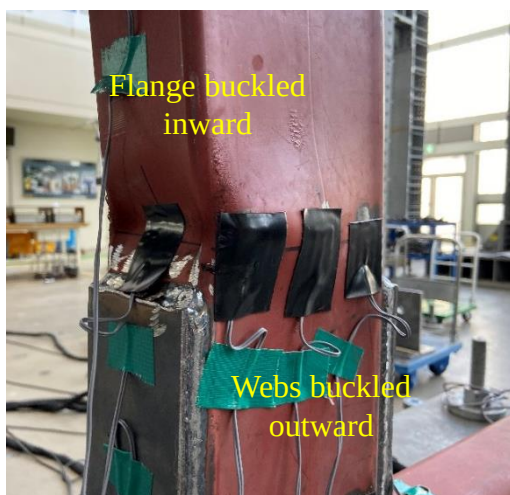
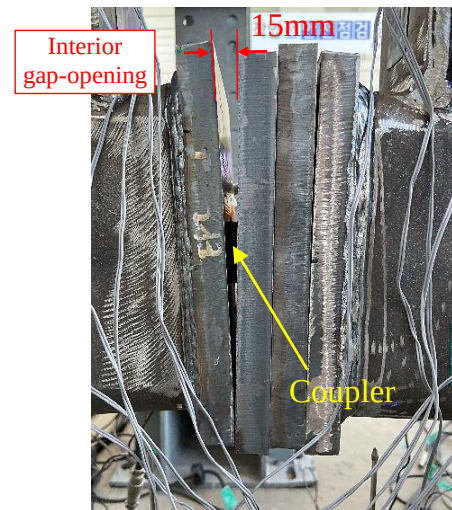
The columns and panel zones also remained in the elastic range throughout the tests, and there were no signs of cracks or fractures developing at the welded connections between the column and beam during the test.



(a) Deformed shape with the interior gap-opening at displacement 38mm (drift 2.47%) with $F_u = 104kN$



(b) Deformed shape with the interior gap-opening at displacement 76mm (drift 4.94%)



(c) Local buckling in the ceiling beam



(d) Bent thread of the SRB in column-2F at the AJ connection

Figure 4.62: Test observation of P-DB-M under monotonic load

2) P-SB-M

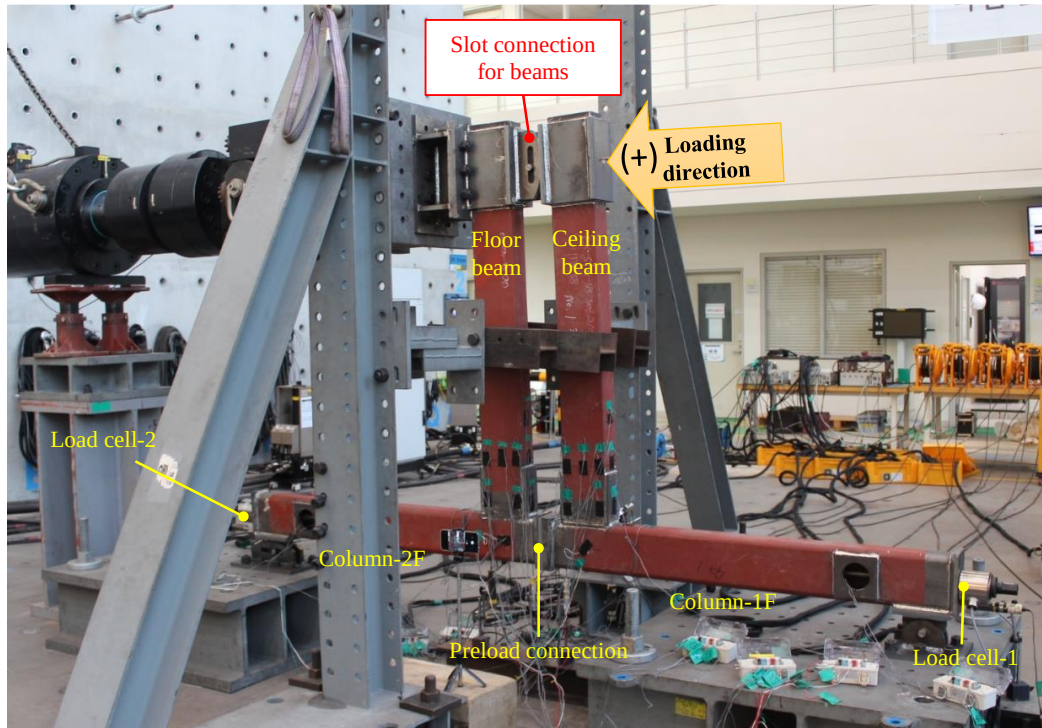


Figure 4.63: P-SB-M test assembly - before conducting the test

For the P-SB-M specimen, the demand preload of 145kN was applied to the AJ connection. Figure 4.64 presents the monotonic response curves of P-SB-M.

When the lateral load reached 68kN at the displacement of 10mm (+0.65% drift), a slip occurred between the plates at the AJ connection with a loud sound from the slot connection at the tip of the beams like for the P-DB-M specimen, and the lateral load decreased from 68kN to 60kN. When the lateral load reached approximately 73kN at the displacement of 11mm (+0.71% drift), the interior gap-opening began to increase at the connection as shown in Figure 4.65, as did the tensile force in the SRBs as shown in Figure 4.66. As the lateral load increased to 101kN at the displacement of 18mm (+1.2% drift), the ceiling beam yielded just above the reinforcing plates. After the ceiling beam yielded, the gap-opening at the interior part of the connection widened faster and the tensile stress in the SRB also increased. At the displacement of 31mm (+2.0% drift), one unloading-reloading cycle was applied to estimate the stiffness under reloading. When the lateral load reached 140kN at the displacement of 34mm (+2.2% drift), local buckling started to occur in the ceiling beam, triggering a more rapid increase of the gap-opening and the tensile stress in the SRB. The strength degradation began after the local buckling of the ceiling beam, but it was not severe in the SRBs resisting the lateral load. The floor beam yielded very slightly prior to failure of the ceiling beam. The test was stopped at the displacement of 93mm (+6.0% drift) because the tensile force in the SRBs almost reached the capacity of the load-cell (300kN) This implies that a tensile force of about 155kN was applied to the SRBs due to gap-opening in addition to the initial preload of 145kN. Figure 4.68 and Figure 4.69 (b) present the final failure shape.

Figure 4.67 presents the stress variations in the SRBs as functions of the interstorey drift. The stress measured at a distance from the connection (see Figure 4.67 (a)) shows a similar tendency to the tensile load variation measured by the load cells in Figure 4.66, i.e. the tensile stress increase was due to the effect of gap-opening. Similarly, it is observed that the stress measured on the bolt shank near the connection (see Figure 4.67 (b)) was subjected to an additional tensile stress due to gap-opening and the bending stress caused by a rotation of coupler when bearing against the plates after the occurrence of slip at the connection. The SRB within column-2F where the gap-opening occurred was subjected to more bending compared to the SRB within column-1F. Although the bolt shank stress remained below the yield stress ($f_{y,SRB}$) until the test was finished, it was found that the bolt thread in column-2F at the connection was bent slightly by the effect of gap-opening and the associated bending action, as shown in Figure 4.69 (d).

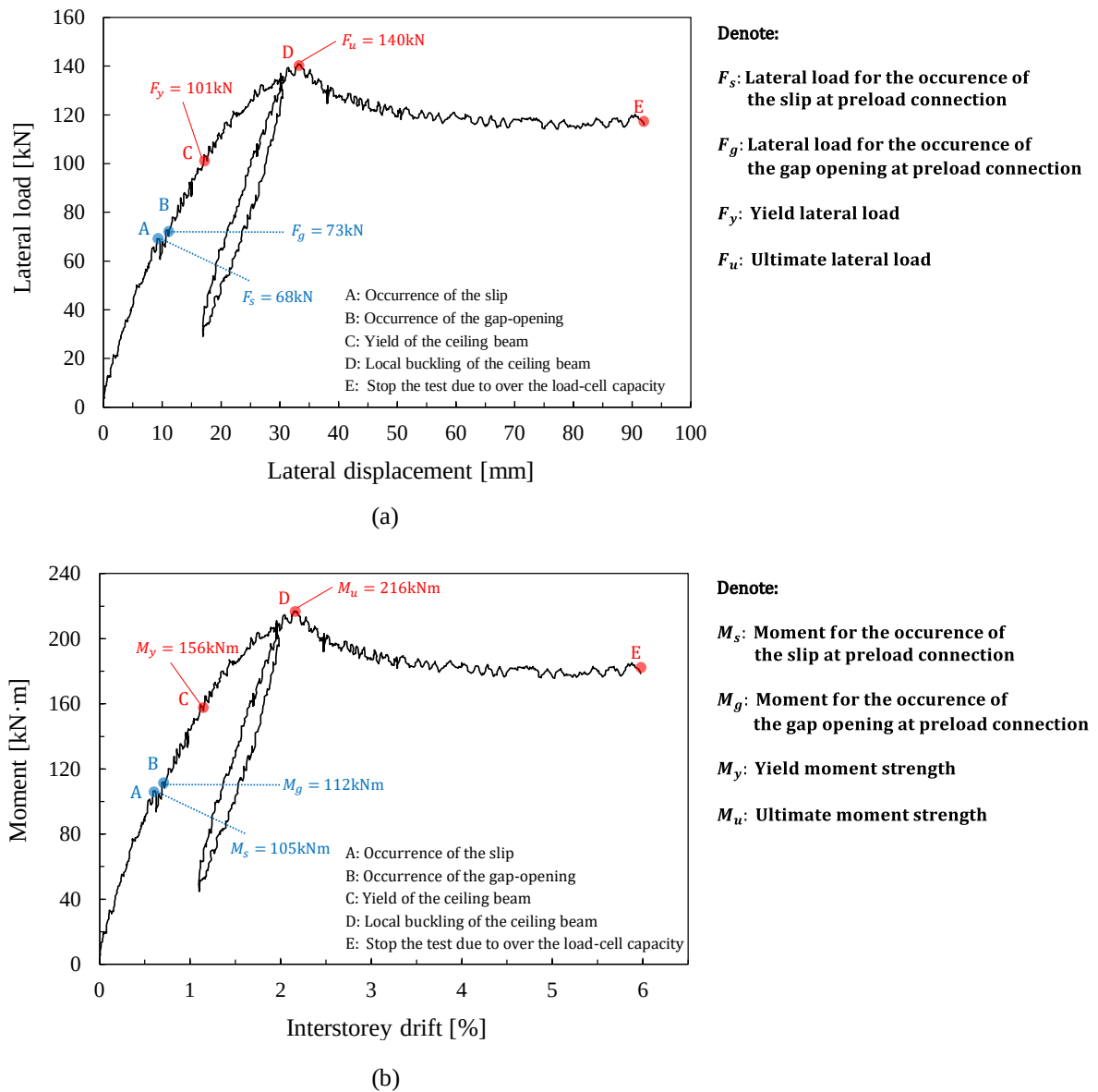


Figure 4.64: Monotonic response curves of P-SB-M: (a) Lateral load-displacement curve; (b) Moment-drift curve

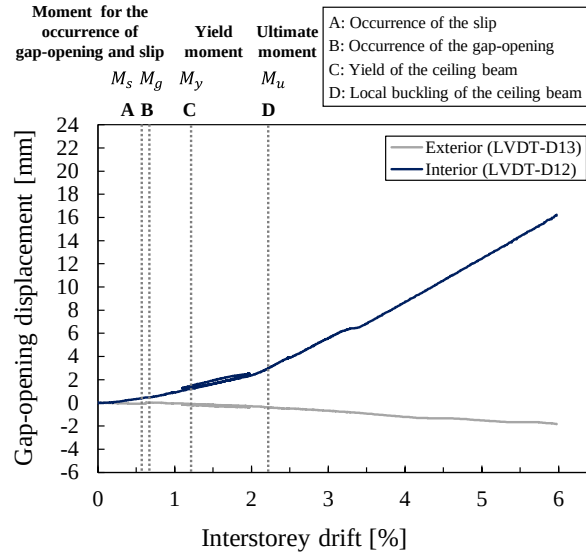


Figure 4.65: Displacement for the gap-opening at the AJ connection in P-SB-M

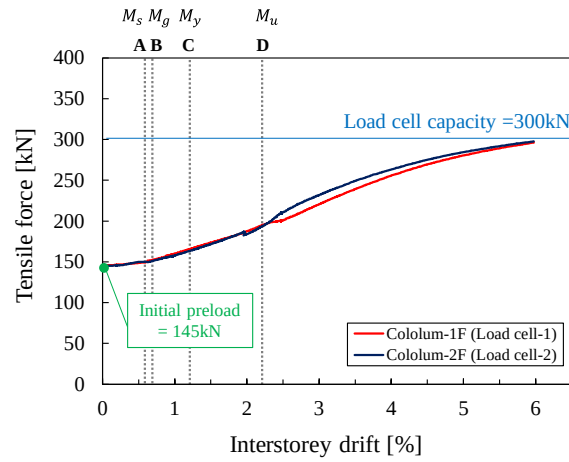
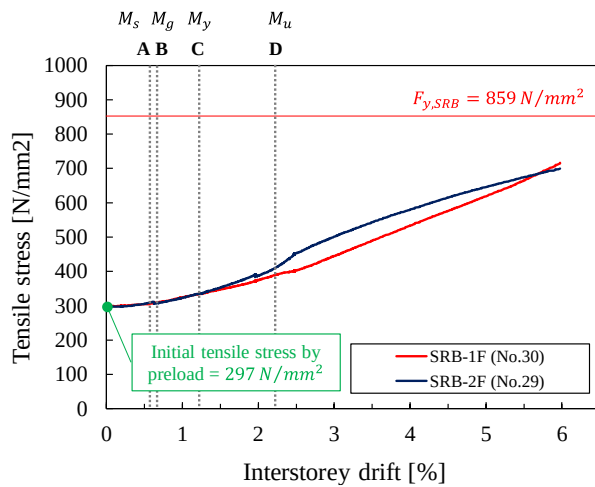
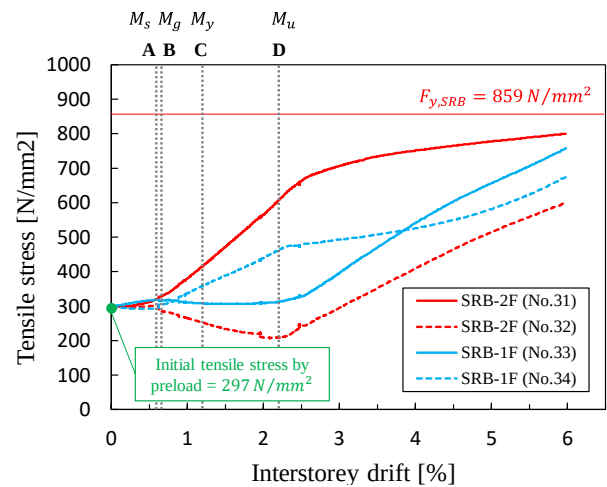


Figure 4.66: Tensile force variation for the SRBs in the columns measured by load cells in P-SB-M



(a) Away from the connection



(b) Near the connection

*The location of strain gauges is shown in Figure 4.45.

Figure 4.67: Tensile stress variation at the shank of SRBs measured by strain gauges in P-SB-M

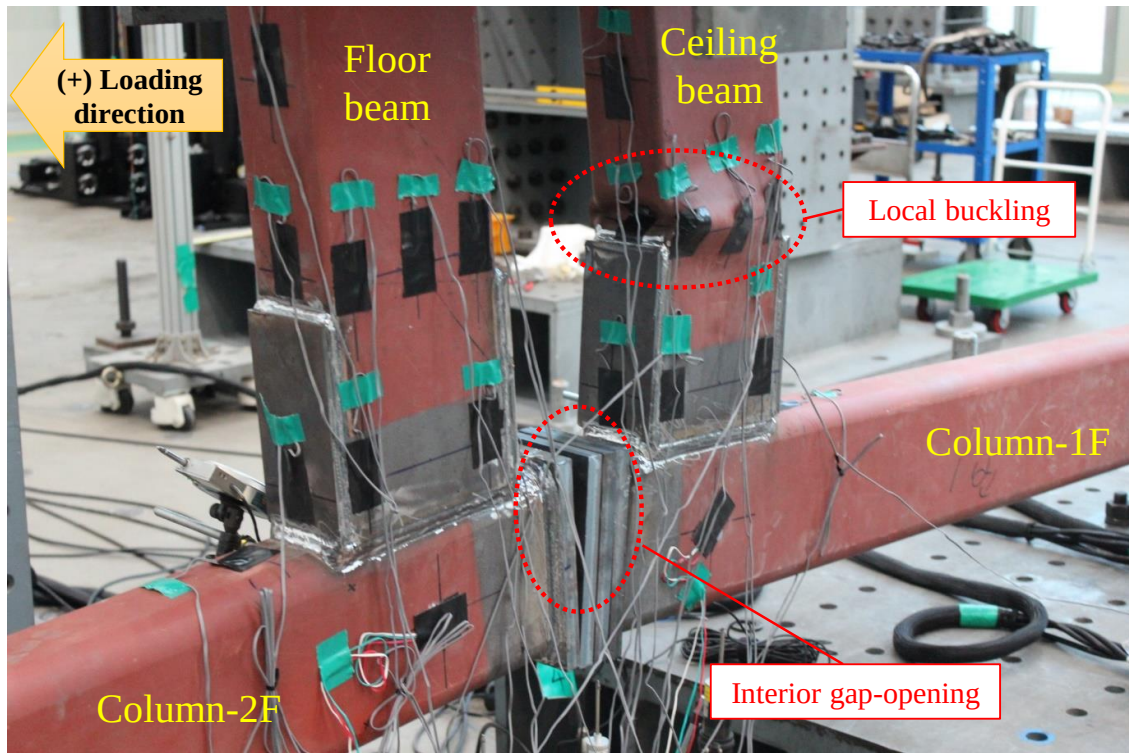
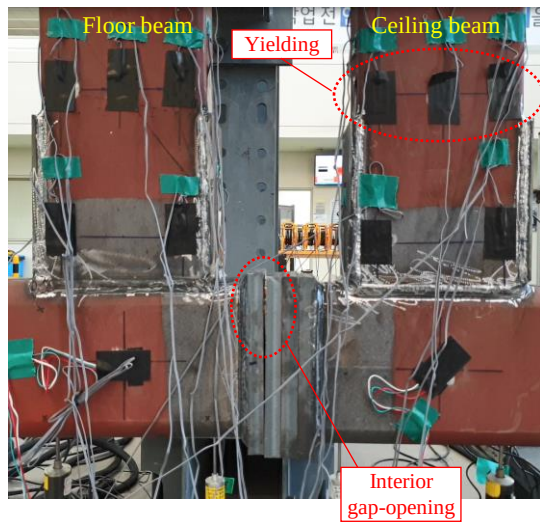


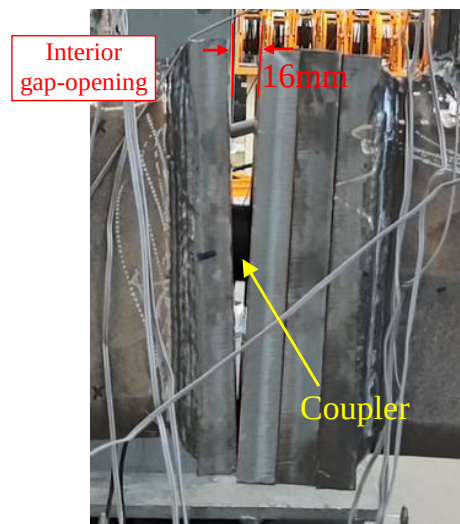
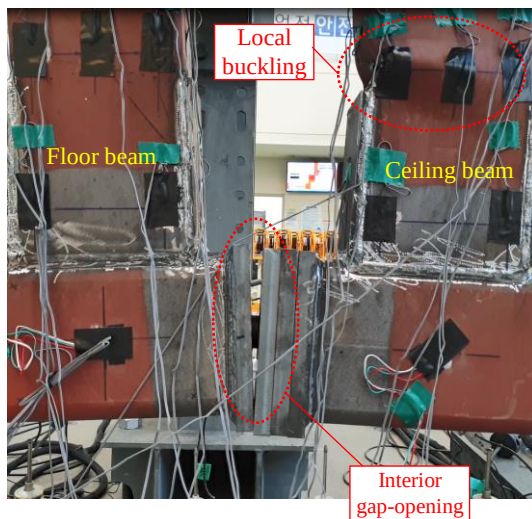
Figure 4.68: Failure mode of P-SB-M under monotonic load

There was no visible damage to the coupler, and no signs of the coupler being loosened from the SRBs after finishing the test. As mentioned previously, the coupler was not instrumented because of the difficulty of attaching strain gauges. Nevertheless, given that its yield strength was much higher than the tensile strength of the SRBs (refer to Table 4.17), which was confirmed by the uniaxial tensile test of the coupler and SRBs as described in Section 4.2.2, it is likely that the coupler behaved within the elastic range, as desired.

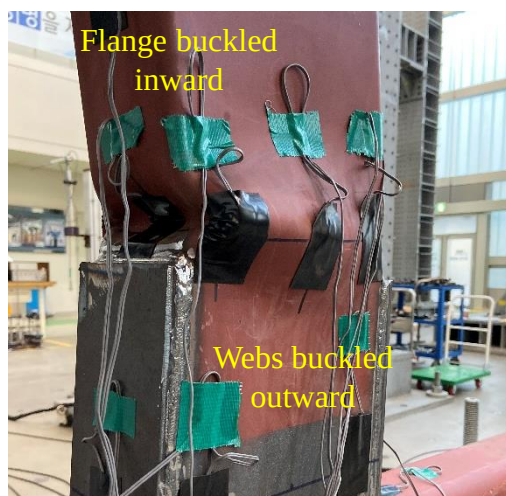
The columns and panel zones also remained in the elastic range throughout the tests, and there were no signs of cracks or fractures developing at the welded connections between the column and beam during the test.



(a) Deformed shape with the interior gap-opening at displacement 34mm (drift 2.21%) with $F_u = 140kN$



(b) Deformed shape with the interior gap-opening at displacement 92mm (drift 5.97%)



(c) Local buckling in the ceiling beam



(d) Bent thread of the SRB in column-2F at the AJ connection

Figure 4.69: Test observation of P-SB-M under monotonic load

4.4.9.2. Cyclic Test

1) P-DB-Q

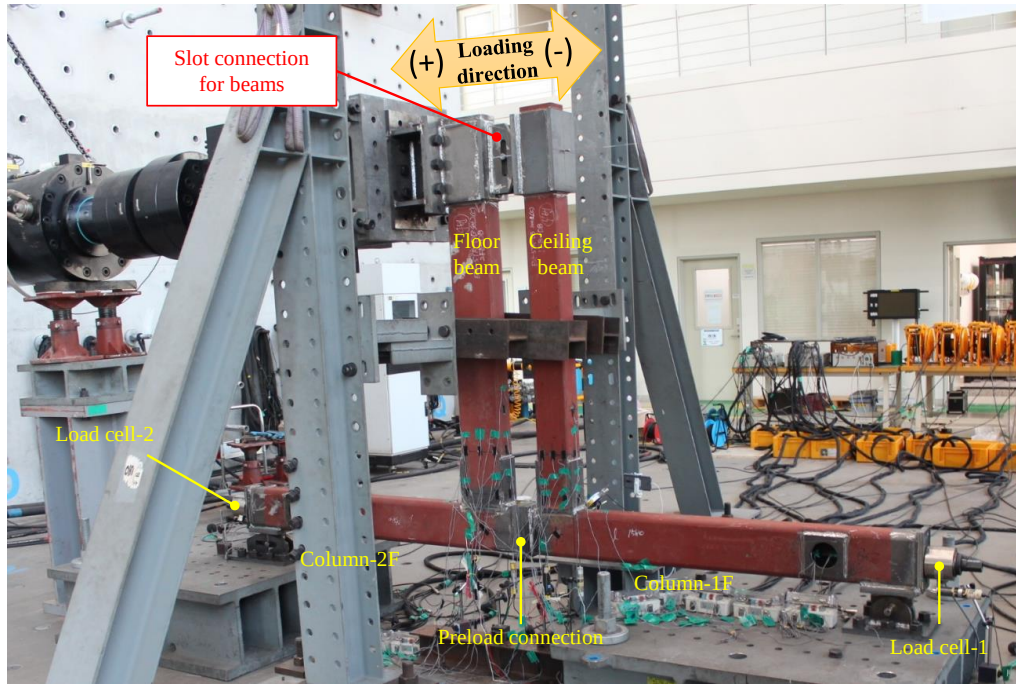


Figure 4.70: P-DB-Q test assembly - before conducting the test

For the P-DB-Q specimen, a preload of 185kN was applied to the connection. The hysteretic response under quasi-static cyclic loading is shown in Figure 4.71. Corresponding to the peak drift in each set of cycles, the displacements measured for the gap-opening at the AJ connection, the tensile force variation measured by the load cells, and the stress variation measured on the SRBs are indicated in Figure 4.72, Figure 4.73, and Figure 4.74, respectively.

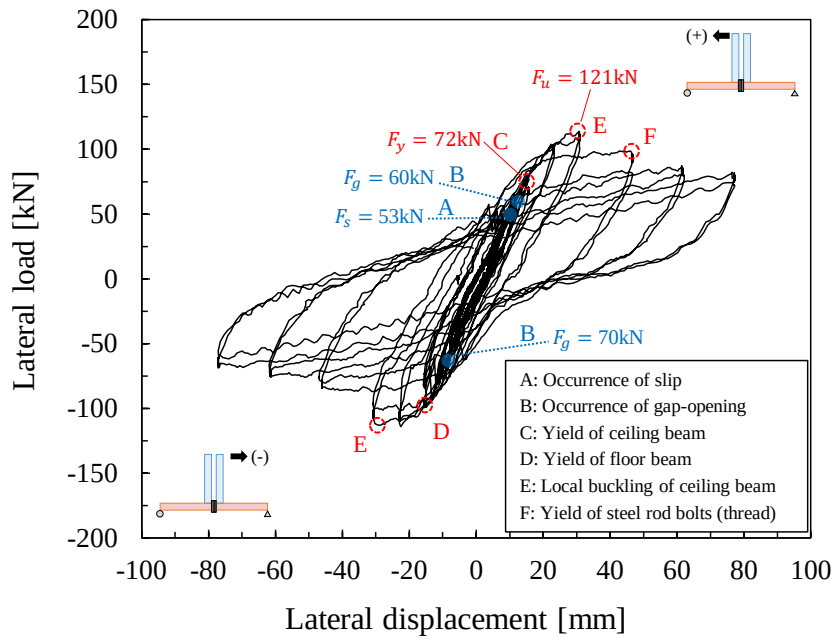
As the lateral load reached 53kN at the displacement of 10mm (+0.65% drift), a slip occurred between the plates at the AJ connection. Then, the interior gap-opening occurred when the lateral load reached approximately 60kN at the displacement of 11.8mm (+0.8% drift) in the positive loading direction. In the negative loading direction, the exterior gap-opening occurred at the connection when the lateral load was around 70kN at the displacement of -10.4mm (-0.67% drift). As the lateral load reached 72kN at the displacement of 15.4mm (+1.0% drift), the ceiling beam yielded just above the reinforcing plates, and then both the interior and exterior gap-opening widened faster in the positive and negative loading directions, respectively, as shown in Figure 4.72. The tensile force of the SRBs within column-1F and column-2F also increased, as presented in Figure 4.73. The floor beam yielded at the displacement of -17.3mm (-1.1% drift) corresponding to the lateral load of 99kN. At the displacement of 30.7mm (+2.0% drift) and lateral load of 121kN, the ceiling beam failed by local buckling, initiating the strength degradation. Since the SRBs within the columns resisted the lateral load, it prevented the brittle failure. After the local buckling of the ceiling beam, the gap-opening at the connection and the tensile force in

the SRBs increased rapidly. The test was stopped after the loading cycle for the drift of $\pm 5\%$. The floor beam yielded, but did not locally buckle in the course of the test. Figure 4.75 and Figure 4.76 present the final failure shape at the drift of $\pm 5\%$. Depending on the loading direction, the connection experienced interior gap-opening or exterior gap-opening, as shown in Figure 4.75 and Figure 4.76. A summary of the test results is presented in Table 4.44.

Figure 4.74 presents the stress variation of the SRBs within the columns. The stress measured at a distance from the connection (see Figure 4.74 (a)) represents the change in tensile force in the SRBs caused by gap-opening at the connection, which proved to have a similar tendency to the tensile force variation measured by the load cells (see Figure 4.73), as would be expected. On the other hand, it was observed that the bolt shanks near the connection (see Figure 4.74 (b)) were subjected to a change in tensile force due to the gap-opening as well as bending imposed by the coupler rotating due to bearing against the plates after the occurrence of slip at the connection. As a result, the shanks of the SRBs within the columns yielded or reached stresses near the actual yield stress ($f_{y,SRB}$). Based on this variation of stress in the shanks of the SRBs, it was predicted that the bolt threads of the SRBs (the cross-sectional area of which is approximately 70% of that of the bolt shank) in column-1F and column-2F yielded between the drifts of 2% and 3% in both the positive and negative loading directions. Figure 4.77 shows the bent thread of the SRBs due to the effect of gap-opening at the AJ connection.

There was no visible damage to the coupler, and no signs of the coupler being loosened from the SRBs after finishing the test. As mentioned previously, the coupler was not instrumented because of the difficulty of attaching strain gauges. Nevertheless, given that its yield strength was much higher than the tensile strength of the SRBs (refer to Table 4.17), which was confirmed by the uniaxial tensile test of the coupler and SRBs as described in Section 4.2.2, it is likely that the coupler behaved within the elastic range, as desired.

The columns and panel zones also remained in the elastic range throughout the tests, and there were no signs of cracks or fractures developing at the welded connections between the column and beam during the test.



(a)

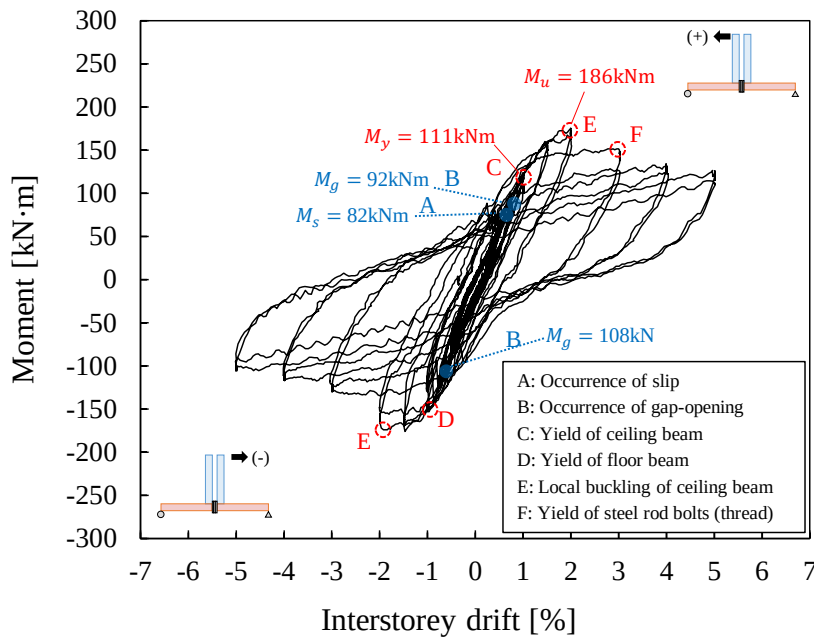
Denote:

F_s : Lateral load for the occurrence of the slip at preload connection

F_g : Lateral load for the occurrence of the gap opening at preload connection

F_y : Yield lateral load

F_u : Ultimate lateral load



(b)

Denote:

M_s : Moment for the occurrence of the slip at preload connection

M_g : Moment for the occurrence of the gap opening at preload connection

M_y : Yield moment strength

M_u : Ultimate moment strength

Figure 4.71: Hysteretic curves of P-DB-Q: (a) Lateral load-displacement curve; (b) Moment-drift curve

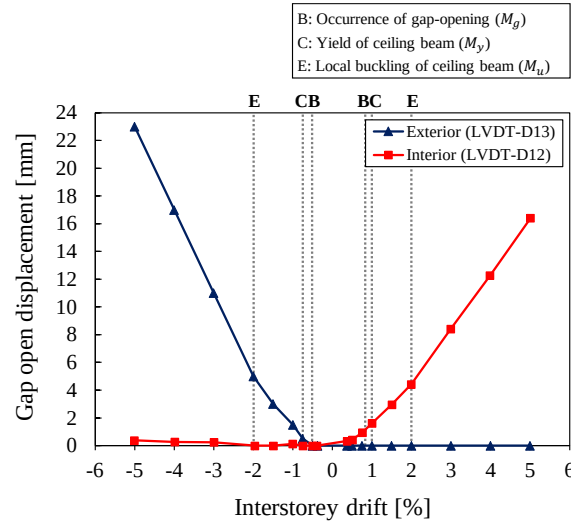


Figure 4.72: Displacement for the gap-opening at the AJ connection in P-DB-Q (corresponding to the peak drift in each set of cycles)

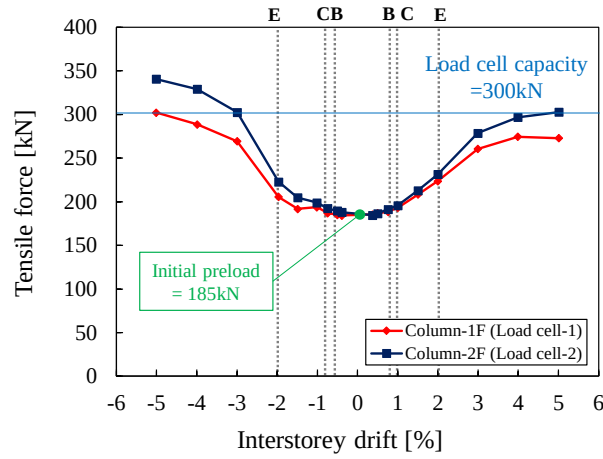
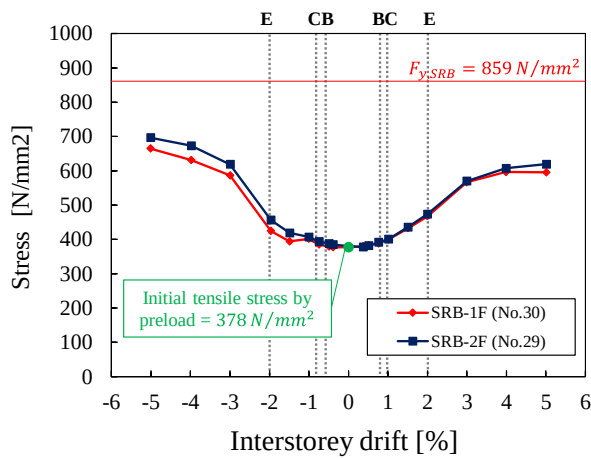
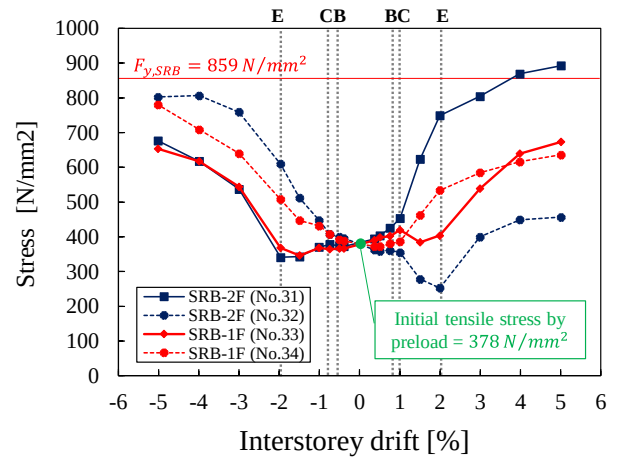


Figure 4.73: Tensile force variation for the SRBs in the columns measured by load cells in P-DB-Q (corresponding to the peak drift in each set of cycles)



(a) Away from the connection



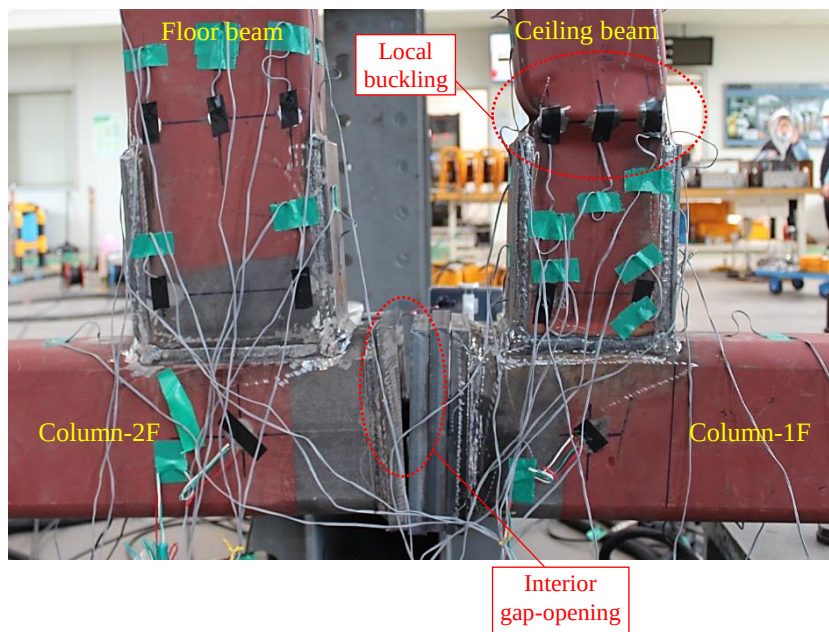
(b) Close to the connection

*The location of strain gauges is shown in Figure 4.45.

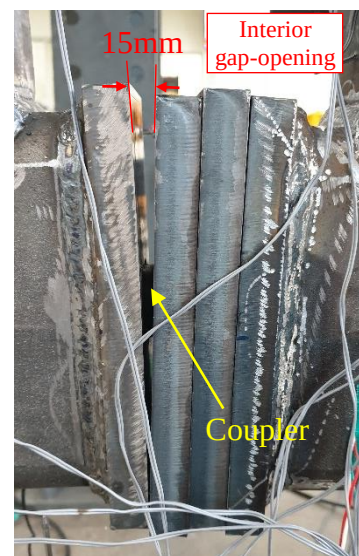
Figure 4.74: Tensile stress variation at the shank of SRBs measured by strain gauges in P-DB-Q (corresponding to the peak drift in each set of cycles)



(a) Deformed shape of P-DB-Q

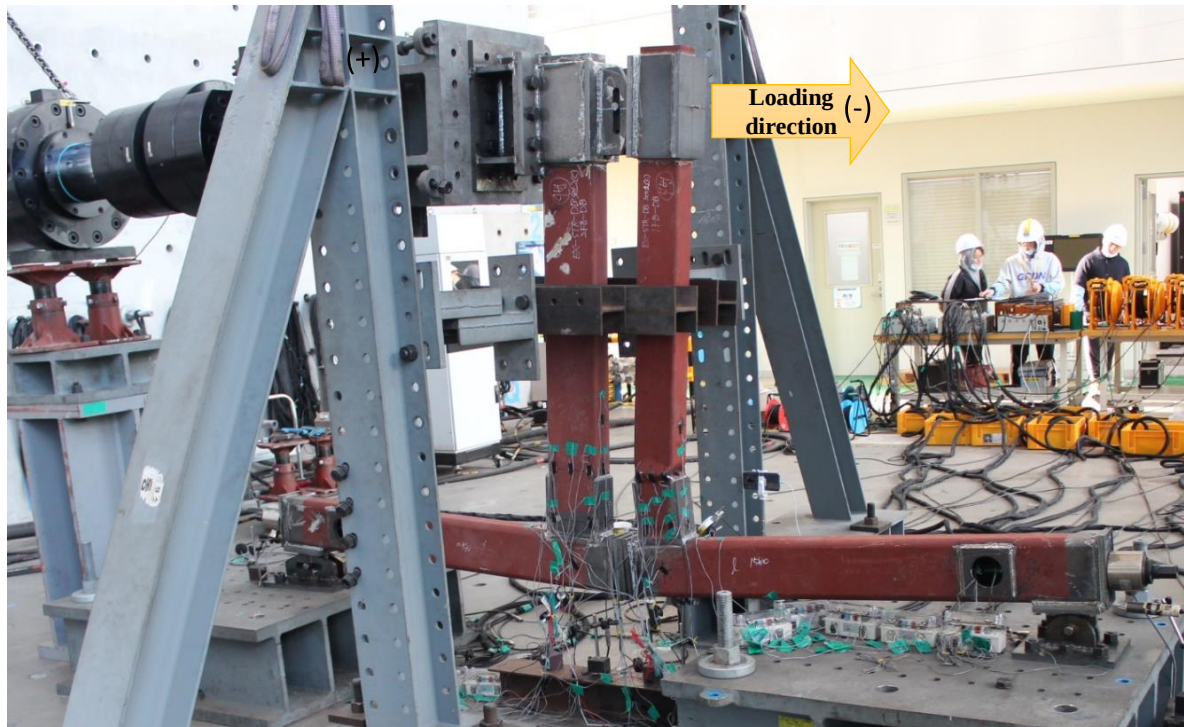


(b) Failure mode: the interior gap-opening and the local buckling in the ceiling beam

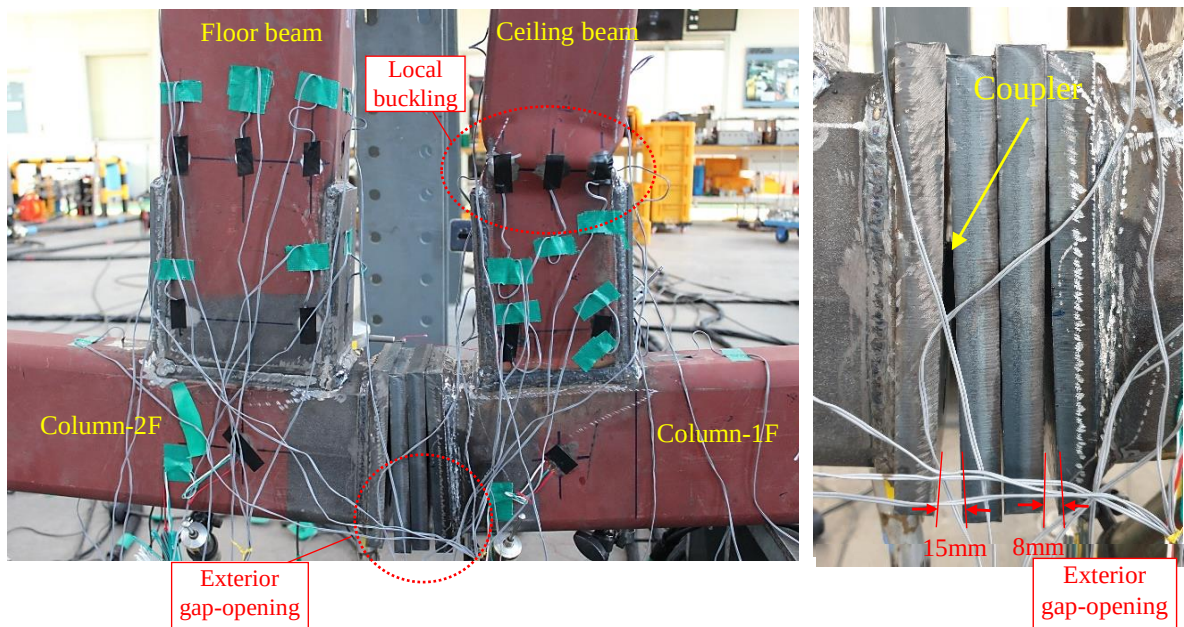


(c) Location of the interior gap-opening

Figure 4.75: Failure mode of P-DB-Q under quasi-static cyclic loading at the drift of 5% in (+) loading direction



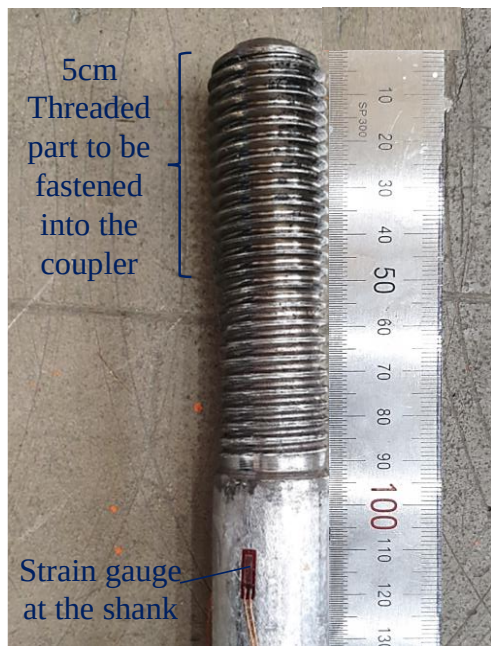
(a) Deformed shape of P-DB-Q



(b) Failure mode: the exterior gap-opening and the local buckling in the ceiling beam

(c) Location of the exterior gap-opening

Figure 4.76: Failure mode of P-DB-Q under quasi-static cyclic loading at the drift of 5% in (-) loading direction



(a) Bolt thread in column-2F



(b) Bolt thread in column-1F

Figure 4.77: Bent SRBs at the AJ connection in P-SB-Q

2) P-SB-Q

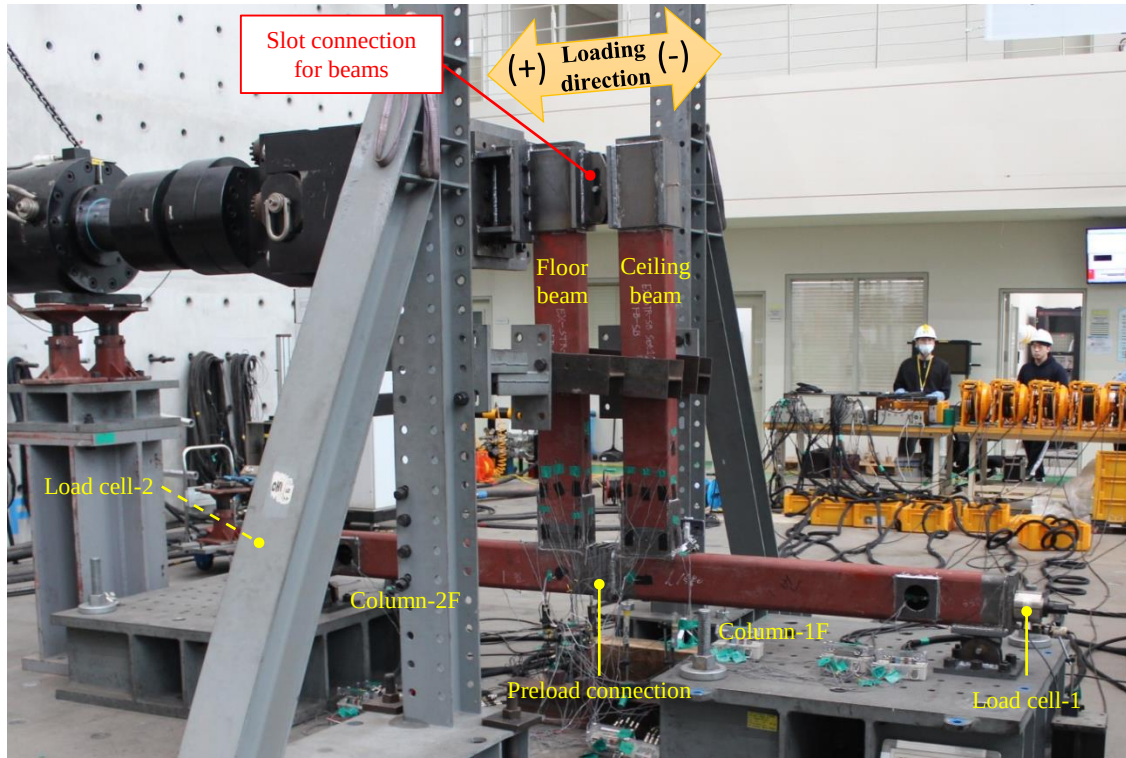


Figure 4.78: P-SB-Q test assembly – before conducting the test

For the P-SB-Q specimen, the demand preload of 145kN was applied to the AJ connection. The hysteretic response under quasi-static cyclic loading is shown in Figure 4.79. Corresponding to the peak drift in each set of cycles, the displacements measured for the gap-opening at the AJ connection, and the tensile force variation measured by the load cells are indicated in Figure 4.80 and Figure 4.81, respectively. Unfortunately, all data measured by strain gauges attached to the test specimen was lost due to a technical error that occurred during the test. Therefore, the stress variation of the beams, SRBs, columns, and panel zones could not be known.

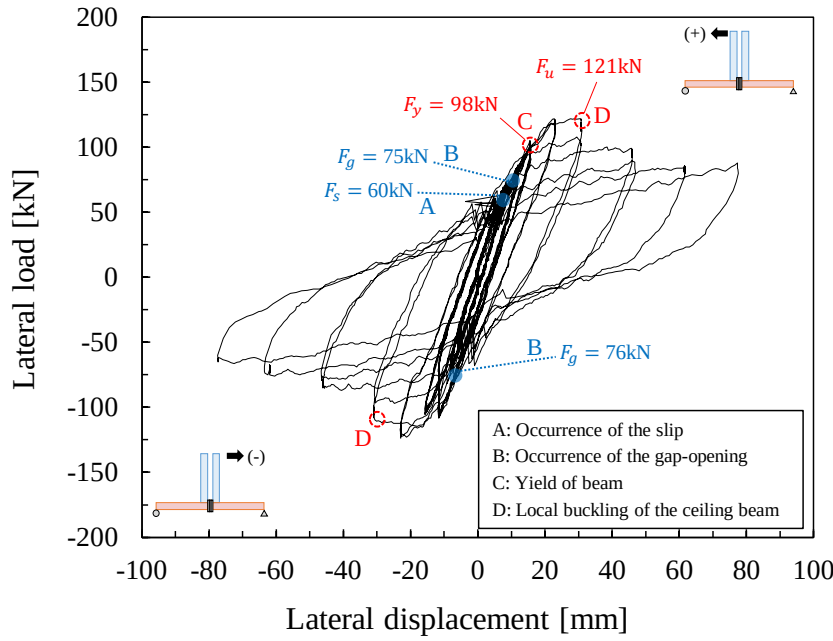
As the lateral load reached 60kN at the displacement of 8mm (+0.52% drift), slip occurred between the plates at the AJ connection. Then, an interior gap-opening occurred when the lateral load reached approximately 75kN at the displacement of 10.5mm (+0.68% drift) in the positive loading direction. In the negative loading direction, the exterior gap-opening occurred at the connection when the lateral load was around 76kN at the displacement of -6.5mm (-0.42% drift). After the displacement of ± 15.4 mm ($\pm 1.0\%$ drift), the interior and exterior gap-openings widened at a faster rate in the positive and negative loading directions (see Figure 4.80) and the tensile force in the SRBs within the columns increased (see Figure 4.81). These results suggested that ceiling beam yielded at the drift of $\pm 1.0\%$. At the displacement of 30.7mm (+2.0% drift) and lateral load of 121kN, the ceiling beam failed by local buckling, triggering a strength degradation and rapid increase in the gap-opening at the connection and the tensile force of the SRBs. The test was stopped after the loading cycle for the drift of $\pm 5\%$. Based

on the monotonic test results for P-SB-M, which had a similar failure mechanism to P-SB-Q, it is presumed that the floor beam yielded slightly.

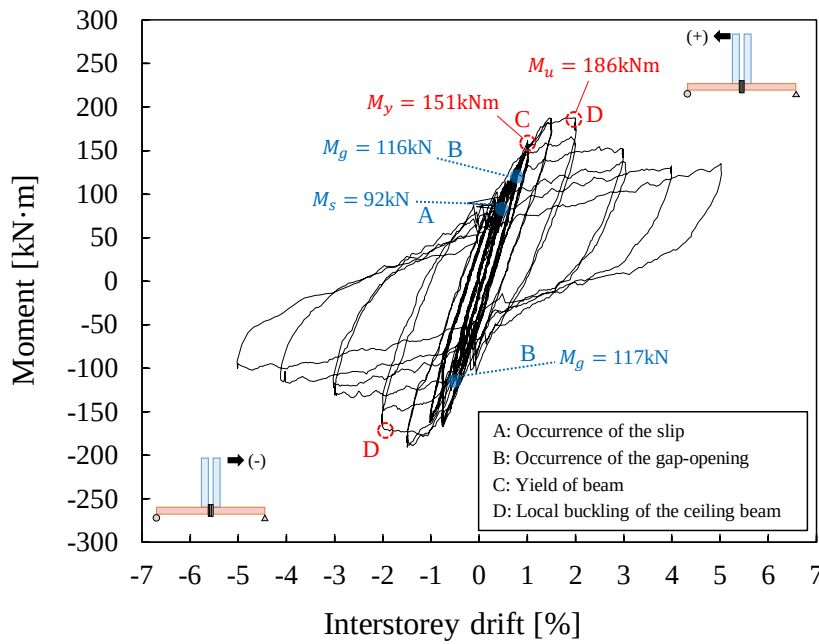
Figure 4.82 and Figure 4.83 present the final failure shape at the drift of $\pm 3.0\%$. Depending on the loading direction, the connection experienced an interior or exterior gap-opening. The failure mechanism was the same as that of P-DB-Q. Figure 4.84 shows the bent thread of the SRBs in column-1F and column-2F, indicating the SRBs were subjected to the induced tensile force as well as bending caused by gap-opening and slip at the connection, similar to the P-DB-Q specimen. No cracks or fractures were observed at the welded connection between the column and beam in the course of the test. A summary of the test results is presented in Table 4.44.

There was no visible damage to the coupler, and no signs of the coupler being loosened from the SRBs after finishing the test. As mentioned previously, the coupler was not instrumented because of the difficulty of attaching strain gauges. Nevertheless, given that its yield strength was much higher than the tensile strength of the SRBs (refer to Table 4.17), which was confirmed by the uniaxial tensile test of the coupler and SRBs in described in Section 4.2.2, it is likely that the coupler behaved within the elastic range, as desired.

The columns and panel zones also remained in the elastic range throughout the tests, and there were no signs of cracks or fractures developing at the welded connections between the column and beam during the test.



(a)



(b)

Figure 4.79: Hysteretic curves of P-SB-Q: (a) Lateral load-displacement curve; (b) Moment-drift curve

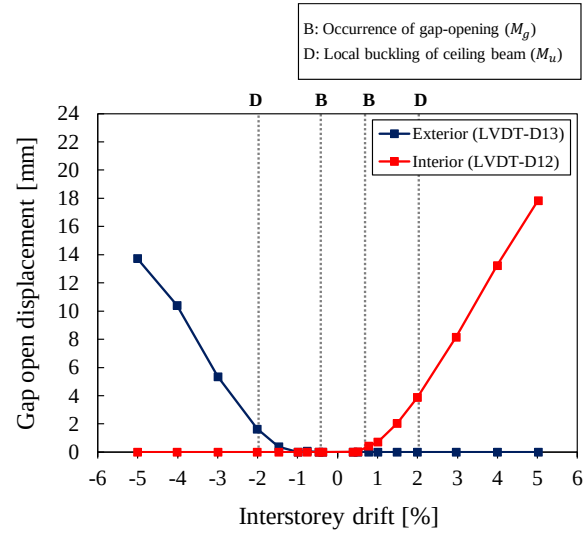


Figure 4.80: Displacement for the gap-opening at the AJ connection in P-SB-Q (corresponding to the peak drift in each set of cycles)

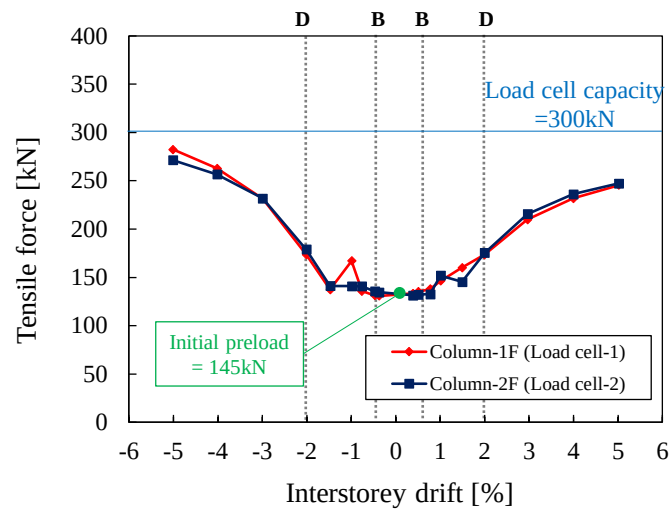
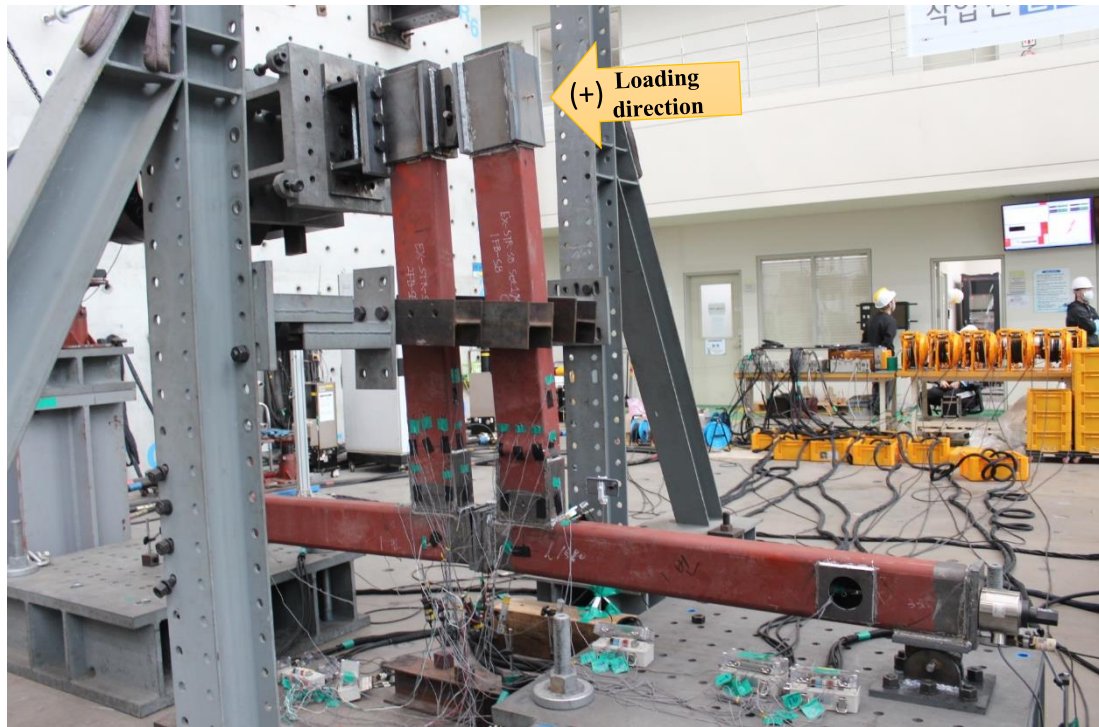
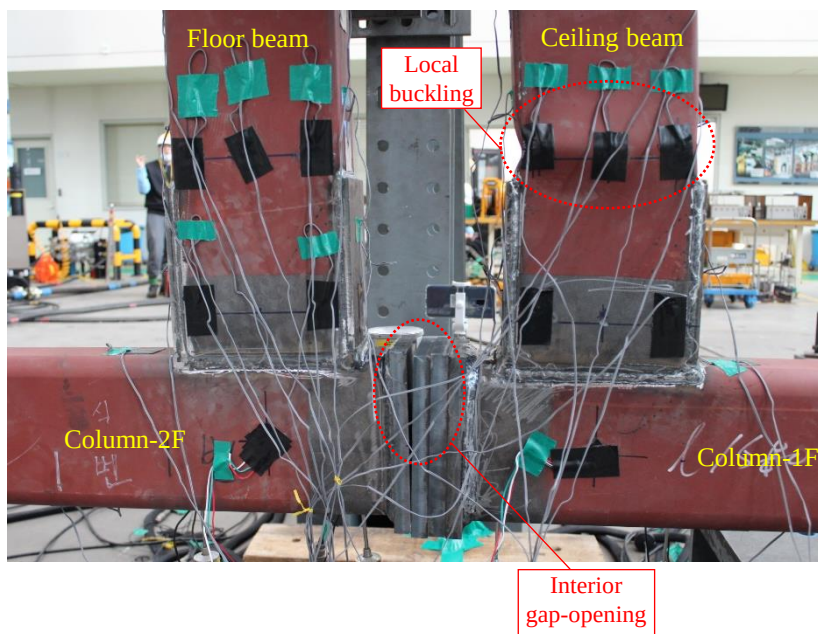


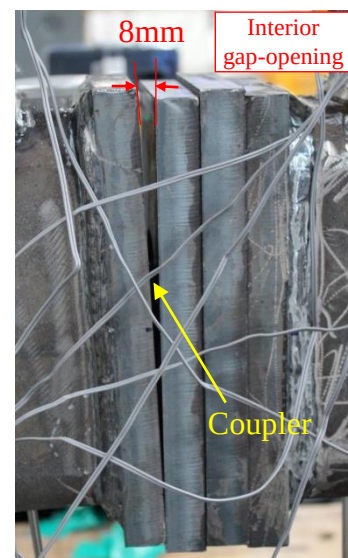
Figure 4.81: Tensile force variation for the SRBs in the columns measured by load cells in P-SB-Q (corresponding to the peak drift in each set of cycles)



(a) Deformed shape of P-DB-Q

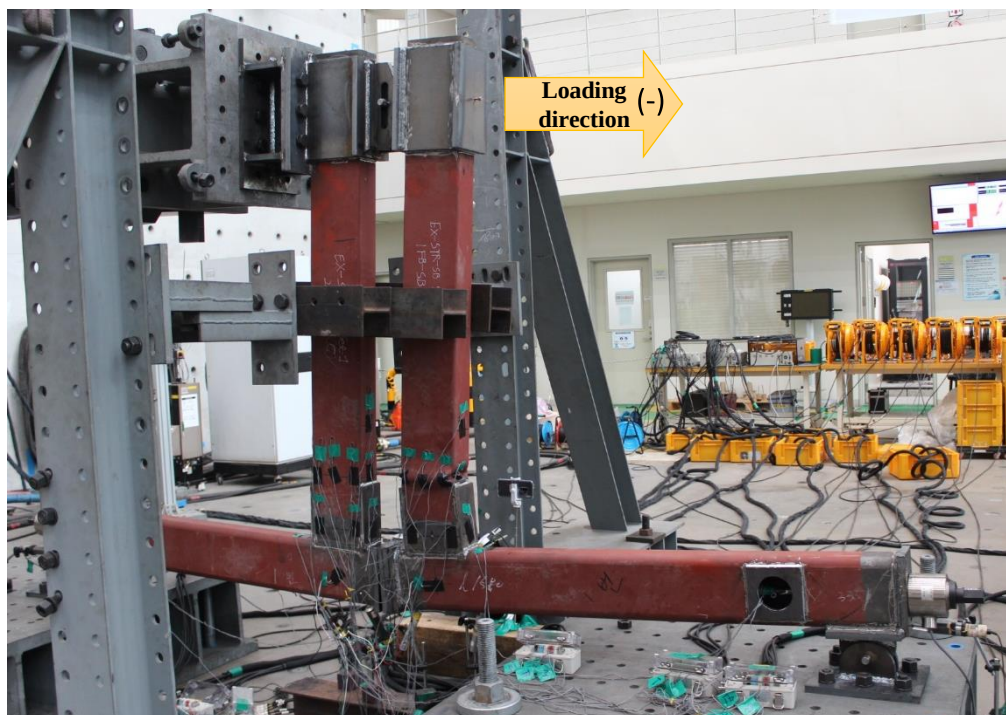


(b) Failure mode: the interior gap-opening and the local buckling in the ceiling beam

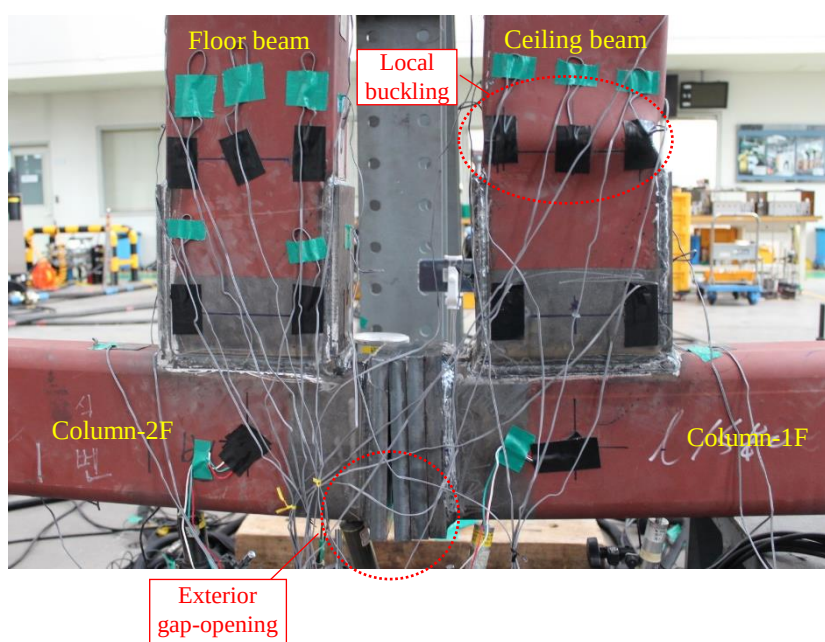


(c) Location of the interior gap-opening

Figure 4.82: Failure mode of P-SB-Q under quasi-static cyclic loading at the drift of 3% in (+) loading direction



(a) Deformed shape of P-SB-Q

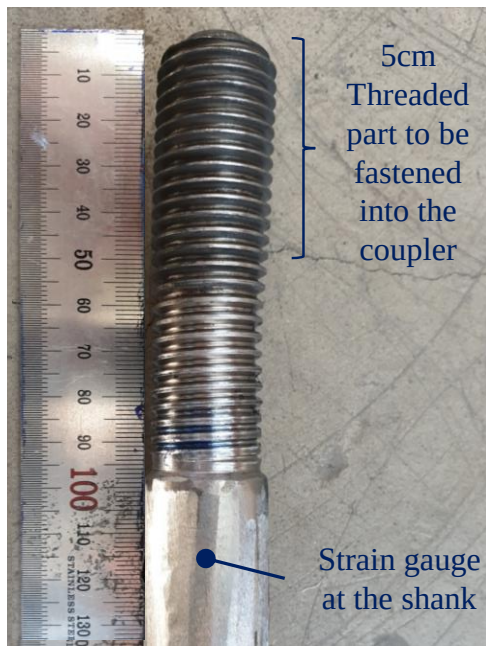


(b) Failure mode: the exterior gap-opening and the local buckling in the ceiling beam

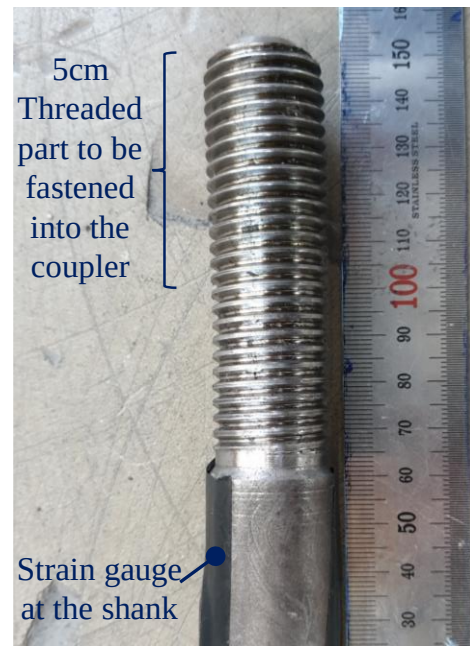


(c) Location of the exterior gap-opening

Figure 4.83: Failure mode of P-SB-Q under quasi-static cyclic loading at the drift of 3% in (-) loading direction



(a) Bolt thread in column-2F



(b) Bolt thread in column-1F

Figure 4.84: Bent SRBs at the AJ connection in P-SB-Q

3) W-SB-Q

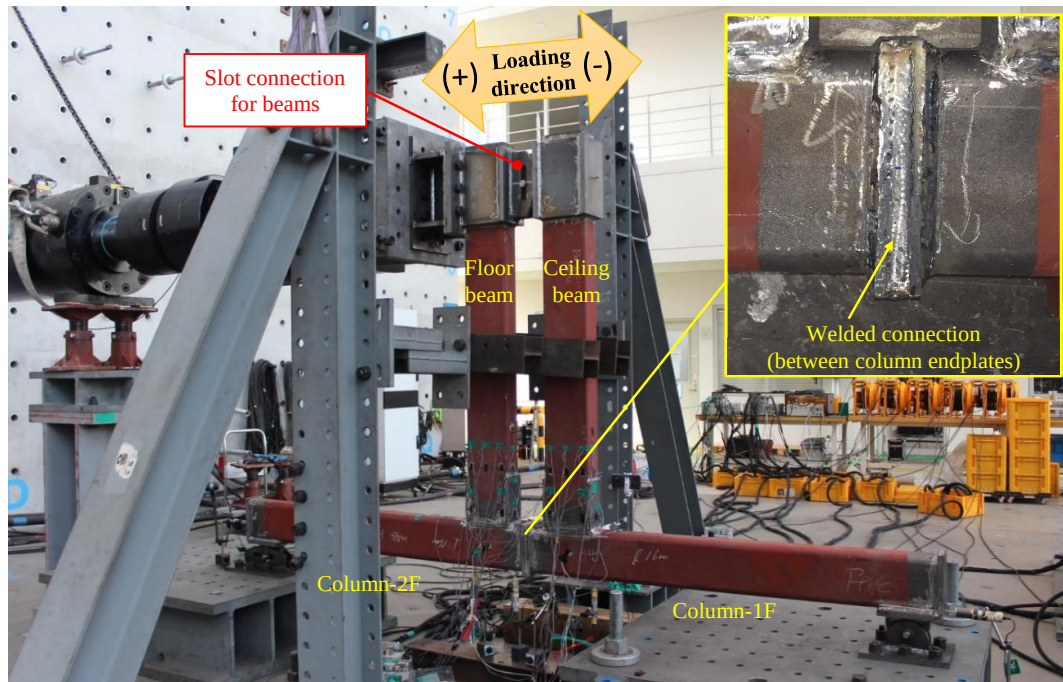


Figure 4.85: W-SB-Q test assembly – before conducting the test

The W-SB-Q specimen featured a welded connection between column-2F and column-1F. The hysteretic response under quasi-static cyclic loading is shown in Figure 4.86. As the lateral load reached 113kN at the displacement of 15.5mm (+1.0% drift), the floor beam and the ceiling beam yielded just above the reinforcing plates. At the displacement of 30.7mm (+2.0% drift) at 153kN, local buckling occurred in the floor beam first. In the next cycle, the ceiling beam experienced local buckling at the displacement of 32.3mm (+2.1% drift) and lateral load of 138kN. The strength degradation started after the local buckling of the floor beam and accelerated after the ceiling beam buckled. Unexpectedly, it was observed that the upper part of the floor beam just below the slot connection experienced local buckling at the displacement of 62mm (+4.0% drift) in the positive loading direction, as shown in Figure 4.88. The test stopped after the loading cycle for the drift of $\pm 5\%$. Figure 4.87 presents the final failure shape at the drift of $\pm 5\%$.

The columns and panel zones also remained in the elastic range throughout the tests, and there were no signs of cracks or fractures developing at the welded connections between the column and beam during the test. A summary of the test results is presented in Table 4.44.

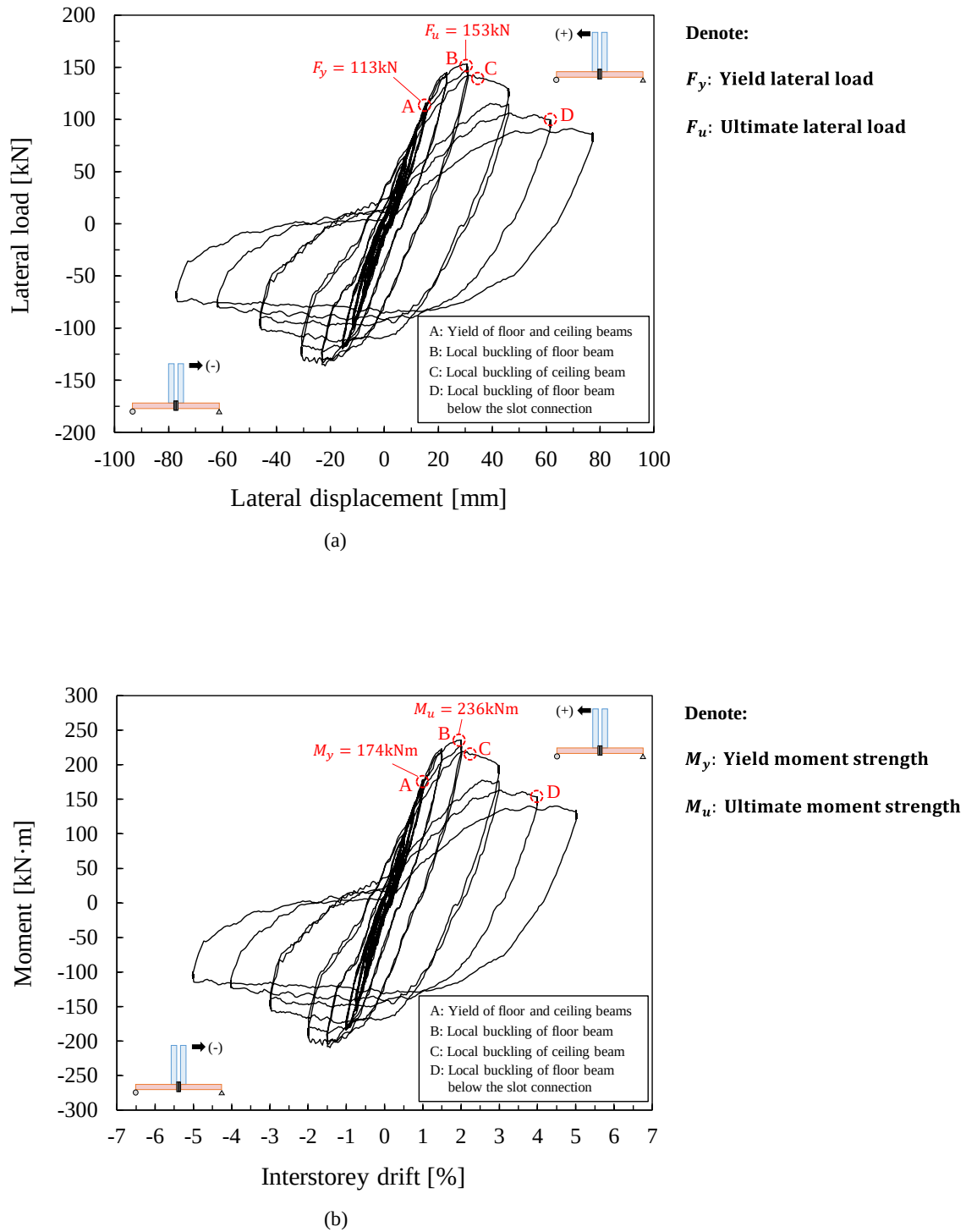
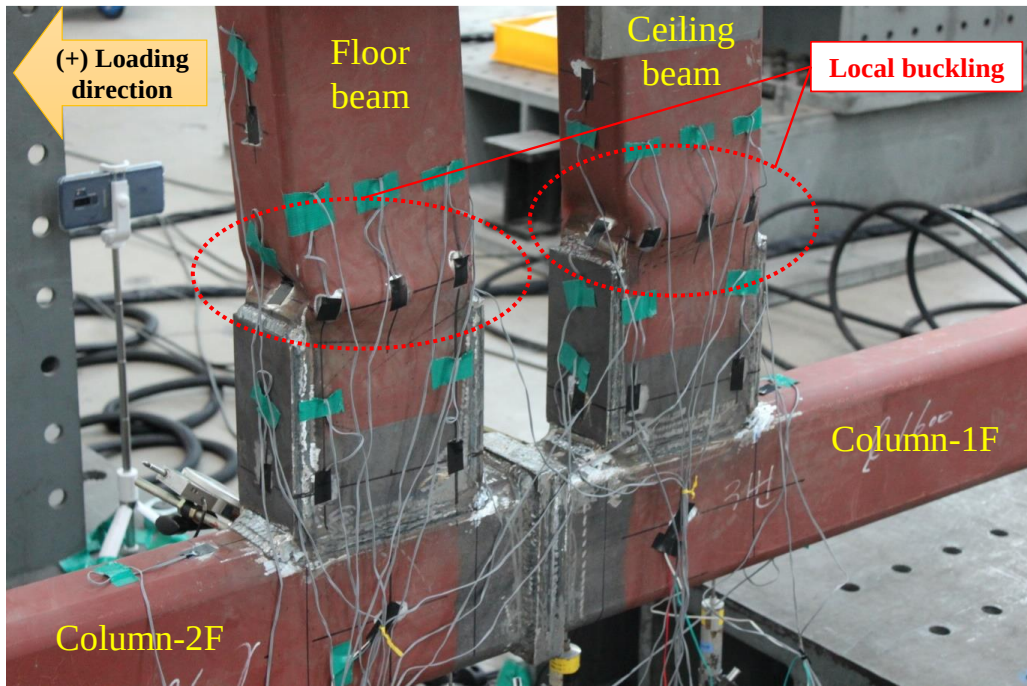
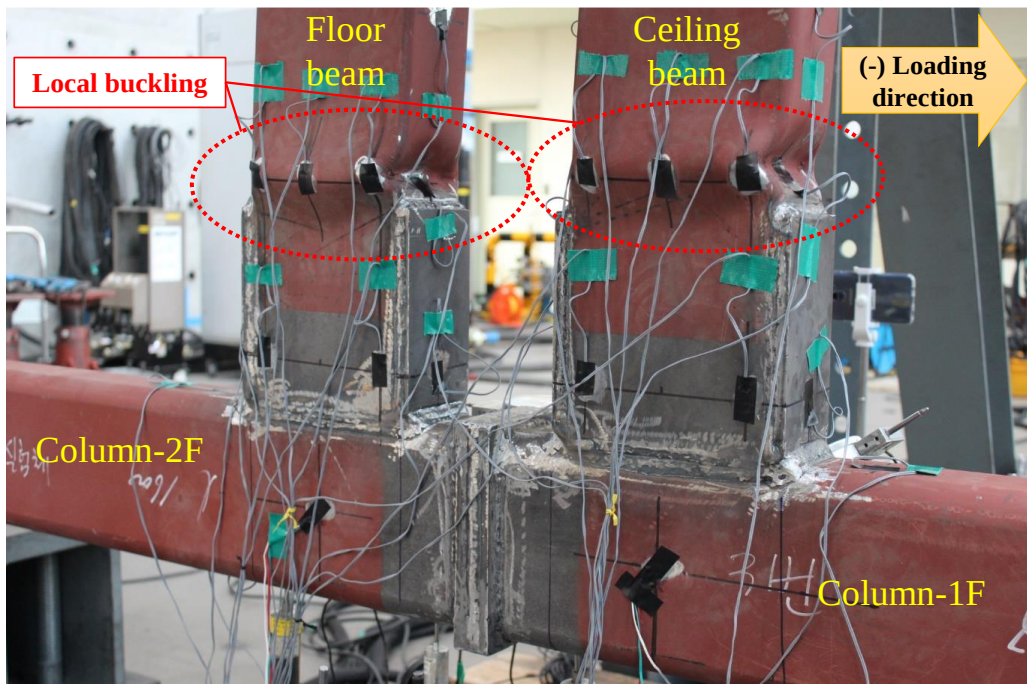


Figure 4.86: Hysteretic curves of W-SB-Q: (a) Lateral load-displacement curve; (b) Moment-drift curve



(a)



(b)

Figure 4.87: Failure mode of W-SB-Q under quasi-static cyclic loading: (a) at drift 5% in (+) loading direction, (b) at drift 5% in (-) loading direction.

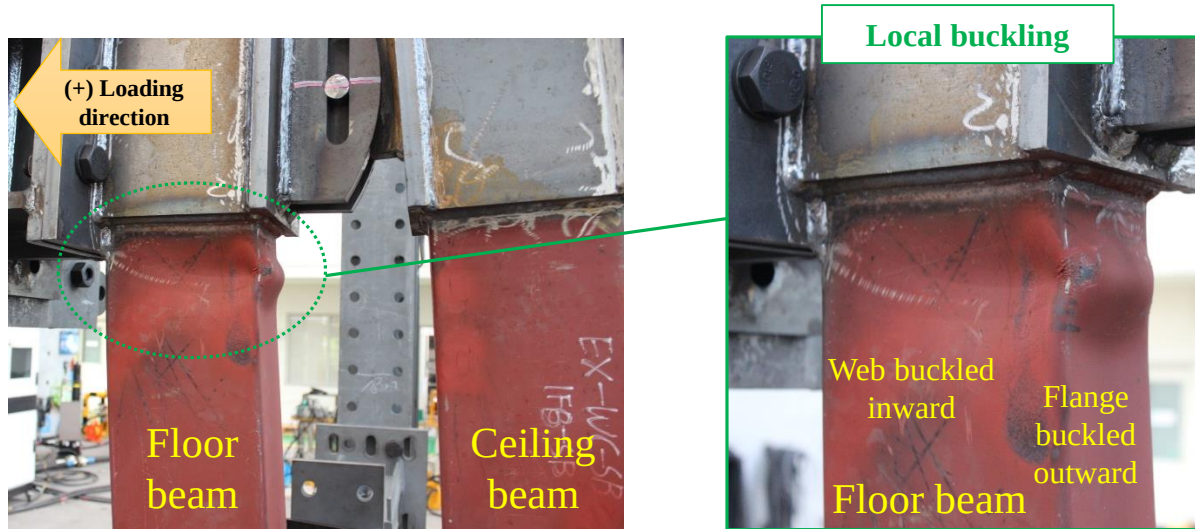


Figure 4.88: Local buckling of the floor beam just below the slot connection at the drift of 4% in (+) loading direction

4.4.10. Test Results and Evaluation

The main objective of the beam-to-column tests was to evaluate the seismic performance (failure mode, ductility, energy dissipation, etc.) of the proposed AJ connection for steel-framed modular buildings. Two types (P-DB and P-SB) of preloaded beam-to-column joints were tested under monotonic and quasi-static cyclic loading. To evaluate the performance of the AJ connection by comparison to a welded connection, the W-SB specimen was tested under quasi-static cyclic loading. Another objective of the tests was to demonstrate if the analytically calculated demand preload could prevent the gap-opening at the connection. A third objective was to confirm the effect of using different cross-sectional sizes of the floor and ceiling beams (implying different bending stiffnesses) on the demand preload required to prevent gap-opening at the connection.

However, after completing the tests, it transpired that the stiffness and strength of all tests were significantly higher than the values predicted by the numerical analyses. Figure 4.89 presents the moment versus interstorey drift curves for all the specimens from the tests and the pre-test numerical simulations presented in Section 4.4.8, and Table 4.43 presents the initial rotational stiffness and ultimate moment strength for all the specimens from the tests and the simulations. Regarding the initial rotational stiffness (K_i), overall, the test results were 1.61 - 2.17 times greater than the simulations. For the strength, the ultimate experimental moments (M_u) were 1.28 - 1.62 times greater than the simulation results in which the actual yield stress of the steel grade and the hardening effect were considered for the beams by the material overstrength factor γ_{ov} and the hardening overstrength factor γ_{sh} . Compared to the expected plastic moment strength of the joint (M_p) calculated based on the actual yield stress obtained from the material tests and the measured beam thickness, the ultimate moments obtained from the test results were 1.56 - 1.98 times larger than M_p .

These were significantly large differences, which were traced back to inconsistencies in the experimental set-up of the monotonic and quasi-static cyclic loading tests. Specifically, it was first observed that the pin in the connection between the floor and ceiling beams got stuck in the slot hole (see Figure 4.90) and then suddenly slid as the test progressed with a loud sound at the slot connection. This implied a partially composite behaviour of the two beams and an increase in both stiffness and strength. Secondly, after completing extensive numerical analyses, it became apparent that due to the relatively large size and heavy weight of the actuator compared to the test specimens (see Figure 4.90), the pin of the actuator did not rotate freely during the tests (refer to Section 5.3.2). The latter effect proved to be the more significant.

As a result of the partial rotational restraint of the pin connecting the actuator to the floor beam, the boundary condition of the tip of the floor beam was changed from a pin to a pin with a vertical spring, and a bending moment generated at the tip of the floor beam just below the lateral loading point (refer

to Figure 5.16 (b)). Meanwhile, the tip of the ceiling beam behaved as a hinge as planned. This partial rotational restraint of the actuator caused the increase in both the stiffness and strength of all test specimens.

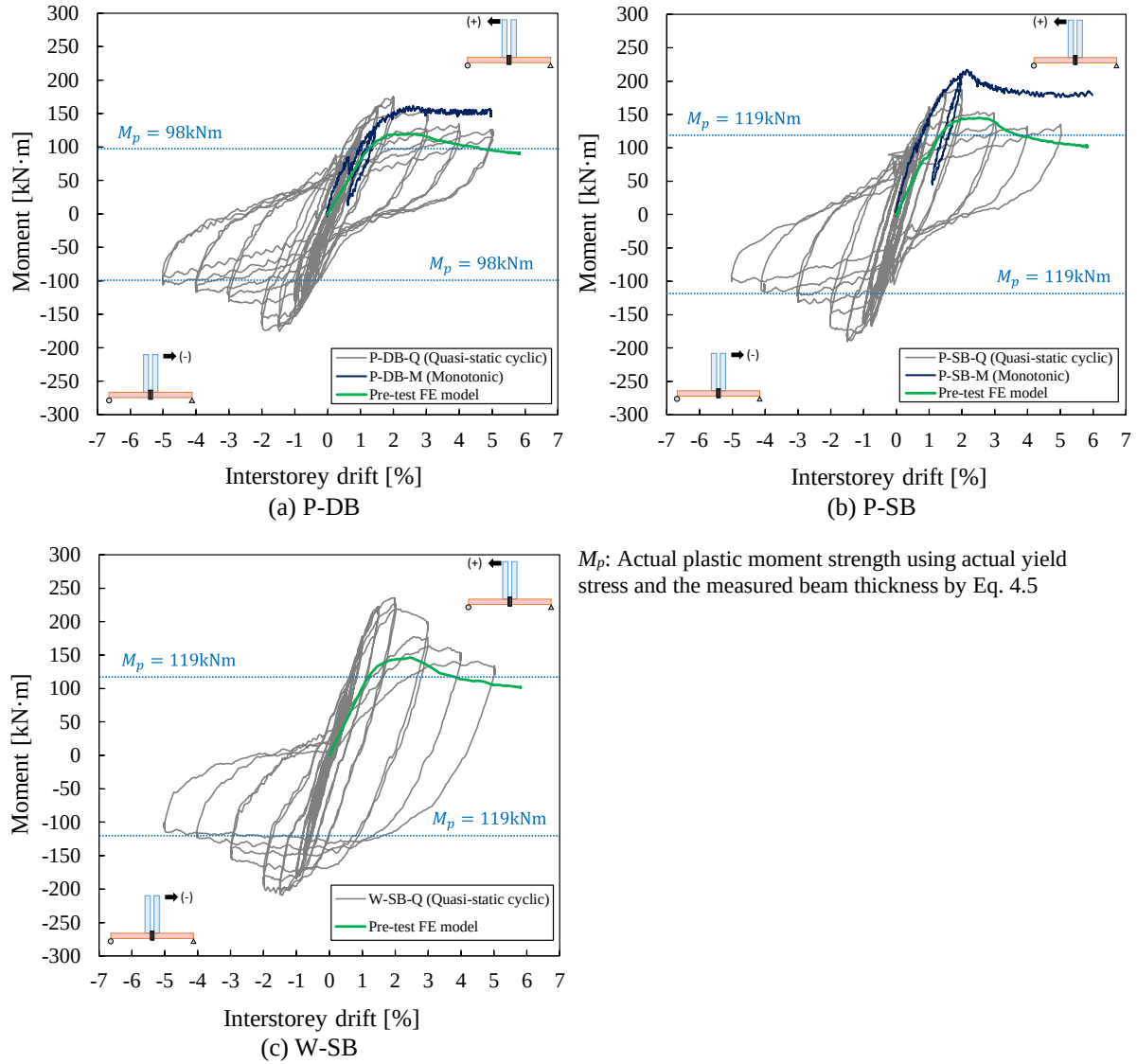


Figure 4.89: Test and pre-test numerical simulation results

Table 4.43: Comparison of test results with pre-test numerical simulations in positive loading direction

Type	Monotonic test		Quasi-Static cyclic test		Pre-test FE model		Ratio			
	$K_{i,m}$	$M_{u,m}$	$K_{i,c}$	$M_{u,c}$	$K_{i,FE}$	$M_{u,FE}$	Initial stiffness		Ultimate moment	
	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]	$\frac{K_{i,m}}{K_{i,FE}}$	$\frac{K_{i,c}}{K_{i,FE}}$	$\frac{M_{u,m}}{M_{u,FE}}$	$\frac{M_{u,c}}{M_{u,FE}}$
P-DB	14710	161	13024	175	8068	120	1.82	1.61	1.34	1.46
P-SB	19530	216	21931	187	10091	146	1.94	2.17	1.48	1.28
W-SB	-	-	19220	236	10112	146	-	1.90	-	1.62

* K_i denotes the initial rotational stiffness

* M_u denotes the ultimate moment strength

The partial rotational restraint also affected the test specimens in different ways depending on the type of the column-to-column connection. In the case of the welded joint (W-SB-Q), it caused local buckling to occur in only the floor beam just below the slot connection, as shown in Figure 4.88. For AJ joints (P-DB and P-SB), the gap-opening started on the floor beam side at the connection from the early elastic stage at a lower than expected moment.

Since all specimens were affected by the issues mentioned above, the seismic performance of the preload beam-to-column joint could not be accurately evaluated through the test results. In addition, the analytically suggested demand preload for preventing the gap-opening at the connection could not be validated based on the test results. Therefore, using the finite element models calibrated based on the monotonic test results, the seismic performance of AJ joints was re-evaluated, as detailed in Section 5.4. Also, the analytically calculated demand preload for the connection was validated in Section 5.5.1.3.

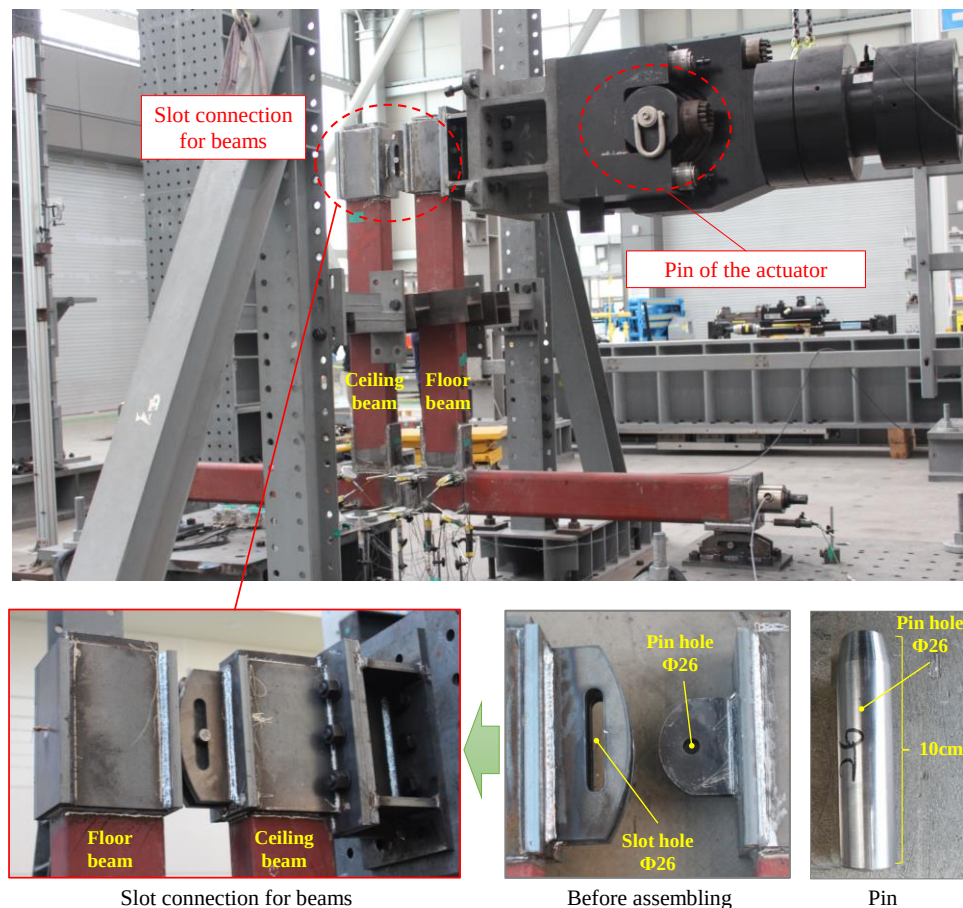


Figure 4.90: Actuator used for the test and the slot connection for the beams

In this Chapter, while the results obtained from the tests have been recorded as-are, only the test results that were considered meaningful have been discussed in detail. Among these was the failure mode of the AJ joint due to gap-opening at the connection. Due to the aforementioned partial rotational restraint of the actuator, the lateral forces applied to the floor and ceiling beams were not distributed according to the ratio of the bending stiffness of the two beams (refer to Section 3.5.1). Rather, the partial restraint caused an increase and decrease in the lateral forces applied to the floor and ceiling beams, respectively. So, in all tests of AJ joint specimens, the gap-opening occurred on the floor beam side at the connection from an early elastic stage of loading. The tests allowed confirmation of the effect of the occurrence of gap-opening in the initial elastic stage at the connection on the behaviour of the preload beam-to-column joint, such as the failure mode, load-bearing mechanism, inelastic behaviour, and the stress variation of the SRBs. Simply, the tests can be regarded as representing the possible failure mode in the case of severe gap-opening at the connection or applying no preload or a small amount of preload to the joint.

4.4.10.1. Summary of Test Results

Table 4.44 presents a summary of the test results. In the case of the W-SB-Q specimen (welded connection), it had the intended beam failure mode under the quasi-static cyclic loading by fully forming the plastic hinges at the floor and ceiling beams. The non-dissipative components such as the columns and panel zones remained in the elastic range until the beams failed. There were no cracks or fractures at the welded connections between the columns and the beams. For the P-DB and P-SB specimens (AJ preloaded joints), a gap-opening occurred at the AJ connection in the monotonic and quasi-static cyclic loading tests. As a result, only the ceiling beam reached its plastic moment capacity and failed by local buckling, while the floor beam slightly yielded or remained elastic. More details of the monotonic loading and quasi-static cyclic loading test results are described in this section.

Table 4.44: Summary of the beam-to-column joint test results

Specimen	Preload [kN]	Loading direction	$K_i \times 10^3$ [kNm]	M_y [kNm]	M_u [kNm]	M_u/M_p	θ_y (%)	θ_u (%)	M_s [kNm]	M_g [kNm]	Failure mode
P-DB-M	155	(+)	14.7	120	161	1.64	1.28	2.47	85	77	Only ceiling beam (with gap-opening)
P-SB-M	145	(+)	19.5	156	216	1.82	1.15	2.18	105	112	Only ceiling beam (with gap-opening)
P-DB-Q	185	(+)	13.0	111	175	1.79	1.05	2.00	82	92	Only ceiling beam (with gap-opening)
		(-)	17.5	108	174	1.78	0.71	1.47	-	108	
P-SB-Q	145	(+)	21.9	151	186	1.56	0.98	2.00	92	116	Only ceiling beam (with gap-opening)
		(-)	23.1	160	191	1.61	0.82	1.50	-	117	
W-SB-Q	NA	(+)	19.2	174	236	1.98	1.00	2.00	-	-	Both floor & Ceiling beam
		(-)	22.1	162	210	1.76	0.82	1.43	-	-	

Note: K_i denotes the initial rotational stiffness; M_y denotes the yield moment strength; M_u denotes the ultimate moment strength; M_p denotes the actual plastic moment strength of the joint by (Eq. 4.5); θ_y denotes the yield interstorey drift; θ_u denotes the ultimate interstorey drift; M_s denotes the moment for the occurrence of the slip at the AJ connection; M_g denotes the moment for the occurrence of the gap-opening at the AJ connection.

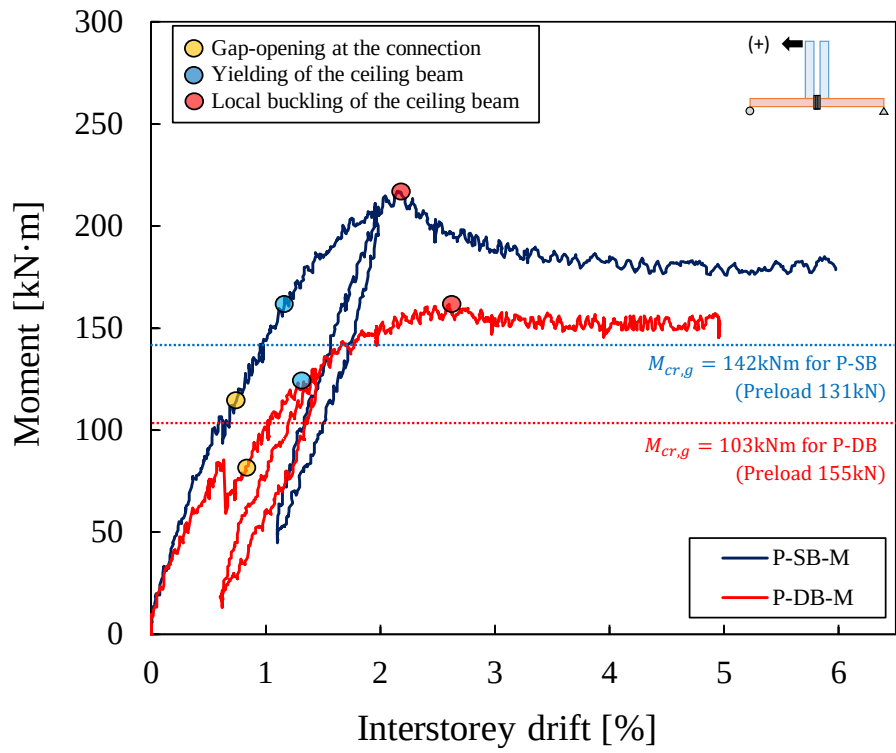
▪ Monotonic loading test

Table 4.44 and Figure 4.91 (a) present the monotonic test results and the moment versus interstorey drift curves for P-DB-M and P-SB-M. The P-SB-M specimen had a greater initial stiffness and ultimate strength than P-DB-M because of the larger cross-section used for the ceiling beam. Both specimens experienced a sudden slip at a drift of approximately 0.65% during the tests because the pin was stuck in the slot holes and then moved suddenly at the slot connection at the tip of the floor and ceiling beams. As shown in Figure 4.91 (a), both specimens developed an interior gap-opening (on the floor beam side) at the connection at a lower moment than the critical moment $M_{g,cr}$. Despite applying a relatively lower amount of preload to P-SB-M, the gap-opening occurred at a larger moment than P-DB-M. It was confirmed that the demand preload to prevent the gap-opening at the connection can be reduced by reducing the difference in the cross-sectional size (i.e. bending stiffness) of the two beams. P-DB-M and P-SB-M both developed interior gap-opening at the AJ connection. As a result, they failed by the same failure mechanism, described as follows:

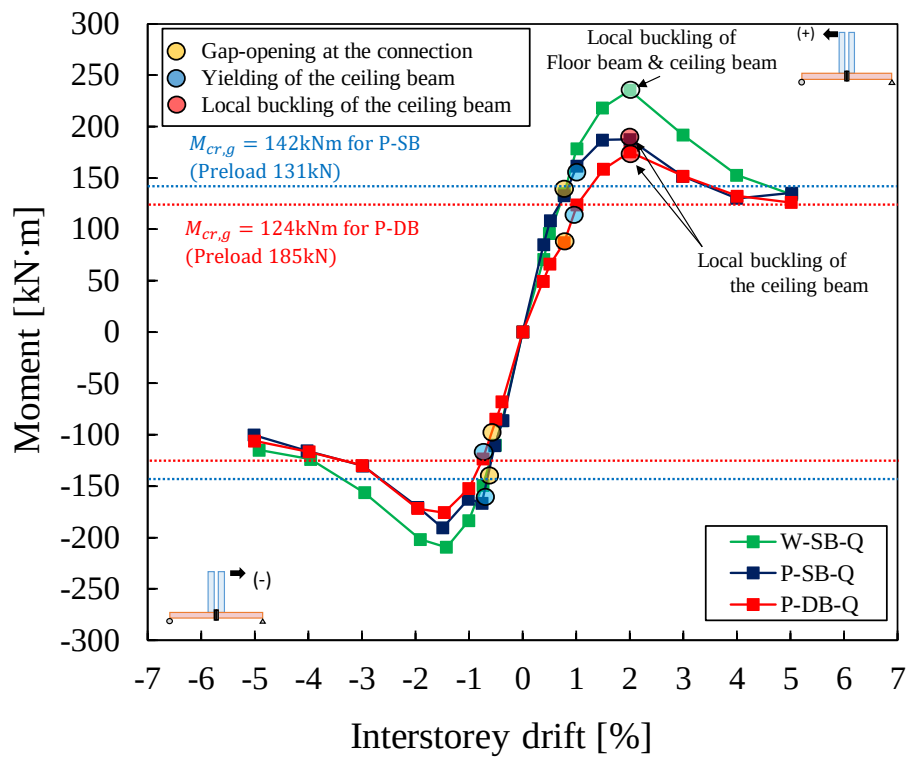
1. The interior gap-opening occurred at the AJ connection.
(→ Tensile load to be applied to the SRBs in the column.)
 2. The gap-opening caused the ceiling beam to yield first.
(→ The amount of the tensile load to the SRBs increased rapidly.)
 3. Only the ceiling beam reached its plastic moment capacity.
 4. Only the ceiling beam failed by local buckling.
(→ Local buckling caused the tensile force in the SRBs to increase more rapidly.)
- * The floor beam yielded only slightly or not at all in the tests and did not buckle locally.

The failure mode due to the gap-opening was identical to the expected failure mechanism as discussed in Section 3.5.4 (refer also to Figure 3.38 (b)). The test results showed that the gap-opening at the connection prevented the AJ joint from fully utilising its total plastic moment capacity, which is the sum of the plastic moments of the floor and ceiling beams.

The strength degradation was not severe for both specimens. In the case of P-DB-M, there was almost no reduction in strength after reaching the ultimate moment. For P-SB-M, the strength reduction was less than 20% of its ultimate strength at the conclusion of the test. The capacity to maintain strength was because the SRBs within the columns increasingly resisted the lateral load as the ceiling beam failed. As a result, the bolt threads near the coupler yielded because they were subjected to both a large tensile force due to preload and gap-opening and a bending moment, the latter induced by the coupler rotating due to bearing against the plates after the occurrence of slip at the connection.



(a) Monotonic test results



(b) Quasi-static cyclic test results – skeleton curves

Figure 4.91: Moment versus interstorey drift curves from the tests

▪ Quasi-static cyclic loading test

The seismic performance (e.g. ductility, energy dissipation, etc.) of the AJ joint could not be evaluated from the test results. They are evaluated in Section 5.4, using a finite element model calibrated based on the test results. However, the effect of the gap-opening on the behaviour of the AJ joint under cyclic loading could be determined from the test results.

Table 4.44 and Figure 4.91 (b) present the test results and the skeleton curves for the three specimens. The curves reflect the hardening and strength degradation of the specimens. In the case of W-SB-Q, both the floor and ceiling beams yielded and failed at a similar time. On the other hand, P-DB-Q and P-SB-Q experienced interior gap-opening in the positive loading direction and exterior gap-opening in the negative loading direction on the floor beam side. Due to the gap-opening at the AJ connection, their failure mechanism was the same as that in their respective monotonic test. As a result, only the ceiling beam yielded and formed a plastic hinge, and then failed by local buckling, while the floor beam yielded only slightly or not at all. Both specimens experienced gap-opening at the connection at a lower moment than the critical moment of the gap-opening ($M_{g,cr}$), as presented in Figure 4.91 (b).

P-SB-Q had a greater initial stiffness than P-DB-Q because of its larger cross-section of the ceiling beam. The P-SB-Q and W-SB-Q specimens, which were identical specimens except for the connection type, had similar initial stiffness. This means that the AJ connection had a similar initial stiffness to the welded connection while no gap-opening occurred at the connection, as would be expected. However, W-SB-Q had larger ultimate strength than P-SB-Q because it failed by both the ceiling and floor beams forming plastic hinges, whereas P-SB-Q only formed a plastic hinge in the ceiling beam due to the gap-opening at the connection. The inelastic behaviour of the AJ connection with gap-opening is discussed in Section 4.4.10.2.

Both P-DB-Q and P-SB-Q experienced strength degradation after the local buckling of the ceiling beam. Compared to the monotonic test results, P-DB-Q and P-SB-Q showed that the strength degradation was relatively severe due to the accumulated damage under the cyclic loading. In the case of W-SB-Q, the strength degradation started after both the floor and ceiling beams failed by local buckling. Compared to W-SB-Q, the P-DB-Q and P-SB-Q specimens exhibited a gradual decline in the rate of strength degradation because the SRBs in the columns resisted the lateral load after the failure of the ceiling beam. If there had been no gap-opening at the connection in the tests of P-DB-Q and P-SB-Q, it is likely that the strength degradation would have been similar to that of W-SB-Q.

4.4.10.2. Inelastic Behaviour of the Preload Beam-to-column Joint with Gap-opening

In seismic design, ductility means the ability of a structure to undergo large amplitude cyclic deformations in the inelastic range without a substantial reduction in strength. Ductile structures are generally able to dissipate significant amounts of energy in the event of an earthquake. For a moment-resisting frame, ductility is mainly obtained from the plastic behaviour of the beam-to-column joints, specifically the beams. Looking at the graph of moment-interstorey drift of P-DB and P-SB in Figure 4.91 (a) and (b), although gap-opening occurred, their inelastic behaviour indicates a ductile behaviour with no severe strength degradation. However, it is not possible to conclude that the behaviour is ductile, or to evaluate the ductility, for the reason as set out below.

In the monotonic and quasi-static cyclic loading tests, due to the gap-opening at the connection, the inelastic behaviour of P-DB and P-SB started after the ceiling beam yielded, and their strength degradation began as the ceiling beam buckled locally (see Figure 4.91 (a) and (b)). However, the inelastic rotational deformations did not only consist of the deformation of the structural members but included the rotational deformation associated with the gap-opening. In fact, after the yielding and failure of the ceiling beam, the contribution of the gap-opening to the total rotational deformation increased because the gap-opening increased more rapidly. Thus, since the inelastic behaviour is not purely the deformation of the structural members, the degree of ductility is impossible to quantify. In addition, in the strength degradation, most of the lateral load was resisted by the SRBs in the columns. In the monotonic and quasi-static cyclic loading tests, this phenomenon was observed in both P-DB and P-SB specimens. As a result, the SRBs, which are the most critical members along with the coupler for connecting the module vertically, were yielded and exposed to rupture.

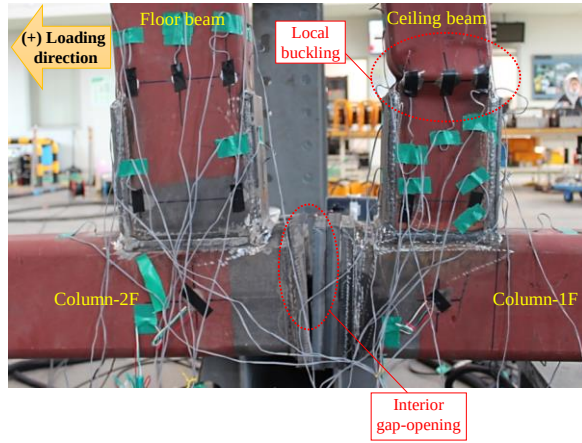
Consequently, if the AJ connection develops gap-opening, the gap-opening prevents the joint from obtaining the ductility implied by plastic hinges forming in the two beams. Therefore, gap-opening should be avoided by introducing an appropriate preload to the connection.

4.4.10.3. Effect of Gap-opening at the AJ Connection

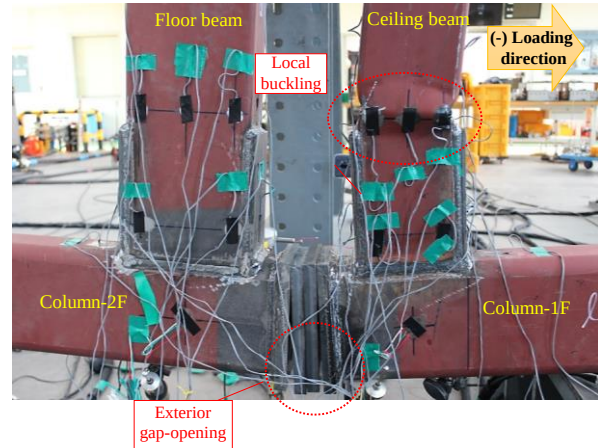
The P-DB and P-SB specimens showed failure due to gap-opening at the AJ connection in the monotonic and quasi-static cyclic loading tests. Due to the partial rotational restraint of the pin connecting the actuator to the floor beam, the lateral force was not distributed to the floor beam and the ceiling beam according to the ratio of the bending stiffness of the two beams (refer to Section 3.5.1), so the floor beam was subjected to a larger lateral force than expected (refer to Section 5.3.2.1). As a result, all the P-DB and P-SB specimens experienced gap-opening on the floor beam side at the connection from the early elastic stage. Their failure mechanism was the same as the expected failure in case the gap-opening occurs on the floor beam side, as presented in Figure 4.92. The effect of gap-opening on the joint behaviour as confirmed through the experiments is summarised as follows:

- (1) A gap-opening at the AJ connection changes the failure mechanism of the beam-to-column joints. It makes one of the beams fail earlier, and prevents the other beam from developing a full plastic hinge. That is, a gap-opening hinders the formation of plastic hinges in both the floor and ceiling beams, resulting in reduced ductility of the beam-to-column joint.
- (2) The SRBs within the columns are subjected to a large tensile force to resist the lateral load. As a result, the SRBs, which are the critical members for connecting modules, are exposed to the risk of yielding or fracture.

Table 4.45 presents a comparison of the test results and the analytical predictions of the moments causing gap-opening and slip at the AJ connection. The test results of P-DB and P-SB showed that the gap-opening occurred at an approximately 25% lower moment than the predicted moment $M_{cr,g}$. Similarly, the test results showed that the P-DB specimens experienced slip at a lower moment than the predicted slip moment $M_{cr,s}$, while the P-SB specimens experienced slip at the predicted moment $M_{cr,s}$. One of the purposes of the beam-to-column joint tests was to prove the validity of the demand preload to prevent gap-opening and slip at the connection, as suggested in Sections 3.4.3 and 3.4.4. However, it was not possible to draw this conclusion from the test results because of the abovementioned inaccuracies introduced in the test setup. Therefore, using the calibrated FE model, the validity of the design method for preventing gap-opening and slip at the AJ connection is evaluated in Section 5.5.1.3.



The ceiling beam failed with the interior gap-opening in (+) loading direction



The ceiling beam failed with the interior gap-opening in (-) loading direction

(a) Test results of P-DB-Q at drift 5%

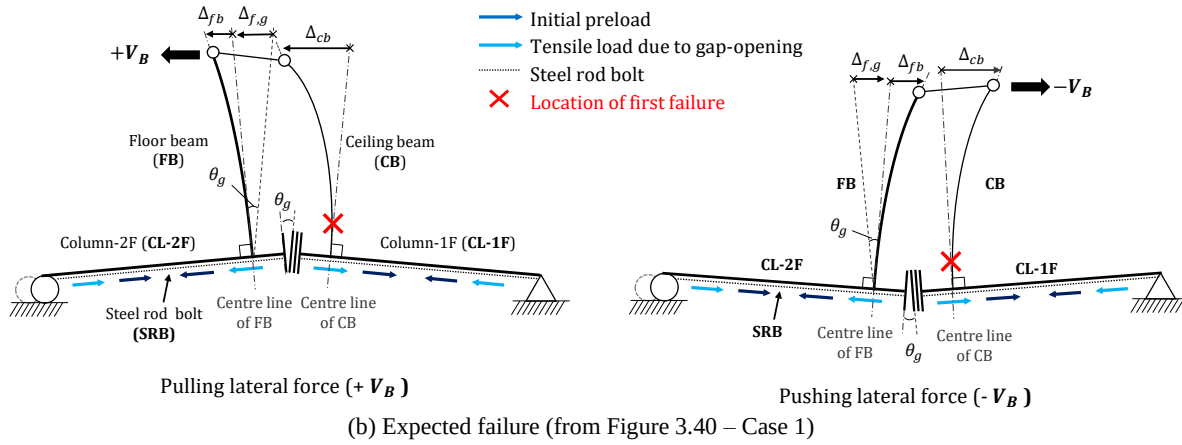


Figure 4.92: Comparison of the test results and the expected failure due to the gap-opening at the AJ connection

Table 4.45: Comparison of the test results with the analytical expected strength on the gap-opening and slip at the AJ connection

Specimens	Preload [kN]	Loading direction	Test results		Analytical expected strength		Ratio	
			M_s [kNm]	M_g [kNm]	$M_{cr,s}$ [kNm]	$M_{cr,g}$ [kNm]	$\frac{M_s}{M_{cr,s}}$	$\frac{M_g}{M_{cr,g}}$
P-DB-M	155	(+)	85	77	93	103	0.91	0.75
P-DB-Q	185	(+)	82	92	112	124	0.73	0.74
		(-)	95	108			0.85	0.88
P-SB-M	145	(+)	105	112	94	157	1.12	0.71
P-SB-Q	145	(+)	92	116	94	157	0.98	0.74
		(-)	95	117			1.02	0.74

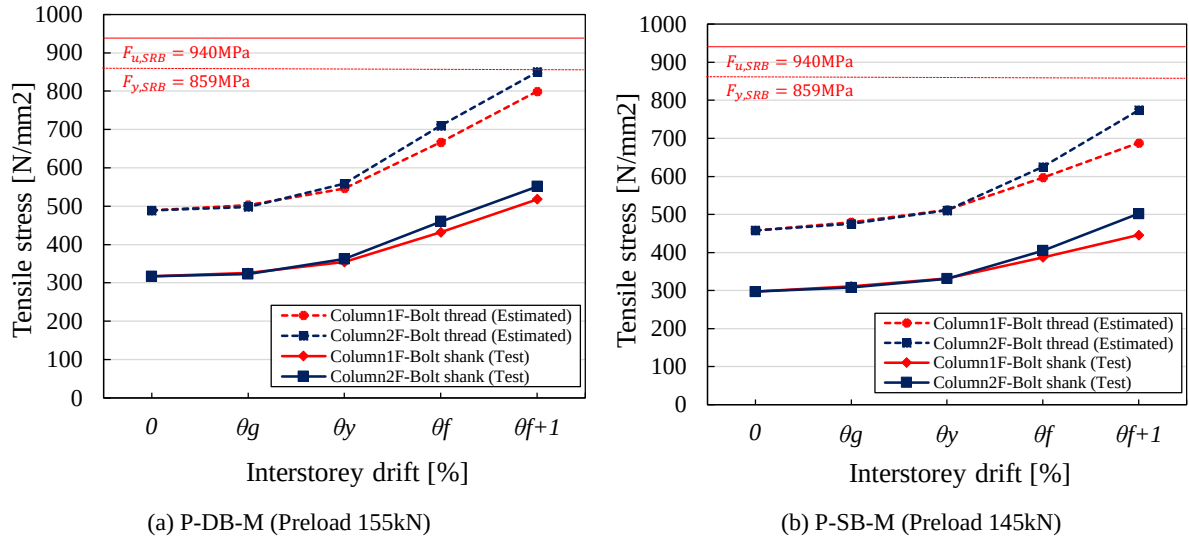
Note: M_s denotes the moment for the occurrence of the slip at the AJ connection; M_g denotes the moment for the occurrence of the gap-opening at the AJ connection; $M_{cr,s}$ denotes the critical moment strength for preventing the slip at the AJ connection by (Eq. 3.82); $M_{cr,g}$ denotes the critical moment strength for preventing the gap-opening at the AJ connection by (Eq. 3.69).

4.4.10.4. Stress Variation of the SRBs Within the Columns Due to Gap-opening at the Connection

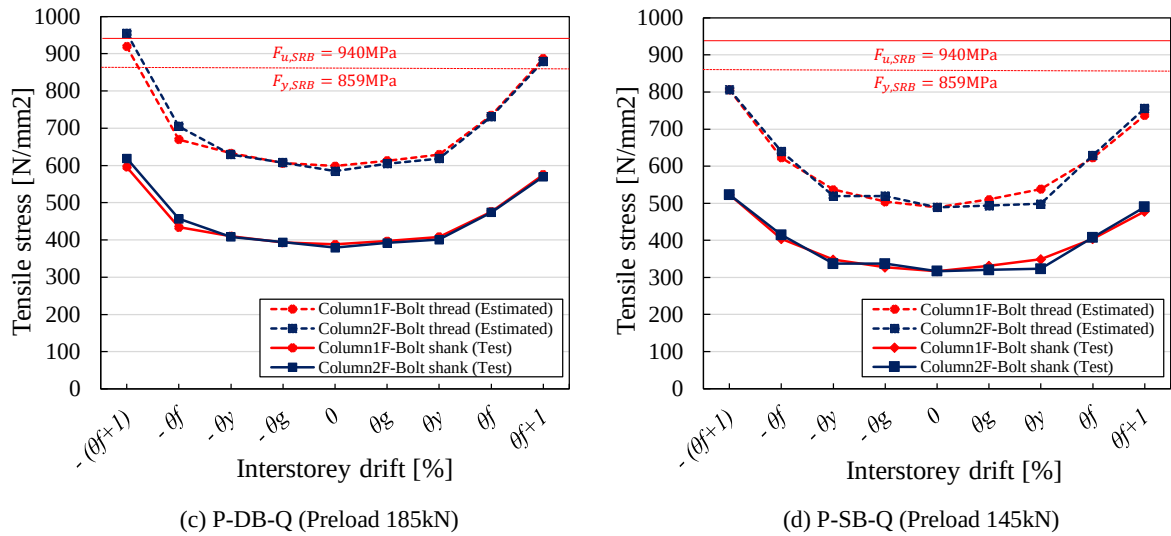
The test results of the P-DB and P-SB specimens showed that the SRBs within the columns were subjected to a large increase in tensile force after gap-opening occurred at the connection. Figure 4.93 shows the tensile stress variation measured on the shank of the SRBs by strain gauges for increasing levels of drift deformation (θ). Figure 4.94 shows photos of the gap-opening at the AJ connection of the P-DB-Q specimen for increasing levels of drift deformation. (The P-SB and P-DB specimens experienced almost the same sequence of deformation). The most vulnerable part of the SRBs is the bolt thread having the smallest cross-section, but it was difficult to attach strain gauges to this part of the bolts. So, based on the nominal cross-sectional area of the bolt thread (refer to Table 4.16) and the stress measured on the shank, the estimated stress variation of the bolt thread was calculated as shown by the dotted lines in Figure 4.93. According to the overall trend of the figure, the tensile stress of the SRBs within column-1F and column-2F increased only slightly after the gap-opening occurred at the connection (θ_g). However, after the ceiling beam yielded due to the effect of the gap-opening (θ_y), the tensile stress of the SRBs increased rapidly. After local buckling occurred in the ceiling beam (θ_f), the monotonic test results showed that the tensile stress of the SRBs increased continually at a similar rate whereas the cyclic test results showed they increased very fast. The estimated stress of the bolt thread almost reached the yield stress ($F_{y,SRB}$) at the drift of $\theta_f+1\%$. Overall, after the ceiling beam yielded (θ_y), the rate at which the stress levels increased in the SRBs was larger in P-DB than it was in P-SB.

If the SRB is stretched by yielding locally due to the lateral force, the preload induced at the connection is reduced when the force is removed. In the event of an earthquake, the preload applied to the connection could be gradually reduced if the SRB incurs even a small amount of local yielding under repeated loading. Therefore, gap-opening and slip should be prevented by applying a sufficient amount of preload to the SRBs, which are the critical members for connecting modules. Further information on the stress variation of the SRBs within the columns is provided in Section 5.5.1.2.

Monotonic test



Quasi-static cyclic test



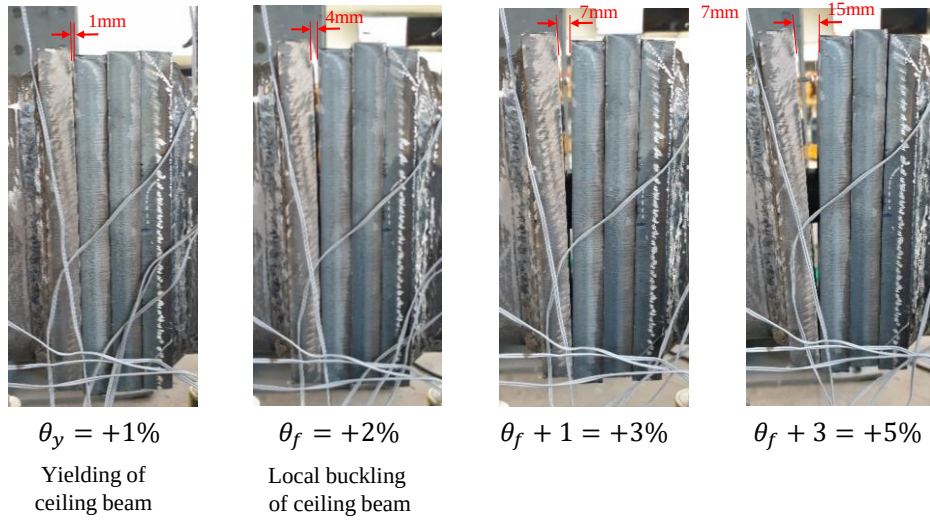
Denote:

θ_g : Drift of the occurrence of the gap opening at the preload connection
 θ_y : Yield drift of the ceiling beam
 θ_f : Failure drift of the ceiling beam by the local buckling

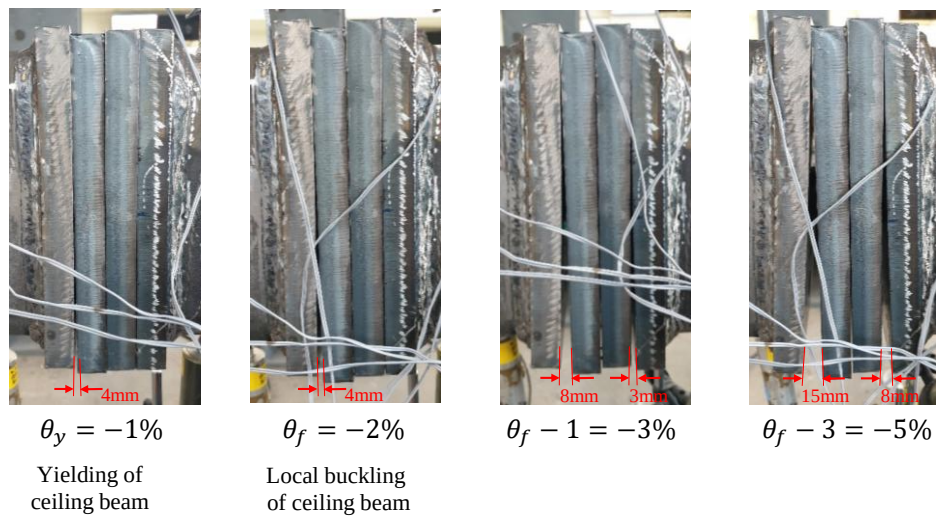
θ_{f+1} : Drift after 1% of the failure drift
 $F_{y,SRB}$: Yield stress of the steel rod bolt
 $F_{u,SRB}$: Ultimate strength of the steel rod bolt

Figure 4.93: Tensile stress variation of SRBs in P-DB and P-SB due to the effect of gap-opening according to the failure mechanism in the monotonic and quasi-static cyclic tests

(+) Loading direction - Interior gap-opening



(-) Loading direction - Exterior gap-opening



Denote:

θ_y : Yield drift of the ceiling beam

θ_f : Failure drift of the ceiling beam by the local buckling

Figure 4.94: Gap-opening at the AJ connection in the P-DB-Q in accordance with the failure mechanism

4.5. Conclusion

(1) Material tests

- (i) The material tests were conducted to determine the material properties of each component used in the beam-to-column joint test specimen. All tensile coupons were prepared and tested according to the Korean standard. Test results satisfied the minimum requirements for each steel grade as specified by the standards.
- (ii) The developed coupler is the most critical member of the AJ connection for connecting modules. Therefore, the coupler should be designed to behave within the elastic range until the SRBs are fully yielded to the point of fracture. Uniaxial tensile tests were conducted on the coupler and SRBs. The test results satisfied the minimum requirements of KS D 0249 [99]. The test confirmed that all failures occurred in the SRBs, and the coupler did not yield until the SRBs failed.

(2) Tests of applying preload to steel rod bolts

The current standards EN 1090-2:2018 [100] and KBC 2016 [101] provide the torque reference values for specific values of preload for high tensile bolts tensioned using the torque method. However, it was uncertain whether this method is applicable to the 3m-SRB investigated herein. Therefore, tests were conducted to determine the relationship between the torque and the resulting preload for the 3m-SRB with the preload applied by a torque wrench. Based on the torque wrench tests, the capacity of the cordless impact wrench, which would be used on construction sites in practice, was selected, and impact wrench tests were conducted to evaluate the amount of preload applied. Lastly, it was investigated whether the amount of applied preload in the lower column was reduced when the upper column was disassembled by loosening the coupler for reuse of the upper module.

- (i) As a result of the torque wrench tests, the average torque coefficient (non-lubricated applications) of the 3m-SRB was 0.53. This was approximately 2.7 times greater than the theoretical torque coefficient for the general M24 high tensile bolt (featuring the same bolt size as the SRBs).
- (ii) In the cordless impact wrench test, the applied preload was 20-35% lower than the average preload induced in the torque wrench test by the same amount of torque. The relationship between torque and preload was irregular. In addition, it was found that the performance of the cordless impact wrench was affected by the battery state of charge. Compared to the preload applied by the impact wrench with a fully-charged battery, the amount of preload applied by a half-charged impact wrench was lower by 13% - 40% at the same torque.
- (iii) As a result of the torque wrench and the impact wrench tests, it was confirmed that the preload in the lower column was maintained after the upper column was disassembled from the lower column.

(3) Pre-test numerical simulations (for beam-to-column joint)

Before conducting the beam-to-column joint tests, pre-test numerical simulations were performed under monotonic loading to predict the accurate behaviour of the joint and to avoid unexpected failure during the tests. The results showed that the beam-to-column joints with the AJ connection had ductile behaviour ensured by plastic hinges forming at the expected locations in both floor and ceiling beams without the occurrence of gap-opening at the connection prior to the two beams failing. The non-dissipative components including the columns, panel zone, couplers, and SRBs remained in the elastic range. It was confirmed that the AJ connection could have the same performance as the welded connection by ensuring sufficient preload is induced to prevent gap-opening at the connection. Consequently, the pre-test simulations demonstrated a good design of the preload beam-to-column joint.

(4) Beam-column joint tests

The main objective of the beam-to-column tests was to evaluate the seismic performance of the proposed AJ connection and to prove the validity of the demand preload calculated analytically for preventing gap-opening (P_g) and slip (P_g) at the connection. Two types of specimens (P-DB and P-SB) were tested under monotonic loading and quasi-static cyclic loading.

After completing the tests, inconsistencies in the experimental set-up of the monotonic and quasi-static cyclic loading tests were identified causing all the test specimens to fail due to gap-opening at the connection. As a result, the seismic performance of the preload beam-to-column joint could not be evaluated from the test results. Therefore, using the finite element models calibrated based on the monotonic test results, the seismic performance of AJ joints is re-evaluated in Section 5.4. Also, the analytically calculated demand preload for the connection is validated in Section 5.5.1.3.

While the results obtained from the test have been recorded as-are, only the test results that were considered meaningful have been discussed in detail. A meaningful observation from the test results was the failure mode of the AJ joint resulting from gap-opening at the connection.

- (i) The gap-opening at the AJ connection changed the failure mechanism of the beam-to-column joints. As a result of the gap-opening at the connection in all specimens (P-DB-M/Q and P-SB-M/Q), only the ceiling beam reached its plastic moment capacity and failed by local buckling, while the floor beam only yielded slightly or not at all. Consequently, it was confirmed that the gap-opening at AJ connection hinders the formation of plastic hinges in both the floor and ceiling beams, resulting in reduced ductility of the beam-to-column joint.
- (ii) The SRBs within the columns were subjected to large tensile forces in addition to the preloads to resist the lateral load produced by gap-opening at the AJ connection. In particular, the threads of

the SRBs near the coupler were subjected to additional bending by the rotation of the coupler, caused by bearing against the plates as slip occurred at the connection. All test results of the AJ joint (P-DB-M/Q and P-SB-M/Q) showed that the bolt threads were bent. Therefore, gap-opening and slip should be minimised by applying a proper amount of preload to the SRBs, which are the critical members for connecting modules.

- (iii) Regarding the effect of using floor and ceiling beams with different sizes (i.e. different bending stiffness EI) on the gap-opening at the AJ connection, the test results showed that the detrimental effect of gap-opening can be minimised if the two beams have the same size.

Chapter 5

Numerical Investigation of the AJ Connection

5.1. Introduction

Full-scale beam-to-column joints tests under monotonic and quasi-static cyclic loading were described in Sections 4.4.9 and 4.4.10. The tests were conducted to evaluate mainly the seismic performance of the AJ joint and to validate the effectiveness of the analytical equation (Sections 3.4.3 and 3.4.4) for the demand preload for preventing gap-opening and slip at a preload column-to-column connection. However, unfortunately, an unexpected issue from the test setup affected all test results and did not allow adequate evaluation of the joints, as described in Section 4.4.10. Originally, the tests on the beam-to-column joint were planned to be conducted in two series in South Korea in 2020. After completing the first test series in February 2020, the second test series was scheduled for a few months later. However, it could not be performed as scheduled because prolonged COVID-19 restrictions imposed at that time prevented travel from Australia to South Korea, and was eventually removed from the test program.

Therefore, in this chapter, using advanced finite element models, the seismic performance of the AJ beam-to-column joint was re-evaluated as follows: Firstly, FE joint models were created in Abaqus with material properties obtained from coupon tests. Using the FE joint models, it was investigated which factors from the test setup might have affected the test results. Then, the joint models with the factors included were calibrated based on the test results. Finally, the calibrated joint models excluding the factors were used for re-evaluating the seismic performance of the AJ joint.

In addition, parametric studies were conducted using the calibrated joint models to investigate the influence of different parameters (e.g. amount of preload, size of the coupler hole, and application of axial force to the columns) on the behaviour of the overall beam-to-column joints and SRBs in columns. All simulation analyses were conducted using the program Abaqus (Version 6.14) and the High-Performance Computing (HPC) facility at the University of Sydney.

5.2. Calibration of Material Models

Material models for the steel grades used in the joint were calibrated based on tensile coupon test results, which were reported in Section 4.2.1. The tensile coupon test provides the engineering stress and strain, which do not reflect a change in the cross-sectional area of a material specimen during the test. The actual cross-sectional area decreases while the specimen is being deformed due to elastic and plastic Poisson deformations. Therefore, the engineering stress and strain needs to be converted to the true stress and strain that can describe the actual material behaviour of the steel component. The true stress and strain curve can be found through (Eq. 5.1) and (Eq. 5.2) from C.1 in EN 1993-1-5 [113] based on the engineering stress-strain curve obtained from the tensile coupon test.

$$\sigma_{true} = \sigma(1 + \varepsilon) \quad (\text{Eq. 5.1})$$

$$\varepsilon_{true} = \ln(1 + \varepsilon) \quad (\text{Eq. 5.2})$$

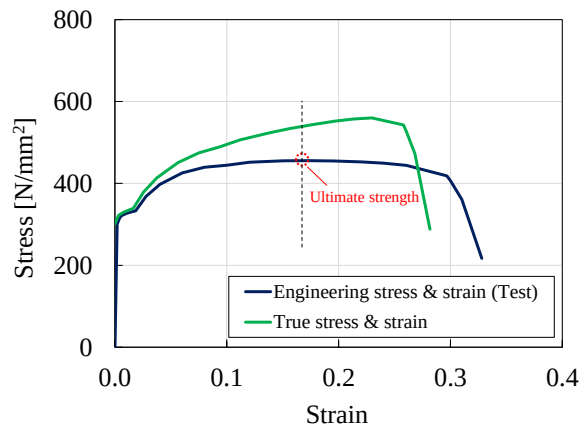
where :

σ_{true}	True stress;
ε_{true}	True strain;
σ	Engineering stress;
ε	Engineering strain.

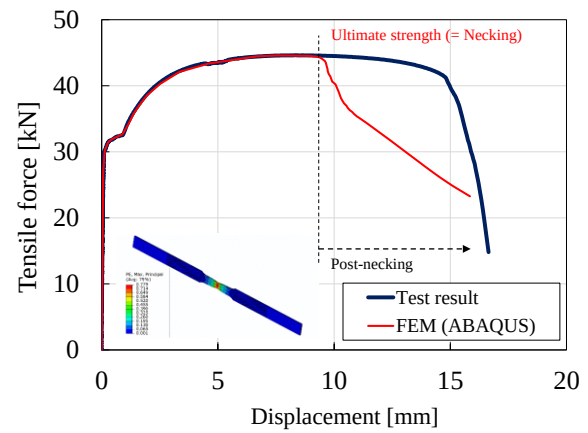
Figure 5.1 (a) shows the engineering stress-strain curve of the material specimen for the RHS ceiling beam $150 \times 100 \times 4$ and the true stress-strain curve computed using (Eq. 5.1) and (Eq. 5.2). Figure 5.1 (b) presents the calibration result in the force-displacement curve using the true stress and strain. The result showed that (Eq. 5.1) and (Eq. 5.2) are valid up to the ultimate strength, where necking occurred and the stress no longer remained uniform. In order to obtain a good agreement with the test in the post-necking stage, additional values need to be used for the true stress and strain beyond the ultimate strength. The values were found using the weighted average method [114], as presented in Figure 5.1 (c). This allowed obtaining a very close reproduction of the tensile test, as shown in Figure 5.1 (d).

Using this procedure, the material models for the RHS column (SRT355), RHS beams (SRT275), steel plates (SS275), reinforcing plate (SS275) and steel rod bolt (AISI 4140) were calibrated based on tensile tests conducted on the same materials. In terms of the Young modulus (E), the average value obtained from the tensile test was used, and the Poisson ratio of 0.3 was used for the material model. Figure 5.2 - Figure 5.7 show the engineering and true stress-strain curves obtained from the coupon tests and FE simulation for each steel grade with the distribution of von Mises stress and the plastic strain at fracture. The simulation results were almost perfectly matched with the test results. In the case of the coupler, a uniaxial tensile test of a mechanical joint was performed (refer to Section 4.2.2). The strength of the coupler was confirmed through the test, which showed that the coupler behaved completely in the elastic stage until the connected high-tensile SRBs fractured. Therefore, an elastic-perfectly plastic material model was used for the coupler based on its nominal yield stress ($F_y=900\text{MPa}$), as shown in Figure 5.8.

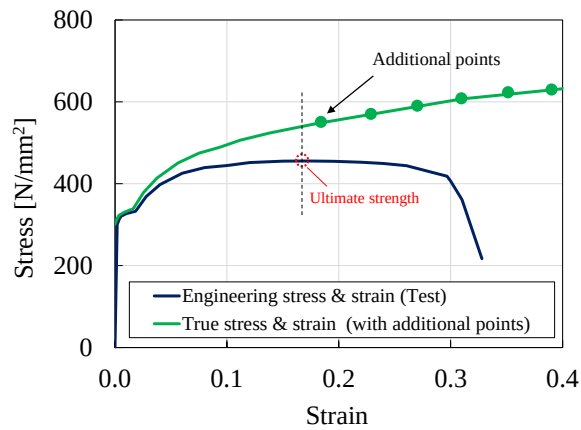
Table 5.1 shows the Abaqus input data (true stress and true plastic strain) of all steel grades for the beam-to-column joint model.



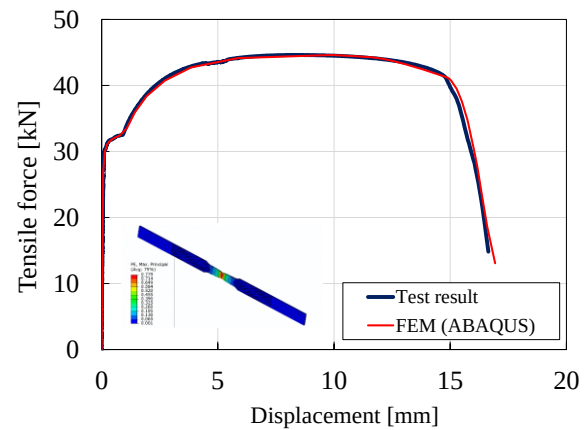
(a) Engineering stress-strain vs True stress-strain computed by equations C.1 from EN 1993-1-5



(b) Test result vs FE simulation result

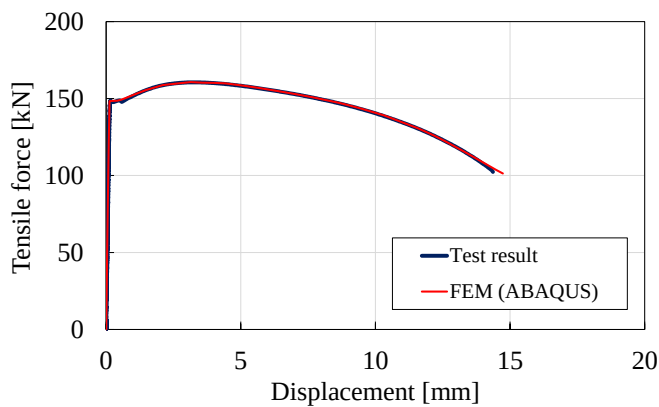


(c) True stress-strain with additional points for post-necking

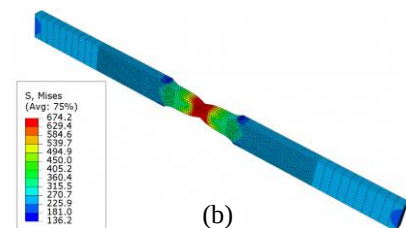


(d) Test result vs FE simulation result

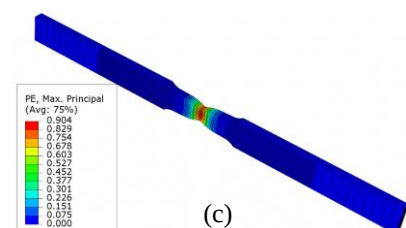
Figure 5.1: Calibration of the material model for steel (SRT275 - RHS ceiling beam 150X100X4)



(a)



(b)



(c)

Figure 5.2: Calibration of RHS column 200X100X12 (SRT355) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

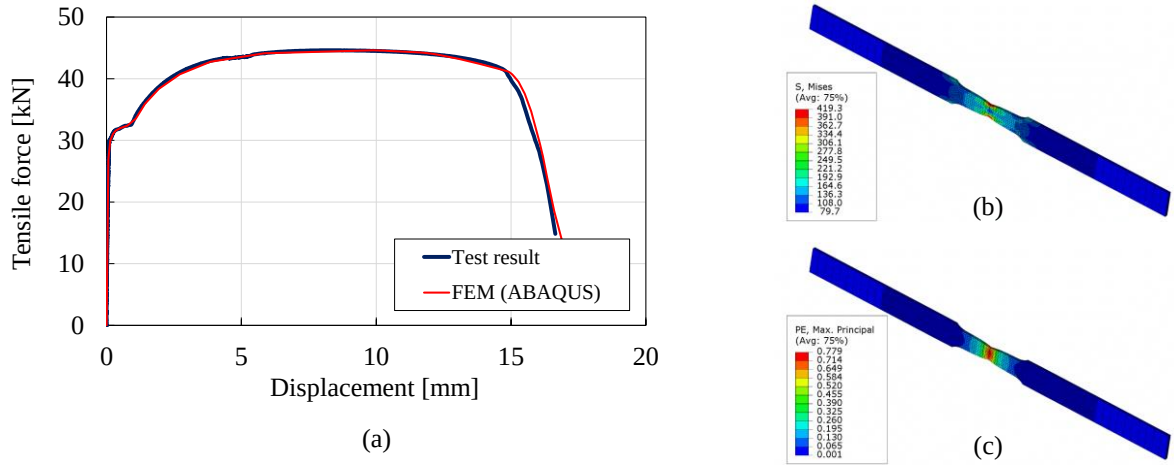


Figure 5.3: Calibration of RHS ceiling beam 150X100X4 (SRT275) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

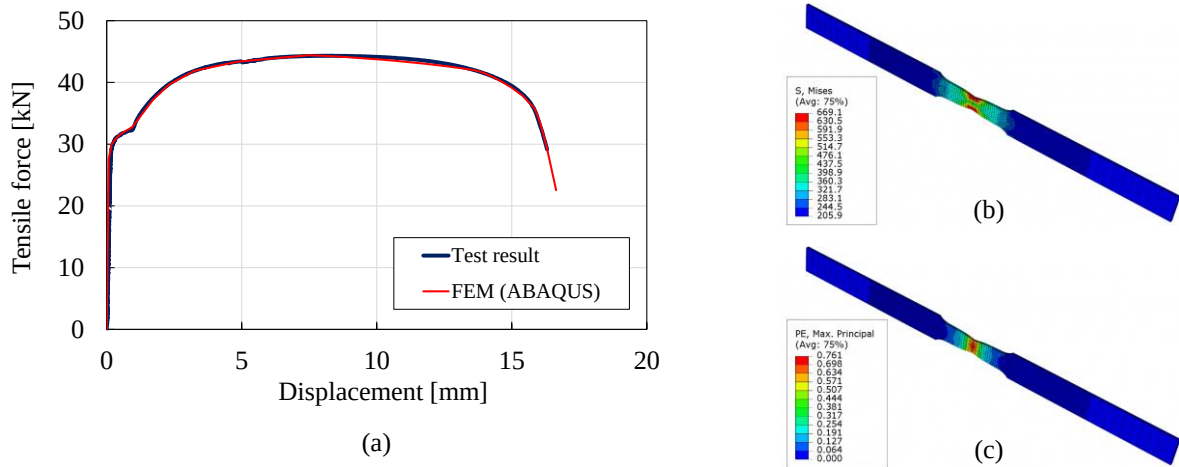


Figure 5.4: Calibration of RHS floor beam 200X100X4 (SRT275) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

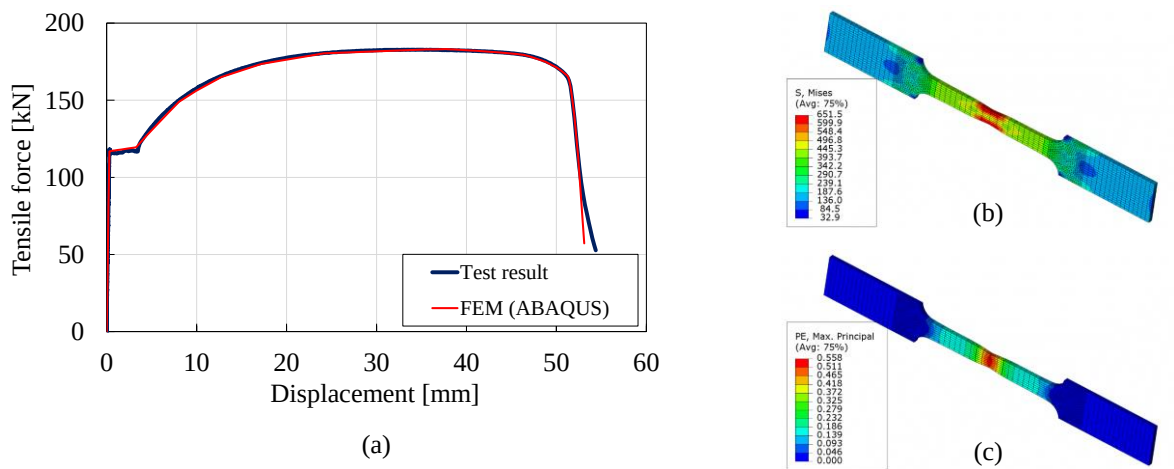


Figure 5.5: Calibration of a reinforcing plate (SS275) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

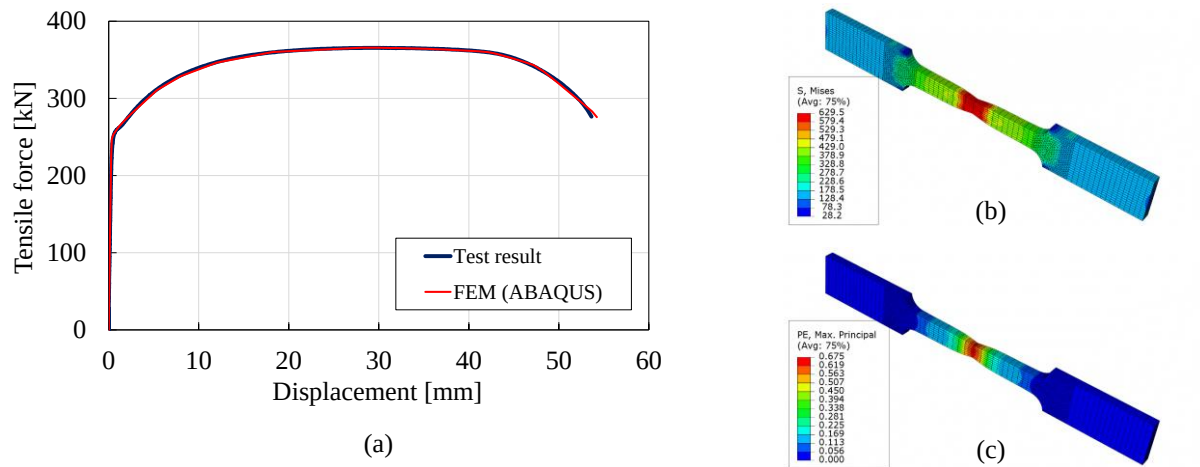


Figure 5.6: Calibration of a column endplate / connecting plate / cover plate (SS275) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

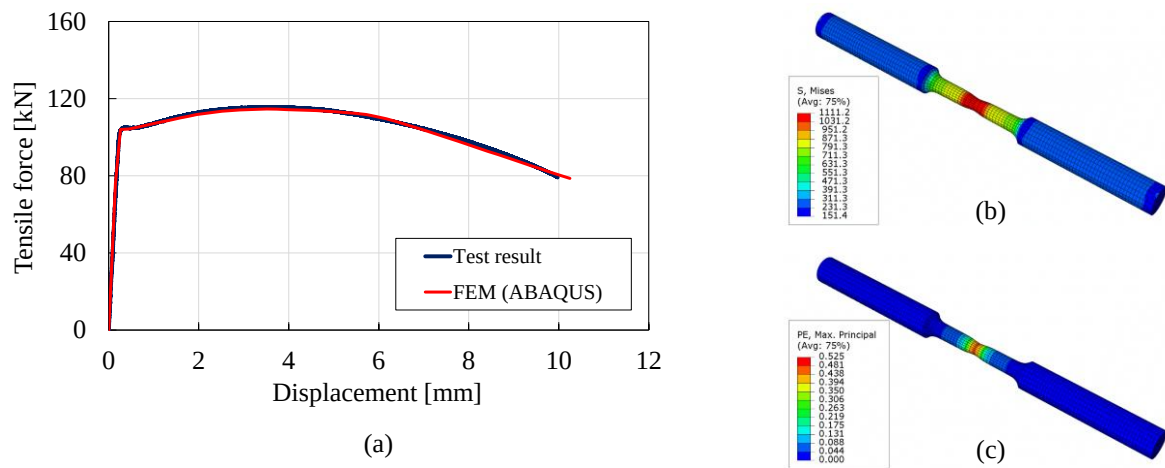


Figure 5.7: Calibration of Steel rod bolt (AISI 4140) material model: (a) Comparison between test and simulation; (b) Mises stress and (c) plastic strain at fracture

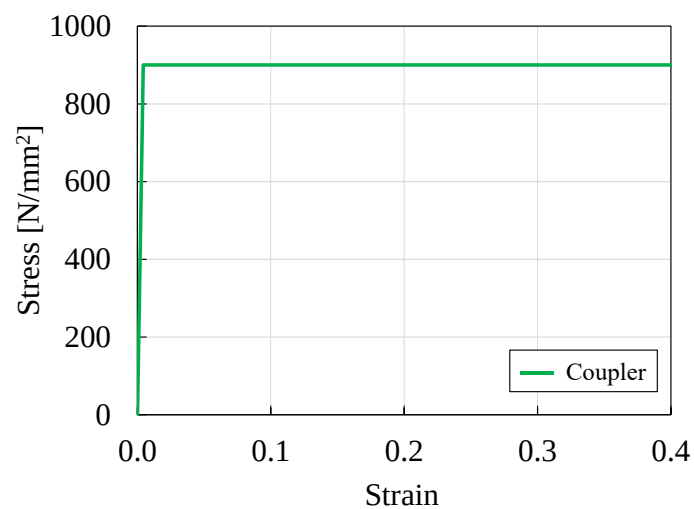


Figure 5.8: Elastic-perfectly plastic material properties for the coupler

Table 5.1: Input data for plastic behaviour of all components of the beam-to-column joint

Column		Ceiling beam		Floor beam		Reinforcing plate	
RHS 200X100X12 SRT355		RHS 150X100X4 SRT275		RHS 200X100X4 SRT275		Plate 220X120X10 SS275	
E = 207588 N/mm ²		E = 202813 N/mm ²		E = 214036 N/mm ²		E = 190107 N/mm ²	
True stress	True plastic strain	True stress	True plastic strain	True stress	True plastic strain	True stress	True plastic strain
N/mm ²	mm/mm	N/mm ²	mm/mm	N/mm ²	mm/mm	N/mm ²	mm/mm
496	0.000	301	0.000	314	0.000	301	0.000
502	0.010	315	0.002	336	0.007	313	0.016
513	0.015	322	0.003	346	0.014	328	0.018
549	0.036	330	0.008	353	0.016	401	0.038
572	0.062	336	0.013	422	0.035	447	0.057
580	0.083	339	0.016	460	0.054	479	0.076
590	0.100	379	0.026	484	0.072	503	0.094
655	0.350	414	0.037	499	0.091	520	0.112
685	0.550	451	0.056	522	0.118	534	0.130
670	0.800	475	0.075	538	0.143	545	0.147
		489	0.093	550	0.173	555	0.164
		506	0.112	685	0.500	565	0.180
		524	0.138	650	0.710	645	0.350
		534	0.157	80	0.850	660	0.430
		547	0.180			648	0.490
		607	0.310			80	0.560
		680	0.575				
		80	0.800				

Steel rod bolt		Column endplate / Connecting plate / Cover plate		Coupler	
D25 AISI 4140		Plate 220X120X20 SS275		*See Figure 4.16 SM45C (=F10T bolt)	
E = 195040 N/mm ²		E = 187863 N/mm ²		E = 21000 N/mm ²	
True stress	True plastic strain	True stress	True plastic strain	True stress	True plastic strain
N/mm ²	mm/mm	N/mm ²	mm/mm	N/mm ²	mm/mm
863	0.000	304	0.000	900	0.000
866	0.003	315	0.001		
870	0.005	330	0.004		
932	0.025	345	0.008		
962	0.035	408	0.027		
984	0.044	446	0.047		
1001	0.054	471	0.066		
1018	0.066	490	0.084		
1036	0.090	503	0.102		
1052	0.110	515	0.120		
1067	0.130	525	0.138		
1081	0.150	535	0.158		
1094	0.170	605	0.300		
1106	0.190	630	0.350		
1111	0.200	660	0.480		
		610	0.670		
		80	0.850		
		80	0.800		

5.3. Calibration of Numerical Models for Preload Beam-to-column Joints

5.3.1. Finite Element Model of the Preload Beam-to-column Joint

All components of the beam-to-column joint assemblies were modelled identically to the test specimen using solid elements, as shown in Figure 5.9 and Figure 5.10. The boundary conditions were set with roller and pinned supports at both ends of the columns. Static analysis was adopted for the welded joint model (W-SB). In the case of the AJ joint models (P-DB and P-SB), dynamic explicit analysis was used because the models had a large amount of contact surfaces, which made convergence difficult to achieve in using static analysis. All parts of the models are described below in detail.

Material model

Basically, the calibrated material models in Section 5.2 was used, which is shown in Table 4.1. However, depending on the type of loading (monotonic or cyclic loading), different steel constitutive models were used. In the beam-to-column joint model under monotonic loading, the isotropic hardening model was used for steel components [115, 116] in which the true stress and strain obtained from the tensile coupon test became input data. On the other hand, a combined isotropic-kinematic hardening model was adopted for the joint model under cyclic loading, which gives a good agreement with experiments [117-120]. The combined isotropic-kinematic hardening model requires a set of material parameters (i.e. for kinematic hardening: C_k = kinematic hardening modulus; γ_k = dynamic recovery term associated with non-linear kinematic hardening, and for isotropic hardening: Q = representative of the maximum change in size of the yield surface; b = parameter determining the rate of change of size of the yield surface) which are evaluated by cyclic tests of material samples. Since the cyclic tests were not conducted in this study, the half-cycle method, which allows C_k , and γ_k to be obtained from the stress-strain data of a standard tensile coupon test, was used for the kinematic parameters. The parameters for the isotropic hardening ($Q=117$ and $b=14.3$ for steel grade of SS275) were obtained from the literature [121]. The combined isotropic-kinematic hardening model was applied to the RHS floor and ceiling beams, which are the dissipative members, in all FE joint models under cyclic loading.

Geometry, element types and meshes

The geometry corresponded to the nominal dimensions of the drawings of W-SB, P-DB, and P-SB (see Figure 4.36 and Figure 4.37). Linear hexahedral elements of type C3D8R were adopted for all components in the joint models. In particular, to capture a more precise response and results, the size of the mesh in the region of the beam plastic hinges, the column panel, and slot connection for the floor ceiling beams was reduced to 4-6mm. For the coupler, the steel rod bolt and the plate, the mesh size was 3-5mm.

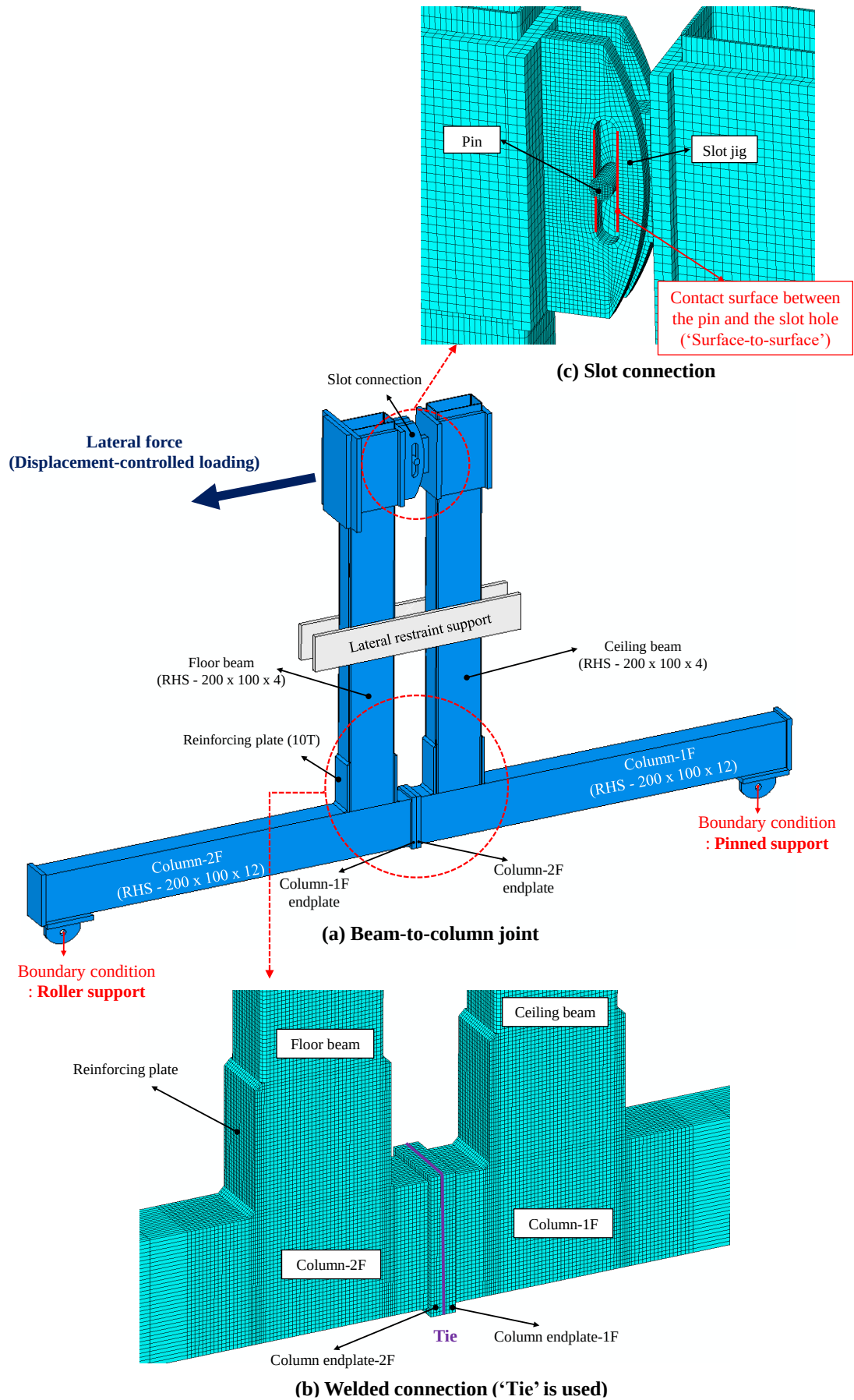
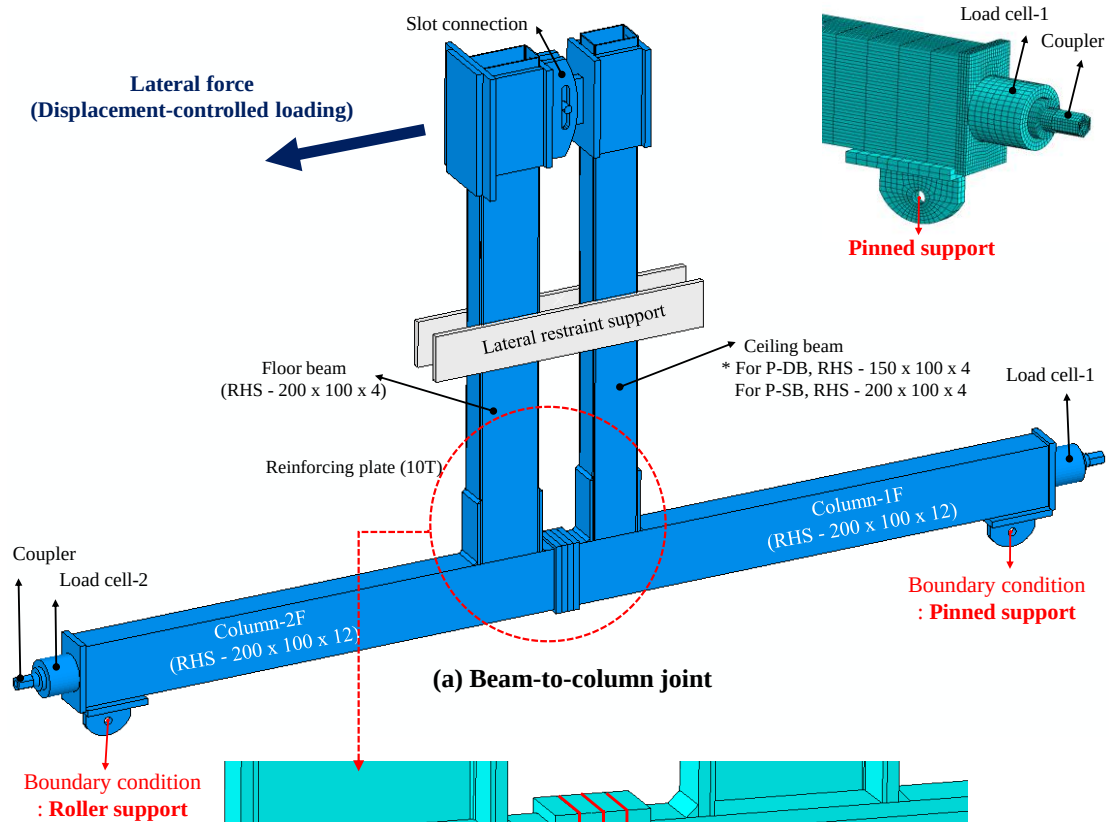
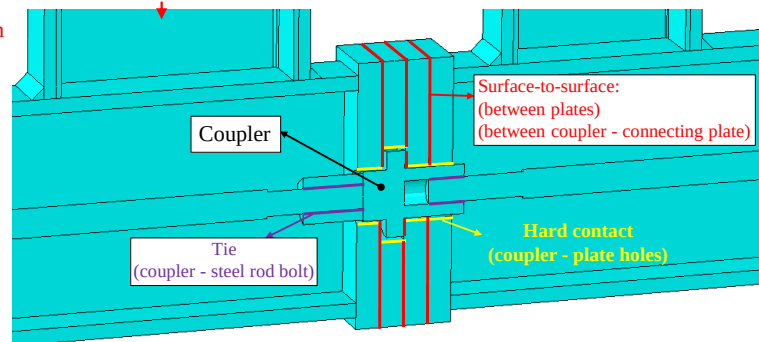


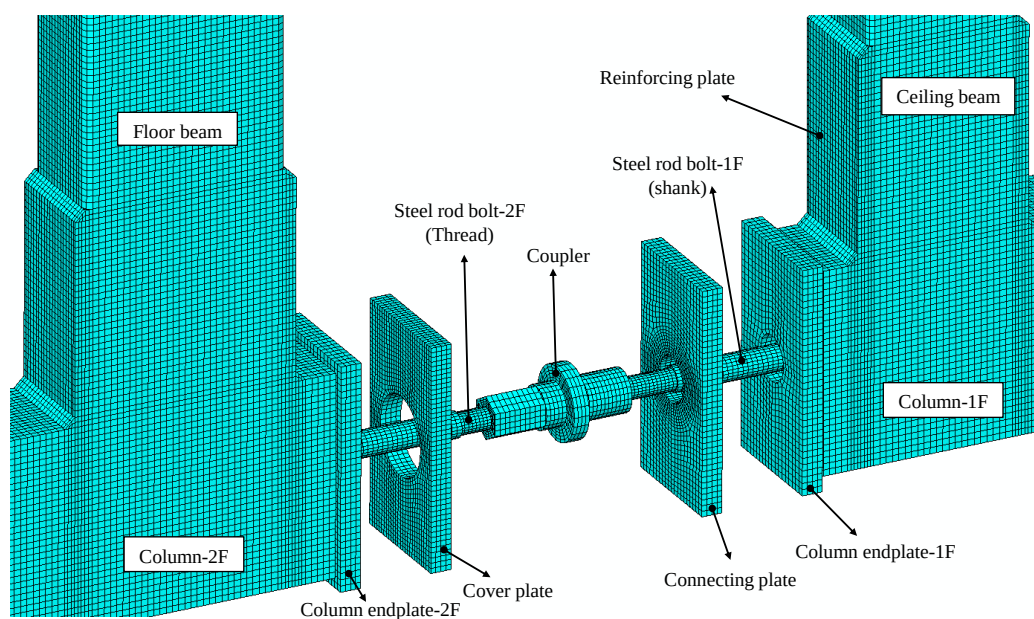
Figure 5.9: Finite element model for W-SB (welded connection)



(a) Beam-to-column joint



(b) Contact setting (Cross-sectional view)



(c) Components at preload connection

Figure 5.10: Finite element model for P-DB and P-SB (AJ connection)

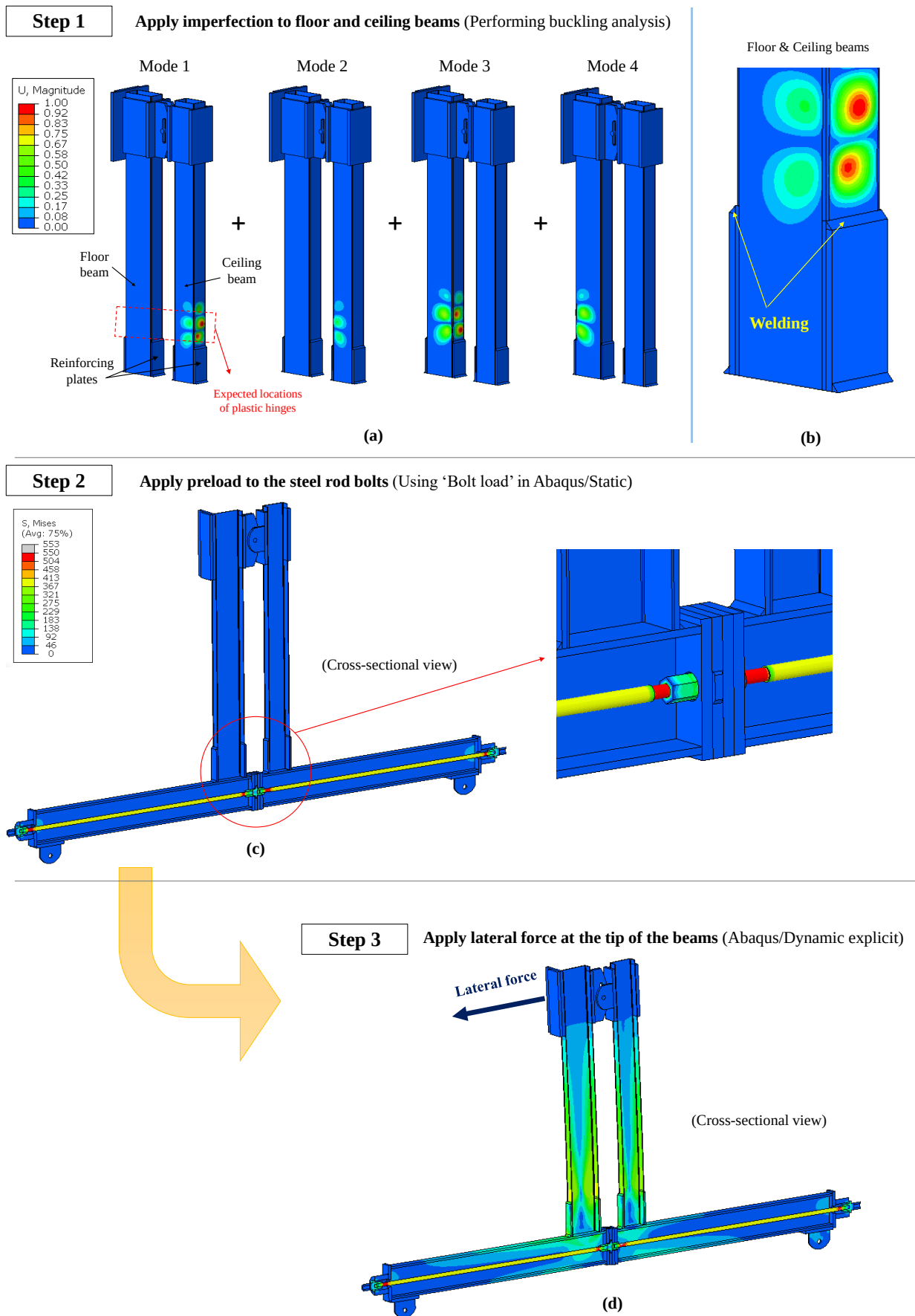


Figure 5.11 Analysis process of the preloaded joint model (P-DB and P-SB)

Contact properties

"Surface-to-Surface" defined by tangential and normal behaviour was used for the contact surfaces where two components are likely to slide and have associated friction. The tangential behaviour was defined as penalty friction, and the normal behaviour was defined as "Hard" contact. The tangential friction coefficient was set as 0.3, which was for cleaned surfaces by wire-brushing with loose rust removed [100]. The contact surfaces between the coupler and the plate holes were defined as "Hard" contact using "General contact". Figure 5.10 (b) presents the specific descriptions.

Constraint

"Tie" constraint is normally used to make two separate surfaces deform together without having any relative displacement. So, this constraint was used to represent welded connections (i.e. the connections between the beam and column, the beam and reinforcing plate, the column and reinforcing plate, and the column and end plate). It was also used for parts where a SRB was fastened to the coupler as shown in Figure 5.10 (b).

Loading process of the FE joint models

The lateral loading was applied at the tip of the beams by displacement-controlled loading. In the case of the welded joint model (W-SB), only lateral loading was applied in Abaqus/Static. On the other hand, the preloaded joint model (P-DB and P-SB) consisted of two loading steps (see Figure 5.11 (c) and (d)). In the preloading stage, the preload was applied to the SRBs in the columns using "Bolt load" in Abaqus/Static. In the second step, the lateral loading was applied using Abaqus/Dynamic explicit. "Bolt load" is not available in dynamic explicit. So, after completing the application of preload to the FE joint model in Abaqus/Static, using "Predefined field" tool, the analysis result was used as the initial state of stress in the joint model for the second analysis step in dynamic explicit. For monotonic loading, the displacement was gradually increased until failure. For cyclic loading, the ANSI/AISC 341-10 loading protocol was used (see Figure 4.47), as was also used in the beam-to-column joint test.

Types of structural analyses for the FE joint models

Different types of structural analyses are shown in Figure 5.12 to demonstrate the effects of material and geometric nonlinearities. Among them, the geometrically and materially nonlinear analysis with imperfections (GMNIA) is considered the most sophisticated and accurate structural analysis method available to verify the strength capacity of a structure, accounting for both plasticity and buckling failure modes [122-124]. Therefore, GMNIA is used for the FE beam-to-column joint model in this chapter.

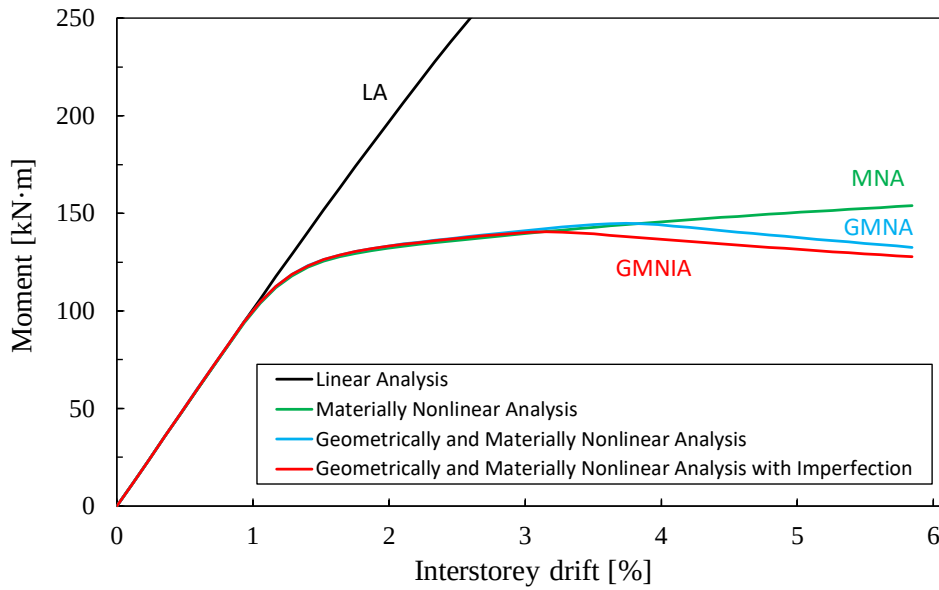


Figure 5.12 Different methods of structural analysis (ex. W-SB model under monotonic loading)

Geometric imperfection for the RHS floor and ceiling beams

Most RHSs have an approximately ‘bow-out’ imperfection varying sinusoidally along the length of the member, as shown in Figure 5.13 (a) and (b) [125, 126]. This imperfection is applied locally at the expected hinge location on the RHS floor and ceiling beams near the reinforcing plates (i.e. 50mm away from the end of the reinforcing plates). The buckled shape was obtained from an eigenvalue buckling analysis, and the final buckling shape is determined by the linear superposition of various eigenmodes as shown in Figure 5.11 (a). Generally, the imperfection size for the FE model may be determined based on the value providing the most accurate correlation to the test results. However, because the test results of the beam-to-column joint in Section 4.4 already included other factors affecting the results, it was difficult to find an accurate imperfection size by comparing the analysis to the test results. In addition, according to the literature [125], localised imperfections are induced in the RHS section near welds because of differential cooling after welding. In the beam-to-column joint in this study, reinforcing plates were welded to both floor and ceiling beams near the expected hinge location. Those hinge locations may have been affected by localised imperfections due to the welding, which were not measured. Therefore, the imperfection size for the RHS floor and ceiling beams were determined by referring to the literature [125, 126] as $\delta_f = b/1500$ for flange and $\delta_w = d/1500$ for web (see Figure 5.13 (a)). The effect of the imperfection is shown in Figure 5.12. The GMNIA model of W-SB shows the reduced ultimate strength and rotation capacity due to the imperfection compared to the GMNIA model. The same magnitudes of the imperfection were applied to the RHS floor and ceiling beams in all joint models.

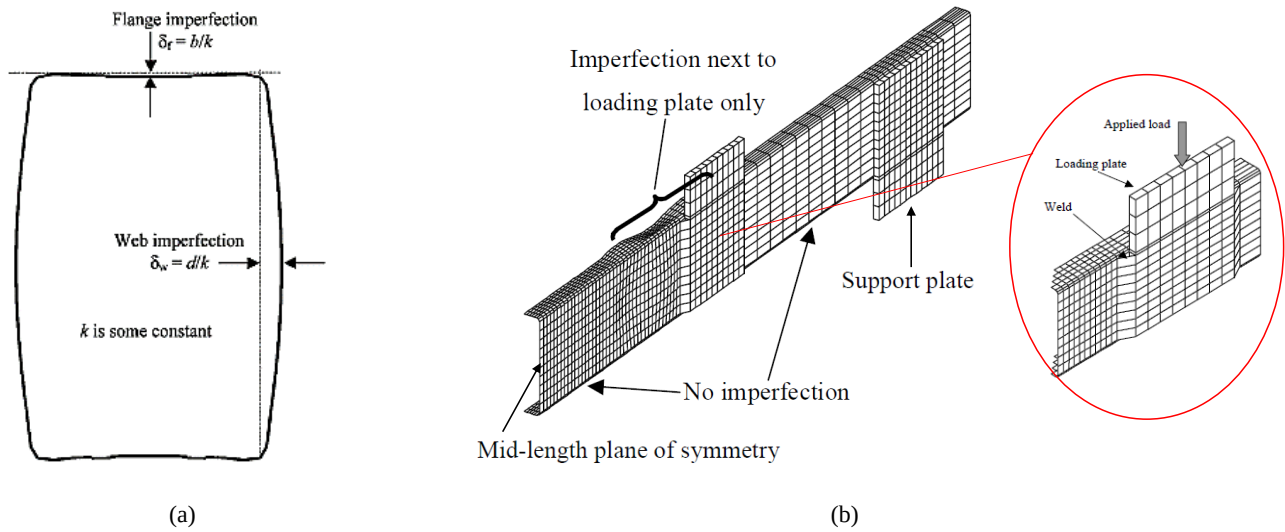


Figure 5.13: (a) 'Bow-out' imperfection in RHS, and (b) RHS with imperfection close to the loading plate (due to welding) [125, 126]

Considerations for dynamic explicit analysis

Dynamic explicit analysis, while advantageous in handling a large amount of contact surfaces, necessitates careful attention to details such as time step determination, loading rates, and mass scaling in order to obtain stable curves and reduce simulation runtime in quasi-static simulations. The process adopted in this study to address this is detailed below:

The time step in dynamic explicit analysis is determined by the smallest element in the model and the speed of wave propagation through this element. By meticulously modelling and meshing each component of the joint, and ensuring proper connectivity between nodes of adjoining components, the smallest mesh size was identified. Initially, a monotonic loading simulation was conducted using Abaqus/Explicit at a relatively slow loading rate without the application of mass scaling. To see if inertial effects could be neglected, the kinetic energy output was requested from the simulation results, monitoring whether the kinetic energy represented a very small fraction of the total energy. Once a stable curve was obtained through this process, it was set as a reference curve for optimising the simulation runtime. With this reference curve, subsequent simulations were executed iteratively by gradually increasing the loading rate and applying mass scaling to find the optimised simulation runtime until stable outputs mirroring the reference curve were achieved. Regarding the mass scaling, it was limited to a value less than ten times the size of the smallest element. Lastly, based on the optimised parameters derived from the monotonic loading simulations, quasi-static cyclic loading simulations were performed.

The extensive simulations required significant computational costs. Their feasibility was made possible only due to the availability of the High-Performance Computing (HPC) facility at the University of Sydney, which played a crucial role in obtaining stable curves from the dynamic explicit analysis, especially for the cyclic loading simulations.

5.3.2. Investigation of the Causes Affecting the Experiment

As mentioned in Section 4.4.10, all test specimens for the beam-to-column joint were affected by unintended influencing factors of the test set-up, resulting in much greater stiffness and strength than expected. In order for the beam-to-column joint model to be calibrated based on the test results, it is necessary to confirm the causes affecting the test results and their specific effects. There were two possible causes:

- (1) Partial rotational restraint of the actuator pin (see Figure 4.90);
- (2) Lack of free movement of the pin in the slot connection between floor and ceiling beams, preventing the two beams to behave individually (see Figure 4.90).

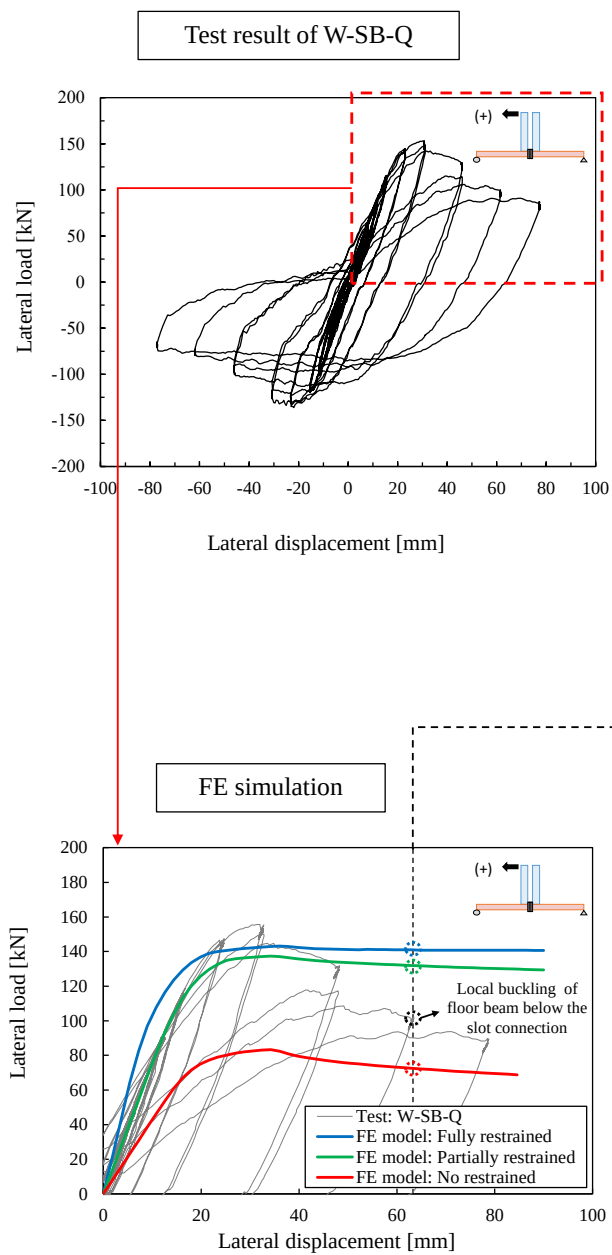
The welded joint model (W-SB) was used for this study because it does not have slip and gap-opening at the column-to-column connection, and hence allowed a focused study of the abovementioned effects on the joint behaviour.

5.3.2.1. Effect of the Rotationally Restrained Pin of the Actuator

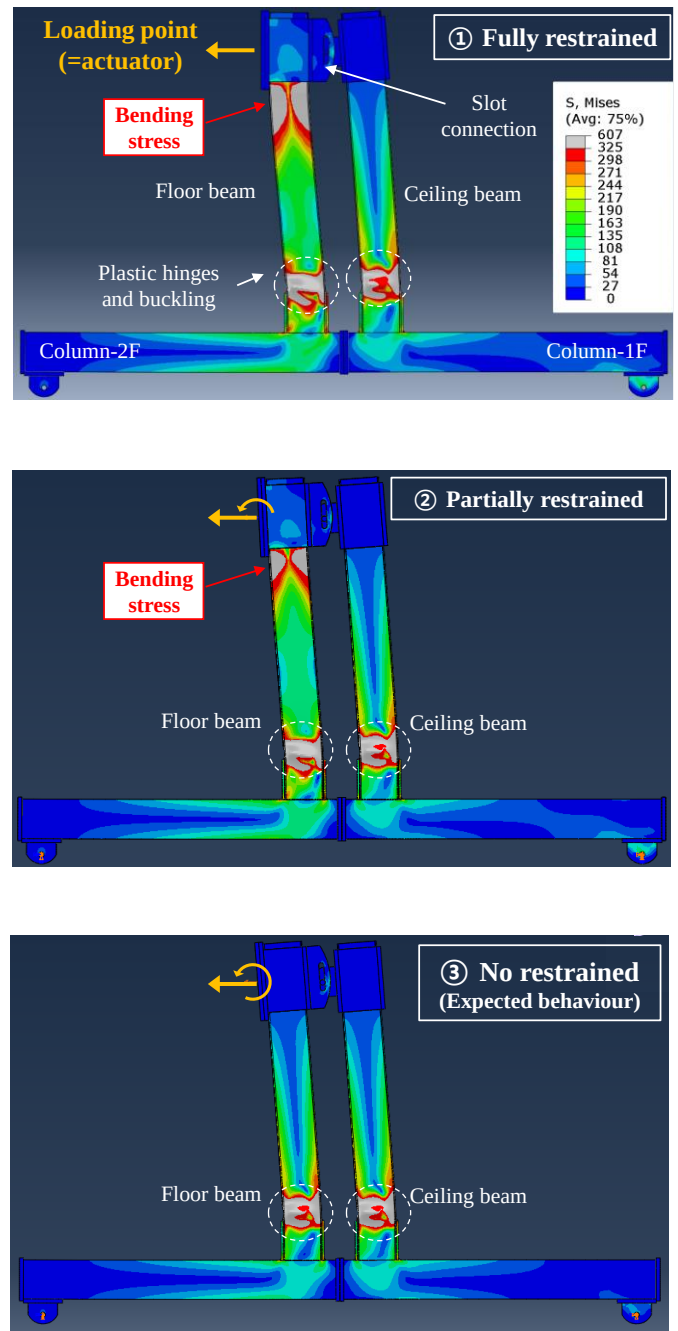
In order to investigate the effect of the partially restrained actuator rotation, the rotation of the loading point was set as a parameter in the FE model as follows:

- (1) Fully restrained against rotation;
- (2) Partially restrained against rotation;
- (3) Not restrained against rotation (freely rotating at the tip of the beams, which was the intended behaviour in the experiment).

Figure 5.14 (a) presents the hysteretic curves of W-SB-Q and the FE simulation results of W-SB under monotonic loading. The simulation results showed that the stiffness and strength increased as the rotational restraint increased. In the case of the FE model with the partially restrained rotation, the stiffness was similar to that obtained in the test. (It was found through several trials by changing the stiffness of the partial restraint.) In addition, as the rotational restraint of the loading point increased, significantly large bending stress occurred right below the tip of the floor beam connected to the loading point (i.e. actuator), as shown in Figure 5.14 (b) ①. During testing of the W-SB-Q specimen, it was observed that buckling occurred at the exact location where the bending stress appeared in the FE simulation (see Figure 4.88), thus confirming the suspected presence of rotational restraint of the actuator. In the FE simulations, the effect of the partial rotational restraint of the actuator was implemented by changing the boundary condition of the tip of the floor beam, which was subjected to the applied lateral force, from a pin to a semi-rigid connection, as shown in Figure 5.15. On the other hand, the tip of the ceiling beam was free to rotate by the pin moving downward in the slot connection.



(a) Comparison of the test result with FE simulation results



(b) Stress distribution (Mises stress) at the lateral displacement of 62mm

Figure 5.14: Effect of the actuator rotation on the response of the beam-to-column joint

The strength and stiffness of the beam-to-column joint increased significantly due to the change of the boundary condition of the tip of the floor beam. In terms of the failure mechanism, both the floor and ceiling beams in all FE models yielded and formed plastic hinges at the expected locations, as also observed in the test of W-SB-Q. Figure 5.16 shows the failure shape from the test and the FE simulation of W-SB. By comprehensively demonstrating agreement between the test results and the FE simulation results (i.e. considering strength, stiffness, location of local buckling of the floor beam right below the slot connection, and failure mechanism), it is believed that the main cause affecting the test results was the partial rotational restraint of the actuator. It seemed that the heavy weight of the actuator, which was of a substantially larger size than the test specimens, implied friction in the joint supporting it and partially restrained the rotation of the end of the floor beam.

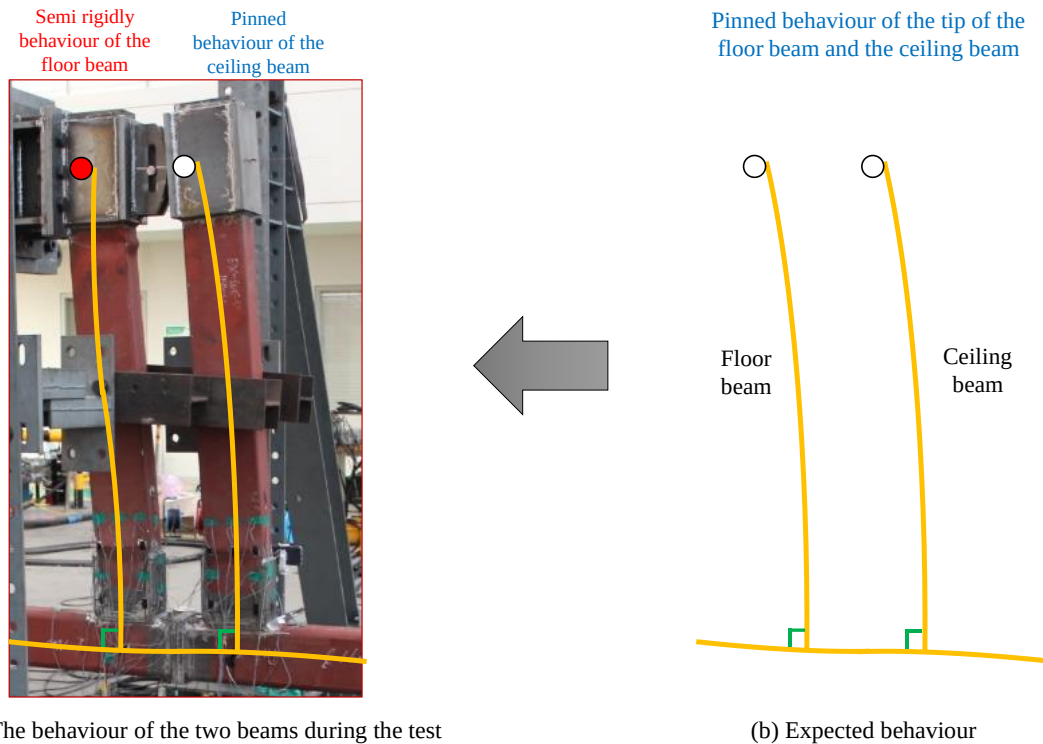
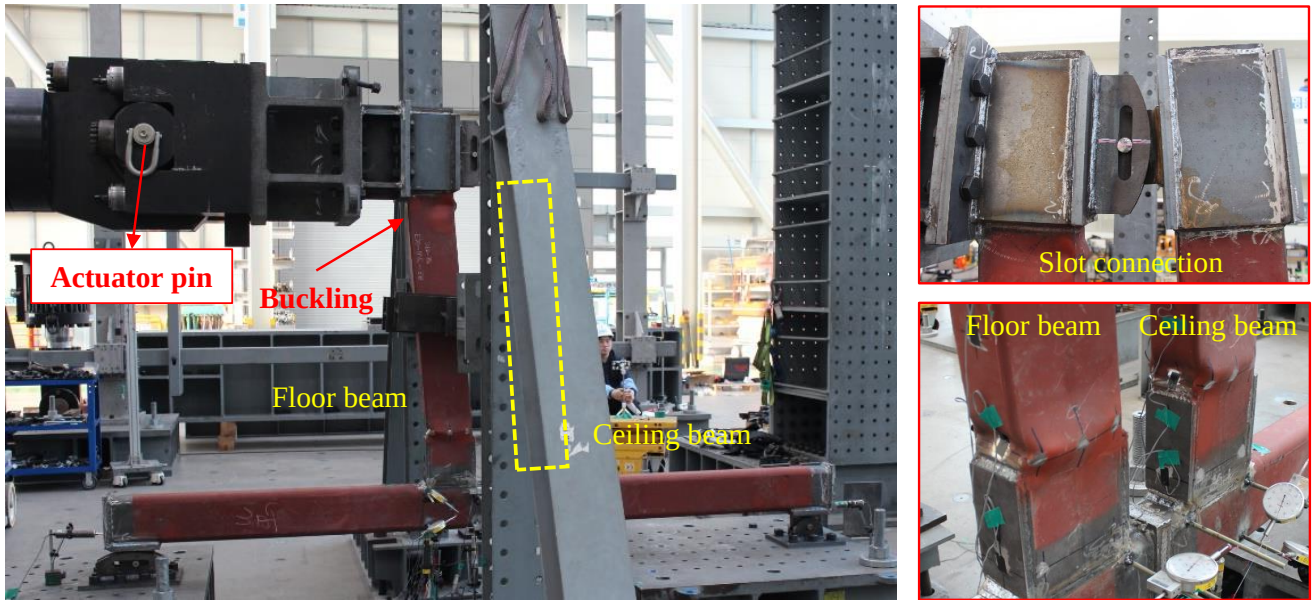
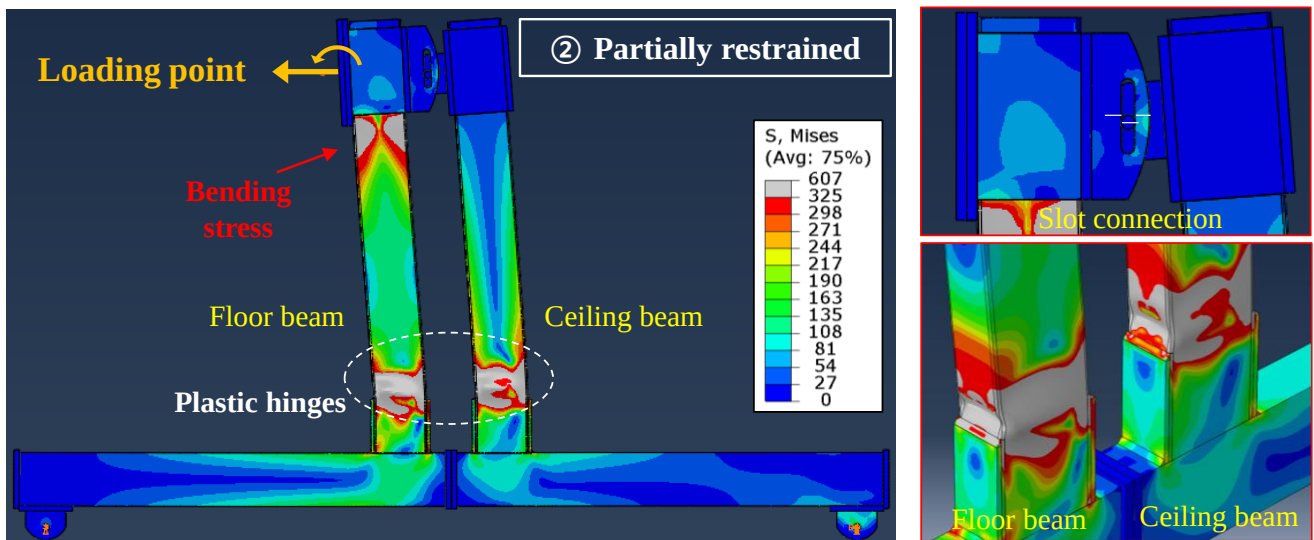


Figure 5.15: Change of the boundary condition of the tip of a floor beam due to the partial rotational restraint of the actuator



(a) Test result of W-SB-Q



(b) FE simulation result (Mises stress distribution)

Figure 5.16: Comparison of the test result with the FE simulation result for W-SB at the lateral displacement of 60mm

5.3.2.2. Lack of Free Movement of the Pin at the Slot Connection

Another possible effect on the test results is the lack of free movement of the pin at the slot connection because it was observed that the pin dropped suddenly in the slot connection with a loud sound in most of the specimens during the tests. If the pin in the slot connection, which was installed to allow free vertical movement of the floor and ceiling beams, did not move sufficiently, it would cause an increase in the stiffness and strength of the joint by the partial composite effect on the two beams.

There are two possible causes that may have prevented the smooth movement of the pin in the slot connections:

- (1) Slightly smaller width of the slot hole than the nominal value due to manufacturing tolerance;
- (2) Friction on the contact surfaces between the pin and the slot hole.

In order to investigate the above effects on the response of the beam-to-column joint, the width of the slot hole (w) and the friction coefficient (μ) were set as parameters in the FE model, as shown in Figure 5.17 (a) and (b).

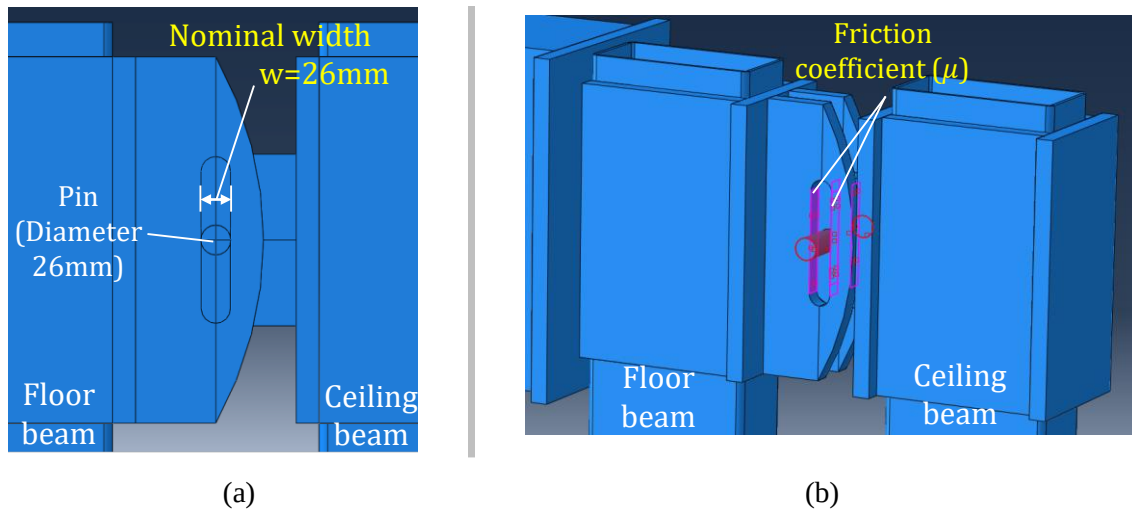


Figure 5.17: Parameters in the FE model: (a) Width of the slot hole; and (b) Friction coefficient on the contact surfaces between the pin and the slot hole.

Figure 5.18 presents the simulation results for the effect of the width of the slot hole. The stiffness and strength of the beam-to-column joint increased as the width of the slot hole was reduced. Although the FE model of $w=25.0\text{mm}$ showed a similar stiffness and strength to the test result, the failure mechanism differed. In the FE model, only the floor beam experienced local buckling (see Figure 5.18 (b)), while in the test of W-SB-Q, both beams experienced local buckling (see Figure 5.16 (a)). In addition, the slot hole was manufactured by a laser cutting machine, which generally has a tolerance $\pm 0.1\text{--}0.5\text{mm}$ [127–130] for 20mm thickness of a steel plate. Therefore, considering the failure mechanism of the joint and the manufacturing tolerance of the laser cutting machine, the smaller width of the slot hole due to the manufacturing tolerance was not regarded as the primary cause affecting the test results.

Figure 5.19 shows the simulation results for the effect of friction on the contact surfaces between the slot hole and the pin. According to the literature, the friction coefficient (μ) between non-lubricant mild steel surfaces ranges between 0.3 and 0.75 [100, 131]. Figure 5.19 (a) presents that the stiffness and strength increased as the friction coefficient increased. However, compared to the test results, the effect of friction on the stiffness and strength of the joint was quite small. While the width of the slot hole and the friction on the contact surface between the pin and the slot hole could influence the test results by a small amount, it is concluded that it is highly unlikely to be the major influencing parameters.

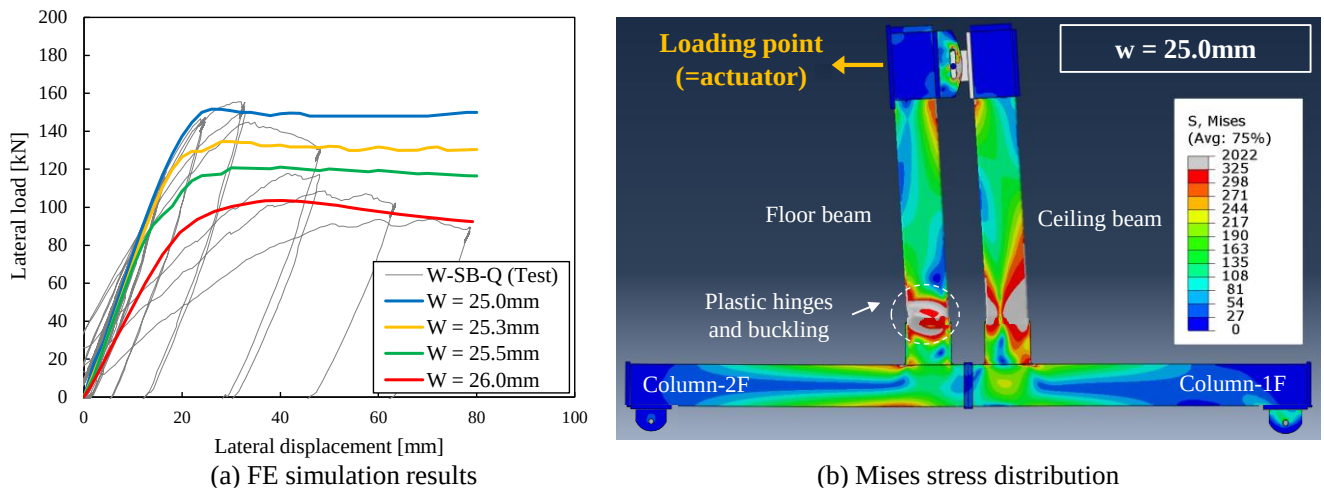


Figure 5.18: Comparison of the simulation results with the test result for the effect of the width of the slot hole (refer to Figure 5.17 (a))

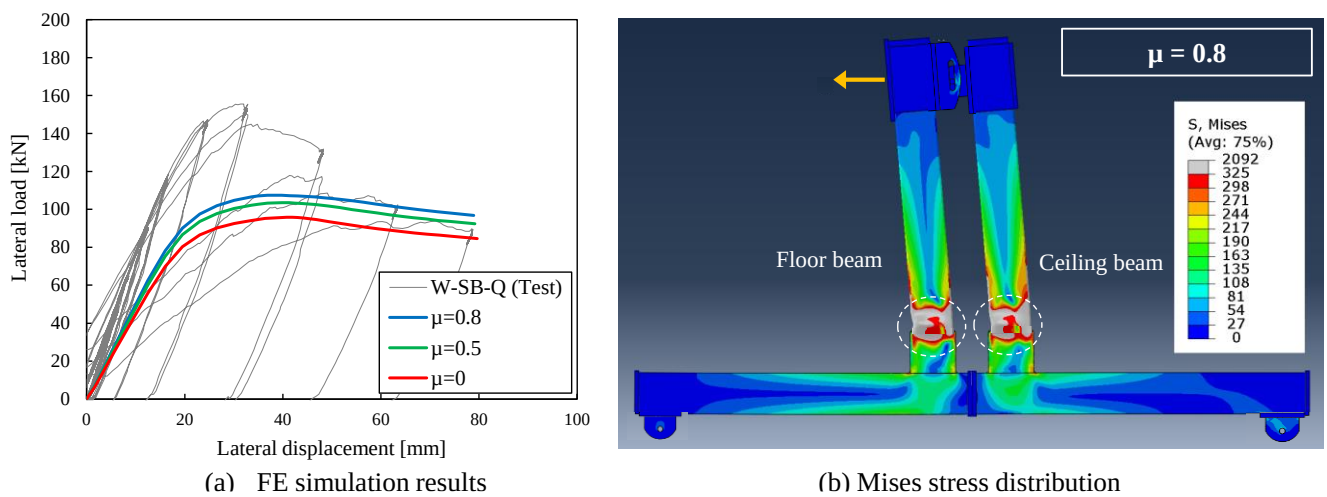


Figure 5.19: Comparison of the simulation results with the test result for the effect of the friction coefficient (μ) on the contact surfaces between the slot hole and the pin (refer to Figure 5.17 (b))

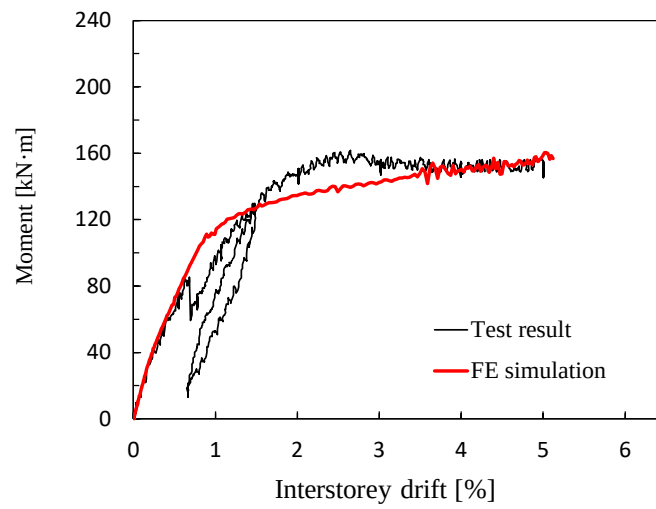
5.3.2.3. Conclusion and Recommendations

The comparative analysis of the FE simulation with the test result shows that the likely main cause affecting the beam-to-column joint tests was the partial rotational restraint of the actuator. It seems that the relatively large size and heavy self-weight of the actuator compared to the test specimens prevented it from rotating freely while applying lateral load to the specimen. In addition, the width of the slot hole and friction on the contact surfaces between the pin and the slot hole might have affected the test results but most likely only to a small extent.

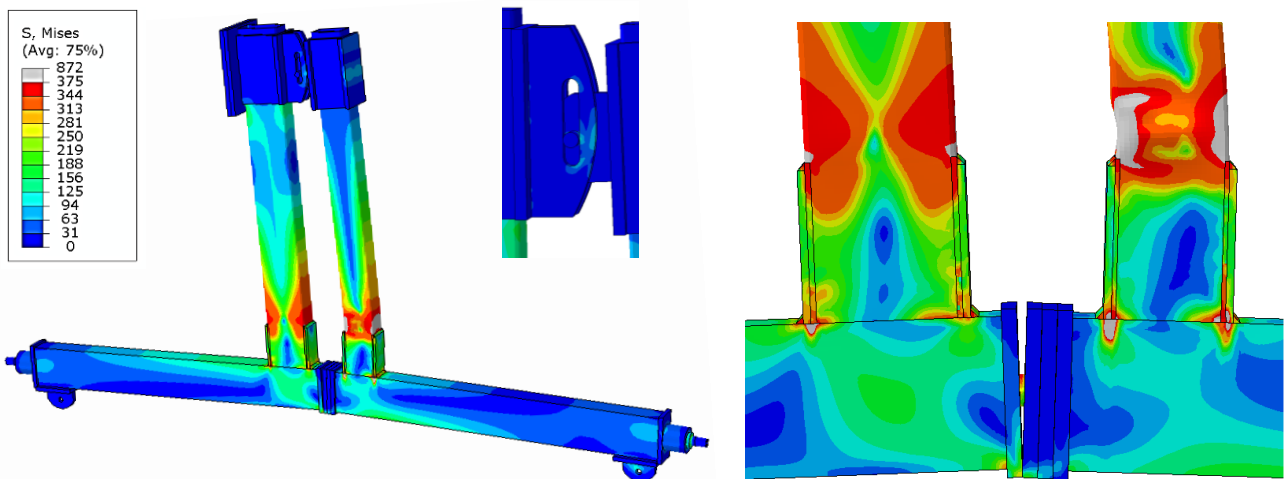
For the beam-to-column joint test of modular building structures, it is difficult to achieve true roller supports allowing free relative movement of the floor and ceiling beams. If the test setup in this study is considered for the exterior beam-to-column joint test in the future, it is recommended to use an actuator with appropriate capacity, size, and weight commensurate with the properties of the test specimen. In addition, in terms of the slot connection between the floor and ceiling beams, it is suggested to make the diameter of the pin slightly smaller (e.g. 0.5 mm) than the width of the slot hole to prevent the pin from being stuck in the slot hole in order to ensure the free relative movement of the two beams. It is also recommended to lubricate the contact surfaces between the pin and the slot hole to minimise friction.

5.3.3. Calibration of Numerical Models for the Beam-column Joint

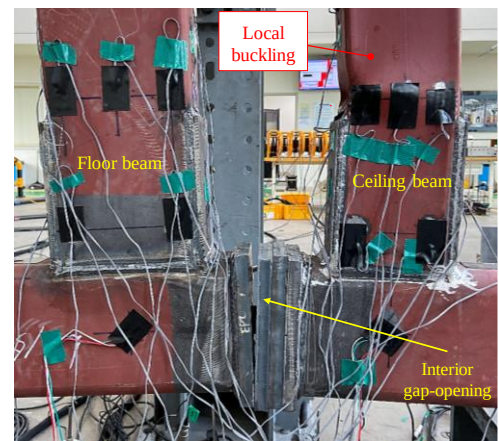
Considering the effects of partial rotational restraint of the actuator, the FE joint model was calibrated based on the beam-to-column joint test results. In terms of friction between plates at the AJ connection, a friction coefficient of 0.3 was used. Also, for the contact surfaces between the pin and slot hole in the slot hole connection, a friction coefficient of 0.3 were applied as well. Since it was not possible to know exactly how much the actuator was rotated while the lateral force was applied to the test specimens during the test, and since slip and gap-opening occurred simultaneously at the connection, it was difficult to reproduce the test results accurately. Therefore, the calibration was focused on the initial stiffness and failure mechanism (e.g. formation of the plastic hinge on the beam, and the location and shape of the gap-opening) for the AJ joint. Figure 5.20, Figure 5.21, and Figure 5.22 show the FE simulation results from the calibration under monotonic and quasi-static cyclic loading.



(a) Moment-interstorey drift curve

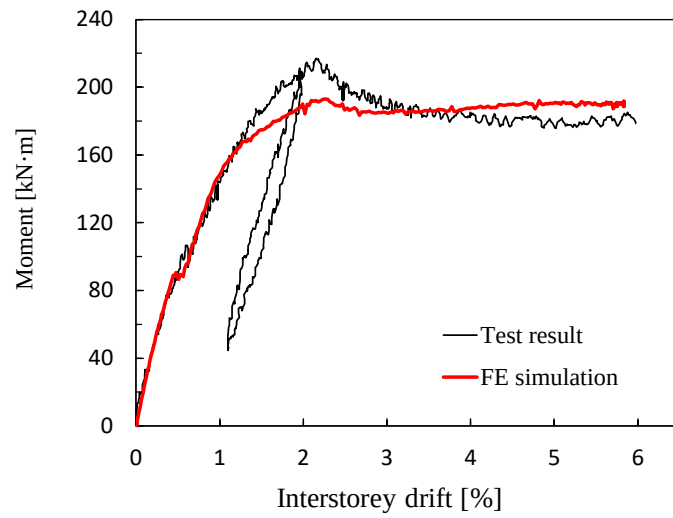


(b) FE simulation - Mises stress distribution at drift of 5%

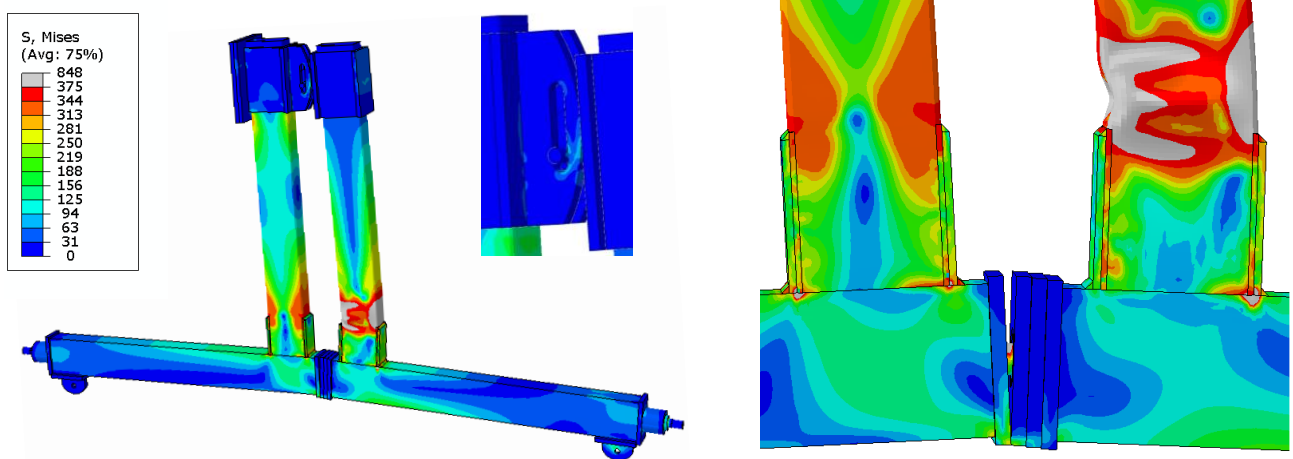


(c) Test result at drift of 5%

Figure 5.20: Calibration result for P-DB-M



(a) Moment-interstorey drift curve

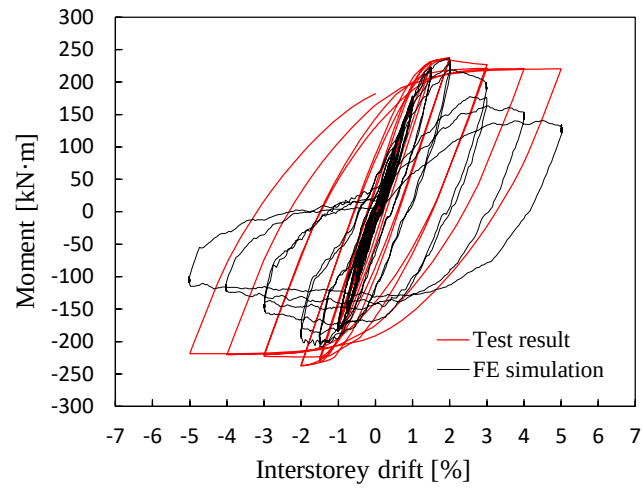


(b) FE simulation - Mises stress distribution at drift of 6%

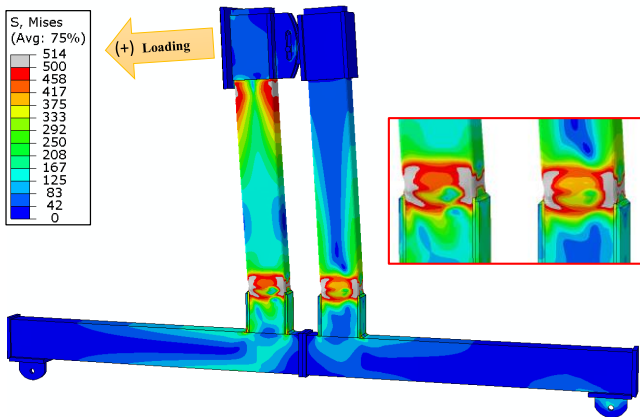


(c) Test result at drift of 6%

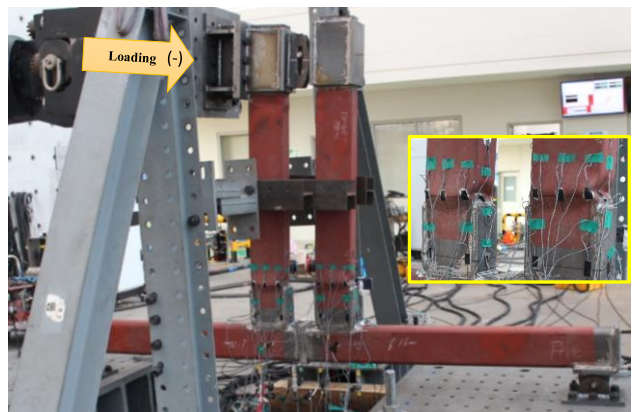
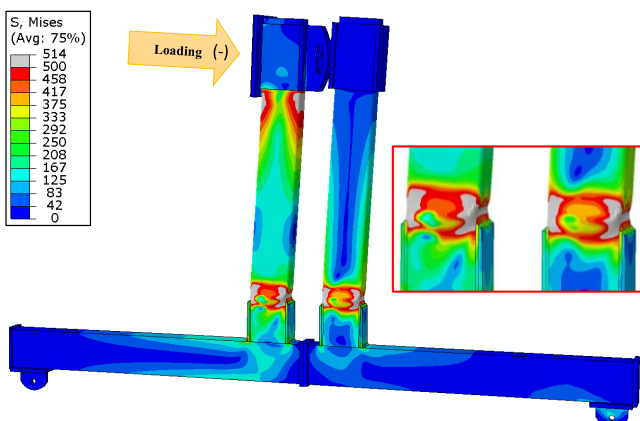
Figure 5.21: Calibration result for P-SB-M



(a) Moment-interstorey drift curve



(b) FE simulation and test result at drift 5% in (+) loading direction



(c) FE simulation and test result at drift 5% in (-) loading direction

Figure 5.22: Calibration result for W-SB-Q

5.4. Re-evaluation of Seismic Performance of the AJ Connection

Using the calibrated FE beam-to-column joint models, the performance of the AJ joints was re-evaluated under monotonic and cyclic loading by removing the factors that affected the test results. Additionally, a welded joint model of W-DB was analysed for comparison with P-DB. The demand preload ($P_g=188\text{kN}$) was applied to the P-DB. For P-SB, since it had a gap-opening when the demand preload ($P_g=131\text{kN}$) was applied in the pre-test simulation, a 10% higher preload of 145kN was used.

There are two issues to point out before discussing the simulation analysis results as set out below.

Firstly, slightly different strength degradation results were obtained for the AJ joints and welded joints after passing the ultimate strength under monotonic and cyclic loading simulation. It appeared that the difference was caused by using different analysis solvers (i.e. Static vs Dynamic explicit).

Secondly, the beam-column welded connection in a steel-framed modular building is susceptible to weld fracture due to the stress concentration in the corners of the connection, leading to brittle failure under cyclic loading [58, 60] and a decrease in the ductility of the joints. Therefore, the base metal capacity and the weld quality are quite important for the ductile behaviour of the joint. In the present study, in order to prevent weld fracture at the beam-column connection and ensure the formation of plastic hinges in floor and ceiling beams, reinforcing plates were installed on both beams in all joint specimens. The beam-to-column joint test results showed that the weld quality at the beam-column connection enabled plastic hinges to develop in both floor and ceiling beams without any weld fracture or even cracks occurring until the beams failed, (refer to Section 4.4.9). So, fracture of the weld at the beam-column connection was not considered in the FE beam-to-column joint models for re-evaluating the seismic performance of joints.

5.4.1. Simulation of Monotonic Loading

The FE joint models used for pre-test simulation (see Section 4.4.8) are also used for re-evaluating the monotonic tests in this section, except for two differences: the actual material properties obtained from the tensile coupon tests are applied and imperfections are introduced to the RHS floor and ceiling beams. The simulation results (overall joint behaviour, failure mode, stress variation in SRB, etc.) are similar to the pre-test simulation results, except for the joint strength and rotational capacity. Therefore, the simulation results are briefly discussed in this section.

5.4.1.1. Simulation Results

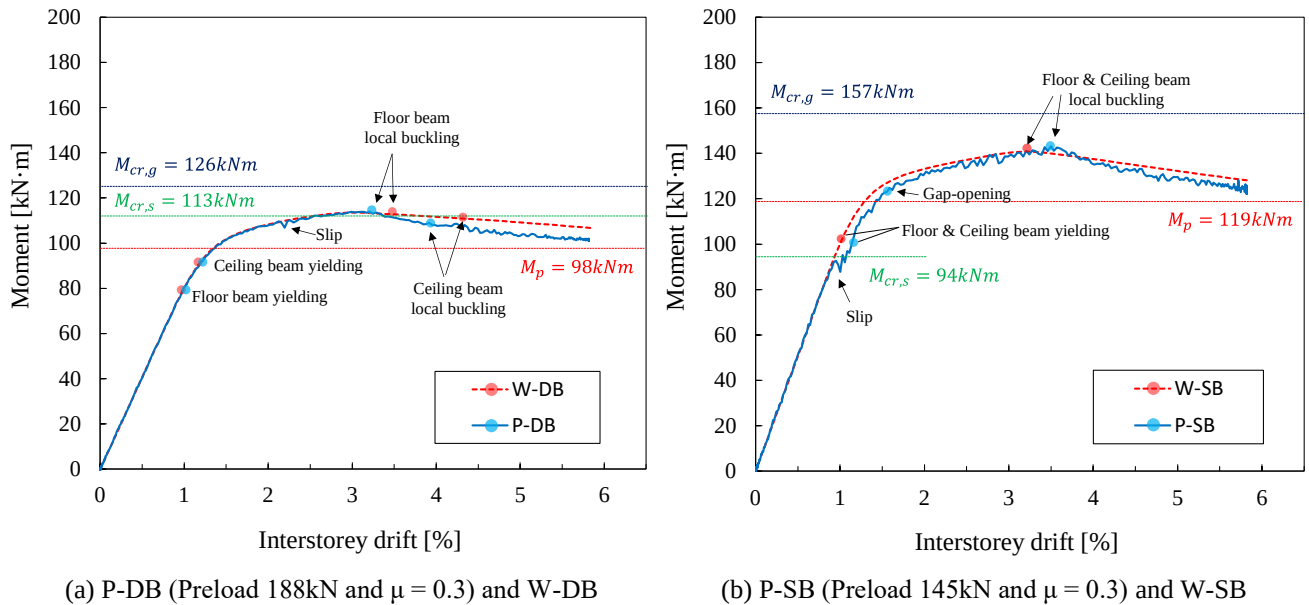
Figure 5.23 shows the moment versus interstorey drift curves for all joints. The AJ joints (P-DB and P-SB) had overall similar performance to welded joints (W-DB and W-SB), except for the slip in the AJ connection. All joints showed ductile behaviour by plastic hinges forming in both floor and ceiling

beams near the reinforcing plates. The non-dissipative components (columns, couplers and SRBs) remained in the elastic range until the beams failed. Figure 5.24 (a) and Figure 5.25 (a) show the failure shapes and stress distributions of P-DB and P-SB, respectively.

Both P-DB and P-SB experienced slip near the expected strength ($M_{cr,s}$), as shown in Figure 4.52. In the case of P-DB, the gap-opening was prevented completely by the demand preload (P_g in Table 4.41) prior to the floor and ceiling beams failing (see Figure 5.24 (b)). P-SB had very minor exterior gap-opening (see Figure 5.25 (b)), even though a 10% higher preload than the demand preload was applied. However, it had no noticeable effect on the overall joint behaviour. It was revealed by later analysis that the unexpected earlier gap-opening in P-SB was related to slip occurring prior to the gap-opening, which caused a complex bearing between the coupler and the plates at the AJ connection and affected the applied preload (refer to Section 5.5.1.2 (3)).

Figure 5.24 (c) and Figure 5.25 (c) presents the stress variations in the SRBs as functions of the interstorey drift for P-DB and P-SB. The stresses in the SRBs in P-DB and P-SB remained below the yield stress ($f_{y,SRB}$) prior to the floor and ceiling beams failing.

The FE numerical simulation results showed that AJ joints (P-DB and P-SB) developed the intended beam failure modes with ductile behaviour, as required for a strong column-weak beam seismic moment-resisting frame.

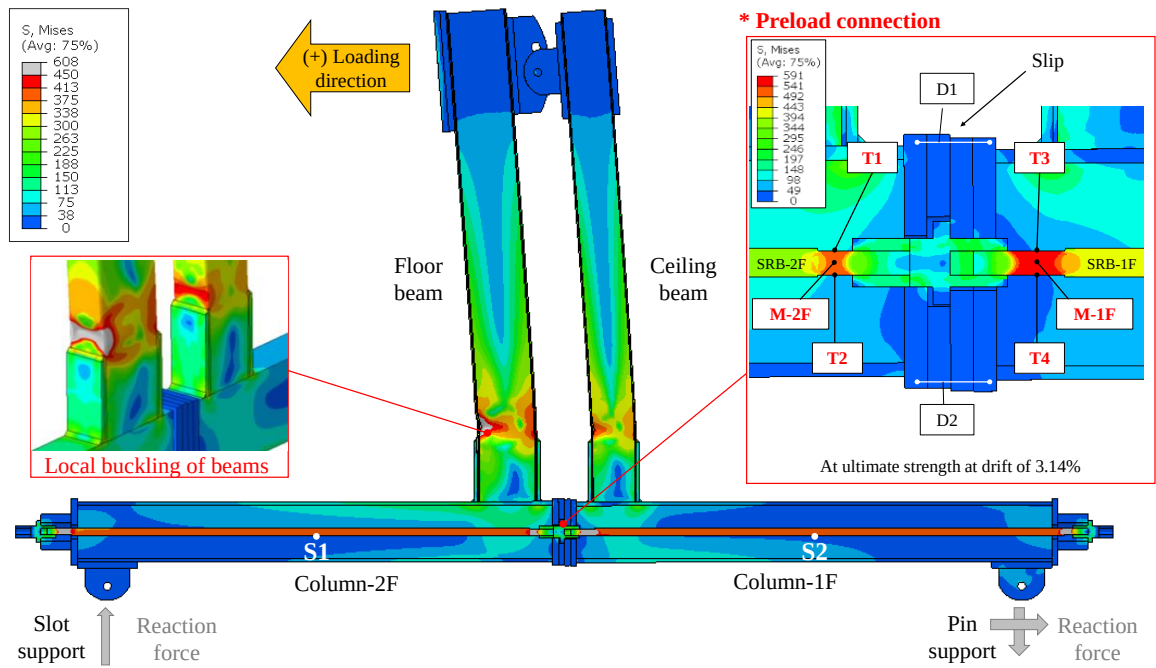


$M_{p,n}$: Nominal plastic moment strength of the beam-to-column joint by (Eq. 4.5).

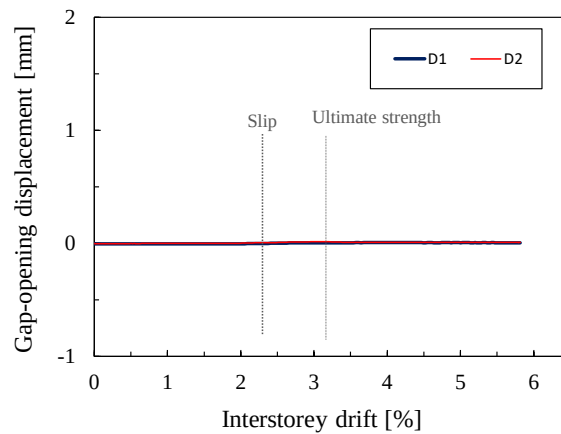
$M_{cr,g}$: Critical moment strength to prevent the gap-opening by the applied preload at the connection by Table 3.3 and (Eq. 3.69).

$M_{cr,s}$: Critical moment strength to prevent the slip by the applied preload at the connection by (Eq. 3.81) and (Eq. 3.82).

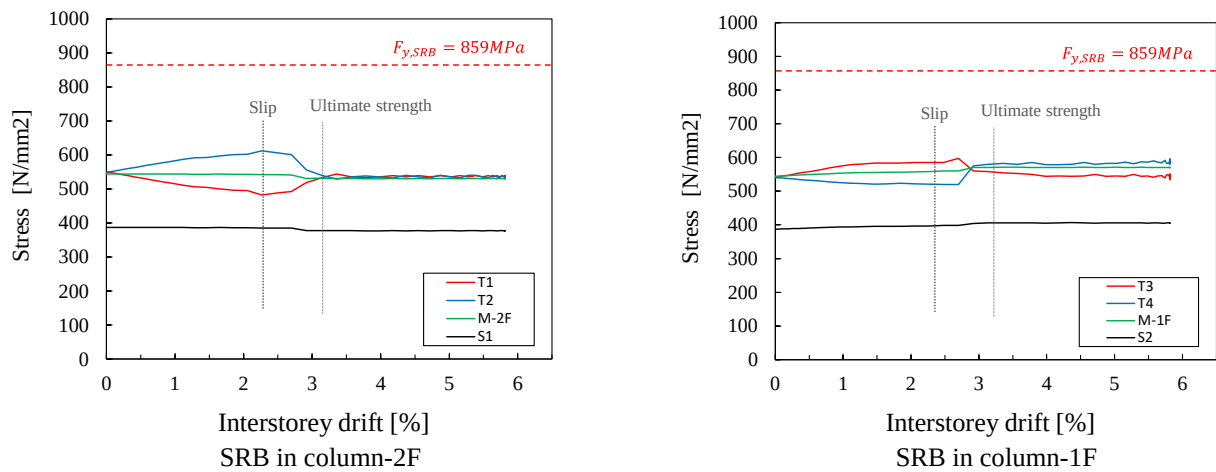
Figure 5.23: Moment-interstorey drift curves for AJ joints and welded joints under monotonic loading



(a) Failure mode and stress distribution at a drift of 6% (von Mises stress at the centre lines of the specimen)

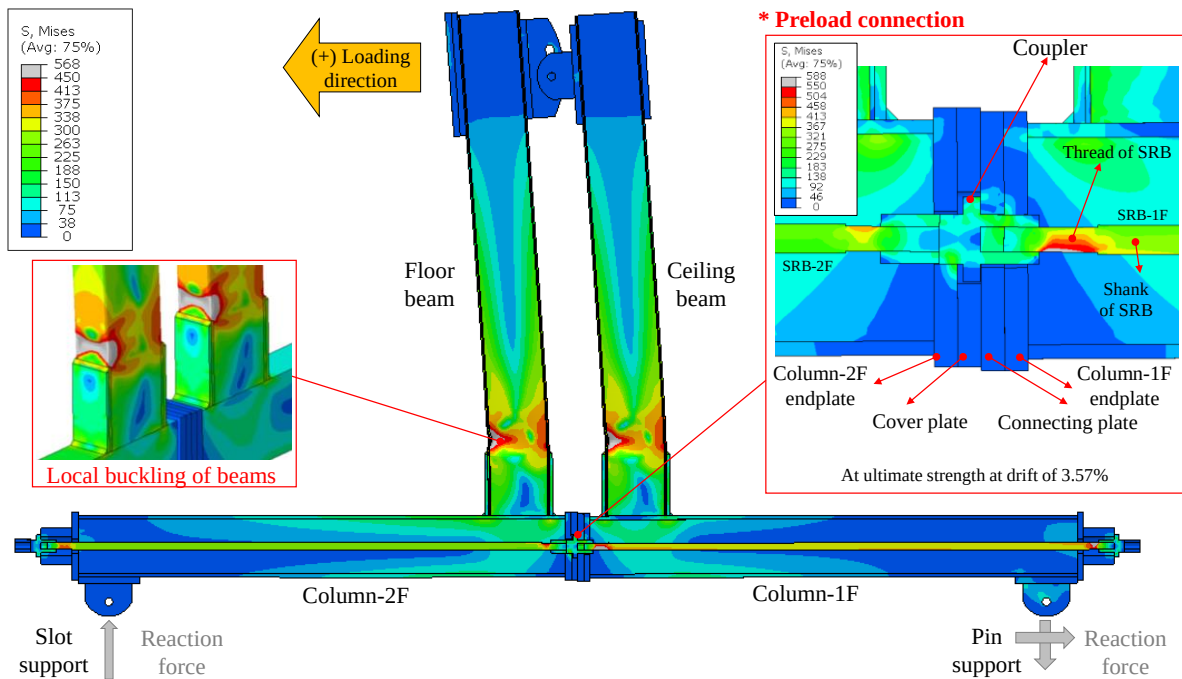


(b) Displacement for gap-opening at AJ connection

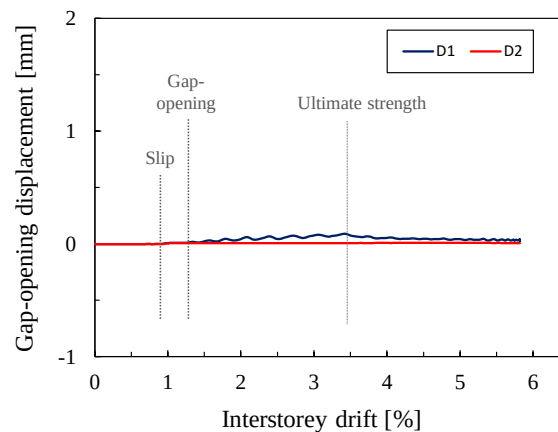


(c) Stress variation of SRBs

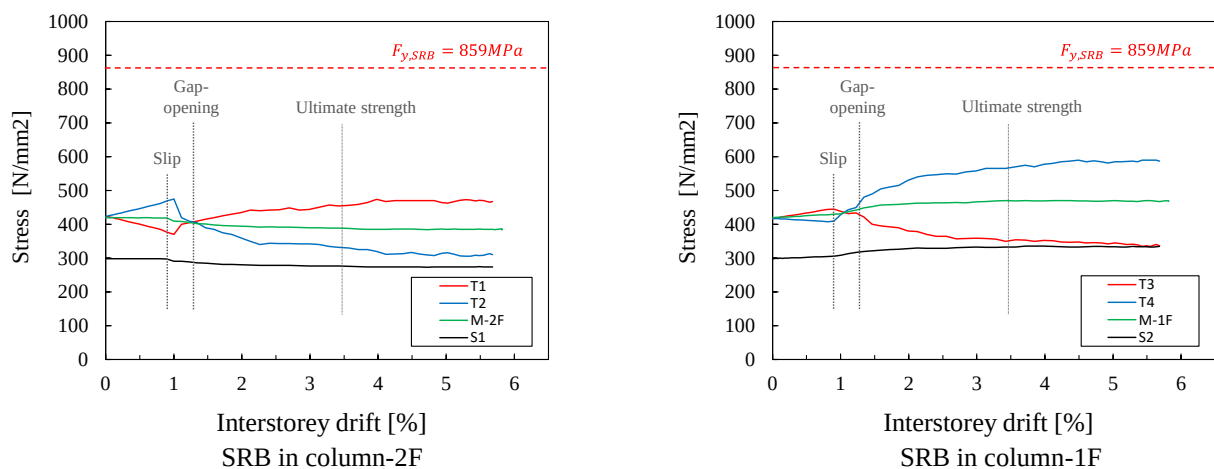
Figure 5.24: FE simulation results of P-DB (Preload 188kN) under monotonic loading



(a) Failure mode and stress distribution at a drift of 6% (von Mises stress at the centre lines of the specimen)



(b) Displacement for gap-opening at AJ connection



(c) Stress variation of SRBs

Figure 5.25: FE simulation results of P-SB (Preload 145kN) under monotonic loading

5.4.1.2. Joint Classification

EN 1993-1-8 [85] provides a joint classification based on the rotational stiffness and strength of the connection to determine the type of joint modelling for global frame analysis. In terms of stiffness, the joint is classified as rigid, semi-rigid, or pinned by comparing its initial rotational stiffness ($S_{j,ini}$) with the stiffness boundaries as follows:

$$\text{Rigid joint:} \quad S_{j,ini} \geq \frac{k_b EI_b}{L_b} \quad (\text{Eq. 5.3})$$

$$\text{Semi-rigid joint:} \quad \frac{0.5EI_b}{L_b} \leq S_{j,ini} \leq \frac{25EI_b}{L_b} \quad (\text{Eq. 5.4})$$

$$\text{Pinned joint:} \quad S_{j,ini} \leq \frac{0.5EI_b}{L_b} \quad (\text{Eq. 5.5})$$

where :

- k_b $k_b=8$ for frames where the bracing system reduces the horizontal displacement by at least 80% and $k_b=25$ for other frames, provided that in every storey $K_b/K_c \geq 0.1$;
- K_b The mean value of I_b/L_b for all the beams at the top of that storey;
- K_c The mean value of I_c/L_c for all the columns in that storey;
- I_b The second moment of area of a beam;
- I_c The second moment of area of a column;
- L_b The span of the beam (centre-to-centre of columns);
- L_c The storey height of a column.

The joint can also be classified in terms of strength as either full strength, partial-strength or nominally pinned. The design resistance of a full-strength joint should be not less than that of the connected members. Namely, in the case the joint strength ($M_{j,Rd}$) is higher than the design plastic moment resistance of a beam ($M_{b,pl,Rd}$), it may be classified as a full strength joint. If the joint strength is lower than 25% of $M_{b,pl,Rd}$, it becomes a nominally pinned joint. If the joint has a higher strength than the pinned joint but lower strength than the full-strength resistance, it is a partial-strength joint. For the modular building structure, $M_{b,pl,Rd}$ becomes the sum of the design plastic moment resistances of the floor and ceiling beams.

Figure 5.26 and Figure 5.27 show the joint classification for the AJ joints (P-SB and P-DB) and the welded joints (W-SB and W-DB). The AJ joints, which have similar stiffness and strength to the welded joints, are classified as semi-rigid and full-strength joints.

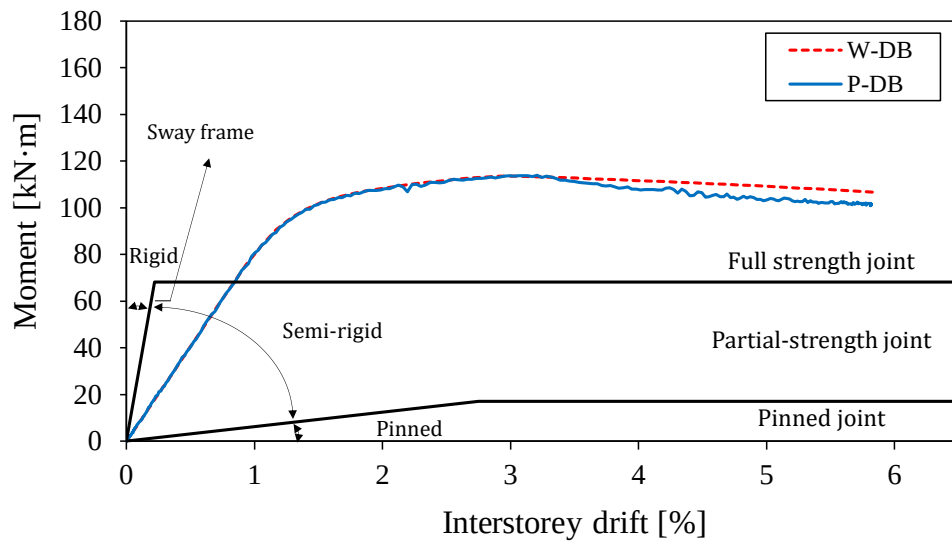


Figure 5.26: Joint classification of P-DB and W-DB

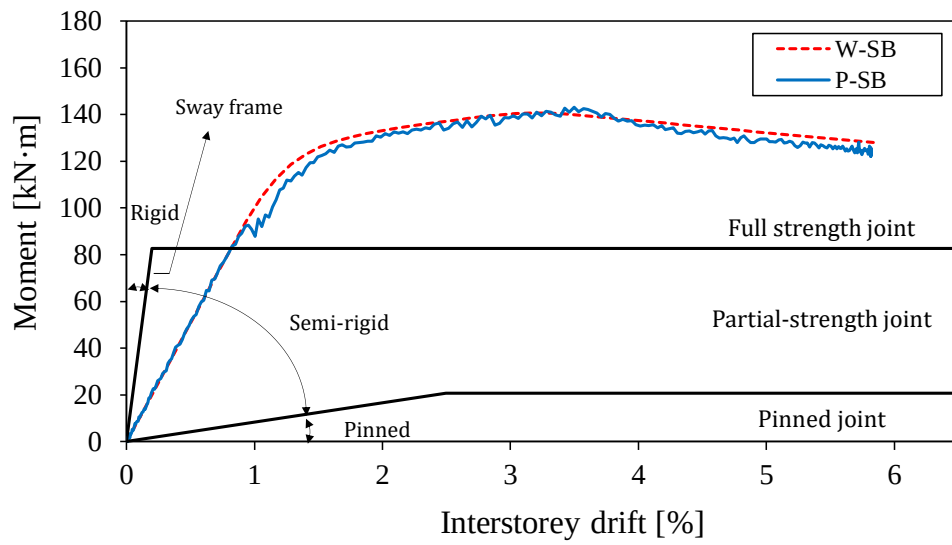


Figure 5.27: Joint classification of P-SB and W-SB

5.4.2. Simulation of Cyclic Loading

5.4.2.1. Failure Mode

Figure 5.28 - Figure 5.33 show the failure modes of each specimen under the quasi-static cyclic loading. All joint specimens (P-DB-Q, P-SB-Q, W-DB-Q, and W-SB-Q) failed in the intended ductile beam failure modes, forming plastic hinges on both floor and ceiling beams. The non-dissipative components (columns, panel zones, coupler and SRB) remained in the elastic range until the beams failed, as shown in Figure 5.28 (c) - Figure 5.33 (c). The failure mechanism of the joints was similar to that observed under monotonic loading. In the case of P-DB-Q and W-DB-Q, the floor beam yielded first, followed shortly after by the ceiling beam. The floor and ceiling beams developed local buckling at the almost same time, but the buckling deformations increased at a faster rate in the floor beam. For P-SB-Q and W-SB-Q, the floor and ceiling beams yielded, reached their plastic moment capacities, and failed by local buckling simultaneously. Figure 5.29 (a) and Figure 5.32 (a) show the measured gap-opening at the AJ connection of P-DB-Q and P-SB-Q, respectively. The gap-opening was completely prevented by the induced preload in P-DB-Q, while in P-SB-Q the gap-opening occurred at a moment 30% lower strength than the expected strength $M_{cr,g}$ (see Figure 5.31). The amount of gap-opening was 0.7mm. In terms of slip, P-DB-Q and P-SB-Q exhibited slip at 7% and 5% lower slip strength than the expected strength $M_{cr,s}$. For the stress variation in the threads of the SRBs, both P-DB-Q and P-SB-Q showed stable behaviour under the yield strength, while P-SB-Q developed relatively larger bending stress after the occurrence of slip (see Figure 5.29 (b) and Figure 5.32 (b)).

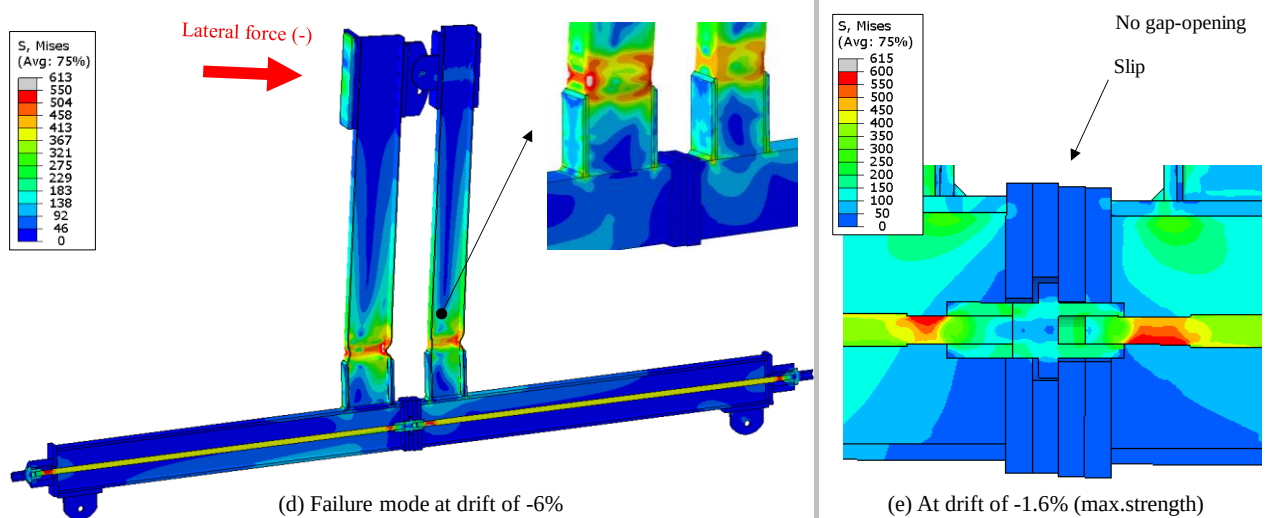
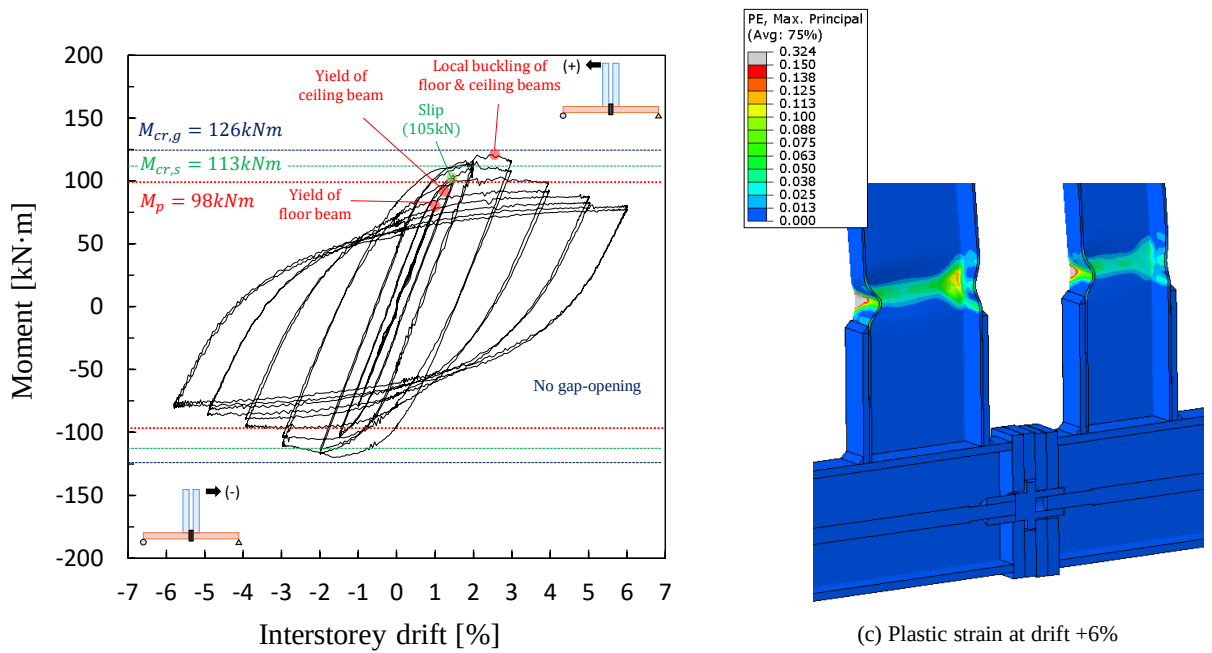
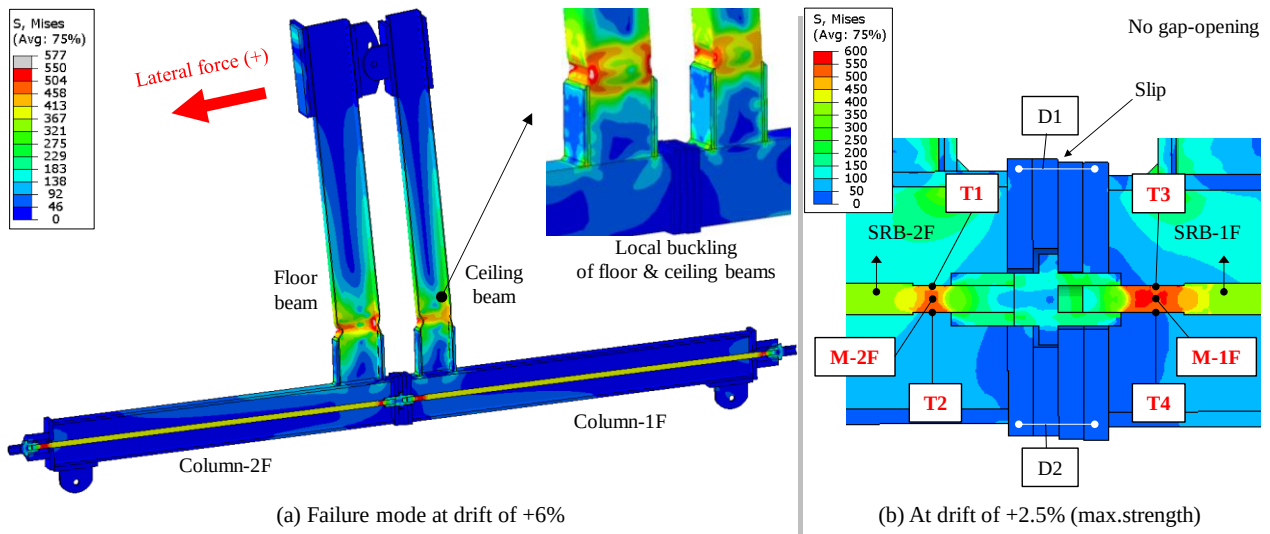
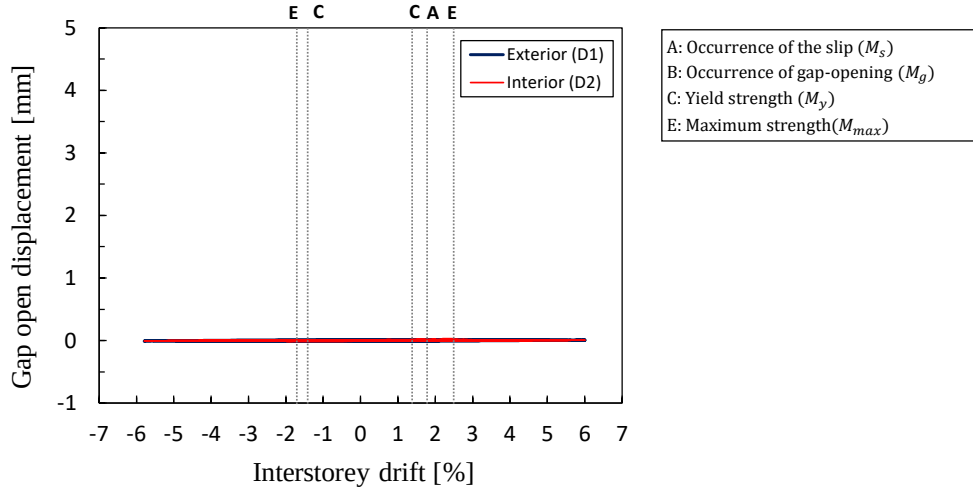
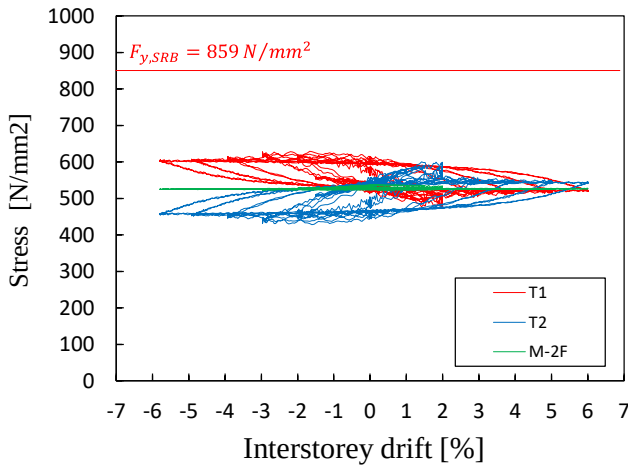


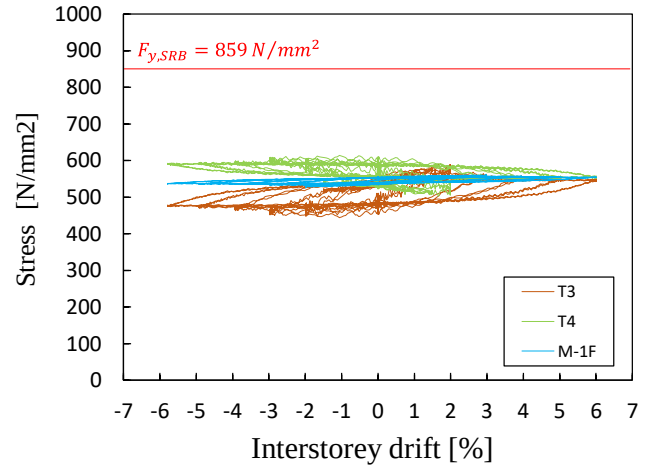
Figure 5.28: Hysteretic curve and failure mode of P-DB-Q (Preload 188kN) - (Mises stress distribution in cross-sectional view)



(a) Displacement for the gap-opening at the AJ connection

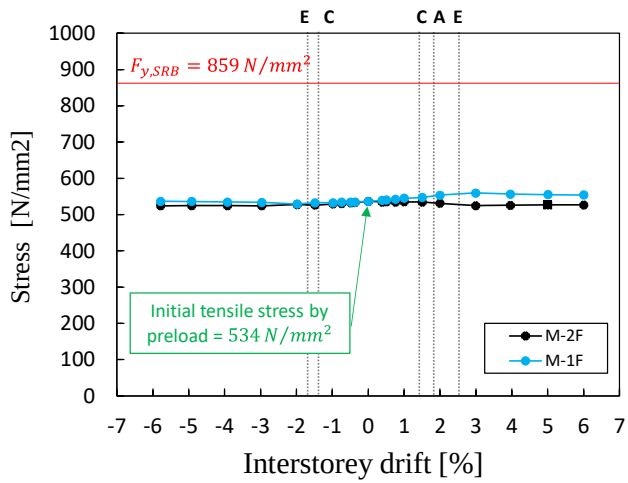


SRB in column-2F



SRB in column-1F

(b) Stress variation in the SRB (bolt thread near the connection)



(c) Stress variation in the SRB corresponding to the peak drift in each set of cycles

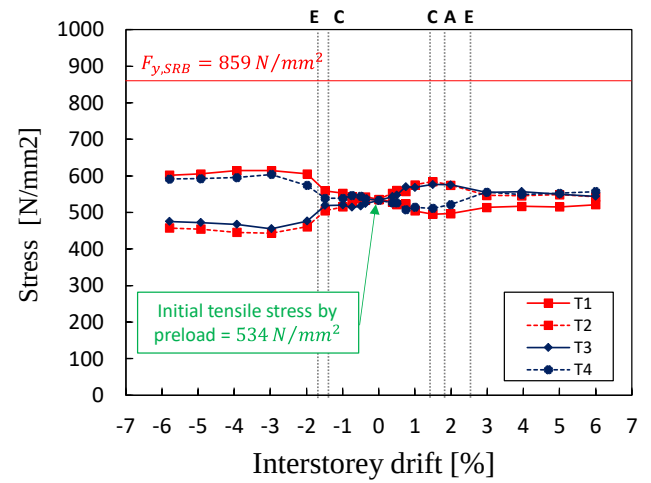


Figure 5.29: Measured gap-opening and stress variations in the SRBs in P-DB-Q (Preload 188kN) (the measured locations are shown in Figure 5.28 (b))

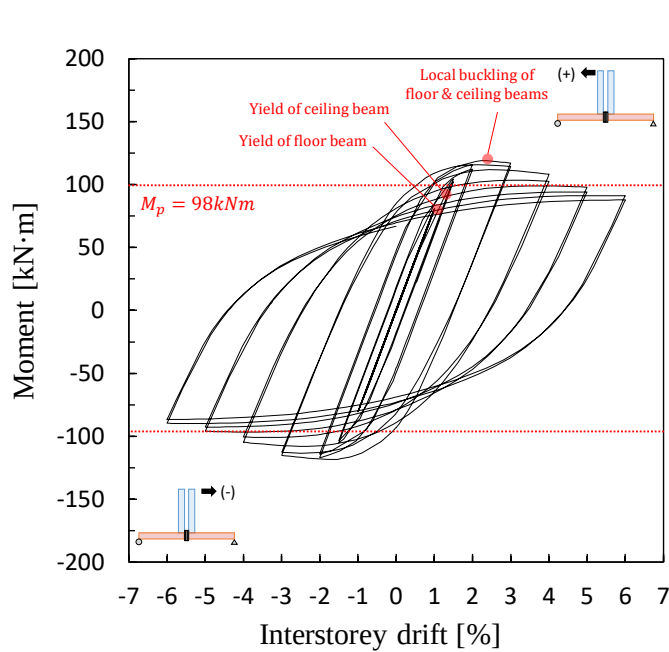
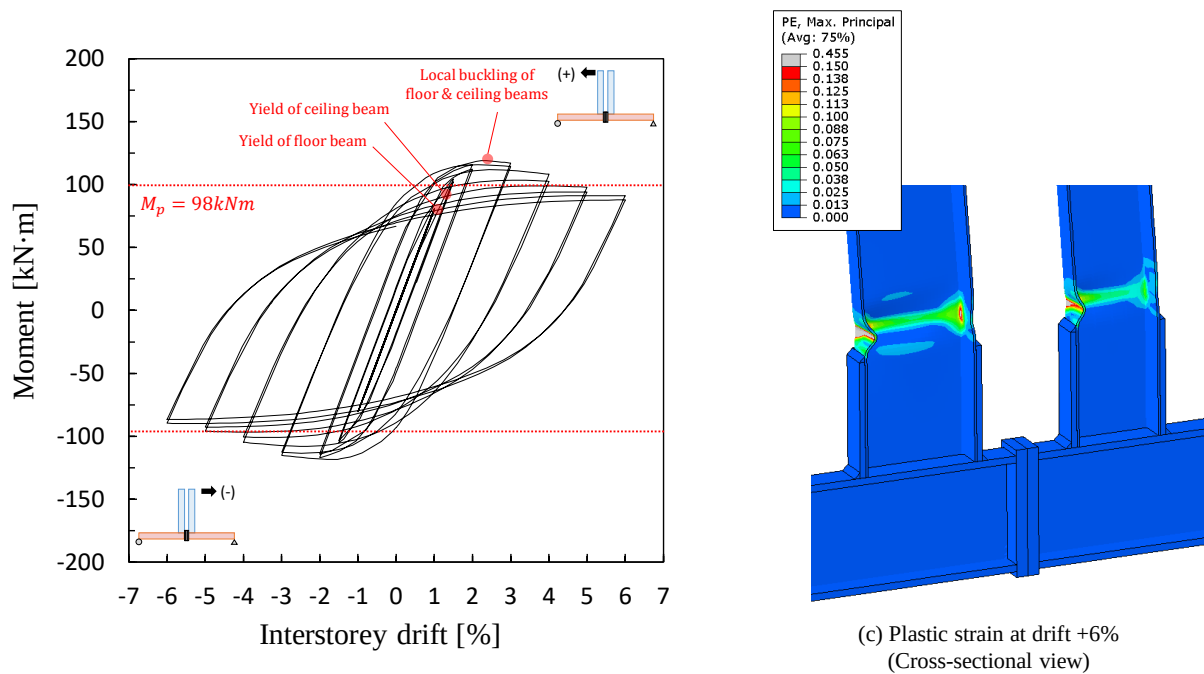
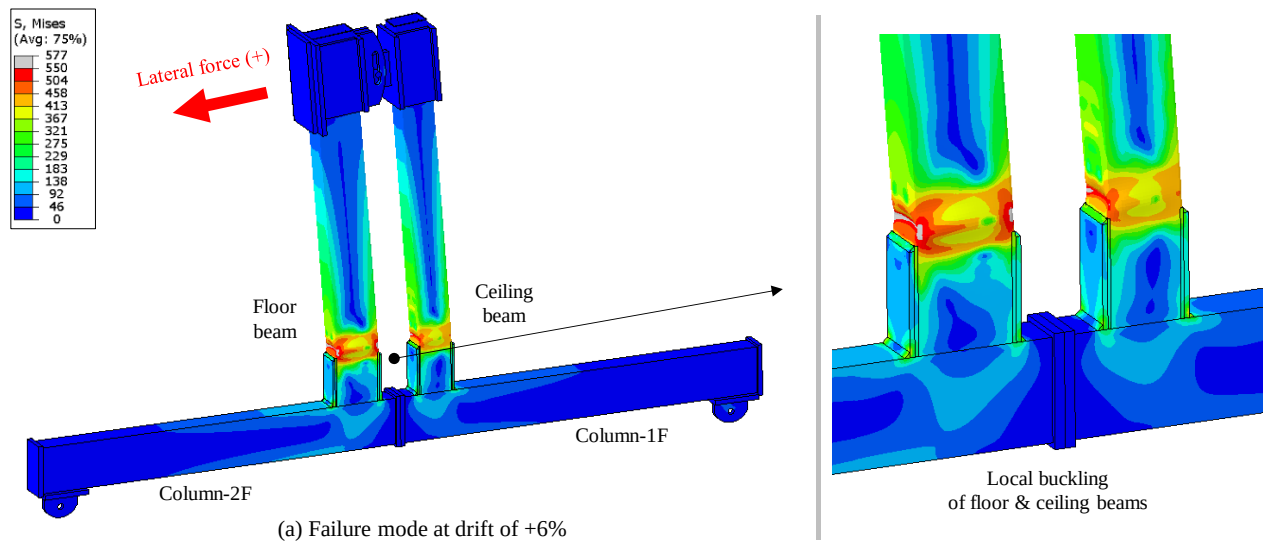


Figure 5.30: Hysteretic curve and failure mode of W-DB-Q (Mises stress distribution)

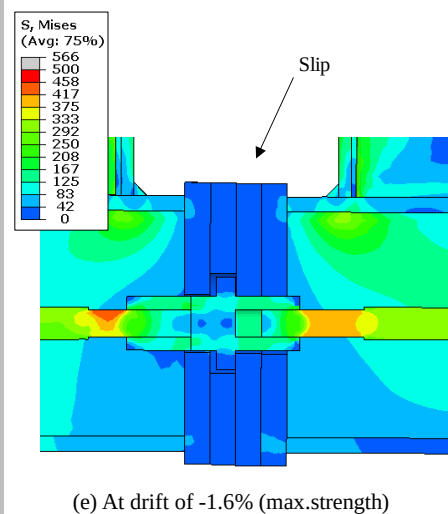
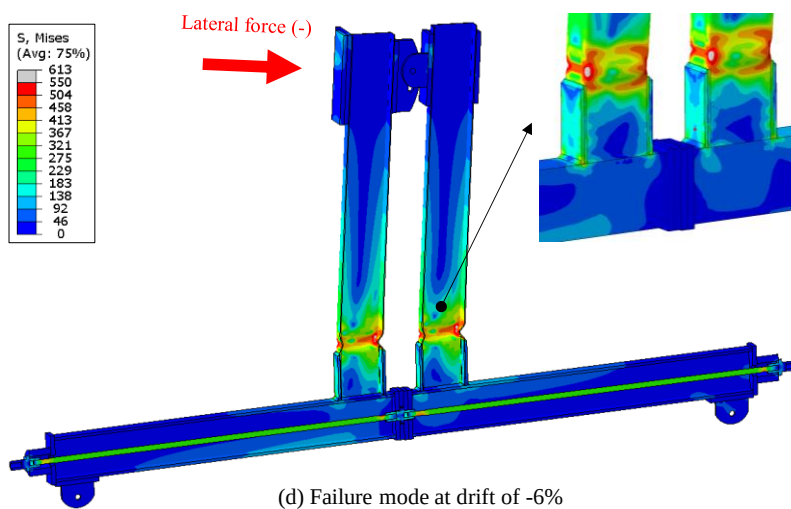
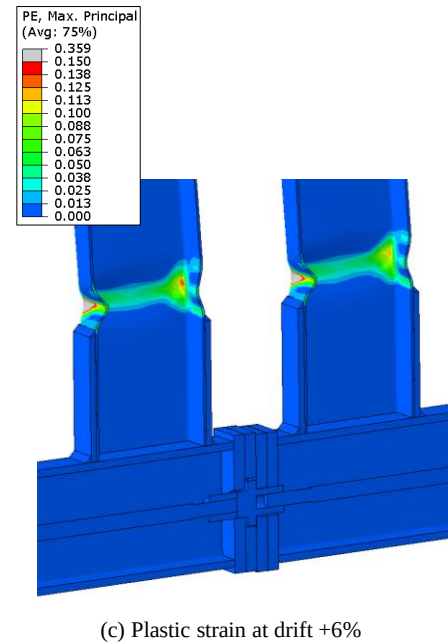
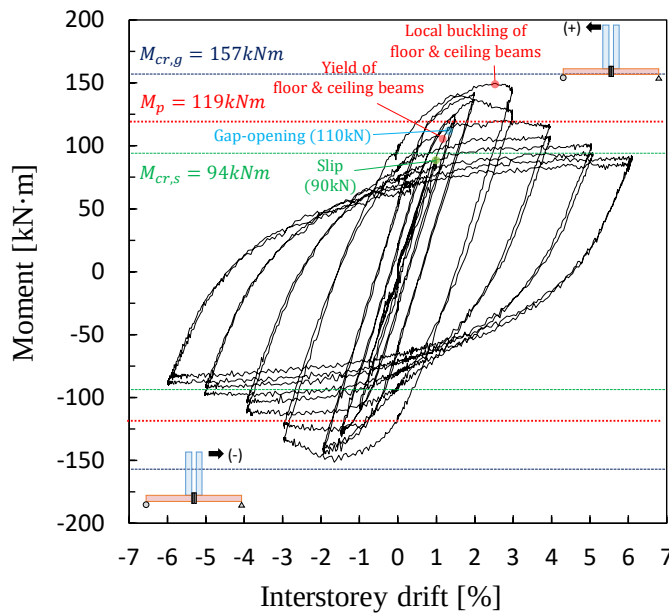
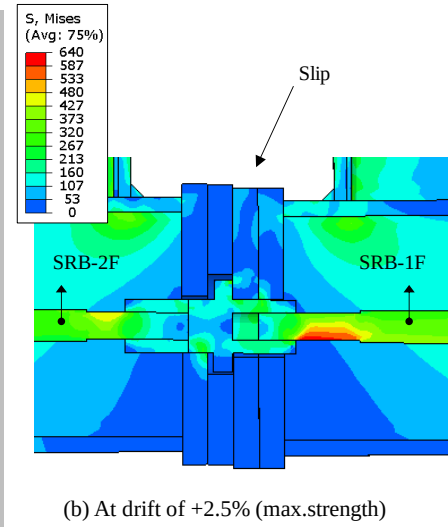
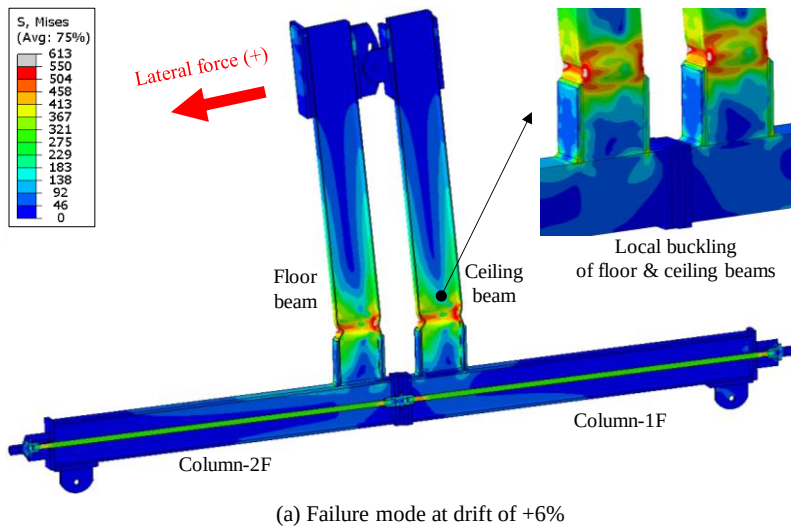
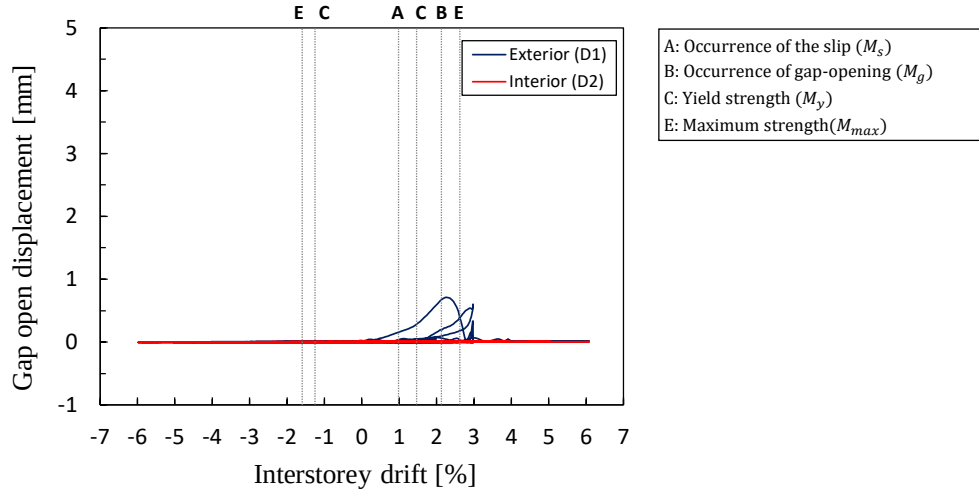
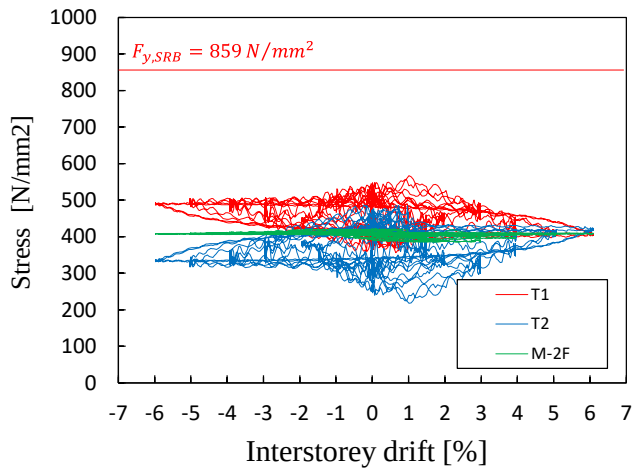


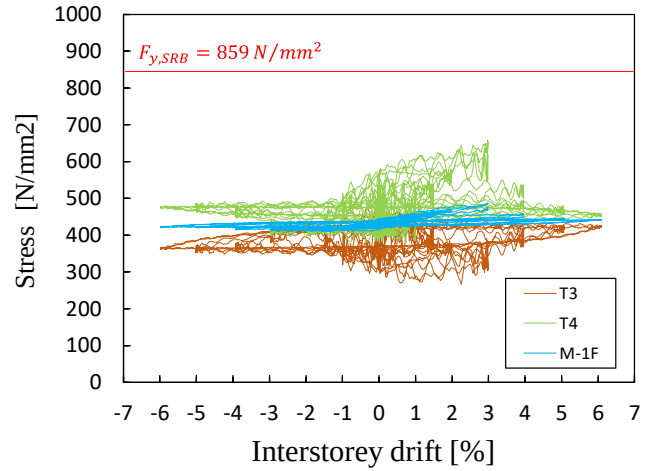
Figure 5.31: Hysteretic curve and failure mode of P-SB-Q (Preload 145kN) - (Mises stress distribution in cross-sectional view)



(a) Displacement for the gap-opening at the AJ connection

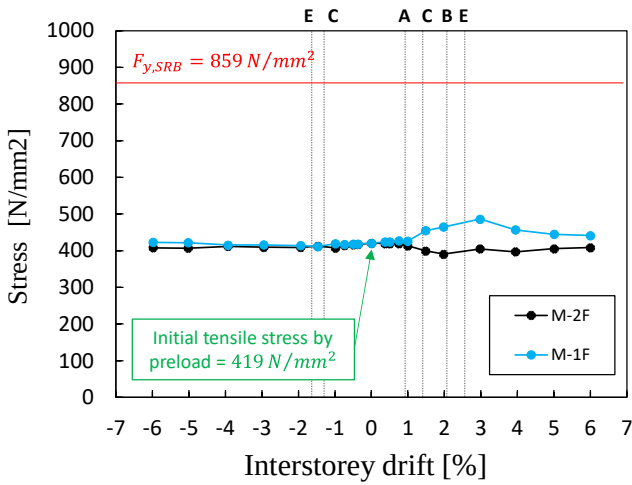


SRB in column-2F



SRB in column-1F

(b) Stress variation in the SRB (bolt thread near the connection)



(c) Stress variation in the SRB corresponding to the peak drift in each set of cycles

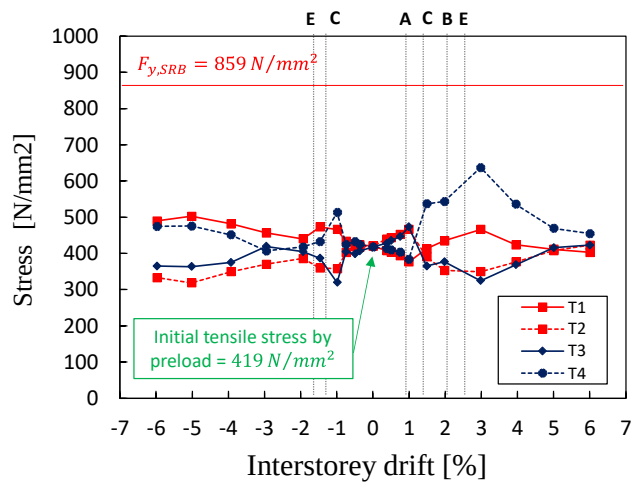


Figure 5.32: Measured gap-opening and stress variations in the SRBs in P-SB-Q (Preload 145kN)
(the measured locations are shown in Figure 5.28 (b))

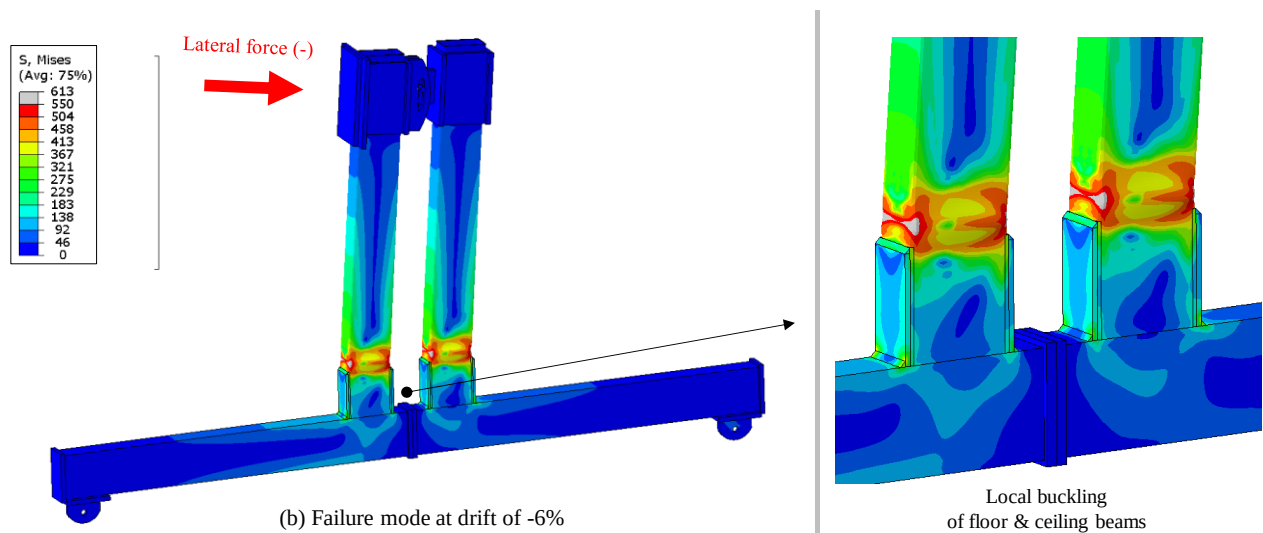
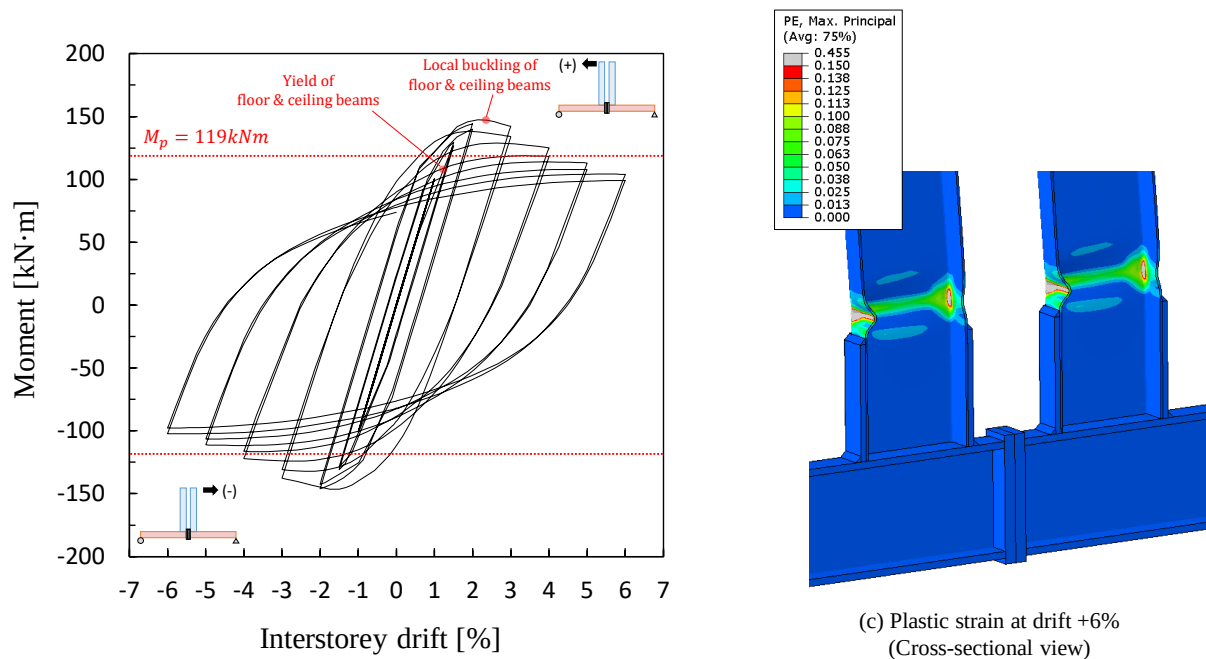
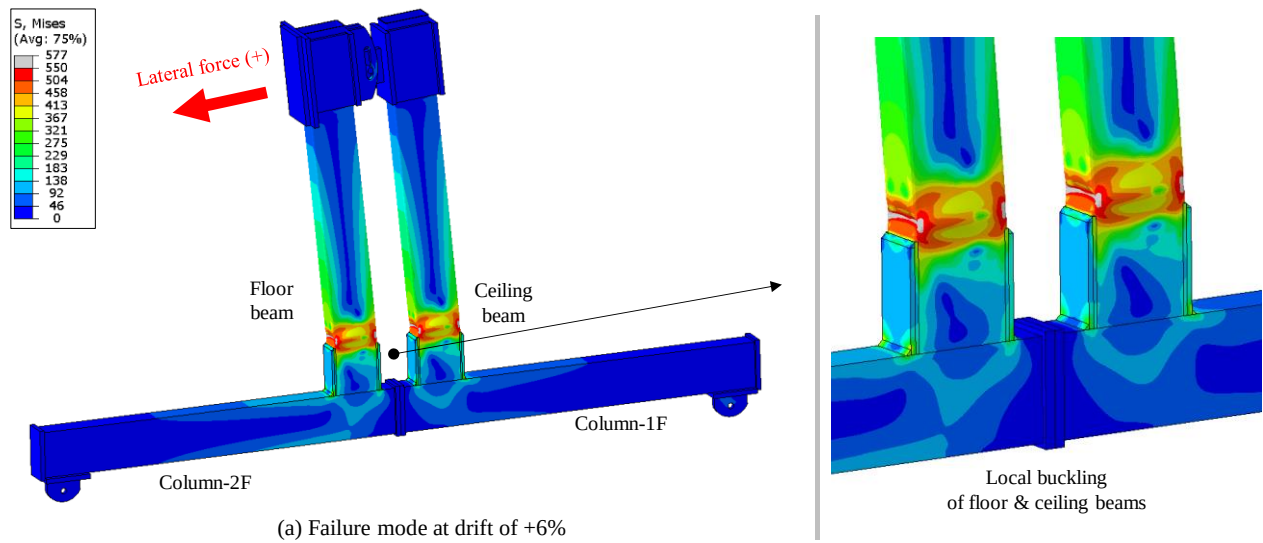
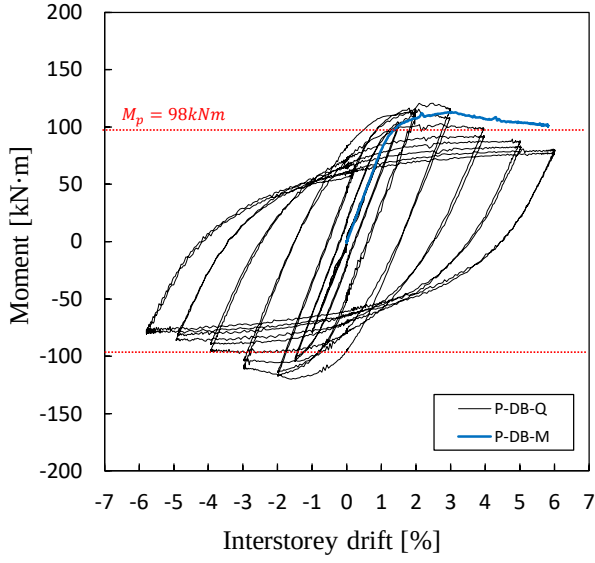


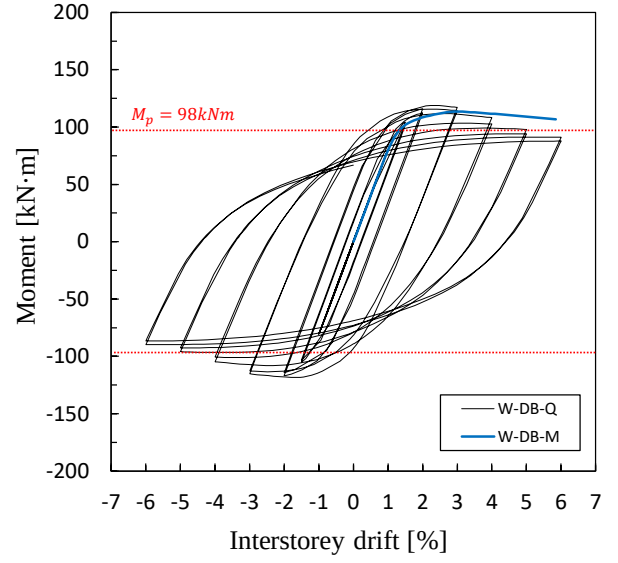
Figure 5.33: Hysteretic curve and failure mode of W-SB-Q (Mises stress distribution)

5.4.2.2. Hysteretic Performance and Skeleton Curves

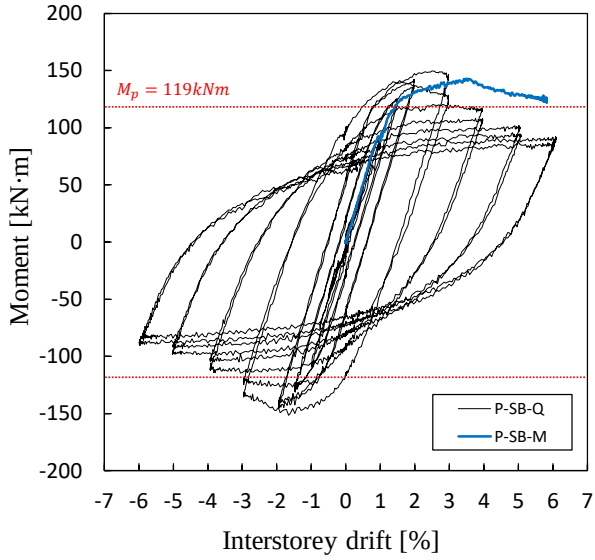
Figure 5.34 presents hysteretic curves and monotonic curves of the moment versus interstorey drift ratio (computed at the column centreline) for AJ and welded joints. Table 5.2 presents a summary of the simulation results. The hysteretic loops of all specimens indicated good energy dissipation capacity. Although the AJ joints (P-DB-Q and P-SB-Q) developed slip at the column-to-column connection, they showed a similar shape of the hysteretic loops to the welded joints (W-DB-Q and W-SB-Q) because the induced preload prevented gap-opening from developing prior to the failure of the floor and ceiling beams. All specimens developed the full plastic moment capacity of the floor and ceiling beams (M_p). Figure 5.35 (a) shows the skeleton curves for all specimens. The skeleton curves are the backbone of the hysteretic loops, and were drawn using the peak moments of each set of cycles with the corresponding interstorey drifts. The curves reflect the hardening and strength degradation of the specimens. The AJ joints had almost the same initial rotational stiffness and maximum moment strength as shown in Figure 5.35 (b) and (c). (As mentioned previously, after reaching the ultimate strength, the AJ joints degraded at a relatively faster rate. It appears that this slight difference in response after reaching the maximum strength might be caused by using different analysis solvers (i.e. Static vs Dynamic explicit)). P-SB-Q and W-SB-Q, which have the same size floor and ceiling beams, had higher initial stiffness (K_i) and maximum moment resistance capacity (M_{max}) than P-DB-Q and W-DB-Q because of the larger cross-sectional size of the ceiling beam. On the other hand, in terms of ductile behaviour, P-DB-Q and W-DB-Q showed better performance as shown in Figure 5.35 (d) and (e). P-SB-Q and W-SB-Q featured larger strength reduction after reaching the maximum moment because the floor and ceiling beams failed simultaneously by local buckling. In the case of P-DB-Q and W-DB-Q which had different size floor and ceiling beams, the floor beam buckled at a relatively faster rate than the ceiling beam, although the moments initiating local buckling of the two beams were similar.



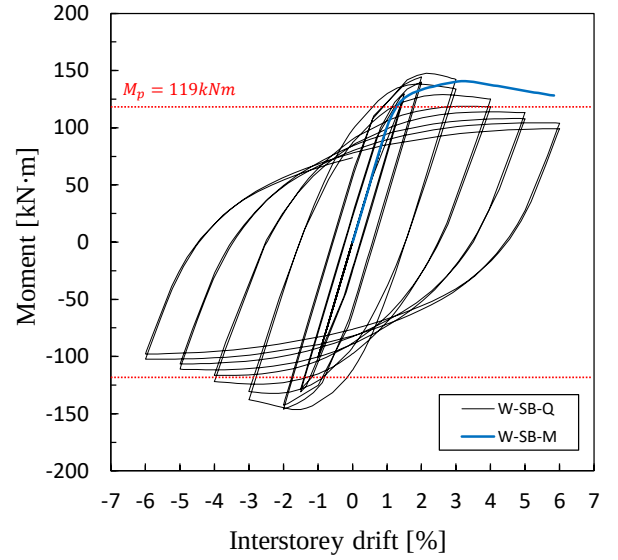
(a) P-DB-Q



(b) W-DB-Q



(c) P-SB-Q



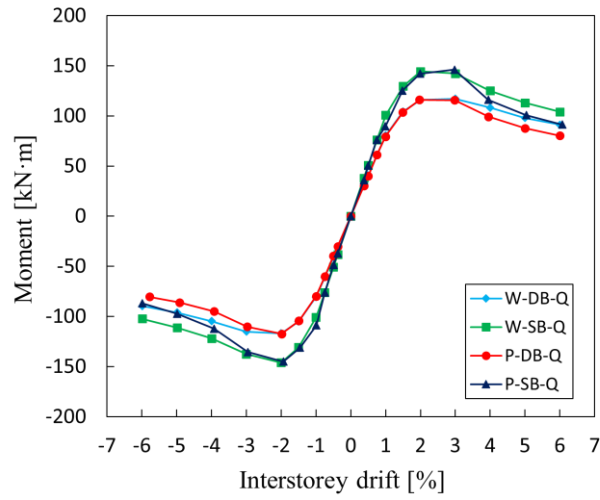
(d) W-SB-Q

Figure 5.34: Hysteretic curves of FE beam-to-column joint models under quasi-static cyclic loading

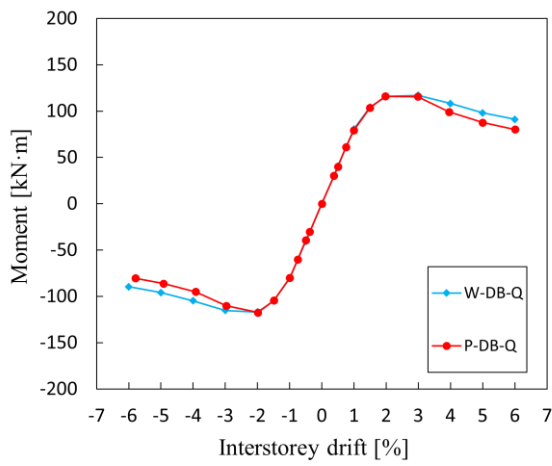
Table 5.2: Summary of the revised cyclic test results through the FE numerical simulations

Specimen	Preload [kN]	Loading direction	$K_i \times 10^3$ [kNm/rad]	M_y [kNm]	θ_y [%]	M_{max} [kNm]	θ_{max} [%]	$0.8M_{max}$ [kNm]	θ_u [%]	μ	γ ($= M_{max}/M_y$)
P-DB-Q	188	(+)	8171	98	1.42	120	2.50	96	3.98	2.80	1.22
		(-)	8067	101	1.46	119	1.61	95	3.97	2.72	1.18
W-DB-Q	N/A	(+)	8124	101	1.44	119	2.36	95	5.00	3.47	1.18
		(-)	8120	103	1.47	118	1.72	95	5.00	3.40	1.15
P-SB-Q	145	(+)	10199	114	1.34	150	2.50	120	3.00	2.24	1.31
		(-)	10221	123	1.35	150	1.60	120	3.00	2.22	1.22
W-SB-Q	N/A	(+)	10151	117	1.36	148	2.38	118	4.00	2.94	1.26
		(-)	10170	127	1.45	145	1.70	116	4.00	2.76	1.14

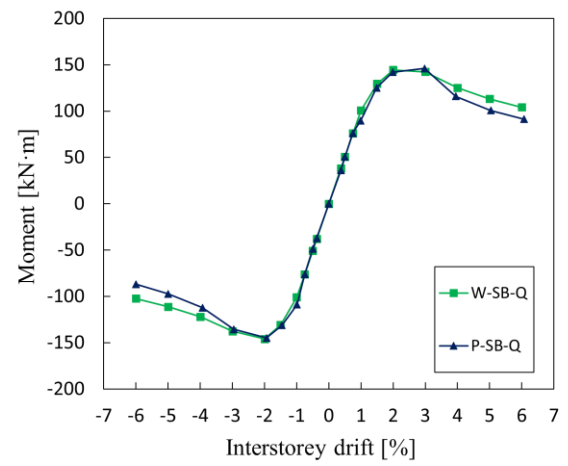
Note: K_i : the initial rotational stiffness; M_y : the yield moment strength; M_{max} : the maximum moment strength; θ_y : the yield interstorey drift; θ_{max} : the interstorey drift at the maximum moment strength; θ_u : the ultimate interstorey drift; μ : the ductility factor; M_p : the actual plastic moment strength of the joint by (Eq. 4.5); γ : the strain hardening coefficient.



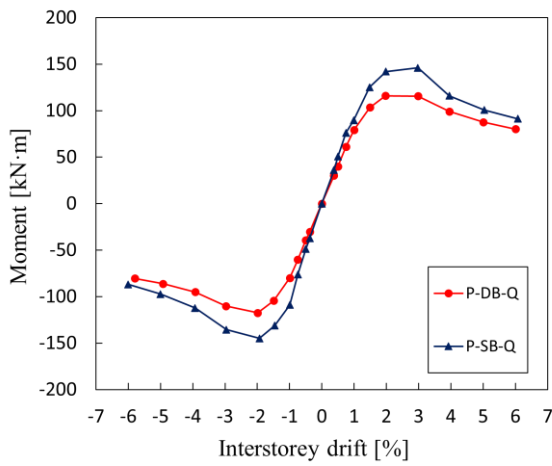
(a)



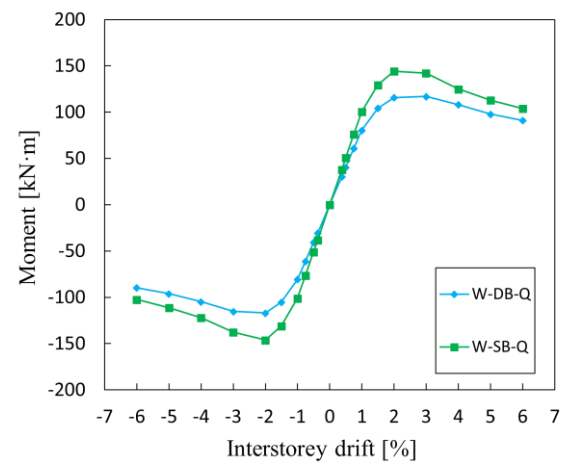
(b)



(c)



(d)



(e)

Figure 5.35: Skeleton curves for AJ joints and welded joints

5.4.2.3. Deformation Capacity and Ductility

EC8-1 [85] and AISC 341-16 [110] were adopted to evaluate the deformation capacity and ductility of the joints to verify their suitability for seismic applications. The ductility is the inelastic deformation capacity available without incurring a significant reduction in load bearing capacity, which can be obtained from hysteretic loops. The ductility coefficient (μ) is regarded as an important index for evaluating the seismic performance of a joint, and is defined as the ratio of the ultimate interstorey drift to the yield interstorey drift, as follows:

$$\mu = \frac{\theta_u}{\theta_y} \quad (\text{Eq. 5.6})$$

For the ultimate interstorey drift θ_u , EC8-1 states that the rotation capacity of the plastic hinge region should be ensured under cyclic loading without degradation of strength and stiffness greater than 20% for beam-to-column joints in moment-resisting frames. Based on this requirement, the ultimate interstorey drift θ_u is determined as the drift corresponding to a strength degradation of 20% from the maximum strength M_{max} . The yield interstorey drift θ_y is the drift corresponding to the yield moment strength (M_y) defined by the 0.2% offset method [132]. Figure 5.36 presents envelope curves for all joints and the determination of θ_u in the positive (+) and negative (-) loading directions. Table 5.2 shows these interstorey drift values and ductility factors for all specimens.

In the case of the AJ joints, the ductility factors were in the range of 2.22 - 2.90 with a mean value of 2.50, while the ductility factors of the welded joints were in the range of 2.76 - 3.47 with a mean value of 3.14. P-DB-Q and W-DB-Q, which had different floor and ceiling beam sizes, showed a larger ductility compared to P-SB-Q and W-SB-Q.

The ductility values were compared with experimental results from the literature obtained under similar conditions to this study (RHS beam-to-RHS column exterior joints of steel-framed modular buildings). The experimental investigations by Chen et al. [60] indicated that the ductility factor ranged from 2.09 to 2.79 with a mean value of 2.42. A similar experimental study by Deng et al. [58] showed the ductility factor ranged from 2.66 to 4.70 with a mean value of 3.24. In actuality, the ductility factor can be quite different depending on the determination of the yield interstorey drift (θ_y). Different definitions and methods are used in the literature, including the 0.2% offset method, the intersection between the tangent of initial stiffness S_j and the tangent that has a slope of $S_j/10$ from the maximum moment [133], the energy equivalent area method [60], and the two thirds of the maximum moment method [58, 134]). If the yield interstorey drift θ_y is defined by the two thirds of the maximum moment method, the ductility factors for the AJ joints are in the range of 2.73 - 4.0. Consequently, it is demonstrated that the proposed joints in this study have good ductility compared to other reported inter-module connections for steel-framed modular buildings.

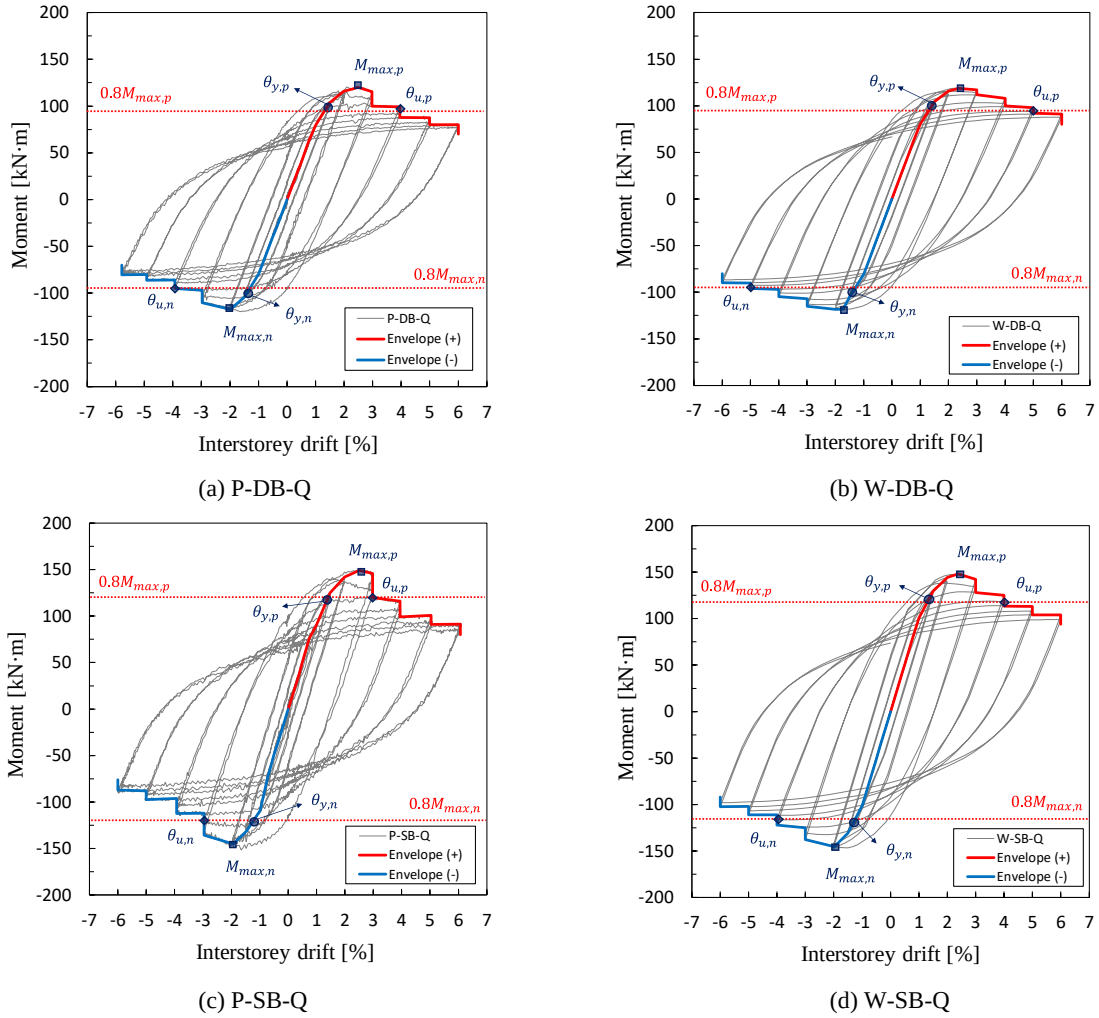


Figure 5.36: Determination of ultimate interstorey drift θ_u according to EC8-1

According to EN 1998-1, the required rotation capacity of the plastic hinge region should be over 3.5% drift and 2.5% drift for structures of ductility classes DCH and DCM, respectively, for beam-to-column joints in moment resisting frames. This criterion for assessing the rotation capacity of plastic hinges takes into account the total beam rotation (i.e. elastic and plastic rotation) and the deformation of the connection including the column web panel shear deformation [135]. The column elastic deformation should be excluded. In all the specimens, the column exhibited elastic behaviour under the cyclic loading (refer to Figure 5.42). Figure 5.37 shows the hysteretic curves drawn by excluding the contribution of the column elastic deformation to evaluate the rotation capacity of the plastic hinge region. The welded joints of W-SB-Q and W-DB-Q satisfied the requirement for DCH. In the case of the AJ joint, for both the (+) and (-) loading directions, P-SB-Q fulfilled the required limit of 2.5% for DCM, and P-DB-Q satisfied the requirement for DCH by maintaining a moment slightly higher than $0.8M_{max}$ at the required drift of 3.6% in both the positive and negative loading directions.

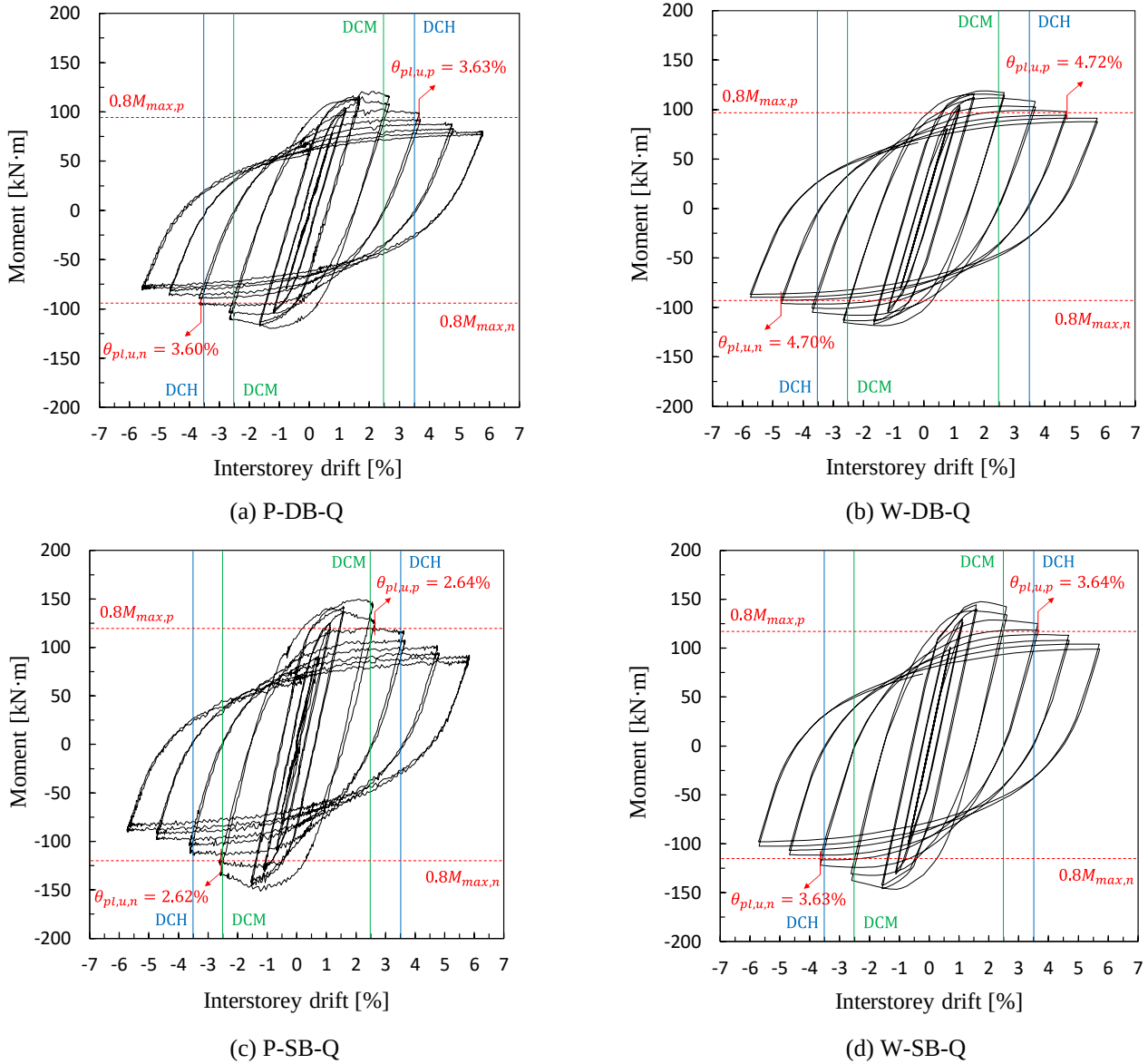


Figure 5.37: Determination of ductility class for joints according to EC8-1

In accordance with AISC 341-16 [136], the following two requirements should be satisfied for using the beam-to-column joint in a seismic force resisting system:

- (1) The required interstorey drift angle for special moment frames (SMF) and intermediate moment frames (IMF) are at least 4% and 2% radians, respectively.
- (2) The measured flexural resistance of the connection, determined at the column face, should sustain a moment over $0.8M_{p,n}$ at the aforementioned interstorey drift angles for SMF and IMF, where $M_{p,n}$ is the nominal plastic beam moment calculated using the specified minimum yield strength, F_y .

The required interstorey drift angle includes an elastic drift of 1%. In other words, SMF and IMF should accommodate at least approximately 3% and 1% inelastic rotation, respectively. Figure 5.37 shows the moments (calculated at the column face) and interstorey drift curves for all joints. All AJ joints and

welded joints satisfied the requirements of SMF provided by AISC 341-16 by maintaining more than $0.8M_{p,n}$ at the interstorey drift of 4%. However, FEMA 350 [137], which was the source of this requirement, also requires that no significant strength degradation should occur due to the failure of the connection up to the strength degradation qualifying drift angle (4% radian) for SMF. In light of this requirement, all joints had strength degradation due to the local buckling failure of the floor and ceiling beams at the drift of 2-3%, resulting in a 20% decrease in strength at the drift of 4%, compared to the maximum strength. This finding suggests that while all specimens satisfied the requirement for IMF by accommodating an interstorey drift ratio of 2% without strength degradation in both positive and negative loading directions, not all joints are suitable for SMF systems. In particular, the preloaded P-DB-Q and P-SB-Q joints can only be used for IMF systems.

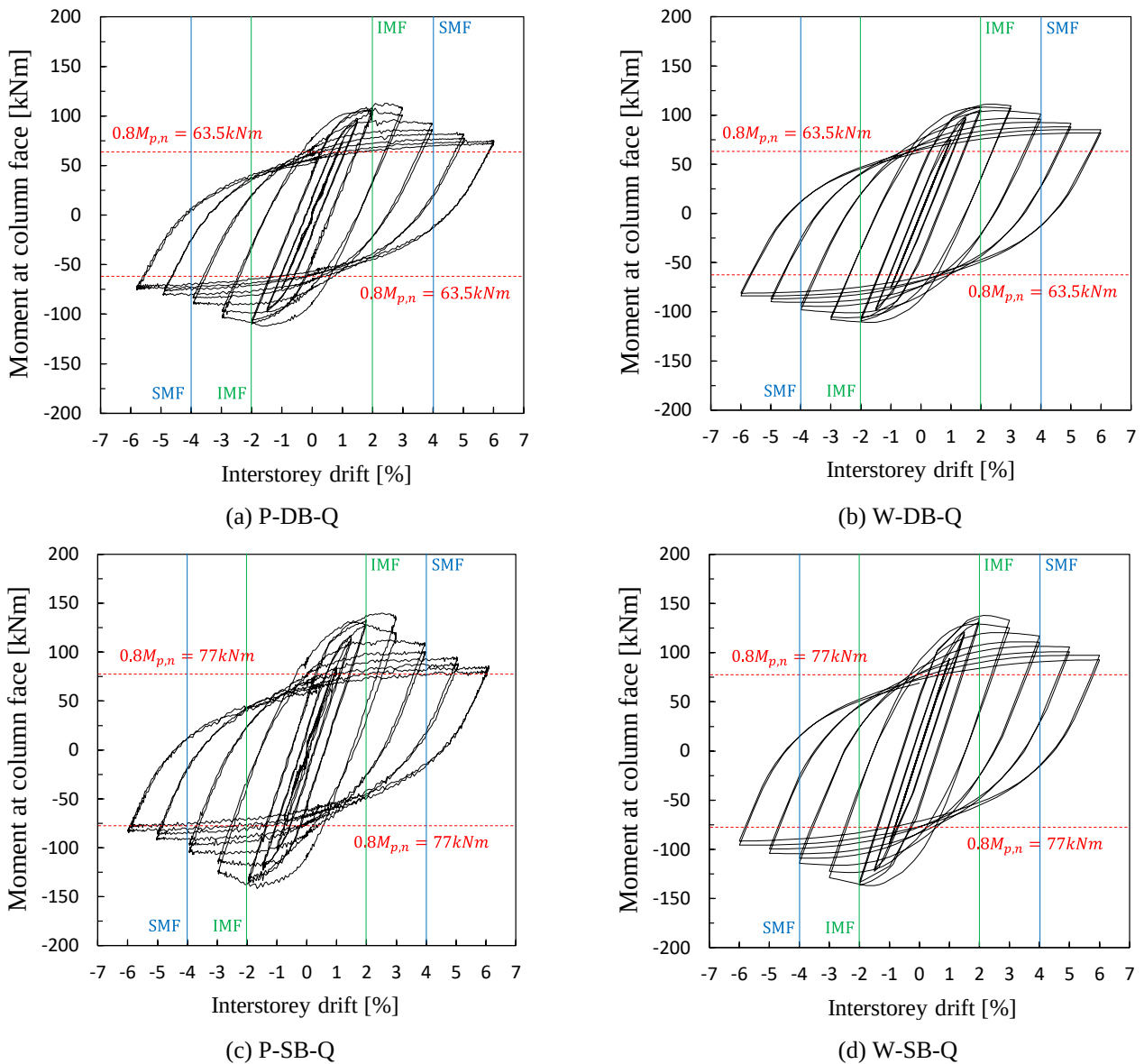
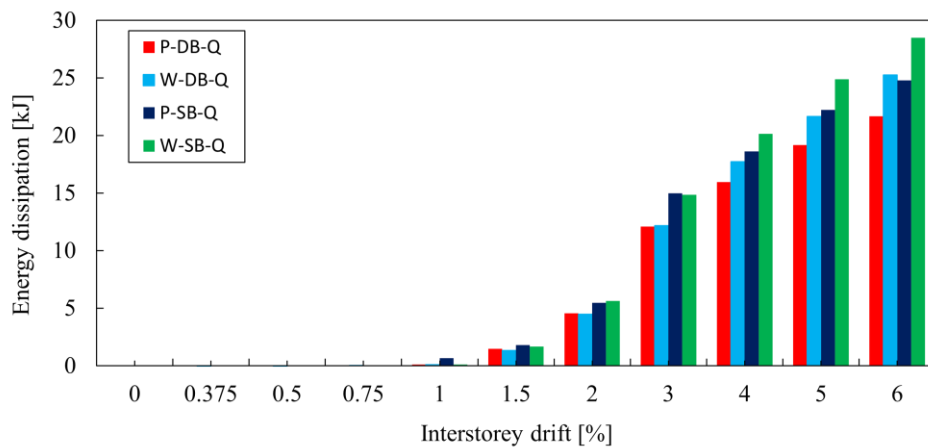


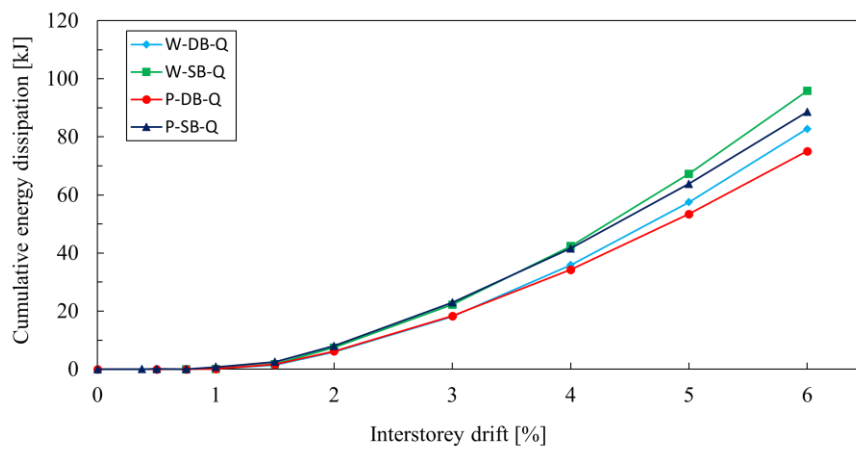
Figure 5.38: Determination of steel moment-resisting frame types of joints according to AISC 341-16

5.4.2.4. Energy Dissipation Capacity

The energy dissipation capacity is an important index that evaluates the seismic performance of a beam-to-column joint, reflecting both the strength and deformation of the joint. The energy dissipated by the inelastic deformation of the joint can be measured as the area enclosed by the load-displacement hysteresis loop. A plump hysteresis loop (with a large enclosed loop area) implies good energy absorption ability and good seismic performance. Figure 5.39 presents the dissipated energy at each level of interstorey drift and the cumulative energy dissipation of each specimen. Both preloaded AJ joints and welded joints exhibited pronounced plastic behaviour by forming plastic hinges in the floor and ceiling beams, gradually dissipating energy. As a result, for all joints, the energy dissipation capacity increased steadily with the increase in interstorey drift level (see Figure 5.39 (a)), even after the beams failed at the ultimate strength (M_{max}). P-SB-Q and W-SB-Q, which had larger ceiling beams than P-DB-Q and W-DB-Q, had greater energy dissipation. Overall, the AJ joints (P-DB-Q and P-SB-Q) showed similar energy dissipation to the welded joints (W-DB-Q and W-SB-Q) because the applied preload effectively prevented gap-opening at the column-to-column connection (see Figure 5.39 (b)).



(a)



(b)

Figure 5.39: Energy dissipation for the AJ and welded joint: (a) Each level of interstorey drift; (b) Cumulative energy dissipation

5.4.2.5. Strength and Stiffness Degradation

Under cyclic loading, both the strength and stiffness of the beam-to-column joint gradually decrease due to the effect of cumulative damage. The strength degradation coefficient (λ_i) and cyclic stiffness (K_c) are widely used to evaluate the failure process of a joint under cyclic loading [59, 138, 139].

$$\lambda_i = \frac{M_i}{M_{max}} \quad (\text{Eq. 5.7})$$

$$K_c = \frac{|+F_i| + |-F_i|}{|+\Delta_i| + |-\Delta_i|} \quad (\text{Eq. 5.8})$$

where:

M_i	The maximum moment strength at the i^{th} loading level
M_{max}	The maximum moment strength
$+F_i$ and $-F_i$	The maximum load at the beam end of the i^{th} loading cycle in the positive and negative directions, respectively
$+\Delta_i$ and $-\Delta_i$	The horizontal displacements corresponding to $+F_i$ and $-F_i$

Figure 5.41 (a) shows the strength degradation coefficient curves for the AJ joints and welded joints. The strength degradation coefficient increases gradually with the increase of the interstorey drift level until the maximum moment strength M_{max} , and decreases from interstorey drift of +3% and -2% due to the local buckling of the floor and ceiling beams. P-SB-Q and W-SB-Q experience sudden loss of strength due to the local buckling of the two beams simultaneously. On the other hand, P-DB-Q and W-DB-Q have the sequential effect of the local buckling of two beams, causing a relatively moderate strength degradation and higher ductility.

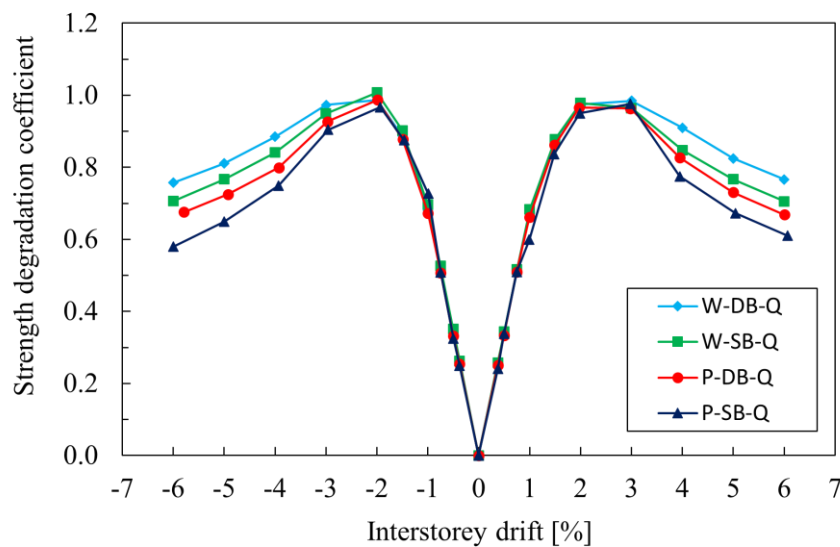


Figure 5.40: Strength degradation

Figure 5.41 shows the cyclic stiffness degradation of each specimen. In all specimens, the stiffness degradation mainly occurred after the beams yielded at the drift of approximately 1-1.2%. The stiffness reduction was caused mainly by the cumulative plastic deformation. The AJ joints (P-DB-Q and P-SB-Q) had similar stiffness degradations to the welded joints (W-DB-Q and W-SB-Q). However, P-DB-Q and P-SB-Q presented different stiffness degradation patterns. In the case of P-SB-Q, the stiffness decreased fast after the floor and ceiling beams yielded simultaneously, while P-DB-Q exhibited a relatively moderate stiffness degradation because the two beams yielded and failed sequentially.

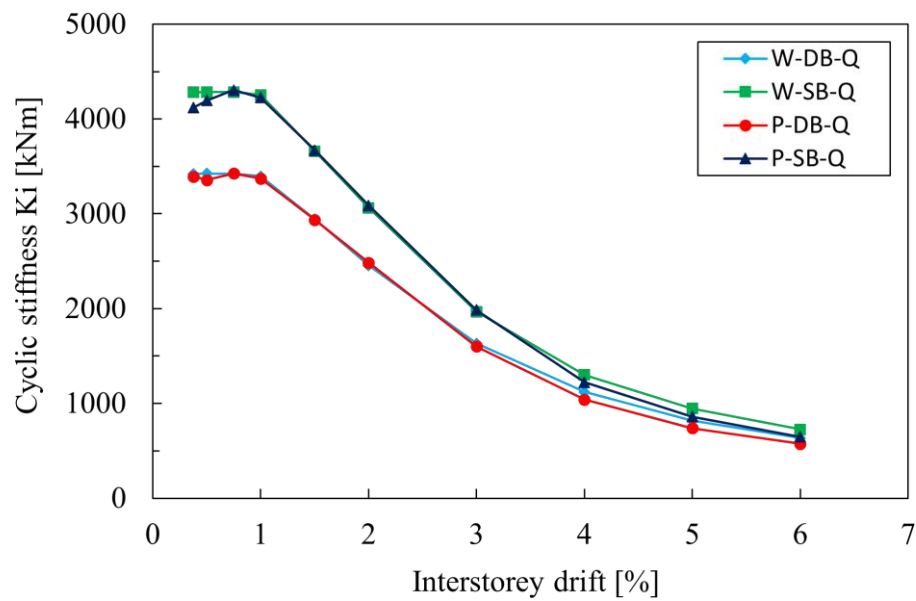


Figure 5.41: Cyclic stiffness degradation

5.4.2.6. Contribution of Components to the Joint Rotation

The beam-to-column joint rotation under lateral force consists of the flexural deformation of the column, the shear deformation of the panel zone (column web panel) and the flexural deformation of the beam. All specimens showed that the plastic deformations occurred in only the floor and ceiling beams (dissipative members), while the columns and panel zone (non-dissipative members) behaved elastically. Also, there was no gap-opening in P-DB-Q and P-SB-Q. The contributions of the abovementioned components to the overall joint rotation are shown in Figure 5.42. It appears that most of the joint rotation was caused by the plastic hinges of the beams. While small, the contribution of the columns to the overall joint rotation was larger than that of the panel zone.

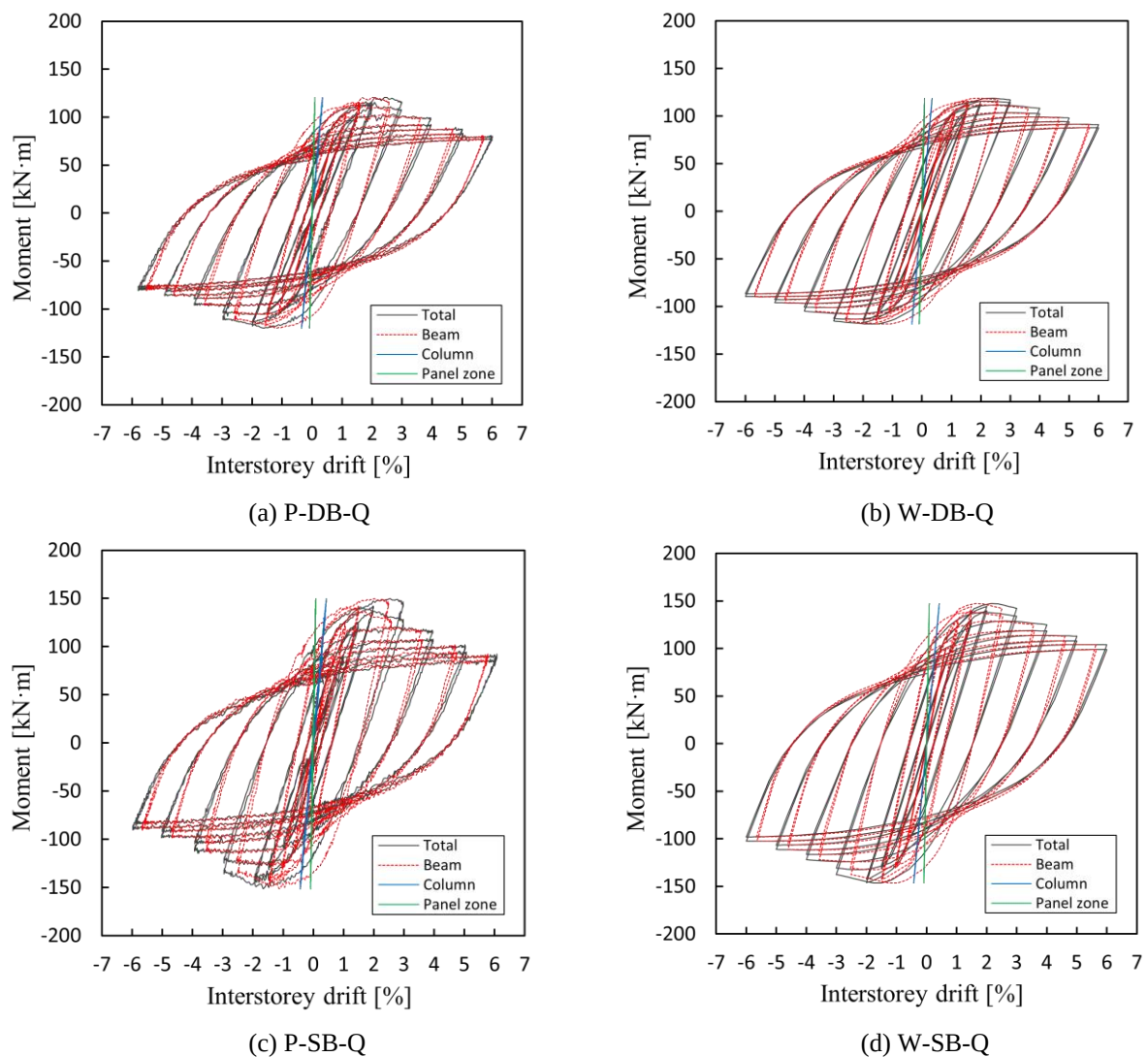


Figure 5.42: Contribution of components to the joint rotation

5.5. Parametric Study

The calibrated FE models are used to perform a parametric study to investigate the influence of critical parameters that were not studied in the experiments.

The beam-to-column tests in Section 4.4 were carried out by placing the columns horizontally because it would have been difficult to apply preload to the columns if the columns had been arranged vertically. For this reason, the supports at the end of the columns were installed under the columns as shown in Figure 5.43 (a), and consequently, the horizontal reaction force R_{x1} generated by the applied lateral force was located on the centreline connecting the pinned support, which was 180mm lower than the column centreline. Since it was expected that the eccentricity of the horizontal reaction force R_{x1} would affect the gap-opening of the AJ connection, the eccentricity was taken into account when calculating the demand preloads for the P-DB and P-SB specimens used for the beam-to-column joint tests (refer to Table 4.41). For the parametric study, the impact of the eccentricity was removed by locating the supports on the column centreline in the FE joint model as shown in Figure 5.43 (b), which reflects the condition of the real structure. Then, the demand preloads for gap-opening P_g and slip P_s at the AJ connection were re-calculated for the parametric study using the same process as in Section 4.4.7.

Table 5.3 presents the demand preloads P_g and P_s for the P-DB and P-SB specimens for the beam-to-column joint tests and parametric study. The demand preloads are those required to prevent the gap-opening and slip from developing at the AJ connection prior to the failure of the floor and ceiling beams. In particular, the demand preload P_g for P-SB was found to be significantly reduced by removing the eccentricity of the horizontal reaction force generated by the pinned support. The parametric study was conducted using the demand preloads P_g shown in the two right-most columns of Table 5.3 as the reference preload. (The detailed calculations of the demand preloads P_g for the P-DB and P-SB specimens are shown in Appendix C.)

Table 5.3: Demand preloads for the P-DB and P-SB specimens for the parametric study

Preload	Beam-to-column joint test (from Table 4.41)		Parametric study	
	P-DB	P-SB	P-DB	P-SB
Demand preload for gap-opening (P_g)	188 kN	131 kN	213 kN	55 kN
Demand preload for slip (P_s) *	229 kN	221 kN	162 kN	202 kN

* The friction coefficient (μ) of 0.3 is used.

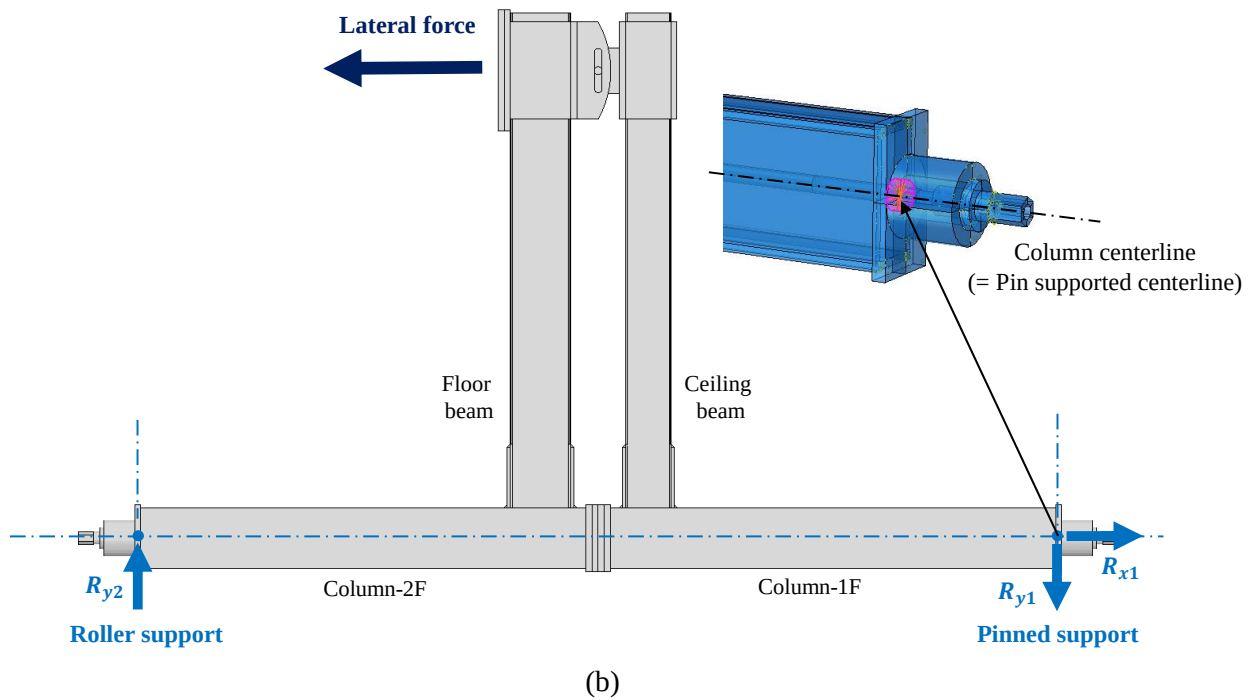
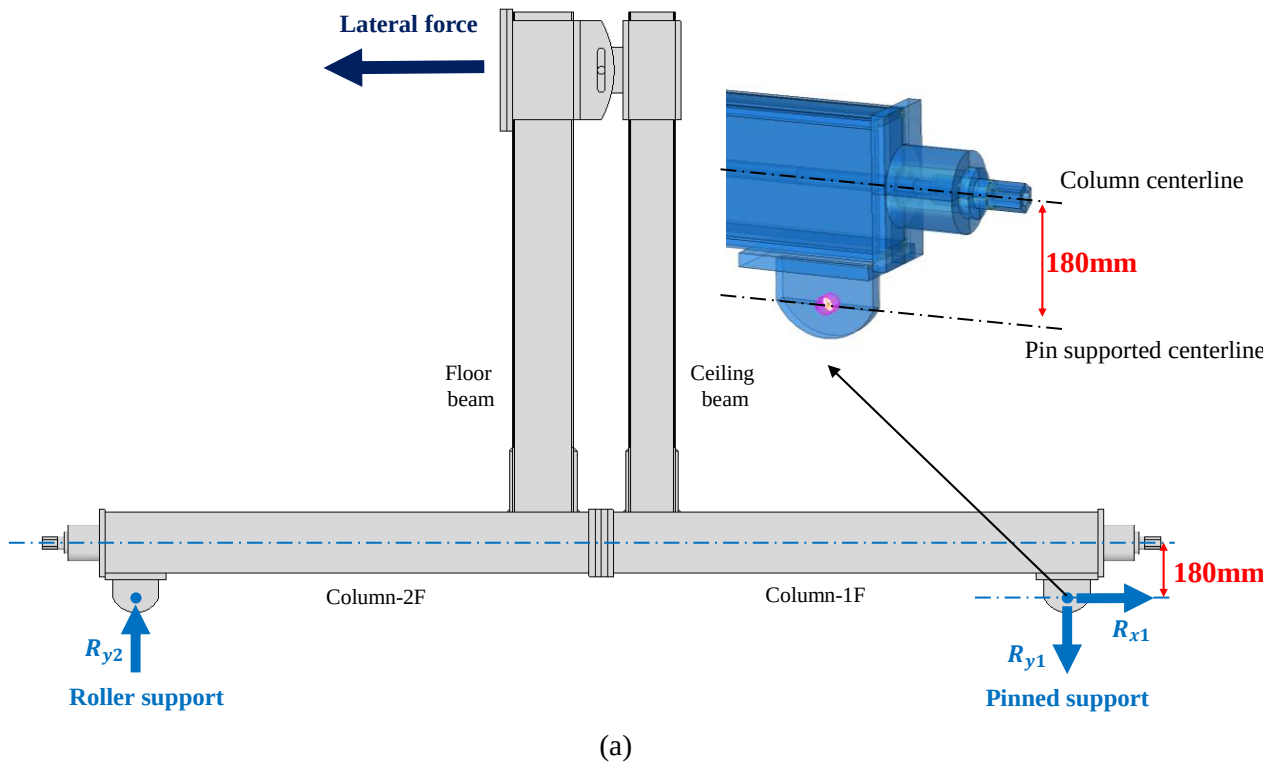


Figure 5.43: Locations of the roller and pinned supports with reaction forces: (a) for the beam-to-column joint tests, and (b) for the parametric study

5.5.1. Influence of Preload Magnitudes

In order to investigate the effects of gap-opening of the AJ connection on the behaviour of beam-to-column joints and SRBs inside columns, the magnitude of the preload P_g was used as a parameter. Specifically, the preload was applied to the FE models for P-DB and P-SB by gradually reducing the preload from the reference preload shown in Table 5.3.

5.5.1.1. Behaviour of Joints Depending on the Preload Magnitudes

(1) P-DB models (Different cross-sectional sizes for the floor and ceiling beams)

The FE simulation results of P-DB (referred to as the ‘DB’) are shown in Figure 5.44 and Table 5.4. The failure mechanism and stress variation in the threads of the SRBs in columns are presented in Figure 5.45 and Figure 5.46, respectively. As shown in Figure 5.44, P-DB eventually developed plastic hinges at the expected locations in both the floor and ceiling beams, and reached the plastic moment strength of the joint (M_p). However, depending on the magnitude of preload, due to the changing effects of the gap-opening, P-DB experienced significantly different failure mechanisms when reaching the ultimate strength.

In the case of DB-1.0P, to which the reference demand preload $P_g=213\text{kN}$ was applied, both gap-opening and slip were prevented completely at the connection. The floor beam yielded first followed by the ceiling beam (see Figure 5.44 and Figure 5.45 (4)). Both beams reached their plastic moment capacities, and the floor beam developed a local buckle at the ultimate strength slightly before the ceiling beam. The coupler and the SRBs inside the columns showed stable behaviour in the elastic range with little changes in tensile stress (see Figure 5.46 (g)). Overall, DB-1.0P exhibited safe and stable plastic behaviour as intended. On the other hand, DB-NoP ($P_g=0\text{kN}$) was affected by gap-opening and slip at the connection as soon as the lateral load was applied, causing the lateral stiffness to decrease. Due to the effect of gap-opening, the ceiling beam yielded earlier than the floor beam (see Figure 5.45 (1)), which was the opposite failure mechanism compared to DB-1.0P. In addition, as shown in Figure 5.44, DB-NoP reached the ultimate strength at a much greater drift than DB-1.0P because of additional drift caused by gap-opening and slip. Regarding the stress variations in the SRBs, the stress in the bolt threads near the connection was subjected to tensile stress (M-1F and M-2F) due to gap-opening and bending stress (T1-T4) caused by a rotation of the coupler when bearing against the plates after the occurrence of slip at the connection (see Figure 5.46 (a)). (Refer to Figure 5.51 (c)-(4) and Section 5.5.1.2 describing the stress variations in the SRBs in detail.) Since the gap-opening and slip occurred from the beginning, the bolt threads of the SRBs in column-2F yielded after the joint reached the yield moment, as shown in Figure 5.46 (a).

As the amount of preload increased, the behaviour of the beam-to-column joint became closer to DB-1.0P because the effect of the gap-opening and slip was reduced. In the case of DB-0.2P and DB-0.4P, the overall behaviour of the joint was similar to that of DB-NoP because of the occurrence of gap-opening and slip in the early elastic stage. DB-0.6P showed improved behaviour, but the bolt thread in column-2F still yielded (see Figure 5.46 (d)). DB-0.8P had very similar behaviour to DB-1.0P (see Figure 5.44). The gap-opening occurred late after the floor beam yielded, so it had little effect on the overall behaviour of the joint and the SRBs. In addition, since the slip was prevented, there was no bending action by the rotation of the coupler on the SRBs (see Figure 5.46 (f)). Consequently, DB-0.8P (i.e. 80% of P_g) showed almost identical behaviour to DB-1.0P with respect to lateral stiffness, failure mechanism, and stress variation in the SRBs.

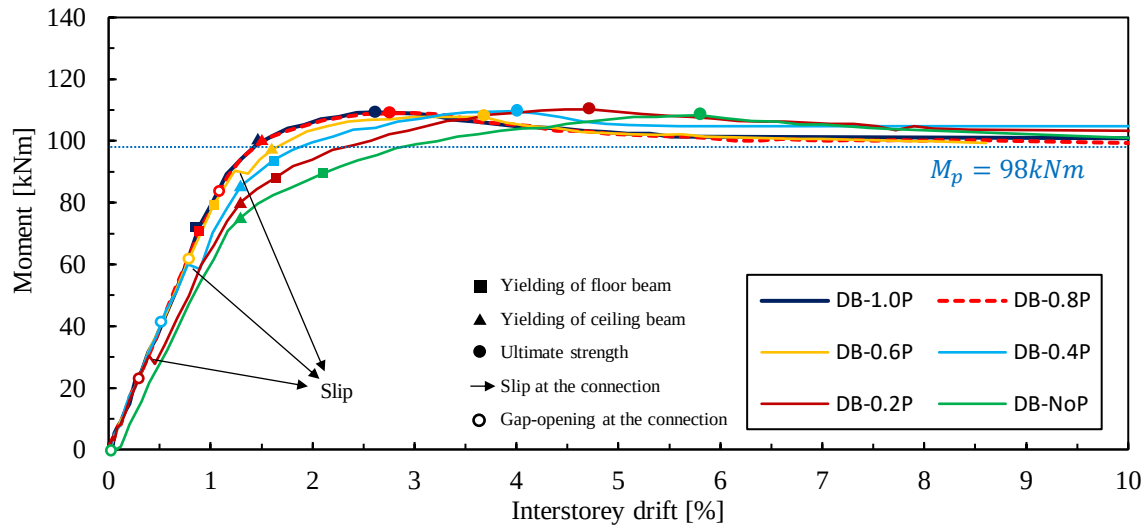


Figure 5.44: FE simulation results: Moment-interstorey drift curves for P-DB models for different magnitudes of preload

Table 5.4: Summary of simulation results for P-DB models for different magnitudes of preload

FEM Model	Preload		K_i [kNm/rad]	M_y [kNm]	θ_y [rad]	M_u [kNm]	θ_u [rad]	Ratio (to DB-1.0P)				
	Ratio	Magnitude [kN]						K_{ini}	M_y	θ_y	M_u	θ_u
DB-1.0P (Ref)	$1.0P_g$	213	8116	72	0.89	109	2.62	1.00	1.00	1.00	1.00	1.00
DB-0.8P	$0.8P_g$	170	7978	71	0.90	109	2.96	0.98	0.99	1.01	1.00	1.05
DB-0.6P	$0.6P_g$	128	7709	79	1.03	108	3.67	0.95	1.09	1.15	0.99	1.40
DB-0.4P	$0.4P_g$	85	6581	85	1.30	110	3.97	0.81	1.18	1.46	1.00	1.52
DB-0.2P	$0.2P_g$	43	6166	80	1.30	109	4.73	0.76	1.11	1.46	1.00	1.81
DB- NoP	$0 P_g$	0	5766	75	1.31	108	5.80	0.71	1.04	1.47	0.99	2.22

FEM Model	Preload		M_s [kNm]	θ_s [rad]	M_g [kNm]	θ_g [rad]	Failure mechanism
	Ratio	Magnitude [kN]					
DB-1.0P (Ref)	$1.0P_g$	213	No slip	No Gap- opening			Floor beam → Ceiling beam
DB-0.8P	$0.8P_g$	170	No slip		81	1.06	Floor beam → Ceiling beam
DB-0.6P	$0.6P_g$	128	90	1.25	61	0.76	Floor & Ceiling beam simultaneously
DB-0.4P	$0.4P_g$	85	61	0.82	41	0.52	Ceiling beam → Floor beam
DB-0.2P	$0.2P_g$	43	30	0.42	22	0.29	Ceiling beam → Floor beam
DB- NoP	$0 P_g$	0	0	0	0	0	Ceiling beam → Floor beam

Note: P_g : the demand preload for preventing gap-opening; K_i : the initial rotational stiffness; M_y : the yield moment strength; M_u : the ultimate moment strength; θ_y : the yield interstorey drift; θ_u : the ultimate interstorey drift; M_s : the moment for the occurrence of slip; M_g : the moment for the occurrence of gap-opening; θ_s : the interstorey drift for the occurrence of slip; θ_g : the interstorey drift for the occurrence of gap-opening.

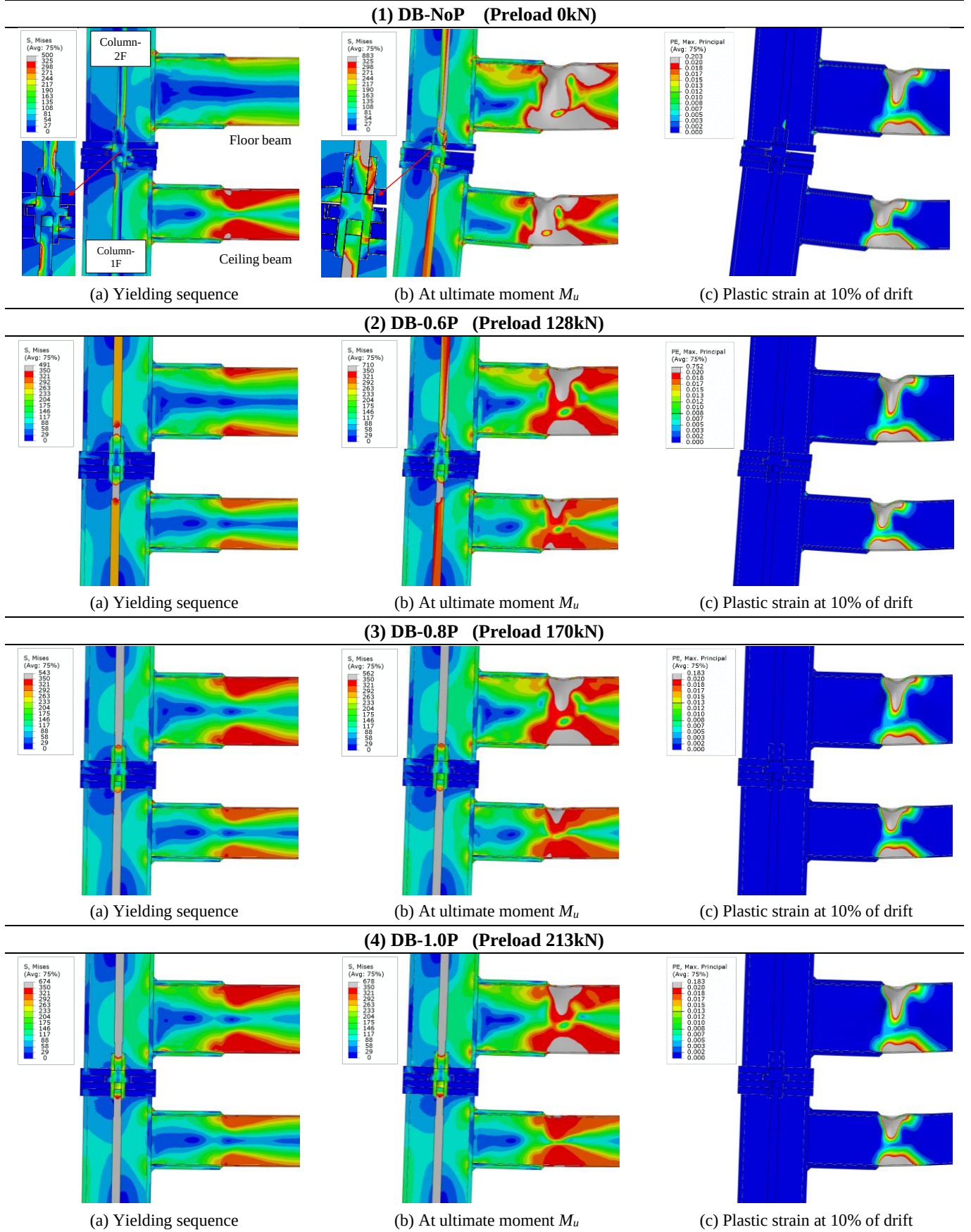


Figure 5.45: Failure mechanism of DB-Models depending on magnitude of preload – cross-sectional view: (a) yielding sequence (Mises stress); (b) local buckling at ultimate moment (von Mises stress); and (c) plastic strain at 10% of interstorey drift.

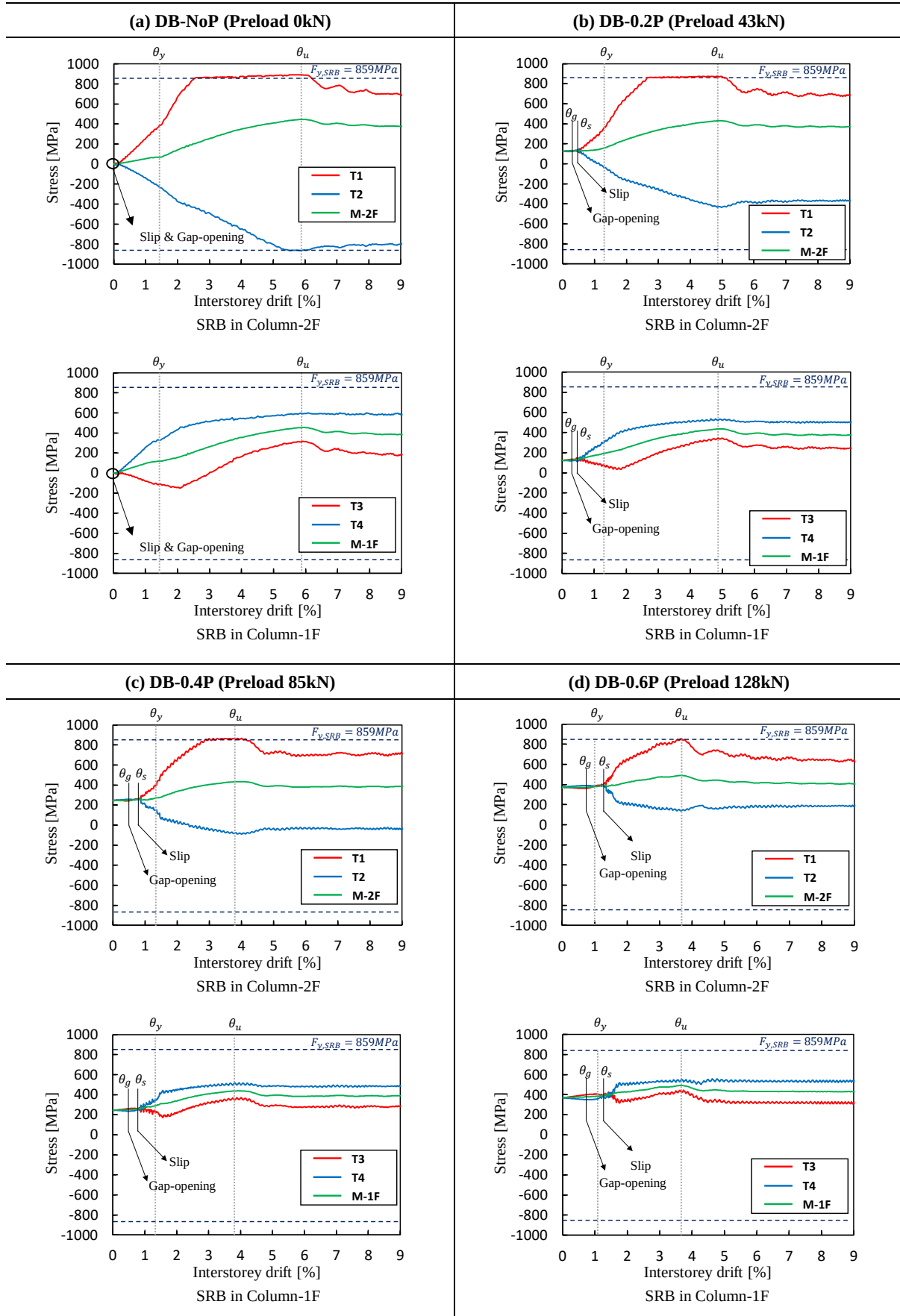


Figure 5.46: Stress variation of SRBs depending on magnitude of preload in DB models (refer to Figure 5.50 and Figure 5.51 for the location of the extracted stress in the SRBs)

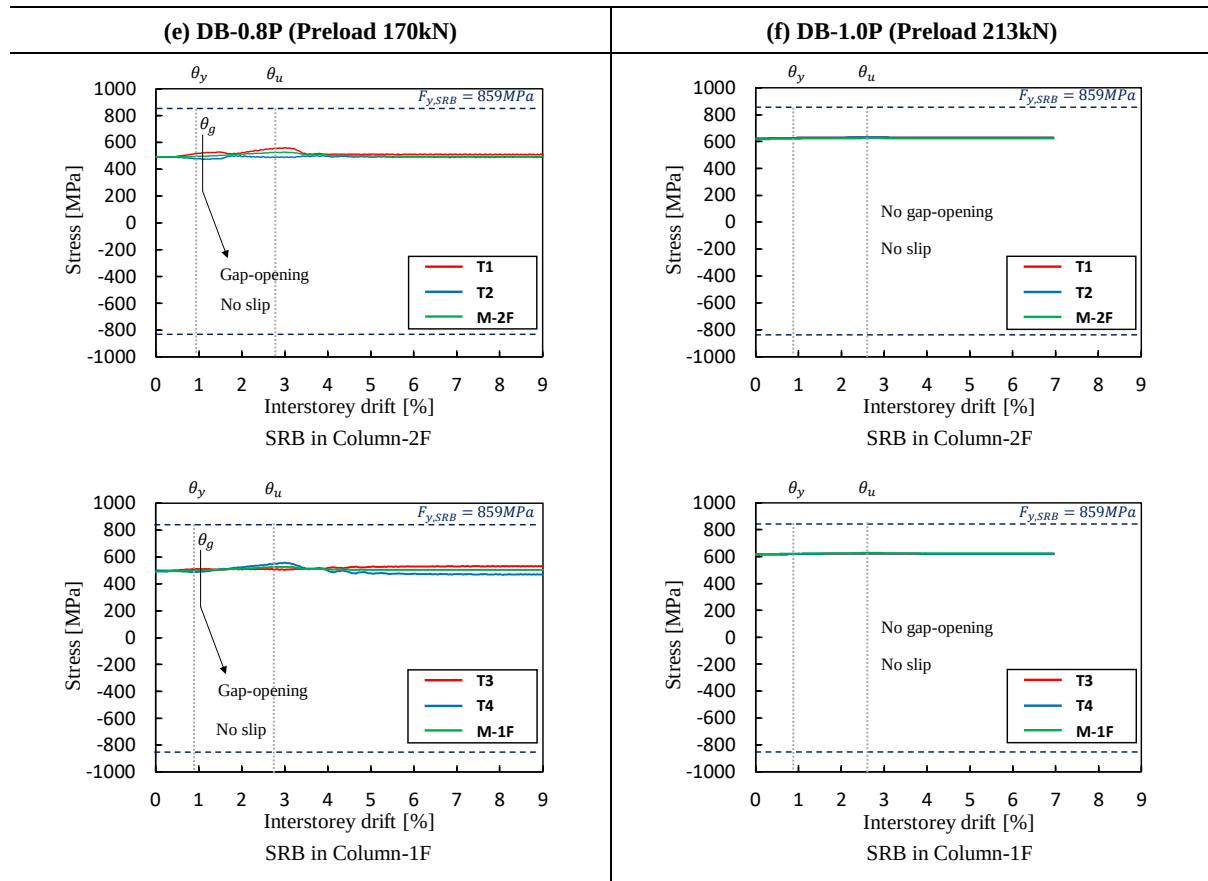


Figure 5.47: Stress variation of SRBs depending on magnitude of preload in DB models (Continued)
(refer to Figure 5.50 and Figure 5.51 for the location of the extracted stress in the SRBs)

(2) P-SB models, (same cross-sectional sizes for floor and ceiling beams)

The FE simulation results of the P-SB (referred to as the ‘SB’) are shown in Figure 5.47 and Table 5.5. The failure mechanism and stress variation in the threads of the SRBs are presented in Figure 5.48 and Figure 5.49, respectively. The simulation results showed that all the P-SB models developed gap-opening, even SB-1.0P to which the analytically calculated demand preload ($P_g=55\text{kN}$) was applied. However, as shown in Figure 5.47, all the SB models presented similar moment-drift response curves, unlike P-DB which showed that the joint behaviour was greatly changed depending on the magnitude of preload. All the SB models reached the plastic moment strength of the joint (M_p) by the floor and ceiling beams yielding and forming the plastic hinges simultaneously. All P-SB models, from SB-NoP to SB-1.0, had the same failure mechanism, and similar initial stiffness.

In terms of the stress variation in the SRBs, the tensile stress increase caused by the gap-opening was insignificant (see M-1F and M2F in Figure 5.49). However, the bolt threads near the connection were subjected to large bending stress after slip occurred (see T1 - T4 in Figure 5.49). Consequently, SB-NoP, SB-0.4P, and SB-0.6P showed that the bolt threads of the SRB in column-2F yielded or almost reached the yield strength. For SB-0.8P, the maximum stress of the SRBs was 30% lower than the yield strength, which is considered acceptable elastic behaviour. SB-1.0P showed more stable stress variation in the SRBs.

In the case of SB models, the demand preload to prevent gap-opening P_g was much lower than the preload required to prevent slip P_s . Therefore, all SB models developed slip in the early stage (see Figure 5.47), which caused large bending stress in the SRBs. In the case of the SB-NoSlip model, in which the slip was prevented by the preload using a large friction coefficient of 0.5, the simulation results showed that both slip and gap-opening could be prevented, resulting in a stable stress variation in the SRBs (see Figure 5.49 (g)).

The FE simulation results showed that P-SB is superior to P-DB in terms of preventing gap-opening. However, unexpectedly, in all P-SB models gap-opening occurred at about 30% lower strength than the expected strength, unlike the P-DB models in which gap-opening occurred at the expected strength. This finding is related to a complicated internal loading mechanism changing the induced preload in P-SB when slip occurs, as described in detail in Section 5.5.1.2 (3).

(3) Summary

Although much larger demand preloads were applied to the DB models than the SB models, they were more vulnerable to gap-opening and were greatly affected by it (i.e. reducing the lateral stiffness of the joint, tensile stress increase of the SRBs in columns, and change of the failure mechanism and plastic behaviour of the joint resulting in delaying the reach plastic moment strength of the joint). On the other hand, the SB models had similar moment-drift response curves regardless of the gap-opening.

Consequently, the FE simulation results confirmed that the AJ connection is much less affected by the gap-opening if the floor and ceiling beams have the same cross-section size (i.e. the same bending stiffness EI). In other words, the demand preload for preventing gap-opening can be significantly reduced, compared to using different cross-sectional sizes for the floor and ceiling beams.

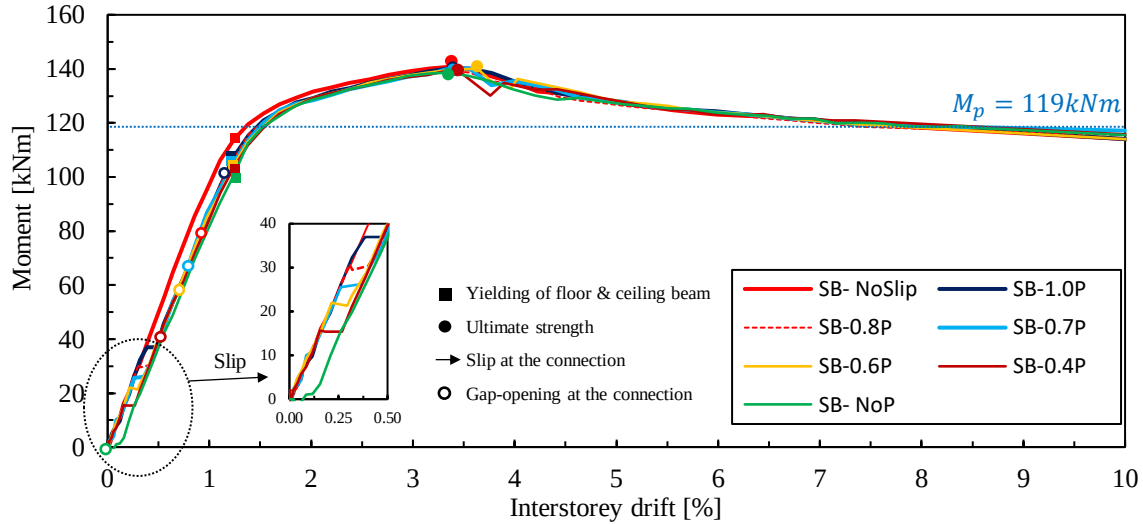


Figure 5.47: FE simulation results: Moment-interstorey drift curves for P-SB models for different magnitudes of preload

Table 5.5: Summary of the simulation results on P-SB models depending on different magnitudes of preload

FEM Model	Preload Ratio	Preload Magnitude [kN]	K_i [kNm/rad]	M_y [kNm]	θ_y [rad]	M_u [kNm]	θ_u [rad]	Ratio (to SB-1.0P)				
								K_{ini}	M_y	θ_y	M_u	θ_u
SB-1.0P (Ref)	$1.0P_g$	55	8692	108	1.24	140	3.36	1.00	1.00	1.00	1.00	1.00
SB-0.8P	$0.8P_g$	44	8600	106	1.24	139	3.36	0.99	0.99	1.00	0.99	1.00
SB-0.6P	$0.6P_g$	33	8433	104	1.24	139	3.36	0.97	0.97	1.00	1.00	1.00
SB-0.4P	$0.4P_g$	22	8363	103	1.23	140	3.44	0.96	0.96	0.99	1.00	1.03
SB- NoP	$0 P_g$	0	8138	100	1.23	139	3.35	0.94	0.93	0.99	0.99	1.00
SB- NoSlip	$1.0P_s$	140*	9188	114	1.24	141	3.36	1.06	1.06	1.00	1.01	1.00

FEM Model	Preload Ratio	Preload Magnitude [kN]	M_s [kNm]	θ_s [rad]	M_g [kNm]	θ_g [rad]	Failure mechanism
SB-1.0P (Ref)	$1.0P_g$	55	37	0.39	102	1.17	Floor and ceiling beams simultaneously
SB-0.8P	$0.8P_g$	44	31	0.31	79	0.92	Ceiling beam → Floor beam
SB-0.6P	$0.6P_g$	33	22	0.21	59	0.70	Ceiling beam → Floor beam
SB-0.4P	$0.4P_g$	22	17	0.16	41	0.51	Ceiling beam → Floor beam
SB- NoP	$0 P_g$	0	0	0	0	0	Ceiling beam → Floor beam
SB- NoSlip	$1.0P_s$	140*	135	2.38	No Gap- opening		Floor and ceiling beams simultaneously

* Demand preload for preventing the slip (P_s) is calculated using friction coefficient (μ) 0.5 for blasted surface.

Note: P_g : the demand preload for preventing gap-opening; K_i : the initial rotational stiffness; M_y : the yield moment strength; M_u : the ultimate moment strength; θ_y : the yield interstorey drift; θ_u : the ultimate interstorey drift; M_s : the moment for the occurrence of slip; M_g : the moment for the occurrence of gap-opening; θ_s : the interstorey drift for the occurrence of slip; θ_g : the interstorey drift for the occurrence of gap-opening.

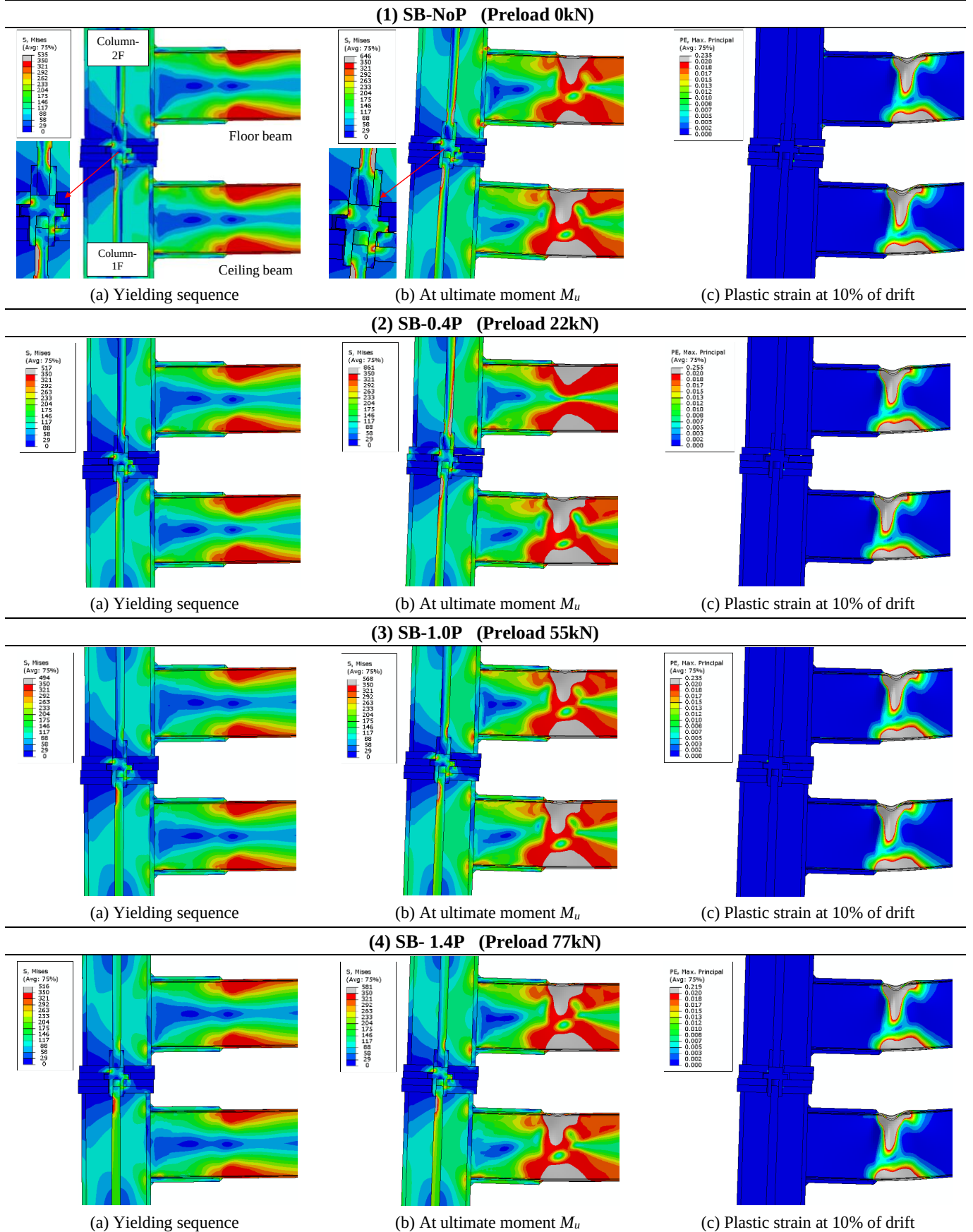


Figure 5.48: Failure mechanism of SB-Models for different magnitudes of preload – cross-sectional view: (a) yielding sequence (Mises stress); (b) local buckling at ultimate moment (Mises stress); and (c) plastic strain at 10% of interstorey drift.

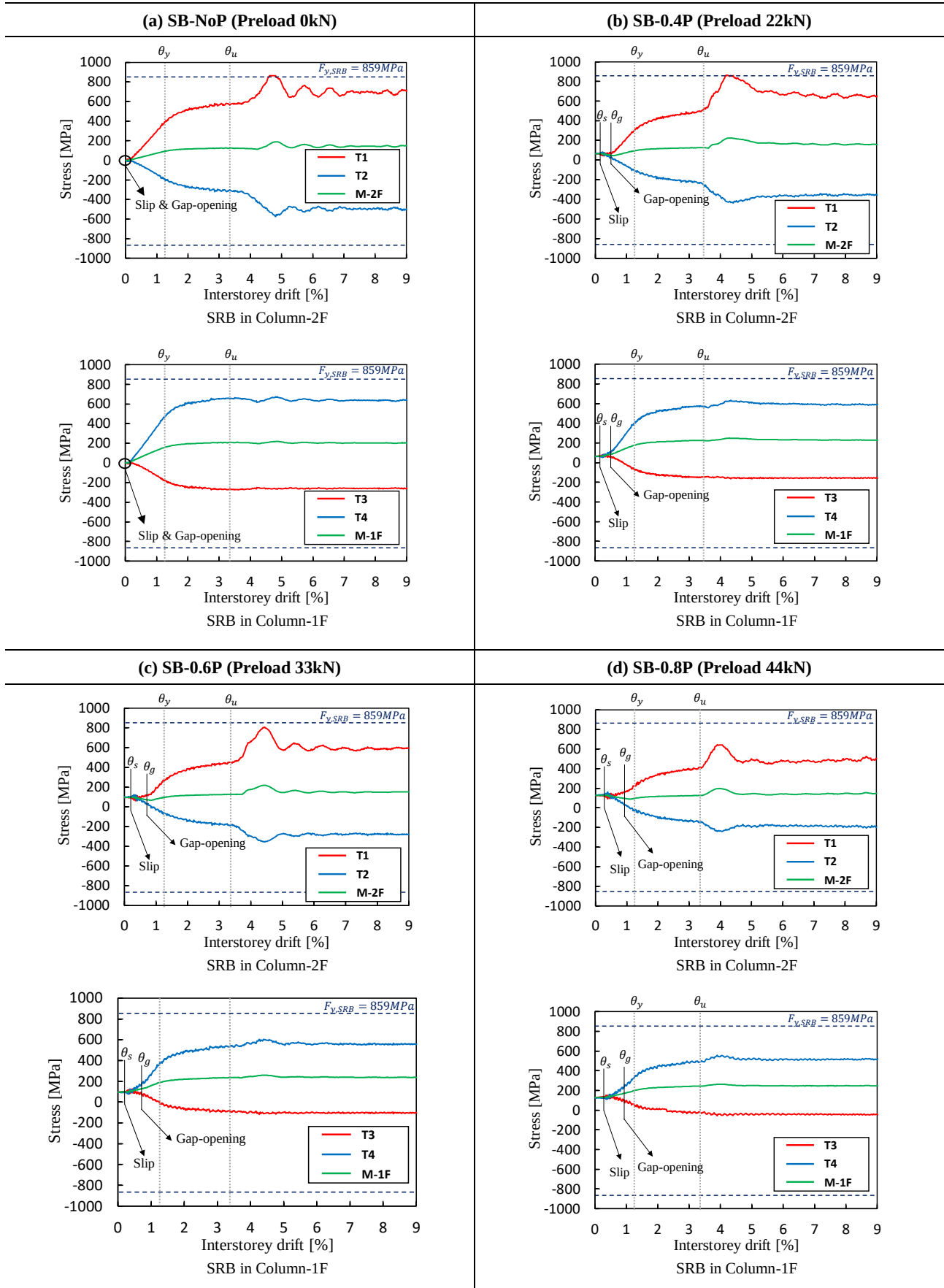


Figure 5.49: Stress variation of SRBs for different magnitudes of preload in SB models (refer to Figure 5.50 and Figure 5.51 for the location of the extracted stress in the SRBs)

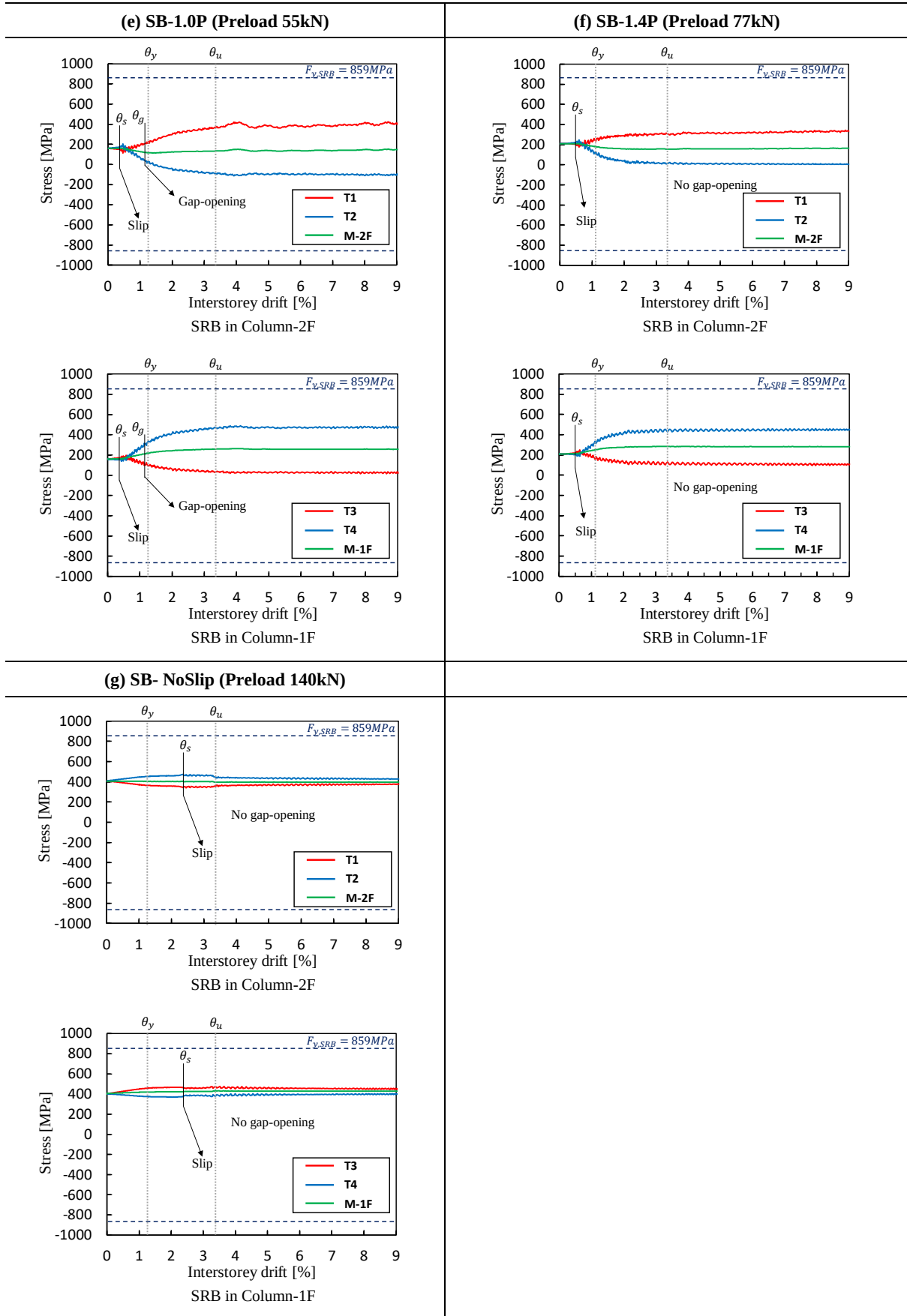


Figure 5.50: Stress variation of SRBs for different magnitudes of preload in SB model (Continued)
(refer to Figure 5.50 and Figure 5.51 for the location of the extracted stress in the SRBs)

5.5.1.2. Stress Variations in the Steel Rod Bolts (SRB)

(1) Locations of the measured stresses on the SRBs

Figure 5.46 and Figure 5.49 show the stress variations in the SRBs for the P-DB and P-SB models, respectively. Figure 5.50 shows the locations of the measured stresses in the SRBs. The stresses were measured on the bolt thread (T1, T2, T3, and T4) for effective stress (i.e. bending and axial stresses). To measure the pure tensile stress caused by the effect of the gap-opening at the AJ connection, stresses were measured at the centre of the bolt threads (M-1F and M-2F).

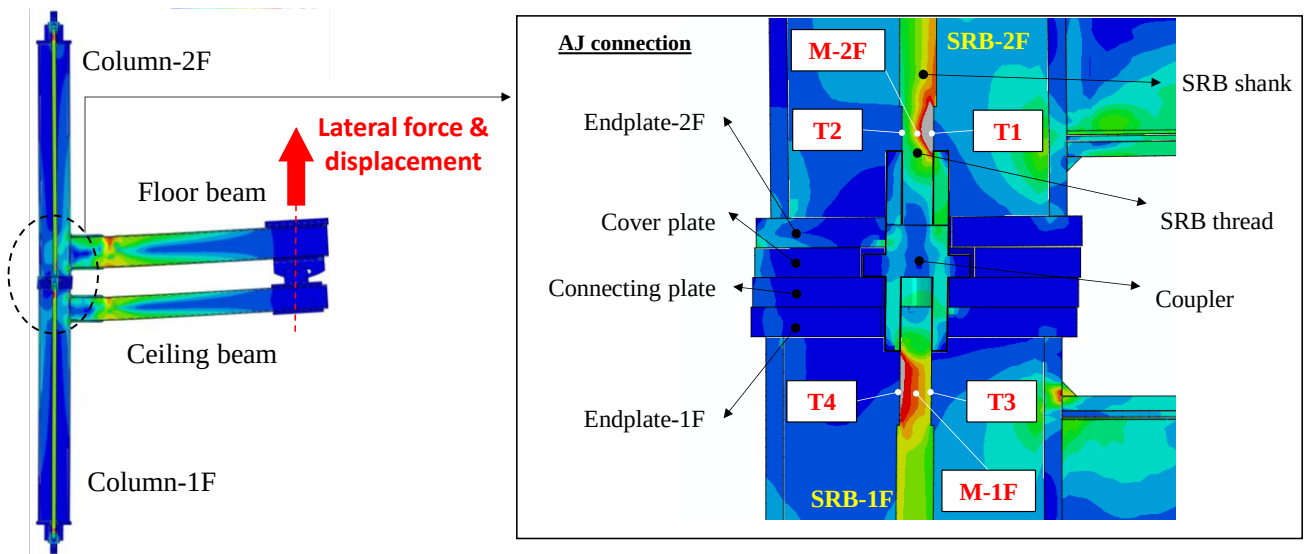


Figure 5.50: Locations of the measured stresses on the SRBs – Cross-sectional view

(2) The effect of gap-opening and slip at the connection on the SRBs

If gap-opening and slip occur at the AJ connection, the SRBs near the connection are subjected to complex combinations of stresses, (see Figure 5.51 (c)), including:

- (i) Tensile stress due to induced preload (σ_p)
- (ii) Bending stress due to flexural deformation of the columns induced by the lateral force (σ_f)
- (iii) Tensile stress due to gap-opening at the AJ connection (σ_g)
- (iv) Bending stress due to slip at the AJ connection (caused by the rotation of the coupler when bearing against plates after the occurrence of slip) (σ_s)

Figure 5.51 explains the effect of the gap-opening and slip at the connection on the SRBs. Figure 5.51 (a) shows the stress variations in the SRBs of the DB-0.6P model as an example, and Figure 5.51 (b) presents the stress variation of the bolt thread (T1 and T2) in column-2F. First, the bolt thread has the initial tensile stress σ_p induced by the preload. Then, the bolt threads near the connection are subjected

to bending stress σ_f due to the flexural deformation of the columns when the lateral force is applied. After the gap-opening occurs at the connection (θ_g), the bolt thread and shank are subjected to tensile stress σ_g by the SRBs resisting the lateral force. When the slip occurs at the connection (θ_s), the plates bear against the coupler, making the coupler rotate. As a result, the bolt threads near the connection are subjected to large bending stress σ_s . Both σ_g and σ_s increase until the beam-to-column joint reaches the ultimate strength (θ_u). Therefore, the bolt thread in column-2F almost reaches the yield stress $F_{y,SRB}$ due to the various stresses (σ_p , σ_f , σ_g and σ_s). On the other hand, while the bolt threads in column-1F (see Figure 5.51 (a)) are subjected to the same effective stresses as the threads in column-2F, the magnitude of stresses is significantly lower.

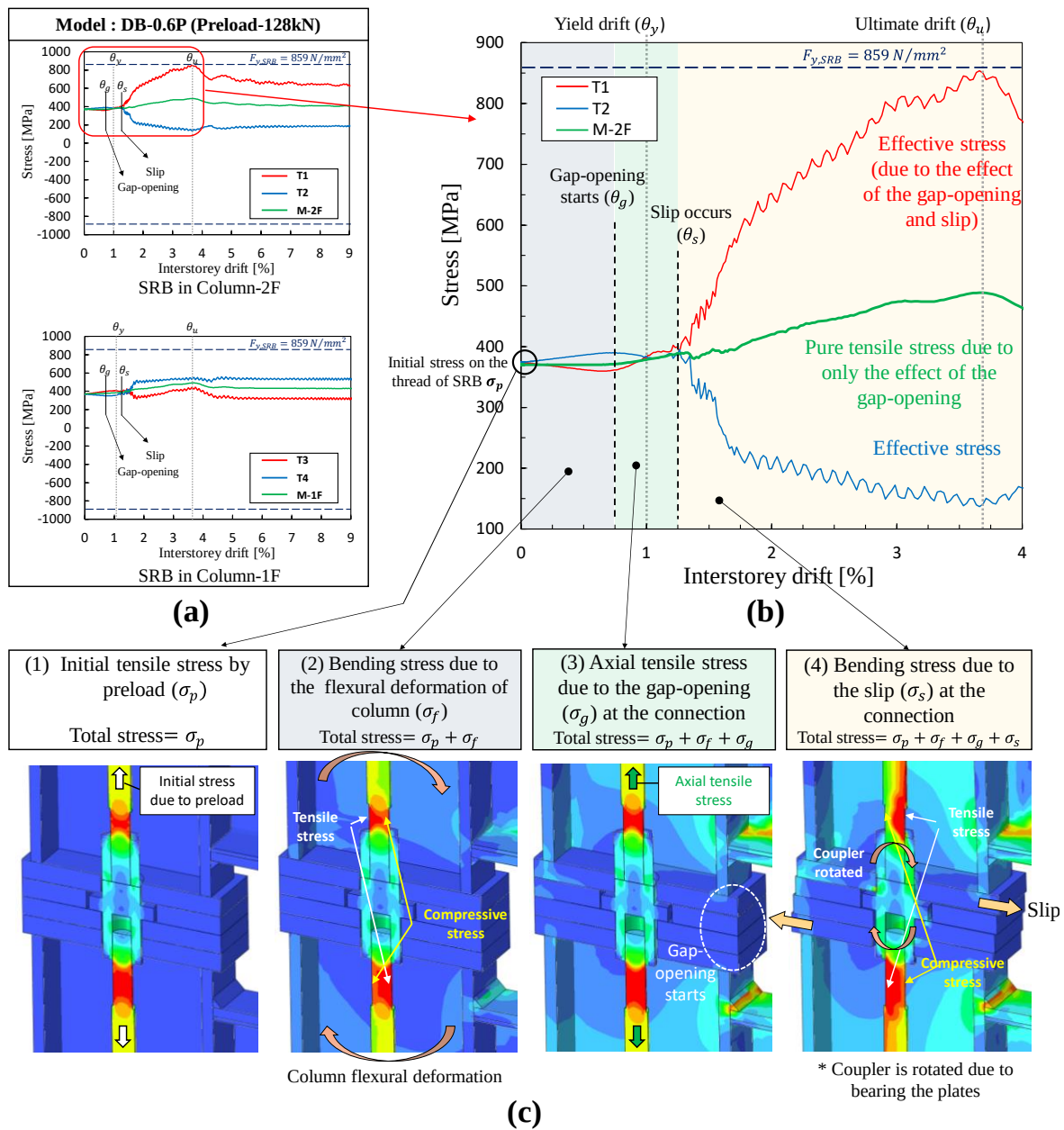


Figure 5.51: Effective stresses acting on the bolt thread of SRBs near the AJ connection: (a) and (b) Stress variations in the SRBs of DB-0.6P model; (c) Various stresses acting on the SRBs

The FE simulation results of the P-DB and P-SB joints (see Figure 5.46 and Figure 5.49) showed the following behaviour of the SRBs:

- In case gap-opening occurs at the connection, the SRBs in the upper and lower columns are subjected to additional tensile stress by resisting the lateral force. The effect of gap-opening is the same for the SRBs in the upper and lower columns.
- In case slip occurs at the connection, the bolt threads near the connection are subjected to large bending stress caused by a rotation of the coupler when bearing against plates. The effect of slip is different for the bolt threads in the upper and lower columns depending on the difference in the floor and ceiling beam size. Hence, it is difficult to predict the critical location and amount of bending stress acting on the bolt threads.
- If the preload prevents gap-opening and slip, the SRB maintains the stress due to the initial preload without significant change until the floor and ceiling beams form plastic hinges and fail (see Figure 5.46 (g)).
- Bending stress due to slip at the connection is more critical for the SRBs than tensile stress due to the gap-opening. The magnitude of bending stress acting on the bolt threads varies greatly with the occurrence of slip. Therefore, even if the AJ connection is designed as a bearing type connection (implying the demand preload is determined to prevent gap-opening P_g), it is recommended to check the demand preload for preventing the slip P_s and apply at least 50% of P_s .

(3) Reduction of induced preload in P-SB due to the effect of slip

Figure 5.52 (a) and (b) show the axial tensile stress variation measured at the centre of the SRBs in the DB-0.4P and SB-0.6P models, respectively. In both models, the axial stress variation of the SRBs starts from the initial tensile stress (σ_p) by the induced preload. The axial stress in the SRBs increases if gap-opening occurs at the connection. If the gap-opening occurs and disappears in the elastic range, the axial stress in the SRBs reverts to σ_p . It cannot be decreased below σ_p unless the couplers are untightened.

In DB-0.4P, the axial stresses of the SRBs in column-1F and column-2F increase almost identically from the initial stress σ_p after gap-opening occurs (θ_g), regardless of the occurrence of slip. On the other hand, in SB-0.6P, the axial tensile stresses of the SRBs in column-1F and column-2F had different variations after the slip occurs (θ_s), even though gap-opening did not occur. After slip occurred, the stress of the SRB in column-1F increased, while the stress of the SRB in column-2F decreased. In other words, stress reduction in SRB in column-2F means its clamping force is reduced after the occurrence of slip. The reduction in stress is approximately 30% of the initial stress σ_p . This phenomenon in the P-SB model seems to be related to the behaviour of the coupler. Figure 5.53 shows the stress distribution at the connection after the slip occurred in the DB-0.4P and SB-0.6P models at the same lateral force (50kN). In both models, one side of the coupler plate pressed the connecting plate. In the SB-0.6P model, the coupler incurs a larger rotation and a larger pressing force was generated, compared to the DB-0.4P model. In this process, the coupler pulls the SRB in column-1F (causing an increase in tensile axial stress), while the SRB in column-2F is compressed slightly (causing a decrease in axial tensile stress). This is the reason why the P-SB models developed earlier gap-opening than expected, as described in Section 5.5.1.1.

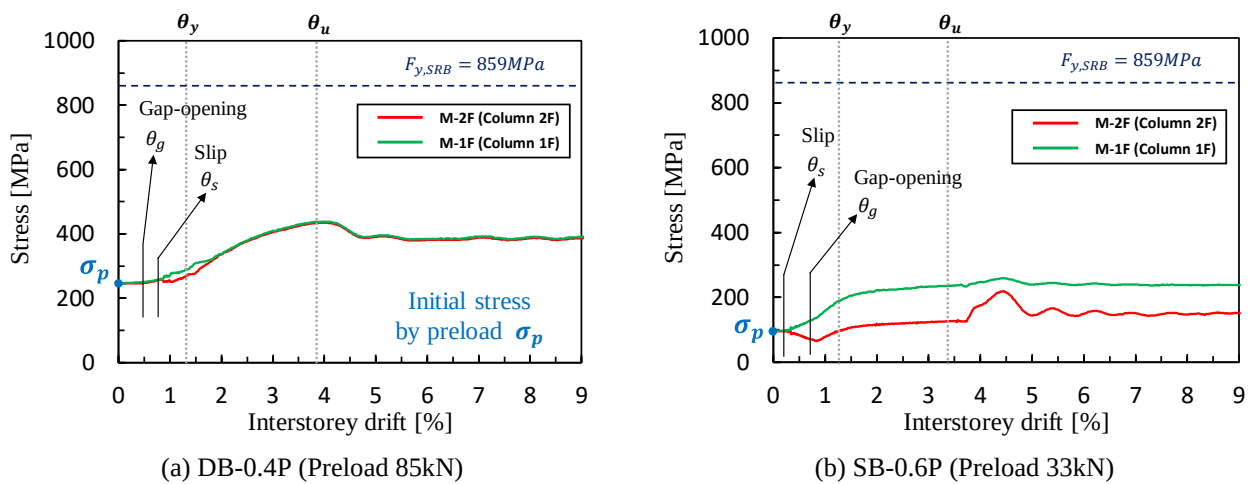
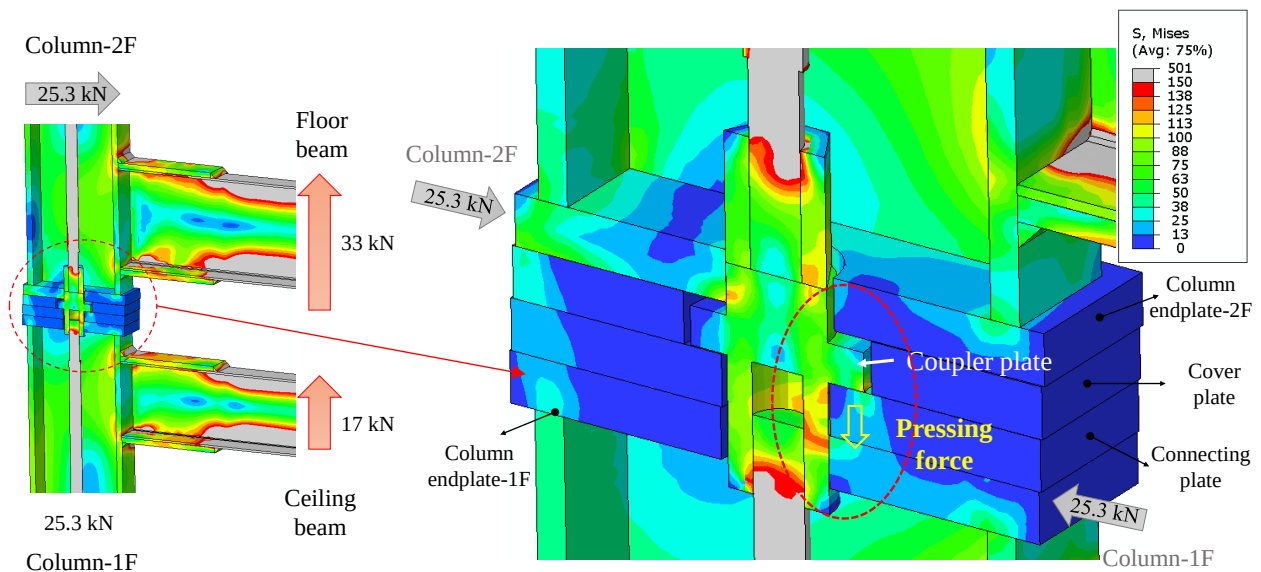
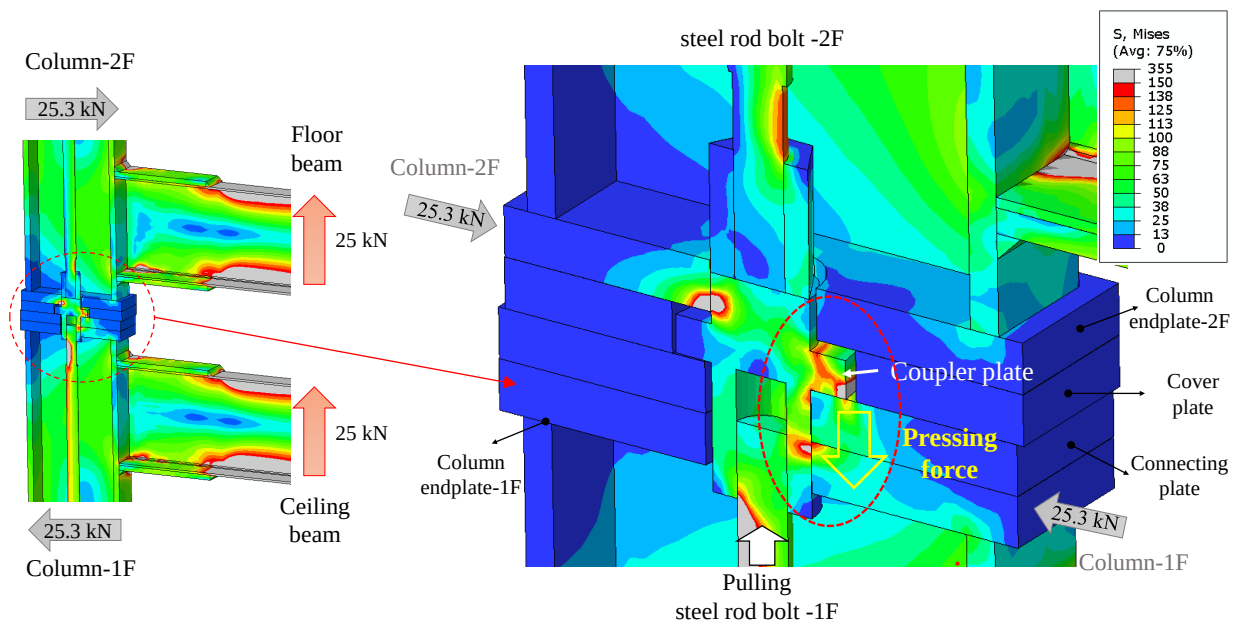


Figure 5.52: Axial stress variation of SRBs in column-1F and column-2F (using DB-0.4P and SB-0.6P models as an example)



(a) DB-0.4P (Preload 85kN): Lateral force 50kN and drift 1.16%



(b) SB-0.6P (Preload 33kN): Lateral force 50kN and drift 0.86%

Figure 5.53: Von Mises stress distribution at the connection of DB-M and SB-M specimens - Cross-sectional view

5.5.1.3. Validation of the Design Methods for Determining Demand Preloads

The demand preloads for preventing gap-opening P_g and slip P_s at the AJ connection are calculated according to the design methods presented in Sections 3.4.3 and 3.4.4. In this section, the demand preloads determined using these methods are validated by comparing the FE simulation results in the previous sections with the critical strength for gap-opening and slip provided in Section 3.5.3.

(1) Validation of demand preload for preventing gap-opening at connection (P_g)

Section 3.5.3 provides the critical strength for gap-opening $F_{cr,g}$ of the AJ connection of the beam-to-column joint. $F_{cr,g}$ is the maximum lateral force that can be applied to the connection before gap-opening occurs at the induced preload. In the FE simulations, the occurrence of gap-opening is determined based on when the plates at the connection lose clamping stresses at the location where the gap-opening occurs and the axial tensile stress of the SRBs begins to increase.

Figure 5.54 and Table 5.6 presents the comparison of the FE simulation results with the critical strength for gap-opening for the P-DB models. The simulation results showed that the location and strength of the gap-opening agreed closely with the predicted location and the critical strength $F_{cr,g}$. The difference between the simulation results and predicted strength is less than 3%. On the other hand, the gap-opening of the P-SB models occurred at about 30% lower strength than the predicted critical strength as shown in Figure 5.55 and Table 5.7. As described in Section 5.5.1.2 (3), the FE simulation results of P-SB showed that the preload in the SRB of the upper column is reduced by approximately 30% from the initially induced preload due to loading and deformation of the coupler after slip occurs. In order to prevent the gap-opening completely (only in case slip occurs earlier than gap-opening), the P-SB models require approximately 30-40% higher preload than the analytically calculated demand preload. In the SB-1.4p model with a 40% increase in demand preload ($1.4P_g=77\text{kN}$), the gap-opening was prevented completely prior to the failure of the floor and ceiling beams, and the stress variation in the SRBs became more stable (see Figure 5.49 (f)).

Consequently, the FE simulation results confirmed the validity of the suggested design method in Section 3.4.3 for determining the demand preload for preventing gap-opening P_g . However, the simulation results also showed that if slip occurs earlier than gap-opening, it can cause a change in the induced preload. It is necessary to take this potential change into consideration in determining the demand preload.

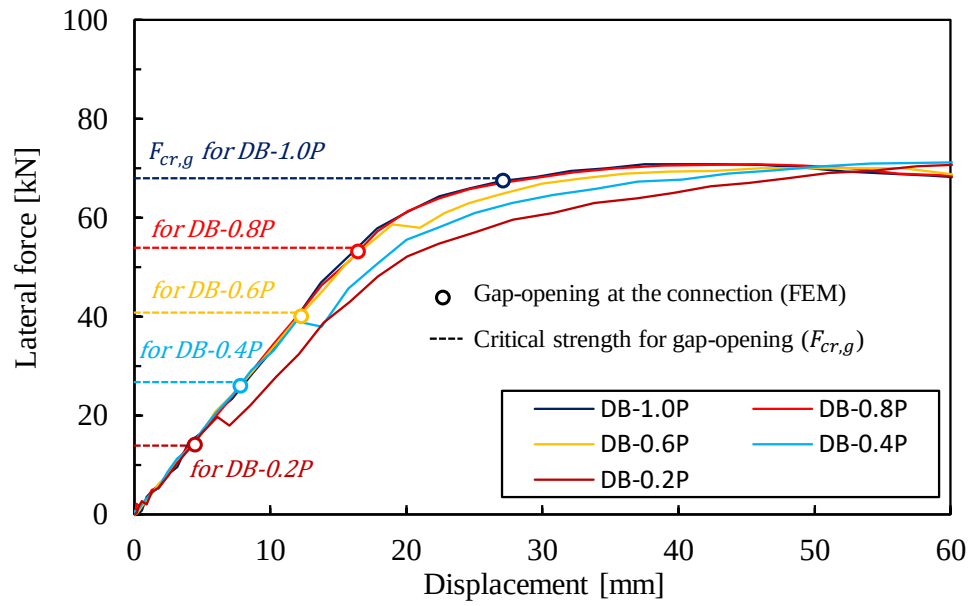


Figure 5.54: Comparison of FE simulation results with the critical lateral force $F_{cr,g}$ for gap-opening in P-DB models

Table 5.6: Comparison of FE simulation results with the critical lateral force $F_{cr,g}$ for gap-opening in P-DB models

FE Model	Preload Ratio	Preload Magnitude [kN]	Analytically expected strength $F_{cr,g}$ [kN]	FEM result $F_{FEA,g}$ [kN]	Ratio $F_{FEA,g} / F_{cr,g}$
DB-1.0P	$1.0P_g$	213	68 (B)	67 (B)	0.99
DB-0.8P	$0.8P_g$	170	54 (B)	53 (B)	0.98
DB-0.6P	$0.6P_g$	128	41 (B)	40 (B)	0.97
DB-0.4P	$0.4P_g$	85	27 (B)	26 (B)	0.98
DB-0.2P	$0.2P_g$	43	14 (B)	14 (B)	1.01
DB- NoP	$0 P_g$	0	0 (B)	0 (B)	0

(): The first gap-opening location at the AJ connection in Figure 3.33.

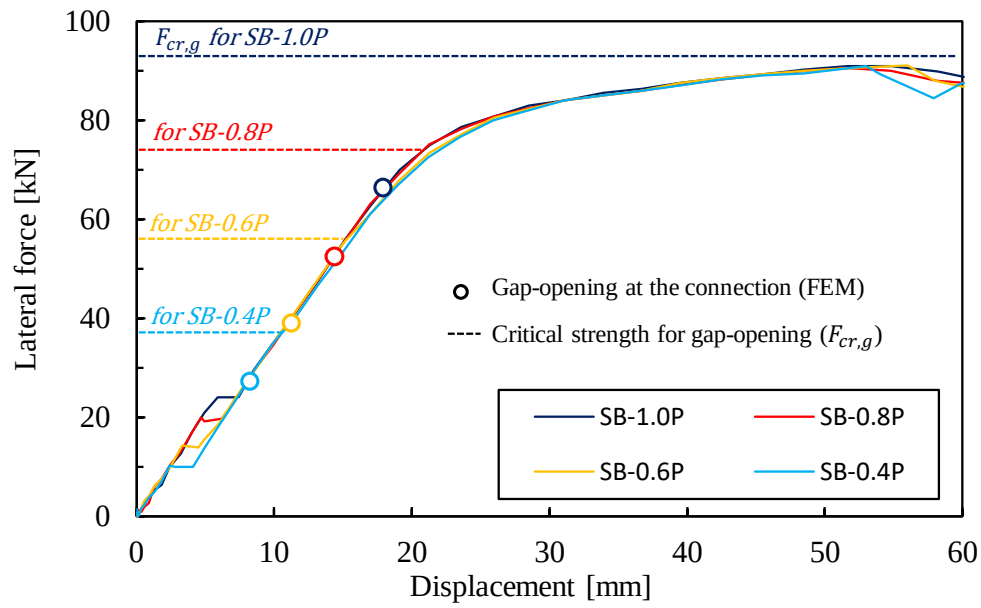


Figure 5.55: Comparison of FE simulation results with the critical lateral force $F_{cr,g}$ for gap-opening in P-SB models

Table 5.7: Comparison of FE simulation results with the critical lateral force $F_{cr,g}$ for gap-opening in P-SB models

FE Model	Ratio	Preload Magnitude [kN]	Analytically expected strength $F_{cr,g}$ [kN]	FEM result $F_{FEA,g}$ [kN]	Ratio $F_{FEA,g} / F_{cr,g}$
SB-1.0P	$1.0P_g$	55	93 (B)	66 (B)	0.71
SB-0.8P	$0.8P_g$	44	74 (B)	52 (B)	0.70
SB-0.6P	$0.6P_g$	33	56 (B)	38 (B)	0.68
SB-0.4P	$0.4P_g$	22	37 (B)	27 (B)	0.73
SB- NoP	$0 P_g$	0	0 (B)	0 (B)	-
SB- 1.4P	$1.4P_g$	77	No gap-opening	No gap-opening	-

(): The first gap-opening location at the AJ connection in Figure 3.33.

(2) Validation of demand preload for preventing slip at connection (P_s)

Section 3.5.3 provides the critical strength for slip $F_{cr,s}$ at the AJ connection. $F_{cr,s}$ is the maximum lateral force that can be applied to the connection before slip occurs at the induced preload.

For the FE joint models and for calculating the critical slip strength $F_{cr,s}$, the friction coefficient (μ) of 0.3 was used for the contact surfaces between the plates at the AJ connection. The comparison of the FE simulation results with the critical strength for slip is shown in Figure 5.56 and Table 5.8 for the P-DB models, and Figure 5.57 and Table 5.9 for the P-SB models. In the case of the P-DB models, the FE simulation results showed that slip occurs at 8-10% higher strengths than the predicted critical strength $F_{cr,s}$. For the P-SB models, the connections developed slip nearly at the critical strength $F_{cr,s}$.

Consequently, the FE simulation results confirmed the validity of the suggested design method in Section 3.4.4 for the demand preload required to prevent slip P_s of the AJ connection.

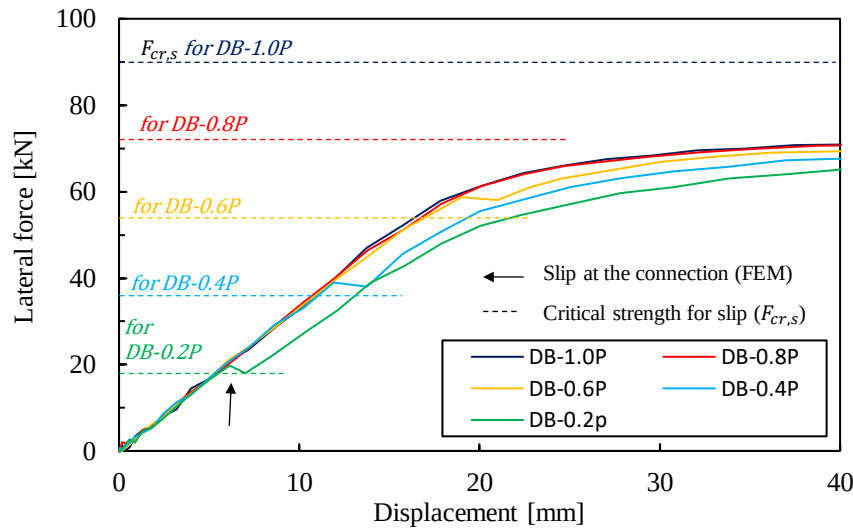


Figure 5.56: Comparison of FE simulation results with the critical lateral force $F_{cr,s}$ for slip in P-DB models

Table 5.8: Comparison of FE simulation results with the critical lateral force $F_{cr,s}$ for slip in P-DB models

FE Model	Preload Ratio	Preload Magnitude [kN]	Analytically expected strength $F_{cr,s}$ [kN]	FEM result $F_{FEA,s}$ [kN]	Ratio $F_{FEA,s} / F_{cr,s}$
DB-1.0P	$1.0P_s$	213	90 (No slip)	No slip	-
DB-0.8P	$0.8P_s$	170	72 (No slip)	No slip	-
DB-0.6P	$0.6P_s$	128	54	59	1.09
DB-0.4P	$0.4P_s$	85	36	39	1.09
DB-0.2P	$0.2P_s$	43	18	20	1.09
DB- NoP	$0 P_g$	0	0	0	0

The friction coefficient (μ) of 0.3 was used for the FE simulations and the critical strength $F_{cr,s}$.

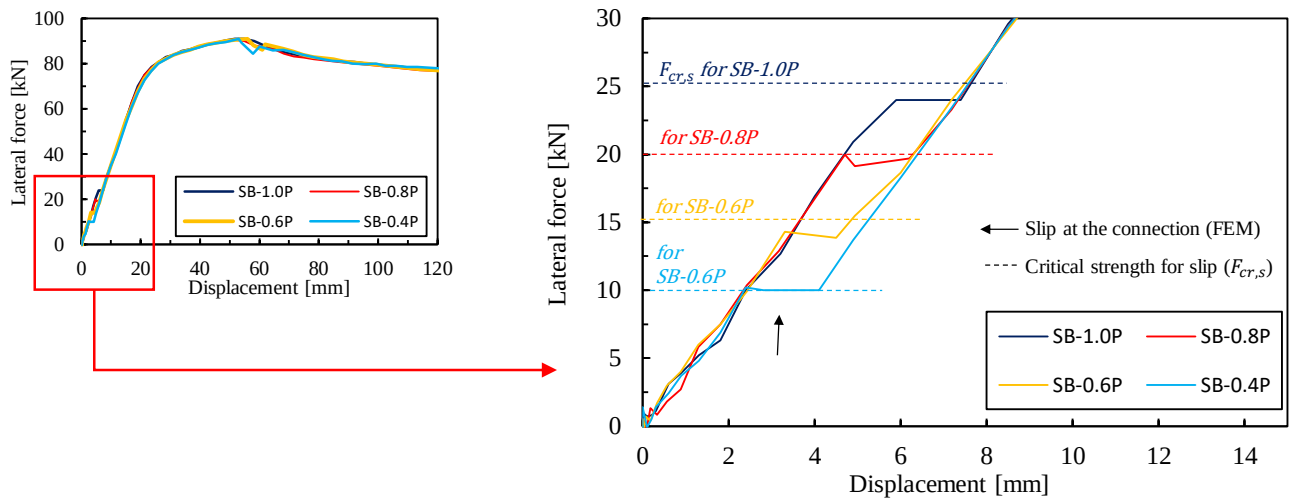


Figure 5.57: Comparison of FE simulation results with the critical lateral force $F_{cr,s}$ for slip in P-SB models

Table 5.9: Comparison of FE simulation results with the critical lateral force $F_{cr,s}$ for slip in P-SB models

FE Model	Ratio	Preload Magnitude [kN]	Analytically expected strength $F_{cr,s}$ [kN]	FEM result $F_{FEA,s}$ [kN]	Ratio $F_{FEA,s} / F_{cr,s}$
SB-1.0P	$1.0P_s$	55	25	24	0.96
SB-0.8P	$0.8P_s$	44	20	20	1.00
SB-0.6P	$0.6P_s$	33	15	14	0.93
SB-0.4P	$0.4P_s$	22	10	11	1.10
SB- NoP	$0 P_s$	0	0	0	-

The friction coefficient (μ) of 0.3 was used for the FE simulations and for calculating the critical strength $F_{cr,s}$.

5.5.2. Influence of the Tolerance of the Coupler Hole

When the AJ connection is used for assembling modules on-site, suitable coupler hole tolerance should be provided to allow positioning of modules during site installation. The tolerance becomes the difference between the diameter of the coupler and the diameter of the coupler hole in plates (see Figure 5.58 (b)). The diameter of the coupler is 42mm (see Figure 5.58 (a)), and the corresponding standard hole is 45mm according to the standard of KBC2019 [101] and KS B ISO 273:2010 [140]. For the beam-to-column joint tests described in Section 4.4, the standard hole size was used for the coupler. In reality, a larger coupler hole tolerance could be required for easier site installation, but as a result, the slip displacement of the connection would increase. In addition to the slip displacement, the tests and FE simulations for the beam-to-column joint showed that after slip occurs at the connection, significant bending stress is generated on the bolt thread of the SRBs by a rotation of the coupler when bearing against the plates (refer to Figure 5.51 (c)-(4)). In the connection design stage, it is difficult to calculate the amount of bending stress on the bolt thread of the SRBs caused by the slip. Therefore, it is necessary to investigate the effect of using a larger coupler hole tolerance on the bending stress of the bolt thread of the SRBs when slip occurs at the connection. Additionally, according to EN 1993-1-8 [85] and KBC2019 standards, the design slip resistance of the preloaded bolt is reduced by the reduction factor (k_s), depending on the size and shape of the hole (i.e. standard hole, oversized hole, and slotted hole). If the bolt hole size is increased, the contact area between the bolt header and the nut is reduced. Consequently, the area onto which the preload can be distributed decreases, thereby increasing the possibility of slip occurrence. In the AJ connection, the preloading mechanism using the coupler and SRB is similar to a preloaded bolt, but the preload is distributed more evenly on the connection through the column endplate (refer to Figure 3.7). FE simulations also confirmed that the slip strength decreases if a larger size coupler hole is used.

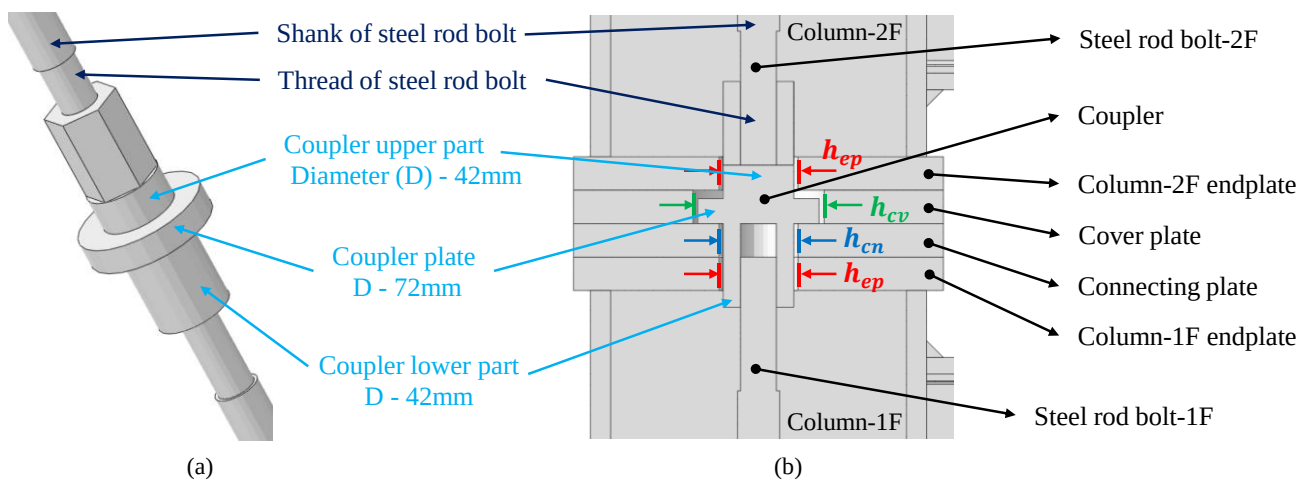


Figure 5.58: Illustration of AJ connection: (a) Coupler and SRBs; and (b) Coupler holes in plates (cross-sectional view)

The P-DB model is used for this parametric study. The coupler hole sizes were determined by referring to the standard of KBS2019 and KS B ISO 273. Table 5.10 shows the details of the FE models (DB-ST, DB-OV1, DB-OV2) and the size of coupler hole tolerance which is the variable parameter. In order to clearly see the stress variation in the SRBs, a preload of 120kN was applied to FE models, implying that both slip and gap-opening would occur. The friction coefficient of 0.3 for surfaces cleaned by wire-brushing with loose rust removed [100] is used for the contact surfaces between the plates at the AJ connection.

Table 5.10: Details of FE models for investigating the influence of coupler hole tolerance

Properties	DB-ST	DB-OV1	DB-OV2
Preload	120 kN (for all)		
Friction coefficient (μ)	0.3		
Type of the hole	Standard hole	Slightly large hole	Oversized hole
Coupler hole tolerance	3mm	5mm	10mm
Column endplate diameter (h_{ep})	45mm	47mm	52mm
Connecting plate diameter (h_{cn})	45mm	47mm	52mm
Cover plate diameter (h_{cv})	78mm (for all)		

Figure 5.59 shows the simulation results for DB-ST, DB-OV1, and DB-OV2. They all exhibit similar joint behaviour (i.e. yield strength, ultimate strength, and failure mechanism), except for the slip displacement at the coupler hole. As expected, the results indicate that the slip displacement increased as the coupler hole size increased, i.e. DB-OV1 had similar behaviour to DB-ST, but DB-OV2 reached the ultimate strength at a noticeably larger displacement than the other models due to the larger slip displacement.

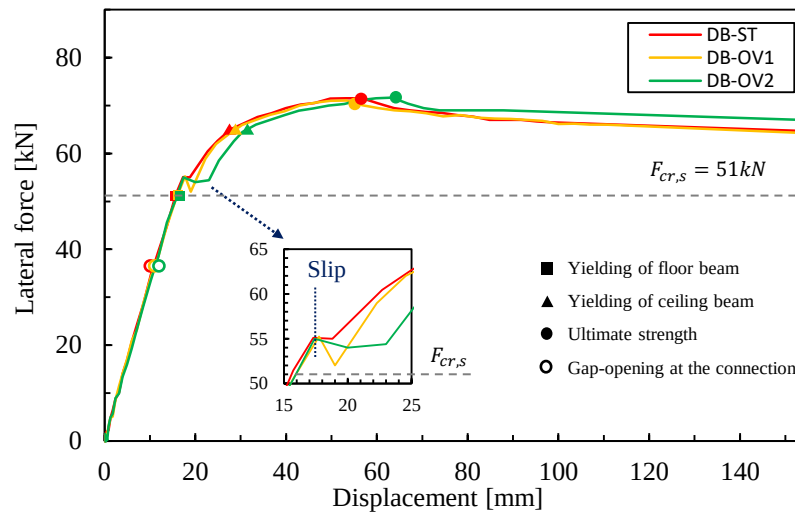


Figure 5.59: Force-displacement curves for DB-ST, DB-OV1, and DB-OV2 under the monotonic loading

Figure 5.60 shows the stress distribution in the coupler and SRBs at the ultimate strength. The location of the maximum stress was at the bolt thread of the SRB in column-2F for all models. The stress variation in the SRBs is presented in Figure 5.61. The bolt thread of the SRB in column-2F had much larger stresses than the thread in column-1F. The stress significantly increased after slip occurred at the connection. As shown in Figure 5.61 (d), DB-ST and DB-OV1 had similar stress variations in the SRBs in column-2F. On the other hand, DB-OV2 had a lower stress variation than the others. The FE simulation results showed that the tolerance of the coupler hole does not negatively affect the bolt thread of SRBs.

Regardless of the size of the coupler hole tolerance, all models developed slip at the same strength, close to the critical strength for slip $F_{cr,s}$. This means that the size of the coupler hole does not affect the slip resistance of the connection. This is because, even if an oversized hole for the coupler hole is used, it represents a relatively small change in area compared to the entire area of the column endplate onto which the preload is evenly distributed.

In summary, the FE simulations confirmed that even if an oversized hole is used for the coupler, it has virtually no effect on the stress of the SRBs compared to the case of using a standard hole. In addition, the slip resistance is not affected by the size of the coupler-hole tolerance. However, when slip occurs at the connection, the slip displacement is large due to the large tolerance of the coupler hole, so it is desirable to prevent slip by applying sufficient preload to the connection.

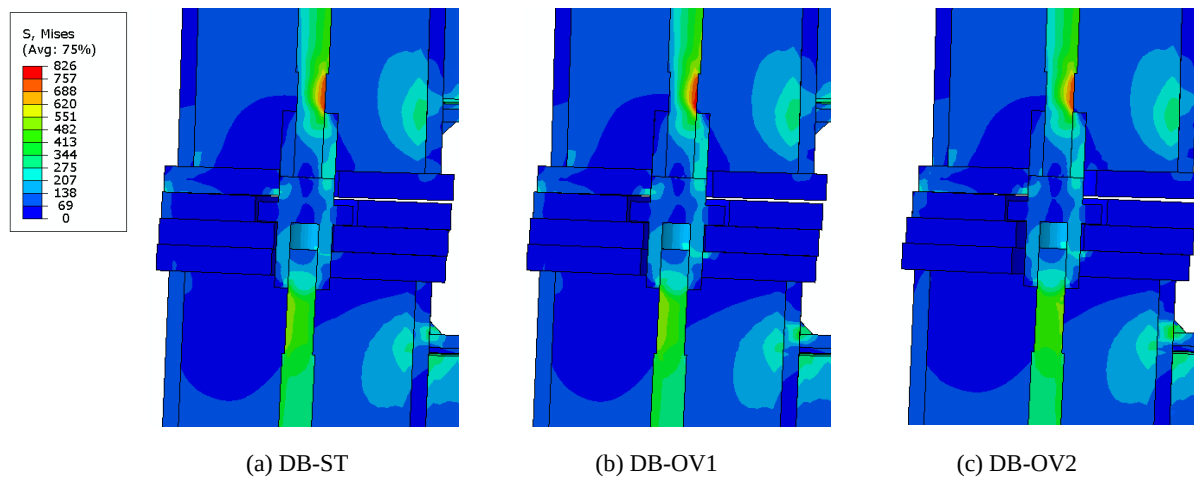


Figure 5.60: Von Mises stress at the ultimate strength in DB-ST, DB-OV1, and DB-OV2

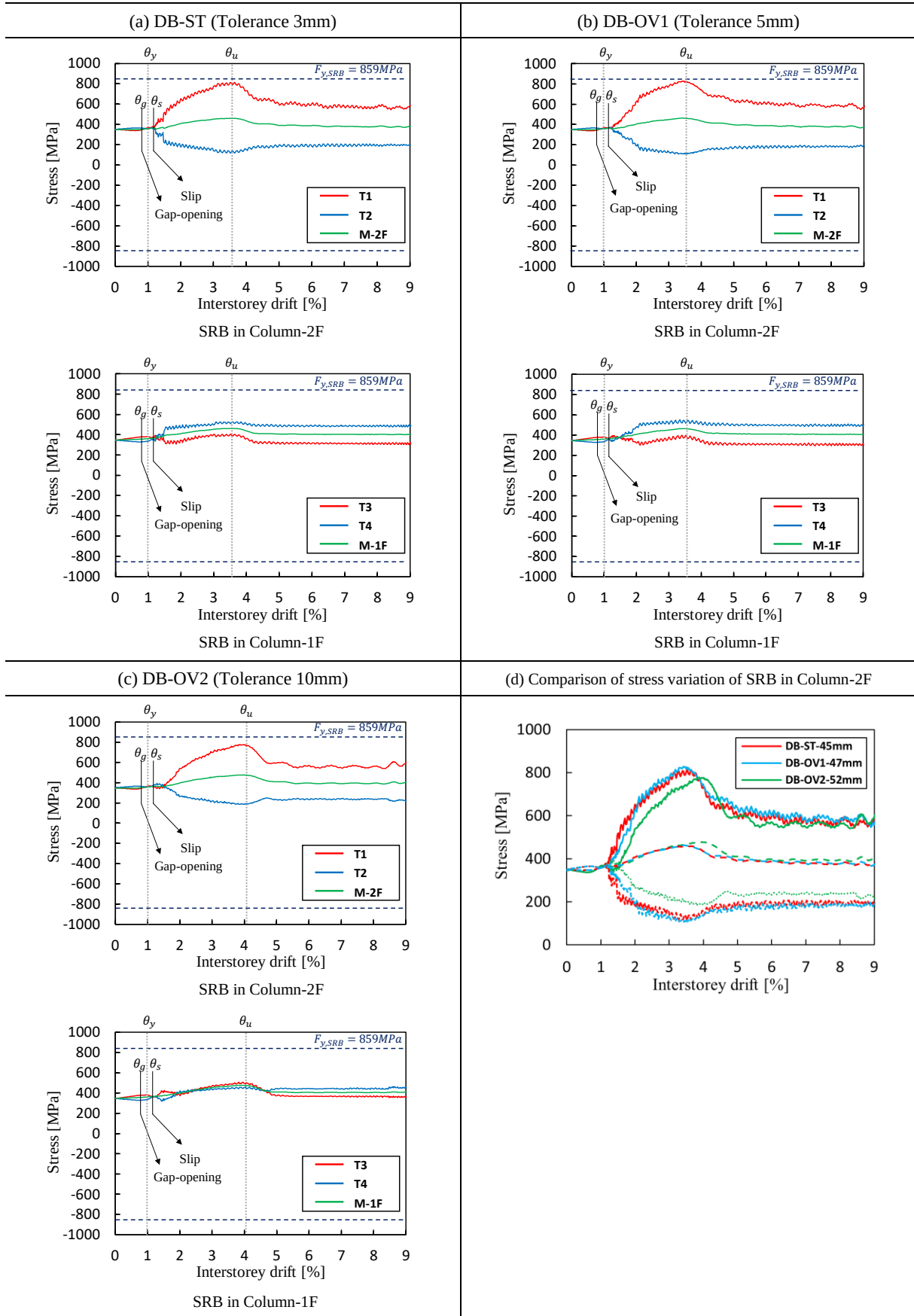


Figure 5.61: Stress variation of SRBs for different coupler hole tolerances (refer to Figure 5.50 for the locations of the measured stresses on the SRBs)

5.5.3. Influence of Axial Forces Acting on the Columns

When a multi-storey modular building is subjected to lateral load, tensile and compressive axial forces are induced in the columns (see Figure 5.62 (3)). The tensile force acting on the column reduces the clamping force in the AJ connection. Conversely, when a compressive force is induced in the column by the lateral load, although gap-opening and slip may not occur, the column is more susceptible to buckling because the induced force increases the pre-existing compressive axial force caused by gravity load and preload. In Section 4.4, the beam-to-column joint tests were performed without axial forces caused by gravity and lateral force. In this section, using the calibrated FE joint models, the more accurate behaviour of the AJ connection was investigated by taking these axial forces into consideration. Figure 5.62 shows the loading process used in the FE joint model. The top row of Figure 5.62 shows the forces induced at the AJ connection in each step. First, the preload is applied to the connection during assembly of the modules, then the connection is subjected to gravity load by the self-weight of modules and lastly, when the frame is subjected to lateral loads such as earthquake load or wind load, the connection is affected by shear forces and axial (tensile and compressive) forces. The bottom row of Figure 5.62 presents the load applied to the FE joint model corresponding to each step shown in the top row of the figure.

In reality, the axial forces in the columns and the shear forces in the beams induced by the lateral load are generated simultaneously. Thus, in the FE models, the axial forces were gradually applied to the column until the joint reached the ultimate strength with the lateral force applied at the tip of the beams as shown in Step 3 of Figure 5.62. The magnitude of the gravity load and associated axial forces are obtained from the exterior joint between the 1st floor and 2nd floor (refer to Figure 4.35) in the 8-storey prototype modular building presented in Section 6.4.1, which was designed (capacity design applied) considering the seismic load in a medium-intensity earthquake area. Table 5.11 shows the magnitude of the gravity loads and axial forces subjected to the columns by an earthquake. Those axial forces are significantly high tensile and compressive forces because they are the forces induced in the column when the floor and ceiling beams fail in an earthquake.

Table 5.11: Magnitudes of axial forces acting on columns in FE simulations and demand preloads for AJ connection

Specimen	Gravity load [kN]	Axial force by earthquake (Tensile or compressive force) [kN]	Demand preload*		Final demand preload (For slip-critical connection) [kN]
			Gap-opening P_g [kN]	Slip P_s [kN]	
P-DB	235	453	431	379	431
P-SB	235	453	272	419	419

* Using the same process in Section 4.4.7, the demand preloads were calculated considering the axial forces. The friction coefficient (μ) of 0.3 was used for P_s

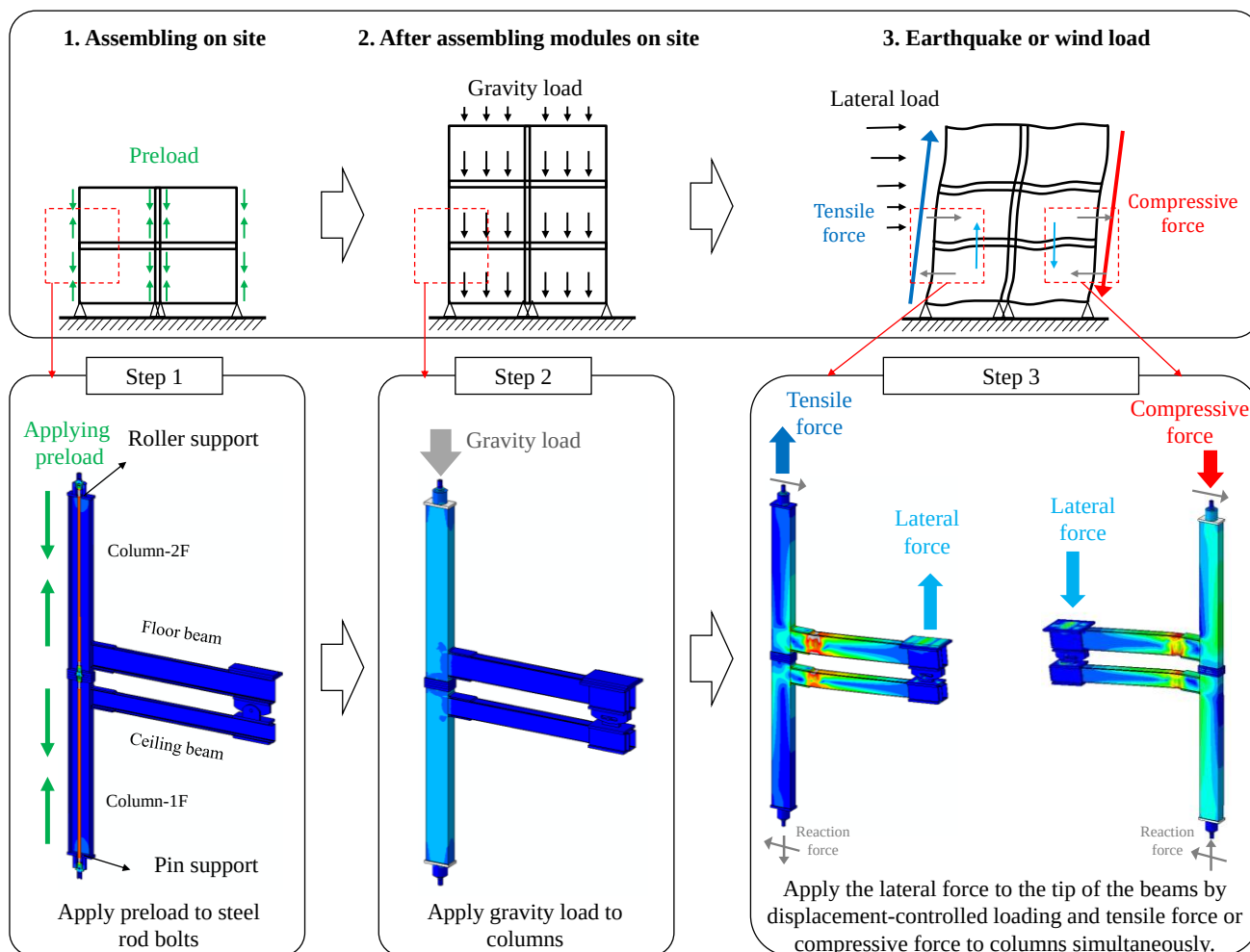


Figure 5.62: Loading process in the FE model of the beam-to-column joint

Suppose the AJ connection is designed as a slip-critical connection. In that case, the demand preloads for P-DB and P-SB are 431kN and 419kN, respectively (see Table 5.11). They exceed the maximum amount of preload which can be applied by an M24 SRB (approx. 200kN). This indicates that if the building is designed according to the capacity design principle for seismic design, the demand preload for the connection can be significantly increased due to the tensile axial force induced in columns by the lateral load. In the first instance, the beam-to-column joint behaviour was investigated using a preload of 200 kN, representative of an M24 SRB. Table 5.12 shows applied axial forces on the columns and preloads for the FE models.

Table 5.12: Applied axial forces on columns and preloads for FE simulation

Models	Gravity load [kN]	Axial force by earthquake [kN]	Applied preload [kN]
P-DB-T	235	453 (Tensile force)	200
P-DB-C	235	453 (Compressive force)	200
P-SB-T	235	453 (Tensile force)	200
P-SB-C	235	453 (Compressive force)	200

Figure 5.63 and Table 5.13 show the moment versus interstorey drift curves obtained from the simulations. In the case of P-DB, there is a large difference in behaviour between P-DB-T and P-DB-C. For P-DB-C, no gap-opening or slip occurred at the connection due to the large compressive force acting on the column. Thus, it showed stable behaviour by forming plastic hinges in the floor and ceiling beams (see Figure 5.64 (2)-(a)). Although the columns are subjected to large compressive forces, they remained below the yield stress ($f_y=355\text{MPa}$) until the floor and ceiling beams failed. The coupler and SRBs in the columns also behaved in the elastic range. Regarding the stress variations in the bolt threads of the SRBs near the connection, the initial stresses induced by the preload were reduced due to the gravity load in both P-DB-T and P-DB-C. This is because the initially stretched length of the SRBs is reduced when the length of the columns shorten as a result of gravity loads (refer to Section 3.4.5). P-DB-C showed that the initial stresses were additionally decreased due to the compressive force induced by the lateral load until the joint reached the ultimate strength (see Figure 5.64 (2)-(b)). This is a temporary reduction due to the elastic axial deformation of the column, so the stress will return to the initial value when the lateral load is removed.

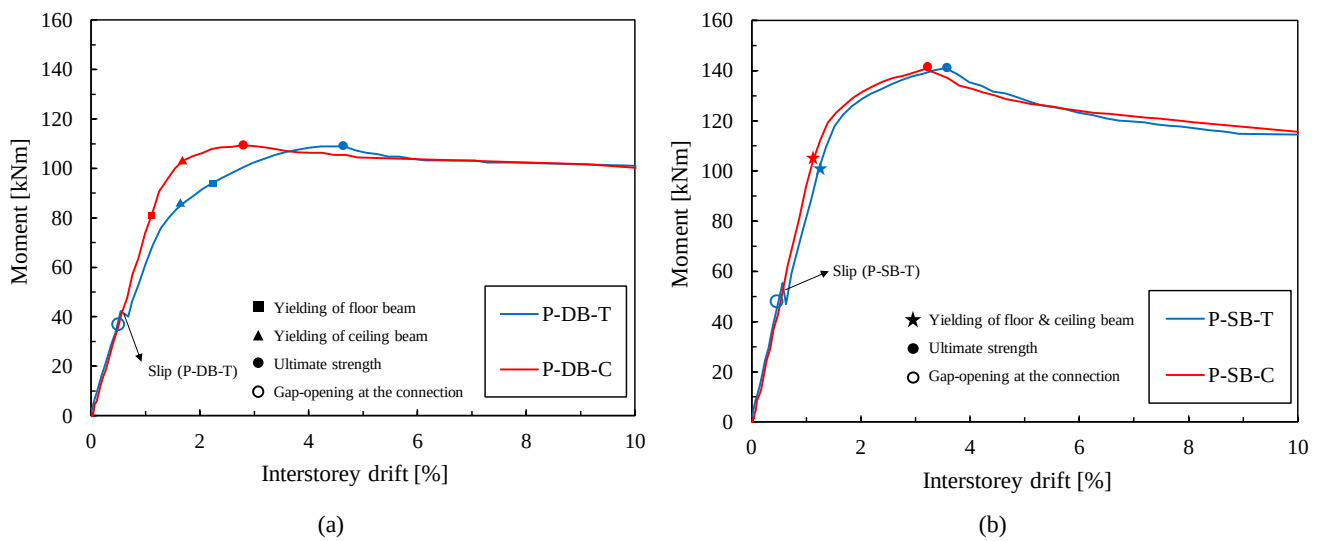


Figure 5.63: Moment-interstorey drift curves: (a) P-DB-T/C and (b) P-SB-T/C

Table 5.13: Summary of the FE simulation results of P-DB-T/C and P-SB-T/C

FEM Model	K_i [kNm/rad]	M_y [kNm]	θ_y [rad]	M_u [kNm]	θ_u [rad]	Slip	Gap-opening	Failure mechanism
P-DB-T	5011	87	1.73	109	4.74	Yes	Yes	Ceiling beam $\rightarrow$ Floor beam SRBs yield
P-DB-C	7276	81	1.12	109	2.78	No	No	Floor beam $\rightarrow$ Ceiling beam
P-SB-T	8142	100	1.22	141	3.55	Yes	Yes	Floor & Ceiling beam simultaneously SRBs yield
P-SB-C	9461	104	1.12	141	3.17	No	No	Floor & Ceiling beam simultaneously

Note: K_i : Initial rotational stiffness; M_y : Yield moment strength; M_u : Ultimate moment strength; θ_y : Yield interstorey drift; θ_u : Ultimate interstorey drift.

On the other hand, P-DB-T showed a different failure mechanism due to the effect of gap-opening at the connection in the early elastic stage as shown in Figure 5.63 (a). Although the floor and ceiling beams failed by forming the plastic hinges as intended, the lateral stiffness of the joint decreased due to the gap-opening and slip at the connection (see Figure 5.64 (1)-(a)). Also, the entire cross-section of the bolt threads of the SRBs in the upper and lower columns yielded before the joint reached the ultimate strength as shown in Figure 5.64 (1)-(b).

In the case of P-SB, although P-SB-T experienced gap-opening and slip in the initial stage due to the tensile force acting on the columns, the gap-opening had little effect on the overall joint behaviour. Except for the slip, the overall joint behaviour was almost same as that of P-SB-C, as shown in Figure 5.63 (b). Regarding the stress variations in the bolt threads of the SRBs (see Figure 5.65 (b)), both P-SB-T and P-SB-C showed that the initial stresses induced by the preload were reduced due to the gravity load. For P-SB-C, the initial stresses were reduced further by the compressive axial force (see Figure 5.65 (2)-(b)), whereas the bolt threads of the SRBs in the upper and lower columns of P-SB-T yielded locally due to bending stress after the slip occurred (see Figure 5.65 (1)-(b)).

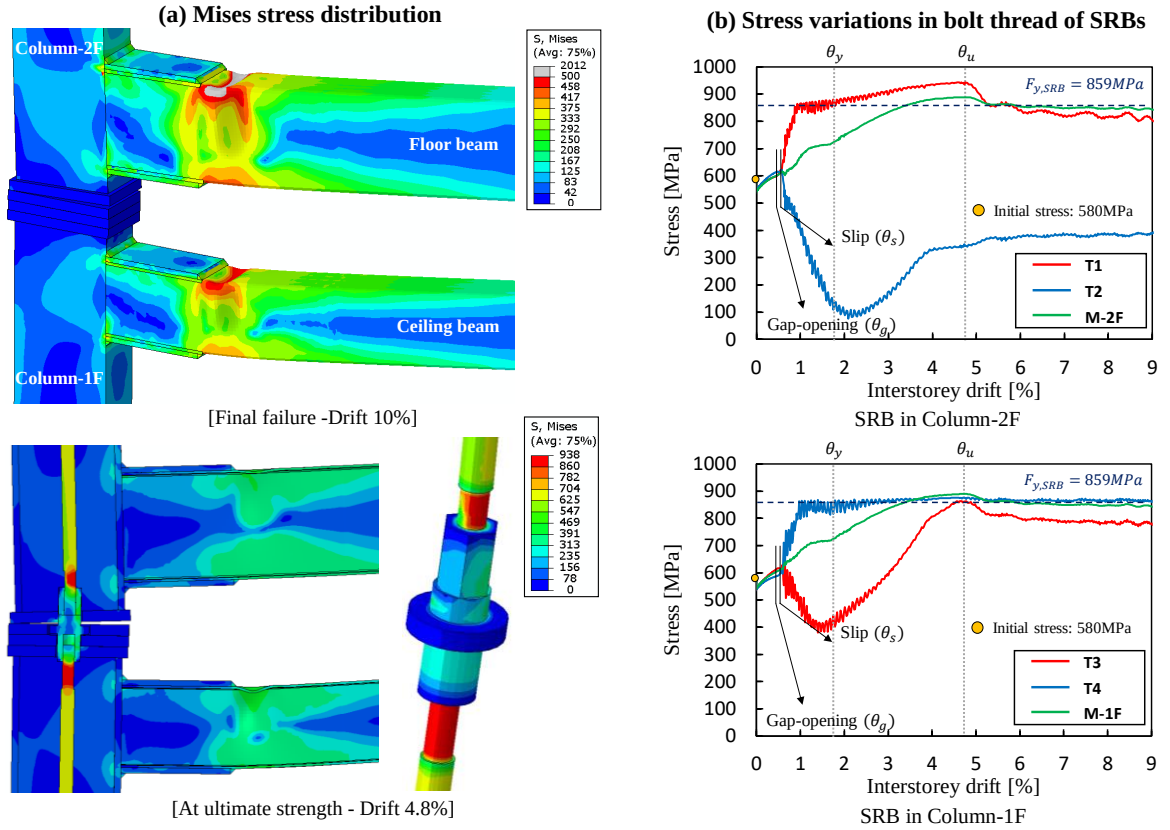
The FE simulation results showed that P-DB and P-SB had stable behaviour under the compressive force acting on the columns, but the column experienced large stresses due to the large compressive forces. Therefore, it is necessary to check whether the column is affected by buckling or yielding due to the compressive force in the design stage. On the other hand, the P-DB and P-SB beam-to-column joints under the tensile force acting on the columns were affected by the gap-opening at the connection from the early stage. The overall joint behaviour of P-DB was governed by the effect of the gap-opening, while P-SB had similar behaviour to the joint under compressive force acting on the columns.

However, when the capacity design principle for the seismic design is applied, the magnitude of the tensile axial force induced in the column by the lateral load is significantly large. It causes unavoidable gap-opening and slip at the connection in the early elastic stage, resulting in yield or fracture of the bolt threads in SRBs. It is expected that yielding or fracture would occur in AJ connections located in the exterior frame and lower floors in the multi-storey modular building. For these AJ connections subjected to such a large tensile axial forces, there are two alternative solutions as follows:

- 1) Using two SRBs (Section 5.5.3.1)
- 2) Using an additional high-tensile bolt (Section 5.5.3.2)

The effectiveness of the two solutions was evaluated through additional FE simulations, as described in the following sections.

(1) P-DB-T



(2) P-DB-C

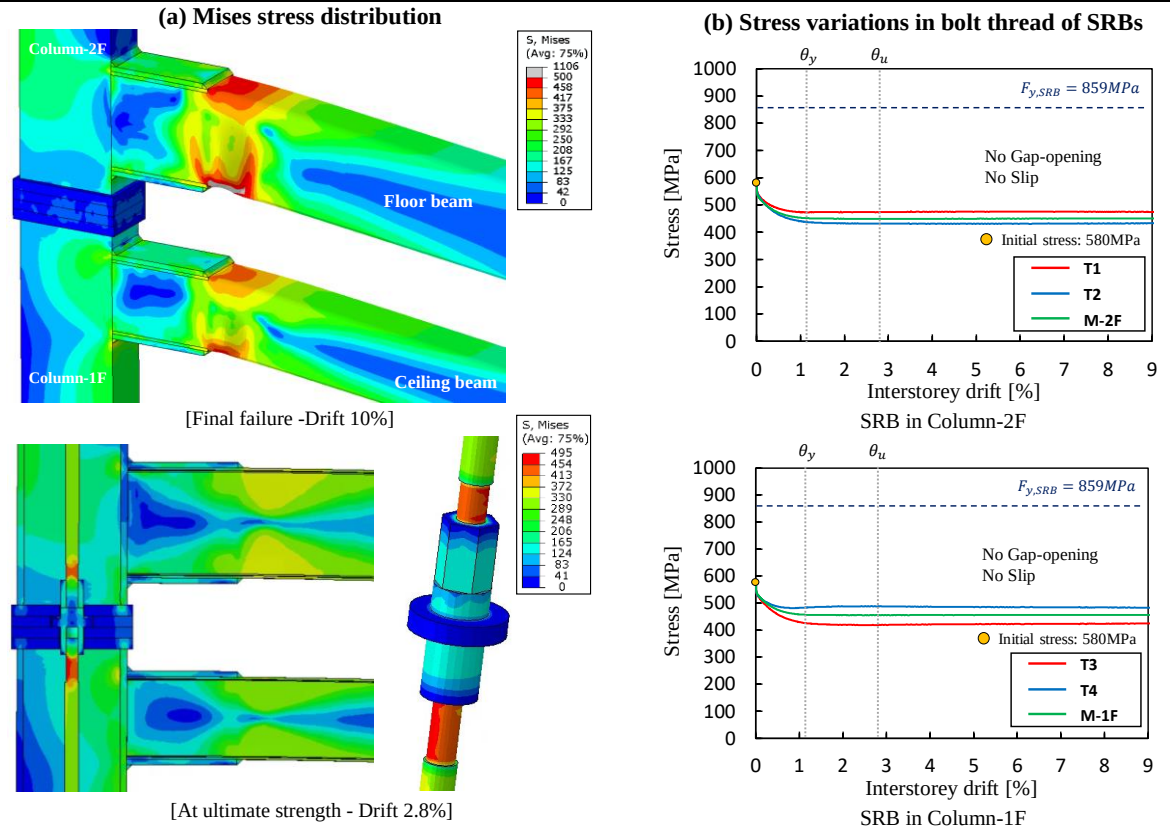
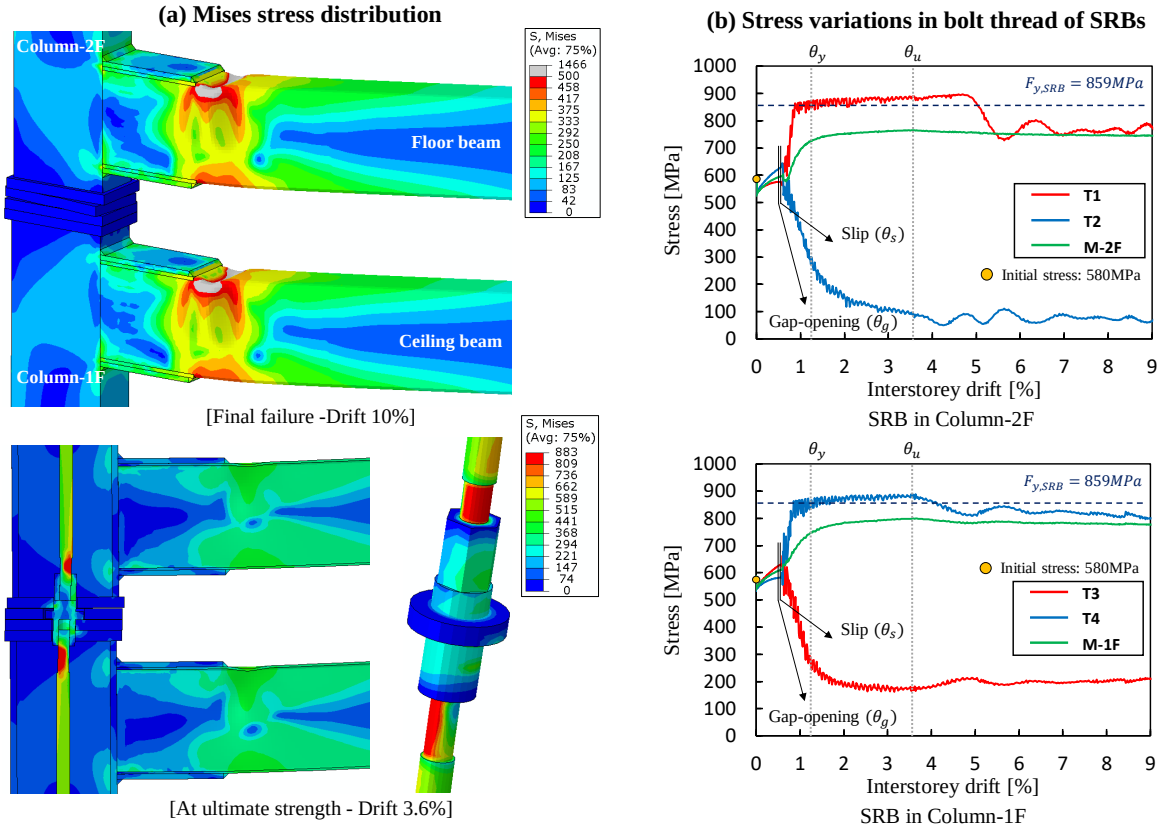


Figure 5.64: FE simulation results of P-DB-T/C: (a) von Mises stress distribution; (b) Stress variations in the SRBs (refer to Figure 5.50 for the locations of the measured stresses on the SRBs)

(1) P-SB-T



(2) P-SB-C

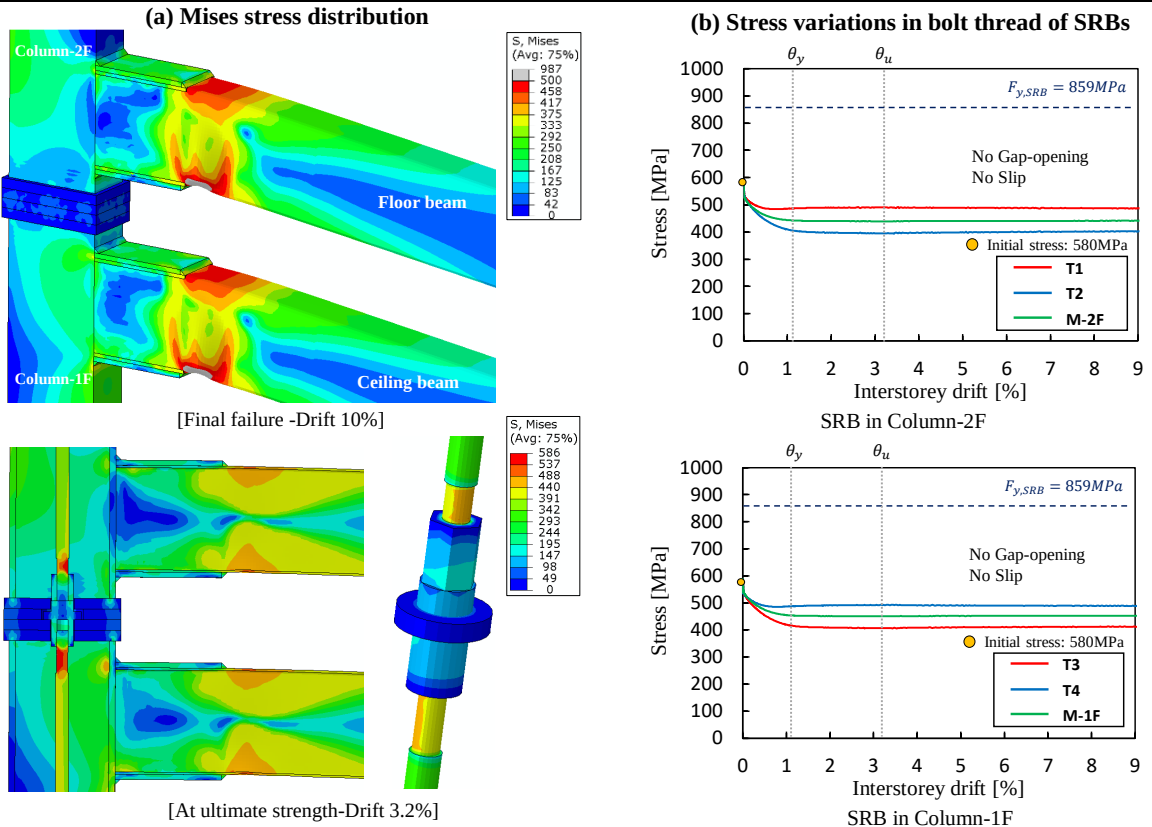


Figure 5.65: FE simulation results of P-SB-T/C: (a) von Mises stress distribution; (b) Stress variations in the SRBs (refer to Figure 5.50 for the locations of the measured stresses on the SRBs)

5.5.3.1. Alternative Option 1: Using Two Steel Rod Bolts in a Column

Figure 5.66 illustrates the P-DB beam-to-column joint with two SRBs inside columns for applying a double amount of preload to the connection. If two SRBs (M24) are used, a preload close to the demand preload for the slip-critical connection can be applied as shown in Table 5.14. However, since the columns are subjected to a doubled compressive force by the preload, the columns become more susceptible to buckling, considering both compressive preload and gravity forces as well as bending moment induced by lateral loads. Therefore, it is noted that the column size may need to be further increased in the design stage. Table 5.14 shows the magnitude of axial forces on columns and applied preloads used in the FE simulations.

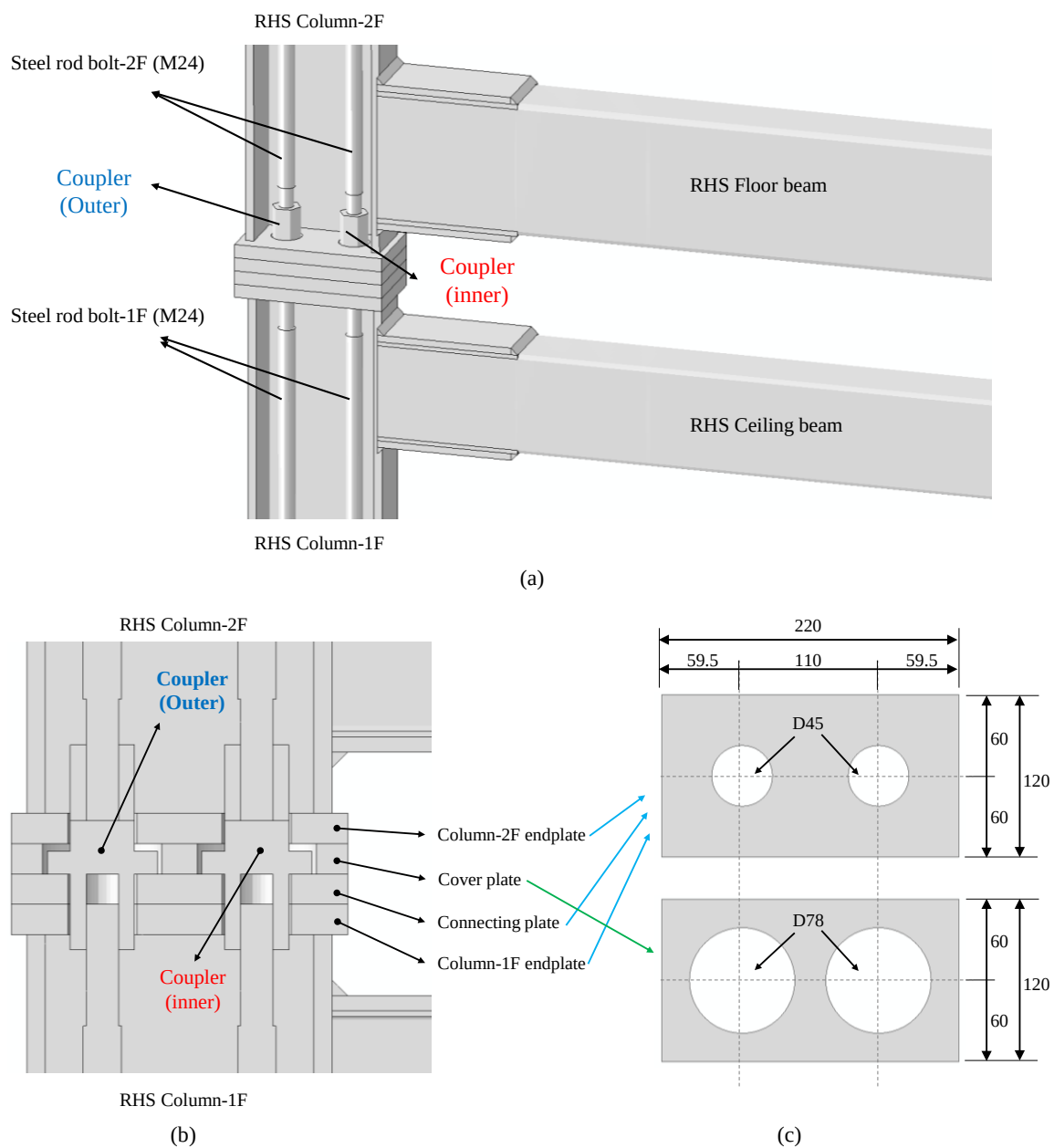


Figure 5.66: AJ connection with two SRBs in P-DB beam-to-column joint: (a) 3D image; (b) Cross-sectional view; and (c) Details of plates

Table 5.14: Applied axial forces on columns and preloads by two SRBs in the P-DB model for FE simulation

Models	Gravity load [kN]	Axial force [kN]	Final demand preload (For slip-critical connection) [kN]	Applied preload [kN]
P-DB-C2-T	235	453 (Tensile force)	419	400 (200kN per SRB)
P-DB-C2-C	235	453 (Compressive force)	419	400 (200kN per SRB)

Figure 5.67 and Table 5.15 present the FE simulation results for models P-DB-C2-T and P-DB-C2-C, which contained two SRBs each. Both had similar elastic and plastic behaviours, except that P-DB-C2-C had very slightly lower stiffness and ultimate strength than P-DB-C2-T. In the case of P-DB-C2-T, gap-opening and slip were prevented at the connection by the preload, and plastic hinges occurred in the floor and ceiling beams. For the columns, the tensile axial force was counteracted by the preload and the gravity load, resulting in a low level of compressive stress in the columns as shown in Figure 5.68 (a). Figure 5.68 (b) presents the stress variations in the bolt threads of SRBs in the upper and lower columns. The initial stresses induced by the preload were reduced due to the compressive deformation of the columns due to the gravity load. Then, when the lateral force and tensile force were applied to the beams and the columns, respectively, the reduced stresses returned to the initial stresses. The stresses in outer SRBs did not vary from the initial stresses induced by the preload, while the stresses in the inner SRBs changed slightly due to the bending action. Overall, the SRBs and the couplers had stable behaviour, as shown in Figure 5.68 (a) and (b).

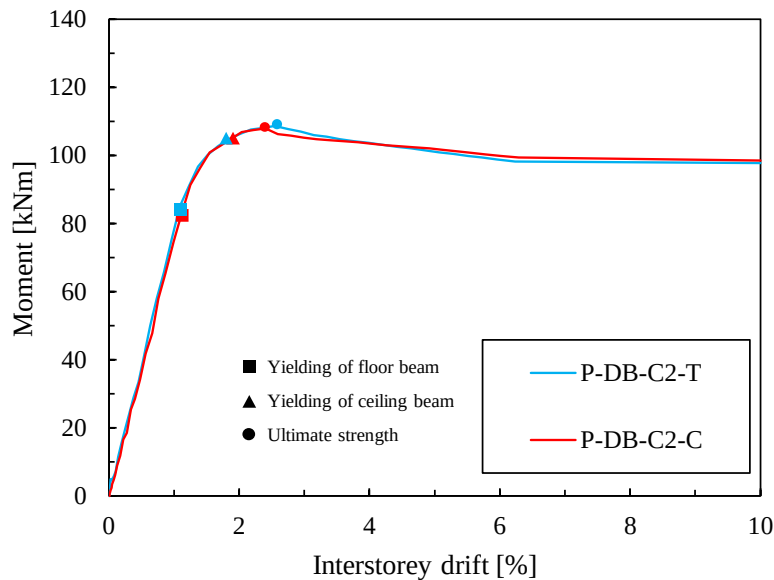


Figure 5.67: Moment-interstorey drift curves of P-DB-C2-T/C

Table 5.15: Summary of the FE simulation results of P-DB-C2-T/C

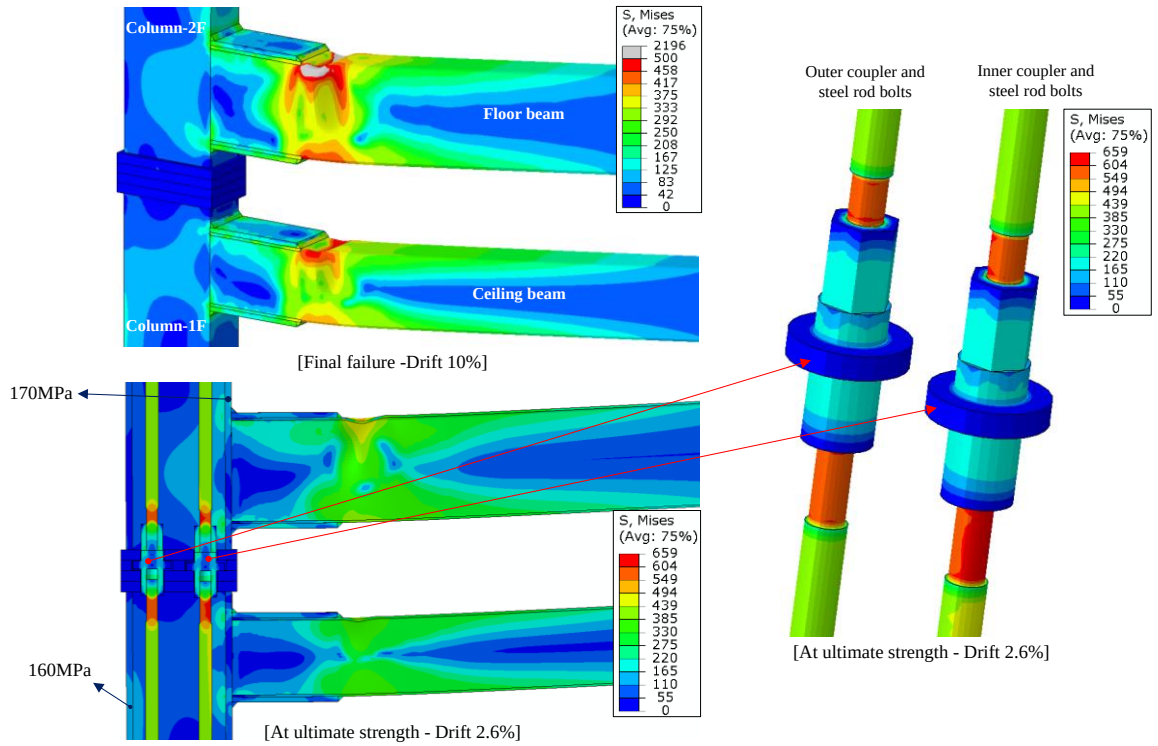
FEM Model	K_i [kNm/rad]	M_y [kNm]	θ_y [rad]	M_u [kNm]	θ_u [rad]	Slip	Gap-opening	Failure mechanism
P-DB-C2-T	7798	85	1.09	109	2.55	No	No	Floor beam → Ceiling beam
P-DB-C2-C	7432	83	1.12	108	2.40	No	No	Floor beam → Ceiling beam

Note: K_i : Initial rotational stiffness; M_y : Yield moment strength; M_u : Ultimate moment strength; θ_y : Yield interstorey drift; θ_u : Ultimate interstorey drift.

P-DB-C2-C had stable plastic behaviour without gap-opening and slip, but the stress in the lower column locally exceeded the nominal yield stress ($f_y=355\text{MPa}$) as shown in Figure 5.69 (a) due to the large compressive force caused by the combination of preload, gravity load, and compressive axial force and bending moment induced by the lateral load. Regarding the stress variations in the bolt threads of the SRBs, the inner and outer SRBs exhibited the same variations as shown in Figure 5.69 (b). The initial stresses were reduced by the compressive deformation of the columns due to the gravity load. They were further reduced due to the compressive axial force acting on the columns until the joint reached the ultimate strength, after which they did not vary.

The FE simulation confirmed that using two SRBs in the column would be a sound alternative for AJ connections subjected to large tensile axial forces. It should be noted that as the amount of the preload increases, a larger and thicker column section may be required.

(a) Mises stress distribution



(b) Stress variations in bolt thread of SRBs

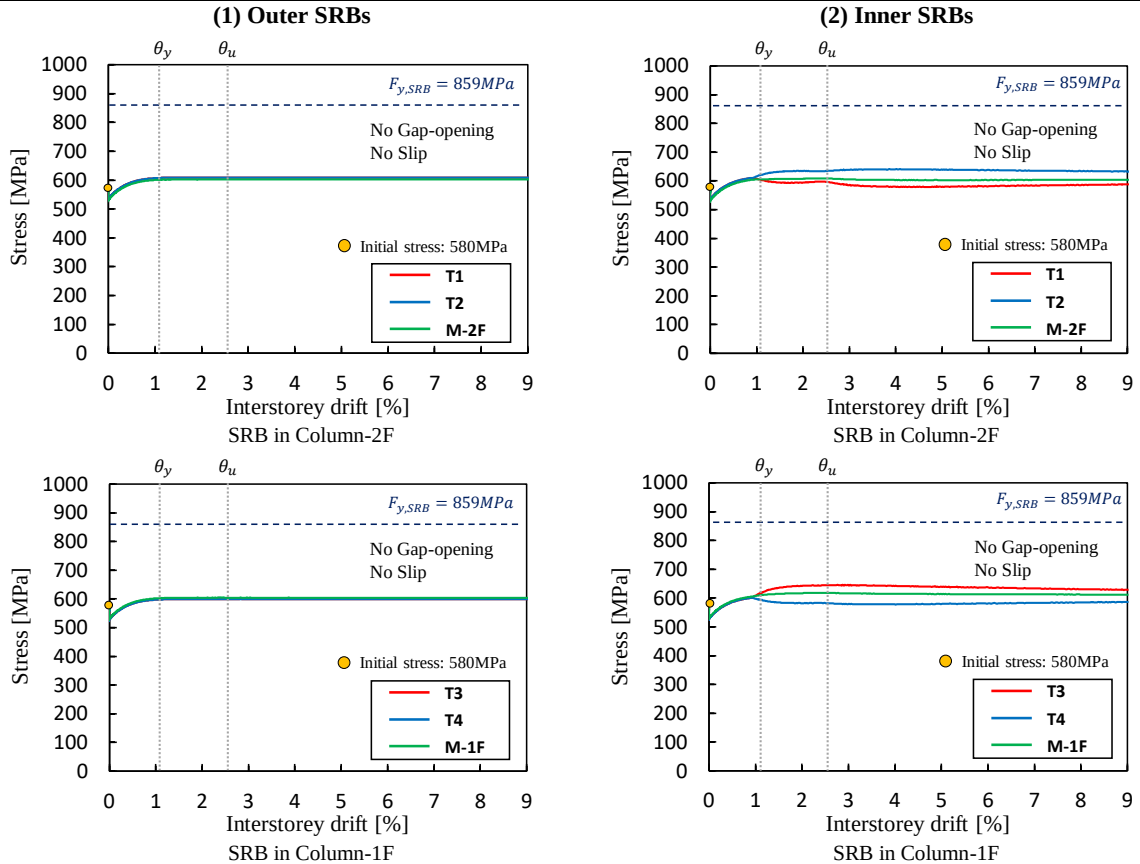
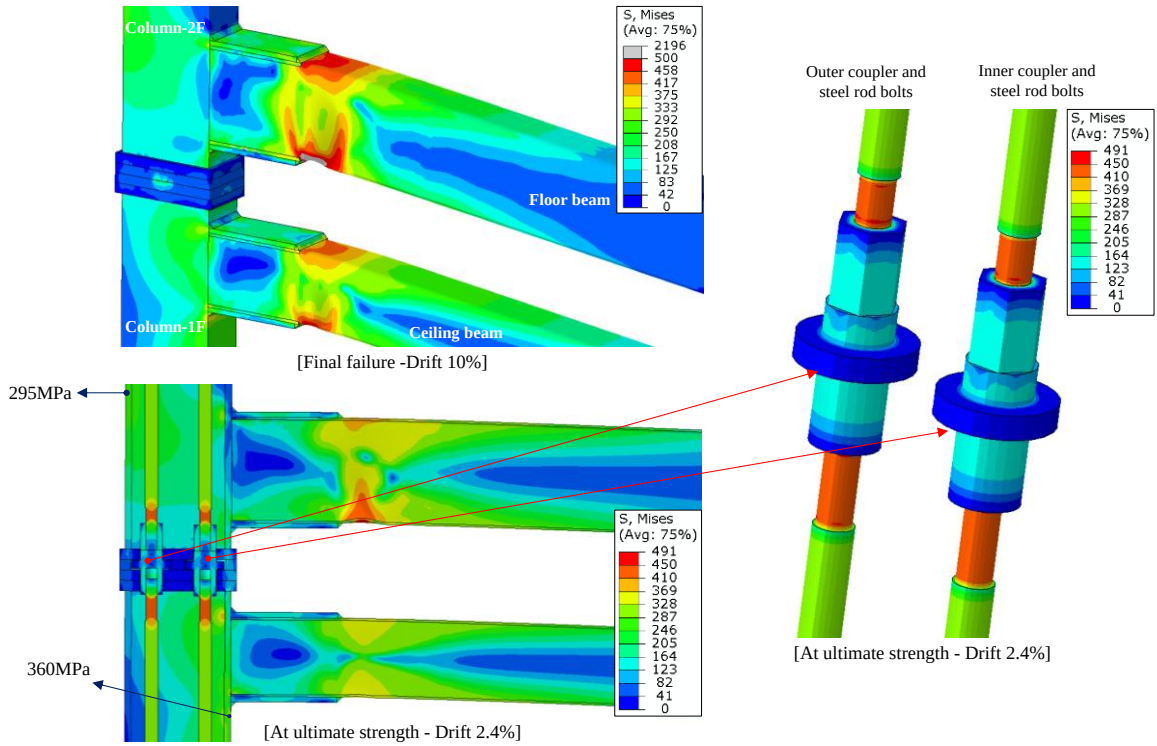


Figure 5.68: FE simulation results of P-DB-C2-T: (a) Mises stress distribution; (b) Stress variations in the SRBs

(a) Mises stress distribution



(b) Stress variations in bolt thread of SRBs

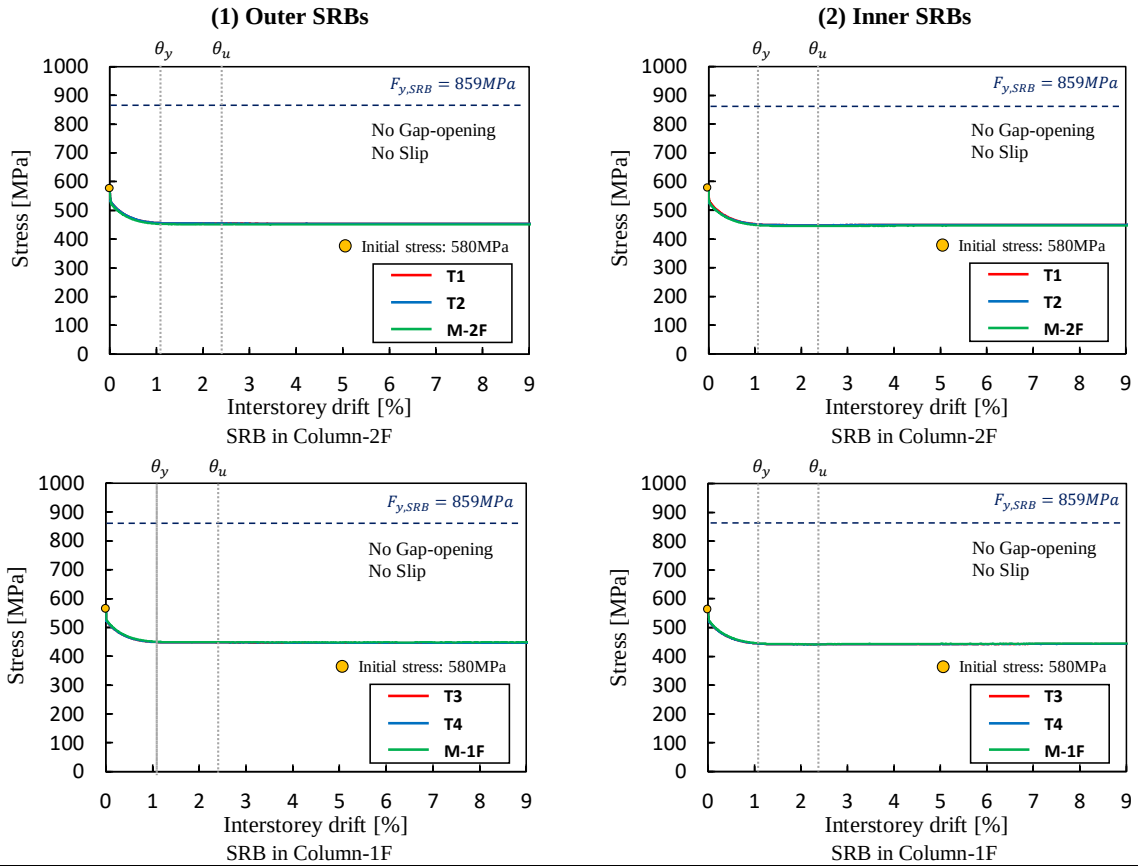


Figure 5.69: FE simulation results of P-DB-C2-C: (a) Mises stress distribution; (b) Stress variations in the SRBs

5.5.3.2. Alternative Option 2: Using an Additional High-tensile Bolt

The second alternative is installing an additional high-tensile bolt at the AJ connection outside the column. The high-strength bolt can be installed on the extended plates as shown in Figure 5.70. The bolt is passed through from the column endplate-2F to the column endplate-1F, and then preload is applied to join the columns. Figure 5.70 illustrates the AJ connection with the high-tensile bolt installed in the P-DB joint for two possible locations of the additional bolt. Since the bolt is installed outside the columns and only through end plates, the columns are not subjected to additional compressive force by preload, unlike the first alternative using two SRBs inside the column. Therefore, even if the demand preload is large, the additional bolt does not cause an increase in the column size. In addition, because it is easier to apply preload to a high-tensile bolt (requiring a relatively low torque (refer to Figure 4.27 (a))), the combined effort of installing preloads in this alternative option is less than for the first alternative option using two SRBs.

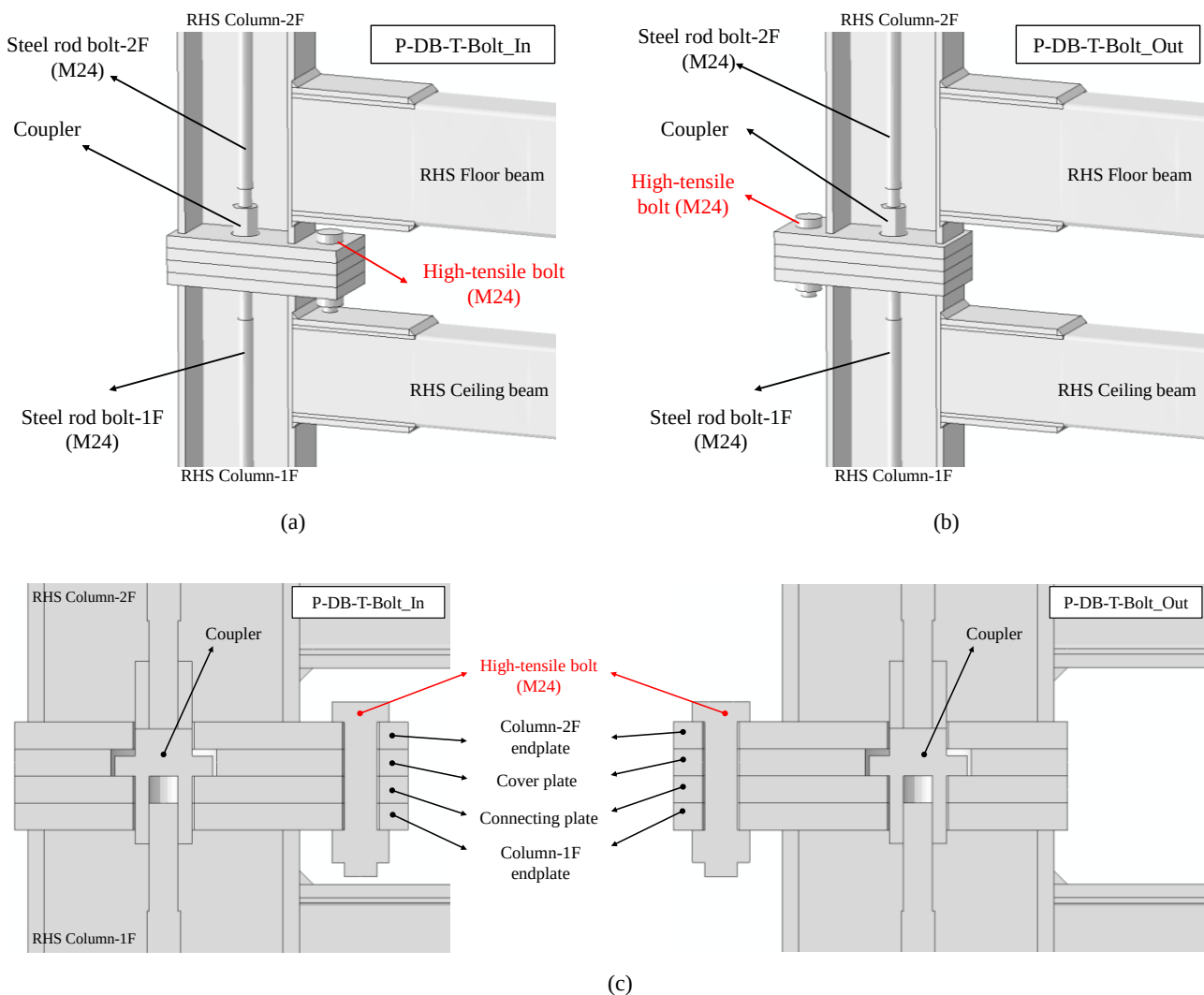


Figure 5.70: AJ connection with the high-tensile bolt in P-DB beam-to-column joint: (a) Bolt installed on the inner side of the connection; (b) Bolt installed on the outer side of the connection; and (c) Cross-sectional view

It is noted that this study focused on the effect of a bolt which is installed near the AJ connection on the joint behaviour. To install the bolt on the inner side of the connection in actuality (see Figure 5.70 (a)), a wider gap between the floor and ceiling beams is required for access space for installation of the bolt.

Table 5.16: Applied axial forces on columns and preloads by the SRB and the high-tensile bolt in the P-DB model for FE simulation

Models	Location of the high-tensile bolt	Gravity load [kN]	Axial force [kN]	Final demand preload (for slip-critical connection) [kN]	Applied preload	
					SRB (M24) [kN]	High-tensile bolt (M24) [kN]
P-DB-T-Bolt_In	Inner side of the connection	235	453 (Tensile)	419	172	247
P-DB-T-Bolt_Out	Outer side of the connection	235	453 (Tensile)	419	172	247

Table 5.16 presents the magnitude of axial forces applied to columns and preloads induced by the SRB and the high-tensile bolt in the FE simulations of the P-DB joint. A high-tensile M24 Grade 10.9 bolt was chosen, to which a preload of 247kN can be applied. Then, a preload of 172kN was applied to the SRBs to meet the demand preload. According to Table 3.2, the location of gap-opening was predicted that the P-DB beam-to-column joint has a gap-opening on the inner side of the connection (refer to Figure 5.45 (1) DB-NoP). In order to investigate the effect of the bolt location on the joint behaviour, FE simulations were performed on two models, P-DB-T-Bolt_In and P-DB-T-Bolt_Out, with bolt locations as shown in Figure 5.70 (a) and Figure 5.70 (b), respectively.

Figure 5.71 and Table 5.17 show the moment versus interstorey drift curves and simulation results. To clearly see the effect of installing the high-tensile bolt, the simulation results of P-DB-T and P-DB-C without the additional bolt (refer to Figure 5.63 (a)) are also shown. The simulation results showed that P-DB-T-Bolt_In and P-DB-T-Bolt_Out had significantly different joint behaviour. In the case of P-DB-T-Bolt_In, where the bolt is installed at the location where the gap-opening is expected, both gap-opening and slip were prevented as shown in Figure 5.72 (a). As a result, the joint exhibited stable plastic behaviour by forming hinges in the floor and ceiling beams, and the coupler and the SRBs remained in the elastic range with no significant changes in stress, as shown in Figure 5.73 (b). The overall joint behaviour of P-DB-T-Bolt_In was almost the same as that of P-DB-C in which the columns were subjected to a compressive axial force. Figure 5.73 (b) and (c) show the stress variations in the high-strength bolt and the SRBs. Due to the compressive deformation of the columns by the gravity load, the initial stresses induced by the preload in both SRBs and bolt were reduced. Then, when the lateral forces and tensile force were applied to the beam and column, respectively, the reduced stresses returned to the initial preload stresses.

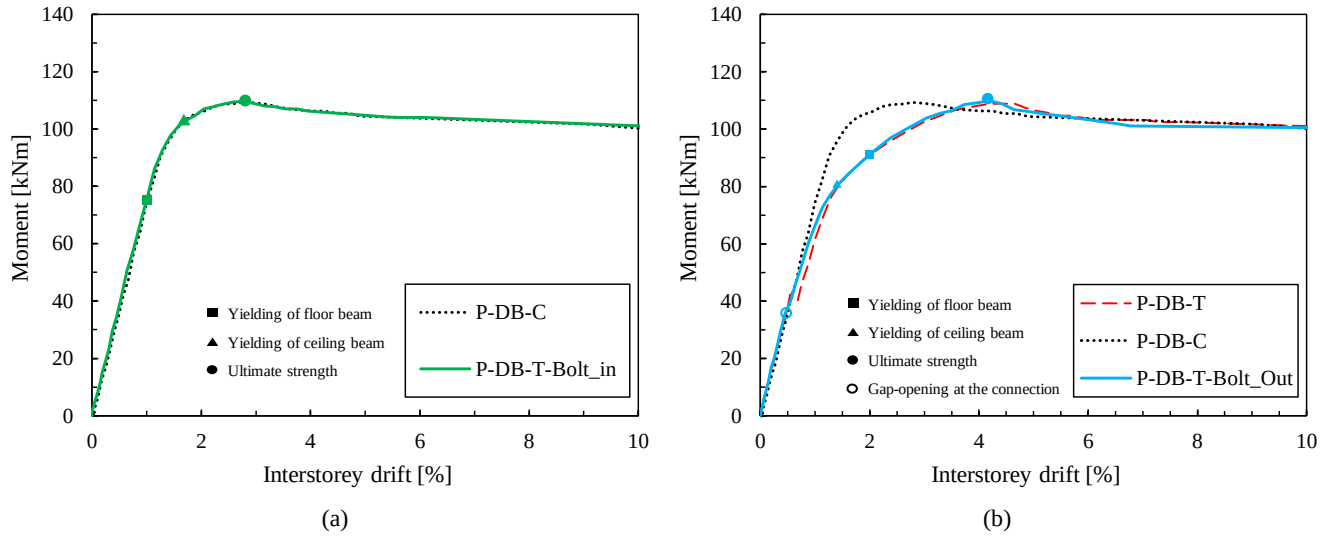


Figure 5.71: Moment-interstorey drift curves: (a) P-DB-T-Bolt_In and (b) P-DB-T-Bolt_Out

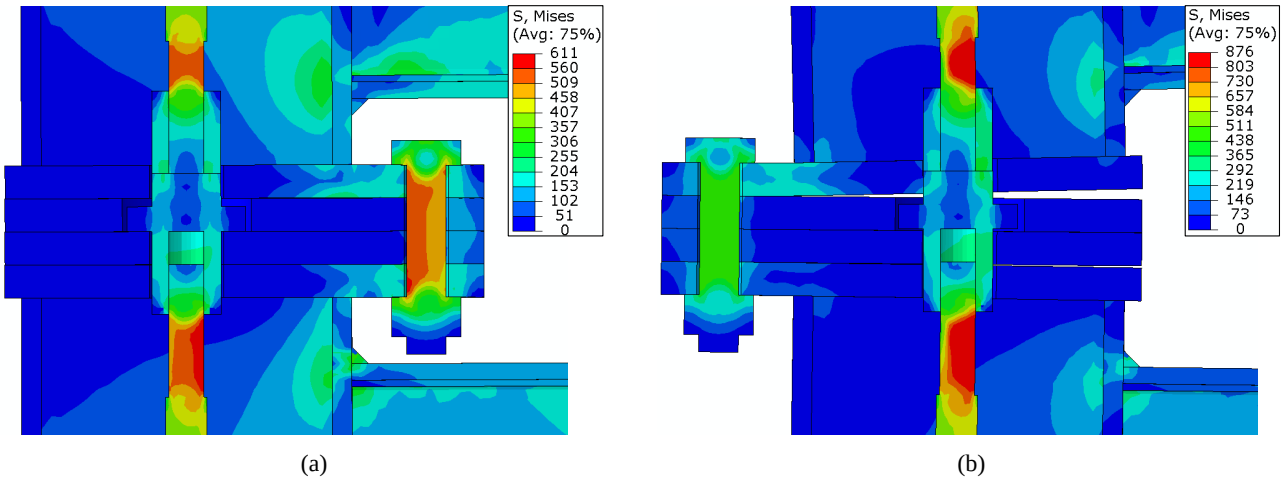


Figure 5.72: Mises stress distribution at the ultimate strength: (a) P-DB-T-Bolt_In and (b) P-DB-T-Bolt_Out

Table 5.17: Summary of the FE simulation results of P-DB-T-Bolt_In/Out

FEM Model	K_i [kNm/rad]	M_y [kNm]	θ_y [rad]	M_u [kNm]	θ_u [rad]	Slip	Gap-opening	Failure mechanism
P-DB-T-Bolt_In	7546	75	1.00	110	2.76	No	No	Floor beam $\rightarrow$ Ceiling beam
P-DB-T-Bolt_Out	5665	81	1.43	110	4.19	No	Yes	Ceiling beam $\rightarrow$ Floor beam
P-DB-C	7276	81	1.12	109	2.78	No	No	Floor beam $\rightarrow$ Ceiling beam
P-DB-T	5011	87	1.73	109	4.74	Yes	Yes	Ceiling beam $\rightarrow$ Floor beam SRBs yield

Note: K_i : Initial rotational stiffness; M_y : Yield moment strength; M_u : Ultimate moment strength; θ_y : Yield interstorey drift; θ_u : Ultimate interstorey drift.

On the other hand, in the case of P-DB-T-Bolt_Out, where the bolt is installed on the opposite side to where the gap-opening is expected, a gap-opening occurred in the initial elastic stage. Consequently, P-DB-T-Bolt_Out displayed similar behaviour to that of P-DB-T as shown in Figure 5.71 (b). The difference between P-DB-T-Bolt_Out and P-DB-T was that the slip at the connection was prevented by sufficient preload provided by the high-tensile bolt. In other words, the high-tensile bolt installed on the outer side of the connection did not contribute to preventing the gap-opening at all (see Figure 5.72 (b)). The stress variations in the high-tensile bolt and the SRBs are shown in Figure 5.74 (b). The bolts thread of SRBs in the upper and lower columns had the same pattern of stress variations. They were subjected to large tensile and bending stresses due to the effect of gap-opening and tensile axial force acting on the columns, resulting in yielding before the joint reached the ultimate strength. The high-tensile bolt showed little stress variation as it was hardly affected by the gap-opening occurring on the opposite side (see Figure 5.72 (b) and Figure 5.74 (c)).

The FE simulation results confirmed that the joint behaviour is significantly different depending on the location of the high-tensile bolt. If the bolt is installed at a location where a gap-opening will not occur, it does not contribute to preventing the gap-opening at all. On the other hand, if the bolt is installed at a location where it can prevent gap-opening, it would be an effective method for improving the performance of AJ connections subjected to large tensile axial forces acting in the columns. Consequently, it is critical to correctly predict the location of gap-opening at a connection. However, since the gap-opening location is determined by the various forces acting on the columns (upper and lower) and beams (floor and ceiling), it is difficult to intuitively predict the location. Therefore, it is recommended to use Table 3.2 to find the gap-opening location at the AJ connection when deciding where to install an additional high-tensile bolt.

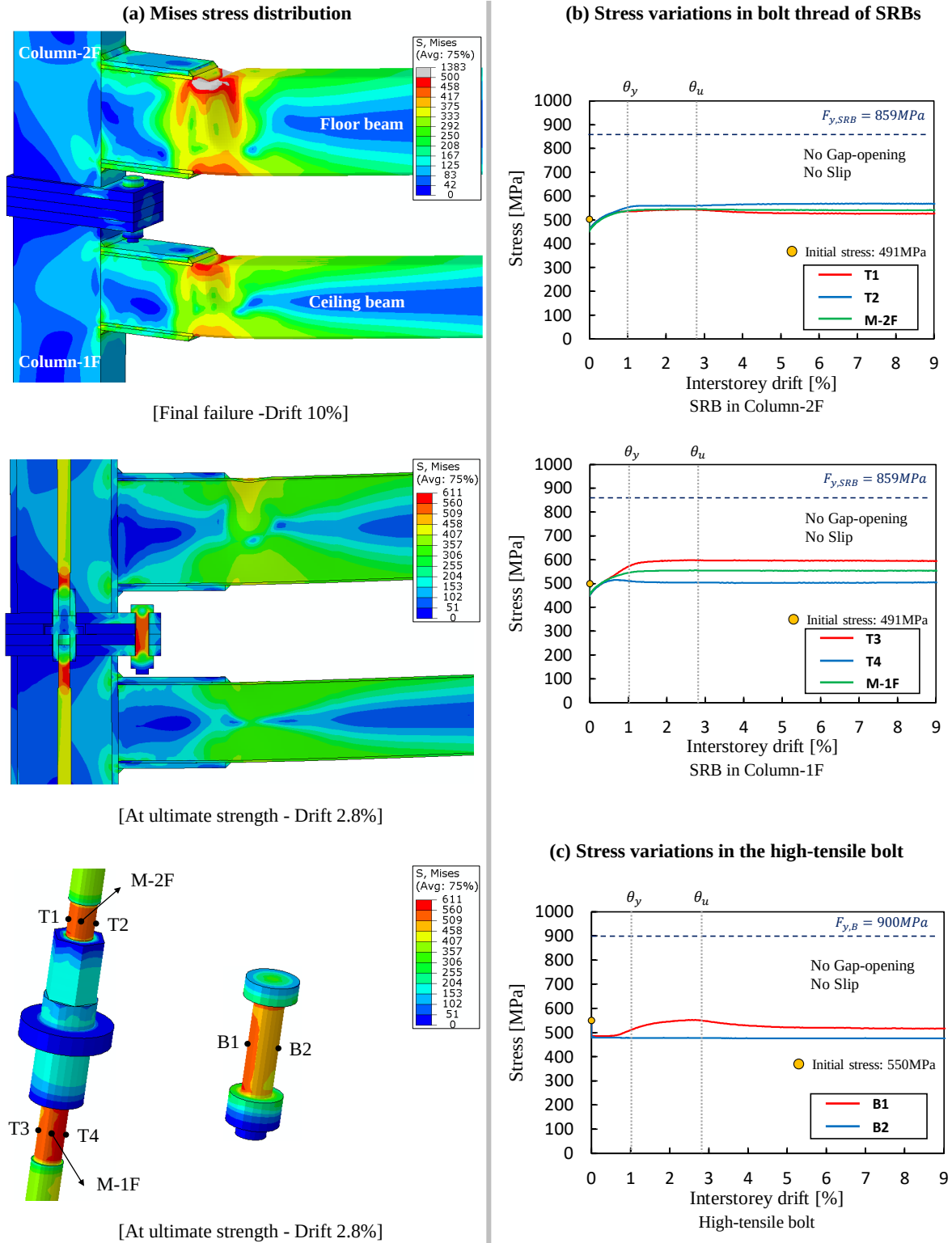


Figure 5.73: FE simulation results of P-DB-T-Bolt_In: (a) Mises stress distribution; (b) and (c) Stress variations in the SRBs and high-tensile bolt, respectively

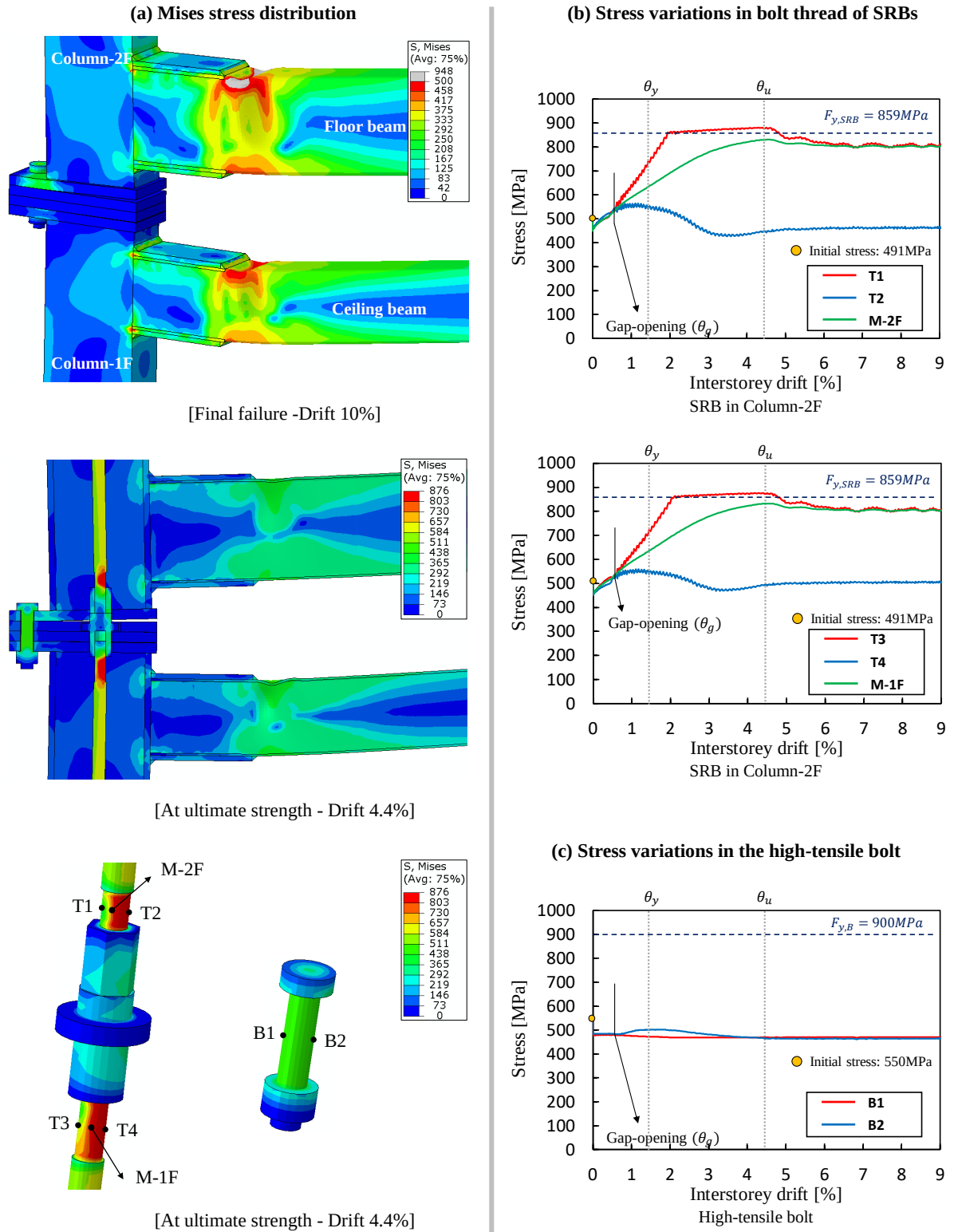


Figure 5.74: FE simulation results of P-DB-T-Bolt_Out: (a) Mises stress distribution; (b) and (c) Stress variations in the SRBs and high-tensile bolt, respectively

5.6. Conclusion

The seismic performance of the beam-to-column joint inclusive of the AJ connection was re-evaluated using the calibrated joint models. In addition, parametric studies were conducted to investigate the influence of various parameters on the behaviour of the joint, including the amount of preload, size of the coupler hole, and axial force in the column. The following main conclusions were drawn from the study:

(1) The AJ joints (P-DB-Q and P-SB-Q) exhibited similar seismic performance to welded joints (W-DB-Q and W-SB-Q) with ductile behaviour and the demand preload P_g preventing gap-opening at the column-to-column connection. The AJ joints showed the intended ductile failure modes with plastic hinges developing in the beams, as required for a strong column-weak beam seismic moment-resisting frame. Based on EN 1993-1-8, the AJ joints are classified as semi-rigid and full-strength joints.

(2) In terms of ductility, the joint with different cross-sectional sizes for the floor and ceiling beams (P-DB-Q) showed better performance than the joint having the same cross-sectional sizes for the two beams (P-SB-Q). Both AJ joints satisfied the requirements for structures of ductility class DCM in accordance with EC8-1, and for intermediate moment frame (IMF) according to AISC 341-16.

(3) The FE simulation confirmed that the design methods (presented in Sections 3.4.3 and 3.4.4) for calculating the demand preload for preventing gap-opening P_g and slip P_s at the column-to-column connection are effective.

(4) In terms of the performance of the SRBs in columns, if the preload prevents gap-opening and slip, the SRB maintains only the initial stress induced by preload until beams form plastic hinges and fail. However, the occurrence of gap-opening at the connection causes the SRBs in the upper and lower columns to be subjected to additional tensile stress by resisting the gap-opening. In addition, if slip occurs at the connection, the threads of the SRBs near the coupler are subjected to bending stress locally caused by a rotation of the coupler when bearing against the plates. This bending stress can be significantly large if slip occurs in the early stage of loading. Also, the FE simulation results of P-SB showed that if slip occurs earlier than the gap-opening, it causes changes in the magnitude of the preload in the SRBs, resulting in the occurrence of gap-opening at a reduced strength. For the particular joint analysed, the reduction was 30%. Therefore, even if the AJ connection is designed as a bearing type connection, it is necessary to minimise the influence of slip by inducing sufficient preload.

(6) The tolerance of the coupler-hole size at the connection did not negatively affect the stress in the bolt thread of the SRBs when slip occurred.

(7) Two alternative solutions (Alt 1: Double coupler and SRBs in the AJ connection / Alt 2: Installing a high-tensile bolt near the AJ connection) were considered for the case where the demand preload for

preventing gap-opening is significantly high (e.g. for AJ connections located at lower storeys and subjected to large tensile axial forces under lateral load). The FE simulations confirmed that both alternative solutions would be effective. In the case of Alt 1, it is noted that a larger column section may be required due to the increase in preload. For Alt 2, it is noted that the high-tensile bolt should be installed at the location where the gap-opening will occur. Otherwise, the bolt has virtually no effect on the gap-opening.

Chapter 6

Frame Analysis with the AJ Connection

6.1. Introduction

The proposed preload column-to-column connection showed almost equivalent performance (i.e. strength, stiffness, ductility, energy dissipation, etc.) to the welded connection at the level of the beam-to-column joint in Section 5.4. This section investigates whether the AJ connection has adequate performance at the frame using 2D FE models.

First, the effect of the inter-module connection stiffness on frame behaviour is investigated, and the minimum required stiffness for individual modules to behave as one building structure are recommended. Then, a 2D simplified FE model of the beam-to-column joint inclusive of the AJ connection, which can describe the essential features (i.e. gap-opening, slip, induced preload, and variation of axial force acting on SRB etc.), is developed and validated by comparing it to the 3D detailed joint model. Lastly, by applying the verified joint model to a moment-resisting frame in an 8-storey mid-rise steel framed modular building, the safety of the AJ connection for the modular building is investigated considering design wind and seismic loads. The seismic performance is assessed using nonlinear static (Pushover) analysis and the N2 method [141].

6.2. Systematic Study of the Stiffness of Inter-Module Connections

A modular building is constructed by connecting a number of modules. In order for the modular building to behave as a compatible structure, the inter-module connection must have sufficient stiffness and strength. The strength of the connections can be secured by designing on demand forces from structural analysis. However, the required stiffness between modules is unknown. Therefore, this section investigates the effect of the inter-module connection stiffness on the behaviour of modular buildings using a 2D FE frame model, and recommends the minimum demand stiffness for inter-module connections.

6.2.1. Introduction of the FE Frame Model

Figure 6.1 shows the 2D FE models used for the numerical simulation. The inter-module connections were expressed simply as shear and axial springs. The vertical inter-module connection for the upper and lower modules was described as a horizontal shear spring ($k_{V,sh}$) and a vertical axial spring ($k_{V,ax}$), while the horizontal inter-module connection between the left and right modules was described as a vertical shear spring ($k_{H,sh}$) and a horizontal axial spring ($k_{H,ax}$). Each spring was treated as a variable in the various width-to-height ratios of modular buildings as shown in Figure 6.2. It was assumed that the same arrangement of the connection was applied to the whole modular building, which meant the stiffness of each spring was changed equally in the models. The simulation was performed by applying a lateral force (150kN) to the top storey, and the top horizontal displacement was used to investigate the effect of each spring stiffness on the behaviour of the modular building. Then, the top displacement of each frame was compared to that of the reference model (Ref.M) which had infinite stiffness for the vertical and horizontal inter-module connections. The minimum demand stiffnesses of each spring (i.e. $k_{V,sh}$, $k_{V,ax}$, $k_{H,sh}$, and $k_{H,ax}$) were defined as the stiffnesses providing differences in top displacement of less than 2%, compared to that of the Ref.M.

6.2.2. FE Simulation Results

(1) Stiffness of vertical inter-module connections (for upper and lower modules)

Figure 6.3 and Table 6.2 show the effect of the horizontal shear spring ($k_{V,sh}$) on the top horizontal displacement of modular buildings. The simulation results show that if the $k_{V,sh}$ is greater than approx. 100kN/mm in the conservative approach, the top displacement difference is less than 2%, compared to the Ref.M. In terms of the vertical axial spring ($k_{V,ax}$) (see Figure 6.4 and Table 6.3), if it is larger than approx. 3000kN/mm, the simulation results show that the top horizontal displacement are similar to the Ref.M. Consequently, the demand minimum shear and axial stiffness of the vertical inter-module connection is 100kN/mm and 3000kN/mm, respectively.

(2) Stiffness of horizontal inter-module connections (for right and left modules)

Figure 6.5 and Table 6.4 show the effect of the vertical shear spring ($k_{H,sh}$) on the top horizontal displacement of modular buildings. If $k_{H,sh}$ is greater than approx. 3000kN/mm, the simulation shows that the top-displacement is almost the same as the one of Ref.M. In terms of the horizontal axial spring ($k_{H,ax}$) (see Figure 6.6 and Table 6.5), if it is larger than approx. 500kN/mm, the simulation results show that the top displacements difference is less than 2%, compared to the Ref.M. Consequently, the demand minimum shear and axial stiffness of the horizontal inter-module connection are 3000kN/mm and 500kN/mm, respectively.

The FE simulation shows that the required stiffness of the inter-module connections for making individual modules behave as one structure (i.e. the Ref.M) is dependent on the width-to-height ratio of the modular building. Adopting a conservative approach, the minimum demand stiffness for the design of inter-module connections, which can cover all investigated width-to-height ratios of modular buildings, is shown in Table 6.1.

Table 6.1: Recommended minimum stiffness for inter-module connections for modular buildings [kN/mm]

Vertical inter-module connection (for upper and lower modules)		Horizontal inter-module connection (for right and left modules)	
Shear stiffness ($k_{V,ax}$)	Axial stiffness ($k_{V,sh}$)	Shear stiffness ($k_{H,sh}$)	Axial stiffness ($k_{H,ax}$)
100	3000	3000	500

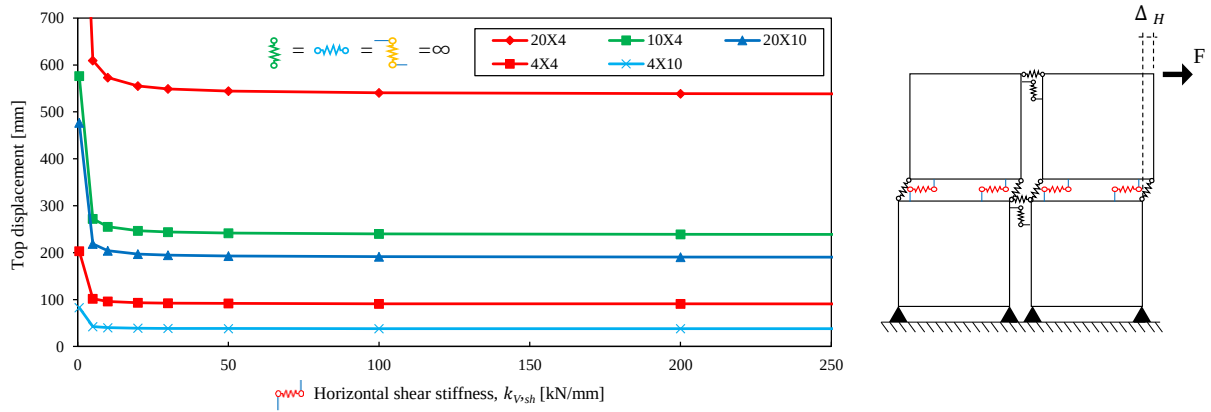


Figure 6.3: Effect of the horizontal shear stiffness ($k_{V,sh}$) on the top displacement for modular buildings

Table 6.2: Top horizontal displacement of modular buildings depending on the horizontal shear stiffness ($k_{V,sh}$) [mm]

Modular building size (Width X Height)	Horizontal shear stiffness ($k_{V,sh}$) [kN/mm]							
	5	10	30	50	100	200	300	∞ (=Ref.M)
4 X 4	102	96	92	92	91	91	91	91
4 X 10	42	40	39	38	38	38	38	38
10 X 4	272	255	244	242	240	239	239	238
20 X 4	610	573	549	544	540	539	538	537
20 X 10	219	204	195	193	191	191	190	190

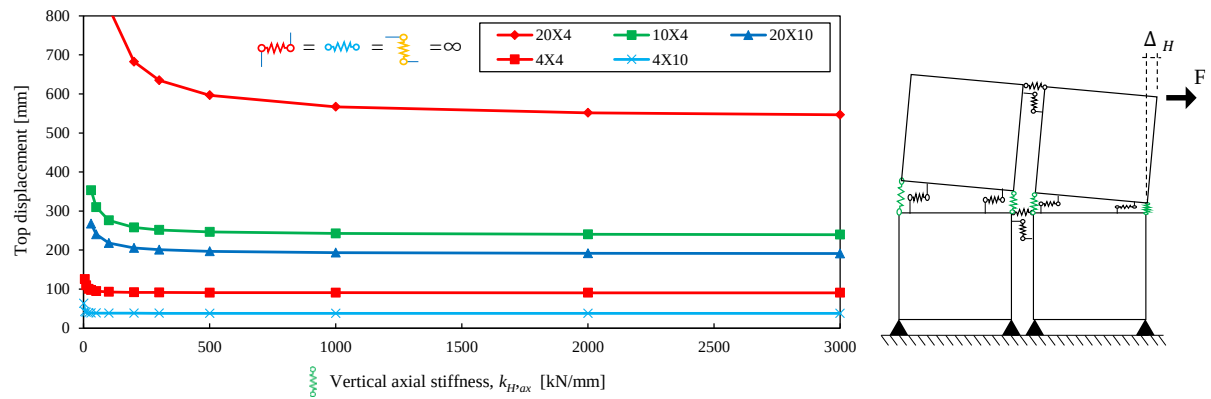


Figure 6.4: Effect of the vertical axial stiffness ($k_{V,ax}$) on the top displacement for modular buildings

Table 6.3: Top horizontal displacement of modular buildings depending on the vertical axial stiffness ($k_{V,ax}$) [mm]

Modular building size (Width X Height)	Vertical axial stiffness ($k_{V,ax}$) [kN/mm]							
	5	10	30	50	100	1000	3000	∞ (=Ref.M)
4 X 4	126	109	98	95	93	91	91	91
4 X 10	42	41	39	39	38	38	38	38
10 X 4	874	563	353	310	276	242	240	238
20 X 4	5936	3248	1454	1093	821	567	547	537
20 X 10	584	396	267	240	218	194	191	190

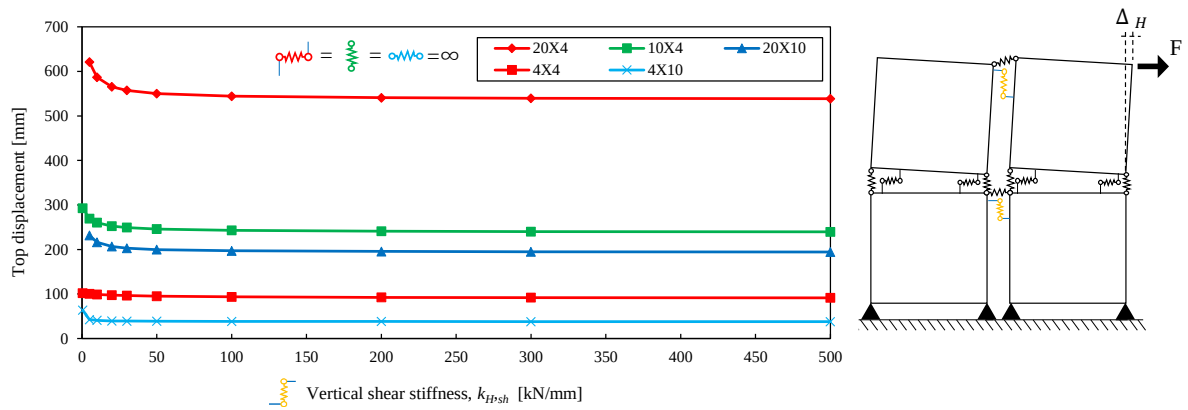


Figure 6.5: Effect of the vertical shear stiffness ($k_{H,sh}$) on the top displacement for modular buildings

Table 6.4: Top horizontal displacement of modular buildings depending on the vertical shear stiffness ($k_{H,sh}$) [mm]

Modular building size (Width X Height)	Vertical shear stiffness $k_{H,sh}$ [kN/mm]								
	0.05	10	30	50	100	300	1000	3000	∞ (=Ref.M)
4 X 4	102	99	96	95	94	92	91	91	91
4 X 10	43	42	41	40	39	39	38	38	38
10 X 4	298	260	249	246	243	240	239	238	238
20 X 4	910	587	557	550	544	540	538	537	537
20 X 10	367	216	203	200	197	195	194	191	190

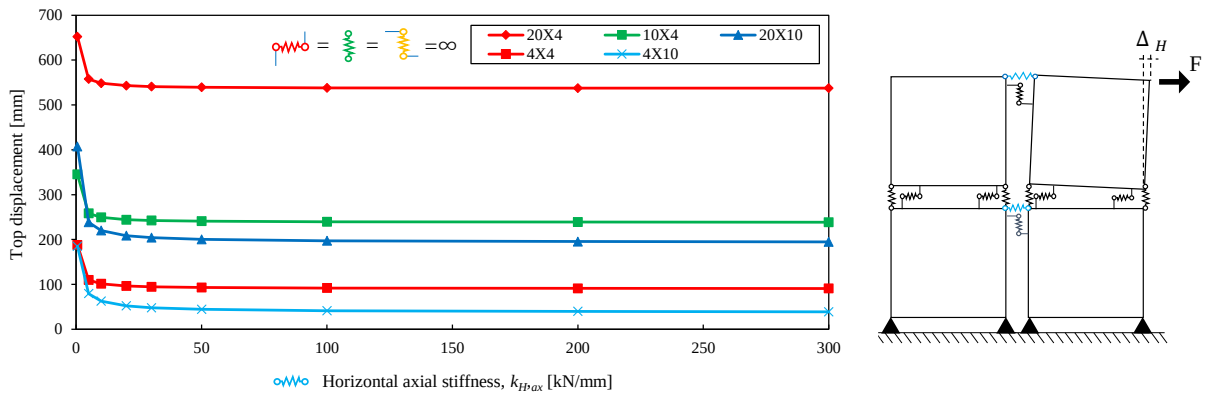


Figure 6.6: Effect of the horizontal shear stiffness ($k_{H,ax}$) on the top displacement for modular buildings

Table 6.5: Top horizontal displacement of modular buildings depending on the horizontal shear stiffness ($k_{H,ax}$) [mm]

Modular building size (Width X Height)	Horizontal axial stiffness $k_{H,ax}$ [kN/mm]								
	0.05	10	30	50	100	200	300	500	∞ (=Ref.M)
4 X 4	321	101	95	93	92	91	91	91	91
4 X 10	321	63	48	44	41	40	39	39	38
10 X 4	627	249	242	241	240	239	239	239	238
20 X 4	1029	548	541	539	538	538	537	537	537
20 X 10	965	220	204	200	197	196	195	192	190

6.2.3. Stiffness of the AJ Connection

(1) Vertical inter-module connection (i.e. column-to-column)

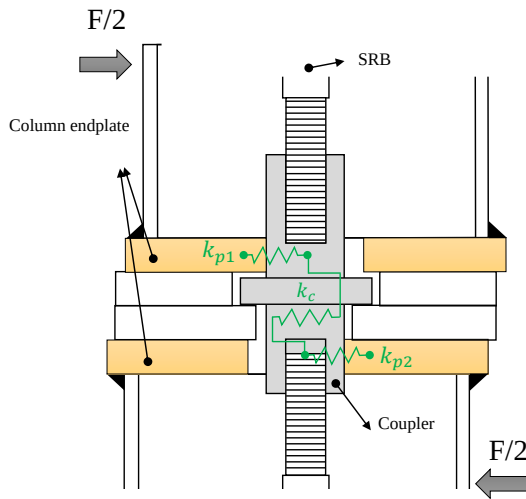
EN1993-1-8 [85] provides the stiffness of a bolt group with two bolts for general steel structures. With reference to this stiffness, the shear and axial stiffness of the AJ connection was estimated. Table 6.6 shows the stiffness per bolt in tension, shear, and bearing.

Table 6.6: Stiffness of one bolt in tension, shear, and bearing

Type	Bolt in tension	Bolt in shear	Bolt in bearing
Preloaded bolt	∞	∞	∞
Non-preloaded bolt	$k_{b,ax} = \frac{EA_s}{L_b}$	$k_b = \frac{8df_{ub}}{d_{M16}}$	$k_p = 12k_bk_tdf_u$

Where E : the Young's modulus; A_s : the tensile stress area of the bolt; L_b : the bolt elongation length, taken as equal to the grip length; d : the bolt diameter; f_{ub} : the ultimate bolt strength, d_{M16} : the nominal diameter of a M16 bolt; f_u : the ultimate strength of the plate material; k_b and k_t depend on the connection detailing (see EN 1993-1-8 [85]).

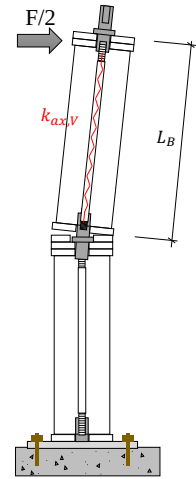
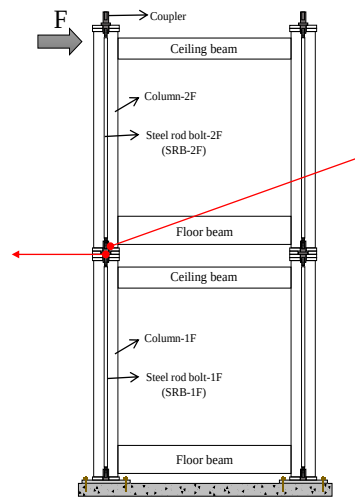
If the AJ connection is designed as a slip-resistant connection, both shear and axial stiffness can be simply assumed as infinite because slip and gap-opening are prevented by the induced preload. On the other hand, if the connection suffers slip and subsequently transfers load by bearing, its shear stiffness ($k_{V,sh}$) can be determined by (Eq. 6.1). Similarly, the axial stiffness ($k_{V,ax}$) can be assumed as infinite before the occurrence of gap-opening, but if the gap-opening occurs, it can be determined by (Eq. 6.2).



$$k_{V,sh} = \frac{1}{k_{p1} + k_c + k_{p2}} \quad (\text{Eq. 6.1})$$

k_c : Shear stiffness of the coupler;
 k_{p1} and k_{p2} are the coupler in bearing stiffness.

(a) Bearing shear stiffness



$$k_{V,ax} = \frac{EA_s}{L_B} \quad (\text{Eq. 6.2})$$

A_s : Tensile stress area of the SRB;
 L_B : Length of column.

(b) Axial stiffness
(after the occurrence of gap-opening)

Figure 6.7: Simplified models for bearing stiffness and axial stiffness of the preload column-to-column connection

(2) Horizontal inter-module connection (i.e. connecting plate)

Figure 6.9 shows the horizontal inter-module connection using a connecting plate. Its shear stiffness ($k_{H,sh}$) between the left and right modules can be found by reference to Figure 6.8 [142] if there is no gap-opening at the column-to-column connection. (Eq. 6.3) shows the relationship between the shear forces acting at the end of the connecting plate and deflection. The shear ($k_{H,sh}$) and axial ($k_{H,ax}$) stiffness of the connecting plate can be determined by (Eq. 6.4) and (Eq. 6.5), respectively.

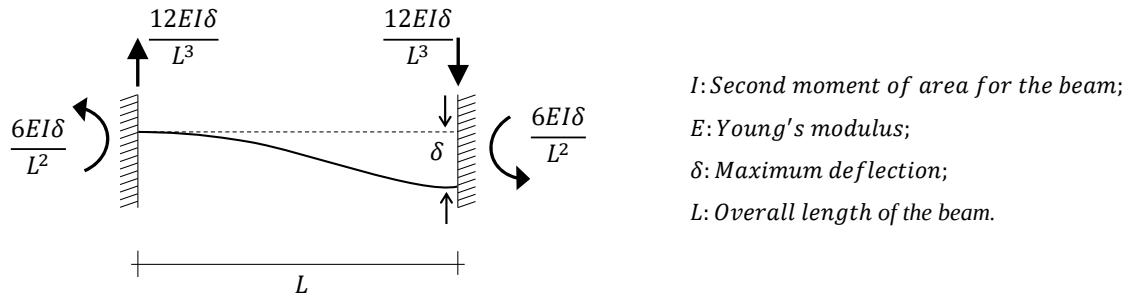


Figure 6.8: Fixed end moments and shear forces for a fixed-fixed beam with deflection [142]

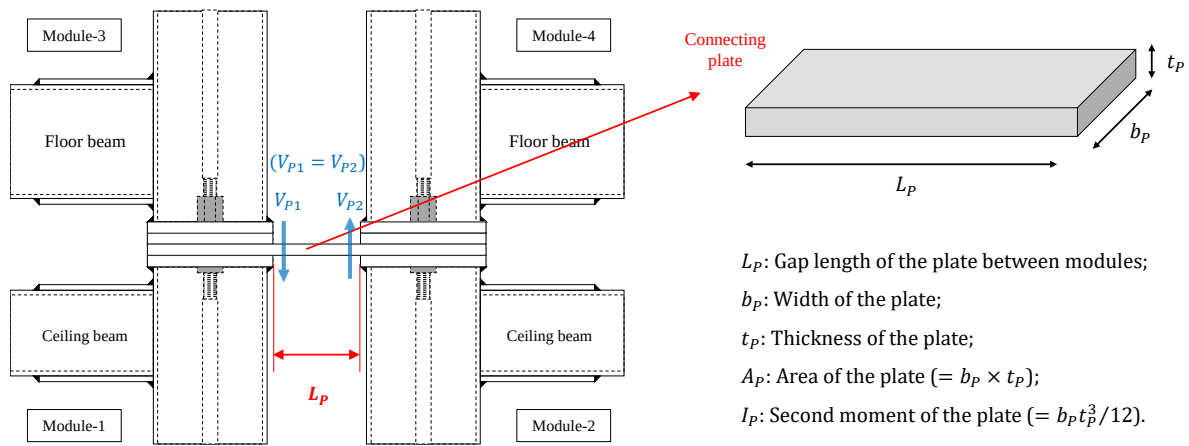


Figure 6.9: Horizontal inter-module connection using the connecting plate in the AJ connection

$$V_{P1} = V_{P2} = \frac{12EI_p \cdot \delta}{L_p^3} \quad (\text{refer to Figure 6.9}) \quad (\text{Eq. 6.3})$$

$$k_{H,sh} = \frac{12EI_p}{L_p^3} \quad (\text{Eq. 6.4})$$

$$k_{H,ax} = \frac{A_p E}{L_p} \quad (\text{Eq. 6.5})$$

Figure 6.10 shows the variation in shear stiffness of connecting plate $k_{H,sh}$ according to its gap-length (L_p) and thickness (t_p). The shear stiffness is strongly affected by the gap-length, and decreases as the gap-length increases.

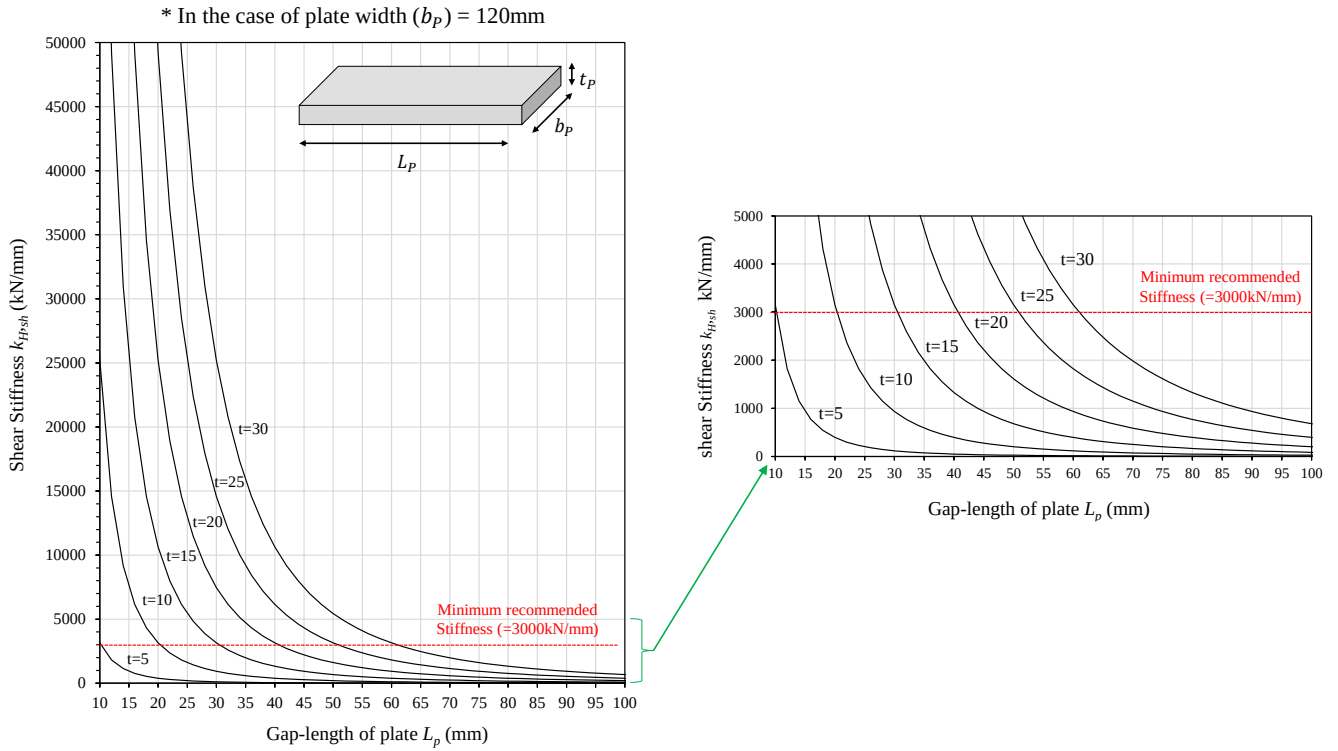
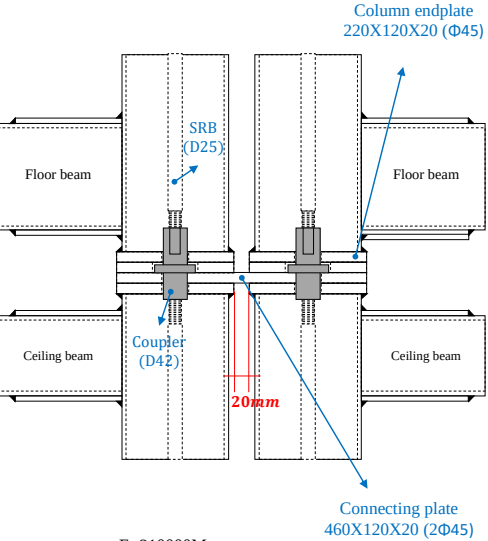


Figure 6.10: Shear stiffness of the connecting plate for horizontal inter-module connection according to the gap-length (L_p)

Table 6.7 presents the stiffness of the AJ connection in the vertical and horizontal directions between modules, which is calculated based on member sizes in test P-DB specimen (refer to Figure 4.36) with the gap-length of 20mm between modules. If the AJ connection is designed as a slip-resistant connection, the minimum demand stiffness will be achieved (see Table 6.1). Even though the connection transfers load in bearing, the bearing stiffness is also over the minimum demand value. If gap-opening occurs, the stiffness of the AJ connection becomes significantly low, and much lower than the minimum required value. Hence, gap-opening should be avoided. (However, it is noted that when gap-opening occurs at the AJ connection due to the lateral force, it occurs only at some and not all connections in the modular building. Therefore, in fact, the overall modular building still would have large stiffness, even though gap-opening occurs at a sub-set of connections.)

Table 6.7: Stiffness of the AJ connection [kN/mm]

Vertical direction (=Column-to-column)		Horizontal direction (=Connecting plate)	
Shear stiffness ($k_{V,ax}$)	Axial stiffness ($k_{V,ax}$)	Shear stiffness ($k_{H,sh}$)	Axial stiffness ($k_{H,ax}$)
$\infty \rightarrow 917^*$	$\infty \rightarrow 34^{**}$	25200	25200
* Bearing stiffness after slip ** Stiffness after gap-opening occurs.			
 - E=210000Mpa - Column endplate: $f_b=410$Mpa - Coupler $f_{ub}=1000$Mpa - Column length $L_b=3000$mm (1) Vertical direction (column-to-column connection) - Axial stiffness, $k_{V,ax}$ $k_{V,ax} = \frac{EA_s}{L_b} = \frac{210000 \times (25\pi^2/4)}{3000} = \mathbf{34 \text{ kN/mm}}$ - Shear stiffness, $k_{V,sh}$ $k_c = \frac{8df_{ub}}{d_{M16}} = \frac{8 \times 42 \times 1000}{16} = 21000 \text{ N/mm}$ $k_{p2} = 12k_b k_t d f_u = 12 \times 1.15 \times 1.88 \times 42 \times 410 = 447117 \text{ N/mm}$ $\frac{1}{k_{V,sh}} = \frac{1}{k_{p1} + k_c + k_{p2}} = \frac{1}{447117 + 21000 + 447117} \rightarrow k_{V,sh} = \mathbf{917 \text{ kN/mm}}$ (2) Horizontal direction (connecting plate) - Axial stiffness, $k_{H,ax}$ $k_{H,sh} = \frac{12EI_p}{L_p^3} = \frac{12 \times 210000 \times (120 \times 20^3/12)}{20^3} = \mathbf{25200 \text{ kN/mm}}$ - Shear stiffness, $k_{H,sh}$ $k_{H,ax} = \frac{A_p E}{L_p} = \frac{(120 \times 20) \times 210000}{20} = \mathbf{25200 \text{ kN/mm}}$			

6.2.4. Demand Preload and Shear Stiffness of the Connecting Plate

The demand preload for the preload connection is significantly affected by axial forces (i.e. tensile force and compressive force) acting on columns. The axial forces acting on columns can be estimated using the axial reaction forces of a modular building. According to the FE simulation, when the modular building is subjected to lateral load, axial reaction forces (e.g. magnitude and location of tensile and compressive forces) change depending on the shear stiffness ($k_{H,sh}$) of the horizontal inter-module connection, as shown in Figure 6.11. Even if the connections have the same stiffness, the distribution of reaction forces differs depending on the width-to-height ratio of the building. Information about the distribution of axial reaction forces is important for finding the location of the vulnerable connections (those subjected to the largest tensile forces) in designing for gap-opening and slip. It also allows for an efficient preload connection design by informing where preload is unnecessary, or only a small amount is needed.

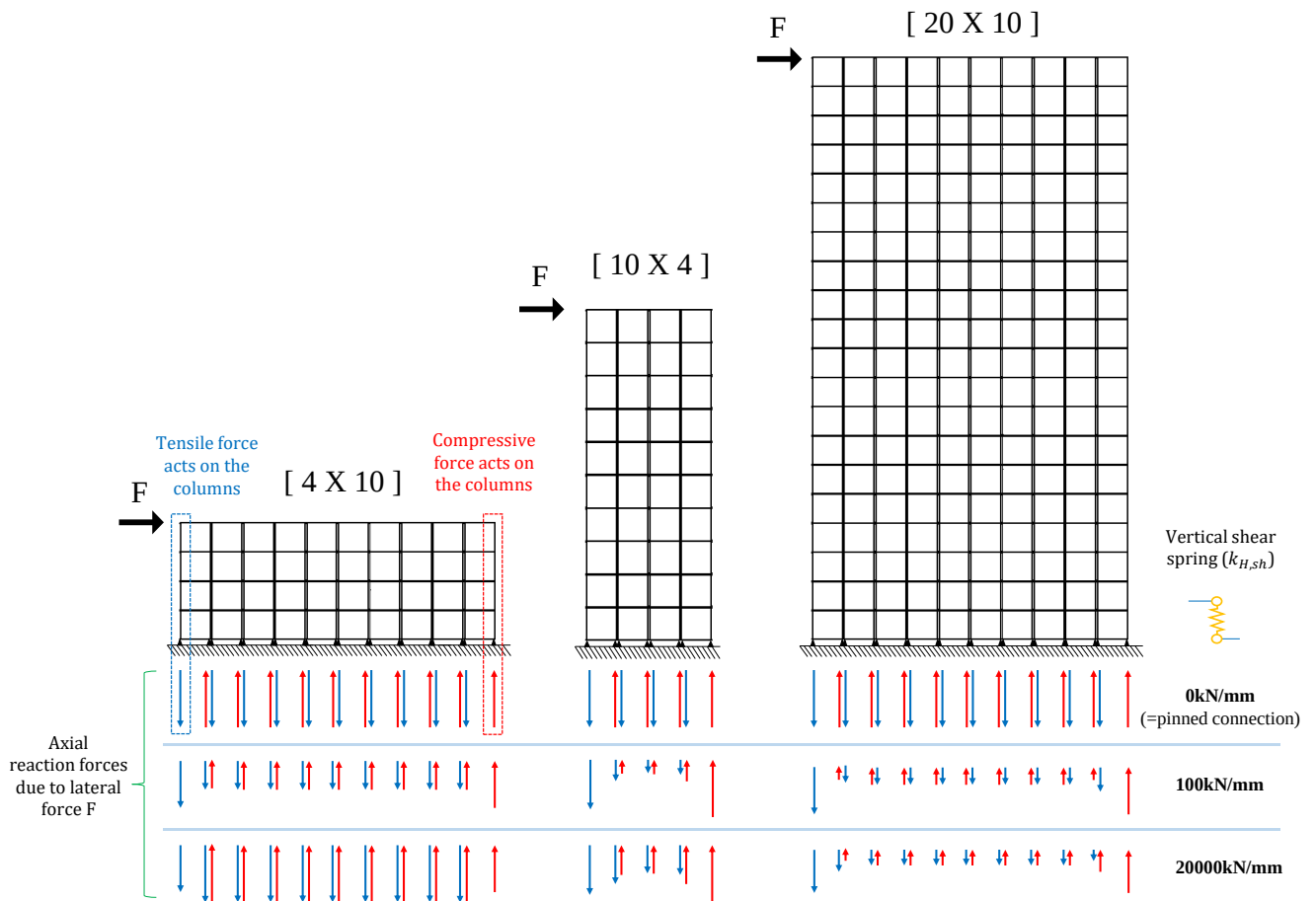


Figure 6.11: Variation of axial reaction forces depending on the width-to-height ratio and the shear stiffness ($k_{H,sh}$) of the horizontal inter-module connection

6.3. Simplified 2D Model of the AJ Connection

6.3.1. Development of the AJ Joint Model

Figure 6.12 shows the 2D simplified FE model of the AJ beam-to-column joint. The model was developed using Abaqus software [112]. The important features for the behaviour of the AJ joint were gap-opening, slip, variation of induced preload at SRBs and plastic hinges of floor and ceiling beams. They were described by spring elements (CONNECTOR in Abaqus). Figure 6.13 shows details of the spring elements used for the AJ joint model.

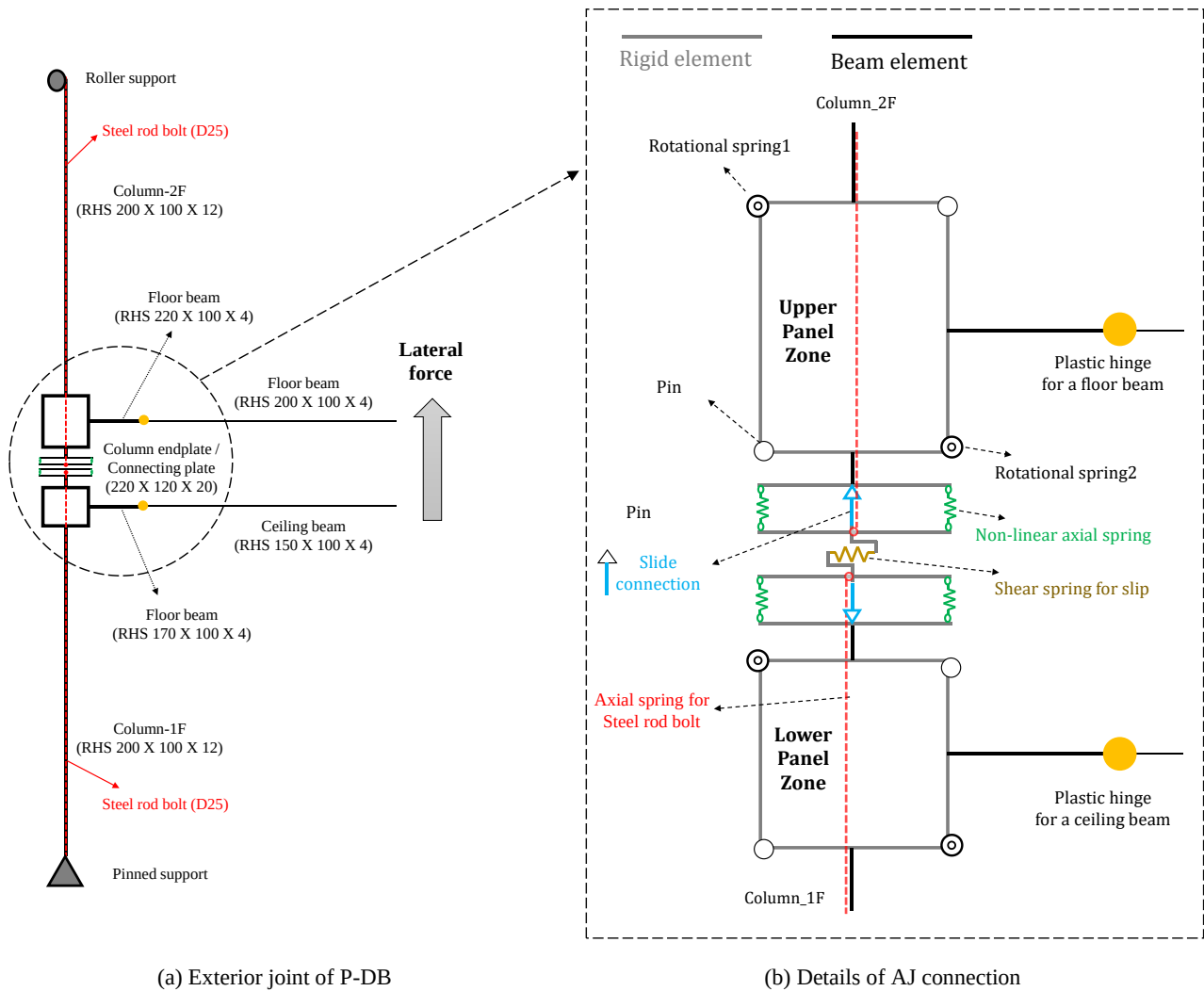


Figure 6.12: Simplified 2D model for the AJ beam-to-column joint

To describe the gap-opening at a column-to-column connection, four nonlinear axial springs were used between column endplates and the connecting plate, as shown in Figure 6.13 (1). Its axial compressive stiffness is infinite, while the axial tensile stiffness is nearly zero. If preload is applied, the springs are subjected to compressive forces, which becomes clamping forces between the plates. Then, when the lateral load is applied, the springs are subjected to tensile or compressive force depending on the location of the springs. When the induced compressive force becomes zero due to the tensile force, gap-opening begins to occur. To describe the lifting of one side of the column endplate when the gap-opening occurred, a slide connector was used to allow free vertical movement, and its end was modelled as a pin to allow free rotation.

The SRB was modelled using an axial spring (see Figure 6.13 (2)). Connections at both ends of the spring were set as pins. The initial stiffness k_{SRB} was calculated by (Eq. 3.44). The magnitude of preload was induced by adjusting the initial length (i.e. reference length in Abaqus) of the spring.

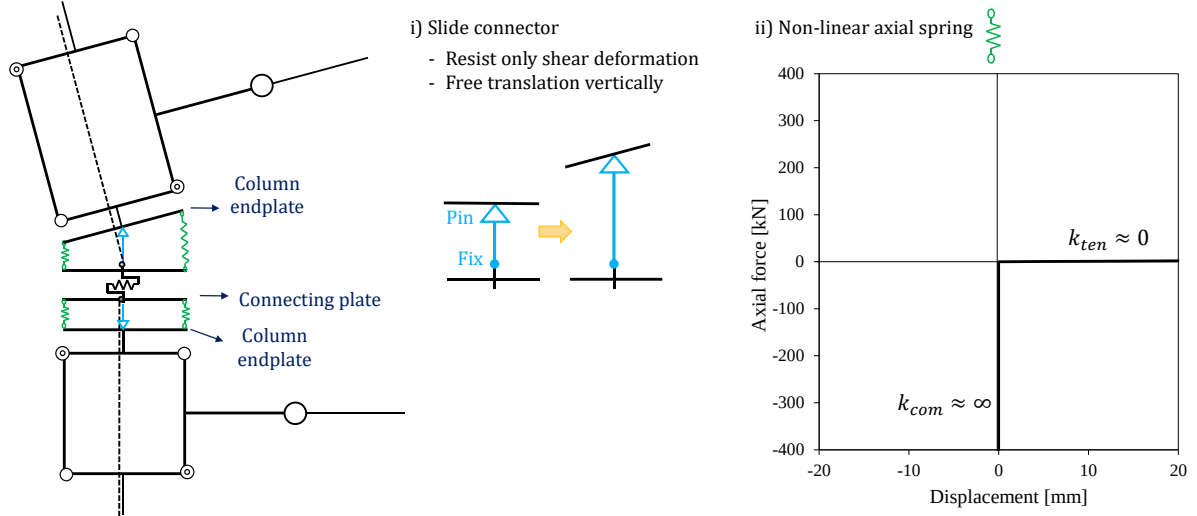
The slip was modelled using an axial spring, which allowed relative translation in the horizontal direction (see Figure 6.13 (3)). Both ends of the spring were set as rigid. The coupler-hole in the plates had a clearance of 2mm at each end to aid installation. Thus, the total slip was 4mm. Slip was resisted until reaching the slip resistance $F_{s,Rd}$. If the load is larger than $F_{s,Rd}$, slip occurs and the shear force is transferred by bearing. The bearing stiffness k_B was assumed as 917kN/mm from Table 6.7.

The plastic hinge properties for the floor and ceiling beams are shown in Figure 6.13 (4). Their moment-rotation relationships were determined from the 3D FE beam-to-column joint model simulated under cyclic loading (P-DB-Q in Section 5.4.2). The procedure to find the plastic hinge properties for the beams is described in Section 6.3.2.

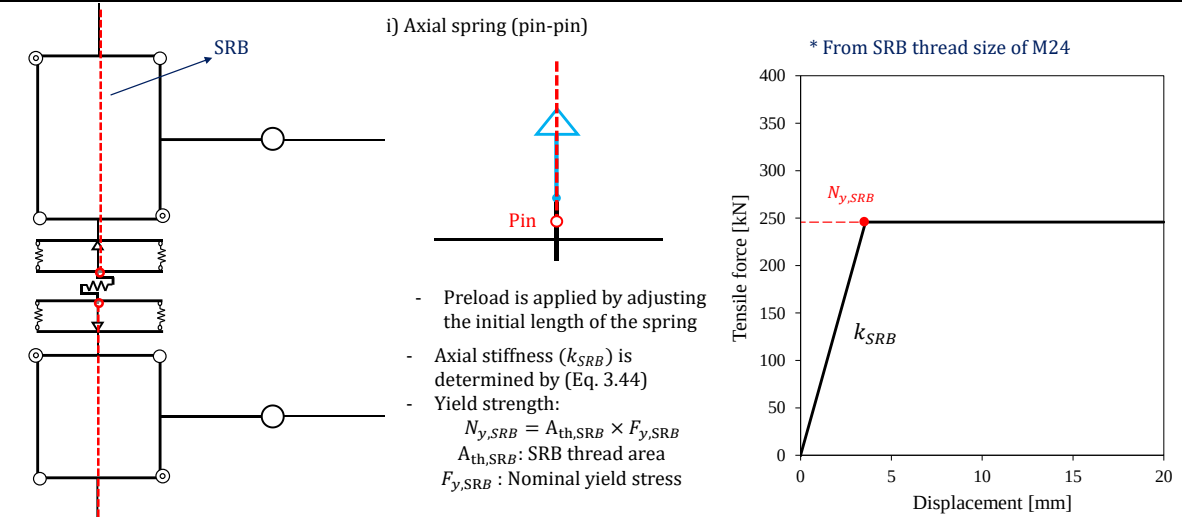
Lastly, panel zone shear behaviour was represented by Krawinkler's model [143]. The strength and stiffness properties of the panel zone were modelled by two rotational springs located in two of the four corners in the panel zone. The input data for the upper and lower panel zones were determined using the bi-linear moment-rotation relationship shown in Figure 6.13 (5).

Figure 6.14 illustrates the deformation of the 2D simplified model when the gap-opening and slip occurred due to the lateral force. The 2D model was created precisely to describe the behaviour of the AJ joint.

(1) Gap-opening at column-to-column connection



(2) Steel rod bolt and preload



(3) Slip at column-to-column connection

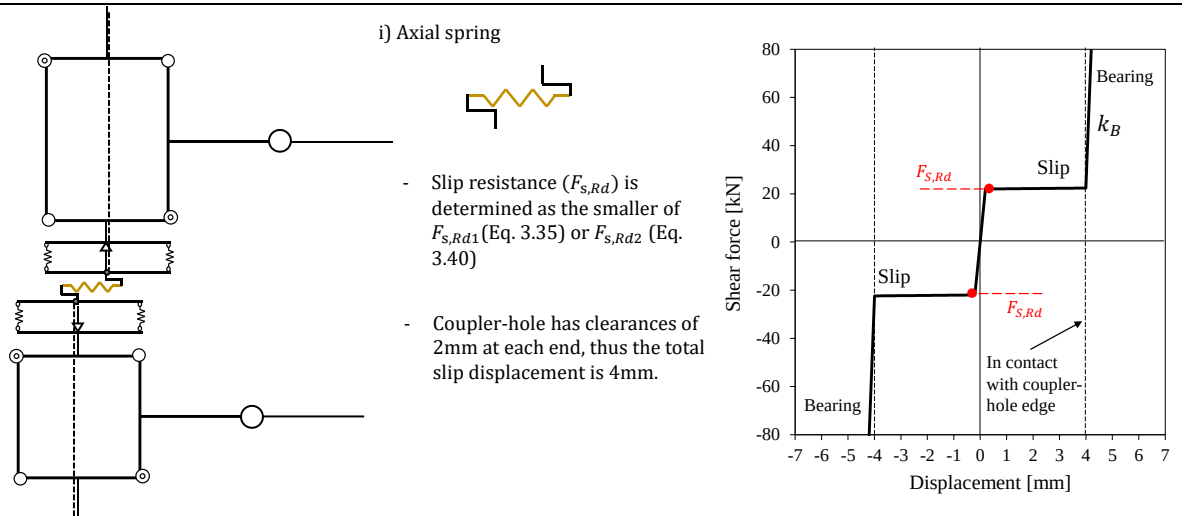
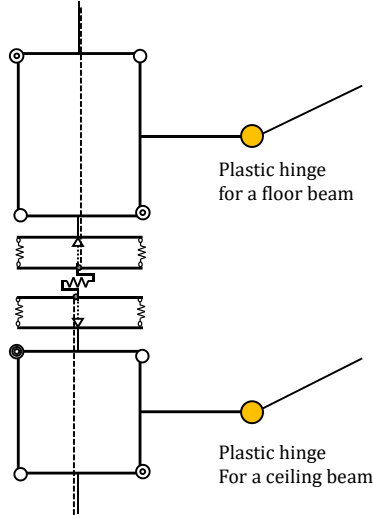


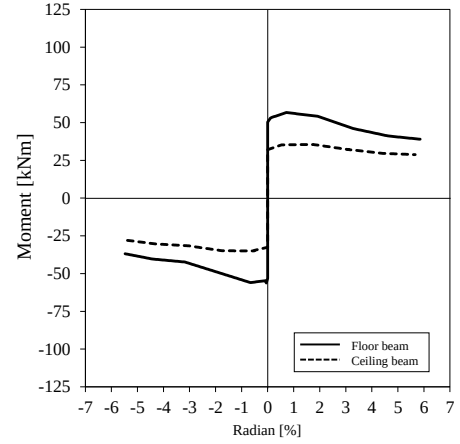
Figure 6.13: Spring elements in the 2D simplified model of AJ beam-to-column joint

(4) Plastic hinges for floor and ceiling beams

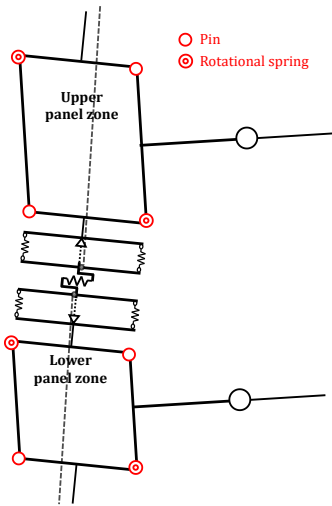


i) Rotational spring

(The detail is described in Section 6.3.2)



(5) Upper and lower panel zone



i) Rotational spring

- Two rotational springs are used for each panel zone to model to represent bilinear behaviour.
- The panel zone shear yield strength (V_y) and ultimate strength (V_u), and the corresponding distortion are determined as follows:

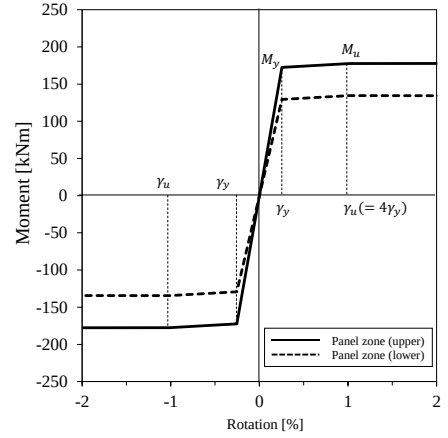
$$V_y = 0.55F_y d_c t_{cw} \quad \gamma_y = \frac{F_y}{\sqrt{3}G}$$

$$V_u = \frac{0.9F_y A_{vc}}{\sqrt{3}} + \frac{4M_{pl,fc,Rd}}{d_s} \quad \gamma_u = 4\gamma_y$$

- The moment for rotational spring is calculated as below:

$$M_y = V_y d_b$$

$$M_u = V_u d_b$$



V_y : The shear yield strength
 V_u : The shear ultimate strength
 γ_y : The yield distortion
 γ_u : The distortion corresponding to M_u or V_u
 $M_{pl,fc,Rd}$: The plastic moment capacity of column flanges
 F_y : The yield stress of the column
 G : The shear modulus of the column material

A_{vc} : The shear area of column
 d_b : The depth of beam
 d_c : The depth of column
 d_s : The distance between the centerline of the beam flanges
 t_{cw} : The thickness of column web
 M_y : The moment yield strength
 M_u : The moment ultimate strength

Figure 6.13 (cont.): Spring elements in the 2D simplified model of AJ beam-to-column joint

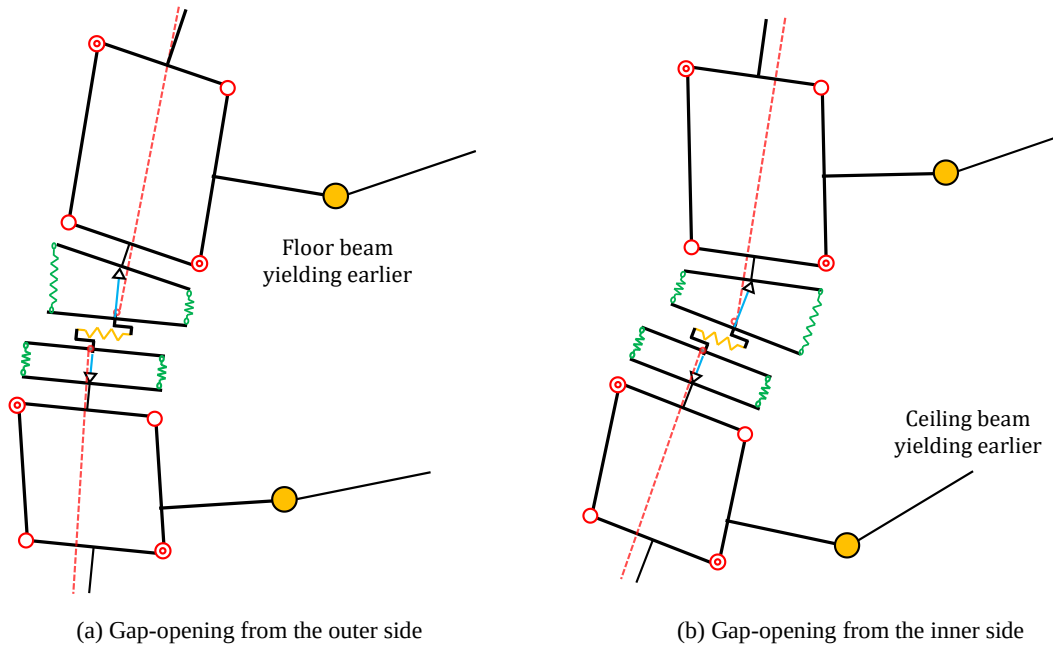


Figure 6.14: Deformation shape of the 2D simplified model in the case of gap-opening and slip

6.3.2. Plastic Hinge Properties of the Floor and Ceiling Beams

The plastic hinge properties of beams for the 2D simplified model were expressed as moment-rotation curves ($M - \theta$). The rotation was defined as the plastic rotation ($\theta_{pl,B}$) by excluding the elastic rotation ($\theta_{y,B}$) from the total beam rotation (θ_B). In the case of beam-to-column joints in modular building structures, θ_B includes the rotation of the floor and ceiling beams. Therefore, it was required to separate θ_B into the rotation of the floor beam (θ_{fb}) and ceiling beam (θ_{cb}). The procedure for determining the plastic hinge properties of the floor and ceiling beams is described below, using the P-DB-Q model introduced in Section 5.4.2.

Step 1: Find moment-beam rotation ($M_B - \theta_B$) curve from the hysteretic curve of joint

Figure 6.15 (a) shows the hysteretic curve of the beam-to-column joint and contribution of each component to the joint rotation. θ_B can be found by deducting the rotation by column flexural deformation (θ_C) and panel zone shear deformation (θ_{pz}) from the hysteretic curve. Figure 6.15 (b) shows the resulting moment-beam rotation θ_B curve (=interstorey drift in Figure 6.15 (b)) at the column face and its skeleton curve.

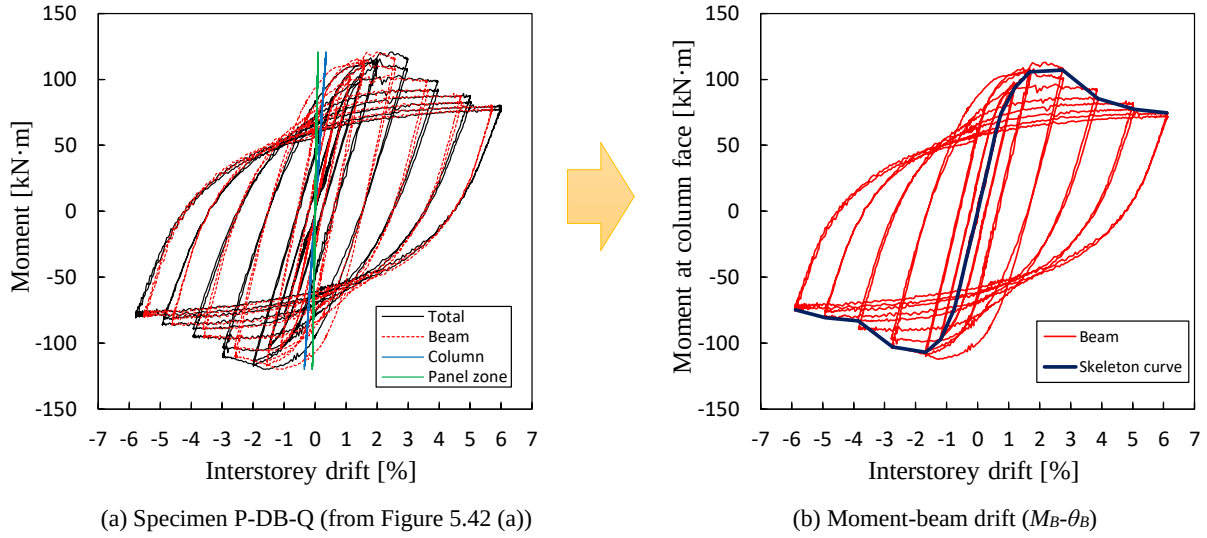


Figure 6.15: Finding Moment-beam rotation curve from hysteretic curve of specimen P-DB-Q

Step 2: Find shear forces acting on the floor beam (F_{fb}) and ceiling beam (F_{cb})

Using stresses extracted from the flanges of the floor (σ_{fb}) and ceiling (σ_{cb}) beams, which remain in the elastic range at some distance away from the plastic hinges (see Figure 6.16), F_{fb} and F_{cb} can be found through (Eq. 6.6) and (Eq. 6.7).

$$F_{fb} = \frac{M_{s,fb}}{L_s} \quad (M_{s,fb} = \sigma_{fb} \times W_{el,y,fb}) \quad (\text{for a floor beam}) \quad (\text{Eq. 6.6})$$

$$F_{cb} = \frac{M_{s,cb}}{L_s} \quad (M_{s,cb} = \sigma_{cb} \times W_{el,y,cb}) \quad (\text{for a ceiling beam}) \quad (\text{Eq. 6.7})$$

where : L_s The distance between the loading point and the location of extracting stresses;
 $W_{el,y,fb}$ and $W_{el,y,cb}$ Elastic section modulus of floor and ceiling beams, respectively.

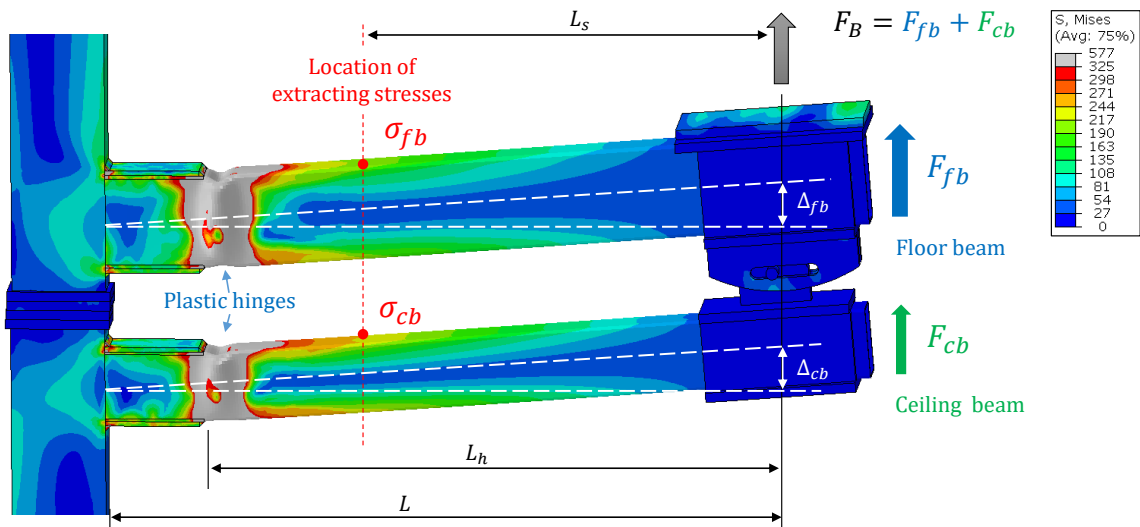


Figure 6.16: Location of extracting stresses on the flange of floor and ceiling beams for finding shear forces F_{fb} and F_{cb} (P-DB-Q model from Section 5.4.2)

Step 3: Draw moment-rotation curves for floor beam ($M_{fb} - \theta_{fb,pl}$) and ceiling beam ($M_{cb} - \theta_{cb,pl}$)

The moment and rotation at the column face for the floor and ceiling beams were determined through (Eq. 6.8) and (Eq. 6.9). Figure 6.17 (a) shows the skeleton curves of the floor and ceiling beams, and the sum of two curves, which matches the total $M_B - \theta_B$ curve.

$$M_{fb} = F_{fb} \times L \quad \theta_{fb} = \frac{\Delta_{fb}}{L} \quad (\text{for a floor beam}) \quad (\text{Eq. 6.8})$$

$$M_{cb} = F_{cb} \times L \quad \theta_{cb} = \frac{\Delta_{cb}}{L} \quad (\text{for a ceiling beam}) \quad (\text{Eq. 6.9})$$

(refer to Figure 6.18 for nomenclature)

The beam plastic rotation θ_{pl} was obtained by deducting the elastic rotation θ_y (determined by the 0.2% offset method) as follows:

$$\theta_{pl,fb} = \theta_{fb} - \theta_{y,fb} = \frac{\Delta_{fb} - \Delta_{y,fb}}{L} = \frac{\Delta_{pl,fb}}{L} \quad (\text{for a floor beam}) \quad (\text{Eq. 6.10})$$

$$\theta_{pl,cb} = \theta_{cb} - \theta_{y,cb} = \frac{\Delta_{cb} - \Delta_{y,cb}}{L} = \frac{\Delta_{pl,cb}}{L} \quad (\text{for a ceiling beam}) \quad (\text{Eq. 6.11})$$

(refer to Figure 6.18 for nomenclature)

Figure 6.17 (b) shows the moment-plastic rotation curves for the floor beam ($M_{fb} - \theta_{pl,fb}$) and ceiling beam ($M_{cb} - \theta_{pl,cb}$) at the column face.

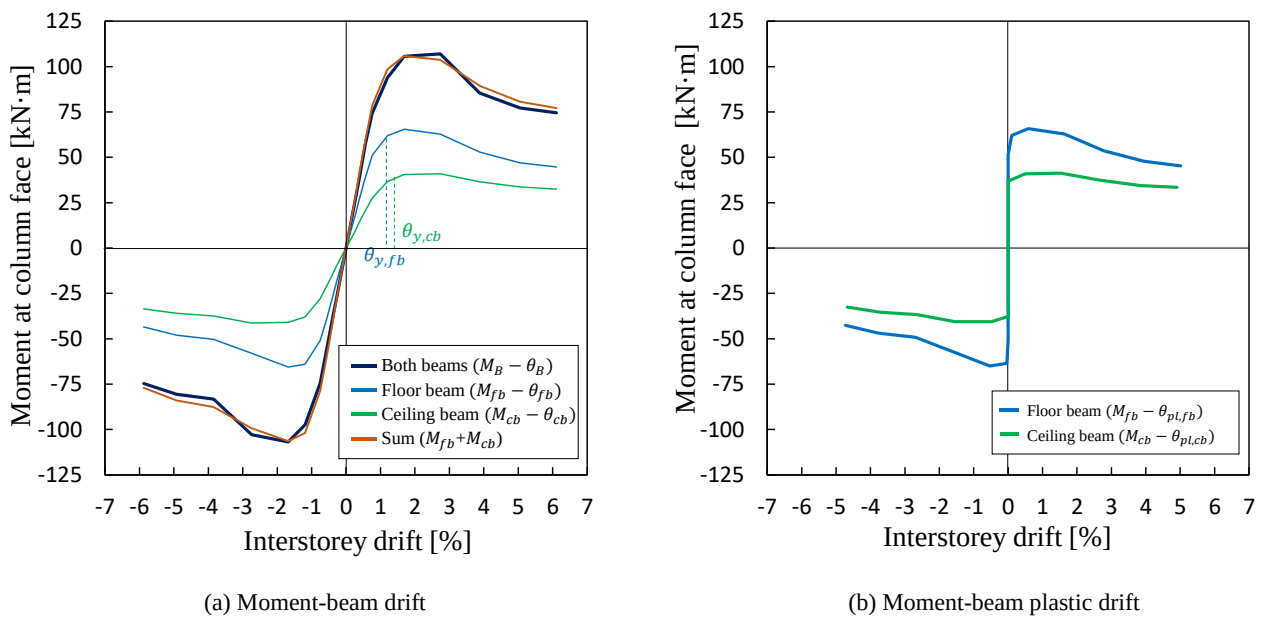
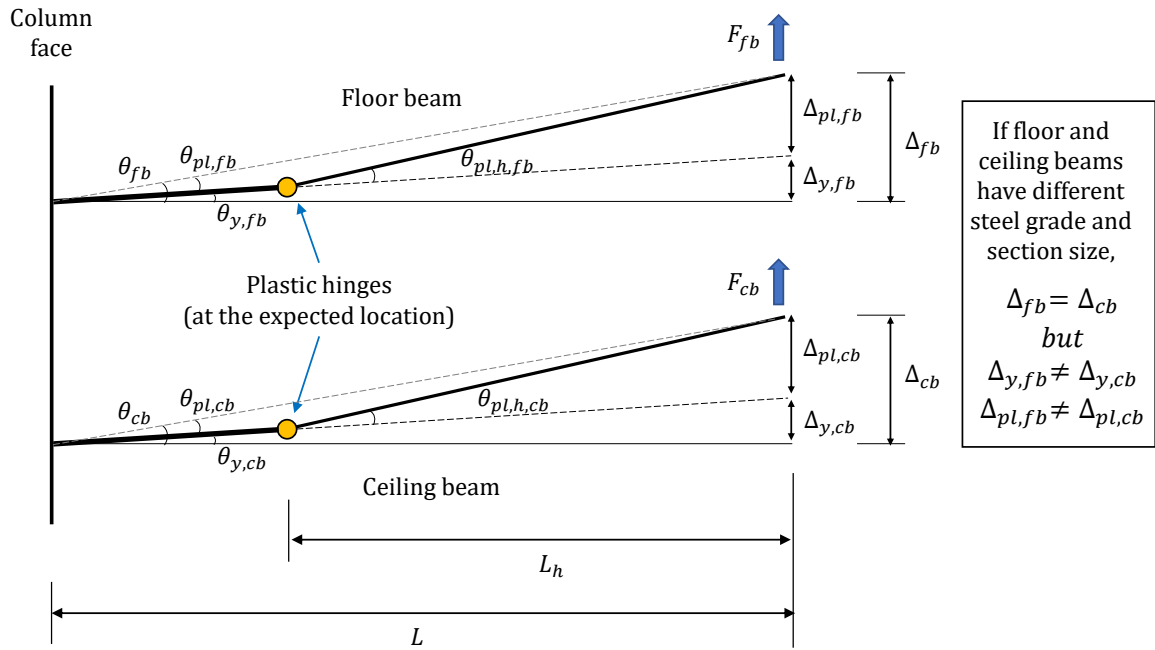


Figure 6.17: Moment- rotation curves for floor and ceiling beams at the column face



L	The distance between the loading point and the column face;
L_h	The distance between the loading point and the expected location of plastic hinge;
F_{fb} and F_{cb}	Shear forces acting on floor and ceiling beams, respectively;
θ_{fb} and θ_{cb}	Total rotation of floor and ceiling beams at column face, respectively;
$\theta_{y,fb}$ and $\theta_{y,cb}$	Yield rotation of floor and ceiling beams at column face, respectively;
$\theta_{pl,h,fb}$ and $\theta_{pl,h,cb}$	Plastic rotation of floor and ceiling beams at the expected location, respectively;
Δ_{fb} and Δ_{cb}	Total displacement at the tip of floor and ceiling beams, respectively;
$\Delta_{y,fb}$ and $\Delta_{y,cb}$	Yield displacement at the tip of floor and ceiling beams, respectively;
$\Delta_{pl,fb}$ and $\Delta_{pl,cb}$	Plastic displacement at the tip of floor and ceiling beams, respectively.

Figure 6.18: Determination of plastic hinge properties for floor and ceiling beams

Step 4: Find plastic hinge properties at the expected location for floor and ceiling beams

The plastic hinge properties at the expected location for the floor beam ($M_{h,fb}$ - $\theta_{pl,h,fb}$) and ceiling beam ($M_{h,cb}$ - $\theta_{pl,h,cb}$) were obtained as follows:

$$M_{h,fb} = F_{fb} \times L_p \quad (\text{Eq. 6.12})$$

$$M_{h,cb} = F_{cb} \times L_p \quad (\text{Eq. 6.13})$$

$$\theta_{pl,h,fb} = \frac{\Delta_{pl,fb}}{L_p} = \theta_{pl,fb} \times \frac{L}{L_p} \quad (\Delta_{pl,fb} = \theta_{pl,fb} \times L) \quad (\text{Eq. 6.14})$$

$$\theta_{pl,h,cb} = \frac{\Delta_{pl,cb}}{L_p} = \theta_{pl,cb} \times \frac{L}{L_p} \quad (\Delta_{pl,cb} = \theta_{pl,cb} \times L) \quad (\text{Eq. 6.15})$$

(refer to Figure 6.18 for nomenclature)

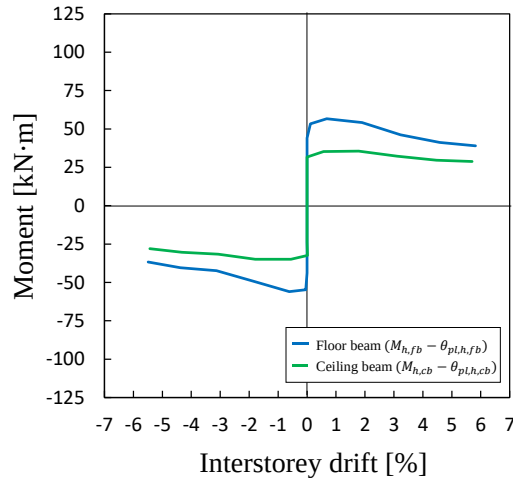


Figure 6.19: Plastic hinge properties for floor and ceiling beams at the expected location

Figure 6.19 shows plastic hinge properties for the floor and ceiling beams at the expected hinge location. Figure 6.20 and Table 6.8 show the simplified hinge properties of the beams for the input data for the 2D joint model and relevant acceptance criteria given by FEMA 356 [144]. EN 1998-3 (Annex B) [145] also provides criteria for the acceptable damage state condition of the plastic hinges related to three limit states, which are similar to those of FEMA 356 when assuming that Immediate Occupancy (IO) corresponds to Damage Limitation (DL), Life Safety (LS) corresponds to Significant Damage (SD), and Collapse Prevention (CP) corresponds to Near Collapse (NC).

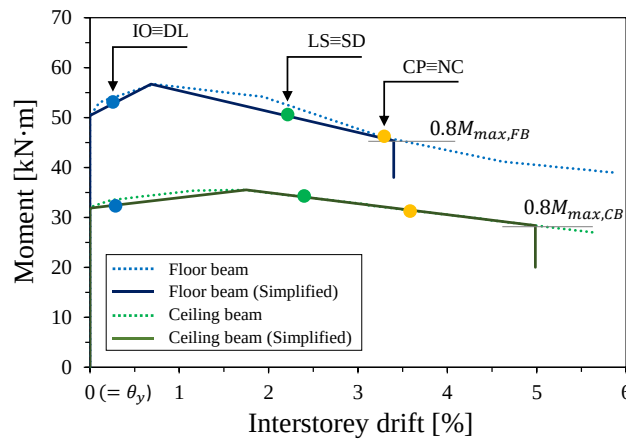


Figure 6.20: Simplified moment-rotation relationship of plastic hinges for beams and relevant acceptance criteria

Table 6.8: Response parameters and acceptance criteria for plastic hinges

Members	3D joint simulation result (P-DB-Q from Section 5.4.2)			FEMA acceptance criteria for plastic hinges [rad]		
	M_y	M_{max}	θ_y	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
	[kNm]	[kNm]	[rad]	$0.25\theta_y$	$2\theta_y$	$3\theta_y$
Floor beam (RHS 200X100X4)	50	57	0.011	0.0028	0.022	0.033
Ceiling beam (RHS 150X100X4)	31	36	0.012	0.0030	0.024	0.036

6.3.3. Verification of the 2D Simplified AJ Joint Model

The 2D simplified AJ joint model of P-DB (see Figure 6.12) was validated by comparing it to the 3D detailed joint model under monotonic loading. (The plastic hinge properties of the floor and ceiling beams for the 2D model were obtained from the monotonic simulation result of the 3D joint model of DB-1.0P shown in Figure 5.45 using the procedure described in Section 6.3.2.) Table 6.9 and Figure 6.21 show three comparisons of the 2D simplified and 3D detailed joint models of P-DB. They show good agreement by the 2D models describing gap-opening and slip (i.e. location and strength of occurrence), indicating the impact of gap-opening on the joint behaviour (i.e. the failure mechanism, reduction in stiffness, and increase in tensile force in SRBs) was accurately simulated. Regarding the initial stiffness and the ultimate strength of the joint, the differences between the 2D and 3D models were less than 2% and 1%, respectively. Also, the variation of tensile force in the SRBs due to the gap-opening in the 2D and the 3D models were quite similar. Their peak forces were 0-8% different (see Figure 6.21 (b)), which is an acceptable level of agreement for predicting the behaviour of SRBs. In addition, as shown in Figure 6.22, the 2D model was compared to the skeleton curve from the 3D model of P-DB-Q (refer to Figure 5.28). The figures show good agreement in the positive and negative directions of lateral forces. In summary, it was confirmed that the 2D simplified joint model can precisely simulate the behaviour of the AJ joint.

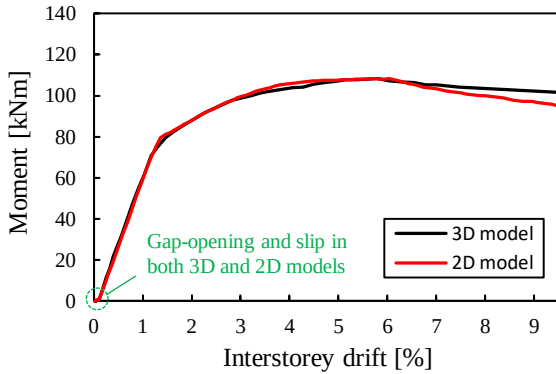
Table 6.9: Comparison of 2D simplified and 3D detailed joint models of P-DB under monotonic loading

FE model	Preload 0kN		Preload 43kN		Preload 213kN	
	3D model (DB-NoP*)	2D model	3D model (DB-0.2P*)	2D model	3D model (DB-1.0P*)	2D model
K_i [kNm/rad]	5766	5790 (+0.4%)	6166	6183 (+0.3%)	8116	7953 (-2.0%)
M_u [kNm]	108	108 (+0.0%)	110	109 (-0.9%)	109	110 (+0.9%)
$N_{s,p}$ [kN]	157	157 (+0.0%)	151	163 (+7.9%)	217	216 (-0.5%)
Failure mechanism	CB → FB	CB → FB	CB → FB	CB → FB	FB → CB	FB → CB

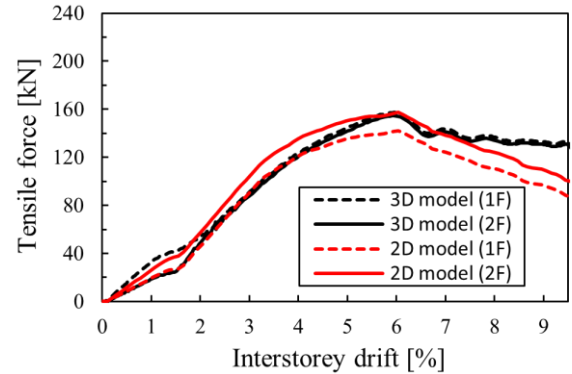
Note: K_i : the initial rotational stiffness; M_u : the ultimate moment strength; $N_{s,p}$: the peak tensile force of the steel rod bolt; CB: Ceiling beam; FB: Floor beam.

* The simulation results of the 3D joint models are from Figure 5.44 and Table 5.4.

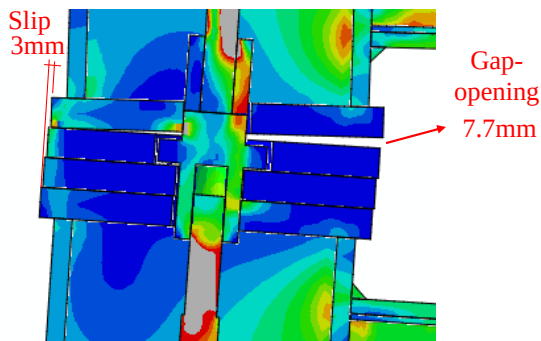
1. Preload 0kN



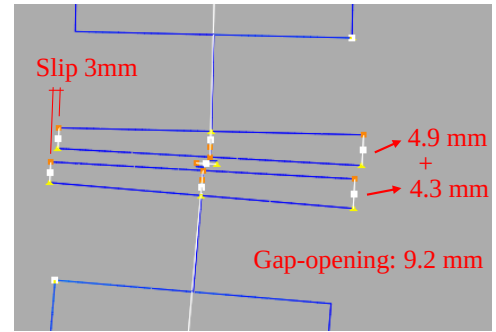
(a) Moment-interstorey drift curve



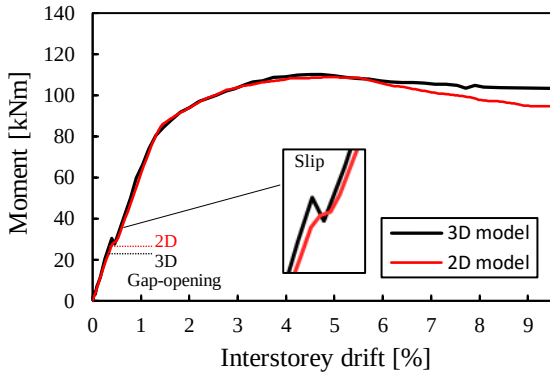
(b) Variation of tensile force in SRBs



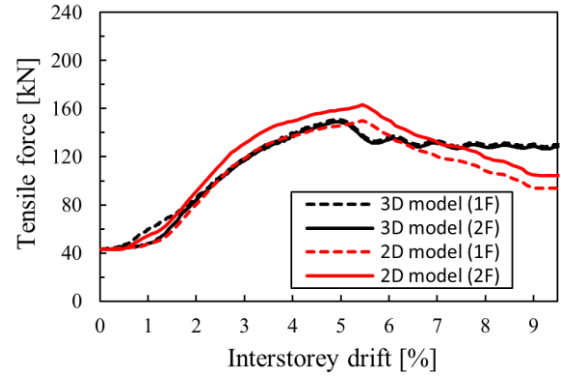
(c) Location and shape of gap-opening and slip at the ultimate strength M_u



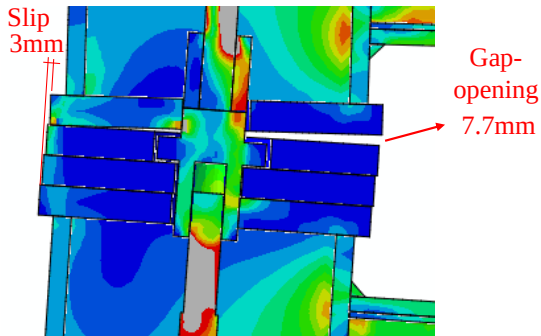
2. Preload 43kN



(a) Moment-interstorey drift curve



(b) Variation of tensile force in SRBs



(c) Location and shape of gap-opening and slip at the ultimate strength M_u

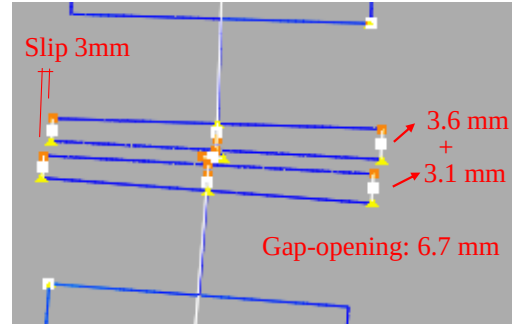


Figure 6.21: Comparison of 2D simplified and 3D detailed joint models of P-DB under monotonic loading

3. Preload 213kN

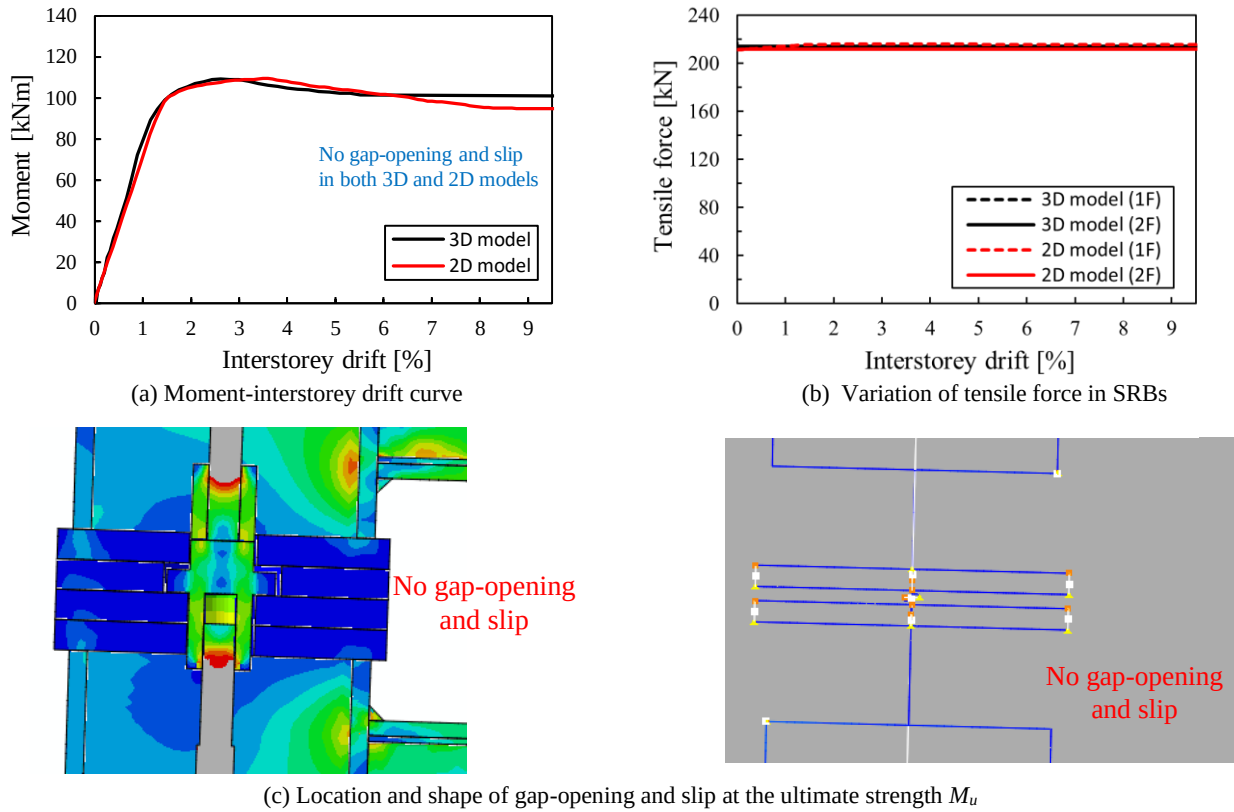


Figure 6.21 (cont.): Comparison of 2D simplified and 3D detailed joint models of P-DB under monotonic loading

Under cyclic loading

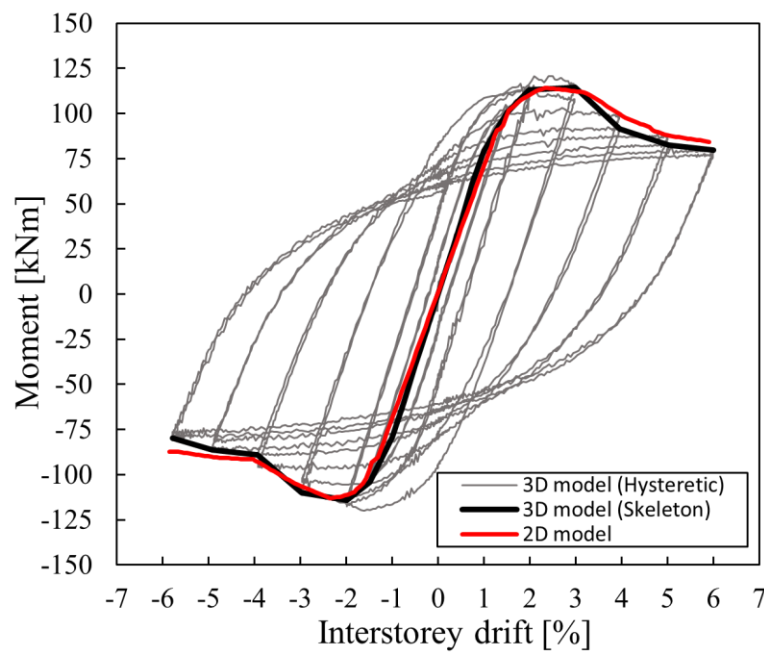


Figure 6.22: Comparison of 2D simplified and 3D detailed joint models of P-DB-Q

6.3.4. Implementation of the Modular Building Frame with the Simplified AJ Joint Model

Prior to conducting wind and seismic analysis with an 8-storey mid-rise modular building, the verified 2D joint model was first applied to a 4-storey modular building frame, which was stacked in a row, to make sure the model works properly in the frame, as shown in Figure 6.23 (a). The loading steps were as follows: (1) inducing preload of 80kN to AJ connections; (2) applying gravity load (31kN per column, which is $\frac{1}{4}$ of a module weight of 121kN); and (3) applying displacement-controlled loading at the top of the frame (see Figure 6.24).

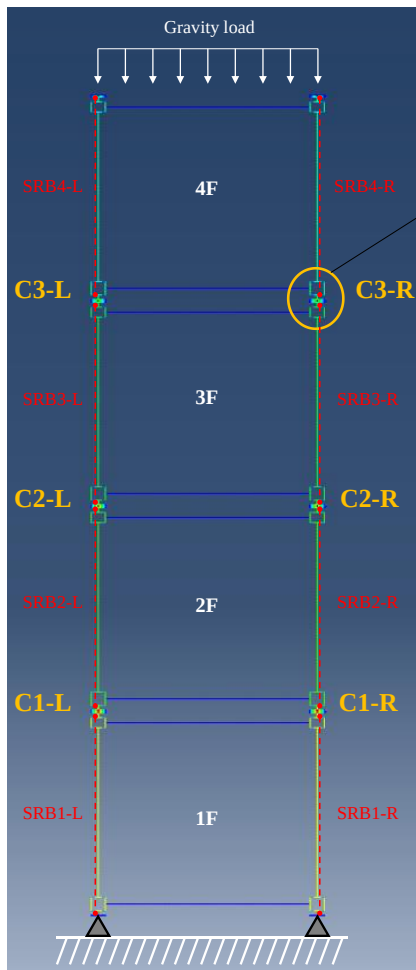
Figure 6.23 (b), (c) and (d) show the variation in compressive forces (=clamping forces) at AJ connections (C-L and C-R) and the tensile force acting on steel rod bolts (SRB-L and SRB-R). As preload was induced to the SRBs, the connections incurred compressive forces, while the SRBs were subjected to tensile forces. As gravity load was applied, the connections experienced additional compressive forces. The connections located in the low storeys had larger clamping forces. However, the final clamping forces at the connections were not equal to the sum of the induced preload and the gravity load because of the 'Weight effect of upper modules' as described in Section 3.4.5. That is, when the modules are stacked, the initially stretched length of SRBs inside columns is shortened by the same amount as the compressive axial deformation of the column, resulting in a reduction of the initially induced preload. Figure 6.23 (e) shows how the tensile forces (= the initially induced preload) in the SRBs are reduced after the gravity load is applied. The amount of reduced preload and final clamping forces at the connections due to the weight effect in the FE frame model are almost the same as the analytically predicted values, as shown in Figure 6.23 (f).

As the frame is subjected to a lateral force, tensile forces on the left-side columns and compressive forces on the right-side columns develop (see Figure 6.24). As a result, AJ connections located on the left side experience a reduction in compressive forces and a gap-opening occurs in one of the springs (see No.1-4 in Figure 6.24 (a)) as the compressive force is reduced to zero. Once the gap-opening occurs, the SRB near the connection is subjected to additional tensile force to resist the lateral force, as shown in Figure 6.24 (h). On the other hand, the clamping forces in all connections located on the opposite side increase due to the additional compressive axial forces, and the induced preload in the SRBs are temporarily reduced as the columns experience further compressive deformation, as shown in Figure 6.24 (e).

In summary, it was confirmed that the FE 2D frame model can represent the actual behaviour of the frame composed of modules connected with the AJ connection.

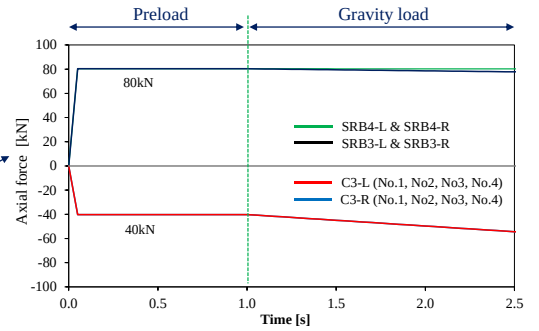
Step1 – Applying preload

Step2 – Applying gravity load

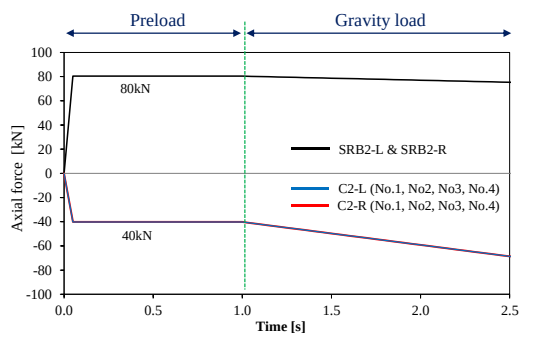


(a)

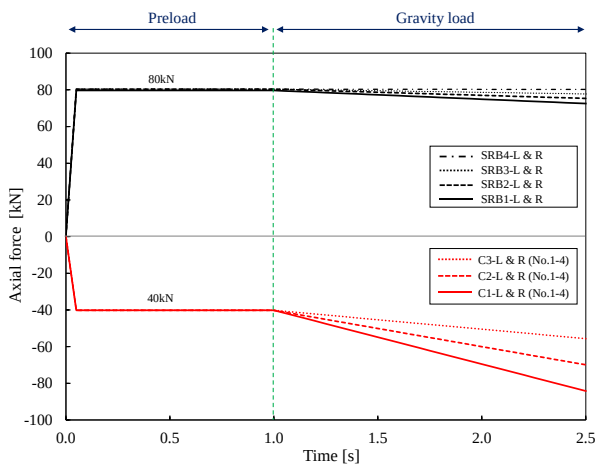
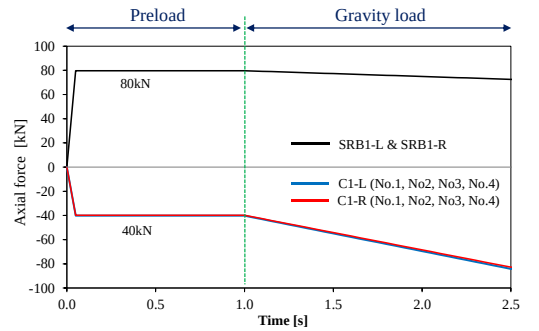
(b) C3-L and C3-R



(c) C2-L and C2-R



(d) C1-L and C1-R



(e)

[Per column]

Location	FE model			Analytically predicted clamping force*	Ratio
	Applied preload (P) [kN]	Gravity Load (G) [kN]	Final clamping force (P+G-P _R) [kN]		
C3-L & R	80	31	110 (-1)	110	1.00
C2-L & R	80	62	139 (-3)	138	0.99
C1-L & R	80	93	167 (-6)	167	1.00

(-): Reduced amount of preload, P_R due to the weight effect of upper modules
* : by Eq. 3.53 and 3.54

(f)

Figure 6.23: Variation in tensile force in SRBs and clamping forces at the AJ connection in the FE frame model:
Step1 - Inducing preload & Step2 - Applying gravity load

Step3 – Applying lateral force

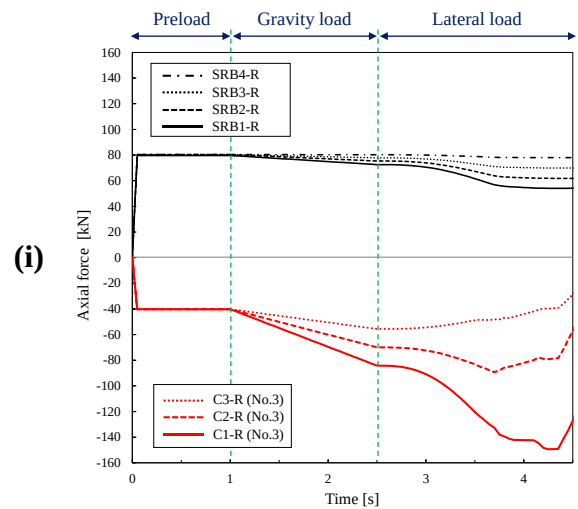
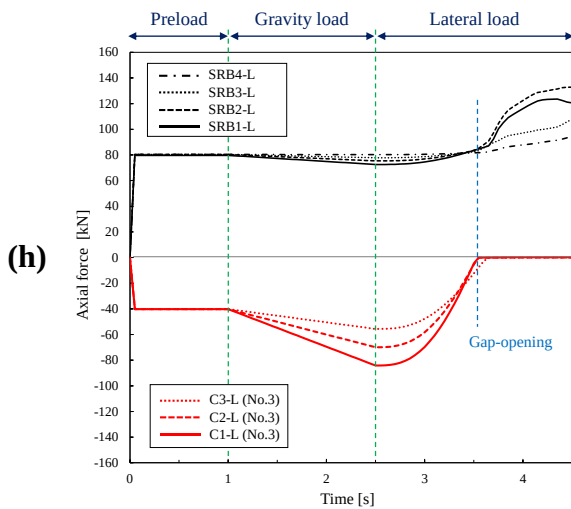
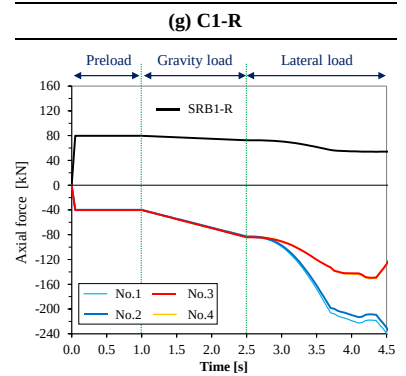
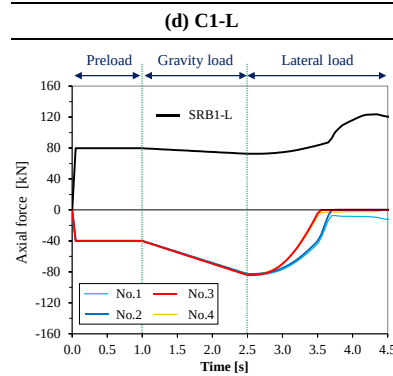
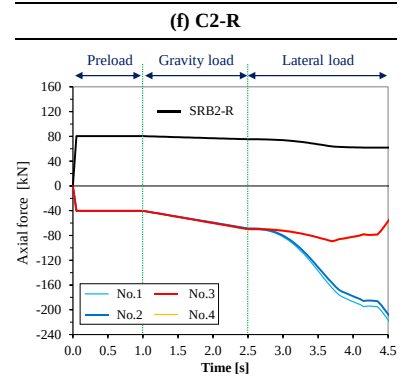
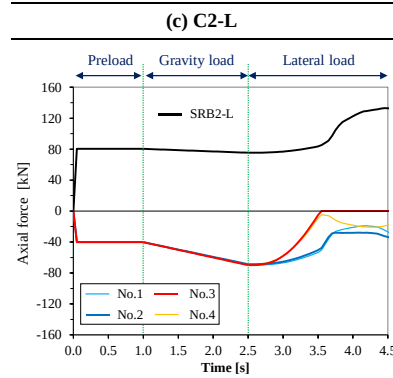
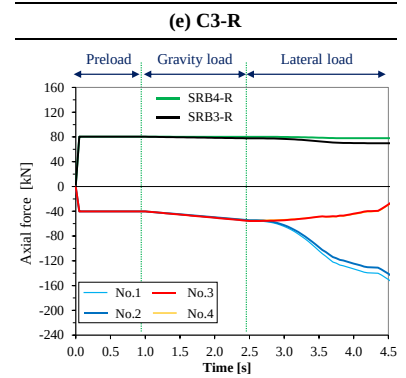
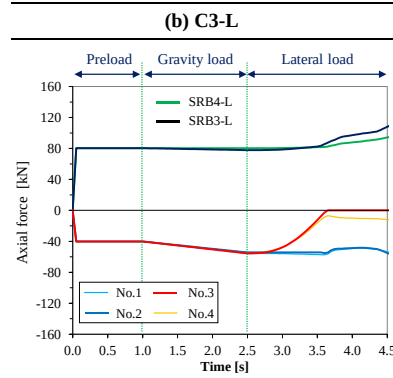
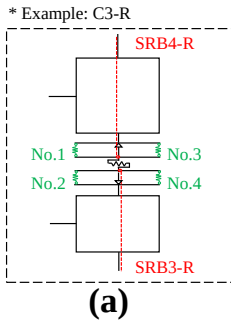
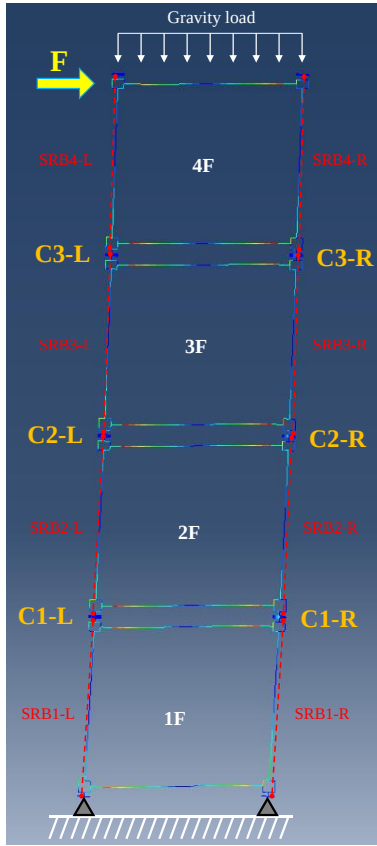


Figure 6.24: Variation in tensile force in SRBs and clamping forces at the AJ connection in the FE frame model:
Step3 - Applying displacement-controlled loading at the top of the frame

6.4. Frame Analysis for the Modular Building with AJ Connections

6.4.1. Design of the Prototype Mid-rise Modular Building

Usually, a mid-rise or high-rise steel-framed modular building has a reinforced concrete (RC) core to resist lateral loads such as wind and earthquake loads. If the module building is constructed with only modules using the AJ connection, construction speed will be maximised because site works (e.g. constructing RC core and additional work after installation of connections between modules) can be reduced significantly. Therefore, a prototype structure for an 8-storey mid-rise modular building was designed with 100% steel-framed modules, as shown in Figure 6.25. Figure 6.27 shows the plan view of the prototype building. The building consists of 200 modules with two types of corner-supported modules (see Figure 6.26). The size of the module is $6.6\text{m} \times 3.3\text{m} \times 3\text{m}$. The module has a one-way concrete slab of 150mm thickness. Regarding the lateral load resisting system, the moment resisting frames (**MRFs**) composed of the shorter side of Type1 modules mainly resist the lateral load in the X-direction, while the concentrically braced frames (**CBFs**) composed of the longer side of Type1 modules resist the lateral load in the Y-direction (see Figure 6.25). Figure 6.28 (a) shows the location of the MRFs and CBFs on the structural layout of the typical floor, and Figure 6.28 (b), (c), and (d) show vertical configurations of the MRFs and CBFs. The structural analysis and design were performed using ETBAS software [146].

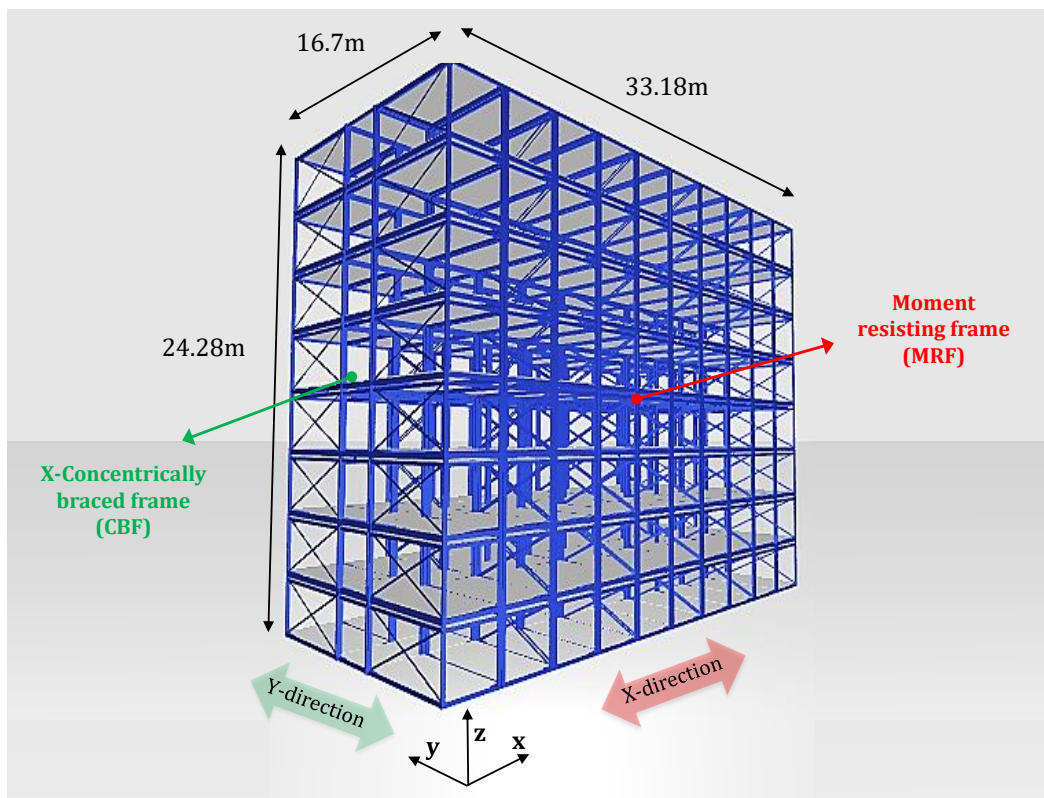
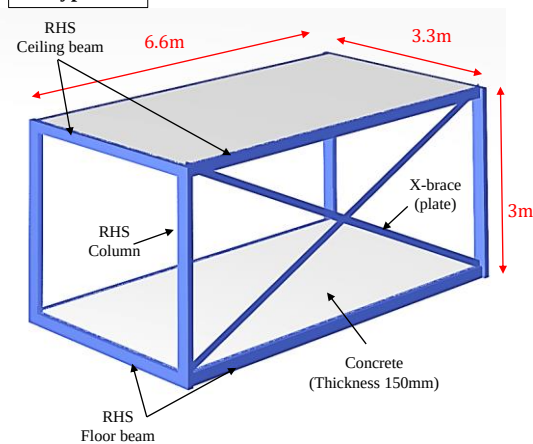


Figure 6.25: 3D view of a prototype structure of 8 storey mid-rise steel-framed modular building

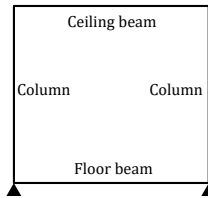
Type 1



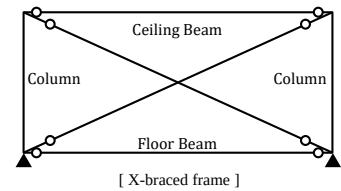
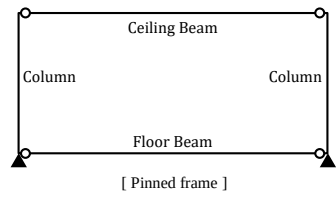
Modeling of module

○ : Pinned connection

Shorter side



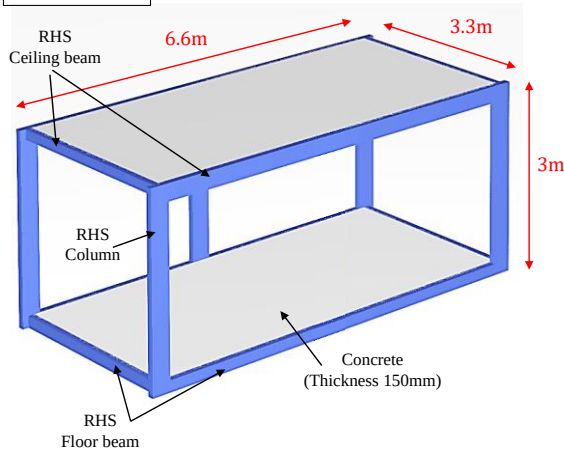
Longer side



Member	Section size [mm]	Steel grade	F_y [N/mm ²]	Section class
Column	RHS 200 X 100 X 12	SRT 355	355	Class 1
Floor beam (Shorter side)	RHS 200 X 100 X 4	SRT 275	275	Class 1
Ceiling beam (Shorter side)	RHS 150 X 100 X 4	SRT 275	275	Class 1
Floor beam (Longer side)	RHS 250 X 100 X 6	SRT 275	275	Class 1
Ceiling beam (Longer side)	RHS 150 X 100 X 5	SRT 275	275	Class 1
Braces (1F-3F)	Plate 110 X 3.7	SS315	315	-
Braces (4F-5F)	Plate 105 X 3.5	SS275	275	-
Braces (6F-8F)	Plate 100 X 3.2	SS235	235	-

(a)

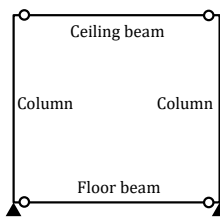
Type 2



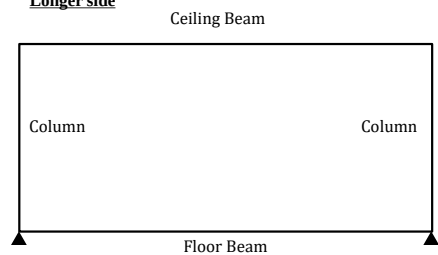
Modeling of module

○ : Pinned connection

Shorter side



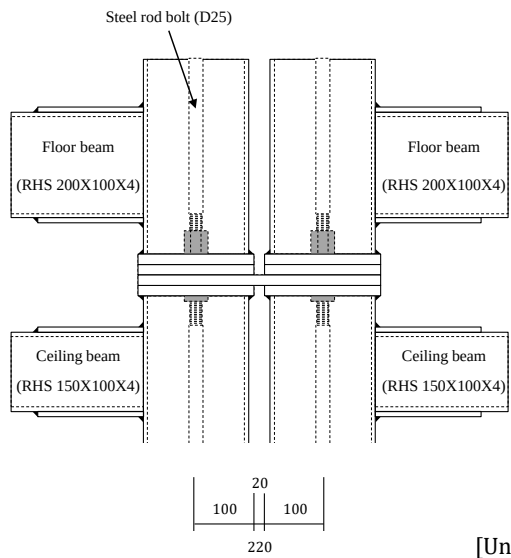
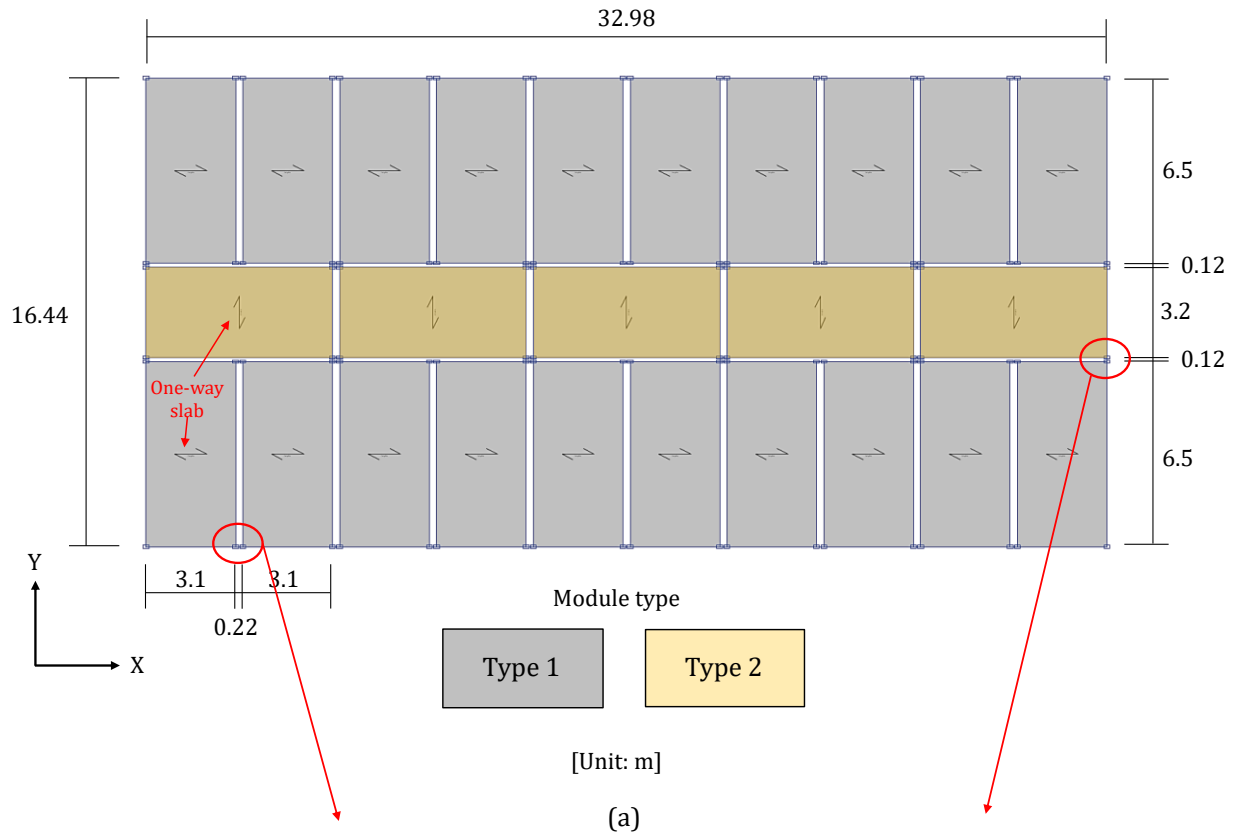
Longer side



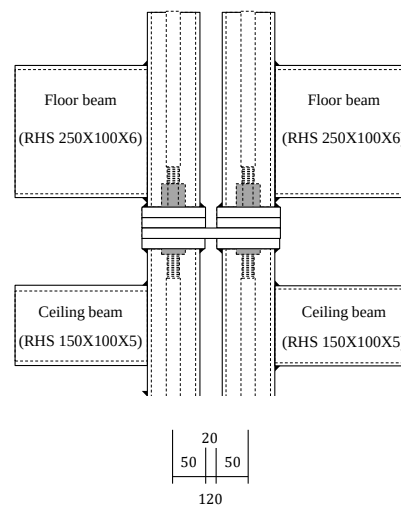
Member	Section size (mm)	Steel grade	F_y [N/mm ²]	Section class
Column	RHS 300 X 100 X 12	SRT 355	355	Class 1
Floor beam (Shorter side)	RHS 250 X 100 X 6	SRT 275	275	Class 1
Ceiling beam (Shorter side)	RHS 150 X 100 X 5	SRT 275	275	Class 1
Floor beam (Longer side)	RHS 250 X 100 X 6	SRT 275	275	Class 1
Ceiling beam (Longer side)	RHS 250 X 100 X 6	SRT 275	275	Class 1

(b)

Figure 6.26: 3D view and modelling of two types of corner-supported modules: (a) Type 1: Single module with X-brace, and (b) Type 2: Corridor module (Note: only the load-bearing elements are modelled)

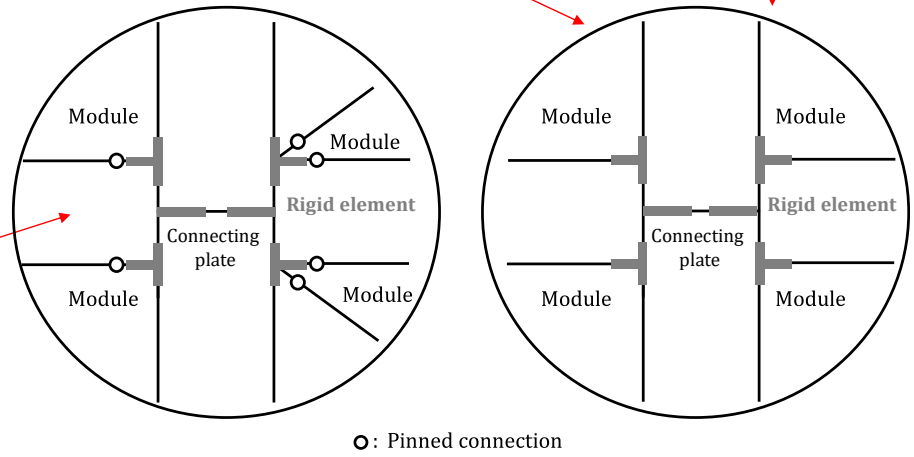
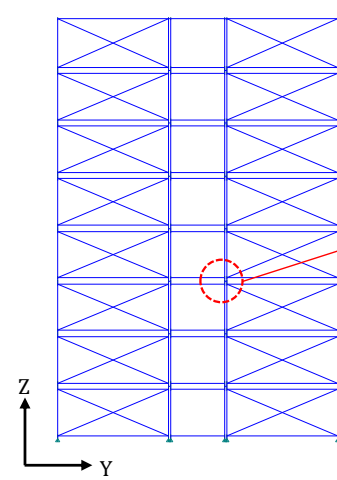
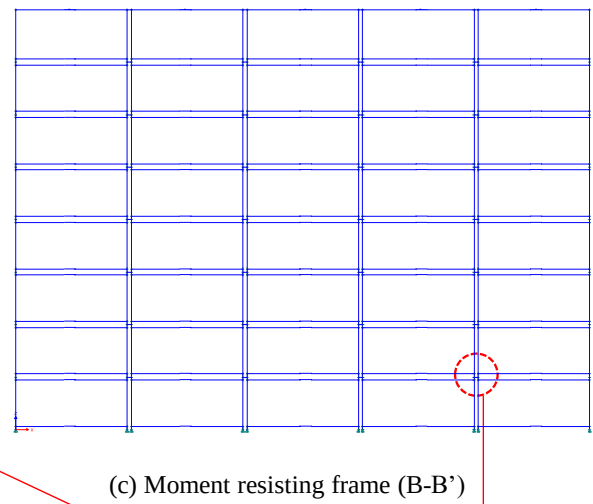
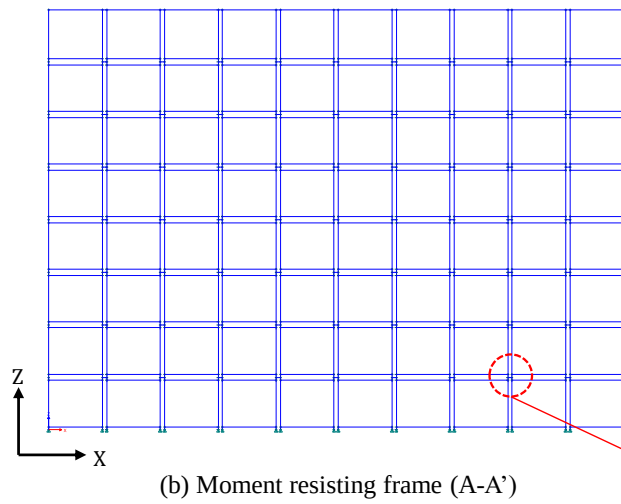
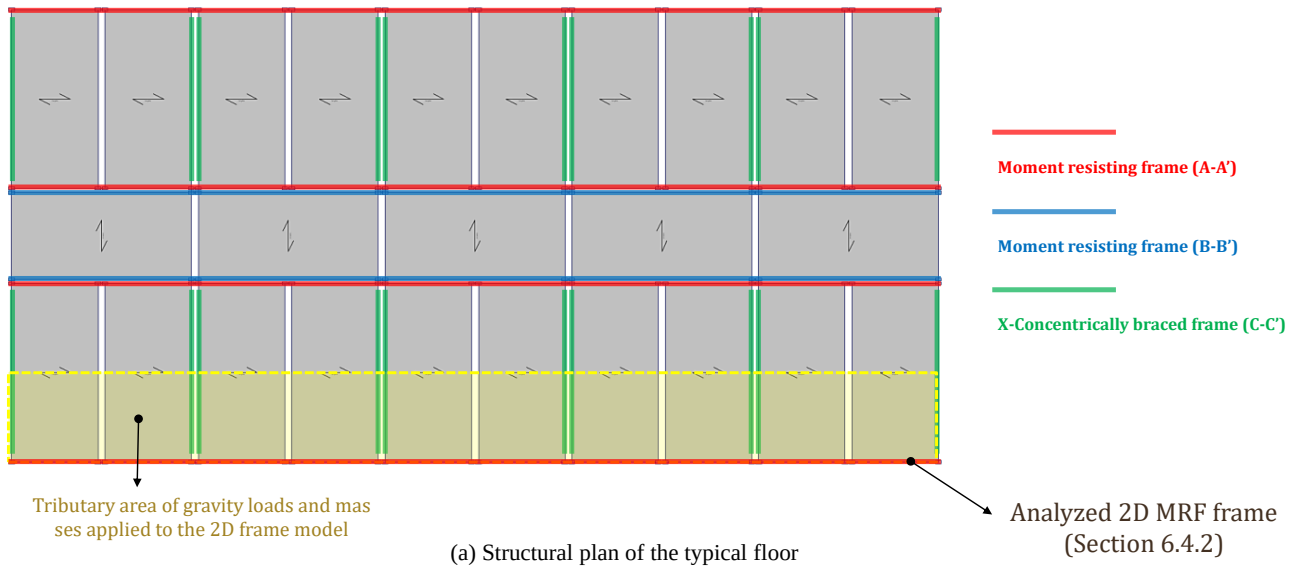


(b)



(c)

Figure 6.27: (a) Plan view of the typical floor for the prototype modular building, (b) and (c) Side views of AJ connections



(e) Modeling of between modules

Figure 6.28: Structural plan and elevation views of the prototype modular building: (a) A-A', B-B' and C-C' views

Assuming that the prototype structure was constructed in Seoul, Korea in a medium seismic area, the characteristics of modules used in Korea (e.g. module size, concrete floor thickness, etc.) were considered. The capacity design was carried out to withstand earthquakes in accordance with EN 1990 [147], EN 1991-1-1 [148], EN 1993-1-1 [88], and EN 1998-1 [86], using the response spectrum in KBC 2016 [101]. The building was analysed for stiff soil condition (Site class S_D) with spectra acceleration $S_a=0.22g$ and importance factor (I_E) equal to 1.2 in accordance with KBC 2016. The behaviour factor $q=3.5$ (ductility class medium) for MRFs and CBFs is used. Figure 6.29 shows the elastic and design response spectra. Table 6.10 summarises the characteristic values of dead and live loads used.

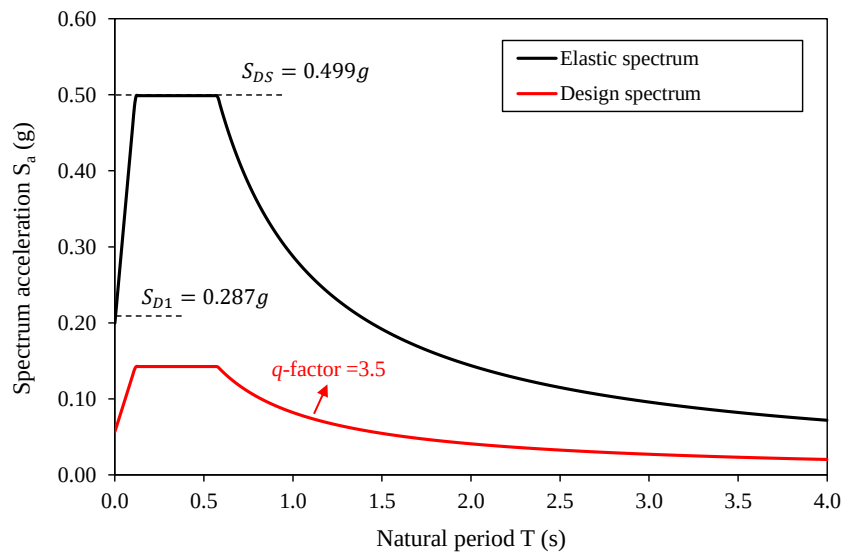


Figure 6.29: Elastic and design response spectra ($S=0.22g$ and Site class SD) according to KBC (2016)

Table 6.10: Dead and live loads used for prototype structure

Gravity load	Dead load (G_k) [kN/m ²]	Live load (Q_k) [kN/m ²]
Roof	4.7	1.0
Ceiling	0.2	-
Floor	4.7*	2.0

* Includes concrete slab, exterior cladding, finishing floor elements, and partition walls.

To evaluate the inertial effects in the seismic design, masses are computed from the following load combination, according to EN 1990 [147].

$$G_k + \Psi_2 \times Q_k \quad (\text{Eq. 6.16})$$

where: G_k The permanent load;
 Ψ_2 The variable load;
 Q_k The factor for quasi-permanent value of a variable action (considered 0.15 for the floor and 0.3 for the roof).

Table 6.11 shows the seismic weights and masses of the prototype building. The weight of each module is 121kN (12.3ton).

Table 6.11: Seismic weight and masses for the 8 storey mid-rise modular building

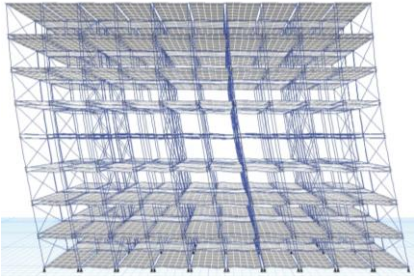
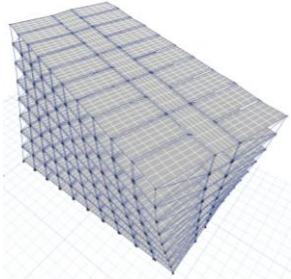
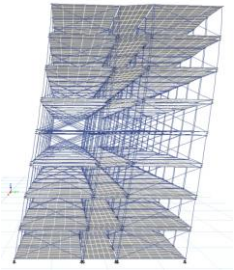
Storey	G_k [kN]	$\Psi_2 \cdot Q_k$ [kN]	Seismic Weight [kN]	Seismic mass [ton]
8 th	5109	298	5407	551
From 1 st – 7 th	2850	149	2999	306
Total	25063	1341	26404	2693

In the structural model, a discrete rigid diaphragm was used for the floor slab in each module. Vertical and horizontal inter-module connections were modelled as rigid connections as shown in Figure 6.28 (e). The gap between modules was 220mm and 120mm in the X-axis and Y-axis, respectively, to account for the centreline offset of columns and a small gap of 20mm (see Figure 6.27 (b) and Figure 6.28 (e)).

The Ultimate Limit State (ULS) design was performed according to EN 1998-1 [1] rules. The dissipative members (i.e. beams in MRFs and braces in CBFs) were designed from the persistent design situation ULS combination (1.0Gk+0.3Qk+1.0Ex), while the non-dissipative members (i.e. columns in MRFs and beams and columns in CBFs) were designed from the seismic design situation combination (1.0Gk+0.3Qk+1.1·rov·Ω·1.0Ex). The interstorey drift limitation of 0.5% of the storey height was used to satisfy the damage limitation requirement under seismic action having a larger probability of occurrence than the design seismic action [86]. Table 6.12 shows modal shapes of the main modes of vibrations, participating mass, and base shear. The sum of the effective modal masses was considered with at least 90% of the total mass of the structure. The modal shape shows that response was governed by the fundamental mode of vibration in the X- and Y-directions.

Figure 6.30 shows the interstorey drift in X-dir. (MRFs) and Y-dir. (CBFs), and they satisfied the damage limitation requirement. It is noted that the cross-sectional size of floor and ceiling beams on the shorter side of the module for MRFs was determined to control drift, not the demand strength.

Table 6.12: Modal shape of the main modes of vibrations and natural period (T_n)

Mode	1 st mode of vibration	2 nd mode of vibration	3 rd mode of vibration
Modal shape			
Period, T_n [s]	1.814	1.173	1.154
UX	75.7%	0%	0%
UY	0%	0%	74.6%
RZ	0%	74.9%	0%
Base shear [kN]	977kN for MRF (in X-direction) 1581kN for CBF (in Y-direction)		

UX: Participation of the modal mass to translational movement in X-direction
 UY: Participation of the modal mass to translational movement in Y-direction
 RZ: Participation of the modal mass to rotational movement in Z (vertical) axis

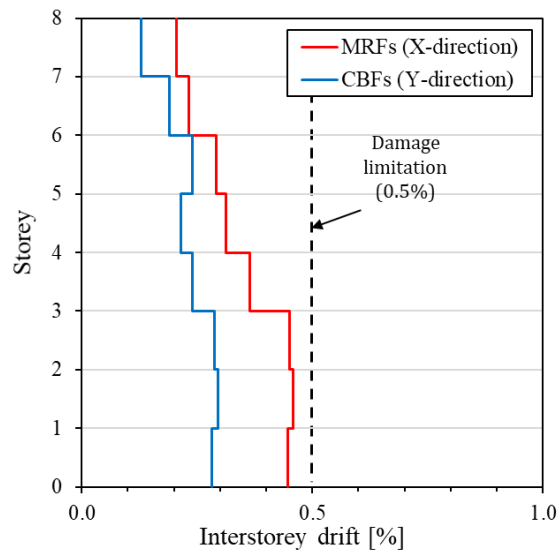


Figure 6.30: Inter-storey drift in X- and Y-directions of 8 storey mid-rise modular building under seismic design load of ULS combination ($1.0G_k+0.3Q_k+1.0E_x$)

6.4.2. Description of the Advanced Frame Model with AJ Connections

The MRF composed of the shorter side of modules in the prototype structure was analysed for wind and seismic analysis (refer to Figure 6.28 (a) for the location of the MRF). Using Abaqus/Static, the 2D MRF model with the simplified joint model (refer to Figure 6.12) was created, as shown in Figure 6.31. The demand preload for the connections was calculated based on the internal force distribution obtained from the ETABS model, using the design method suggested in Sections 3.4.3, 3.4.4, 3.4.5, and 3.6.4.2. The performance of the MRF with AJ connections was evaluated by comparing the response to that of the frame with welded connections. In the frame analysis, P-DF was set as the reference model (see Table 6.13).

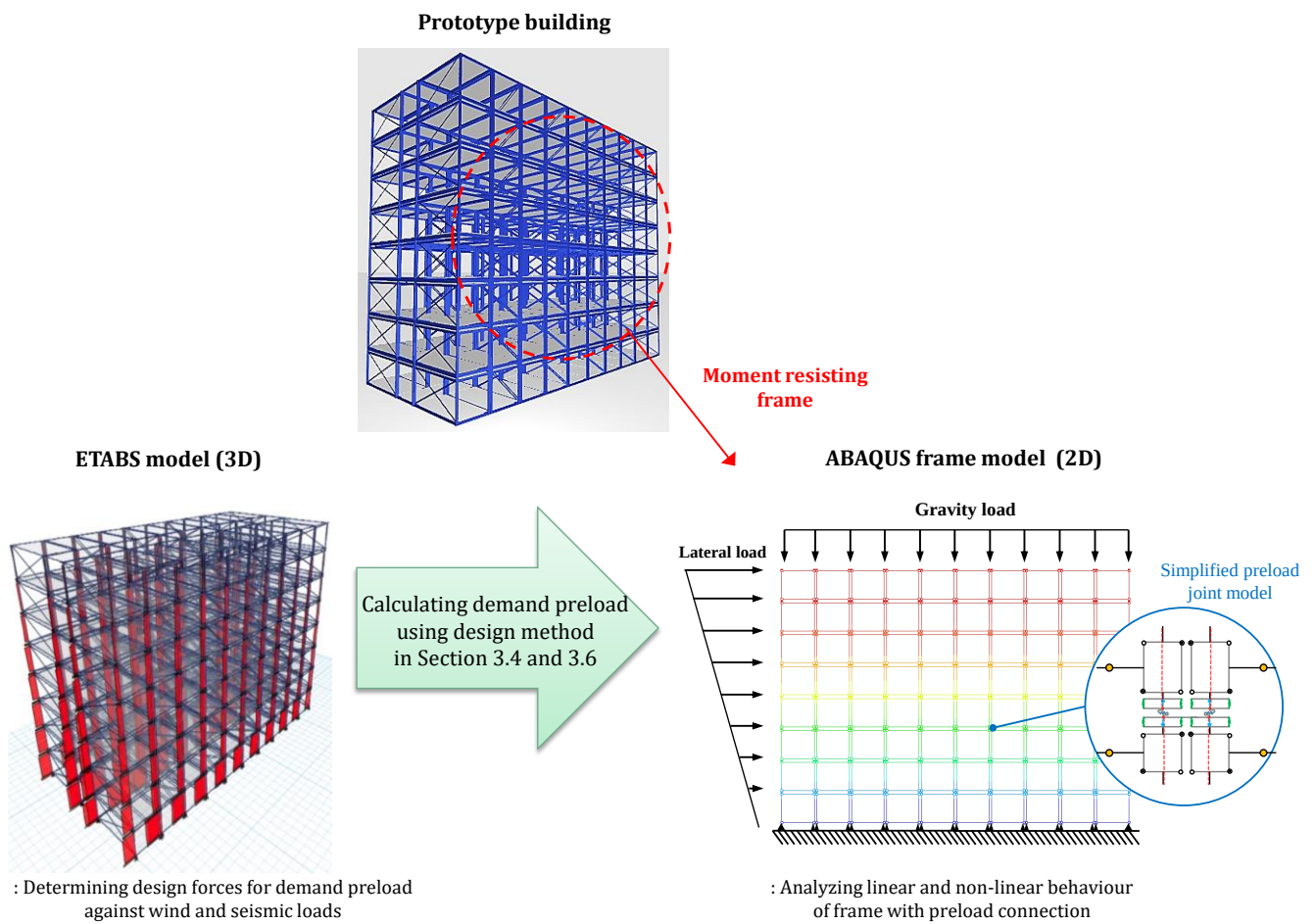


Figure 6.31: Advanced Abaqus 2D MRF model with the simplified AJ joint model

Table 6.13: Reference frame model for frame analysis

Frame	Type of connection (column-to-column)	Column $[d \times b \times t]$	Floor beam $[d \times b \times t]$	Ceiling beam $[d \times b \times t]$	Gap-length between modules [mm]	Slab type & thickness [mm]
PF-D	Preload	200X100X12	200X100X4	150X100X4	20	Concrete 150

6.4.3. Behaviour of the Frame with AJ Connections Against Wind

The MRF composed of modules with AJ connections was investigated for the level of wind load in Sydney and Brisbane, Australia. According to Australian Wind code AS 1170.2 [149], the static wind loads for the ultimate limit state were calculated using a basic wind speed (a 1000-year return period) of 46m/s for Sydney and 60m/s for Brisbane, and the load combination of 0.9G+1.0W [150] was applied to the frame as shown in Figure 6.32.

The self-weight of a module affects the demand preload for the AJ connection. The heavier self-weight of a module allows for reducing the demand preload required for preventing gap-opening and slip at the connection. Generally, a module with a concrete slab is much heavier than a module with a cement board slab because of the larger density and thickness. So, two types of floor slabs (i.e. concrete and cement board slab) in a module were considered in the simulation. The thicknesses of the concrete slab and the cement board slab were assumed to be 150 mm and 18mm, respectively. Table 6.14 shows details of the frame models analysed.

Table 6.14: Details of the moment-resisting frame models for wind analysis

Frame	Type of connection (column-to-column)	Module unit (6.6m X 3.3m X 3m)			
		Type of slab	Thickness [mm]	Density [kg/m ³]	Weight [kN]
PF-D*	Preload	Concrete	150	2400	112
PF-D-CB	Preload	Cement board	18	1200	45
WF-D	Welded	Concrete	150	2400	112
WF-D-CB	Welded	Cement board	18	1200	45

* Reference model

6.4.3.1. Determination of Demand Preload for AJ Connections

Calculating the demand preload for all AJ connections in a frame is inefficient and time-consuming. Therefore, the demand preload was determined from modules in which the most critical connections (i.e. columns subjected to the largest tensile or shear forces) were located. Figure 6.32 shows the frame subjected to gravity and wind loads with resulting axial reaction forces. The exterior modules (MC-1) were expected to feature the critical connections for gap-opening and slip, so the demand preload was determined from AJ connections in MC-1. Table 6.15 shows the demand preload for preventing gap-opening (P_g) and slip (P_s) at all AJ connections in MC-1 for wind loads in Sydney and Brisbane. For the mid-rise modular building, if slip occurred at the column-to-column connection, it was expected that the top storey would have a large displacement from the accumulated slip according to the height of a building. Therefore, the AJ connection needed to be designed as the slip-resistant connection. The

demand preload for the slip-resistant connection ($P_{s,Ed}$) was determined as the greater of P_g and P_s . As a conservative design approach, the largest demand preload of $P_{s,Ed}$ among all the connections (C1-C7) in Table 6.15 was taken as the demand preload for all the connections in the frame.

In the case of P-FD having the concrete slab, $P_{s,Ed}$ was zero for Sydney wind load because both gap-opening and slip were expected to be prevented by the self-weight of modules, while a preload of 16kN was required for preventing slip at the highest storey for Brisbane wind load. Regarding PF-D-CB having the cement board slab, $P_{s,Ed}$ was 16kN and 54kN for the wind loads in Sydney and Brisbane, respectively.

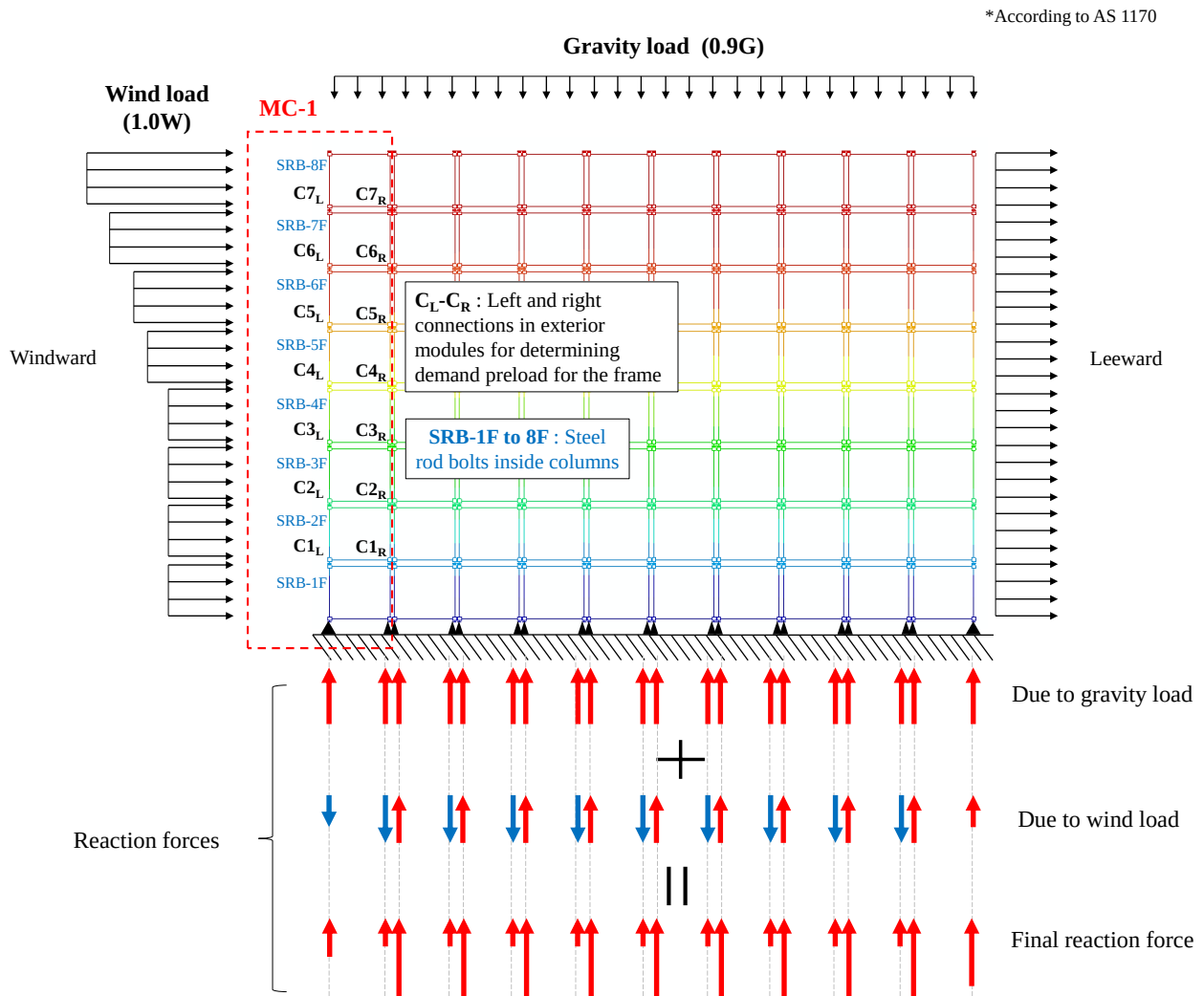


Figure 6.32: External loads (wind and gravity) and the corresponding shape of axial reaction forces in the 8-storey moment resisting frame

Table 6.15: Demand preload for a slip-resistant connection ($P_{S,Ed}$) determined in critical modules (MC-1) for wind load [kN]

Connection (Refer to Figure 6.32 for location)	Sydney (46m/s)				Brisbane (60m/s)			
	PF-D		PF-D-CB		PF-D		PF-D-CB	
	P_g	P_s	P_g	P_s	P_g	P_s	P_g	P_s
C7 _L (C7 _R)	0 (0)	0 (0)	6 (0)	16 (9)	0 (0)	16 (1)	8 (0)	31 (18)
C6 _L (C6 _R)	0 (0)	0 (0)	1 (0)	5 (0)	0 (0)	0 (0)	6 (0)	16 (0)
C5 _L (C5 _R)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	9 (0)	12 (0)
C4 _L (C4 _R)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	6 (0)	9 (0)
C3 _L (C3 _R)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	6 (0)	9 (0)
C2 _L (C2 _R)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	3 (0)	9 (0)
C1 _L (C1 _R)	0 (0)	0 (0)	0 (10)	0 (0)	0 (0)	0 (0)	0 (54)	18 (23)
Demand preload for a slip-resistant connection ($P_{S,Ed}$)	Not needed		16 kN		16 kN		54 kN	

Note: P_g and P_s are demand preload for preventing gap-opening and slip determined by (Eq. 3.29) and (Eq. 3.42), respectively.

Friction coefficient of 0.3 is used for P_s assuming the connection has class C of friction surfaces according to EN 1993-1-8 [85].

To maintain a preload (i.e. clamping force) of $P_{S,Ed}$ at the connections after stacking the upper modules on the lower modules on-site, the amount of reduced preload (P_R) (see Table 6.16) due to the weight effect of upper modules (refer to Section 3.4.5) should be added to $P_{S,Ed}$. The final demand preload ($P_{F,S,Ed}$) is shown in Table 6.17, which was applied to the SRBs in the FE models. The heavier the module is, the greater the reduction in the induced preload, particularly for the lower storeys. For instance, although $P_{S,Ed}$ for PF-D-CB (Sydney) and PF-D (Brisbane) have the same preload of 16kN, the final demand preload $P_{F,S,Ed}$ for PF-D (Brisbane) is greater as shown in Table 6.17.

Table 6.16: Reduced preload due to weight of upper modules (P_R)* [kN]

Location of SRB (Refer to Figure 6.32)	Diameter and the number per column	PF-D	PF-D-CB
SRB-8F	D25 X 1	0	0
SRB-7F	D25 X 1	3.4	1.1
SRB-6F	D25 X 1	5.3	1.8
SRB-5F	D25 X 1	7.2	2.6
SRB-4F	D25 X 1	9.2	3.4
SRB-3F	D25 X 1	11.1	4.1
SRB-2F	D25 X 1	13.0	4.9
SRB-1F	D25 X 1	14.9	5.7

Note: P_R is determined by (Eq. 3.53)

Table 6.17: Final induced preload to steel rod bolts for the slip-resistant connection ($P_{F,S,Ed} = P_{S,Ed} + P_R$) [kN]

Location of SRB (Refer to Figure 6.32)	Diameter and the number per column	Sydney (46m/s)		Brisbane (60m/s)	
		PF-D	PF-D-CB	PF-D	PF-D-CB
SRB-8F	D25 X 1	(No preload needed)	16	16	54
SRB-7F	D25 X 1		17	19	55
SRB-6F	D25 X 1		18	21	56
SRB-5F	D25 X 1		19	23	57
SRB-4F	D25 X 1		19	25	57
SRB-3F	D25 X 1		20	27	58
SRB-2F	D25 X 1		21	29	59
SRB-1F	D25 X 1		22	31	60

6.4.3.2. FE Simulation Results

(1) Frame behaviour without preload

First, frames were simulated without inducing preload in order to investigate the impact of gap-opening and slip at column-to-column connections on the behaviour of the frame, the impact of their location in the frame, and the effect of the self-weight of modules. In other words, the modules were connected by only SRBs, and clamping forces at the connections between modules were dependent on only the self-weight of modules.

Figure 6.33 shows the storey displacements of the frames. In the case of PF-D (see Figure 6.33 (a)), gap-opening and slip were completely prevented by the self-weight of modules for Sydney wind load as expected, while a small slip displacement occurred at the top storey for Brisbane wind load. Overall, PF-D having a concrete slab showed almost the same behaviour as WF-D against both wind loads. On the other hand, PF-D-CB experienced both gap-opening and slip at the connections (see Figure 6.33 (b)). The impact of slip at the connections was very noticeable. Particularly for the Brisbane wind load, the slip displacement occurring at the top two storeys was as large as the clearances of a coupler hole. The amount of slip displacement increased according to the storey height since the wind load was larger while the effect of the self-weight of modules was smaller. As a result, the top displacement increased significantly due to the slip accumulated from each storey. Conversely, gap-opening occurred only at the connections located at the lowest storey. Figure 6.34 (a) and (b) show the locations of gap-opening in PF-D-CB and its amount of 0.2mm. It caused the SRBs at the 1st storey to be subjected to a minor additional tensile force of 2kN. Figure 6.34 (c) shows the variation in clamping forces where the gap-opening occurred at the connections. The initial clamping forces obtained from the weight of modules, were reduced gradually due to the wind load, and the gap-opening began to occur at the top displacement of 19mm.

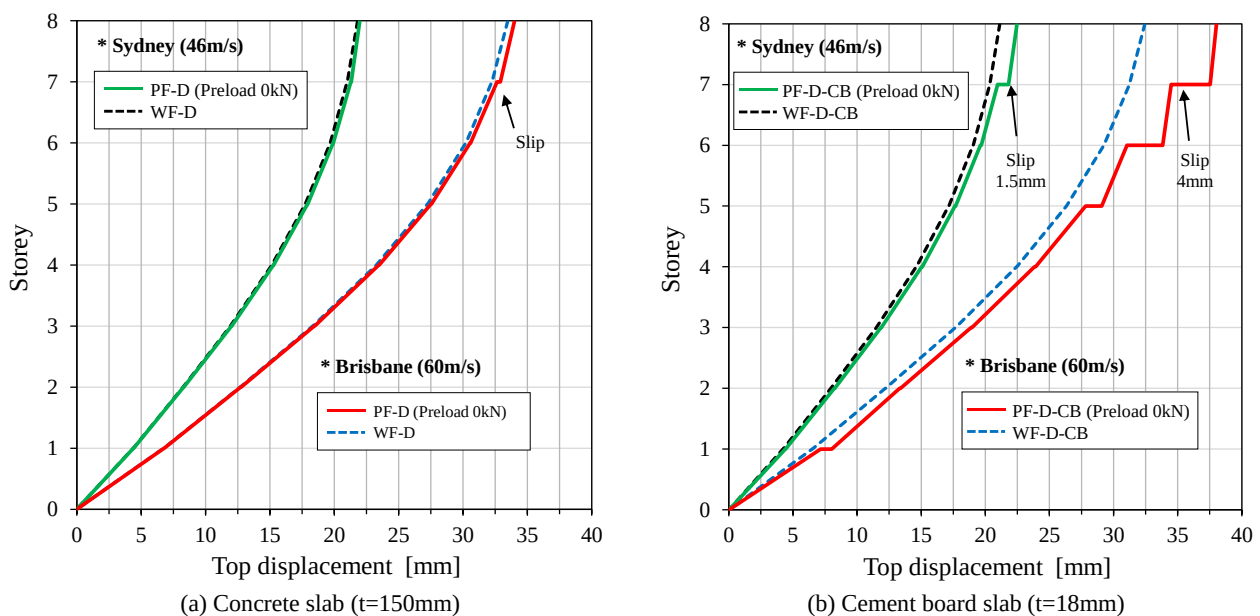


Figure 6.33: Storey displacement of the PF-D and PF-D-CB frames (No preload)

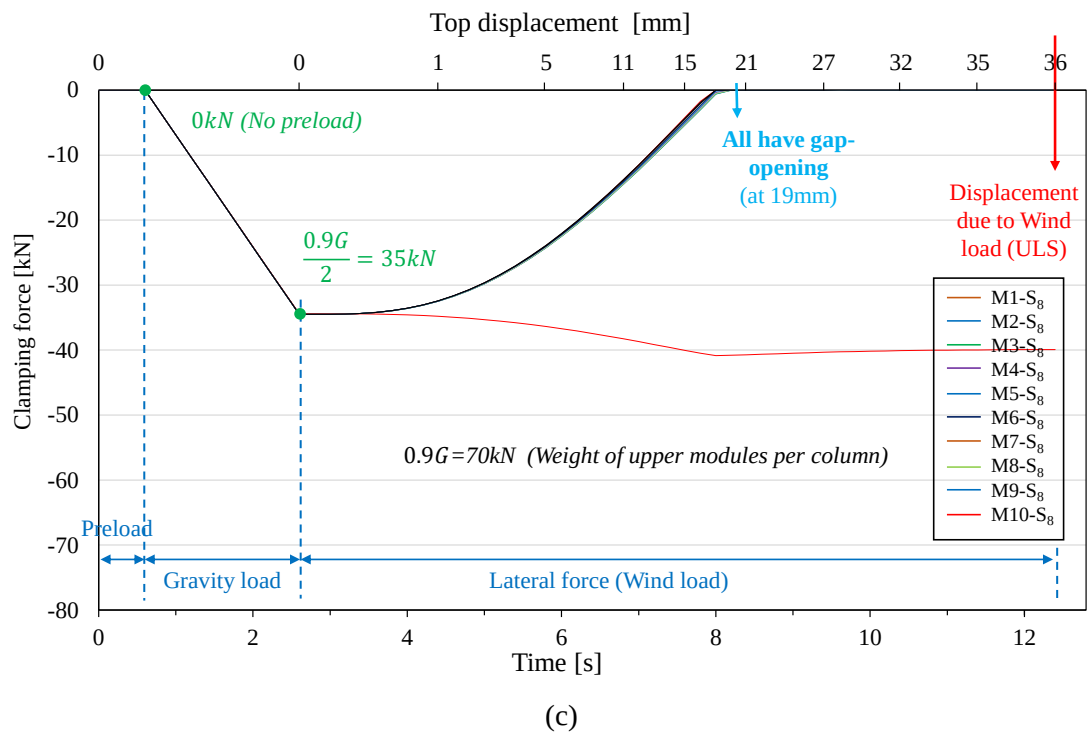
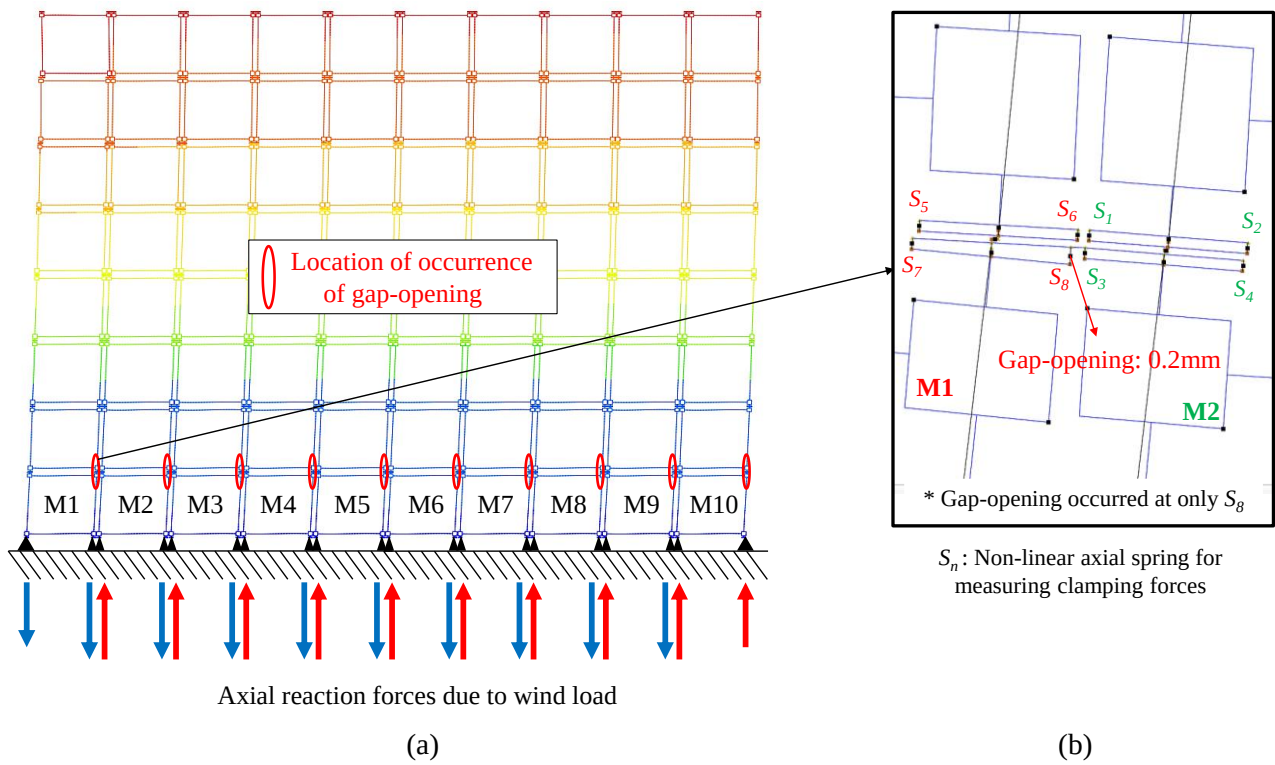


Figure 6.34: PF-D-CB (Preload=0kN) under Brisbane wind load: (a) and (b) Location of gap-opening, and (c) Variation in clamping force of spring (S_8) in which gap-opening occurred

To separately investigate the impact of gap-opening and slip on the lateral displacement of the frame, the PF-D-CB model was re-simulated by restricting slip or gap-opening at the connections. The simulation results are shown in Figure 6.35. The amount of gap-opening of 0.2mm from the lowest storey caused 1.3mm of top displacement. It is a minor displacement compared to the slip displacement, but when proportional to the amount of the gap-opening, the increased top displacement is not a small amount. If the building is higher than this analysed frame, it is expected to be larger.

The locations of connections having gap-opening and slip in the frame almost coincided with those at which preload was required by the analytical approach, as shown in Table 6.15.

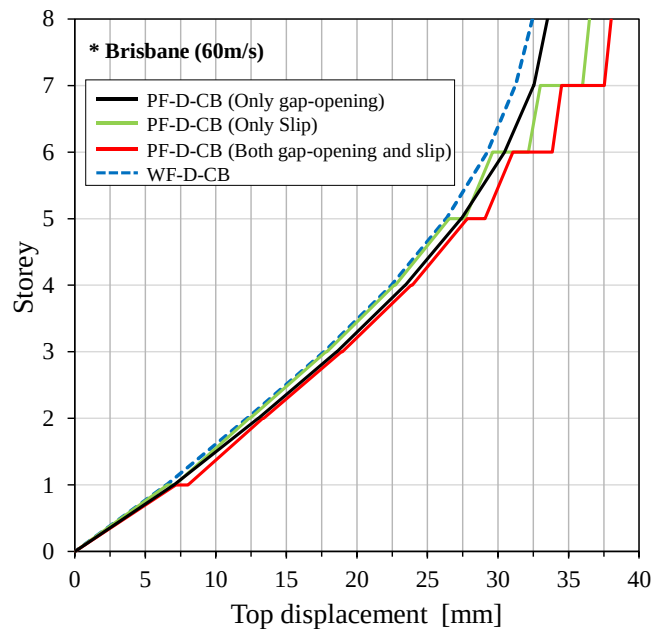


Figure 6.35: Impact of gap-opening and slip on the 8-storey MRF: Storey displacement of PF-D-CB (Preload =0kN)

(2) Frame behaviour with induced demand preload

Figure 6.36 shows the storey displacements of the frame in which the demand preload for the slip-resistant connection ($P_{F,S,Ed}$) was induced. The simulation results showed that slip and gap-opening were completely prevented in PF-D and PF-D-CB for both Sydney and Brisbane wind loads, so they had almost the equivalent behaviour to WF-D and WF-D-CB, respectively.

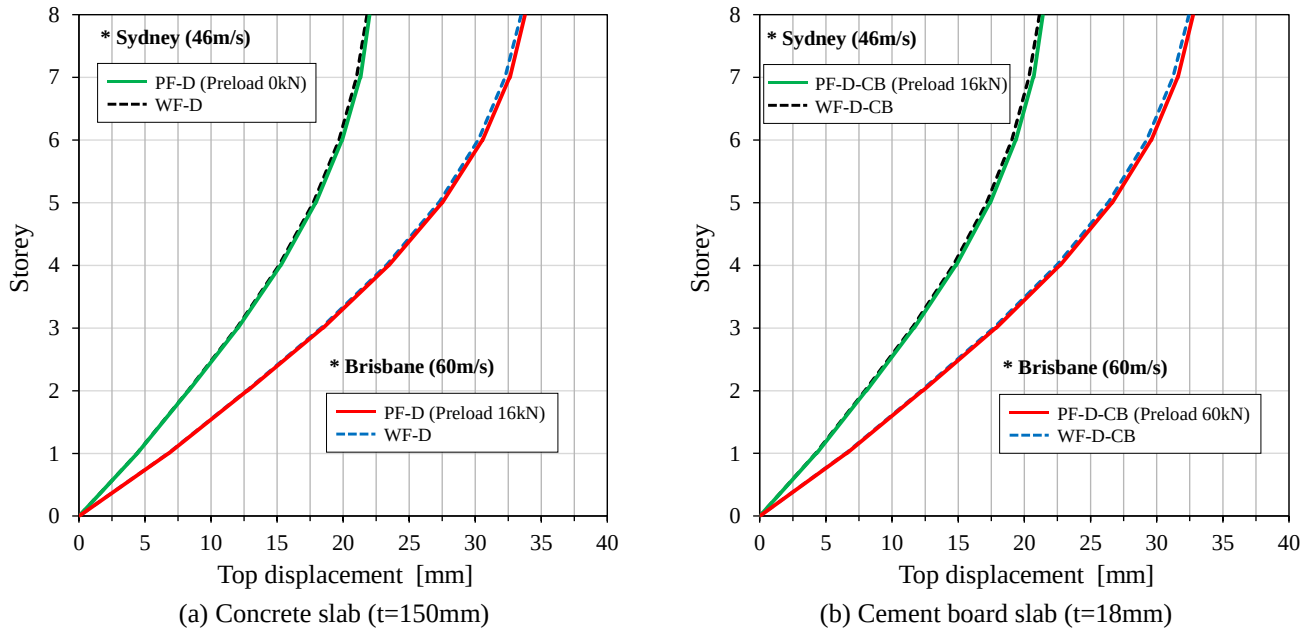
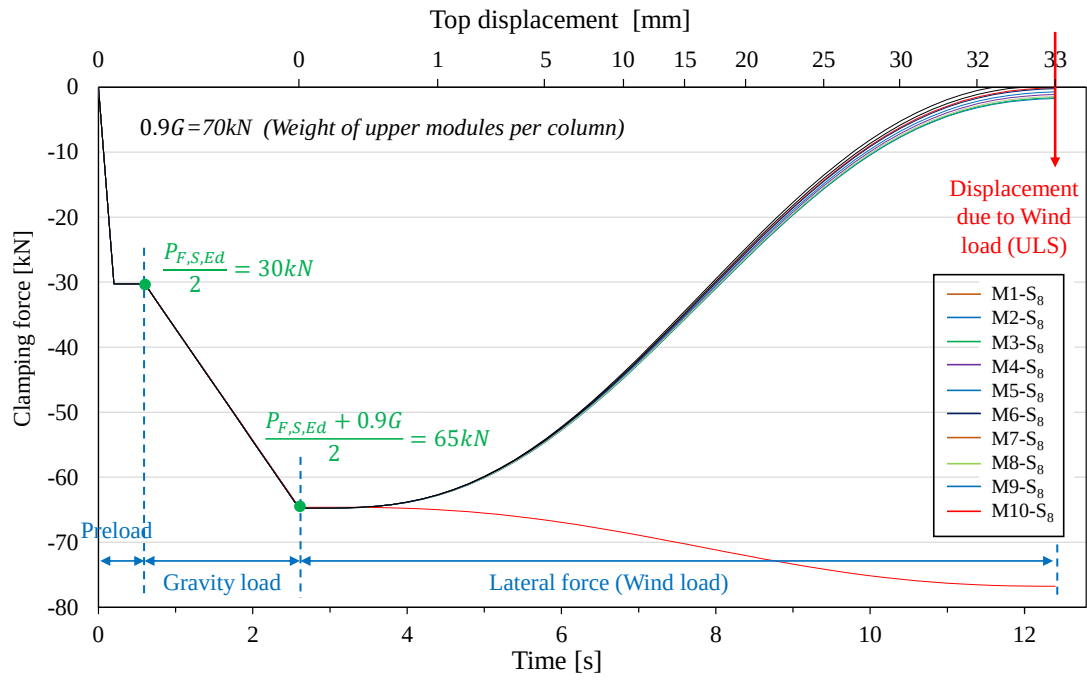


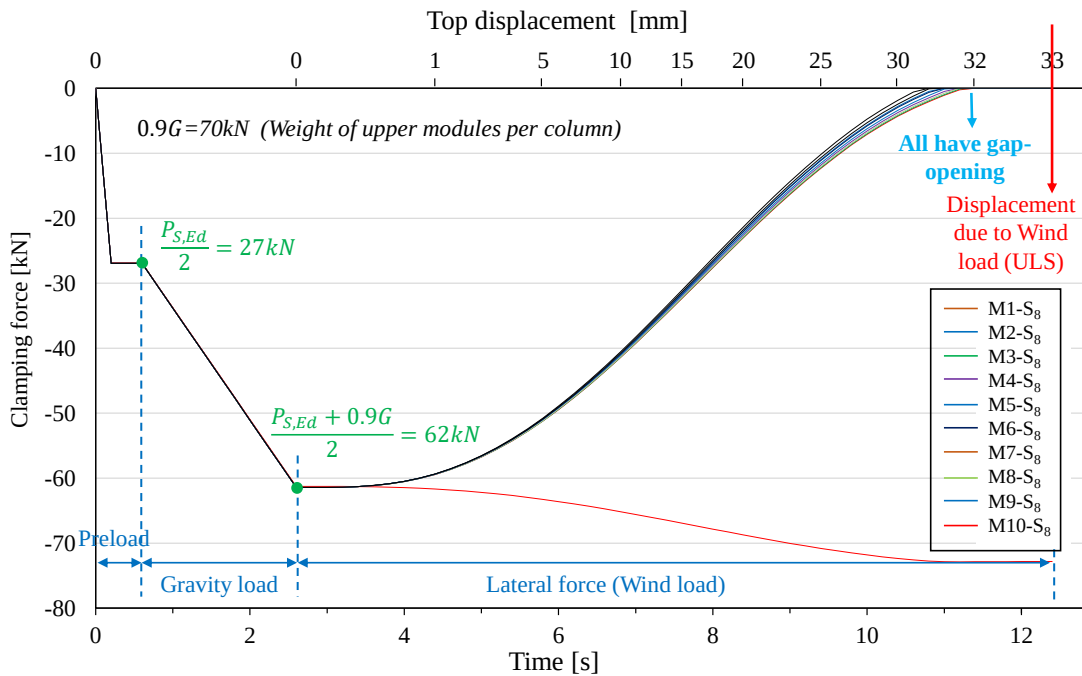
Figure 6.36: Storey displacement of frames with AJ connections with preload $P_{F,slip}$

(3) Validation of demand preload considering the weight effect of upper modules

Additionally, the effectiveness of demand preload considering the weight effect of upper modules was confirmed. Using the PF-D-CB model, two cases were simulated as follows: (1) Inducing demand preload considering the weight effect $P_{F,S,Ed} (=P_{S,Ed} + P_R)$, and (2) Inducing demand preload without considering the weight effect $P_{S,Ed}$. Figure 6.37 shows the variation in clamping forces of the connections at the lowest storey. In the former case, the connections maintained the clamping forces until the top displacement reached the value corresponding to the design wind load (ULS). On the other hand, in the latter case, the connections lost clamping forces at a slightly earlier displacement, which meant $P_{S,Ed}$ could not prevent gap-opening for the design load due to reduced preload from the weight of upper modules. Consequently, the FE simulation shows that $P_{F,S,Ed}$ could accurately provide the required amount of preload for design forces.



(a) Preload ($P_{F,S,Ed}$) = 60 kN (Considering the weight effect of upper modules)



(b) Preload ($P_{S,Ed}$) = 54 kN (Not considering the weight effect of upper modules)

Figure 6.37: Variation in clamping force (springs, S₈) of AJ connections at the lowest storey in PF-D-CB (refer to Figure 6.34 (a) and (b) for the location of springs)

In summary, the FE simulation confirmed that the AJ connection could be realistically applied to the mid-rise modular building against wind load. The amount of required preload is not large because the self-weight of modules contributes to preventing the gap-opening and slip at the connection. For wind design, slip from the column-to-column connections is more critical than gap-opening. Therefore, the AJ connection should be designed as a slip-resistant connection for the mid-rise building. The simulation showed that the frame with AJ connections had equivalent performance to the frame with rigid connections. Therefore, it is possible to simply model AJ connections as rigid connections for structural analysis provided the demand preload $P_{F,S,Ed}$ is induced. The FE simulation also confirmed the induced preload suggested in this study could accurately provide the necessary amount of preload for preventing gap-opening and slip at AJ connections at the frame.

6.4.4. Behaviour of the Frame with AJ Connections Against Earthquakes

The current seismic design in EN 1998-1 [86] is force-based design, whereby a structure is designed by dividing the seismic design load by the behaviour factor (q), assuming it has a certain amount of ductility depending on the structural type. However, it is not guaranteed that the structure actually has the assumed ductility when the earthquake corresponding to the design load occurs. Therefore, in this section, the MRF with AJ connections, which was designed according to EN 1998-1 with $q=3.5$, is first investigated for its nonlinear behaviour using pushover analysis. Subsequently, its seismic performance for design seismic load (ULS) is evaluated using the N2 method [141].

6.4.4.1. Pushover Analysis

Table 6.18 shows details of the frame models analysed. The PF-D/S frames were created to investigate the effect of a difference in the cross-sectional size of floor and ceiling beams on the demand preload and frame behaviour. Their performances were evaluated by comparing to the performance of WF-D/S, which had rigid connections. The pushover analysis was conducted by applying displacement-controlled lateral loading until the frame reached a complete loss of resistance. The P-Delta effect was considered in the analysis. The lateral load was applied at the centroid of floor beams where concrete slabs are located. The lateral storey forces were assumed to be proportional to the fundamental mode-shape, as shown in Figure 6.38.

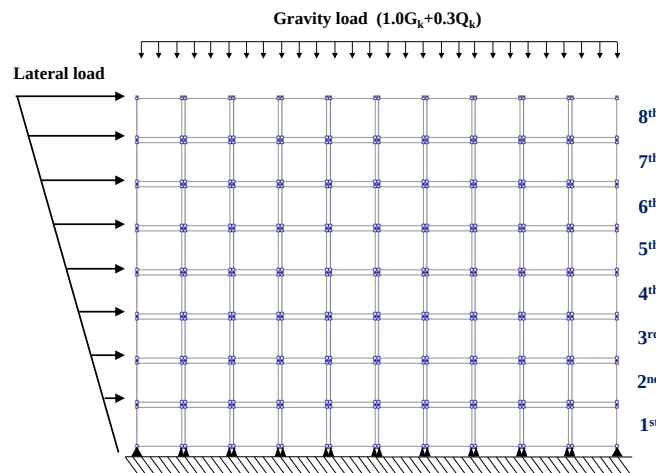


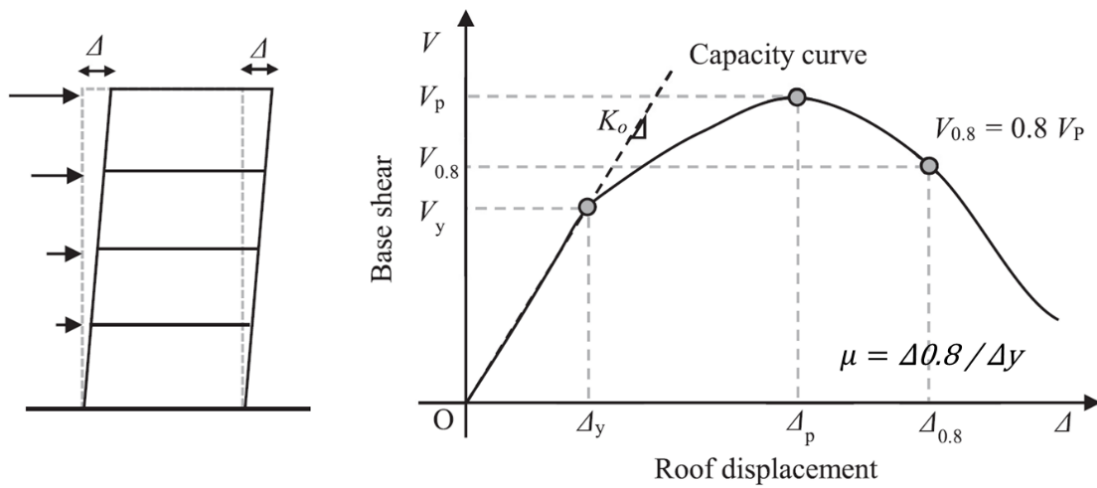
Figure 6.38: Applied forces on an 8-storey moment resisting frame in a modular building for pushover analysis

Table 6.18: Details of the moment-resisting frame models for pushover analysis

Frame	Type of connection (column-to-column)	Column [$d \times b \times t$]	Floor beam [$d \times b \times t$]	Ceiling beam [$d \times b \times t$]	Slab type & thickness [mm]
PF-D*	Preload	200X100X12	200X100X4	150X100X4	Concrete 150
PF-S	Preload	200X100X12	200X100X4	200X100X4	Concrete 150
WF-D	Welded	200X100X12	200X100X4	150X100X4	Concrete 150
WF-S	Welded	200X100X12	200X100X4	200X100X4	Concrete 150

* Reference model

The pushover analysis results were used to determine the frame ductility ($\mu = \Delta_{0.8}/\Delta_y$), as demonstrated in Figure 6.39 [62]. However, in the case of the frame with AJ connections, the displacement for evaluating μ is defined as the smaller of $\Delta_{0.8}$ and $\Delta_{y,SRB}$, where $\Delta_{y,SRB}$ is the displacement at which SRBs yield. This is because, even if the SRBs are partially yielded, the induced preload would be reduced continually under cyclic loading, which causes the effect of gap-opening at the connection on the frame behaviour to increase continually as well. In addition, since the difference between the yield and ultimate strength of the SRB made of high-strength steel (refer to Figure 4.14) is small, there is a risk that the SRB might undergo rupture if stresses close to the yield load. Therefore, it is reasonable to regard the yield point of the SRBs as the maximum ductility displacement of the frame.



- V_y and Δ_y : The force capacity corresponding to the first yield, which is defined as the point where the capacity curve leaves the line of elastic stiffness of the structure, K_o and the corresponding roof displacement, respectively;
- V_p and Δ_p : The force capacity corresponds to the peak point and the corresponding roof displacement, respectively;
- $V_{0.8}$ and $\Delta_{0.8}$: The load of $0.8V_p$ on the descending branch and the corresponding roof displacement, respectively;
- μ : The frame ductility.

Figure 6.39: Typical capacity curve for frames subjected to lateral load [62]

(1) Determination of demand preload for AJ connections

The demand preload was determined based on the internal force distribution in the prototype structure obtained from the response spectrum analysis (RSA) in Section 6.4.1. The most critical connections for gap-opening and slip were located in the exterior modules (see MC-1 in Figure 6.40). The demand preload was calculated based on the concept of capacity design described in Section 3.6.4.2. The amplification factor α_B (Eq. 3.98) was based on the first yielding of a beam, as shown in Figure 6.40, since otherwise, the demand preload would become significantly large. In this case, the value of α_B becomes the same as the magnification coefficient Ω for the non-dissipative member design in the seismic design code EN 1998-1 [86]. Then, α_B was used to determine the design seismic load for calculating the demand preload for the AJ connection as follows:

$$1.0 \cdot G_k + 0.3 \cdot Q_k + 1.1 \cdot r_{ov} \cdot \alpha_B \cdot 1.0 \cdot E_x \quad (\text{Eq. 6.17})$$

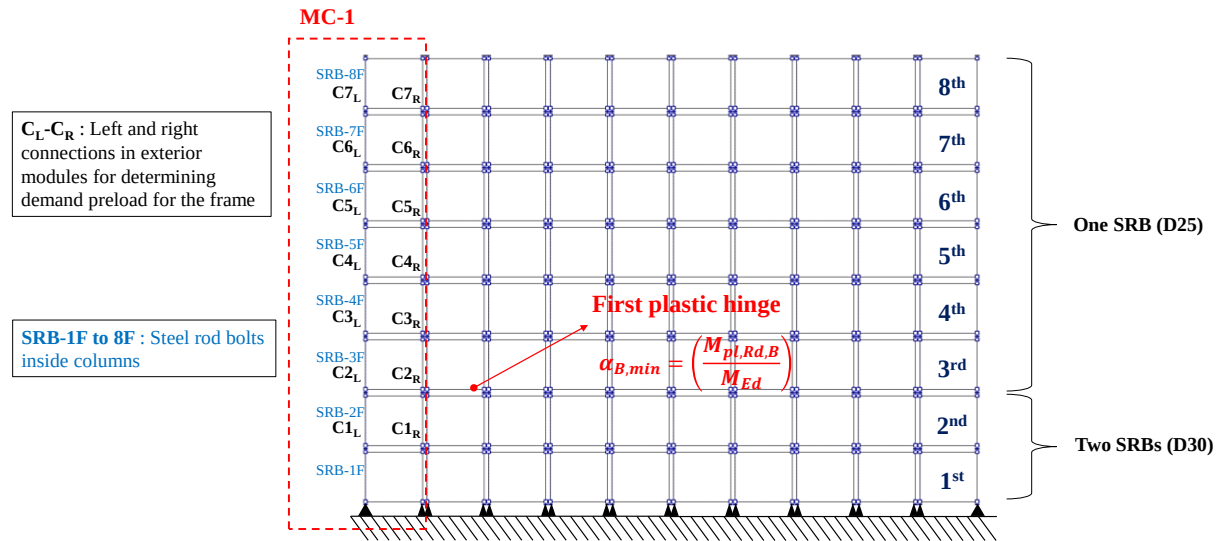


Figure 6.40: Location of critical connections and determining the amplification factor α_B in an 8-storey moment resisting frame

Table 6.19 shows the required preload for preventing gap-opening (P_g) and slip (P_s) of the connections and the demand preload for a slip-resistant connection ($P_{S,Ed}$). Regarding the demand preload P_g , PF-S required approximately 30-70% less preload than PF-D. Overall, a significantly large amount of preload was needed for the connections located at the low storeys, so the preload was applied to the frame by dividing it into 2 or 3 zones, as shown in Figure 6.40. For the connections between the 1st and 2nd storeys, two SRBs (D30) were required, while the connections between the 2nd and 8th storeys needed one SRB (D25). Table 6.20 and Table 6.21 shows the reduced preload (P_R) due to the weight of upper modules and the final induced preload ($P_{F,S,Ed}$), respectively.

Table 6.19: Demand preload for a slip-resistant connection ($P_{S,Ed}$) determined in critical modules (MC-1) for seismic load [kN]

Connection (Refer to Figure 6.32 for location)	PF-D		PF-S	
	P_g	P_s	P_g	P_s
C7L (C7R)	151 (153)	38 (83)	66 (93)	40 (84)
C6L (C6R)	116 (186)	50 (87)	36 (98)	49 (84)
C5L (C5R)	140 (190)	75 (79)	43 (86)	80 (77)
C4L (C4R)	172 (183)	103 (60)	64 (60)	116 (62)
C3L (C3R)	215 (173)	134 (30)	92 (32)	156 (39)
C2L (C2R)	218 (218)	169 (36)	121 (70)	201 (46)
C1L (C1R)	387 (698)	228 (467)	264 (494)	272 (442)
Demand preload for a slip-resistant connection ($P_{S,Ed}$)	218 kN (for C2 – C7) 633 kN (for C1)		156 kN (for C3 – C7) 201 kN (for C2) 494 kN (for C1)	

Note: P_g and P_s are demand preload for preventing gap-opening and slip determined by (Eq. 3.29) and (Eq. 3.42), respectively. Friction coefficient of 0.5 is used for P_s assuming the connection has class A of friction surfaces according to EN 1993-1-8 [85].

Table 6.20: Reduced preload due to weight of upper modules (P_R) [kN]

Steel rod bolts (Refer Figure 6.40)	Diameter and the number per column	PF-D	PF-S
SRB-8F	D25 X 1	0	0
SRB-7F	D25 X 1	4.2	4.2
SRB-6F	D25 X 1	6.6	6.6
SRB-5F	D25 X 1	9.0	9.0
SRB-4F	D25 X 1	11.4	11.4
SRB-3F	D25 X 1	13.8	13.8
SRB-2F	D30 X 2	46.4	46.4
SRB-1F	D30 X 2	53.3	53.3

Note: P_R is determined by (Eq. 3.53).

Table 6.21: Final induced preload to steel rod bolts for the slip-resistant connection ($P_{F,S,Ed} = P_{S,Ed} + P_R$) [kN]

Steel rod bolts (Refer Figure 6.40)	Diameter and the number per column	PF-D	PF-S
SRB-8F	D25 X 1	232	167
SRB-7F	D25 X 1	232	167
SRB-6F	D25 X 1	232	167
SRB-5F	D25 X 1	232	167
SRB-4F	D25 X 1	232	167
SRB-3F	D25 X 1	232	215
SRB-2F	D30 X 2	751 (375 per SRB)	548 (274 per SRB)
SRB-1F	D30 X 2	751 (375 per SRB)	548 (274 per SRB)

(2) Nonlinear behaviour and failure mechanism of the AJ connection

Figure 6.41 and Table 6.22 show the capacity curves and the summarised key features. Overall, both WF-D/S and PF-D/S featured the strong column-weak beam failure mechanism. All frames formed plastic hinges in beams evenly throughout the frames, while the columns remained in the elastic range. PF-D/S had the same stiffness as WF-D/S in the elastic range, but after gap-opening developed at the AJ connections between the 1st and 2nd storeys (see Figure 6.43), the lateral stiffness deteriorated, resulting in lower peak strength and ductility compared to WF-D/S. Figure 6.42 (b) presents the distribution of plastic hinges in PF-D shortly after the gap-opening occurred. It shows that only ceiling beams formed plastic hinges, but floor beams could not form the hinges near the connections having the gap-opening. Although in the end, a floor beam and other beams in the frame formed hinges while the lateral load was being resisted by the SRBs-1F, the lateral stiffness of the frame decreased and the SRBs-1F yielded.

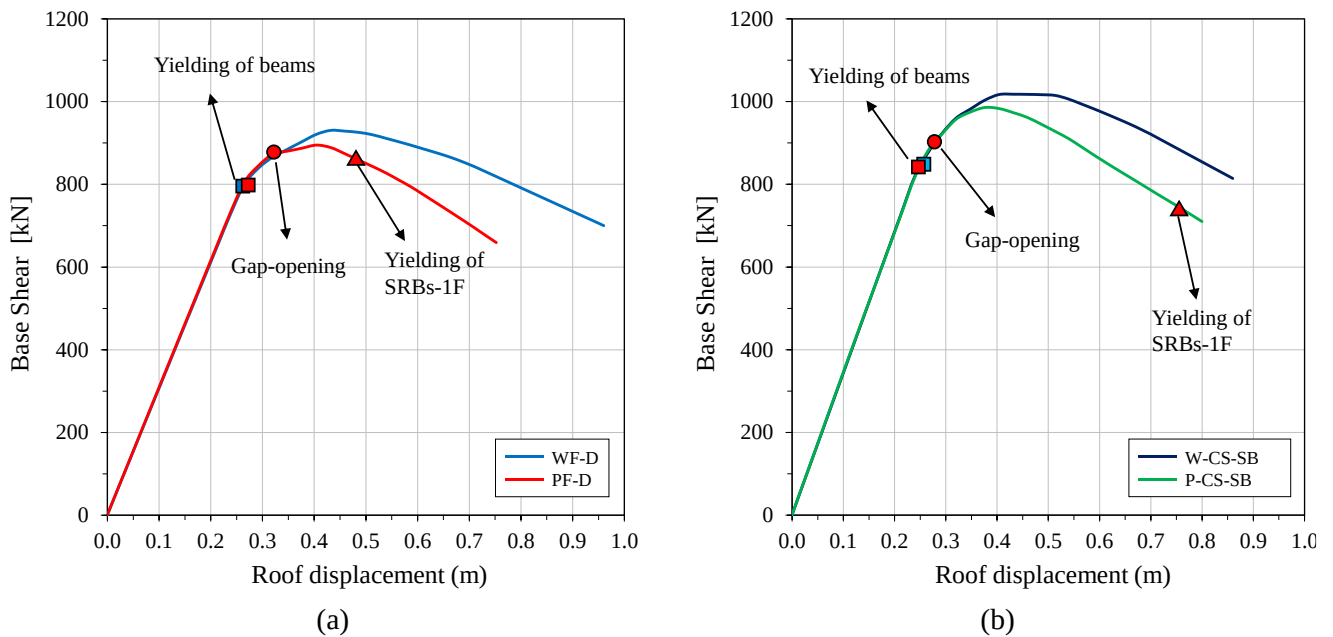


Figure 6.41: Capacity curves from pushover analysis of the frames: (a) PF-D and WF-D, and (b) PF-S and WF-S

Table 6.22: Summary of the pushover analysis results (refer to Figure 6.39 for denomination)

Frame	Δ_y [mm]	V_y [kN]	Δ_p [mm]	V_p [kN]	$\Delta_{0.8}$ [mm]	$V_{0.8}$ [kN]	Δ_g [mm]	V_g [kN]	$\Delta_{SRB,y}$ [mm]	$V_{SRB,y}$ [kN]	μ -
WF-D	265	800	432	930	880	744	N/A	N/A	N/A	N/A	3.3
PF-D	263	802	404	895	682	716	318	873	487	858	1.9
WF-S	253	853	432	1018	860	814	N/A	N/A	N/A	N/A	3.4
PF-S	250	852	376	986	697	789	750	882	750	762	2.8

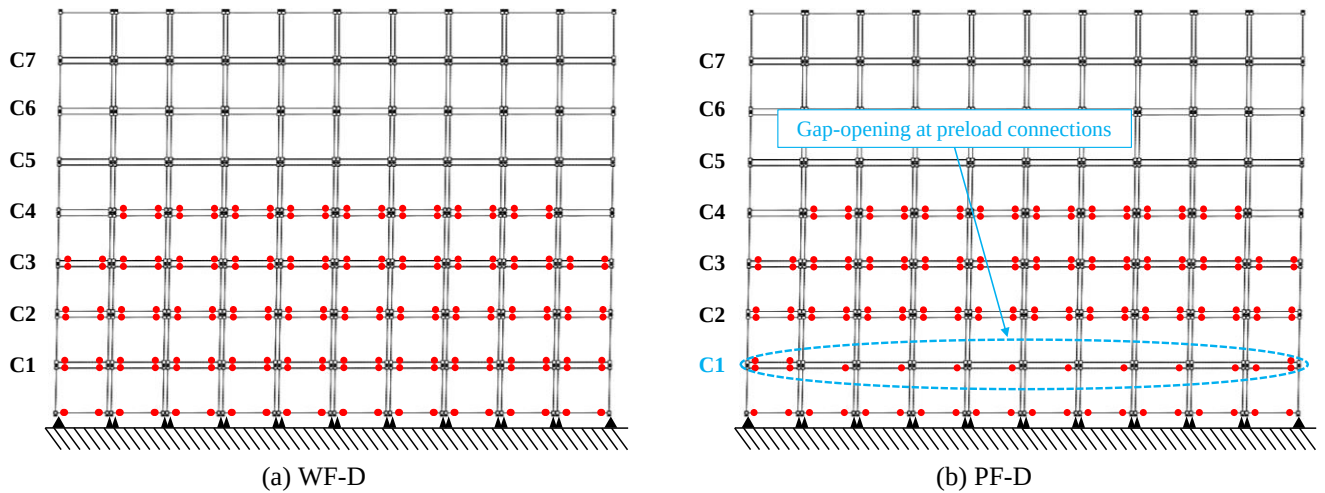


Figure 6.42: Distribution of plastic hinges (at the roof displacement of 350mm)

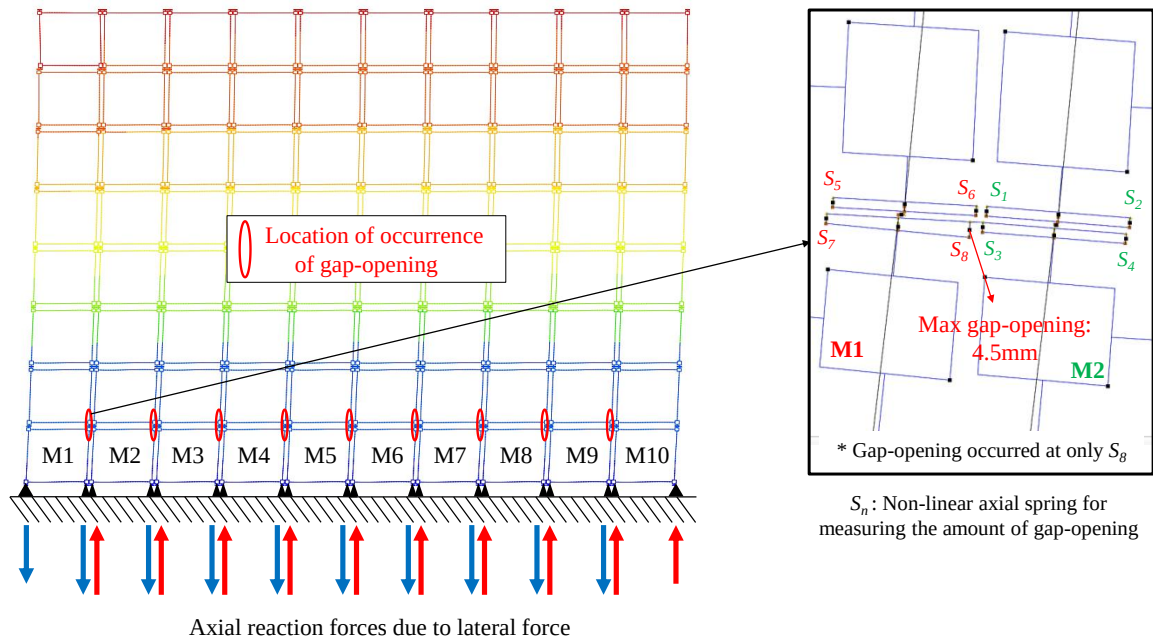


Figure 6.43: Location of gap-opening in PF-D

(2) Effect of the difference in the cross-section size of floor and ceiling beams

The pushover analysis results confirmed that PF-S having the same size of floor and ceiling beams was less impacted by gap-opening than PF-D. First, although the smaller amount of preload was induced to PF-S, the timing of the gap-opening was similar to PF-D. Even though the gap-opening occurred, PF-S did not lose the lateral stiffness immediately unlike PF-D (see Figure 6.41 (a) and (b)), resulting in the larger peak strength. Additionally, PF-S had greater ductility because the SRBs yielded much later than in the case of PF-D. Hence, the impact of gap-opening was less, and the SRBs had greater capacity to resist the tensile force due to a smaller initially induced preload than the SRBs of PF-D. It can be concluded that for seismic design, the AJ connection of PF-S is more favourable than that of PF-D.

(3) Efficient application of the AJ connection for seismic design

PF-D/S experienced gap-opening at the connections in the lowest storey immediately after the commencement of nonlinear material behaviour (yielding), which reduced the remaining strength of the frame. Generally, even if several beams form plastic hinges, the strength of the frame increases until plastic hinges are evenly formed in other beams. Therefore, in order to secure stable ductile behaviour without gap-opening at AJ connections, it is necessary to apply a larger value of amplification factor α_B in order to prevent or delay the initiation of gap-opening.

However, the large value of α_B might cause a significant increase in demand preload, which practically could be difficult to achieve. In fact, despite using the minimum α_B , the induced preload for the lower storeys in PF-D/S (i.e. 375kN and 274kN per SRB) would be difficult to apply in practice. In addition, the large demand preload becomes a heavy burden on columns in the lower storeys, which resist large compressive gravity loads. Therefore, in order to use the AJ connection efficiently for modular buildings in seismic zones, using other types of connections (e.g. bolted or welded connections) together with the AJ connection would be a good solution. Figure 6.44 shows such PF-D* and PF-S* models, for which AJ connections at the lower storey were replaced with bolted (or welded) connections. The pushover analysis results show that PF-D* and PF-S* had the same performance as WF-D and WF-S without gap-opening and yielding of SRBs in the frame, as shown in Figure 6.45.

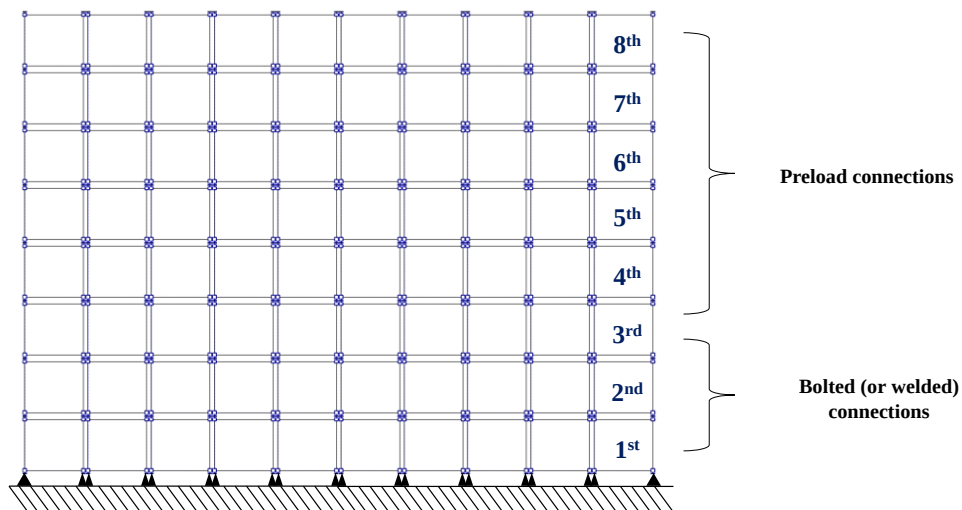
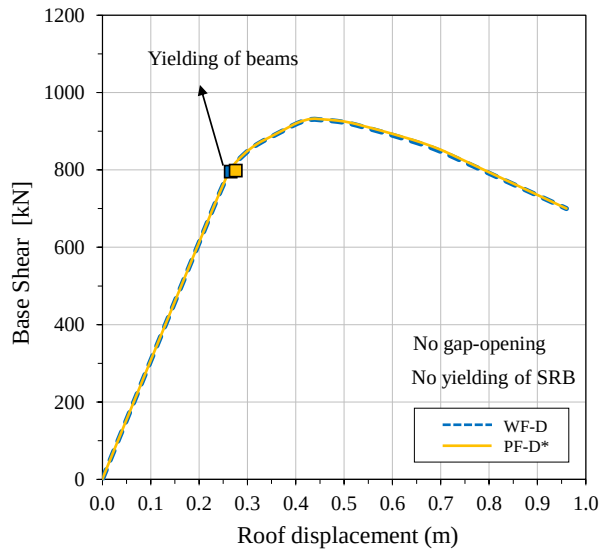
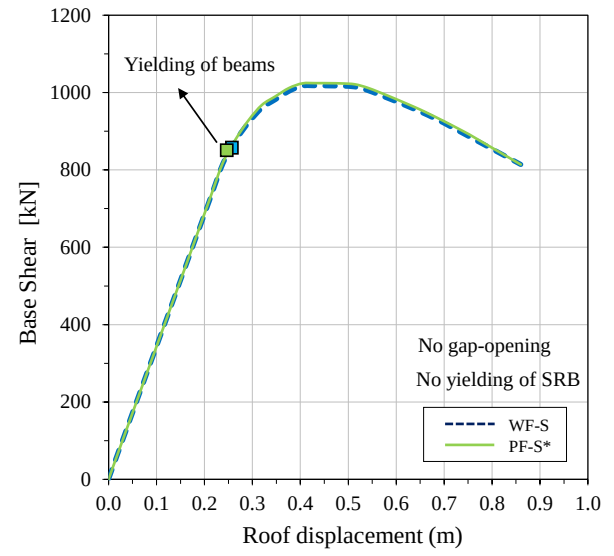


Figure 6.44: Application of the AJ connection with other types (e.g. bolted or welded) of connections for modular buildings



(a) PF-D*



(b) PF-D*

Figure 6.45: Capacity curves of the frames with other types (e.g. bolted or welded) for lower storeys (1st -3rd storeys)

6.4.4.2. Performance Based Evaluation (N2 Method)

In the previous section, the frames with AJ connections (PF-D/S) demonstrated less ductile behaviour than the ones with welded connections (WF-D/S). In this section, using the N2 method [141], the seismic performance of PF-D/S was evaluated for the design seismic actions (i.e. SLS and ULS). Additionally, it is evaluated whether the induced preload (for design seismic load ULS) can prevent gap-opening at the AJ connection in the event of the design earthquake.

The N2 method is a particular method to evaluate the seismic performance of structures designed by the current code (i.e. response spectral analysis). It was developed at the University of Ljubljana by Fajfar [141]. The method combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system to obtain the "target displacement (D_t)", which is the maximum displacement for the MDOF system to be experienced during the design earthquake. That is, D_t becomes the seismic demand of the designed structure, and the structure is evaluated whether it has sufficient deformation capacity to reach D_t by performing a pushover analysis.

The structural performance was evaluated for three limit states shown in Table 6.23, where a_{gr} is the reference peak ground acceleration of 0.22g and a_g represents the peak ground acceleration for each earthquake level. The performance of the structure was evaluated by comparing the plastic beam rotation and interstorey drift at D_t to the FEMA 356 [144] acceptance criteria, as shown in Table 6.24.

Table 6.23: Limit states and corresponding peak ground acceleration (a_g)

Limit state	Return period years	Probability of exceedance	a_g	$\lambda (= a_g/a_{gr})$
Damage Limitation (DL / SLS)	95	10% / 10 years	0.13g	0.58
Significant Damage (SD / ULS)	475	10% / 50 years	0.22g ($=a_{gr}$)	1.00
Near Collapse (NC / CPLS)	2475	2% / 50 years	0.38g	1.73

* SLS: serviceability limit state; ULS: ultimate limit state; CPLS: collapse prevention limit state.

Table 6.24: Seismic performance assessment provided by FEMA 365 for each limit state

Limit state	FEMA 365 - acceptance criteria		
	Plastic beam rotation [rad]		For steel moment frame
	Floor beam (RHS 200X100X4)	Ceiling beam (RHS 150X100X4)	Inter-storey drift
Damage Limitation (DL / SLS)	0.002	0.003	0.7%
Significant Damage (SD / ULS)	0.019	0.025	2.5%
Near Collapse (NC / CPLS)	0.029	0.038	5%

Figure 6.46 shows the comparison between the seismic demand and the capacity curves of the equivalent SDOF of PF-D/S and WF-D/S. Table 6.25 shows D_t computed for each of the three limit states. (D_t was slightly different for each frame because of the different periods of vibration T^* of the equivalent SDOF system, which were affected by the peak strengths of the MDOF system.) The modal pushover up to D_t was performed.

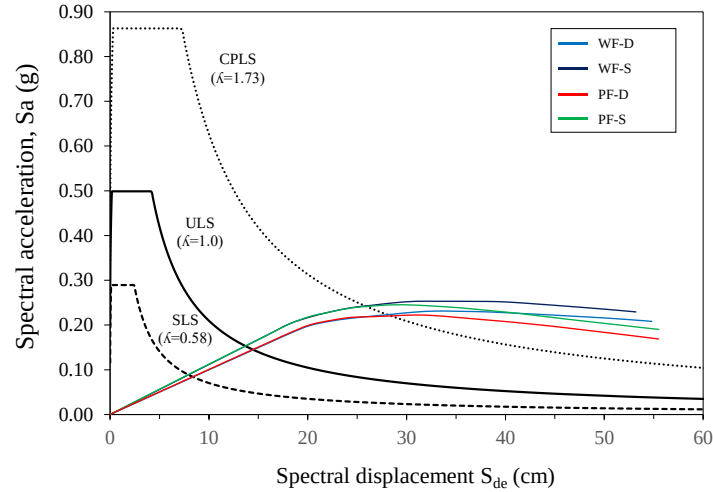


Figure 6.46: Seismic demand and the capacity curves of the equivalent SDOF of PF-D/S and WF-D/S

Table 6.25: N2 method - Target displacements D_t [mm]

Limit state	PF-D	PF-S	WF-D	WF-S
Damage Limitation (DL / SLS)	177	171	184	174
Significant Damage (SD / ULS)	302	294	323	297
Near Collapse (NC / CPLS)	460	465	491	474

Figure 6.47 and Figure 6.49 present the pushover curves for PF-D and PF-S and the target displacements corresponding to the three limit state levels. Figure 6.48 and Figure 6.50 show the distribution of plastic hinges in PF-D/S and WF-D/S for the ULS and CPLS seismic loads. Table 6.26 and Figure 6.51 compare the beam plastic rotation and interstorey drift at D_t with the FEMA 356 acceptance criteria.

At the Damage Limitation (DL) limit state, both PF-D/S and WF-D/S remained in the elastic range and satisfied the FEMA 356 limits for the beam plastic rotation, but their interstorey drifts exceeded the limits of 0.7% (approximately 1.0% for all frames).

At the Significant Damage (SD) limit state, many floor and ceiling beams formed plastic hinges evenly in PF-D without gap-opening at the AJ connection (see Figure 6.48). In the case of PF-S, some beams had plastic hinges, but most were still in the elastic range (see Figure 6.50). The gap-opening occurred at the lowest storey just before reaching D_t . The plastic beam rotation of floor and ceiling beams in PF-D/S fully satisfied the FEMA 356 limits, and the interstorey drift also did not exceed the limit of 2.5%.

At the Near Collapse (NC) limit state, the peak strength and lateral stiffness of PF-D/S were reduced due to the impact of gap-opening at the lowest storey, compared to the WF-D/S. However, they showed ductile behaviour by forming plastic hinges in the beams while SRBs inside the columns were resisting

the lateral force without yielding. Most plastic hinge rotation of the floor and ceiling beams in PF-D and PF-S did not exceed the SD limit. Both PF-D/S satisfied the plastic rotation limit of the beams and the interstorey drift limit of the FEMA 365 criteria.

In summary, the MRFs with the AJ connection (PF-D/S) displayed excellent seismic performance. Even though PF-D/S had lower strength and ductility due to the impact of gap-opening compared to the frames with rigid connections (WF-D/S), they met most performance objectives with significant elastic capacity for SD/ULS and NC/CPLS. However, the MRFs designed based on the current code did not satisfy the interstorey drift limits for DL/SLS, while it was confirmed that the gap-opening at the connections could be prevented by the induced preload until PF-D/S reached the design seismic load (ULS). Therefore, it is concluded that the AJ connection can be modelled simply as a rigid connection in structural analyses for seismic design (i.e. inelastic design) under the assumption that the demand preload suggested in this study is induced.

It should be noted that the MRF composed of modules may have a fairly large elastic capacity, implying they are not designed cost-effectively. This is because of the following:

- 1) As a characteristic of modular buildings, they feature both floor and ceiling beams when an upper module is assembled with a lower module, which causes the frame to have more beam strength capacity than necessary. In addition, the frames of modules (comprising beams and columns) are mostly designed to have the same size for reasons related to convenient manufacture in the factory, transportation, and lifting on-site for assembling, etc. So, the section sizes of floor and ceiling beams are not designed flexibly based on demand strength, and as a result, a frame composed of multiple modules will have overstrength.
- 2) Generally, the MRF has greater lateral deflection and drift compared to that of braced frames or shear walls. So, when designing the MRF according to EN 1998-1 for seismic design, the damage limit state (i.e. SLS) often governs [151]. This means that the cross-section sizes of columns and beams in the MRF are determined to control the drift, and so their sizes are larger than necessary for demand strength, also causing the MRF to have overstrength.

All in all, even if a modular building is designed using the behaviour factor q for seismic design in accordance with the current code, it will have a significantly larger elastic strength than an ordinary steel structure. The large elastic strength means that the demand ductile behaviour for the design seismic load becomes smaller. In other words, even though the ductility of the modular building may be insufficient, the building is likely to have sufficient seismic performance to resist the design seismic load.

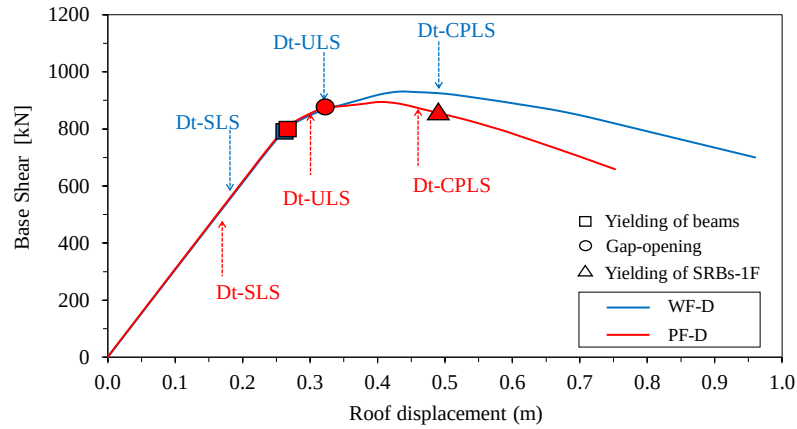


Figure 6.47: Capacity curves of PF-D and WF-D with target displacement (D_t) corresponding to the limit state levels

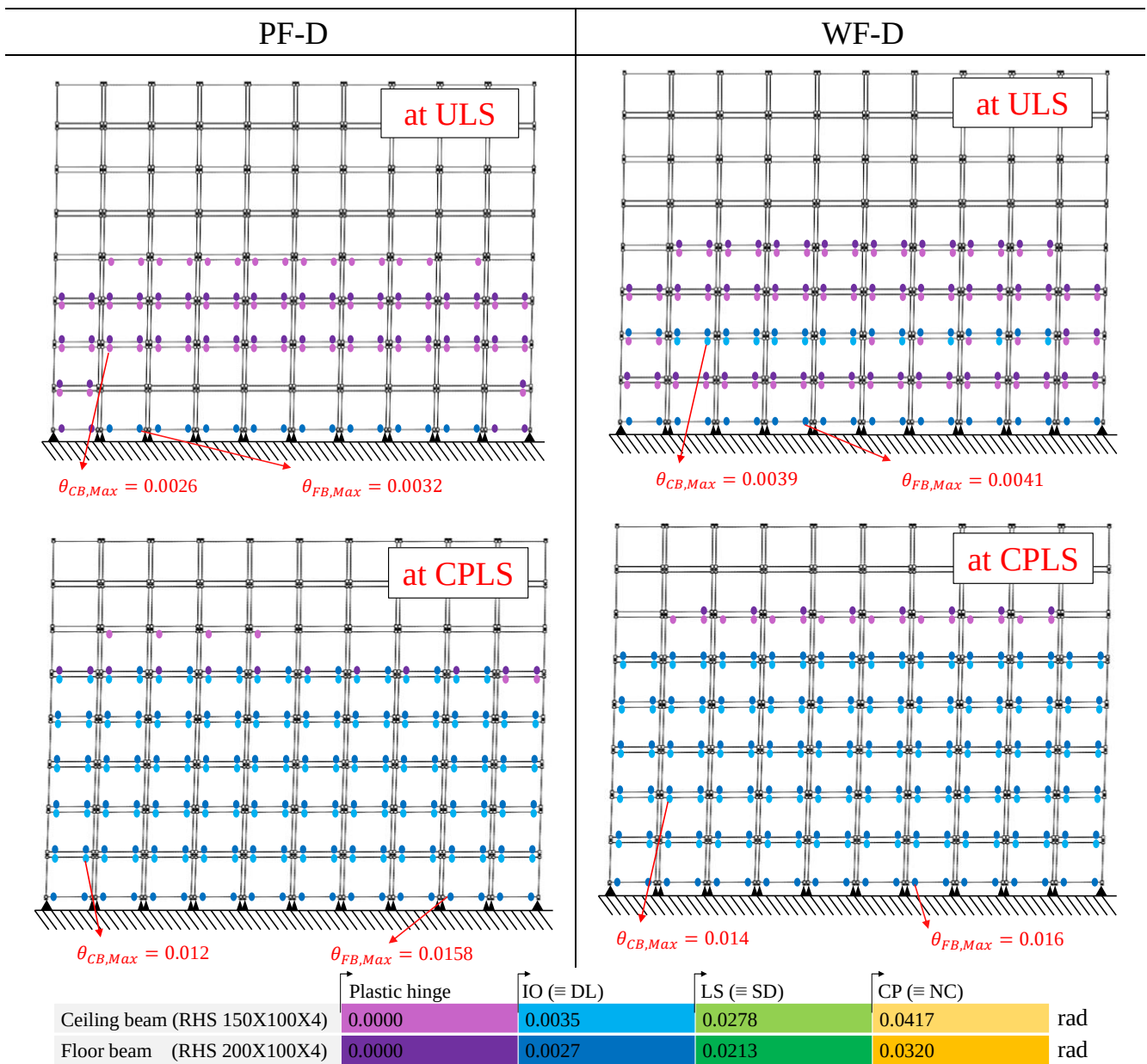


Figure 6.48: Distribution of plastic hinges on floor and ceiling beams in PF-D and WF-D at ULS and CPLS

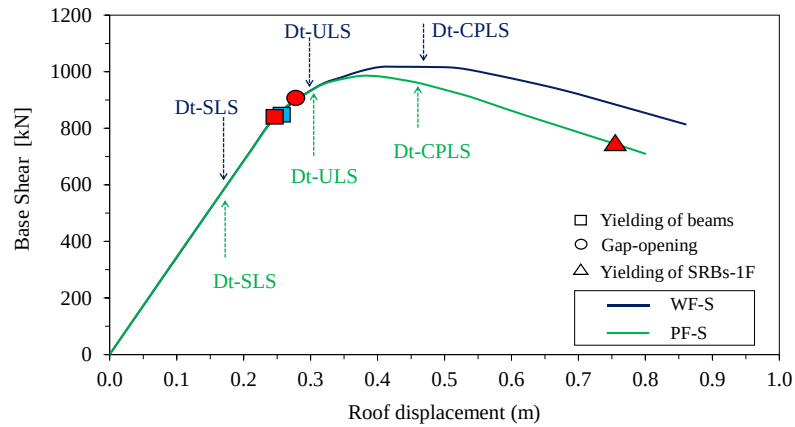


Figure 6.49: Capacity curves of PF-S and WF-S with target displacement (D_t) corresponding to the limit state levels

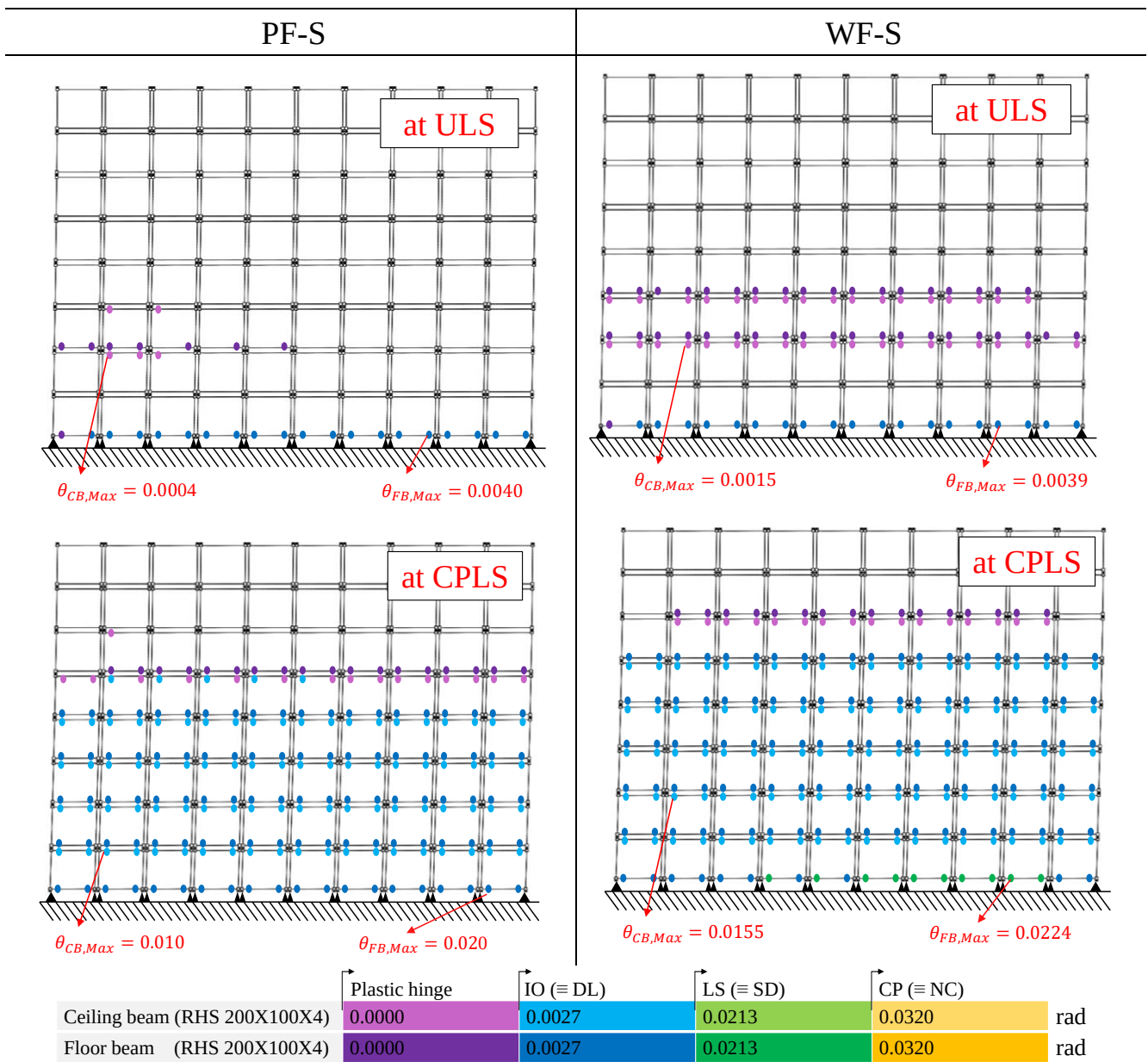


Figure 6.50: Distribution of beam plastic hinges on floor and ceiling beams in PF-S and WF-S at ULS and CPLS

Table 6.26: Maximum plastic rotation angle of hinges of floor and ceiling beams at target displacements (D_t) [rad]

(a) PF-D

Limit state Member	DL / SLS		SD / ULS		NC / CPLS	
	Floor beam	Ceiling beam	Floor beam	Ceiling beam	Floor beam	Ceiling beam
N2 - Beam rotation at D_t	0	0	0.0041	0.0041	0.0158	0.0121
FEMA acceptance criteria	0.0027	0.0035	0.0213	0.0278	0.0320	0.0417

(b) PF-S

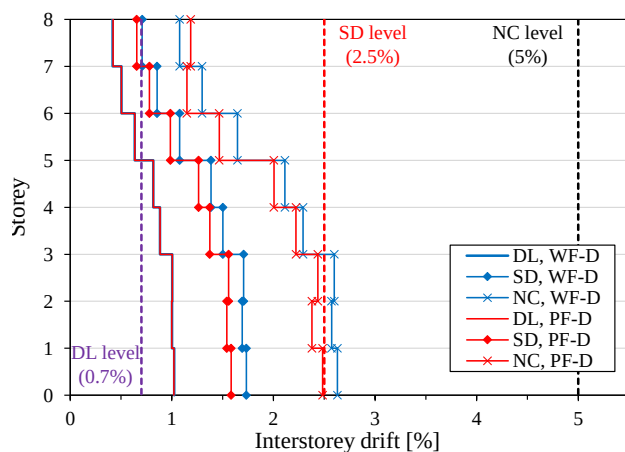
Limit state Member	DL / SLS		SD / ULS		NC / CPLS	
	Floor beam	Ceiling beam	Floor beam	Ceiling beam	Floor beam	Ceiling beam
N2 - Beam rotation at D_t	0	0	0.006	0.0015	0.0206	0.0100
FEMA acceptance criteria	0.0027	0.0027	0.0213	0.0213	0.0320	0.0320

(c) WF-D

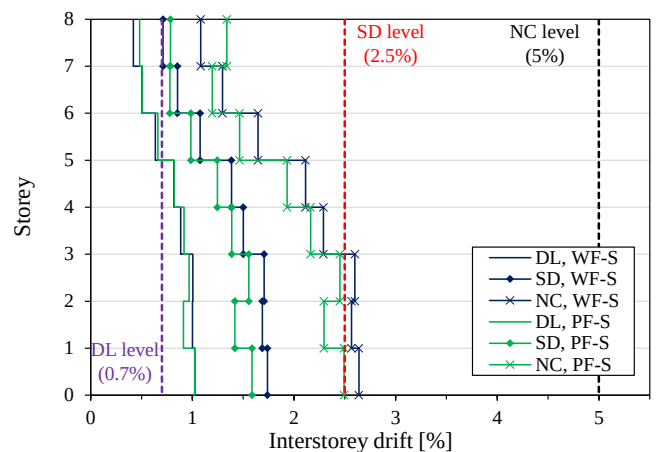
Limit state Member	DL / SLS		SD / ULS		NC / CPLS	
	Floor beam	Ceiling beam	Floor beam	Ceiling beam	Floor beam	Ceiling beam
N2 - Beam rotation at D_t	0	0	0.0041	0.0041	0.0189	0.0155
FEMA acceptance criteria	0.0027	0.0035	0.0213	0.0278	0.0320	0.0417

(d) WF-S

Limit state Member	DL / SLS		SD / ULS		NC / CPLS	
	Floor beam	Ceiling beam	Floor beam	Ceiling beam	Floor beam	Ceiling beam
N2 - Beam rotation at D_t	0	0	0.006	0.0015	0.0206	0.0100
FEMA acceptance criteria	0.0027	0.0027	0.0213	0.0213	0.0320	0.0320



(a)



(b)

Figure 6.51: Inter-storey drift at the corresponding limit state levels: (a) PF-D and WF-D, and (b) PF-S and WF-S

6.5. Conclusion

The mid-rise moment resisting frame (MRF) composed of modules with the AJ connection was investigated using a 2D FE frame model. The main purpose was to evaluate whether the induced preload, determined using the design method described in Sections 3.4.3, 3.4.4, and 3.4.5, can secure the safe performance of MRFs to resist design wind and seismic loads.

- (1) The effect of the inter-module connection stiffness on the behaviour of the frame was studied, and the minimum demand stiffness for the inter-module connections was obtained to make the assembly of individual modules behave as a compatible structure.
- (2) A 2D simplified FE joint model, which could describe the important features of AJ connection, was developed. This model could accurately simulate the impact of gap-opening on the behaviour of a beam-to-column joint (i.e. the failure mechanism, reduction in stiffness, and increasing the tensile force in the steel rod bolt under gap-opening). The joint model was verified by comparing it to the 3D detailed joint model for the advanced frame analysis against wind and seismic load.

Wind analysis

- (3) The impact of slip is more critical than gap-opening at the connection for wind loads. The connections at the upper storeys are more vulnerable to slip due to the larger wind load combined with the smaller self-weight of modules. Accumulated slip at the column-to-column connections according to the height of the building causes a significant top displacement. Therefore, the connections should be designed as slip-resistant connections in mid-rise buildings.
- (4) The heavier weight of a module contributes to preventing gap-opening and slip at the AJ connection, resulting in a reduced demand preload for the connections. The simulation showed that, for instance, in case the prototype 8-storey mid-rise modular building is assembled with modules having a concrete slab ($t=150\text{mm}$), gap-opening and slip were prevented by the self-weight of the modules alone without a need for inducing preload for the level of wind load in Sydney (46m/s).
- (5) The FE simulation confirmed that the induced preload could provide the required amount of preload for preventing slip and gap-opening at the connection in the frame for wind load (i.e. elastic design). As a result, the MRF with AJ connections demonstrated a behaviour corresponding to the MRF with rigid connections. Therefore, it is possible to simply model AJ connections as rigid connections in structural analyses.

Seismic analysis

- (6) The MRFs with the AJ connection (PF-D/S) demonstrated ductile strong column-weak beam behaviour. Even though gap-opening occurred at the lowest storey shortly after yielding was reached, most beams could form plastic hinges evenly throughout the frames while the SRBs inside columns were resisting the lateral force. However, compared to the MRF with rigid connections (WF-D/S), they had lower ductility and peak strength due to the effect of gap-opening.
- (7) Using the N2 method, the seismic performance of MRFs with the AJ connection (PF-D/S) was evaluated for the seismic design actions (ULS, $a_g=0.22g$). They displayed excellent seismic performance with a considerable elastic capacity, and met most performance objectives stipulated by the FEMA 365 criteria (i.e. beam plastic rotation and interstorey drift) for SD/ULS and NC/CPLS, but not for the interstorey drift for DL/SLS.
- (8) The MRF composed of modules might have a significantly large elastic capacity associated with the overstrength resulting from the characteristic of a modular building. (i.e. double beams comprising floor and ceiling beams, and the use of module units having the same sizes of beams and columns throughout the modular building). As a result, even if the ductility of the modular building may prove insufficient, the building is likely to have sufficient seismic performance to resist the design seismic loads. Therefore, the minimum amplification factor α_B (Eq. 3.98) can be used for MRFs to determine the design seismic load for calculating the demand preload for the AJ connection.
- (9) The pushover analysis confirmed that the induced preload could almost prevent gap-opening at the level of the design seismic load (ULS). Therefore, AJ connections can be simply modelled as rigid connections when performing structural analysis for seismic design (i.e. inelastic design).
- (10) The MRF with the same cross-section size of floor and ceiling beams (PF-S) shows better ductile behaviour than the MRF with different sizes of beams (PF-D). Compared to PF-D, PF-S required approximately 30-70% smaller demand preload for the connections for the same design seismic load. In addition, even if gap-opening occurs, PF-S was less impacted. As a result, PF-S had larger lateral stiffness, peak strength, and ductility with stable behaviour of SRBs in the inelastic behaviour than PF-D.
- (11) AJ connections located at the low storeys may require a significantly large preload for seismic design. Therefore, if required, it would be an efficient solution to replace AJ connections in the low storeys with other types of connections (e.g. bolted or welded connections) for the modular building.

Chapter 7

Design Recommendations for the AJ Connection

7.1. Introduction

This Chapter aims to provide practicing engineers with guidance on the design procedure for the AJ connection proposed in this study for steel-framed modular buildings. The design guidelines are based on the research results from the analytical approach (Chapter 3), the experiments (Chapter 4) and numerical simulations (Chapters 5 and 6), as well as the design provisions of EN 1993-1 [85, 88] and EN 1998-1 [86].

7.2. Design Flowchart

Figure 7.1 illustrates the design procedure for a modular building with AJ connections.

7.3. Preliminary Design and Seismic Design

Modelling of inter-module connections for structural analysis

A module unit needs to be designed first, and then a frame composed of modules is analysed according to current codes for the SLS and ULS load combinations to obtain design forces for the AJ connection. In the frame model for the structural analysis, vertical and horizontal inter-module connections should be modelled as fixed connections. In particular, depending on the shear stiffness of the horizontal inter-module connections (i.e. the connecting plate), axial forces acting on columns that affect the demand preload for the AJ connection are significantly changed (i.e. location of tensile and compressive forces and magnitude), refer to Section 6.2.4. Therefore, for use in structural analyses, the horizontal inter-module connections must be designed as fixed connections that reflect the shear stiffness. Additionally, it is recommended to design the connecting plate so that it satisfies the minimum demand stiffness suggested in Table 6.1 to secure all individual modules in the frame to behave as a compatible structure. This can be achieved without difficulty if the gap-length between modules is not too large (refer to Section 6.2.3 (2)).

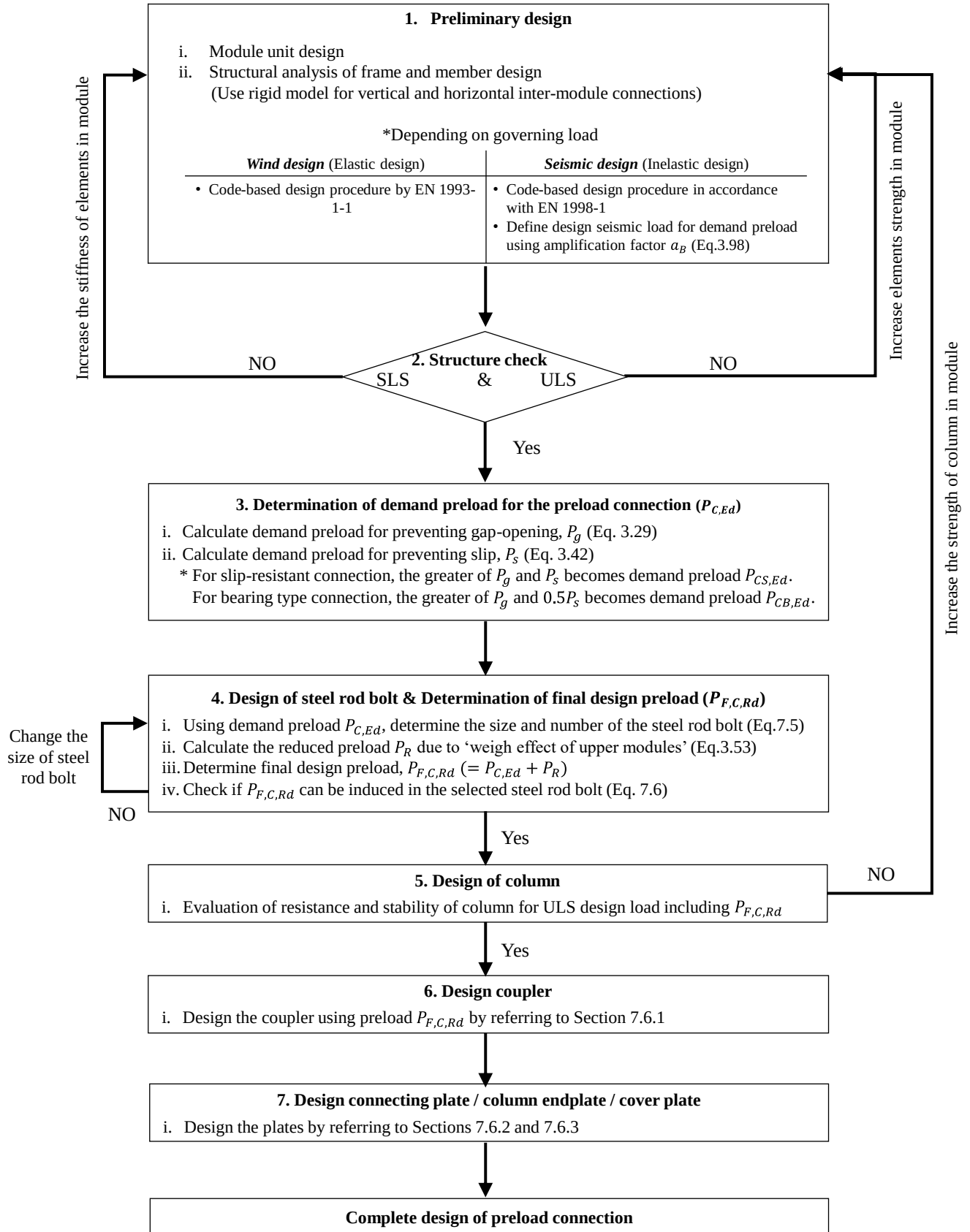


Figure 7.1: Design flowchart for design of a modular building with AJ connections

Seismic design and amplification factor, α_β for MRF

When conducting a capacity seismic design, dissipative and non-dissipative members are designed according to a code, herein using EN 1998-1 [86]. The demand preload for the AJ connection is determined according to the same concept as for non-dissipative members as the preload required to prevent gap-opening and slip until both floor and ceiling beams have developed ductile failure by forming plastic hinges. In order to achieve this, the amplification factor α_β is proposed in Section 3.6.4.2 for determining the design seismic forces for the demand preload. The value of α_β is always greater than or equal to Ω (i.e. the minimum design overstrength of the dissipative elements in EN 1998-1), but otherwise can be adjusted as needed. The higher the value, the more ductile behaviour can be secured by reducing the impact of gap-opening on the frame. However, since a higher value causes the demand preload to increase, it should be noted that a significantly large preload may not be achievable in practice. Therefore, the minimum α_β , which is the same value as Ω , can be used to determine the seismic design load for the demand preload (refer to Figure 6.40).

7.4. Determination of Demand Preload

Calculating the demand preload for all AJ connections in a frame can be inefficient and time-consuming. Therefore, it is recommended to calculate the demand preload for the most critical connection (i.e. columns subjected to the largest tensile or shear forces) in each storey and then apply the same demand preload to all connections located in a particular storey. The location of the critical connections can be predicted by checking the distribution of axial reaction forces. Normally, they are located in the exterior modules in the building. Section 6.4.3.1 can be referred to for the determination of demand preload in a frame.

In case the preload to be induced to an SRB in an upper column is greater than the one in a lower column, the preload for the lower column needs to be adjusted so that it is equal to or slightly greater than that in the upper column. Otherwise, the preload for the upper column may not be induced. This is because the SRB coupler in the lower column is pulled up the moment the preload for the upper column exceeds the preload induced in the lower column. It becomes the situation where the preload is induced to the equivalent of a two storeys long SRB (refer to Section 3.3 and Figure 3.10).

In case the demand preload is large for the connections at the low storeys, it is suggested to apply two SRBs per column after dividing the frame into 2 or 3 zones according to the height of the building. If the demand preload is significantly large, particularly for the seismic design, it is recommended to design the modules to have the same size floor and ceiling beams to reduce the demand preload or to replace AJ connections with other types of connections (e.g. bolted or welded connections) for the lower storeys (refer to Figure 6.44).

In order to maintain the induced preload after assembling the modular building, the preload to be reduced (P_R) due to the weight effect of the upper modules should be taken into account, which becomes the final design preload ($P_{F,C,Rd}$). Since the magnitude of the gravity load acting on columns in each storey is different, the amount of P_R for each storey is also different. Therefore, it is necessary to calculate P_R for all storeys, and add to the demand preload ($P_{C,Ed}$) for each storey to determine the final design preload. The connections at the lowest storey have the largest P_R . It should be noted that P_R is dependent on the axial stiffness of SRB and column. If the axial stiffness of the column is larger, P_R is smaller. On the other hand, if the axial stiffness of the SRB is larger, P_R is larger. Further, if two SRBs are used in one column, P_R is doubled (refer to Section 3.4.5.1).

7.5. Design of Beams, Columns and Panel Zones

The beams, columns and panel zones are designed for the SLS and ULS load combination in accordance with EN 1993-1-1 [88]. For the columns, it should be noted that the amount of induced preload (i.e. compressive force) must be included in the design forces for checking their resistance and stability. Section 3.6.3 can be referred to for designing the beams, columns, and panel zones for seismic design.

7.6. Design of the AJ Connection

The AJ connection can be designed as a slip-resistant or a bearing type connection:

1) Slip-resistant connection

For mid- and high-rise modular buildings, it is recommended to design the AJ connection as a slip-resistant connection to avoid accumulated slip from column-to-column connections causing a large lateral displacement at the top of the building. Therefore, the design slip resistance $F_{s,Rd}$ should be greater than the design ultimate shear force $F_{v,Ed}$. Additionally, the bearing resistance of the plate $F_{b,Rd}$ should be checked according to EN 1993-1-8 [85].

$$\begin{aligned} F_{v,Ed} &\leq F_{s,Rd} \\ F_{v,Ed} &\leq F_{b,Rd} \end{aligned} \quad (\text{Eq. 7.1})$$

where : $F_{v,Ed}$ Design ultimate shear force;
 $F_{s,Rd}$ Design slip resistance at the ultimate limit state (refer to Eq. 3.31);
 $F_{b,Rd}$ Design bearing resistance of the plate at the ultimate limit state (refer to (1) in Table 7.5).

The demand preload for the slip-resistant connection $P_{CS,Ed}$ becomes the preload to prevent both gap-opening and slip. Therefore, the following criteria should be satisfied:

$$\begin{aligned} P_g &\leq P_{CS,Ed} \\ P_s &\leq P_{CS,Ed} \end{aligned} \quad (\text{Eq. 7.2})$$

where : P_g Demand preload for preventing gap-opening at the ultimate limit state, determined by (Eq. 3.29);
 P_s Demand preload for preventing slip at the ultimate limit state, determined by (Eq. 3.42);
 $P_{CS,Ed}$ Demand preload for slip-resistance connection.

The final design preload for the slip-resistant connection $P_{F,CS,Rd}$ becomes the value obtained by adding P_R representing the weight effect of upper modules to $P_{CS,Ed}$.

2) Bearing type connection

It is recommended to use the bearing type connection only for low-rise modular buildings (i.e. 2 or 3 storeys). According to EN 1993-1-8 [85], the design resistance of a bearing type connection is the minimum of the design bearing resistance of the plates $F_{b,Rd}$ and the design shear resistance of the coupler $F_{c,Rd}$.

$$\begin{aligned} F_{v,Ed} &\leq F_{b,Rd} \\ F_{v,Ed} &\leq F_{c,Rd} \end{aligned} \quad (\text{Eq. 7.3})$$

The demand preload for bearing type connections $P_{CB,Ed}$ becomes the preload to prevent gap-opening. Also, $P_{CB,Ed}$ should be larger than 50% of the demand preload for preventing slip P_s to avoid a slip-induced bending force from being applied to the SRB (refer to Section 5.5.1.2 and Figure 5.51 (c)-(4)). Therefore, the following criteria should be satisfied:

$$\begin{aligned} P_g &\leq P_{CB,Ed} \\ 0.5P_s &\leq P_{CB,Ed} \end{aligned} \quad (\text{Eq. 7.4})$$

The final design preload for the bearing type connection $P_{F,CB,Rd}$ becomes the value obtained by adding P_R to $P_{CB,Ed}$.

7.6.1. Steel Rod Bolt and Coupler

The SRB corresponds to the thread and shank of a general preloaded bolt, while the coupler corresponds to the bolt head and a nut. So, the material properties and size of SRB and coupler were determined by referring to EN 14399 (high strength structural bolting for preloading) for preloaded bolts [152]. The SRB and coupler are recommended to be made of steel having the equivalent yield and tensile strengths to bolt classes 8.8 and 10.9 for general preloaded bolts.

The design forces for the SRB and coupler are similar to those of general preloaded bolts. Therefore, it is recommended to base their design on the design provisions for preloaded bolts according to EN 1993-1-8 [85].

■ Steel rod bolt

The required size and number of SRB is determined as follows:

$$P_{C,Ed} \leq n \cdot F_{p,d} \quad \rightarrow \quad \frac{P_{C,Ed}}{F_{p,d}} \leq n \quad (\text{Eq. 7.5})$$

where :

$P_{C,Ed}$	Demand preload for the AJ connection (i.e. $P_{CS,Ed}$ or $P_{CB,Ed}$);
$F_{p,d}$	Inducible amount of preload per SRB;
n	The number of SRBs.

The inducible amount of preload per SRB, $F_{p,d}$ is calculated using (Eq. 7.6) from EN 1993-1-8 [85].

$$F_{p,d} = \frac{0.7 \cdot f_{ub} \cdot A_s}{\gamma_{M7}} \quad (\text{Eq. 7.6})$$

where :

A_s	Tensile stress area (=Threaded portion of the bolt);
f_{ub}	Ultimate tensile strength of SRB;
γ_{M7}	1.1 for preloads of high strength bolts.

Table 7.1 shows the applicable sizes of SRBs and the corresponding area of gross section and tensile stress area (i.e. area of threaded portion) based on the maximum amount of preload induced in the 3m SRB in the tests. Table 7.2 shows the design preload for each size of SRB.

Table 7.1: Nominal areas of SRBs in accordance with EN ISO 898 [153]

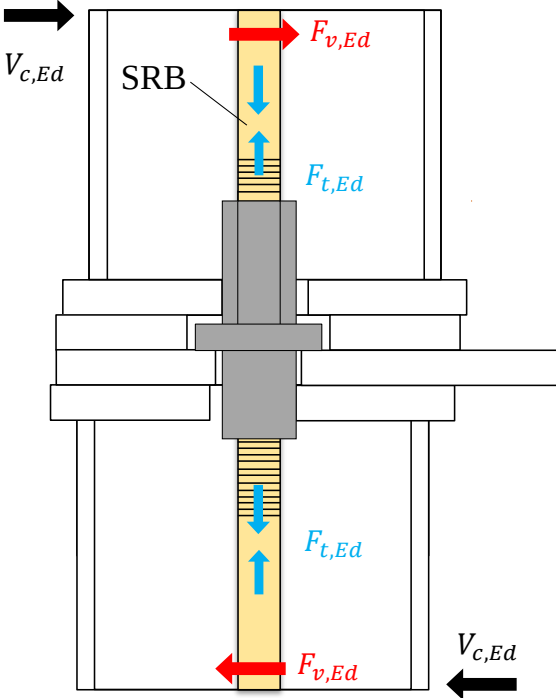
Bolt grade	M20	M22	M24
Diameter, d (mm)	20	22	24
Gross section area, A (mm ²)	314	380	452
Tensile stress area, A_s (mm ²)	245	303	353

Table 7.2: Nominal inducible amount of preload $F_{p,d}$ per SRB

Bolt class	f_{yb} [N/mm ²]	f_{ub} [N/mm ²]	M20 [kN]	M22 [kN]	M24 [kN]
8.8	640	800	125	155	180
10.9	900	1000	156	193	225

Table 7.3 shows the design forces for the SRB. Unless slip occurs, the SRB comprises only tensile force caused by preloading. If slip occurs, it is also subjected to a shear force $F_{v,Ed}$. Therefore, the following conditions should be satisfied for tension, shear, and combined shear and tension.

Table 7.3: Design forces and the corresponding resistance of the SRB

Design forces	Failure mode and resistance
 ($F_{v,Ed} = V_{c,Ed}$, $F_{t,Ed} = P_{Rd}$)	(1) In tension $F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Rd} = \frac{0.9 f_{ub} \cdot A_s}{\gamma_{M2}}$ (2) In shear $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$ (3) In combined shear and tension $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 F_{t,Rd}} \leq 1.0$
where : $F_{t,Ed}$ and $F_{v,Ed}$ $V_{c,Ed}$ P_{Rd} $F_{t,Rd}$ and $F_{v,Rd}$ f_{ub} α_v A_s γ_{M2}	Design tensile and shear forces for the SRB, respectively; Design shear force acting on the column; Induced design preload to the SRB ($=P_{F,C,Rd}$); Design tensile and shear resistance for the SRB, respectively; Ultimate tensile strength of the SRB; 0.5 for bolt class 10.9, 0.6 for bolt class 8.8; Tensile stress area of the SRB (=Threaded portion of the bolt); Partial safety factor 1.25 for bolts.

■ Coupler

The diameter of the coupler body (see Figure 4.16) is determined referring to the nut size of a general preloaded bolt. The thickness and diameter of the coupler plate are determined with reference to the bolt head height and washer size of a preloaded bolt with the same diameter as the coupler body. The minimum thread engagement length between an SRB and coupler is determined by referring to the nut height of a preloaded bolt.

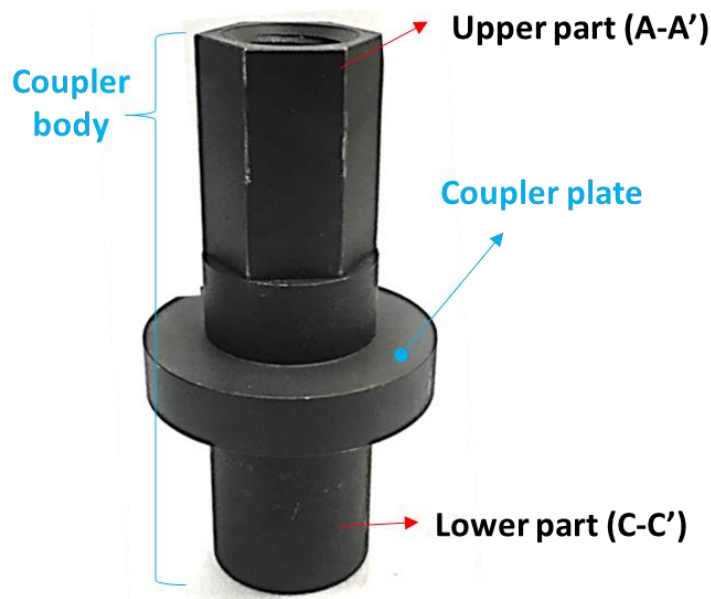
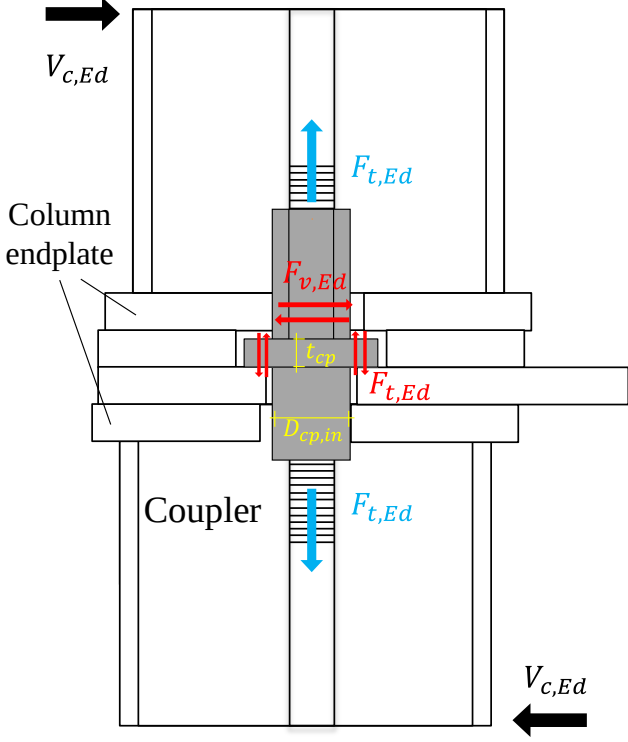


Figure 7.2: 3D image of coupler (from Figure 4.16 (a))

Table 7.4 shows the design forces and resistance for the coupler. As for the SRB, the coupler is subjected to only tensile force unless slip occurs. If slip occurs, it is also subjected to a shear force $F_{v,Ed}$. Therefore, the coupler should satisfy the strength requirements for tension, shear, and combined shear and tension. Since the coupler has different cross-sections along the length, the minimum sectional area of the upper part (see Figure 4.16) should be used for its resistance. Additionally, the shear resistance of the coupler cover plate should be checked against the applied tensile force.

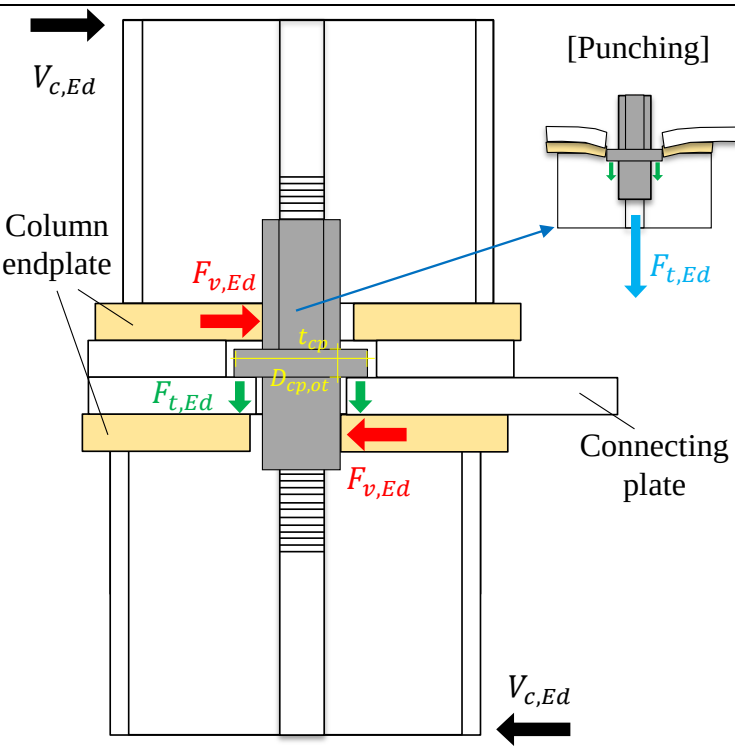
Table 7.4: Design forces and the corresponding resistance of the coupler

Design forces	Failure mode and resistance
 $(F_{v,Ed} = V_{c,Ed}, F_{t,Ed} = P_{Rd})$	(1) In tension (coupler body) $F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Rd} = \frac{0.9f_{ub} \cdot A_{min,s}}{\gamma_{M2}}$ (2) In shear (coupler body) $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A_{min,s}}{\gamma_{M2}}$ (3) In combined shear and tension (coupler body) $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1.0$ (4) In shear (coupler plate) $F_{t,Ed} \leq V_{pl,Rd}$ $V_{pl,Rd} = \frac{A_v \cdot (f_{yb}/\sqrt{3})}{\gamma_{M0}}$
where :	$F_{t,Ed}$ and $F_{v,Ed}$ Design tensile and shear forces for the coupler, respectively; $V_{c,Ed}$ Design shear force acting on the column; P_{Rd} Induced design preload to the SRB ($=P_{F,C,Rd}$); $F_{t,Rd}$ and $F_{v,Rd}$ Design tensile and shear resistance for the coupler, respectively; f_{yb} Yield strength of the coupler plate; f_{ub} Ultimate tensile strength of the coupler; α_v 0.5 for bolt class 10.9, 0.6 for bolt class 8.8; $A_{min,s}$ Minimum tensile stress area of the coupler; A_v Shear area ($=\pi \cdot D_{cp,in} \cdot t_{cp}$); γ_{M2} Partial safety factor 1.25 for bolts; γ_{M0} Partial safety factor 1.0 for resistance of cross-section in tension to fracture.

7.6.2. Column Endplate

Table 7.5 shows design forces for the column endplate. If slip occurs, the coupler and endplate become loaded in bearing. In addition, when preload is induced in an SRB using a coupler, the endplate may have punching failure. Therefore, the column endplate must satisfy design requirements for bearing, shear, and punching.

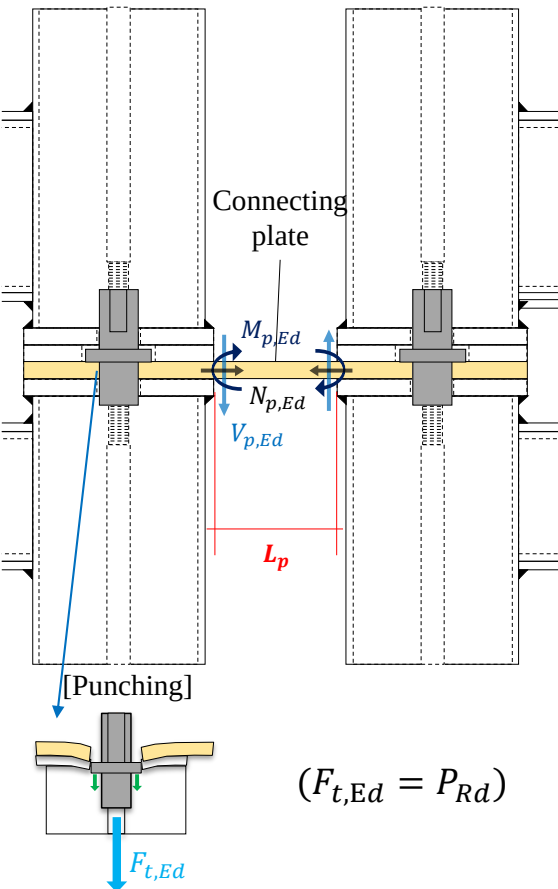
Table 7.5: Design forces and the corresponding resistance of the column endplate

Design forces		Failure mode and resistance
 (F_{v,Ed} = V_{c,Ed}, F_{t,Ed} = P_{Rd})		(1) In bearing $F_{b,Ed} \leq F_{b,Rd}$ $F_{b,Rd} = \frac{k_1 \cdot a_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$ (2) In shear $F_{t,Ed} \leq F_{v,Rd}$ $F_{v,Rd} = \frac{A_v \cdot (f_y / \sqrt{3})}{\gamma_{M0}}$ (3) In punching shear resistance $F_{t,Ed} \leq B_{P,Rd}$ $B_{P,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u}{\gamma_{M2}}$
where :	k_1 Factor considers reduction of bearing resistance; α_b Factor considers shear failure; f_u Ultimate tensile strength of the column endplate; f_y Yield strength of the column endplate; t_p Thickness of the column endplate under the coupler; t Thickness of the column endplate; A_v Shear area of the column endplate (= $\pi \cdot D_{cp,ot} \cdot t_{cp}$); d Nominal coupler body diameter; d_m Diameter of the coupler plate; γ_{M2} Partial safety factor 1.25 for bolts; γ_{M0} Partial safety factor 1.0 for resistance of cross-section in tension to fracture.	

7.6.3. Connecting Plate (Horizontal Inter-Module Connection)

Table 7.6 shows the design forces for the connecting plate. Its resistance must be verified for the applied axial, shear, and moment actions according to EN 1993-1-1 [88]. In addition, when preload is induced in an SRB using a coupler, the connection may have punching failure if the connecting plate is thin. Therefore, the connecting plate must also satisfy a punching requirement.

Table 7.6: Design forces and the corresponding resistance of the connecting plate

Design forces	Failure mode and resistance
	(1) In axial $N_{Ed} \leq N_{Rd}$ $N_{Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$
	(2) In shear $V_{Ed} \leq V_{pl,Rd}$ $V_{pl,Rd} = \frac{A_v \cdot (f_y / \sqrt{3})}{\gamma_{M0}}$
	(3) In bending $M_{Ed} \leq M_{pl,Rd}$ $M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$
	(4) In punching shear $F_{t,Ed} \leq B_{P,Rd}$ $B_{P,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u}{\gamma_{M2}}$
where : N_{Ed}, V_{Ed}, and M_{Ed} $F_{t,Ed}$ P_{Rd} N_{Rd} $V_{pl,Rd}$ $M_{pl,Rd}$ W_{pl} A A_v f_y d_m t_p γ_{M2} γ_{M0}	Design axial force, shear force, and moment for the connecting plate; Design shear forces for the connecting plate, respectively; Induced design preload to the SRB ($=P_{F,C,Rd}$); Design axial resistance; Design plastic shear resistance; Design plastic bending resistance; Plastic section modulus; Cross sectional area for the connecting plate; Shear area for rectangular cross section; Yield strength of the connecting plate; Dimension of bolt head (= diameter of coupler plate); Thickness of the connecting plate under the coupler; Partial safety factor 1.25 for bolts; Partial safety factor 1.0 for resistance of cross-section in tension to fracture.

Chapter 8

Conclusion

8.1. Research Summary

The main aim of the proposed research was to develop an innovative post-tensioned type inter-module connection to maximise construction speed and reusability of modules for mid-rise steel-framed modular buildings as well as to apply the connection in seismic areas. The aim also included providing optimal design principles and structural modelling of the connection to enable engineers to design in practice.

Based on an in-depth analysis of existing inter-module connections from the relevant literature, the AJ connection was proposed for steel-framed modular buildings. This column-to-column connection connects modules vertically by preload, which is applied using the developed coupler and high-tensile SRBs within columns. The AJ connection has a simple shape and installation process for connecting modules, which provides an easy and convenient on-site assembly and disassembly process of the modular building. In addition, it allows for the modules to be manufactured as complete finished products, not requiring additional on-site finishing works after connecting modules, unlike existing bolted connections. It also does not require a hydraulic jack to apply preload, unlike the existing post-tensioned connections. Consequently, the AJ connection can maximise the benefits of 3D volumetric modular construction (i.e. fast on-site construction, reusability of modules, and automated factory production of modules).

In order to apply the AJ connection to steel-framed modular buildings in practice, the structural performance of the connection was investigated using analytical, experimental and numerical methods. Firstly, theoretical principles (e.g. preload mechanism, failure modes, determination of demand preload, seismic design application, etc.) of the AJ connection were established using an analytical approach. Secondly, the applicability of the connection in practice and the established theoretical principles were evaluated and verified through experimental investigations of the component (the developed coupler) and the beam-to-column joint. Thirdly, using the advanced finite element 3D joint models calibrated against the test results, the performance of the AJ connection was additionally investigated at the joint level. Fourthly, by applying the 2D simplified FE model of the AJ connection to a mid-rise modular building frame, the behaviour of the frame was investigated when subjected to wind and seismic loads. Based on these studies, guidelines for the optimal design of the AJ connection were developed.

Concluding remarks of the work in this thesis are presented in Section 8.2. The conclusions are broken into three main areas: analytical investigation, experimental investigation, and numerical investigation.

8.2. Final Remarks

8.2.1. Analytical Investigation

The preloading mechanism of the AJ connection has been explained in detail according to the assembling process of an upper column and a lower column. The connection has three contact surfaces between the constituent plates. Depending on the magnitude of preload applied from the upper column and the lower column, the clamping forces on the contact surfaces of the connection can differ. It is desirable to design the connection to have a uniform clamping force on all the contact surfaces by applying the same amount of preload to the upper column and lower column.

The failure mode of the AJ connection is defined as ‘gap-opening’ and ‘slip’. These failure modes can significantly reduce the stiffness of the entire frame and affect the overall stability of the building by causing an additional P- Δ effect. Therefore, the demand preload for the connection has been defined as the preload required to prevent these failure modes, and the determination process for the demand preload has been developed accordingly.

When a module is stacked on another module, the weight of the upper module causes the preload in the lower module to decrease. The calculation process for the reduction in preload caused by the weight of upper modules was presented. Therefore, in order to maintain the demand preload at the AJ connection after completing the construction of a modular building, an enhanced preload to be subsequently reduced due to the weight of installed upper modules must be applied to the connection to achieve the demand preload in the finished building.

Regarding seismic design, depending on the occurrence of gap-opening at the AJ connection, the behaviour of the beam-to-column joint may be significantly different. In case no gap-opening occurs at the connection, both the floor and ceiling beams (dissipative members) reach their full plastic moment capacities, M_p , and the SRBs within the columns stably maintain the initial preload. On the other hand, if a gap-opening occurs at the connection, it has the following negative effects: (i) Reduced initial lateral stiffness, (ii) Decreased ductility by delaying or preventing the formation of plastic hinges in both beams (i.e. only the smaller beam reaches its full M_p , while the larger beam may or may not reach its M_p , and (iii) The SRBs within columns are subjected to large tensile force in addition to the initial preload, exposing them to the risk of yielding and rupture. As a result, the gap-opening at the AJ connection significantly reduces the seismic performance of the joint and therefore, the gap-opening at the connection should be prevented by the preload.

The gap-opening at the AJ connection is affected by differences in the flexural stiffness (EI) of the floor and ceiling beams. The impact of gap-opening increases with increasing difference between the flexural stiffness of the two beams. If the same cross-sectional size (same EI) is used for the two beams, the impact of gap-opening is minimised, resulting in the demand preload being reduced to prevent gap-opening. Therefore, in the case of AJ connections subjected to large lateral forces (e.g. on the lower floor in a multi-storey building), it is a good strategy to use the same beam size for the floor and ceiling beams.

This study aimed to produce moment-resisting frames consisting of steel-framed modules connected by AJ connections that exhibit strong column-weak beam behaviour and a ductile response to earthquake loading. Based on the design criteria and detailing rules of EN 1998-1, design details for the beam-to-column joint with the AJ connection are presented, considering the characteristics of the modular building. The AJ connection should be designed as a non-dissipative AJ connection, which can prevent gap-opening and slip at the connections until both the floor and ceiling beams (dissipative members) reach their full plastic moment capacities. The design forces required to obtain the demand preload that satisfies this condition have been derived.

8.2.2. Experimental Investigation

The experimental investigation of the AJ connection consisted of coupon tests, uniaxial tensile tests of the developed coupler and the SRBs, tests on applying preload to 3m-long SRBs using the coupler, and full-scale beam-to-column joint tests.

The coupon tests were conducted to determine the actual material properties of steel used for the beam-to-column joint test specimen and for input to the numerical simulations.

The developed coupler in this research is the most critical member of the AJ connection for connecting modules. Therefore, the coupler should be designed to behave within the elastic range until the high-tensile SRBs are fully yielded to the point of fracture. Uniaxial tensile tests on the coupler and SRBs confirmed that the coupler satisfies this requirement.

Current standards provide torque reference values for specific values of preload for high tensile bolts. However, it was uncertain whether these values are applicable to the 3m-SRB. Accordingly, preload was applied to the 3m-long SRB to determine the relationship between the torque and the resulting preload using both a torque wrench and a cordless impact wrench, as would be used on construction sites in practice. The torque wrench tests showed that the average torque coefficient (non-lubricated applications) of the 3m-SRB was 0.53. This was approximately 2.7 times greater than the theoretical torque coefficient for the general M24 high tensile bolt (featuring the same bolt size as the SRBs). In

the cordless impact wrench tests, the applied preload was 20-35% lower than the average preload induced in the torque wrench test by the same amount of torque. The relationship between torque and preload was irregular.

From the test of applying preload to SRB, it was also investigated whether the amount of applied preload in the lower column was reduced when the upper column was disassembled by loosening the coupler for reuse in the upper modules. As a result of the torque wrench and impact wrench tests, it was confirmed that the preload in the lower column was maintained after the upper column was disassembled from the lower column.

The beam-to-column joints were tested under monotonic loading and quasi-static cyclic loading to evaluate the seismic performance of the AJ connection and prove the validity of the analytical demand preload calculation for preventing gap-opening and slip at the connection. However, after completing the tests, inconsistencies in the experimental setup of the monotonic and quasi-static cyclic loading tests were identified, causing all the test specimens to fail due to gap-opening at the connection. As a result, the seismic performance of the preload beam-to-column joint could not be evaluated from the test results. Therefore, using finite element models calibrated against the monotonic test results, the seismic performance of the AJ joints was re-evaluated. Also, the analytical demand preload calculation for the connection was validated. While the results obtained from the tests have been recorded as-is, only the test results that were considered meaningful, which was the impact of gap-opening on the beam-to-column joint behaviour, are discussed as follows:

- (a) The gap-opening at the AJ connection changed the failure mechanism of the beam-to-column joints. As a result of the gap-opening at the connection in all specimens, only the ceiling beam reached its plastic moment capacity and failed by local buckling, while the floor beam only yielded slightly or not at all. Consequently, it was confirmed that the gap-opening at the AJ connection hinders the formation of plastic hinges in both the floor and ceiling beams, resulting in reduced ductility of the beam-to-column joint.
- (b) The SRBs within the columns were subjected to large tensile forces in addition to the preloads to resist the lateral load produced by gap-opening at the AJ connection. In particular, the threads of the SRBs near the coupler were subjected to additional bending by the rotational deformation of the coupler, caused by bearing against the plates as slip occurred at the connection. All the test results of the AJ joints showed that the bolt threads were bent. Therefore, gap-opening and slip should be minimised by applying a sufficient amount of preload to the SRBs, which are the critical members for connecting modules.
- (c) Regarding the effect of using floor and ceiling beams with different sizes (i.e. different bending stiffness EI) on the gap-opening at the AJ connection, the test results showed that the detrimental effect of gap-opening can be minimised if the two beams have the same size.

The impact of gap-opening at the AJ connection on the beam-to-column joint behaviour was consistent with the expected results from the analytical investigation.

8.2.3. Numerical Investigation

Beam-to-column joint

The seismic performance of the AJ joint inclusive of the AJ connection was re-evaluated using the finite element models calibrated against the beam-to-column joint tests. The AJ joints exhibited similar seismic performance to welded joints with ductile behaviour and the demand preload preventing gap-opening at the connection. They showed the intended ductile failure modes with plastic hinges developing in the beams, as required for a strong column-weak beam seismic MRF. Based on EN 1993-1-8, the AJ joints are classified as semi-rigid and full-strength joints. The AJ joints satisfied the requirements for structures of ductility class DCM in accordance with EC8-1, and for intermediate moment frame (IMF) according to AISC 341-16. In terms of ductility, the joint with different cross-sectional sizes for the floor and ceiling beams showed better performance than the joint having the same cross-sectional sizes for the two beams.

Parametric studies were conducted to investigate the influence of various parameters on the behaviour of the beam-to-column joint. As a result, the FE simulation confirmed that the design methods for calculating the demand preload for preventing gap-opening and slip at the AJ connection are accurate. In terms of the performance of the SRBs in columns, if the preload prevents the gap-opening and slip, the SRB maintains only the initial stress induced by preload until beams form plastic hinges and fail. However, the occurrence of gap-opening at the connection causes the SRBs in the upper and lower columns to be subjected to additional tensile stress by resisting the gap-opening. In addition, if slip occurs at the connection, the threads of the SRBs near the coupler are subjected to bending stress locally caused by rotation of the coupler when bearing against the plates. This bending stress can be significantly large if slip occurs in the early stage of loading. Therefore, even if the AJ connection is designed as a bearing type connection, it is necessary to minimise the influence of slip by inducing sufficient preload. Regarding the tolerance of the coupler-hole size at the connection, it did not negatively affect the stress in the bolt thread of the SRBs when slip occurred.

Additionally, in the case where the demand preload for preventing gap-opening is significantly high (e.g. for AJ connections located at lower storeys and subjected to large tensile axial forces under lateral load), two alternative solutions (Alt 1: Double coupler and SRBs in the AJ connection / Alt 2: installing a high-tensile bolt near the AJ connection) were considered. The FE simulations confirmed that both alternative solutions would be effective. In the case of Alt 1, it is noted that a larger column section may be required due to the increase in preload. For Alt 2, it is noted that the high-tensile bolt should be

installed at the location where gap-opening will occur. Otherwise, the bolt has virtually no effect on the gap-opening.

Frame analysis

The main purpose of the frame analysis was to investigate the impact of gap-opening and slip on the frame behaviour and to evaluate whether the demand preload, which was determined by the design method suggested in this thesis, can secure the safe performance of the frame against design wind and seismic loads. Before conducting the frame analysis, as a fundamental study, the effect of the inter-module connection stiffness on the behaviour of the frames was studied using simple spring models for the inter-module connection, and a minimum demand stiffness for the inter-module connections was recommended to make the assembly of modules behave as a compatible structure. In the case of the AJ connection, the recommended values were satisfied.

A 2D simplified FE joint model, which could describe the important features of the AJ connection, was developed. This model could accurately simulate the impact of gap-opening on the behaviour of a beam-to-column joint (i.e. the failure mechanism, reduction in stiffness and increase in the tensile force in the SRB). The joint model was verified by comparing it to the 3D detailed joint model. For the advanced frame analysis with the 2D FE joint model, a prototype structure for an 8-storey mid-rise modular building was designed with 100% steel-framed modules, i.e. without a reinforced concrete (RC) core, to maximise construction speed with the proposed AJ connection by reducing site-work. The 2D MRF composed of the short-side direction of modules was created from the prototype structure.

Regarding the wind analysis, the impact of slip is more critical than gap-opening at the connection. The connections at the upper storeys are more vulnerable to slip due to the larger wind load and smaller self-weight of the modules. Accumulated slip at the column-to-column connection according to the height of the building causes significant top displacement. Therefore, the connections should be designed as slip-resistant connections in mid-rise buildings. The FE simulation confirmed that the induced preload could provide the required amount of preload for preventing slip and gap-opening at the connections for wind load (i.e. elastic design). As a result, the MRF with AJ connections had a behaviour corresponding to the MRF with a rigid connection.

Regarding the seismic analysis, using the pushover analysis and the N2 method, the seismic performance of the MRF with the AJ connection was evaluated. The MRF exhibited ductile strong column-weak beam behaviour. Even though gap-opening occurred at the lowest storey shortly after the onset of yielding, most beams could form plastic hinges evenly throughout the frames while SRBs inside the columns resisted the lateral force. However, compared to the MRF with rigid connections, they had lower ductility and peak strength due to the effect of gap-opening.

Despite the occurrence of the gap-opening, the MRF composed of modules with the AJ connection demonstrated excellent seismic performance by meeting most of the performance objectives stipulated by the FEMA 365 criteria for SD/ULS and NC/CPLS. This is because of a significantly large elastic capacity caused by the overstrength of the modular building derived from using double floor and ceiling beams and using module units with the same size of beams and columns throughout the modular building.

In terms of the effect of differences in the flexural stiffness (EI) of the floor and ceiling beams, the MRF with the same cross-section size of floor and ceiling beams showed better seismic performance than the MRF with different sizes of beams (different EI). Even if gap-opening occurs, the former was less impacted, resulting in larger lateral stiffness, peak strength, and ductility with stable behaviour of SRBs in the inelastic behaviour than the latter. In addition, the former required approximately 30-70% smaller demand preload for the connections to resist the same design seismic load.

Regarding the connection modelling for structural analysis, the FE simulation confirmed that the demand preload determined using the analytical design method proposed in this thesis could prevent gap-opening and slip when subjected to the design wind and seismic loads (ULS). Therefore, AJ connections can be simply modelled as fixed connections under the assumption that the demand preload will be induced in all connections.

8.3. Recommendations for Future Studies

Recommended future research to improve the AJ connection (the post-tensioned type connection) is outlined below:

The relationship between the torque and resulting preload for the 3m-SRBs induced by a torque wrench or cordless impact wrench was determined from a small number of test specimens and limited conditions (non-lubricated applications). Particularly, the test results of the cordless impact wrench was irregular and therefore, additional experiments with many SRB specimens to obtain a stable torque coefficient by considering different conditions (e.g. lubricated applications, using impact wrenches manufactured by several companies, various sizes of SRBs, etc.) are required. Also, the tests were performed by placing columns horizontally in this thesis, which is a different condition from reality. Therefore, it is recommended to conduct tests using the full-scale frame modules where the columns are upright for determining the torque coefficient for 3m-SRBs in the same condition as on an actual construction site.

In this thesis, the seismic performance of the AJ connection was investigated by conducting only the exterior RHS beam-to-RHS column joint. Further tests are required for the interior beam-to-column

joints under monotonic and cyclic loading to fully understand the characteristics and performance of the AJ connection. In addition, it is recommended to investigate the optimal joint (e.g. H beam-to-H column or H beam-to-RHS column) with the AJ connection to obtain a more efficient system with enhanced seismic performance.

In a post-tensioned type connection like the AJ connection, the axial forces (i.e. tensile or compressive forces induced by lateral forces) acting on the columns are closely related to the behaviour of the connection and demand preload. However, the axial force cannot be included in the beam-to-column joint test unless artificially applied by an actuator. If applying the axial force to the column using an actuator, the test set-up becomes quite complicated considering the additional application of preload to the column. Therefore, it is recommended to conduct experiments using full-scale frame modules, which can induce all forces acting on the AJ connection, to evaluate the performance of the AJ connection most accurately.

Regarding the numerical simulation work, the seismic performance of only an 8-storey mid-rise MRF frame with the AJ connection was evaluated using nonlinear static (pushover) analysis. Therefore, further evaluation of the seismic performance of the AJ connection is required using advanced nonlinear dynamic analysis (e.g. time history analysis and incremental dynamic analysis) considering various systems (e.g. high-rise buildings, modular frames consisting of braced frame modules, hybrid structural systems etc.) for application of the AJ connection to various structural types of modular buildings in seismic and non-seismic areas.

References

- [1] HICKORY. "One9 Apartment Tower," from <https://www.hickory.com.au/news/one9-apartment-tower-constructed-in-only-five-days/>; 2014.
- [2] C. D. Annan, M. A. Youssef, and M. H. Ei-Naggar, "Effect of directly welded stringer-to-beam connections on the analysis and design of modular steel building floors," *Advances in Structural Engineering*, vol. 12, no. 3, pp. 373-383, 2009.
- [3] Z. Chen, J. Liu, and Y. Yu, "Experimental study on interior connections in modular steel buildings," *Engineering Structures*, vol. 147, pp. 625-638, 2017.
- [4] G. Zhang, L. H. Xu, and Z. X. Li, "Development and seismic retrofit of an innovative modular steel structure connection using symmetrical self-centering haunch braces," *Engineering Structures*, vol. 229, 2021.
- [5] Z. Chen, J. Wang, J. Liu, and K. Khan, "Seismic behavior and moment transfer capacity of an innovative self-locking inter-module connection for modular steel building," *Engineering Structures*, vol. 245, 2021.
- [6] R. Sanches, O. Mercan, and B. Roberts, "Experimental investigations of vertical post-tensioned connection for modular steel structures," *Engineering Structures*, vol. 175, pp. 776-789, 2018.
- [7] A. W. Lacey, W. Chen, H. Hao, and K. Bi, "Simplified structural behaviours of post-tensioned inter-module connection for modular buildings," *Journal of Constructional Steel Research*, vol. 175, 2020.
- [8] W. Pan, and R. Sidwell, "Demystifying the cost barriers to offsite construction in the UK," *Construction Management and Economics*, vol. 29, no. 11, pp. 1081-1099, 2011.
- [9] A. Rogan, R. Lawson, and N. Bates-Brkljac, "Value and benefits assessment of modular construction," *Steel Construction Institute, Ascot, UK*, 2000.
- [10] R. E. Smith, *Prefab architecture: A guide to modular design and construction*: John Wiley & Sons, 2010.
- [11] X. Liu, A. Zhang, J. Ma, Y. Tan, and Y. Bai, "Design and Model Test of a Modularized Prefabricated Steel Frame Structure with Inclined Braces," *Advances in Materials Science and Engineering*, 2015.
- [12] E. M. Generalova, V. P. Generalov, and A. A. Kuznetsova, "Modular Buildings in Modern Construction," in *Procedia Engineering*, 2016, pp. 167-172.
- [13] D. Lopez, and T. M. Froese, "Analysis of Costs and Benefits of Panelized and Modular Prefabricated Homes," in *Procedia Engineering*, 2016, pp. 1291-1297.
- [14] Quebec Wood Export Bureau. "Light Wood Frame," from <https://quebecwoodexport.com/en/products/wood-construction/light-wood-frame/wall-panels/>; 2017.
- [15] SHOWHOUSE. "Why modular homes have failed to gain ground in UK," from <https://www.showhouse.co.uk/news/why-modular-homes-have-failed-to-gain-ground-in-uk/>; 2017.
- [16] R. B. Richard, "Industrialised building systems: Reproduction before automation and robotics," in *Automation in Construction*, 2005, pp. 442-451.
- [17] M. O'Brien, R. Wakefield, and Y. Beliveau, "Industrializing the Residential Construction Site," no. Department of Housing and Urban Development, Office of Policy Development and Research, Washington, DC, 2000.

- [18] L. R. Kawecki, *Environmental performance of modular fabrication: calculating the carbon footprint of energy used in the construction of a modular home*: Arizona State University, 2010.
- [19] A. Baldwin, C. S. Poon, L. Y. Shen, S. Austin, and I. Wong, "Designing out waste in high-rise residential buildings: Analysis of precasting methods and traditional construction," *Renewable Energy*, vol. 34, no. 9, pp. 2067-2073, 2009.
- [20] V. W. Y. Tam, C. M. Tam, S. X. Zeng, and W. C. Y. Ng, "Towards adoption of prefabrication in construction," *Building and Environment*, vol. 42, no. 10, pp. 3642-3654, 2007.
- [21] R. M. Lawson, R. G. Ogden, and R. Bergin, "Application of modular construction in high-rise buildings," *Journal of Architectural Engineering*, vol. 18, no. 2, pp. 148-154, 2012.
- [22] A. Manalo, "Structural behaviour of a prefabricated composite wall system made from rigid polyurethane foam and Magnesium Oxide board," *Construction and Building Materials*, vol. 41, pp. 642-653, 2013.
- [23] C. Mao, Q. Shen, L. Shen, and L. Tang, "Comparative study of greenhouse gas emissions between off-site prefabrication and conventional construction methods: Two case studies of residential projects," *Energy and Buildings*, vol. 66, pp. 165-176, 2013.
- [24] I. Won, Y. Na, J. T. Kim, and S. Kim, "Energy-efficient algorithms of the steam curing for the in situ production of precast concrete members," *Energy and Buildings*, vol. 64, pp. 275-284, 2013.
- [25] L. Aye, T. Ngo, R. H. Crawford, R. Gammampila, and P. Mendis, "Life cycle greenhouse gas emissions and energy analysis of prefabricated reusable building modules," *Energy and Buildings*, vol. 47, pp. 159-168, 2012.
- [26] L. Jaillon, C. S. Poon, and Y. H. Chiang, "Quantifying the waste reduction potential of using prefabrication in building construction in Hong Kong," *Waste Management*, vol. 29, no. 1, pp. 309-320, 2009.
- [27] T. Gunawardena, T. Ngo, P. Mendis, L. Aye, and R. Crawford, "Time-efficient post-disaster housing reconstruction with prefabricated modular structures," *Open House International*, vol. 39, no. 3, pp. 59-69, 2014.
- [28] M. Kamali, and K. Hewage, "Life cycle performance of modular buildings: A critical review," *Renewable and Sustainable Energy Reviews*, vol. 62, pp. 1171-1183, 2016.
- [29] FASTCOMPANY. "The Stack Modular Housing in Manhattan," from <https://www.fastcompany.com/1673057/this-prefab-building-is-a-first-for-new-york>; 2013.
- [30] Tide Construction Ltd. "Apex House," from <https://tideconstruction.co.uk/projects/apex-house-wembley/>; 2017.
- [31] Q. T. Nguyen, T. Ngo, P. Tran, P. Mendis, M. Zobec, and L. Aye, "Fire performance of prefabricated modular units using organoclay/glass fibre reinforced polymer composite," *Construction and Building Materials*, vol. 129, pp. 204-215, 2016.
- [32] D. Martins, M. Proença, J. R. Correia, J. Gonilha, M. Arruda, and N. Silvestre, "Development of a novel beam-to-column connection system for pultruded GFRP tubular profiles," *Composite Structures*, vol. 171, pp. 263-276, 2017.
- [33] R. M. Lawson, and R. G. Ogden, "'Hybrid' light steel panel and modular systems," *Thin-Walled Structures*, vol. 46, no. 7-9, pp. 720-730, 2008.
- [34] J. D. R. Roigé. "Truck carrying a module prefabricated concrete housing in Barcelona," from <https://www.alamy.com/stock-photo-truck-carrying-a-module-prefabricated-concrete-housing-in-barcelona-83449145.html>; 2015.
- [35] Blumer Lehmann. "Modular timber construction," from <https://www.blumer-lehmann.com/en/implement-construction-projects/construction/modular-construction.html>; 2022.

- [36] C. D. Annan, M. A. Youssef, and M. H. El Naggar, "Experimental evaluation of the seismic performance of modular steel-braced frames," *Engineering Structures*, vol. 31, no. 7, pp. 1435-1446, 2009.
- [37] H. Moghimi, and H. R. Ronagh, "Better connection details for strap-braced CFS stud walls in seismic regions," *Thin-Walled Structures*, vol. 47, no. 2, pp. 122-135, 2009.
- [38] T. S. Eom, T. H. Ha, B. H. Cho, and T. H. Kim, "Cyclic loading tests on framed stud walls with strap braces and steel sheathing," *Journal of Structural Engineering (United States)*, vol. 141, no. 7, 2015.
- [39] L. Fiorino, M. T. Terracciano, and R. Landolfo, "Experimental investigation of seismic behaviour of low dissipative CFS strap-braced stud walls," *Journal of Constructional Steel Research*, vol. 127, pp. 92-107, 2016.
- [40] R. Serrette, and K. Ogunfunmi, "Shear resistance of gypsum-sheathed light-gauge steel stud walls," *Journal of Structural Engineering*, vol. 122, no. 4, pp. 383-389, 1996.
- [41] E. Baran, and C. Alica, "Behavior of cold-formed steel wall panels under monotonic horizontal loading," *Journal of Constructional Steel Research*, vol. 79, pp. 1-8, 2012.
- [42] M. Zeynalian, and H. R. Ronagh, "Seismic performance of cold formed steel walls sheathed by fibre-cement board panels," *Journal of Constructional Steel Research*, vol. 107, pp. 1-11, 2015.
- [43] V. Macillo, L. Fiorino, and R. Landolfo, "Seismic response of CFS shear walls sheathed with nailed gypsum panels: Experimental tests," *Thin-Walled Structures*, vol. 120, pp. 161-171, 2017.
- [44] D. Farnsworth, "Modular tall building design at Atlantic Yards B2," *Council on Tall Buildings and Urban Habitat*, pp. 492-499, 2014.
- [45] S. Hong, "Structural design guidelines for middle to high-rise modular buildings for architects and engineers," *Review of Architecture and Building Science*, vol. 58, no. 5, pp. 15-19, 2014.
- [46] P. Andrade, M. Veljkovic, J. Lundholm, and T. Heistermann, "Innovative construction of student residences: Frameup concept," in *Nordic Steel Construction Conference*, 2015, pp. 199-200.
- [47] M. Lawson, "Building design using modules," *New steel construction*, vol. 15, no. 9, pp. 28-29, 2007.
- [48] INHABITANT. "Europe's tallest modular tower snaps together in north London-Apex House," from <https://inhabitat.com/europes-tallest-modular-tower-snaps-together-in-north-london/>; 2017.
- [49] M. J. Hough, and R. M. Lawson, "Design and construction of high-rise modular buildings based on recent projects," in *Proceedings of the institution of civil engineers-civil engineering*, 2019, pp. 37-44.
- [50] Fieldcondition. "Atlantic Yards B2 Tower," from <http://fieldcondition.com/blog/2014/6/23/atlantic-yards-b2-tower>; 2014.
- [51] SkyCiv. "Steel Connection for Beams and Columns," from <https://skyciv.com/technical/steel-connection-design-for-beams-and-columns/>; 2022.
- [52] Y.-H. Choi, H.-C. Lee, and J.-K. Kim, "Seismic Performance Assessment of a Composite Modular System Considering Stiffness of Connections," *Journal of the Korea institute for structural maintenance and inspection*, vol. 23, no. 1, pp. 9-18, 2019.
- [53] T. Gunawardena, T. Ngo, P. Mendis, and J. Alfano, "Innovative Flexible Structural System Using Prefabricated Modules," in *Journal of Architectural Engineering*, 2016.
- [54] T. Gunawardena, "Behaviour of prefabricated modular buildings subjected to lateral loads," PhD thesis, The University of Melbourne, 2016.
- [55] S. Lee, J. Park, E. Kwak, S. Shon, C. Kang, and H. Choi, "Verification of the seismic performance of a rigidly connected modular system depending on the shape and size of the ceiling bracket," *Materials*, vol. 10, no. 3, 2017.

- [56] S. Lee, J. Park, S. Shon, and C. Kang, "Seismic performance evaluation of the ceiling-bracket-type modular joint with various bracket parameters," *Journal of Constructional Steel Research*, vol. 150, pp. 298-325, 2018.
- [57] B. H. Cho, J. S. Lee, H. Kim, and D. J. Kim, "Structural performance of a new blind-bolted frame modular beam-column connection under lateral loading," *Applied Sciences (Switzerland)*, vol. 9, no. 9, 2019.
- [58] E. F. Deng, L. Zong, Y. Ding, X. M. Dai, N. Lou, and Y. Chen, "Monotonic and cyclic response of bolted connections with welded cover plate for modular steel construction," *Engineering Structures*, vol. 167, pp. 407-419, 2018.
- [59] E. F. Deng, L. Zong, Y. Ding, and Y. B. Luo, "Seismic behavior and design of cruciform bolted module-to-module connection with various reinforcing details," *Thin-Walled Structures*, vol. 133, pp. 106-119, 2018.
- [60] Z. Chen, J. Liu, Y. Yu, C. Zhou, and R. Yan, "Experimental study of an innovative modular steel building connection," *Journal of Constructional Steel Research*, vol. 139, pp. 69-82, 2017.
- [61] J. Peng, C. Hou, and L. Shen, "Numerical simulation of weld fracture using cohesive interface for novel inter-module connections," *Journal of Constructional Steel Research*, vol. 174, 2020.
- [62] J. Peng, C. Hou, and L. Shen, "Lateral resistance of multi-story modular buildings using tenon-connected inter-module connections," *Journal of Constructional Steel Research*, vol. 177, 2021.
- [63] J. Dhanapal, H. Ghaednia, S. Das, and J. Velocci, "Structural performance of state-of-the-art VectorBloc modular connector under axial loads," *Engineering Structures*, vol. 183, pp. 496-509, 2019.
- [64] J. Dhanapal, H. Ghaednia, S. Das, and J. Velocci, "Behavior of thin-walled beam-column modular connection subject to bending load," *Thin-Walled Structures*, vol. 149, 2020.
- [65] B. Hajimohammadi, S. Das, H. Ghaednia, and J. Dhanapal, "Structural performance of registration pin connection in VectorBloc modular construction system," *Journal of Constructional Steel Research*, vol. 197, 2022.
- [66] Z. Chen, Y. Liu, X. Zhong, and J. Liu, "Rotational stiffness of inter-module connection in mid-rise modular steel buildings," *Engineering Structures*, vol. 196, 2019.
- [67] Z. Chen, J. Wang, J. Liu, and Z. Cong, "Tensile and shear performance of rotary inter-module connection for modular steel buildings," *Journal of Constructional Steel Research*, vol. 175, 2020.
- [68] Y. Liu, Z. Chen, J. Liu, Y. Bai, X. Zhong, and X. Wang, "Lateral stiffness evaluation on corner-supported thin walled modular steel structures," *Thin-Walled Structures*, vol. 157, 2020.
- [69] Y. Liu, Z. Chen, J. Liu, X. Zhong, X. Liu, and J. Jiang, "Effects of various reinforcing intra-module connection details on seismic behavior of corner-supported steel modular buildings," *Structures*, vol. 40, pp. 555-570, 2022.
- [70] J. Liu, Z. Chen, Y. Liu, Y. Bai, and X. Zhong, "Full-scale corner-supported modular steel structures with vertical inter-module connections under cyclic loading," *Journal of Building Engineering*, vol. 44, 2021.
- [71] X. M. Dai, L. Zong, Y. Ding, and Z. X. Li, "Experimental study on seismic behavior of a novel plug-in self-lock joint for modular steel construction," *Engineering Structures*, vol. 181, pp. 143-164, 2019.
- [72] Z. Chen, H. Li, A. Chen, Y. Yu, and H. Wang, "Research on pretensioned modular frame test and simulations," *Engineering Structures*, vol. 151, pp. 774-787, 2017.
- [73] Y. Yu, Z. Chen, and A. Chen, "Experimental study of a pretensioned connection for modular buildings," *Steel and Composite Structures*, vol. 31, no. 3, pp. 217-232, 2019.

- [74] A. W. Lacey, W. Chen, H. Hao, K. Bi, and F. J. Tallowin, "Shear behaviour of post-tensioned inter-module connection for modular steel buildings," *Journal of Constructional Steel Research*, vol. 162, 2019.
- [75] Y. S. Chua, J. Y. R. Liew, and S. D. Pang, "Modelling of connections and lateral behavior of high-rise modular steel buildings," *Journal of Constructional Steel Research*, vol. 166, 2020.
- [76] D. H. Shin, and H. J. Kim, "Load-Carrying Mechanism of Post-Tensioned Inter-Module Connections Implemented into Modular Steel Buildings," *Journal of Earthquake Engineering*, 2022.
- [77] C. D. Annan, M. A. Youssef, and M. H. El Naggar, "Seismic vulnerability assessment of modular steel buildings," *Journal of Earthquake Engineering*, vol. 13, no. 8, pp. 1065-1088, 2009.
- [78] M. Lawson, R. Ogden, and C. I. Goodier, *Design in modular construction*: CRC Press Boca Raton, FL, 2014.
- [79] S. S. Lee, K. S. Park, S. Y. Hong, and K. W. Bae, "Behavior of C-Shaped beam to square hollow section column connection in modular frame," *Journal of Korean Society of Steel Construction*, vol. 27, no. 5, pp. 471-481, 2015.
- [80] A. W. Lacey, W. Chen, H. Hao, and K. Bi, "New interlocking inter-module connection for modular steel buildings: Simplified structural behaviours," *Engineering Structures*, vol. 227, 2021.
- [81] A. W. Lacey, W. Chen, H. Hao, and K. Bi, "New interlocking inter-module connection for modular steel buildings: Experimental and numerical studies," *Engineering Structures*, vol. 198, 2019.
- [82] C. Ding, Y. Bai, C. Qiu, C. Wan, and X. L. Zhao, "Steel bolted flanged connections in tension: Effects of stiffener configurations," *Thin-Walled Structures*, vol. 154, 2020.
- [83] R. Ma, J. Xia, H. Chang, B. Xu, and L. Zhang, "Experimental and numerical investigation of mechanical properties on novel modular connections with superimposed beams," *Engineering Structures*, vol. 232, 2021.
- [84] E. F. Deng, J. B. Yan, Y. Ding, L. Zong, Z. X. Li, and X. M. Dai, "Analytical and numerical studies on steel columns with novel connections in modular construction," *International Journal of Steel Structures*, vol. 17, no. 4, pp. 1613-1626, 2017.
- [85] CEN, "EN 1993-1-8: Eurocode 3: Design of steel structures," *Part 1-8: Design of joints*, European Committee for Standardization, 2005.
- [86] CEN, "EN 1998-1: Eurocode 8: Design of structures for earthquake resistance," *Part 1: General rules, seismic actions and rules for buildings*, European Committee for Standardization, 2004.
- [87] C. Lee, "Seismic Design of Steel Buildings per Capacity Design Concept," *J. of Arch. Inst. of Korea, AIK*, vol. 52, no. 10, pp. 32-37, 2008.
- [88] CEN, "EN 1993-1-1: Eurocode 3: Design of steel structures," *Part 1-1: General rules and rules for buildings*, European Committee for Standardization, 2005.
- [89] KSA, "KS B 0801 Test pieces for tensile test for metallic materials," Korean Standards Association, 2007.
- [90] KSA, "KS B 0802 Method of tensile test for metallic materials ", Korean Standards Association, 2003.
- [91] KSA, "KS D 3568 Carbon steel square pipes for general structural purposes," Korean Standards Association, 2018.
- [92] KSA, "KS D 3503 Rolled steels for general structure," Korean Standards Association, 2018.
- [93] ASTM, "A193/A193M-20 Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications," 2016.

- [94] KSA, "KS D 3752 Carbon steels for machine structural use," Korean Standards Association, 2019.
- [95] KSA, "KS B 1010 Set of high strength hexagon bolt, hexagon nut and plain washers for friction grip joints ", Korean Standards Association, 2009.
- [96] JSA, "JIS B 1186 Sets of high strength hexagon bolt, hexagon nut and plain washers for friction grip joints," Japanese Standards Association, 2013.
- [97] ASTM, "ASTM F3125/F3125M-19 Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength," 2019
- [98] "ISO 15835-1 Steels for the reinforcement of concrete — Reinforcement couplers for mechanical splices of bars," *Part 1: Requirements*, International Organization for Standardization, 2018.
- [99] KSA, "KS D 0249 Method of inspection for mechanical splicing joint of bars for concrete reinforcement," Korean Standards Association, 2019.
- [100] CEN, "EN 1090-2 Execution of steel structures and aluminium structures " *Technical requirements for steel structures*, European Committee for Standardization, 2018.
- [101] "Korea Building Code," Ministry of Land, Infrastructure and Transport of Korea, 2016.
- [102] J. Bickford, *Introduction to the design and behavior of bolted joints*: CRC Press, 2007.
- [103] Q. Yu, X. Yang, and H. Zhou, "An experimental study on the relationship between torque and preload of threaded connections," *Advances in Mechanical Engineering*, vol. 10, no. 8, 2018.
- [104] J. Bickford, *Handbook of bolts and bolted joints*, New York, USA: CRC press, 1998.
- [105] R. C. Juvinall, and K. M. Marshek, *Fundamentals of Machine Component Design*, New York: Wiley, 2000.
- [106] S. A. Nassar, P. H. Matin, and G. C. Barber, "Thread friction torque in bolted joints," *ASME J. Pressure Vessel Technol.* , vol. 127, no. 4, pp. 7, November 2005.
- [107] H.-Y. Hwang, "Bolted joint torque setting using numerical simulation and experiments," *Journal of Mechanical Science and Technology*, vol. 27, no. 5, pp. 1361-1371, 2013.
- [108] D. Croccolo, M. De Agostinis, and N. Vincenzi, "Influence of tightening procedures and lubrication conditions on titanium screw joints for lightweight applications," *Tribology International*, vol. 55, pp. 68-76, 2012.
- [109] Q. Yu, H. Zhou, and L. Wang, "Finite element analysis of relationship between tightening torque and initial load of bolted connections," *Advances in Mechanical Engineering*, vol. 7, no. 5, 2015.
- [110] ANSI/AISC, "ANSI/AISC 360-10. Specification for structural steel buildings," American Institute of Steel Construction, 2010.
- [111] AISC, "ANSI/AISC 341-10 Seismic provisions for structural steel buildings," American Institute of Steel Construction, 2010.
- [112] Abaqus, "Abaqus 6.14," *Dassault Systemes Simulia Corporation, Providence, RI, USA*, 2014.
- [113] CEN, "EN 1993-1-5: Eurocode 3: Design of steel structures," *Part 1-5: Plated structural elements*, European Committee for Standardization, 2005.
- [114] Y. Ling, "Uniaxial true stress-strain after necking," *AMP Journal of technology*, vol. 5, no. 1, pp. 37-48, 1996.
- [115] K.-J. Shin, Y.-J. Kim, Y.-S. Oh, and T.-S. Moon, "Behavior of welded CFT column to H-beam connections with external stiffeners," *Engineering Structures*, vol. 26, no. 13, pp. 1877-1887, 2004.
- [116] C. Vulcu, A. Stratan, A. Ciutina, and D. Dubina, "Beam-to-CFT high-strength joints with external diaphragm. II: Numerical simulation of joint behavior," *Journal of Structural Engineering*, vol. 143, no. 5, 2017.

- [117] C.-C. Chou, C.-C. Wu, C.-K. Jao, and Y.-Y. Wang, "Weakened and strengthened steel moment connections," in 4th International Conference on Earthquake Engineering, 2006.
- [118] G. Shi, M. Wang, Y. Bai, F. Wang, Y. Shi, and Y. Wang, "Experimental and modeling study of high-strength structural steel under cyclic loading," *Engineering Structures*, vol. 37, pp. 1-13, 2012.
- [119] R. Kiamanesh, A. Abolmaali, and M. Ghassemieh, "The effect of stiffeners on the strain patterns of the welded connection zone," *Journal of Constructional Steel Research*, vol. 66, no. 1, pp. 19-27, 2010.
- [120] E. B. Schäfer D, Amlung L, Vayas I, Karlos V, Spiliopoulos A, Lippe M, Kubon Z, Kander L., *Modern Plastic Design for Steel Structures (Final Report)*, Directorate-General for Research, 2010.
- [121] E. Cho, J. Hyun, and S. W. Han, "Estimation of Combined Hardening Model Parameter Values for Korean Steel Grades," in The 2020 World Congress on Advances in Civil, Environmental, & Materials Research/The 2020 Structures Congress, 2020, pp. 1-9.
- [122] L. Marques, L. S. da Silva, R. Greiner, C. Rebelo, and A. Taras, "Development of a consistent design procedure for lateral-torsional buckling of tapered beams," *Journal of Constructional Steel Research*, vol. 89, pp. 213-235, 2013.
- [123] W. Schneider, "Stimulating equivalent geometric imperfections for the numerical buckling strength verification of axially compressed cylindrical steel shells," *Computational Mechanics*, vol. 37, no. 6, pp. 530-536, 2006.
- [124] R. Ofner, "Buckling check of members and frames based on numerical simulations," in Steel Structures and Bridges, 2003, pp. 469-474.
- [125] T. Wilkinson, and G. Hancock, "Predicting the rotation capacity of cold-formed RHS beams using finite element analysis," *Journal of Constructional Steel Research*, vol. 58, no. 11, pp. 1455-1471, 2002.
- [126] T. J. Wilkinson, "The plastic behaviour of cold-formed rectangular hollow sections," PhD thesis, The University of Sydney, 1999.
- [127] EKO LASER. "Laser cutting," from <https://ekolaser.pl/en/laser-cutting/>; 2022.
- [128] The Australian Manufacturing Technology Institute Limited. "Determining the best process for metal cutting," from <https://amtil.com.au/determining-the-best-process-for-metal-cutting/>; 2015.
- [129] GEOMIQ. "Sheet metal fabrication design guide-Laser cutting tolerance," from <https://geomiq.com/sheet-metal-design-guide/>; 2022.
- [130] Cutting Technologies Ltd. "Mild steel-Laser cutting tolerance," from <https://www.cut-tec.co.uk/materials/laser-cutting/mild-carbon-steel/mild-steel/#.Yt6RHXbP2Uk>; 2022.
- [131] R. T. Barrett, *Fastener design manual*, Lewis Research Center, Cleveland, Ohio, United States: National Aeronautics and Space Administration, 1990.
- [132] R. Landolfo, F. M. Mazzolani, D. Dubina, L. S. da Silva, and M. D'Aniello, "Design of Steel Structures for Buildings in Seismic Areas: Eurocode 8: Design of Structures for Earthquake Resistance. Part 1-1-General Rules, Seismic Actions and Rules for Buildings," 2017.
- [133] European Convention for Constructional Steelwork. Steelwork, *Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads*, Brussels, Belgium: ECCS General Secretariat, 1986.
- [134] G. Shi, X. Chen, and D. Wang, "Experimental study of ultra-large capacity end-plate joints," *Journal of Constructional Steel Research*, vol. 128, pp. 354-361, 2017.
- [135] M. Fardis, E. Carvalho, A. Elnashai, E. Faccioli, P. Pinto, and A. Plumier, *Designers' Guide to EN 1998-1 and 1998-5. Eurocode 8: Design Provisions for Earthquake Resistant Structures*: Thomas Telford Publishing, 2005.

- [136] AISC, "ANSI/AISC 341-16 Seismic provisions for structural steel buildings," American Institute of Steel Construction, 2016.
- [137] FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, Washington DC, USA: Federal Emergency Management Agency, 2000.
- [138] J.-F. Wang, L.-H. Han, and B. Uy, "Hysteretic behaviour of flush end plate joints to concrete-filled steel tubular columns," *Journal of Constructional Steel Research*, vol. 65, no. 8-9, pp. 1644-1663, 2009.
- [139] J. Wang, L. Zhang, and B. Spencer Jr, "Seismic response of extended end plate joints to concrete-filled steel tubular columns," *Engineering Structures*, vol. 49, pp. 876-892, 2013.
- [140] KSA, "KS B ISO 273 Diameter of clearance holes and counterbores for bolts and screws," Korean Standards Association, 2010.
- [141] P. Fajfar, "A nonlinear analysis method for performance-based seismic design," *Earthquake spectra*, vol. 16, no. 3, pp. 573-592, 2000.
- [142] T. H. G. Megson, *Structural and stress analysis*: Butterworth-Heinemann, 2019.
- [143] FEMA 355C, *State of the art report on systems performance of steel moment frames subject to earthquake ground shaking (SAC Joint Venture)*, 2000.
- [144] FEMA 356, *Prestandard and commentary for the seismic rehabilitation of buildings*, Washington DC, USA: Federal Emergency Management Agency and American Society of Civil Engineers, 2000.
- [145] CEN, "EN 1998-3: Eurocode 8: Design of structures for earthquake resistance," *Part 3: Assessment and retrofitting of buildings*, European Committee for Standardization, 2005.
- [146] I. Computers & Structures, "ETABS 17—Integrated Building Design Software," 2017.
- [147] CEN, "EN 1990: Eurocode: Basis of structural design," European Committee for Standardization, 2002.
- [148] CEN, "EN 1991: Eurocode 1: Actions on structures " *Part 1-1: General actions: Densities, self-weight, imposed loads for buildings*, European Committee for Standardization, 2002.
- [149] "AS/NZS 1170.2: Structural design actions," *Part2: Wind actions*, Standards Australia, 2021.
- [150] "AS/NZS 1170.0: Structural design actions," *Part0: General principles*, Standards Australia, 2002.
- [151] M. T. Naqash, G. De Matteis, and A. De Luca, "Seismic design of steel moment resisting frames- European versus American practice," *NED University Journal of Research*, vol. 9, no. 2, pp. 45-60, 2012.
- [152] CEN, "EN 14399-1: High-strength structural bolting assemblies for preloading," *Part 1: General requirements*, European Committee for Standardization, 2015.
- [153] CEN, "EN ISO 898-1: Mechanical properties of fasteners made of carbon steel and alloy steel," *Part 1: Bolts, screws and studs with specified property classes - Coarse thread and fine pitch thread*, European Committee for Standardization, 2013.

Appendix A

Example calculation of reduced preload due to the weight effect of upper modules

Example: 12-storey modular building

The 12-storey modular building is constructed using the same size of module and SRBs. The demand preload P_g of 150kN is applied to all AJ connections in the building.

Condition:

1. Module size: 3.3m (W) X 3m (H) X 6.6m (L) – four columns in one module
2. Weight per module (W_M) : 136kN (with thickness of concrete floor slab 150mm)
3. Member: Column: RHS 200 X 100 X 12, ($A_c=6369\text{mm}^2$)
SRB: Bolt shank diameter 25mm, ($A_{sh}=491\text{ mm}^2$)

1. Reduction in preload from the initial preload ($P_g=150\text{kN}$) in column-1F and column-2F after constructing the 12-storey modular building

The axial stiffness of column k_M and bolt k_B (according to Eq. 3.44 and Eq. 3.45)

$$k_M \approx k_{cot} = \frac{E \cdot A_c}{L_c} = \frac{210000 \cdot 6469}{3000} = 452830\text{N/mm} \quad \text{for all columns (from 1F to 12F)}$$

$$k_B \approx k_{sh} = \frac{E \cdot A_{sh}}{L_{sh}} = \frac{210000 \cdot 491}{3000} = 34370\text{N/mm} \quad \text{for SRBs (from 1F to 12F)}$$

The amount of reduced preload P_R (according to Eq. 3.53)

$$\text{Weight of upper module acting on each column} \quad W = \frac{W_M}{4} = \frac{136\text{kN}}{4} = 34\text{ kN}$$

For column-2F :

$$P_{R,2} = \frac{k_{B,2}}{k_{M,2}} \times \sum_{n=3}^{12} W_n = \frac{k_{B,2}}{k_{M,2}} \times (W_3 + W_4 + \dots + W_{12}) = \frac{34370}{452830} \times (10 \times 34\text{kN}) = 26\text{ kN}$$

For column-1F :

$$P_{R,1} = \frac{k_{B,1}}{k_{M,1}} \times \sum_{n=2}^{12} W_n = \frac{k_{B,1}}{k_{M,1}} \times (W_2 + W_3 + \dots + W_{12}) = \frac{34370}{452830} \times (11 \times 34\text{kN}) = 28\text{ kN}$$

* After constructing the 12-storey modular building, the initially applied preload of 150kN to column-1F and column-2F are reduced by 19% (28 kN) and 17% (26 kN), respectively.

2. Remaining preload (net preload) in column-1F and column-2F

(according to Eq. 3.54)

$$\text{For column-2F: } P_{n,2} = P_{i,2} - P_{R,2} = 150 \text{ kN} - 26 \text{ kN} = 124 \text{ kN}$$

$$\text{For column-1F: } P_{n,1} = P_{i,1} - P_{R,1} = 150 \text{ kN} - 28 \text{ kN} = 122 \text{ kN}$$

3. Clamping forces provided by net preloads at connection-1F (refer to Figure 3.32) between column-1F and column-2F

$$C_{1F,2} = C_{1F,3} = 124 \text{ kN} \quad (\leftarrow \text{reduced from } 150 \text{ kN})$$

$$C_{1F,1} = 122 \text{ kN} \quad (\leftarrow \text{reduced from } 150 \text{ kN})$$

* Initially, the same amount of preload is applied to column-1F and column-2F, and it is expected the connection will have uniform clamping forces. However, due to the weight effect of upper modules, the clamping forces are reduced and non-uniform at the connection.

4. In order to achieve the clamping force ($P_g = 150 \text{ kN}$) at the connection-1F after constructing the 12-storey modular building, the required initial preloads for column-1F and column-2F are as follows:

(according to Eq. 3.55)

$$\text{For column-2F: } P_{F,g2} = P_g + P_{R,2} = 150 \text{ kN} + 26 \text{ kN} = 176 \text{ kN}$$

$$\text{For column-1F: } P_{F,g1} = P_g + P_{R,1} = 150 \text{ kN} + 28 \text{ kN} = 178 \text{ kN}$$

* As a result, the connection-1F will have the uniform clamping force of 150 kN

$$C_{1F,1} = C_{1F,2} = C_{1F,3} = 150 \text{ kN} \quad (\text{refer to Figure 3.32})$$

*** In case all conditions are the same but the thickness of the column is reduced by 4mm to RHS 200 X 100 X 8, the reduced preload becomes significantly larger because of greater compressive axial deformation of the column under the same weight effect.**

Results :

$$\text{For column-2F: } P_{R,2} = 37 \text{ kN} \quad (\text{corresponding to } 25\% \text{ of the initial preload } P_g \text{ of } 150 \text{ kN})$$

$$\text{For column-1F: } P_{R,1} = 41 \text{ kN} \quad (\text{corresponding to } 27\% \text{ of the initial preload } P_g \text{ of } 150 \text{ kN})$$

*** In case all conditions are the same but the preload of 150kN is applied by two SRBs of the same diameter (75kN per bolt) in the column RHS 200X100X12. The reduction in preload becomes significantly larger as it doubles under the same weight effect.**

For column-2F:

$$P_{R,2} = \frac{2 \times k_{B,2}}{k_{M,2}} \times \sum_{n=3}^{12} W_n = \frac{2 \times k_{B,2}}{k_{M,2}} \times (W_3 + W_4 + \dots + W_{12}) = \frac{2 \times 34370}{452830} \times (10 \times 34\text{kN}) = 52 \text{ kN}$$

For column-1F:

$$P_{R,1} = \frac{2 \times k_{B,1}}{k_{M,1}} \times \sum_{n=2}^{12} W_n = \frac{k_{B,1}}{k_{M,1}} \times (W_2 + W_3 + \dots + W_{12}) = \frac{2 \times 34370}{452830} \times (11 \times 34\text{kN}) = 57 \text{ kN}$$

*** After constructing the 12-storey modular building, the initially applied preload of 150kN in column-1F and column-2F is reduced by 38% (57 kN) and 35% (52 kN), respectively. In order to avoid loss of preload due to the weight effect, use a smaller diameter of the SRBs to reduce their axial stiffness.**

Appendix B

Calculation of demand preloads for the test specimens

■ P-DB-M/Q specimens

1. Determination of the lateral force $F_{p,fb}$ and $F_{p,cb}$, according to (Eq. 4.10) and (Eq. 4.11)

The lateral forces making the floor beam $F_{p,fb}$ and the ceiling beam $F_{p,cb}$ reach their plastic moment, respectively:

$$F_{p,fb} = \left(\frac{M_{g,Ed,fb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{fb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 kNm}{1.24m} \right) \cdot \left(\frac{12226260 + 6072927}{12226260} \right) = 68.7 kN$$

$$F_{p,cb} = \left(\frac{M_{g,Ed,cb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{cb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 26.8 kNm}{1.24m} \right) \cdot \left(\frac{12226260 + 6072927}{6072927} \right) = 89.5 kN$$

2. Determine the design forces acting on beams and columns for demand preload P_g

The design forces for demand preload P_g are as bellows:

1) For floor beam and column-2F

$$V_{Ed,B2} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = \left(\frac{12226260}{12226260 + 6072927} \right) \cdot 68.7 kN = 45.9 kN \quad (\text{for floor beam})$$

$$V_{Ed,C2} = \frac{F_{p,fb}}{H} \cdot L = \frac{68.7 kN}{3040 mm} \cdot 1720 mm = 38.9 kN \quad (\text{for column 2F})$$

2) For ceiling beam and column-1F

$$V_{Ed,B1} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,cb} = \left(\frac{6072927}{12226260 + 6072927} \right) \cdot 89.5 kN = 29.7 kN \quad (\text{for floor beam})$$

$$R_{y1} = V_{Ed,C1} = \frac{F_{p,cb}}{H} \cdot L = \frac{89.5 kN}{3040 mm} \cdot 1720 mm = 50.6 kN \quad (\text{for column 1F})$$

$$*R_{x1} = F_{p,cb} = 89.5 kN \quad (\text{for column 1F})$$

3. Determine the demand preload P_g to prevent the gap-opening in the AJ connection.

(Refer to the process Steps 1, 2 and 3 in Section 3.4.3)

Step 1: Demand preload (P_{b2}) for the column 2F-floor beam

1) The minimum demand preload to prevent the gap-opening at point A ($P_{b2,A}$) and point B ($P_{b2,B}$)

$$P_{b2,A} = N_{c2} + \frac{2}{b_e} \left(V_{c2} \cdot \frac{h_2}{2} - V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{b2} \cdot d_{e2} \right)$$

$$= 0 + \frac{2}{0.22m} \left(38.9kN \cdot \frac{3m}{2} - 45.9kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) + 0 \cdot 0.17m \right) = -66.2kN$$

$$P_{b2,B} = N_{c2} + \frac{2}{b_e} \left(V_{b2} \cdot \frac{L_M}{2} - V_{c2} \cdot \frac{h_2}{2} - N_{b2} \cdot d_{e2} \right)$$

$$= 0 + \frac{2}{0.22m} \left(45.9kN \cdot \frac{3.3m}{2} - 38.9kN \cdot \frac{3m}{2} - 0 \cdot 0.17m \right) = 158kN$$

The larger preload between $P_{b2,A}$ and $P_{b2,B}$ becomes the demand preload (P_{b2}) for contact surface-2.

$$P_{b2} \geq \text{Max} (P_{b2,A}, P_{b2,B}) = \text{Max} (-66.2kN, 158kN) = 158kN$$

Step 2: Demand preload (P_{b1}) for the lower column-beam

1) The minimum demand preload to prevent the gap-opening at point C ($P_{b1,C}$) and point D ($P_{b1,D}$)

$$P_{b1,C} = N_{c1} + \frac{2}{b_e} \left(V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) - V_{c1} \cdot \frac{h_1}{2} + N_{b1} \cdot d_{e1} \right) + R_{x1} \cdot \left(0.18m + \frac{b_e}{2} \right) \cdot \frac{2}{b_e}$$

$$= 0 + \frac{2}{0.22m} \left(29.7kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) - 50.6kN \cdot \frac{3m}{2} + 0 \cdot 0.17m \right) + 89.5kN \cdot \left(0.18m + \frac{0.22m}{2} \right) \cdot \frac{2}{0.22m}$$

$$= -303.9kN + 235.8kN = -68.1kN$$

$$P_{b1,D} = N_{c1} + \frac{2}{b_e} \left(V_{c1} \cdot \frac{h_1}{2} - V_{b1} \cdot \frac{L_M}{2} - N_{b1} \cdot d_{e1} \right) + R_{x1} \cdot \left(0.18m - \frac{b_e}{2} \right) \cdot \frac{2}{b_e}$$

$$= 0 + \frac{2}{0.22m} \left(50.6kN \cdot \frac{3m}{2} - 29.7kN \cdot \frac{3.3m}{2} - 0 \cdot 0.17m \right) - 89.5kN \cdot \left(0.18m - \frac{0.22m}{2} \right) \cdot \frac{2}{0.22m}$$

$$= 244.5kN - 56.9kN = 188kN$$

* R_{x1} (reaction force acting on the pinned support) was considered for determination of demand preload P_g .

The larger preload between $P_{b1,C}$ and $P_{b1,D}$ becomes the demand preload (P_{b1}) for contact surface-1

$$P_{b1} \geq \text{Max} (P_{b1,C}, P_{b1,D}) = \text{Max} (-68.1, 188) = 188kN$$

Step 3: Determination of the final demand preload (P_g) for the connection

The final demand preload (P_g) for preventing the gap-opening at the connection

$$P_g = \text{Max} (P_{b1}, P_{b2}) = \text{Max} (158kN, 188kN) = 188kN$$

■ P-SB-M/Q specimens

1. Determination of the lateral force $F_{p,fb}$ and $F_{p,cb}$, according to (Eq. 4.10) and (Eq. 4.11)

The lateral forces making the floor beam $F_{p,fb}$ and the ceiling beam $F_{p,cb}$ reach their plastic moment, respectively:

$$F_{p,fb} = \left(\frac{M_{g,Ed,fb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{fb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 \text{ kNm}}{1.24 \text{ m}} \right) \cdot \left(\frac{12226260 + 12226260}{12226260} \right) = 91.8 \text{ kN}$$

$$F_{p,cb} = \left(\frac{M_{g,Ed,cb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{cb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 \text{ kNm}}{1.24 \text{ m}} \right) \cdot \left(\frac{12226260 + 12226260}{12226260} \right) = 91.8 \text{ kN}$$

2. Determine the design forces acting on beams and columns for demand preload P_g

The design forces for demand preload P_g are as bellows:

1) For floor beam and column-2F

$$V_{Ed,B2} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = \left(\frac{12226260}{12226260 + 12226260} \right) \cdot 91.8 \text{ kN} = 45.9 \text{ kN} \quad (\text{for floor beam})$$

$$V_{Ed,C2} = \frac{F_{p,fb}}{H} \cdot L = \frac{91.8 \text{ kN}}{3040 \text{ mm}} \cdot 1720 \text{ mm} = 51.9 \text{ kN} \quad (\text{for column 2F})$$

2) For ceiling beam and column-1F

$$V_{Ed,B1} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,cb} = \left(\frac{12226260}{12226260 + 12226260} \right) \cdot 91.8 \text{ kN} = 45.9 \text{ kN} \quad (\text{for floor beam})$$

$$R_{y1} = V_{Ed,C1} = \frac{F_{p,cb}}{H} \cdot L = \frac{91.8 \text{ kN}}{3040 \text{ mm}} \cdot 1720 \text{ mm} = 51.9 \text{ kN} \quad (\text{for column 1F})$$

$$*R_{x1} = F_{p,cb} = 91.8 \text{ kN} \quad (\text{for column 1F})$$

3. Determine the demand preload P_g to prevent the gap-opening in the AJ connection.

(Refer to the process Steps 1, 2 and 3 in Section 3.4.3)

Step 1: Demand preload (P_{b2}) for the column 2F-floor beam

1) The minimum demand preload to prevent the gap-opening at point A ($P_{b2,A}$) and point B ($P_{b2,B}$)

$$P_{b2,A} = N_{c2} + \frac{2}{b_e} \left(V_{c2} \cdot \frac{h_2}{2} - V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{b2} \cdot d_{e2} \right)$$

$$= 0 + \frac{2}{0.22m} \left(51.9kN \cdot \frac{3m}{2} - 45.9kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) + 0 \cdot 0.17m \right) = 111kN$$

$$P_{b2,B} = N_{c2} + \frac{2}{b_e} \left(V_{b2} \cdot \frac{L_M}{2} - V_{c2} \cdot \frac{h_2}{2} - N_{b2} \cdot d_{e2} \right)$$

$$= 0 + \frac{2}{0.22m} \left(45.9kN \cdot \frac{3.3m}{2} - 51.9kN \cdot \frac{3m}{2} - 0 \cdot 0.17m \right) = -20kN$$

The larger preload between $P_{b2,A}$ and $P_{b2,B}$ becomes the demand preload (P_{b2}) for contact surface-2.

$$P_{b2} \geq \text{Max} (P_{b2,A}, P_{b2,B}) = \text{Max} (111kN, -20kN) = 111kN$$

Step 2: Demand preload (P_{b1}) for the lower column-beam

1) The minimum demand preload to prevent the gap-opening at point C ($P_{b1,C}$) and point D ($P_{b1,D}$)

$$P_{b1,C} = N_{c1} + \frac{2}{b_e} \left(V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) - V_{c1} \cdot \frac{h_1}{2} + N_{b1} \cdot d_{e1} \right) + R_{x1} \cdot \left(0.18 + \frac{b_e}{2} \right) \cdot \frac{2}{b_e}$$

$$= 0 + \frac{2}{0.22m} \left(45.9kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) - 51.96kN \cdot \frac{3m}{2} + 0 \cdot 0.17m \right) + 91.85kN \cdot \left(0.18m + \frac{0.22m}{2} \right) \cdot \frac{2}{0.22m}$$

$$= -111.8kN + 242kN = 130.3kN$$

$$P_{b1,D} = N_{c1} + \frac{2}{b_e} \left(V_{c1} \cdot \frac{h_1}{2} - V_{b1} \cdot \frac{L_M}{2} - N_{b1} \cdot d_{e1} \right) + R_{x1} \cdot \left(0.18 - \frac{b_e}{2} \right) \cdot \frac{2}{b_e}$$

$$= 0 + \frac{2}{0.22m} \left(51.96kN \cdot \frac{3m}{2} - 45.9kN \cdot \frac{3.3m}{2} - 0 \cdot 0.17m \right) - 91.85kN \cdot \left(0.18m - \frac{0.22m}{2} \right) \cdot \frac{2}{0.22m}$$

$$= 20kN - 58.5kN = -38.5kN$$

* R_{x1} (reaction force acting on the pinned support) was considered for determination of demand preload P_g .

The larger preload between $P_{b1,C}$ and $P_{b1,D}$ becomes the demand preload (P_{b1}) for contact surface-1

$$P_{b1} \geq \text{Max} (P_{b1,C}, P_{b1,D}) = \text{Max} (130.3kN, -38.5kN) = 130.3kN$$

Step 3: Determination of the final demand preload (P_g) for the connection

The final demand preload (P_g) for preventing the gap-opening at the connection

$$P_g = \text{Max} (P_{b1}, P_{b2}) = \text{Max} (111.8kN, 130.3kN) = 131kN$$

Appendix C

Calculation of demand preloads for the FE joint models for parametric study

No considering R_{x1} (=reaction force acting on the pinned support)

■ For P-DB model

1. Determination of the lateral force $F_{p,fb}$ and $F_{p,cb}$, according to (Eq. 4.10) and (Eq. 4.11)

The lateral forces making the floor beam $F_{p,fb}$ and the ceiling beam $F_{p,cb}$ reach their plastic moment, respectively:

$$F_{p,fb} = \left(\frac{M_{g,Ed,fb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{fb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 kNm}{1.24m} \right) \cdot \left(\frac{12226260 + 6072927}{12226260} \right) = 68.7 kN$$

$$F_{p,cb} = \left(\frac{M_{g,Ed,cb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{cb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 26.8 kNm}{1.24m} \right) \cdot \left(\frac{12226260 + 6072927}{6072927} \right) = 89.5 kN$$

2. Determine the design forces acting on beams and columns for demand preload P_g

The design forces for demand preload P_g are as bellows:

1) For floor beam and column-2F

$$V_{Ed,B2} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = \left(\frac{12226260}{12226260 + 6072927} \right) \cdot 68.7 kN = 45.9 kN \quad (\text{for floor beam})$$

$$V_{Ed,C2} = \frac{F_{p,fb}}{H} \cdot L = \frac{68.7 kN}{3040 mm} \cdot 1540 mm = 34.8 kN \quad (\text{for column 2F})$$

2) For ceiling beam and column-1F

$$V_{Ed,B1} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = \left(\frac{6072927}{12226260 + 6072927} \right) \cdot 68.7 kN = 22.8 kN \quad (\text{for floor beam})$$

$$R_{y1} = V_{Ed,C1} = \frac{F_{p,fb}}{H} \cdot L = \frac{68.7 kN}{3040 mm} \cdot 1540 mm = 34.8 kN \quad (\text{for column 1F})$$

$$*R_{x1} = N_{c1} = F_{p,cb} = 68.7 kN \quad (\text{for column 1F})$$

3. Determine the demand preload P_g to prevent the gap-opening in the AJ connection.

(Refer to the process Steps 1, 2 and 3 in Section 3.4.3)

Step 1: Demand preload (P_{b2}) for the column 2F-floor beam

1) The minimum demand preload to prevent the gap-opening at point A ($P_{b2,A}$) and point B ($P_{b2,B}$)

$$\begin{aligned}
 P_{b2,A} &= N_{c2} + \frac{2}{b_e} \left(V_{c2} \cdot \frac{h_2}{2} - V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{b2} \cdot d_{e2} \right) \\
 &= 0 + \frac{2}{0.22m} \left(34.8kN \cdot \frac{3m}{2} - 45.9kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) + 0 \cdot 0.17m \right) = -122kN \\
 P_{b2,B} &= N_{c2} + \frac{2}{b_e} \left(V_{b2} \cdot \frac{L_M}{2} - V_{c2} \cdot \frac{h_2}{2} - N_{b2} \cdot d_{e2} \right) \\
 &= 0 + \frac{2}{0.22m} \left(45.9kN \cdot \frac{3.3m}{2} - 34.8kN \cdot \frac{3m}{2} - 0 \cdot 0.17m \right) = 214kN
 \end{aligned}$$

The larger preload between $P_{b2,A}$ and $P_{b2,B}$ becomes the demand preload (P_{b2}) for contact surface-2.

$$P_{b2} \geq \text{Max} (P_{b2,A}, P_{b2,B}) = \text{Max} (-122kN, 214kN) = 214kN$$

Step 2: Demand preload (P_{b1}) for the lower column-beam

1) The minimum demand preload to prevent the gap-opening at point C ($P_{b1,C}$) and point D ($P_{b1,D}$)

$$\begin{aligned}
 P_{b1,C} &= N_{c1} + \frac{2}{b_e} \left(V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) - V_{c1} \cdot \frac{h_1}{2} + N_{b1} \cdot d_{e1} \right) \\
 &= 68.7kN + \frac{2}{0.22m} \left(22.8kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) - 34.8kN \cdot \frac{3m}{2} + 0 \cdot 0.17m \right) \\
 &= 68.7kN + (-178kN) = -109.3kN \\
 P_{b1,D} &= N_{c1} + \frac{2}{b_e} \left(V_{c1} \cdot \frac{h_1}{2} - V_{b1} \cdot \frac{L_M}{2} - N_{b1} \cdot d_{e1} \right) \\
 &= 68.7kN + \frac{2}{0.22m} \left(34.8kN \cdot \frac{3m}{2} - 22.8kN \cdot \frac{3.3m}{2} - 0 \cdot 0.17m \right) \\
 &= 68.7kN + 132.5kN = 201kN
 \end{aligned}$$

The larger preload between $P_{b1,C}$ and $P_{b1,D}$ becomes the demand preload (P_{b1}) for contact surface-1

$$P_{b1} \geq \text{Max} (P_{b1,C}, P_{b1,D}) = \text{Max} (-109kN, 201kN) = 201kN$$

Step 3: Determination of the final demand preload (P_g) for the connection

The final demand preload (P_g) for preventing the gap-opening at the connection

$$P_g = \text{Max} (P_{b1}, P_{b2}) = \text{Max} (214kN, 201kN) = 214kN$$

■ For P-SB model

1. Determination of the lateral force $F_{p,fb}$ and $F_{p,cb}$, according to (Eq. 4.10) and (Eq. 4.11)

The lateral forces making the floor beam $F_{p,fb}$ and the ceiling beam $F_{p,cb}$ reach their plastic moment, respectively:

$$F_{p,fb} = \left(\frac{M_{g,Ed,fb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{fb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 \text{ kNm}}{1.24 \text{ m}} \right) \cdot \left(\frac{12226260 + 12226260}{12226260} \right) = 91.8 \text{ kN}$$

$$F_{p,cb} = \left(\frac{M_{g,Ed,cb}}{L_p} \right) \cdot \left(\frac{I_{fb} + I_{cb}}{I_{cb}} \right) = \left(\frac{1.1 \cdot 1.25 \cdot 41.4 \text{ kNm}}{1.24 \text{ m}} \right) \cdot \left(\frac{12226260 + 12226260}{12226260} \right) = 91.8 \text{ kN}$$

2. Determine the design forces acting on beams and columns for demand preload P_g

The design forces for demand preload P_g are as bellows:

1) For floor beam and column-2F

$$V_{Ed,B2} = \left(\frac{I_{fb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,fb} = \left(\frac{12226260}{12226260 + 12226260} \right) \cdot 91.8 \text{ kN} = 45.9 \text{ kN} \quad (\text{for floor beam})$$

$$V_{Ed,C2} = \frac{F_{p,fb}}{H} \cdot L = \frac{91.8 \text{ kN}}{3040 \text{ mm}} \cdot 1540 \text{ mm} = 46.5 \text{ kN} \quad (\text{for column 2F})$$

2) For ceiling beam and column-1F

$$V_{Ed,B1} = \left(\frac{I_{cb}}{I_{fb} + I_{cb}} \right) \cdot F_{p,cb} = \left(\frac{12226260}{12226260 + 12226260} \right) \cdot 91.8 \text{ kN} = 45.9 \text{ kN} \quad (\text{for floor beam})$$

$$R_{y1} = V_{Ed,C1} = \frac{F_{p,cb}}{H} \cdot L = \frac{91.8 \text{ kN}}{3040 \text{ mm}} \cdot 1540 \text{ mm} = 46.5 \text{ kN} \quad (\text{for column 1F})$$

$$*R_{x1} = N_{c1} = F_{p,cb} = 91.8 \text{ kN} \rightarrow \text{acting on the centre of the column} \quad (\text{for column 1F})$$

3. Determine the demand preload P_g to prevent the gap-opening in the AJ connection.

(Refer to the process Steps 1, 2 and 3 in Section 3.4.3)

Step 1: Demand preload (P_{b2}) for the column 2F-floor beam

1) The minimum demand preload to prevent the gap-opening at point A ($P_{b2,A}$) and point B ($P_{b2,B}$)

$$\begin{aligned} P_{b2,A} &= N_{c2} + \frac{2}{b_e} \left(V_{c2} \cdot \frac{h_2}{2} - V_{b2} \cdot \left(\frac{L_M}{2} - b_e \right) + N_{b2} \cdot d_{e2} \right) \\ &= 0 + \frac{2}{0.22 \text{ m}} \left(46.5 \text{ kN} \cdot \frac{3 \text{ m}}{2} - 45.9 \text{ kN} \cdot \left(\frac{3.3 \text{ m}}{2} - 0.22 \text{ m} \right) + 0 \cdot 0.17 \text{ m} \right) = 37.3 \text{ kN} \end{aligned}$$

$$\begin{aligned}
P_{b2,B} &= N_{c2} + \frac{2}{b_e} \left(V_{b2} \cdot \frac{L_M}{2} - V_{c2} \cdot \frac{h_2}{2} - N_{b2} \cdot d_{e2} \right) \\
&= 0 + \frac{2}{0.22m} \left(45.9kN \cdot \frac{3.3m}{2} - 46.5kN \cdot \frac{3m}{2} - 0 \cdot 0.17m \right) = 54.4kN
\end{aligned}$$

The larger preload between $P_{b2,A}$ and $P_{b2,B}$ becomes the demand preload (P_{b2}) for contact surface-2.

$$P_{b2} \geq \text{Max} (P_{b2,A}, P_{b2,B}) = \text{Max} (37.3kN, 54.4kN) = 54.4kN$$

Step 2: Demand preload (P_{b1}) for the lower column-beam

1) The minimum demand preload to prevent the gap-opening at point C ($P_{b1,C}$) and point D ($P_{b1,D}$)

$$\begin{aligned}
&= N_{c1} + \frac{2}{b_e} \left(V_{b1} \cdot \left(\frac{L_M}{2} - b_e \right) - V_{c1} \cdot \frac{h_1}{2} + N_{b1} \cdot d_{e1} \right) \\
&= 91.8kN + \frac{2}{0.22m} \left(45.9kN \cdot \left(\frac{3.3m}{2} - 0.22m \right) - 46.5kN \cdot \frac{3m}{2} + 0 \cdot 0.17m \right) \\
&= 91.8kN + (-37.3kN) = 54.5kN \\
P_{b1,D} &= N_{c1} + \frac{2}{b_e} \left(V_{c1} \cdot \frac{h_1}{2} - V_{b1} \cdot \frac{L_M}{2} - N_{b1} \cdot d_{e1} \right) \\
&= 91.85kN + \frac{2}{0.22m} \left(46.5kN \cdot \frac{3m}{2} - 45.9kN \cdot \frac{3.3m}{2} - 0 \cdot 0.17m \right) \\
&= 91.85kN + (-54.4kN) = 37.5kN
\end{aligned}$$

The larger preload between $P_{b1,C}$ and $P_{b1,D}$ becomes the demand preload (P_{b1}) for contact surface-1

$$P_{b1} \geq \text{Max} (P_{b1,C}, P_{b1,D}) = \text{Max} (54.5kN, 37.5kN) = 54.5kN$$

Step 3: Determination of the final demand preload (P_g) for the connection

The final demand preload (P_g) for preventing the gap-opening at the connection

$$P_g = \text{Max} (P_{b1}, P_{b2}) = \text{Max} (54.4kN, 54.5kN) = 55kN$$