

**A Curved Member Retrofit and Reliability-Based Reparability
Assessment of Column Bases in Steel Structures**

LIN KUNSIAN

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CHAPTER 1

INTRODUCTION

1.1 Background

The sustainable development of our society relies on the efficient utilization of resources and the minimization of waste. Earthquake disasters, however, cause casualties and damage to infrastructure, and generate large volumes of waste material. In this context, buildings that can recover their original or nearly original performance with minimal intervention are favored. The critical challenge lies not only in restoring the original capacity of damaged structures through retrofitting, but also in determining whether we can reliably assess the residual capacity of such structures after a major earthquake event. A robust evaluation is essential to ensure confidence in its performance during future events. The issue with residual capacity was highlighted following the 2011 Christchurch earthquake, where numerous slightly damaged structures were demolished due to the absence of clear guidelines for assessing their residual capacity [1-1].

Recent studies in disaster resilience have increasingly focused on developing models for estimating loss and recovery trajectories, with emphasis on tools for assessing residual structural capacities. For reinforced concrete (RC) buildings subjected to earthquake ground motions, various methodologies have been proposed to evaluate residual strength and ductility [1-2]. For steel moment-resisting frames, a method has been established that quantifies damage severity using a dynamic-strain-based damage index, enabling the update of the model for evaluating residual structural capacity [1-3]. Furthermore, the incorporation of residual deformation into seismic performance assessments has gained recognition. For instance, a probabilistic framework for performance-based seismic evaluation considering residual deformation has been proposed [1-4], alongside a methodology for aftershock vulnerability assessment that considers residual drift demands after the mainshock [1-5].

In parallel, frameworks have been developed to guide post-earthquake inspection and assessment, aiding in the determination of whether a structure can meet performance expectations in future earthquakes with minimal or no repair, or whether extensive rehabilitation is required [1-6]. Meanwhile, major seismic events such as the 1995 Great Hanshin-Awaji earthquake, the 2011 Great East Japan earthquake, and the 2016 Kumamoto earthquake have heightened awareness of the importance of earthquake-resistance design for new buildings and retrofitting for existing structures. However, seismic retrofitting techniques, including additional load-resisting systems, dampers, and seismic isolation devices, often impose significant disturbances on both the structure and its occupants. These interventions may increase the force demand on the structural system due

to enhanced strength and stiffness, altering architectural layouts, and necessitate the temporary relocation of residents during the retrofit process. For example, occupancy rates during typical retrofitting construction periods have been reported to drop to approximately 60% [1-7].

In response, recent studies have proposed the development of seismic-resilient structures that minimize both seismic damage and repair time, while also reducing disturbance during implementation [1-8, 1-9]. Such strategies, emphasizing rapid construction and low invasiveness, offer significant benefits for enhancing the seismic resilience of buildings and contribute to the sustainable development of cities by extending the service life of a large volume of building stocks.

1.2 Motivations and Research Gap

A substantial portion of urban building stock consists of low-rise structures [1-10], extending their service life, which is central to improving the earthquake resilience and sustainable development of our society. In Japan, the use of yielding column bases has emerged as a cost-effective design strategy for steel moment-resisting frames (SMRFs) [1-11], in contrast to the strict capacity design principles enforced in other high-seismicity regions such as the United States [1-12]. Observations from the 2010 Maule Earthquake in Chile demonstrated that buildings employing ductile anchor rods at the base experienced rocking behavior and correspondingly reduced damage [1-13]. The most common form of yielding column bases is the exposed type, in which a steel base plate is welded to the column and connected to concrete foundations via anchor rods [1-14, 1-15].

Lifting of strict capacity design at column bases enables compact and economical foundation solutions. Japanese design provisions suggest that if the bending moment capacity of the column base exceeds 1.3 to 1.5 times the plastic moment of the column (connection factor), the plastic hinge will form in the column. Conversely, if the plastic hinge is intentionally relocated to the column base, the force demand on the foundation can be reduced by approximately 30% to 50%. Recent studies have further suggested that the design requirements for column base connections can be relaxed by adopting capacity design principles to the overstrength seismic load (i.e., using the Ω factor), thereby permitting inelastic rotations at the column base connection and resulting in significant cost savings [1-16, 1-17]. In addition, adopting anchor-yielding column bases can effectively mitigate column axial shortening caused by local buckling, i.e., a phenomenon that may be critical in low-rise SMRFs when assessing post-earthquake reparability and informing decisions regarding potential building demolition [1-18]. Furthermore, this design approach offers a unique post-earthquake repair option through the retightening of elongated anchor rods, provided that adequate ductility is ensured in accordance with design specifications [1-19].

However, despite the increasing recognition of yielding column bases as a resilient and cost-effective solution, significant research gaps remain. First, while simplified repair methods such as retightening anchor rods are conceptually appealing, their practical effectiveness, especially under repeated loading, remains insufficiently validated. The retightened anchor rod reduces its deformation capacity; thus, the remaining deformation capacity is a crucial factor in adopting this repair method. Second, the classification of damage states and the decision-making process for repair applicability require alignment with Japanese and international standards. Currently, this area lacks sufficient experimental and numerical data, as well as a unified framework

for assessment.

In terms of retrofit strategies, existing methods for exposed column bases include welding shear plates under the base plate to provide shear resistance [1-20], and driving a wedge into the gap between the nut and the base plate to convert a slip-type column base to a non-slip-type column base [1-21], and encasing damaged column bases with reinforced concrete where additional steel bars being welded to the vertical reinforcement [1-22]. While these methods enhance strength and stiffness, their effects on the global seismic response, especially in low-rise buildings, have not yet been thoroughly examined. Retrofitting is known to change buildings' natural period and internal force distribution, which may inadvertently increase seismic demand on the frame system and adjacent members. A comprehensive understanding of these interactions is currently lacking, and designers lack quantitative guidance on how retrofit strategies impact overall structural performance.

Parallel to these structural considerations, an alternative option is to use the behaviors of curved members as the retrofit technique to overcome these challenges. Curved members have gained attention because of their aesthetic value and structural efficiency. Curved members are commonly used in architectural features such as canopies, spiral staircases, pedestrian pathways, and as structural members, roof members for station buildings and stadiums, and other large-scale structures. Beyond primary structural use, the curved members are frequently utilized in noise barriers and other auxiliary structures, demonstrating their versatility.

Curved geometries, achieved through bending or angular assembly of straight members, introduce intentional eccentricity that provides flexibility in balancing stiffness and strength. Many studies have demonstrated the benefits associated with intentional eccentricity provided by curved members. Specifically, intentional eccentricity enables lateral stiffness loosely coupled with yield strength, offering the potential to meet multiple, and sometimes conflicting, seismic design objectives related to stiffness, strength, and ductility [1-23]. In contrast, conventional straight members, commonly placed diagonally as braces in concentrically braced frames (CBFs) and buckling-restrained braced (BRB) frames, offer efficient structural performance but provide limited flexibility in tailoring the combination of stiffness and strength. When designed solely based on force demand, such braces often lead to substantial increases in stiffness, which in turn elevate seismic acceleration demands and consequently amplify the base shear forces acting on the structure.

Notable applications include crescent-shaped and curved knee braces, which have been proposed to enhance seismic performance through their unique geometric configuration [1-24, 1-25, 1-26]. The intended initial eccentricity also controls the buckling direction, avoids the severe loss of strength due to buckling, and places the member into post-buckling behavior [1-27]. A novel brace featuring a curved geometry and partial strengthening through induction heating has been proposed. The curved deformation along the brace length has been proven to enhance the stability of its compressive behavior [1-28].

Nevertheless, the application of curved members in seismic retrofit remains underdeveloped. The post-yield behavior, plastic deformation capacity, and buckling modes of curved members under realistic boundary conditions are not well characterized. Additionally, the absence of well-established design equations and detailing guidelines for curved members further limits their adoption in earthquake engineering practice.

Taken together, these gaps point to an urgent need for systematic research to:

- Experimentally and numerically assess the reparability of yielding anchor-rod-based column bases following seismic events;
- Develop decision-making frameworks and performance-based criteria for the reuse or replacement of damaged column bases;
- Quantify the system-level effects of common retrofit strategies on low-rise steel frame performance;
- Advance the understanding of the nonlinear seismic behavior of curved members and establish reliable design procedures for their use in retrofit applications.

Addressing these issues is essential to unlock the full potential of cost-effective, low-disruption seismic retrofit strategies that align with the goals of resilience, reparability, and sustainability.

1.3 Objectives

The goal of this dissertation is to investigate the effectiveness and applicability of the simple repair method, specifically, the retightening of yielded anchor rods, for the exposed column bases in low-rise steel structures, and to develop a novel retrofit technique that leverage the advantages of curved members while minimizing disturbance to the existing building system. The specific objectives of this research are as follows:

- (1) To evaluate the effectiveness of the simple repair method involving the retightening process of anchor rods, aimed at rapidly restoring the stiffness of exposed column bases after earthquakes. The repair method is examined through experimental testing and numerical analyses to quantify its applicability, reparability, and residual performance.
- (2) To propose a seismic retrofit framework of column bases grounded in residual capacity assessment and structural reliability, which identifies damage states of anchor rods and recommends appropriate repair/retrofit strategies. The framework is validated through experimental application to a four-story steel moment-resisting.
- (3) To develop and validate a novel seismic retrofit technique that utilizes curved members, achieving a moderately enhanced strength and stiffness, improved ductility, and reduced seismic demand, while minimizing construction-related disturbance.
- (4) To investigate the fundamental properties of curved members used in the proposed retrofit technique. Key parameters, including plastic strengths, initial elastic stiffness, and the effects of slenderness and compactness, are examined through a comprehensive analytical study. Parametric studies using numerical models and finite element methods are conducted with various configurations and geometric dimensions.
- (5) To establish a component- and system-level design framework for the proposed curved member retrofit technique, including connection detailing and strength estimation procedures. The design framework is validated through subassembly-level tests.

To demonstrate the seismic performance of retrofitted structures subjected to sequential earthquake excitations by examining global seismic responses and residual capacities through pushover and nonlinear dynamic analyses, with comparative evaluation of pre- and post-retrofit conditions.

1.4 Thesis Organization

This dissertation is structured into eight chapters, excluding the introduction (Chapter 1) and the conclusions (Chapter 8). The structure of the dissertation is illustrated in Figure 1-1. A summary of the main contents of Chapters 2 to 7 is provided below.

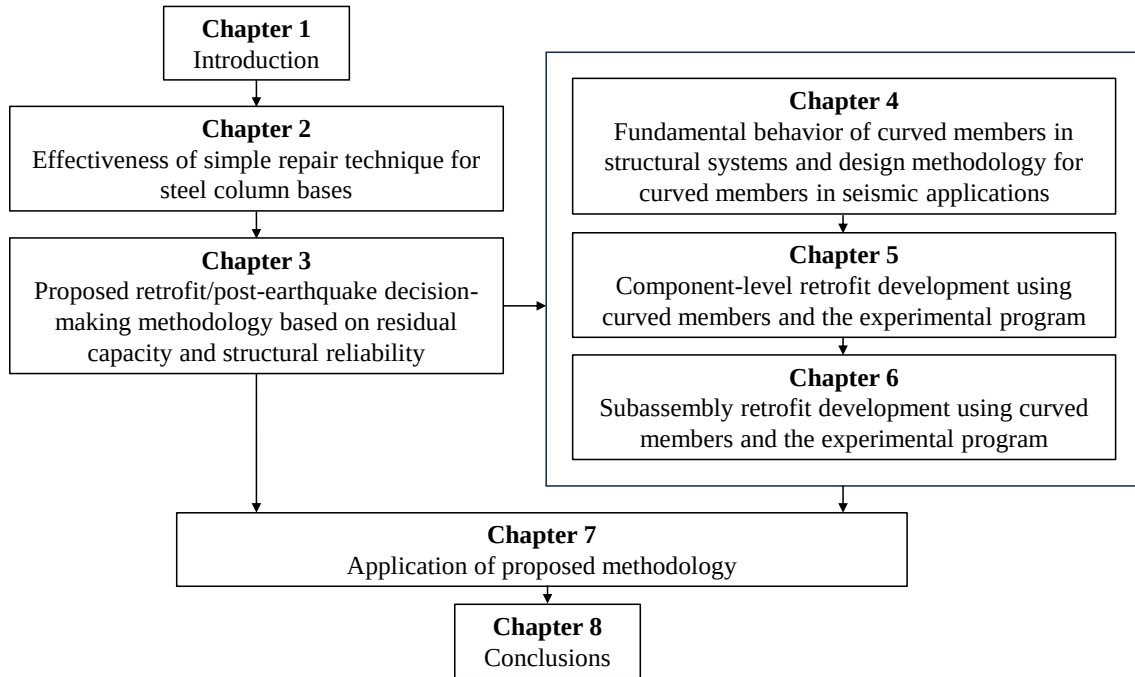


Figure 1-1 Dissertation structure

Chapter 2 provides a comprehensive literature review on conventional retrofit techniques for column bases and the required deformation capacity of ductile anchor rods. To address gaps in current knowledge, experimental tests are conducted on threaded anchor rods fabricated from various steel grades and with different screw thread configurations, to clarify their deformation capacity and ductility performance. Additionally, numerical models of four- and six-story steel moment-resisting frame buildings are developed and validated using data from shaking table tests. These models serve as benchmark structures throughout the dissertation for investigating the reparability and residual performance of exposed column base connections. Portions of the content presented in this Chapter are based on the author's previously published paper [3, 6, 7].

Chapter 3 introduces a repair methodology and a decision-making flowchart based on basic structural reliability theory, aimed at identifying damage states in anchor rods and selecting appropriate repair or retrofit strategies for exposed steel column bases. The anchor rod properties obtained in Chapter 2 are statistically analyzed to identify the residual performance of column bases.

Chapter 4 investigates the basic mechanism and plastic behavior of curved members through static analysis and extensive parametric studies. The study focuses on initial elastic stiffness, plastic strength, slenderness effects, and section compactness. Estimation equations for yield strength and buckling behavior under pinned and fixed boundary conditions are developed and validated through numerical simulations. The results

contributed to a connection design methodology that enhances material efficiency while preventing overly conservative designs. Portions of the content presented in this Chapter include parts of the author's paper that is currently under view [2].

Chapter 5 and 6 present the development and validation of a low-disturbance seismic retrofit technique that exploits the advantages of curved members to improve seismic performance while reducing the seismic demand to brittle or aged structural components. Component-level testing is conducted on four curved member specimens with variations in fabrication processes and boundary conditions, examining overall behavior, strength backbones, and deformation shapes. Furthermore, subassembly-level tests are conducted to evaluate improvements in the repairability and ductility limits of column bases and beam-column connections when retrofitted with curved members. These chapters incorporate parts of the author's published journal papers as well as a paper currently under review [1, 4].

Chapter 7 presents a case study that adopts the proposed decision-making methodology for retrofitting damaged or seismically deficient buildings, which integrates both the retightening of anchor rods and a curved member retrofit technique. A four-story building model adopting curved member retrofit is developed. Pushover and nonlinear dynamic analyses with sequential ground motions are conducted to assess the effectiveness and the seismic performance of the retrofitted structure. Finally, the bidirectional behavior of the proposed curved member retrofit was examined. This chapter incorporates parts of the author's conference paper and journal paper [4, 5].

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List of Publications

Journal papers

- [1] **Lin, K. S.**, Kurata, M., Kitamura, K., Kitatani, Y. 2025. “Subassembly Tests of Seismic Retrofit Technique Using Curved Members for Steel Frames.” *Journal of Structural Engineering (ASCE)*. (Under review)
- [2] **Lin, K. S.**, Kawasaki, Y., Kurata, M. 2025. “Plastic Behavior and Design Methodology for Curved Members in Seismic Applications”. *Japan Architectural Review*. (Under review)
- [3] **Lin, K. S.**, Kurata, M., Pettinga, D., Suzuki, Y., Matsuo, S., Perea, T., & Ito, M. 2024. “Effectiveness of repairing yielding anchor rods in exposed column bases in steel structures.” *Earthquake Engineering & Structural Dynamics*, 53(4), 1656-1675. <https://doi.org/10.1002/eqe.4087>
- [4] **Lin, K. S.**, Kurata, M., Kawasaki, Y., & Kitatani, Y. 2024. “Investigation of low-disturbance seismic retrofit method for steel column bases using curved members.” *Japan Architectural Review*, 7(1), e12429. <https://doi.org/10.1002/2475-8876.12429>

International conference papers (reviewed)

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- [6] **Lin K. S.**, Kurata M., Matsuo S., Pettinga D., Suzuki Y., Perea T. (2024.7) Examination of simple repair method for anchor-rod-yield-type exposed column bases in steel structures. In *Proceedings of the 18th World Conference on Earthquake Engineering-WCEE 2024*.
- [7] **Lin K. S.**, Kurata M., Kawamoto T., Matsuo S., Pettinga D. (2023.10) Identification and reparation of earthquake-induced irregularity of steel frames with weak-base/strong-column design. 10th European Workshop on the Seismic Behaviour of Irregular and Complex Structures, Catania.

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CHAPTER 2

EFFECTIVENESS OF SIMPLE REPAIR TECHNIQUE FOR STEEL COLUMN BASES

2.1 Introduction

2.1.1 Background

In the design of steel moment-resisting frames in Japan, a cost-effective strategy known as the weak-base/strong-column design approach [2-1], in which exposed-type column bases are designed to yield before first-story columns, is widely adopted. This practice contrasts with the strict capacity design requirements for column bases typically enforced in other seismic regions. By shifting the yield mechanism into the column base connection, this approach facilitates a more compact and economical column base design.

Another distinctive feature of this design approach is its simple repair method, which allows constructors to retighten elongated anchor rods, with ensured ductility by specifications, to rapidly restore the overall stiffness of the exposed column bases [2-2], as shown in Figure 2-1. As a standard repair technique for damaged exposed-type column bases, the applicability and effectiveness of the retightening process of anchor rods warrant a thorough investigation. The retightened anchor rod reduces its deformation capacity; therefore, deformation capacity becomes a critical factor in adopting this repair method.

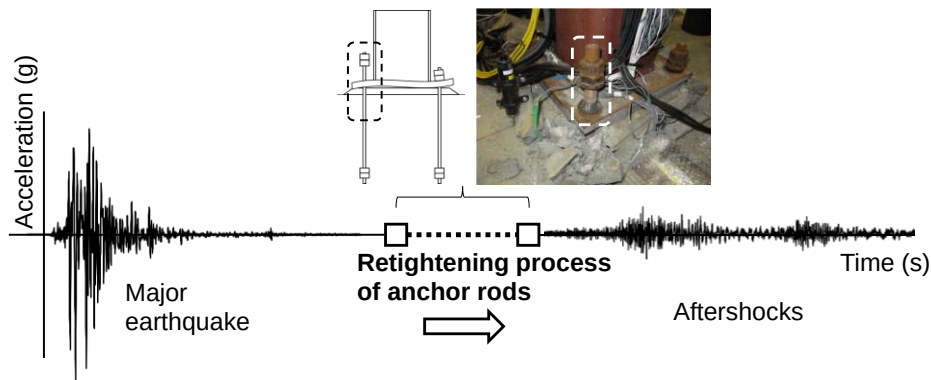


Figure 2-1 Column base repair methodology: sequential earthquakes and retightening process.

In this context, the present study examines the applicability of the retightening process by evaluating the deformation capacity of anchor rods that conform to both Japanese and foreign specifications. Furthermore, the

study verifies the effectiveness of the repair method in restoring the overall stiffness of the exposed column bases through experimental and numerical simulations.

2.1.2 Organization

Section 2.2 summarizes the current specifications on standard retrofit and repair techniques for column bases and the deformation capacity required for ductile anchor rods. Section 2.3 reports the testing of the anchor rod deformation capacity with different steel grades and screw threads commonly used in Japan, Taiwan, Mexico, the U.S. (ASTM standards), and New Zealand. Then, section 2.4 presents numerical models of four- and six-story buildings representing a hospital facility and an office building, respectively. Finally, section 2.5 discusses the applicability and effectiveness of this simple repair method for column bases through dynamic analysis.

2.2 Design concept and retrofit strategies for exposed column bases

Exposed-type column bases consisting of base plates, anchor bolts, and foundation concrete must reliably transfer axial forces, shear forces, and bending moments acting on the bottom of steel columns to the foundation. The design approach for exposed-type column bases can be classified into the following two types: 1) enforce the capacity design to prevent column bases from yielding and expect plastic deformation at the bottom of the first-story columns. The required strength of column bases per AISC 341-22 [2-3] shall be the greater of the following, the required strength of the column or the required strength determined using the overstrength seismic load; 2) allow column bases to yield before the first story columns and expect plastic deformation at the column bases. According to the AIJ Guidebook on Design and Fabrication of Column Base in Steel Structure, the relation between the maximum strength of the column base and column determines the design of the system. If ductile anchor rods are adopted, and the maximum strength of the column base is smaller than that of the column, the plastic hinge will occur at the anchor rods. The first approach expects plastic deformation at the bottom of columns, and thus, the width-to-thickness ratio of columns must be small to ensure sufficient plastic deformation capacity of the columns. Also, sufficient strength must be provided to the column bases so that the column bases remain sound against repeated plastic deformation of the columns. The second approach under the weak-base/strong-column design assumes high ductility at the column bases and thus properly evaluates the various collapse modes at the column bases. The collapse modes with high plastic deformation capacity should be pursued. In addition, the base plate of the column base may yield in bending before the anchor rod yields in tension, which also provides plastic deformation capacity for the column base and adds extra energy dissipation capacity.

According to the Building Standard Law of Japan, to ensure the beam collapse (e.g., entire collapse) mechanism, the column strength ratio associated with the combined collapse mechanism of the beam and panel for each story must satisfy the requirement that the ratio be greater than one. On the other hand, to consider the effect of inelastic deformations on the required lateral strength of structures, the structural characteristics factor D_s was adopted in the Building Standard Law of Japan. The factor D_s is used to reduce the elastic response story shear according to the available ductility. In the case of yielding anchor rods, the elongation of the anchor

rods results in a slip behavior in the column base hysteresis. Moreover, the inflection point shifts downward, leading to damage concentration at the second-floor beam-column connections. Therefore, an increase of 0.05 of D_s is required for design.

Previous studies have proposed several retrofit techniques for exposed column bases. One approach involves welding shear plates beneath the base plate to enhance shear resistance [2-4]. Another method eliminates the gap between the nut and the base plate by inserting a wedge, thereby converting a slip-type column base into a non-slip-type [2-5]. Additionally, a widely used retrofit technique for damaged column bases involves the use of reinforced concrete encasement [2-6]. In this method, supplemental steel bars are welded to the vertical chord reinforcement surrounding the anchor rods, and reinforced concrete is cast around the steel column base to restore strength and stability.

2.3 Required deformation capacity and tensile test for anchor rods

Anchor rods play a critical role in exposed-type column bases, particularly in the context of weak column base design. One objective of this chapter is to promote the adoption of the weak column base design and check the applicability of a simple repair method by examining whether commonly used anchor rod types in different countries possess adequate deformation capacity to accommodate the demand. This section first reviews the specifications in different countries and then presents the results of tensile tests conducted to evaluate the deformation capacity of anchor rods.

2.3.1 Specifications for the required deformation capacity of anchor rods

In designs where plastic deformation is expected in column bases, an anchor bolt is a crucial component that determines the plastic deformation capacity. The shank section of the anchor bolt shall yield to secure a large elongation capacity. The Japan Society of Steel Construction (JSSC) established standards for anchor bolt sets in 2000, guaranteeing the yield of the shank section. The Japan Industrial Standards (JIS) adopted the JSSC standards without changes in 2010. The anchor bolts currently used in Japan follow the specifications of the Japan Industrial Standards (JIS) and are classified as ABM and ABR [2-7, 2-8], depending on the thread manufacturing method. ABM is cut-threaded, and ABR is roll-threaded. The material specification for the anchor bolts specifies a yield ratio to guarantee the yielding of the shank part of anchor bolts in addition to the threaded part, as shown in Table 2-1. The anchorage length is set to $20d$ (d is the diameter of the anchor bolt) or longer.

Although commonly used in the structural engineering industry, there is currently no cited New Zealand Standard that specifically covers the manufacture of threaded bars typically used for column anchor bolts. While there are several international standards covering the specification and manufacture of threaded bars, most bars used in New Zealand are supplied in accordance with the European Standard DIN 976-1, published in 2016 [2-9]. Performance requirements for anchor rods are, however, covered in AS/NZS5131:2016 (Structural Steelwork – Fabrication and erection) [2-10], while chemical and mechanical properties are defined by AS/NZS 4291.1:1995 [2-11]. Notably, the range of threaded bars covered under these Standards is somewhat limited, with a maximum diameter of M39 and common grades 8.8, 10.9, and 12.9. The general design approach with

foundation anchor rods is to maintain their elastic behavior, with the potential plastic hinge of the moment frame located in the column member itself. Base plates and anchor rods are designed according to the capacity design philosophy is ‘capacity protected’, which means that their capacity exceeds the overstrength plastic hinge capacity of the column. In addition, in high-demand scenarios, the anchor rods are often debonded from the concrete via wrapped tape or sleeved tubes over the bars. This is to ensure that the tensile strains in the anchor rods are developed over a sufficient length.

Table 2-1 anchor rod material in Japan

Typ.	Name	Material	Tensile strength min. (N/mm ²)	Yield ratio	Manu. type	Thread type	Washer hardness	Size	Surface treatment
ABR	ABR400	SNR400B	400	below 80%	rolled	Metric coarse	200J	M16-M48	hot-dip or electro galvanizing
	ABR490	SNR490B	490					M16-M48	
	ABR520SUS	SUS304A	520	below 65%				M16-M48	—
ABM	ABM400	SNR400B	400	below 75%	cut	Metric fine	200J	M24-M48	electro galvanizing
	ABM490	SNR490B	490					M24-M100	
	ABM520SUS	SUS304A	520	below 65%		Metric coarse		M24-M48	—

Threaded rods are commonly used as concrete anchors and foundation bolts in many countries, including the U.S., Taiwan, and Mexico. The Specifications for Structural Steel Buildings in the U.S., Taiwan, and Mexico specify that anchor rods should be designed to resist the tensile and shear forces on the column base under the applicable load combinations. The general design recommendations in the U.S., Taiwan, and Mexico for anchor rods and base plates refer to AISC Design Guide 1 [2-12], and the anchor rod materials commonly used in structural engineering meet ASTM standards, as shown in Table 2-2, and ACI 318. The anchor rod deformation capacity defined by ACI 318-19 Appendix D is an element with a tensile test elongation of at least 14 percent and reduction in the area of at least 30 percent, while the 14 percent elongation should be measured over the gauge length specified in the appropriate ASTM standard [2-13]. For anchor bolts that are not threaded over their length, it should be verified that threaded deformed bars satisfy the strength requirement that the tensile strength is greater than 1.25 times the yield stress due to concerns over a fracture in cut threads.

In Taiwan, during the construction of column bases, when the structure needs to eliminate the influence of anchor rod elongation on the deflection of high-rise slender structures or prevent anchor nuts from loosening due to repetitive loading, high-strength anchor bolts (A449) with pre-tension can be used. Pre-tensioned anchor bolts can be applied to the structure, while the anchor plate must have sufficient stiffness to prevent deformation under pressure, and the anchor bolt must be covered with lubricating material to facilitate the process of pre-tension [2-14]. It is noted that the design concept of weak-base/strong-column with an anchor-rod-yield-type exposed column base is not permitted in many countries, including the U.S., Taiwan, and Mexico.

Table 2-2 anchor rod material in the US and Mexico

Material ASTM	Size, inches	Yield strength, ksi (Mpa), min.	Tensile strength, ksi (Mpa)	Elong. % min.	Reduction area % min.	
F1554	Gr 36 ^(a)	½ - 4	36 (250)	58 – 80 (400 – 550)	23	40
	Gr 55	½ - 4	55 (380)	75 – 95 (515 – 655)	21	30
	Gr 105	½ - 3	105 (725)	125 – 150 (860 – 1035)	15	45
A449		¼ - 1	92 (635)	120 (825) min	14	35
		1⅛ - 1½	81 (560)	105 (725) min	14	35
		1⅝ - 3	58 (400)	90 (620) min	14	35
A36 ^(b)	4 max	36 (250)	58 – 80 (400 – 550)	20	-	
A307	¼ - 4	-	60 (415) min	18	-	
A354 Gr BD		¼ - 2½	130 (895)	150 – 173 (1035 – 1195)	14	40
		2⅝ - 4	130 (895)	140 – 173 (965 – 1195)	14	40

Notes: ^(a) Preferred material specification in the U.S. ^(b) Preferred material specification in Mexico.

2.3.2 Tensile tests

In a collaborative project with Kyoto University, Holmes Solutions LP (a member of Holmes Group) performed tensile tests on typical Grade 8.8 and 10.9 threaded rods in accordance with AS1391:2007 Metallic Material - Tensile Testing at Ambient Temperature, at an accredited structural testing facility [2-15]. The specimens have diameters of 30 mm and 36 mm, which are commonly used in New Zealand steel construction. Holmes Solutions procured five specimens of each size and steel grade from a local supplier based in Christchurch, New Zealand. All threaded rods were machined with an ISO metric coarse pitch thread.

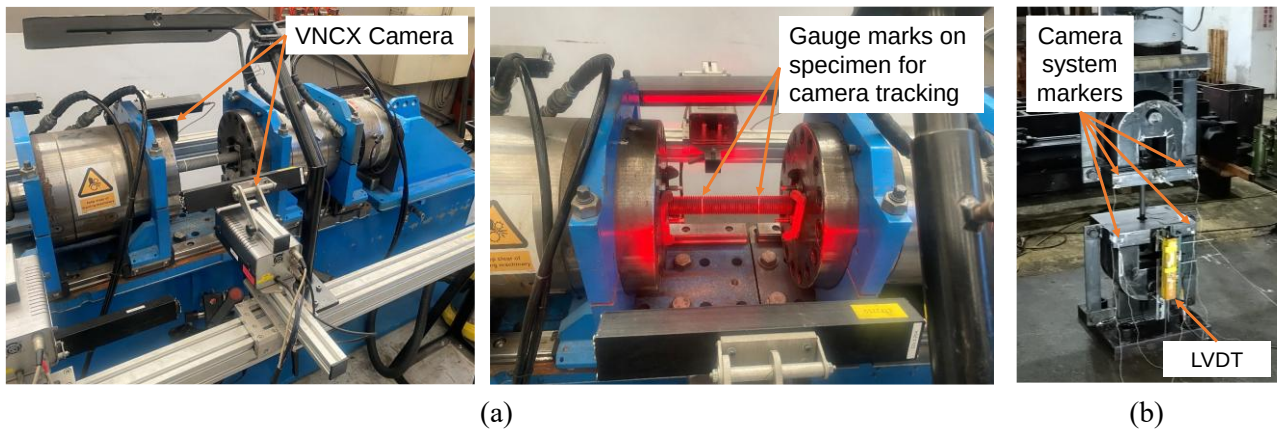


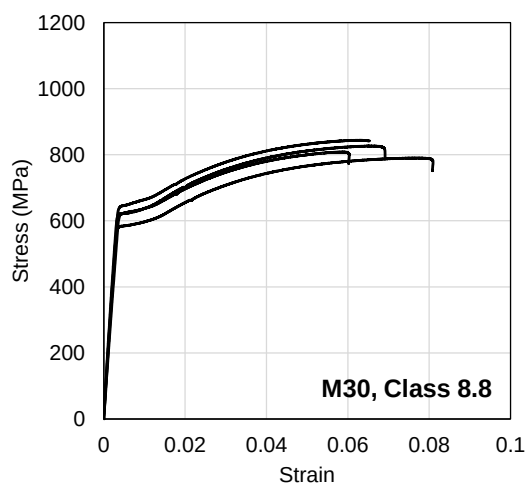
Figure 2-2 Tensile test setup adopted in (a) New Zealand and (b) Taiwan.

The principal test apparatus employed was HTM1, which has a maximum stroke of 600 mm and a peak table velocity of 300 mm/sec. Control of the loading rate in the test machine can be achieved through load control, stress control, stroke control, or strain control, utilizing a Delta Computer Systems RMC151E motion controller and RMC Tools software. This multipurpose test machine has an axial static force capacity of 1 MN

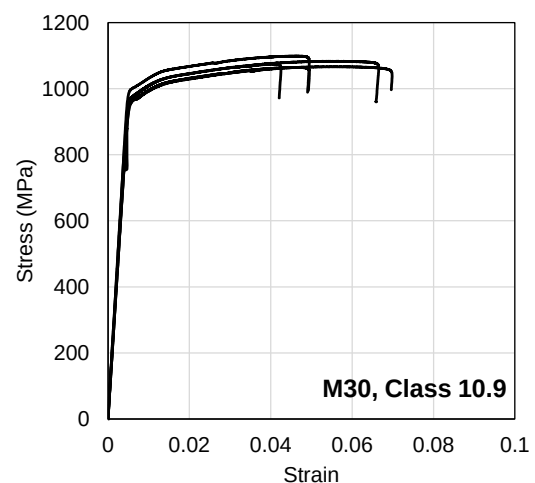
in both compression and tension directions and is equipped with hydraulic wedge-locking jaws for mounting test specimens or fixtures. The test setup is illustrated in Figure 2-2(a). The applied force was measured using a single load cell located at the fixed tailstock end of HTM1, with a total force measurement capacity of ± 1 MN. A video non-contact extensometer (VNCX), an optical-based strain measurement system, was used to measure the elongation of test specimens in HTM1 without requiring a physical connection to the specimen. The VNCX system uses one or more cameras to track painted markers on the test specimen, and after internal data processing, it provides elongation and strain measurements as output values. All specimens were tested following the test procedure outlined in AS 1391:2007. The yield strength ($R_{p0.2}$) is determined by the 0.2% offset method, while the tensile strength (R_M) is determined by the peak tension force the specimen withstands. The detailed and statistical data of all specimens, including mean and standard deviation, are shown in Table 2-3 and Figure 2-3. Threaded bar tensile tests are not often interrogated with regard to plastic strain capacity. The key difference noted is the significantly larger ultimate strain capacity of the M36 G8.8 bars compared to the others, although the strain at peak stress is consistent around 6-7%. The yield ratio of thread rods using steel grade of class 8.8 satisfies part of the material specification for anchor bolts in Japan mentioned in Table 2-1. Therefore, the anchor rod adopting steel grade of class 8.8 used in New Zealand can be used for anchor-yield-type exposed column base in Japan.

Table 2-3 Test result for fully threaded rod property: mean/standard deviation

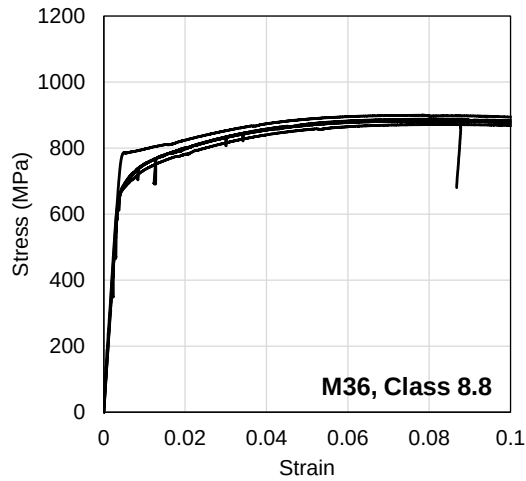
Specimen	Strain at Peak Stress (%)	Yield Strength, $R_{p0.2}$ (MPa)	Tensile Strength, R_M (MPa)	$R_M/R_{p0.2}$
M30 Grade 8.8	6.5/0.6	612/28.9	810/25.9	1.32/0.03
M30 Grade 10.9	5.0/0.7	984/15.8	1078/14.0	1.10/0.01
M36 Grade 8.8	7.9/0.5	715/40.2	884/10.4	1.24/0.05
M36 Grade 10.9	4.2/0.6	957/13.6	1062/9.9	1.11/0.01



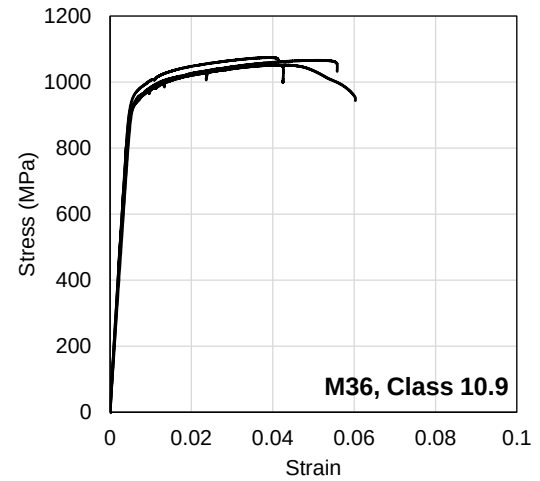
(a)



(b)



(c)



(d)

Figure 2-3 Test result for fully threaded rod.

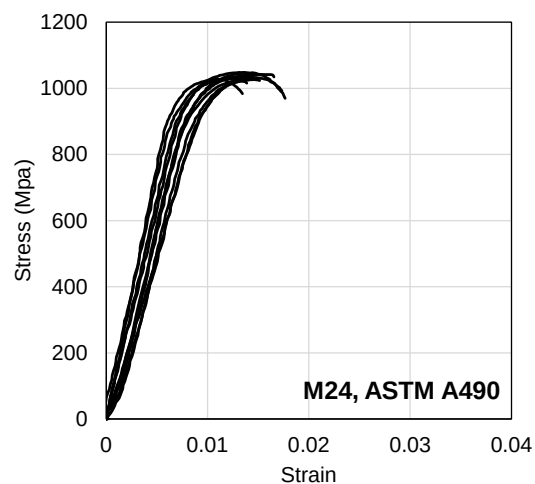
For the anchor rod used in Taiwan, all testing was completed in the structural laboratory at the National Taiwan University of Science and Technology, Taipei City, Taiwan. All specimens were fabricated in Taiwan, and the size is commonly used in structural engineering. The specimens have diameters of 24 mm and 36 mm, with total lengths of 550 mm and 690 mm, respectively. The threaded lengths are 105 mm and 145 mm, respectively. The American steel grade of ASTM-A490 and ISO general-purpose metric screw threads was adopted for all anchor rod specimens in the test.

Twenty anchor rod specimens have been tested by applying monotonic loadings in tension. A set of fixtures with pins was adopted in the study to test the anchor rods where double nuts are placed on both sides of the end plate, and the anchor rods are subjected to tension only, as shown in Figure 2-2(b). A 5,886 kN universal testing machine applied monotonic loadings to the specimens through a set of stiff fixtures designed to induce limited elastic deformation. The speed of testing was determined based on the rate of strain control method as specified in ASTM E8/E8M-13a standard. To evaluate the deformation capacity of the specimens, linear variable displacement transducers (LVDTs) were adopted to directly measure the deformation of the anchor rod specimens as shown in Figure 2-2(b). The Northern Digital Inc. (NDI) motion capture system was adopted in the instrumentation. The markers of the NDI system were attached to the end plate of the fixtures to measure the relative rotation and displacement. The strain gauges were attached to the middle of the anchor rod specimens to measure the strain on the un-threaded segment. The yield strength is determined by the 0.2% offset method, while the tensile strength is determined by the peak tension force the specimen withstands. The elastic modulus was calculated by the strain gauge attached to the unthreaded part of the anchor rods. The elongation is measured by the change in length from the original to the fracture. The detailed and statistical data of all specimens, including mean and standard deviation, are shown in Table 2-4 and Figure 2-4. All the tests were finally terminated due to failure at the threaded portion. In conclusion, the threaded anchor rod with a steel grade of ASTM-A490 exhibits poor deformation capacity, making it unsuitable for an anchor-yield-type exposed column base. Due to concerns over a fracture in cut threads where connections are threaded and the ductile steel

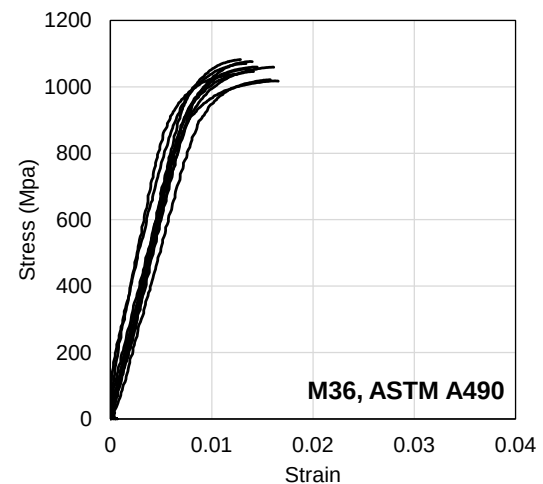
element is not threaded over its entire length, the stress ratio should not exceed 0.8 unless the threaded portions are upset. To use the anchor-rod-yield-type exposed column bases in Taiwan, the anchor rod material must meet the requirements. Alternatively, the anchor rod can be customized to have upset threaded portions or be threaded throughout its length to prevent failure in the threads. Anchor rod materials commonly used in Mexico also follow the ASTM standard. The method of thread cutting and the specifications used are similar to those in Taiwan. Therefore, it can be assumed that the mechanical properties and behavior of commonly used threaded anchor rods in Taiwan are equivalent to those in Mexico.

Table 2-4 Test result for end threaded rod property: mean/standard deviation

Specimen	Yield strength (MPa)	Tensile strength (MPa)	Elastic modulus (MPa)	Elastic stiffness of anchor rods (kN/mm)	Elongation (%)
M24	985/27.7	1035/7.92	221184/7467	133/14.8	1.53/0.14
M36	1003/33.5	1053/20.7	225021/16990	262/27.5	1.41/0.11



(a)



(b)

Figure 2-4 Test result for end threaded rod.

2.4 Numerical model development

To investigate the applicability and effectiveness of the retightening process for anchor rods in frame systems, numerical models of four-, six-, and ten-story buildings, representing a hospital facility and two office buildings, respectively, were developed using the OpenSees framework [2-16]. A simulation method for replacing anchor rod elements in the entire structure's life was established.

2.4.1 Design of prototype buildings

All the buildings adopted the weak-base/strong-column concept, which has a collapse mechanism with the column base yielding before the columns. The four-story SMRF was designed for a full-scale shaking table test of a hospital facility, which was conducted at the Hyogo Earthquake Engineering Research Center (E-defense)

in December 2020 [2-17]. The structural design followed the building law of Japan with a reserved strength of 1.5 times the code minimum and adopted the weak-base/strong-column concept. The seismic design of Japanese buildings is featured by a two-level design for earthquakes. The first-level design is an allowable stress design method that aims to ensure the safety and reparability of buildings during medium earthquakes, and the story drift should not exceed 1/200 under the action of the design shear force. The second-level design aims to assess the available ductility capacity to meet the ductility demand and the overstrength provided by the structure against severe earthquakes through pushover analysis. The general floor plans and elevations of the four-story building are shown in Figure 2-5. The SMRF was designed as two by one bay, with a width of 5.0m and 7.0m, respectively, and a story height of 3.6m for the 1-story bay and 3.4m for the remaining stories. The member sizes of the beams and columns in the four-story SMRF building are listed in Table 2-5. The adopted exposed column bases, EJ250-4-30, consisted of an HSS-250×250×16 column (BCR295) welded to a 410×410×40 base plate (SN490B). The base plate was secured with four rods (30mm nominal diameter, HAB) to the concrete foundation, with a nominal grout pack between the base plate and the foundation. The Housing and Building (HAB) material meeting the Building Standards Act with a stress ratio below 0.7 was adopted for the anchor rods.

Table 2-5 Designed member sizes of the four-story SMRF

Story	Columns (mm)	Beams (mm)
1	BOX-250×250×16	H-400×200×8×13
2	BOX-250×250×16	BH-400×150×6×19
3-4	BOX-250×250×16	H-400×200×8×13

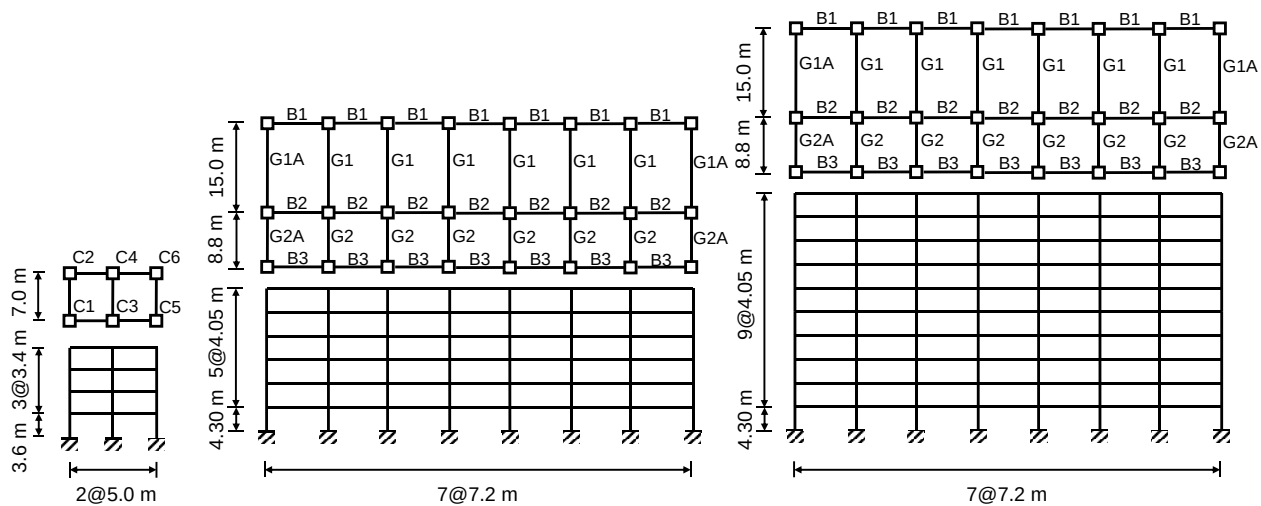


Figure 2-5 Floor plans and elevations of the adopted four-story, six-story, and ten-story building.

The six- and ten-story buildings were designed (by Kobori Research Complex) as an office building with an importance factor I of 1 using the two-step design method per Japanese code. The general floor plans and elevations of the building are shown in Figure 2-5. Both buildings have a story height of 4.3m for the 1-story and 4.05m for the remaining stories, and a floor plan consisting of 7 by 2 bays, with 7.2m-long bays in the X

direction and 8.8m- and 15m-long bays in the Y direction. The region factor Z , which represents the seismic hazard is 1.0 for a location in an urban area such as Tokyo, Osaka, and Nagoya, and the critical period T_c of the soil is 0.6 seconds for the medium soil type. The design spectral factor R_i of 0.99 is determined according to the type of soil profile and the natural period T of the building. A structural characteristic factor, D_s , similar to the response modification factor R as defined in ASCE 7-22, with a value of 0.25, was used for the benchmark building. The final member sizes of the beams and columns for the six- and ten-story buildings are listed in Tables 2-6 to 2-8.

Table 2-6 Designed column sizes of the six- and ten-story SMRF

SMRF-6F			SMRF-10F				
Story	Columns, HSS (mm)	Story	Columns, HSS (mm)				
			C1	C1A	C2	C2A	C3, C3A
1-3	600×600×25	1	600×600×36	700×700×40	700×700×36	700×700×40	700×700×40
4-6	600×600×22	2-4	600×600×36	600×600×40	700×700×36	700×700×40	600×600×40
		5-7	600×600×28	600×600×28	600×600×32	600×600×32	600×600×32
		8-10	600×600×25	600×600×25	600×600×28	600×600×28	600×600×25

Table 2-7 Designed beam sizes of the six-story SMRF

Story	Beams, H (mm)					
	B1, B3	B2	G1	G1A	G2, G2A	
1	600×250×12×22	600×250×12×25	800×250×14×22	800×250×14×22	800×250×14×25	
2-3	600×250×12×22	600×250×12×22	800×250×14×22	800×250×14×22	800×250×14×25	
4-5	600×250×12×19	600×250×12×19	800×250×14×22	800×250×14×22	800×250×14×22	
6	600×250×12×22	600×250×12×22	800×250×16×25	800×250×14×25	800×250×14×25	

Table 2-8 Designed beam sizes of the ten-story SMRF

Story	Beams, H (mm)					
	B1	B2, B3	G1	G1A	G2	G2A
1	850×250×16×32	850×250×16×25	850×350×16×32	850×350×16×36	850×350×16×32	850×350×16×36
2-3	800×250×16×28	800×250×14×25	800×300×14×28	800×300×14×32	800×300×14×32	
4-6	800×250×14×25	800×250×14×22	800×300×14×25	800×300×14×28	800×300×14×28	
7-9	800×250×14×25	800×250×14×22	800×300×14×25	800×300×14×25	800×300×14×25	
10	800×250×14×25	800×250×14×25	800×300×14×32	800×300×14×28	800×300×14×32	

The benchmark building was designed to meet fundamental periods and drift limitations per the Japanese code. JIS steel grade of SN490B and BCP325 was adopted for beams and columns of the building. To ensure that the plastic hinges form solely on the beams instead of the surrounding column, the beam-to-column capacity ratio of each connection must be larger than 1.5. Moreover, beam flanges are widened at the ends to prevent

fractures at connections. The adopted high-performance exposed column, HiBase NEO, allows for yielding at anchor rods and base plates to improve energy dissipation and mitigate the slip behavior of conventional exposed column bases. It is noted that anchor rod deformation dominates, and residual deformation of the base plate is not significant for HiBase NEO. Figure 2-6 shows a comparison of the behavior of conventional and high-performance exposed column bases.

Table 2-9 gives the final design parameters, including the total building weight (W), design base shear (V), and the fundamental period of the structure (T).

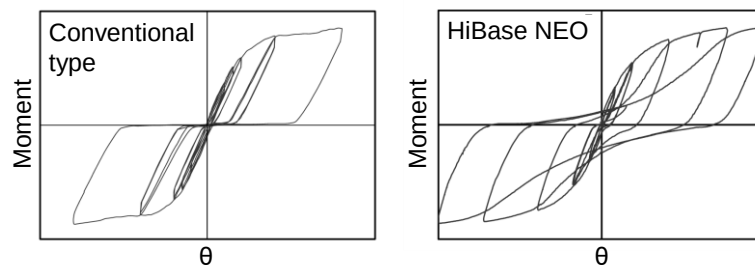


Figure 2-6 Comparison of behavior of (a) conventional and (b) high-performance exposed column bases.

Table 2-9 Design parameters and periods of the benchmark building

	$T_c(s)$	$T(s)$	Z	I	R_t	$W(kN)$	$V(kN)$	V/W
SMRF4F	0.6	0.410	1.0	1.5	1	1637	327	0.200
SMRF6F	0.6	0.737	1.0	1.0	0.99	57430	11367	0.198
SMRF10F	0.6	1.226	1.0	1.0	0.78	101061	15833	0.157

2.4.2 Numerical modeling and validation

All beams and columns were simulated using force-based, nonlinear beam-column elements combined with fiber-type sections, each with five integration points. The panel zone effect was considered in the analysis using the Krawinkler model, which, as shown in Figure 2-7, simulates the panel zone of a rectangle composed of eight very stiff elastic beam-column elements, with one zero-length element serving as a rotational spring to represent shear distortions. At the three corners of the panel zone without the rotational spring, the elements are joined by a simple pin connection, achieved by constraining both translational degrees of freedom using the equalDOF command. The spring has a trilinear backbone which is created with the Hysteretic material and derived using the principle of virtual work applied to the deformed shape of the panel zone [2-18]. The P-delta effect of the frame was included, and the floor weights were distributed on the beams at each floor level of the models. The node horizontal degrees of freedom were constrained to the centre-of-mass node at each floor level to simulate the rigidity of the slabs. The uniaxial material model of Steel02, using the yield strength and the kinematic strain hardening ratio obtained from the material tests, was applied to each fiber. Uniformly distributed gravity loads were applied along the length of each beam at each story in the model. Moreover, lumped masses were applied at nodes of beam-to-column joints in the frame for dynamic analysis. The model of the exposed column base was composed of a base plate and anchor rods [2-19]. The base plate was modeled

with a fiber section assigned to nonlinear elements outside the column depth and elastic elements with 10 times the elastic modulus inside the column depth to simulate the rigidity in between the column depth. The anchor rod was modeled with a circular fiber section assigned to a force-based nonlinear element. The material property of the anchor rods was a Hysteretic material in series with the rigid elastic no-compression material to simulate the slip behavior. The concrete foundation was simulated using truss elements with the material property of Concrete01, so-called Winkler springs. These truss elements were equally spaced along the length of the base plate, as shown in Figure 2-7. A simulation approach for the structure subjected to ground motions, incorporating the adopted anchor rod retightening repair method, was established using the numerical model outlined here. The command to remove components from the model in OpenSees was used to replace the anchor rod elements in the structure with new material properties, simulating the repair process between two ground motion inputs. After checking the longitudinal deformation data of the anchor rods at the end of the ground motion, the damaged components will be removed and assigned the new material property with reduced ductility to simulate the retightening process.

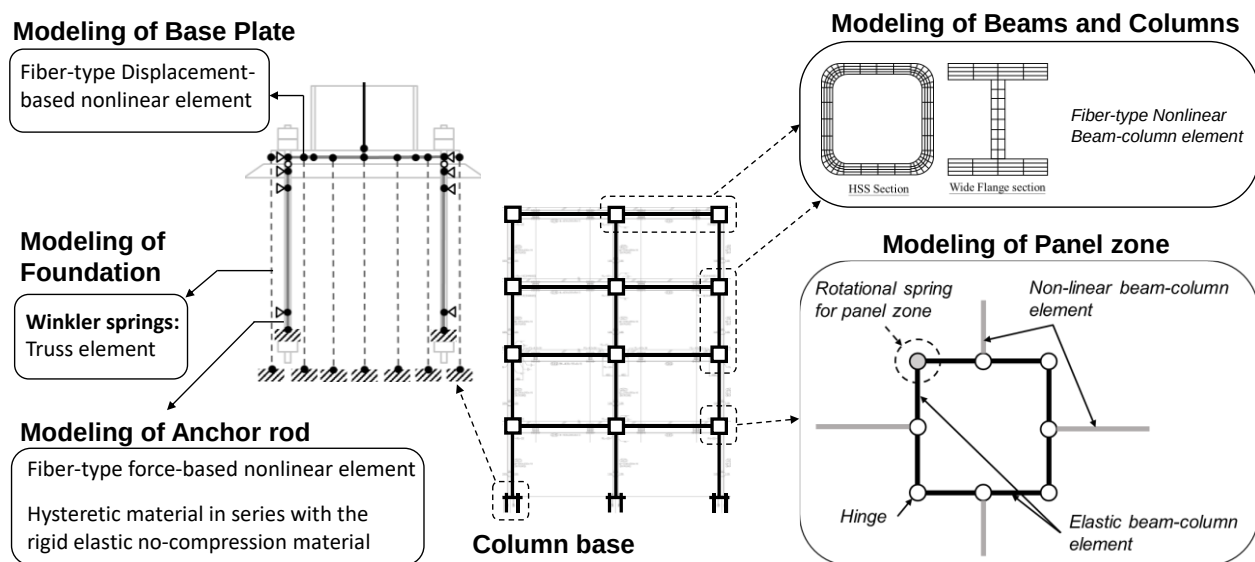


Figure 2-7 Illustrations of adopted numerical model of SMRFs.

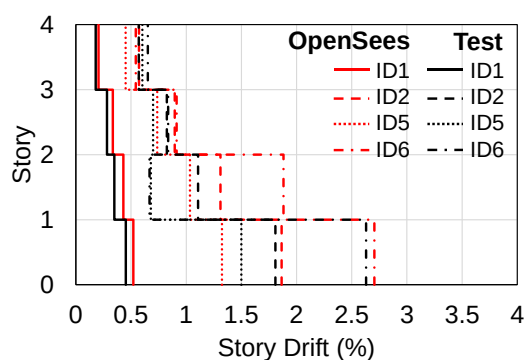
The proposed model was validated using data from component tests and the E-defense test of a four-story building, conducted through both static and dynamic analyses. In the E-defense shaking table test, JMA Kobe NS and long-period ground motions OS2 were scaled to provide six ground motions, including the elastic level, life safety level, and 1.5 times the design level, as shown in Table 2-10. The structural damage can be summarized by the observation after each ground motion in the test. From ID1 to ID3, where the ground motion was scaled to the elastic level, the structure remained elastic. After the ground motion of ID4, yielding was observed in the second-floor beam ends and column bases. In the large long-period ground motion of ID5, slabs cracking in the second and third floors were observed. In the ground motion of ID6, two anchor rods of the column bases fractured 14 seconds after inputting acceleration. In static component tests, the specimens were subjected to lateral cyclic loading and axial force, which corresponded to ID4 and ID5 in the shaking table test.

The resulting analytical responses of the frame system and column bases in the X direction are shown in Figures 2-8 and 2-9, compared with the measured responses. The errors of the X direction maximum first-floor drift in the simulation and experimental results of the ground motion input IDs 1, 4, 5, and 6 are 15.3%, 3.1%, -11.7%, and 2.9%, respectively. The maximum acceleration errors under the ground motion input ID1 to ID4 are 29.2%, 6.6%, 14.6%, and -2.5%, respectively. The errors in the X and Y direction maximum acceleration in ground motion input ID5 and 6 are -18.6%, -5.8%, -21.9%, and -51.3%, respectively. The larger difference under the ground motion of ID6 is that two anchor rods of the column bases fractured at 14 seconds after inputting the ground motion of ID6, resulting in a soft story effect. In the simulation, we did not simulate the fracture of anchor rods.

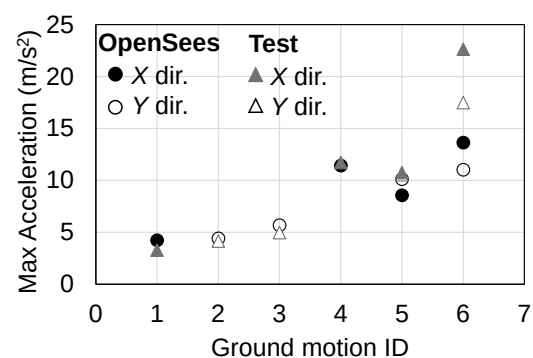
The analytical results of each specimen in the static test are shown in Figure 2-10 compared with the experimental responses. It is shown that the numerical model generally enables the capture of the responses of the frame system, including maximum story drift and acceleration, as well as the behavior of column base specimens and beam-column connections in terms of initial stiffness and strength backbones. It should be noted that the fiber-type discrete models were based on the plane-section-remain-plane assumption, which was unable to simulate the strength degradation caused by local buckling around the member ends in static tests, while no strength degradation was observed in the shaking table test. Therefore, the effect of the strength degradation of the beam ends was considered negligible for the effectiveness of the repair process in the numerical model.

Table 2-10 A list and details of the loading cases

ID	Ground Motion	Reference
1	JMA Kobe NSX dir. 16%	Elastic limit
2	JMA Kobe NSY dir. 16%	Elastic limit
3	OS-2Y dir. 20%	Long period small
4	JMA Kobe NSX dir. 50%	Life safety limit
5	OS-2XY dirs. 50%	Long period large
6	JMA Kobe NS XY dirs. 50%, UD Z dir. 100%	1.5-time life safety limit



(a)



(b)

Figure 2-8 Comparisons of simulated and measured shaking table test results: (a) maximum story drift and (b) maximum acceleration.

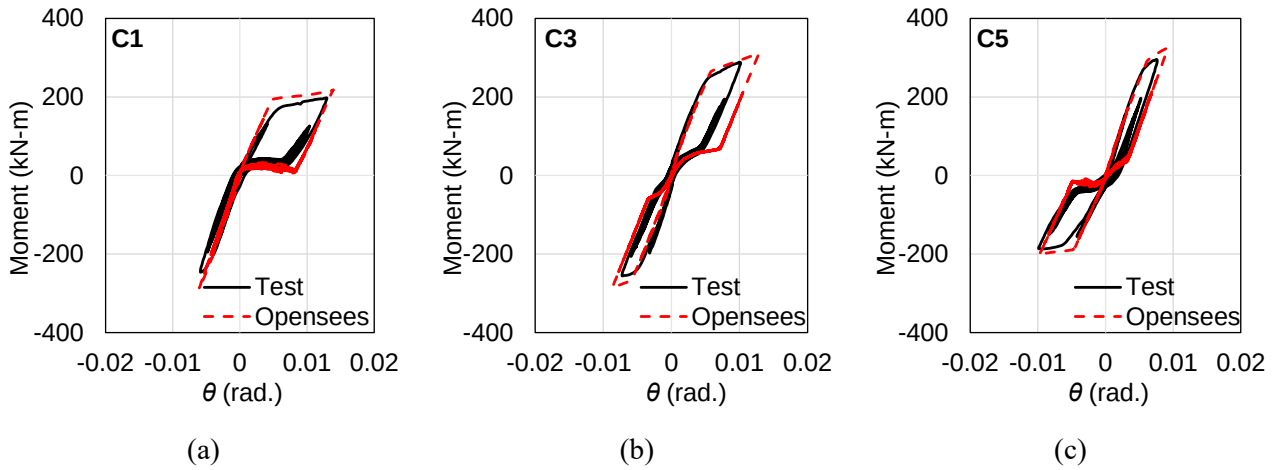


Figure 2-9 Comparisons of simulated and measured ID4 shaking table test results of column bases after re-tightening of anchor rods.

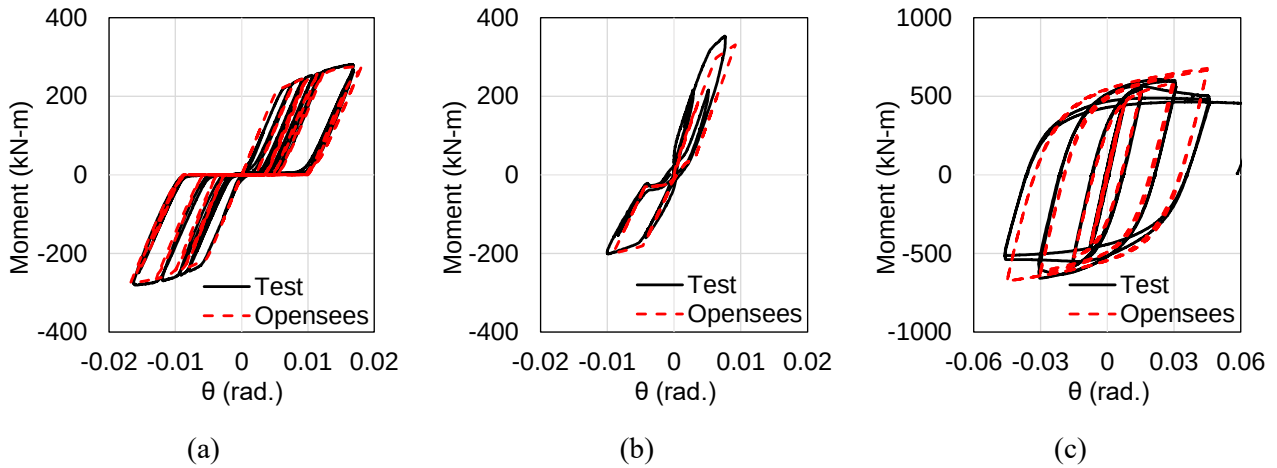


Figure 2-10 Simulated and measured static loading results: (a) column base (without axial force); (b) column base (with axial force); (c) beam-to-column connection.

2.5 Applicability and effectiveness of re-tightening anchor rods

In the shaking table test, after inputting the long-period ground motion of ID5, the natural frequency was measured before and after adopting the re-tightening process of anchor rods in the E-defense shaking table test as shown in Figure 2-11. The experimental results showed that the natural frequency of the frame system recovers 90% and 85% of its original value in X and Y direction respectively after the anchor rods of column bases were re-tightened. However, even with the adoption of the re-tightening process of anchor rods, the natural frequency of the structure does not return to its original undamaged state. This is likely due to slab cracking and the panel zone yielding, that occurred on the second floor observed in the test. In addition, the coupling effect of bi-directional attack on the column base, and the cracking of the base-plate packing mortar also resulted in overall stiffness reductions. These test observations also explained why simulated natural frequencies were larger than the test results for the higher intensity ground motions.

The analytical results of the natural frequency after each input of ground motions are shown in Figure 2-11 comparing with the experimental responses. It is shown that the results of the test and simulation have a similar trend. It should be noted that an additional re-tightening process was performed after inputting the ground motion of ID4 in the simulation because a significant decrease in the natural frequency was observed in both the test and the simulation.

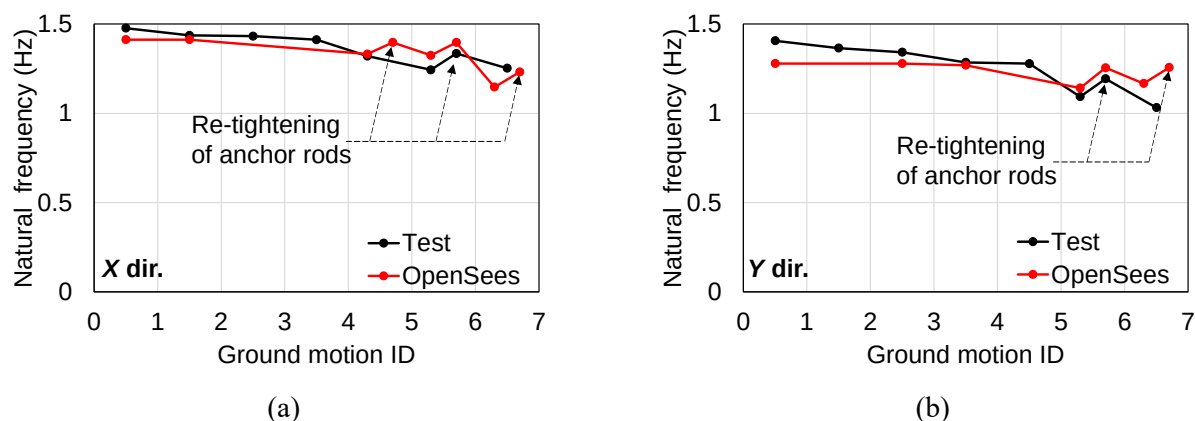


Figure 2-11 Comparisons of simulated and measured natural frequency in (a) X direction and (b) Y direction.

To verify the effectiveness of adopting the retightening process for anchor rods in the structural system, dynamic analyses were performed with and without the retightening process of anchor rods for each input ground motion using the developed numerical model mentioned above.

The ground motion measured at the Japan Meteorological Agency (JMA) Kobe station (the NS direction) during the Hyogo-ken Nanbu earthquake in 1995 was used. These ground motions were scaled to four levels, ranging from 16% to 100%, representing the elastic level, design level, and 1.5 times the design level for the dynamic analysis, as shown in Table 2-11. It is noted that the ground motions A1 and A2 have experimental data from the shaking table test, but ground motions A3 and A4 do not.

Table 2-11. A list and details of the ground motions

ID	Ground motion	Reference	Test data
A1	JMA Kobe NS 16%	Elastic level	Yes
A2	JMA Kobe NS 50%	Design level	Yes
A3	JMA Kobe NS 75%	1.5-time design level	No
A4	JMA Kobe NS 100%		No

Nonlinear time history analysis involves subjecting the models to ground motions that are scaled to multiple levels of intensity. After anchor rods have yielded under an earthquake, there will be a gap between the anchor rod nut and the base plate due to the plastic elongation of the anchor rods. This repair method aims to rapidly restore the overall stiffness of the exposed column base and reduce the risk and damage of aftershocks

to the structures. However, the retightened anchor rod will reduce its deformation capacity and ductility. Therefore, the anchor rods used with the expectation that retightening might be applied will need to have sufficient deformation and energy dissipation capacity to satisfy design and performance expectations pre- and post-repair. The retightening process of anchor rods was executed after A2 and A3 in the analysis since the components remained elastic after A1, as shown in Figure 2-12.

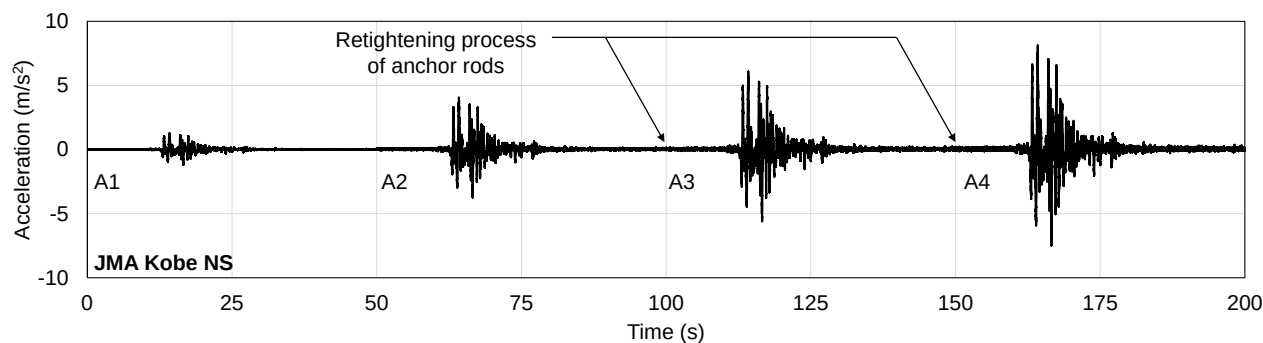


Figure 2-12 Illustrations of sequential earthquakes of JMA Kobe NS and retightening process.

Seismic performance evaluations of the buildings were performed through time-history analysis under both sequential and single ground motions. The dynamic results of each building were evaluated and discussed through the maximum inter-story drift. The analysis results for the four-story building under the ground motions of A1 and A2 are consistent with the shaking table test results, which indicate that yielding was observed in the second-floor beam ends and column bases. It is considered that the seismic responses of the structure are within the life safety level as per FEMA 356 (2000). The results show that the maximum inter-story drift of the four-story building were 3.6% and 4.9% in X direction and 3.8% and 4.3% in Y direction at the ground motion of A3 and A4 respectively, as shown in Figure 2-13, and it is considered that the structure is within performance level of collapse prevention. In addition to the low-rise building, the seismic performance in mid- and high-rise buildings was also evaluated. Figures 2-14 and 2-15 show the maximum inter-story drift in the six- and ten-story buildings. At A1, where ground motion was scaled to elastic level, both structures were undamaged, while the maximum inter-story drift was 1.91% and 1.41% in the Y direction for both buildings at the ground motion of A4, and it is considered that the structure is within the performance level of life safety. The overall responses of inter-story drift in six- and ten-story buildings are smaller than those in the four-story building due to relatively strong design. Additionally, the maximum inter-story drift occurred at the first, second, and fourth stories of the four-, six-, and ten-story buildings, respectively, indicating the influence of different building heights.

A single ground motion of A3, representing the maximum considered earthquake (1.5 times the design level), was input and compared with the seismic responses obtained by adopting the retightening process of anchor rods at each sequential ground motion. The results of single ground motion are slightly smaller than those of sequential ground motion with retightened anchor rods for all the buildings, indicating that the structural components other than column bases have accumulated damage from previous ground motions.

The dynamic results, with and without the retightening process for anchor rods, are compared through seismic responses. For the four-story building, after inputting the ground motion of A2, the maximum story drift on the first floor after inputting the ground motion of A3 is reduced by 9.6% and 7.7% in the X and Y direction, respectively, as shown in Figure 2-13. For the six- and ten-story buildings, the responses of maximum inter-story drift with and without the retightening process of anchor rods are similar, as shown in Figures 2-14 and 2-15. It is considered that the column bases used for the six- and ten-story buildings exhibit a behavior where the base plate yields before the anchor rods, and the contributions of the column bases in the six- and ten-story buildings are relatively small compared to those in the four-story building.

To summarize, the retightening process of the anchor rod can reduce story drift after a design-level earthquake, such as A2, and rapidly restore the overall stiffness of the column base, thereby recovering functionality for low-rise buildings that adopt the weak-base/strong-column design. However, the improvement of the retightening process for the high-rise building is limited. It should be noted that further study and analysis are necessary to verify the effectiveness of the retightening process for anchor rods in low-rise cases with varying numbers of spans.

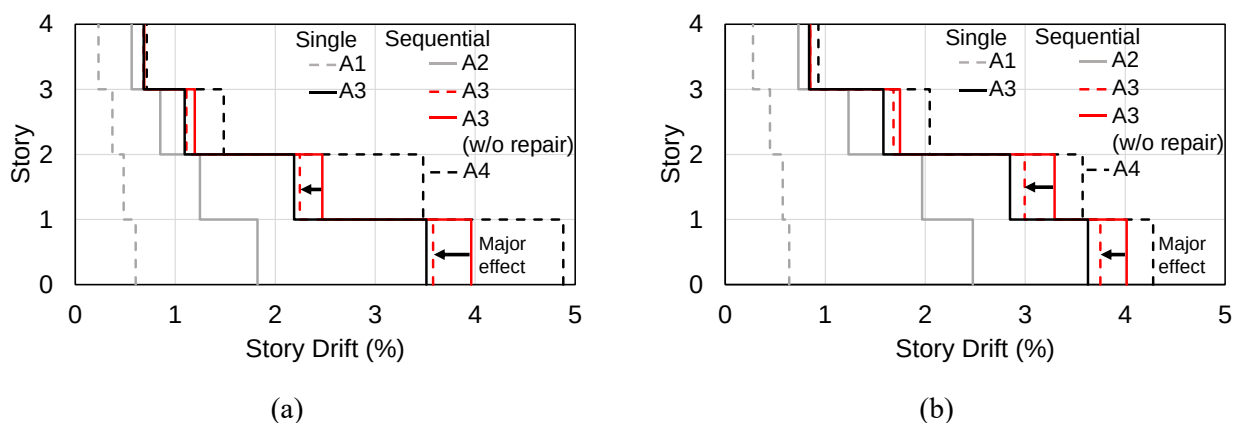


Figure 2-13 The dynamic responses of four-story SMRF in (a) X direction and (b) Y direction.

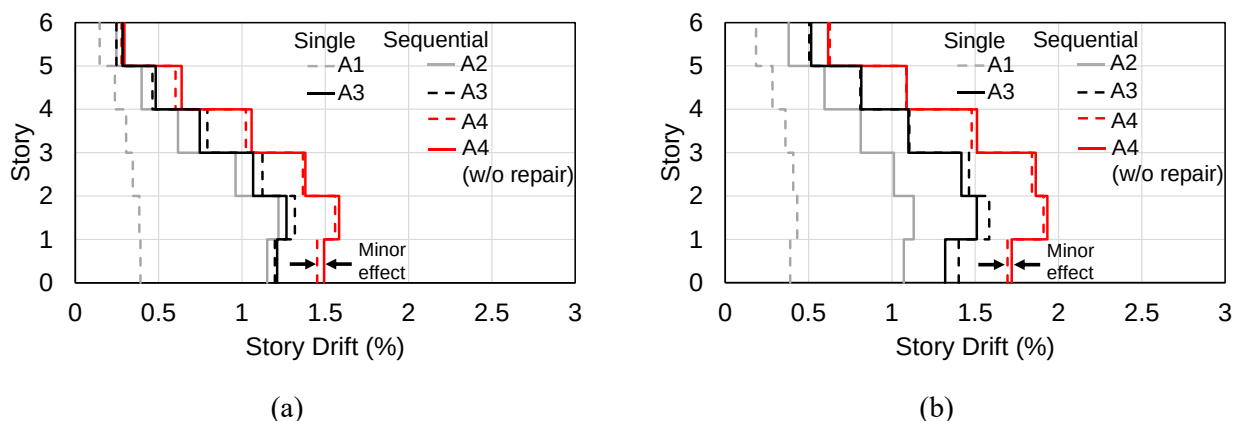


Figure 2-14 The dynamic responses of six-story SMRF in (a) X direction and (b) Y direction.

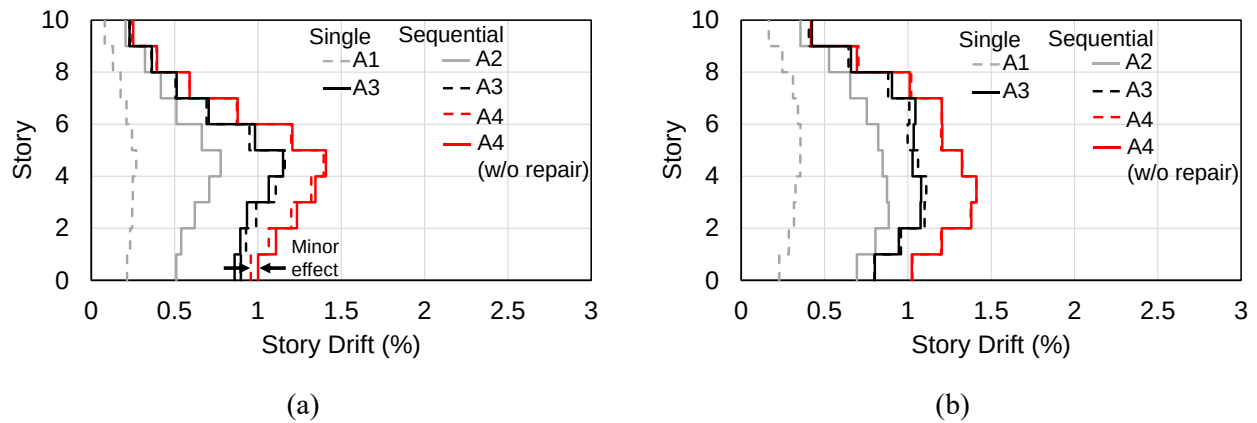


Figure 2-15 The dynamic responses of ten-story SMRF in (a) X direction and (b) Y direction.

2.6 Summary

This chapter presents an investigation focusing on a standard repair method for damaged exposed-type column bases. An effective repair method for retightening the anchor rods, aiming to rapidly reinstate the overall stiffness of the exposed column base after an earthquake, was examined through experimental testing and numerical analyses. A series of dynamic analyses were performed for four-, six-, and ten-story benchmark buildings using a discrete modeling approach through OpenSees frameworks to quantify the applicability and effectiveness of this repair method. The major findings are summarized as follows.

- (1) The repair method is not always applicable for standard anchor rods, which are assumed for pin or fixed column bases, to remain elastic. This chapter introduced and integrated the Japanese and foreign specifications for the required deformation capacity of ductile anchor rods.
- (2) The test data for threaded anchor rod deformation capacity with different steel grades and screw threads commonly used in Japan and abroad were prepared to verify the required deformation capacity of anchor rods through the tensile test and analyses.
- (3) A simulation method for replacing anchor rod elements throughout the entire structure's life was established using OpenSees frameworks and validated by a shaking table test. It is demonstrated that the numerical model effectively captures the frame system response, including maximum story drift and acceleration, as well as the local behavior of column base specimens and beam-column connections in terms of initial stiffness and strength backbones.
- (4) A simple repair method, involving the retightening process of anchor rods, was proven to be effective for low-rise buildings through analysis, which showed that the maximum story drift on the first floor was reduced by 9.6% and 7.7% when the structure was subjected to subsequent ground motion. In contrast, the effectiveness of this repair method for mid- and high-rise buildings is not significant. The effectiveness of this simple repair method will be influenced by the contribution of column bases to the system, the different types of exposed column bases, and the building's story height.

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CHAPTER 3

POST-EARTHQUAKE DECISION-MAKING METHODOLOGY FOR EXPOSED COLUMN BASES

3.1 Introduction

3.1.1 Background

As verified in Chapter 2, the simple repair method of re-tightening, intended to rapidly restore the overall stiffness of exposed column bases and reduce the risk and damage from aftershocks, has proven effective. However, re-tightened anchor rods may exhibit reduced deformation capacity and ductility. Therefore, when re-tightening is anticipated, the anchor rods must possess sufficient deformation capacity to satisfy design and performance expectations before and after repair.

To ensure the feasibility of the simple repair approach, engineers and building stakeholders must assess its reliability. Accordingly, an inspection and assessment framework is required to determine whether a damaged column base can be expected to perform adequately in future earthquakes with minimal or no repair, or whether more extensive retrofitting is necessary.

This chapter introduces a decision-making methodology proposed for identifying the damage states of anchor rods and selecting appropriate repair or retrofit strategies for the exposed steel column base. Then, the damage levels and residual capacities of both the original and repaired column bases are examined through dynamic analyses under multiple ground motion scenarios.

3.1.2 Organization

Section 3.2 introduces a decision-making methodology based on the structural reliability theory, incorporating both the required ductility factor and the remaining deformation capacity of the anchor rods. Section 3.3 presents the ductility responses of anchor rods obtained from dynamic analyses and employs reliability methods to evaluate the residual performance of exposed column bases.

3.2 Proposed retrofit methodology for column bases

A repair methodology that identifies the damage states of anchor rods and selects appropriate repair/retrofit methods for the exposed steel column base is proposed. The repair methodology, illustrated in Figure 3-1(a), consists the following steps:

- 1) Following the earthquake, conduct a visual inspection of anchor rods at column bases, particularly at the corners of the structure/where higher demands are anticipated.
- 2) Assess the plastic deformation of anchor rods through visual inspection or other methods, or loose nuts/washers. If there is no deformation, no retrofit is required.
- 3) If there is deformation, determine the required and residual ductility of deformed anchor rods through mill sheet and visual inspection.
- 4) Evaluate the probability of failure of anchor rods. If the probability of failure, P_f , is less than 0.05, the simple repair method for column bases, re-tightening of anchor rods, can be adopted.

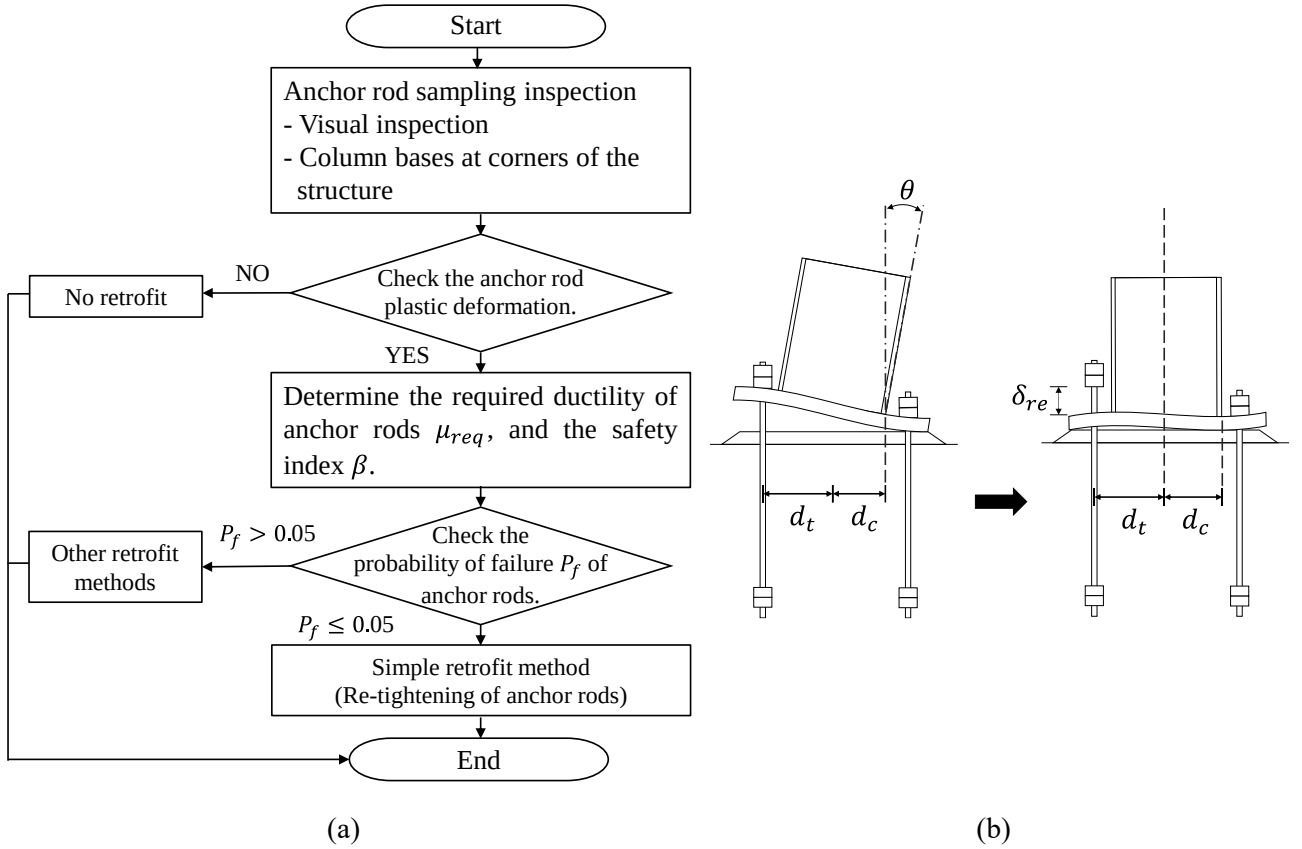


Figure 3-1 Column base repair methodology: (a) flowchart; (b) anchor rod plastic deformation.

The probability of failure can be estimated by comparing the remaining and required ductility for anchor rods. The required ductility factor μ_{req} of anchor rods, as shown in Figure 3-1(b), can be estimated by Eq. (3.1).

$$\mu_{req} = \frac{\theta(d_t + d_c)}{\delta_y} \quad (3.1)$$

where d_t is the distance between anchor rods under tension and the column centerline; d_c is the distance between the outer column surface under compression and the column centerline; δ_y is the displacement at the yield strength of anchor rods; θ is the required plastic rotational capacity of the column base. It should be noted that

the value of θ was suggested referring to the global acceptance criteria that the residual story drift shall not exceed $0.015 h_{sx}$ per ASCE 7-22 [3-1], where h_{sx} is the height of story below level x . In addition, the value of θ can also be referred to the reparability limit base on the actual data of earthquakes that have a maximum residual story drift of $1/71$ [3-2].

After determining the required ductility factor of the anchor rods, their remaining deformation capacity needs to be identified and confirmed to satisfy the performance requirements as defined via statistical methods. As mentioned above, the re-tightened anchor reduces its deformation capacity after each ground motion. Once the remaining ductility factor of the anchor rod becomes less than the required ductility factor, the anchor rod has a higher possibility of failure in the next earthquake. The failure possibility of anchor rods can be estimated by the basic structural reliability theory as shown in Figure 3-2(a) and 3-2(b) obtained by Eqs. (3.2) to (3.5).

$$Z = R - S \quad (3.2)$$

$$\mu_z = \mu_R - \mu_S \quad (3.3)$$

$$\sigma_z^2 = \sigma_R^2 + \sigma_S^2 \quad (3.4)$$

The equations consider the remaining ductility R and the required ductility S of anchor rods. Each is described by a known probability density function, $f_R()$ and $f_S()$. The safety margin Z has a mean and variance given by Eqs. (3.3) and (3.4). The probability of failure, P_f , of the anchor rod can be stated in Eq. (3.5).

$$P_f = \Phi \left[\frac{-(\mu_z)}{\sigma_z} \right] = \Phi(-\beta) \quad (3.5)$$

where $\Phi()$ is the standard normal distribution function extensively tabulated in statistics texts; β is defined as the safety index.

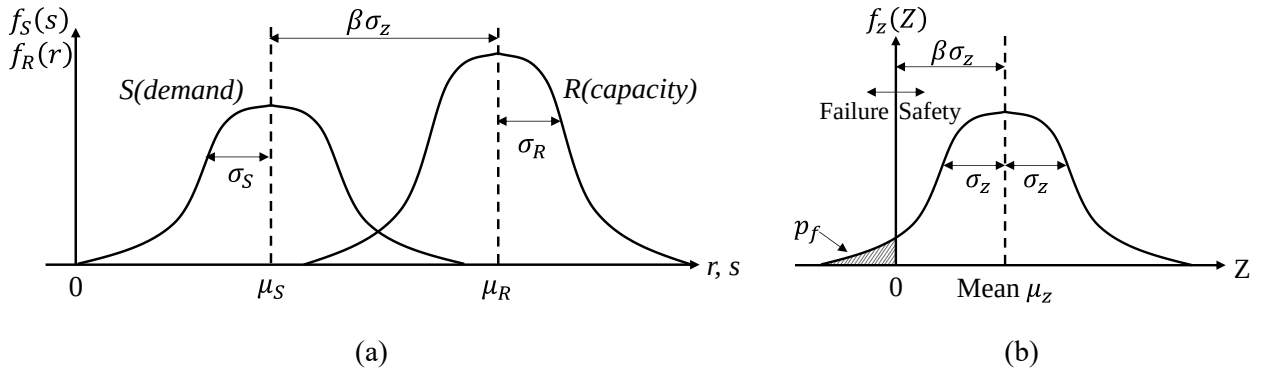


Figure 3-2 Failure probability of anchor rods: (a) reliability index when R and S are independent and normally distributed; (b) with $Z = R - S$ denoting the safety margin.

The applicability of a re-tightening repair method can be assessed by referencing the probability of failure of anchor rods and the corresponding safety index. The acceptable probability of failure serves as a critical index

for representing structural performance requirements. However, this threshold can vary significantly depending on factors such as the risk category of buildings, loading conditions, and failure type. In Step 4 of the above flowchart, the target failure probability of 0.05 is assumed, based on the target reliability criteria for the conditional limit states where failure could pose a substantial risk to human life, per ASCE 7-22 [3-1].

As shown in Figure 3-3, the ductility factor of anchor rods can be checked after each ground motion by the proposed repair flowchart. Suppose the resulting probability of failure of anchor rods is less than 0.05, which indicates that anchor rods have sufficient deformation capacity, and a simple repair method of re-tightening the anchor rods can be adopted. Conversely, if the resulting probability of failure exceeds 0.05, the anchor rods are deemed to have an unacceptable risk of failure in future earthquakes, necessitating more robust retrofit measures, such as reinforced concrete encasing or the installation of supplemental members. It should be noted that the repair methodology assumes the column base and anchor rod had been correctly installed and tightened before the earthquake event.

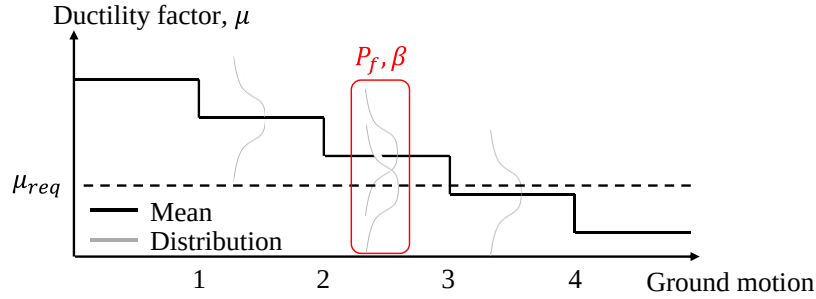


Figure 3-3 Remaining ductility capacity and failure probability of anchor rods after ground motions.

3.3 Dynamic analysis

The residual performance of the four-story SMRF, used at the E-Defense shaking table test reported in Chapter 2, was evaluated through the dynamic analyses.

The re-tightened anchor rod reduces its ductility by the re-tightening process, and thus, the remaining ductility is essential for adopting this repair method. The remaining ductility of anchor rods after each ground motion is examined by dynamic analysis and test results to check the applicability of the re-tightening process. To quantify it, the ductility factor μ of anchor rods can be calculated by Eq. (3-6).

$$\mu = \frac{\delta_{u,0} - \delta_{re}}{\delta_y} \quad (3-6)$$

where δ_y is the yield displacement and $\delta_{u,0}$ is the displacement at the ultimate tensile strength before the earthquake; δ_{re} is the permanent displacement resolved by the re-tightening process. For example, the anchor rod used in the four-story SMRF constructed for the shaking table test had an initial ductility factor of 7.5 calculated from the tensile test result. The required ductility of 2.44 was calculated by Eq. (3-1) with the θ of 0.015 rad.

Sequential ground motion analyses were conducted using the OpenSees model developed in Chapter 2. Table 3-1 lists the reduced ductility factor of anchor rods calculated based on the maximum longitudinal deformation of the anchor rod. Using the reliability method mentioned in Section 3.2, the ductility factor and the probability of failure of the anchor rods after each ground motion were estimated. The remaining deformation capacity of anchor rods, shown in Figure 3-4(a), where the mean and standard deviation were estimated by fitting to the normal distribution, distributions of the ductility factor of a total of 24 anchor rods, as well as the probability of failure of anchor rods, were compared.

Table 3-1 lists the ductility factor of each anchor rod after ground motions. The analytical results show that the ductility factor of anchor rods remains unchanged before the ground motion of ID4, indicating that the anchor rods were not damaged. After the ID4 ground motions, which correspond to a life-safety level earthquake, the mean value of the ductility factor of anchor rods was reduced by 12.7%. Following the ground motion of ID5 and ID6, which represented large, long-period levels and a 1.5-time life safety level earthquake, the mean values of the ductility factor of anchor rods were 66.9% and 36.1% of the initial value, respectively. The degree of dispersion of the ductility factor of anchor rods becomes larger after inputting several accelerations due to the direction of ground motions and the location of column bases. Figure 3-4(b) shows the distribution of the remaining ductility factor of anchor rods after the ground motion of ID6. The analytical results show that one of the anchor rods in each of the column bases C1, C2, and C3 has a ductility factor less than zero, while fractures of anchor rods observed in the shaking table test were also at the exact location of the column bases C1 and C3. The responses indicated that the re-tightening process of anchor rods would be applicable to rapidly repair column bases if the remaining ductility of anchor rods was sufficient.

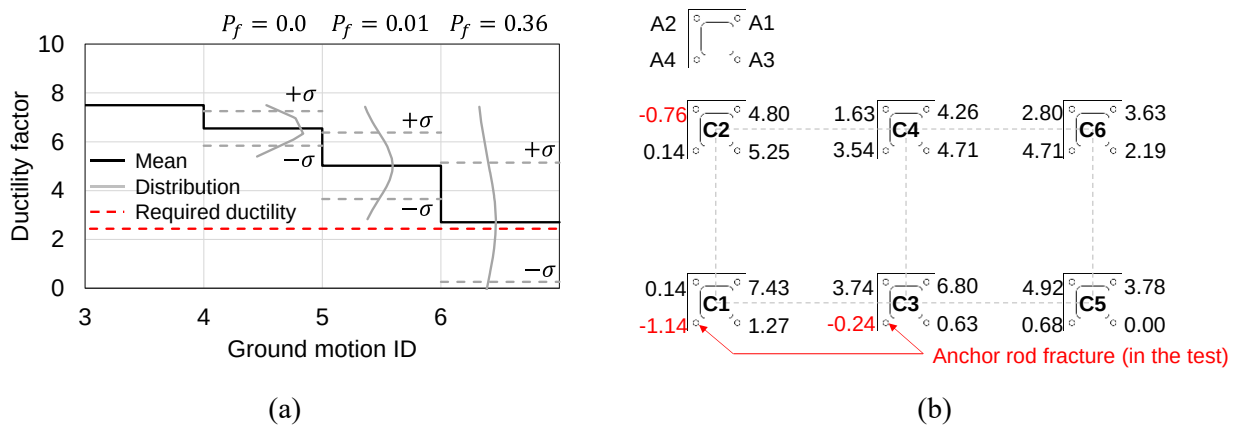


Figure 3-4 Anchor rod evaluation: (a) remaining ductility for 4-story SMRF (b) ductility factor after ground motion ID6.

Table 3-1 Ductility factor of anchor rods in 4-story SMRF

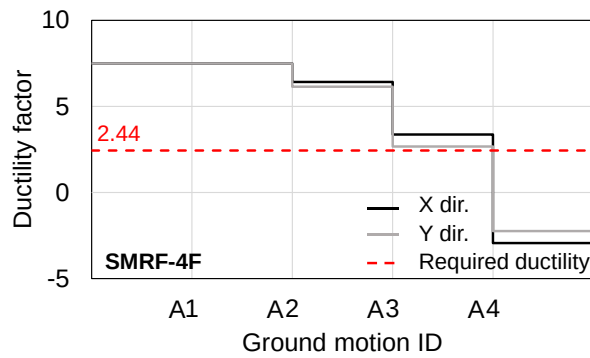
Column No.	C1				C3				C5			
Anchor rod No.	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
Initial to ID3	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50
ID4	7.50	5.40	7.50	5.40	6.96	6.09	6.96	6.09	6.33	7.01	6.33	7.01

ID5	7.43	4.11	5.04	2.83	6.80	6.01	4.40	3.53	4.17	6.94	3.77	4.45
ID6	7.43	0.14	1.27	-1.14	6.80	3.74	0.63	-0.24	3.78	4.92	0.00	0.68
Column No.	C2				C4				C6			
Anchor rod No.	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
Initial to ID3	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50
ID4	7.50	5.40	7.50	5.40	6.96	6.09	6.96	6.09	6.33	7.01	6.33	7.01
ID5	5.31	3.21	7.22	4.11	4.78	3.90	6.68	5.81	4.17	4.83	4.17	6.73
ID6	4.80	-0.76	5.25	0.14	4.26	1.63	4.71	3.54	3.63	2.80	2.19	4.71

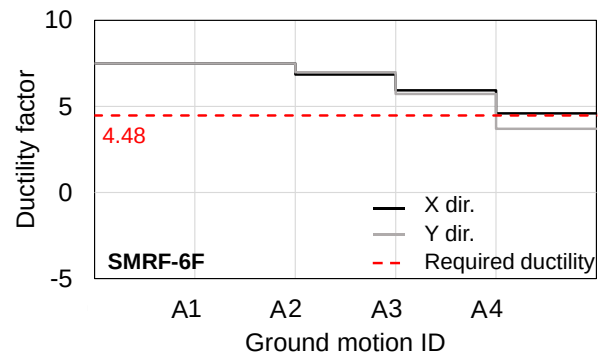
In addition to the ground motions used in the shaking table test, the scaled ground motions listed in Table 2-11 were employed to evaluate the reparability and residual capacity of column bases. It should be noted that each column base has eight anchor rods in the six- and ten-story SMRF and assumes the same initial ductility factor of 7.5 as the anchor rod used in the 4-story SMRF in the analysis. The required ductility of 4.48 and 5.16 was calculated by Eq. (3-1) with the θ of 0.015 rad for the column bases in six- and ten-story SMRF, respectively.

The remaining ductility of anchor rods in the four-story SMRF is shown in Figure 3-5(a), where the mean value of the remaining ductility factor of anchor rods in two directions, as well as the required ductility, was compared. The average ductility factor μ , safety index β , and probability of failure P_f of anchor rods after each ground motion are listed in Tables 3-2 and 3-3. The results indicate that the probability of failure for anchor rods is 0.0 and 0.007 in the X and Y directions, respectively, for a four-story SMRF following the ground motion of A2, which is below the 0.05 limit specified in step 4 of the flowchart. This suggests that the simple repair method, involving re-tightening, is feasible. Thus, tightening the permanently deformed anchor rods observed in step 2 can rapidly revert the overall stiffness of column bases with sufficient deformation capacity. However, the probability of failure of anchor rods for the four-story SMRF after the ground motion of A3 exceeds the limit of 0.05 in step 4, indicating that the ductility of anchor rods will be insufficient during the next seismic event and may result in severe damage to the structure. Therefore, other retrofit methods are necessary to ensure the required capacity of column bases.

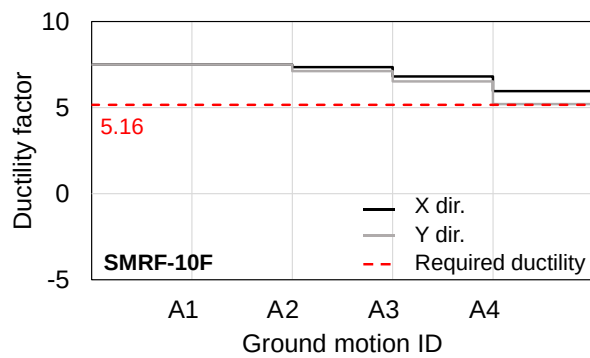
The remaining and required ductility of anchor rods in six- and ten-story SMRF is shown in Figure 3-5(b) and 3-5(c), and the safety index β and probability of failure P_f of anchor rods are listed in Tables 3-2 and 3-3. The results indicate that the simple repair method, involving re-tightening the anchor rods, can be applied to six- and ten-story SMRFs following the A2 ground motion, provided the probability of failure of the anchor rods meets the requirements. The probability of failure of anchor rods exceeds the limit of 0.05 in step 4 for both buildings after ground motion A3, which represents the MCE level earthquake, indicating that the fracture at anchor rods may occur in upcoming earthquakes or aftershocks. The analytical results also show that all buildings had the maximum cumulative damage to the column base at the corners of the structure, which can be used to identify the damage level and residual performance of column bases.



(a)



(b)



(c)

Figure 3-5 Average remaining ductility of anchor rods in (a) four-story SMRF, (b) six-story SMRF, and (c) ten-story SMRF.

Table 3-2 Remaining ductility factor, safety index, and probability of failure of anchor rods in X direction

X dir.	SMRF-4F			SMRF-6F			SMRF-10F		
ID	μ	β	P_f	μ	β	P_f	μ	β	P_f
A1	7.50			7.50			7.50		
A2	6.42	4.93	0.000	7.50	4.52	0.000	7.35	5.50	0.000
A3	3.38	0.26	0.397	6.86	1.56	0.059	6.81	1.55	0.061
A4	-2.90	-0.84	0.800	5.94	0.08	0.468	5.96	0.37	0.356

Table 3-3 Remaining ductility factor, safety index, and probability of failure of anchor rods in Y direction

Y dir.	SMRF-4F			SMRF-6F			SMRF-10F		
ID	μ	β	P_f	μ	β	P_f	μ	β	P_f
A1	7.50			7.50			7.50		
A2	6.15	2.42	0.007	7.50	3.35	0.000	7.12	2.23	0.012
A3	2.67	0.06	0.476	6.86	0.82	0.206	6.52	0.66	0.255
A4	-2.24	-0.73	0.768	5.94	-0.29	0.614	5.20	0.01	0.496

3.4 Summary

Chapter 3 presents a decision-making methodology for the post-earthquake repair and retrofit of exposed steel column bases. Building on the findings from Chapter 2, a performance-based framework was developed to evaluate the applicability of a simple repair method of re-tightening of anchor rods, which is intended to rapidly restore the overall stiffness of column bases and reduce the risk of further damage during aftershocks. The methodology was formulated through structural reliability theory, incorporating inspection procedures, ductility evaluation, and failure probability calculations. Dynamic analyses and experimental data were utilized to verify the feasibility of the repair method for various building scales. The major findings are summarized as follows.

- (1) A repair decision-making flowchart was proposed to evaluate the damage condition of anchor rods and determine appropriate repair or retrofit actions. The flowchart includes steps for visual inspection, evaluation of plastic deformation, estimation of required and remaining ductility, and the use of reliability theory to assess the probability of failure.
- (2) The residual deformation capacity of anchor rods after each ground motion is examined by dynamic analysis and test results to check the applicability for adopting this repair method. A reliability-based methodology was adopted to calculate the probability of anchor rod failure. When the probability of failure was less than 0.05, the simple repair method was considered acceptable. Conversely, if the probability exceeded the limit, the anchor rods were deemed insufficient for future seismic demands.
- (3) When the simple repair method is no longer feasible due to excessive damage or low remaining ductility, other retrofit options must be considered. In this study, a retrofit technique using curved members was proposed as an alternative to ensure the required performance of damaged column bases. The details and evaluation of this retrofit system will be introduced in the next chapter.

Reference

- [3-1] ASCE 7-22 (2022), “Minimum Design Loads and Associated Criteria for Buildings and Other Structures”, American Society of Civil Engineers (ASCE).
- [3-2] Iwata Y, Sugimoto H, Kuwamura H. Reparability limit of steel structural buildings based on the actual data of the Hyogoken-Nanbu earthquake. Proc. of the 38th Joint Panel. Wind and Seismic effects. NIST Special Publication 2006;1057(23-32):30.

CHAPTER 4

PLASTIC BEHAVIOR AND DESIGN METHOD FOR CURVED MEMBERS IN SEISMIC APPLICATIONS

4.1 Introduction

4.1.1 Background

When the simple re-tightening repair method is no longer applicable due to excessive damage or insufficient residual deformation capacity of anchor rods, as discussed in Chapter 3, alternative retrofit strategies must be adopted to ensure the continued seismic safety of structures. However, conventional seismic retrofitting techniques, such as the addition of reinforced concrete encasing, dampers, or base isolation devices, result in substantial disturbances to both the structure and its occupants. These interventions can alter the architectural layout, increase force demands on existing members due to added stiffness, and in many cases, necessitate temporary relocation of residents [4-1].

For exposed column bases, conventional retrofit approaches often involve reinforced concrete encasement or foundation enlargement [4-2], which typically demand excavation, on-site concrete casting, and long curing periods. These methods not only require significant construction effort but can also alter the seismic behavior of the frame system. In low-rise buildings, where column base flexibility contributes more prominently to the overall dynamic characteristics, such retrofit methods can lead to unintended changes in natural period and redistribution of internal forces, thereby affecting the seismic demand on existing members.

To address this challenge, this study proposes the development of a retrofit technique that utilizes the advantages of curved members designed to enhance the seismic performance of damaged column bases while ensuring minimal disturbance to the existing structural and architectural configuration. However, the plastic behavior of these members, particularly the yield, buckling, and ultimate strengths under various boundary conditions, remains insufficiently investigated. Unlike straight members, curved members lack well-established design equations and connection design guidelines, limiting their use for earthquake-resistant applications.

This chapter investigated the plastic behavior of curved members through static analysis and extensive parametric studies, focusing on initial elastic stiffness, plastic strength, slenderness effects, and section compactness. Estimation equations for yield strength and buckling behavior with pinned- and fixed-boundary conditions were developed and validated using numerical simulations, demonstrating high accuracy. Moreover, a boundary constraint condition was identified to effectively control the yielding mechanism and achieve the

desired plastic behavior. Additionally, the bidirectional effect of the proposed curved member retrofit system was investigated to evaluate its capacity to improve seismic performance under loading from multiple directions. These outcomes also serve as the basis for the comprehensive evaluation of the curved member retrofit system in Chapters 5 and 6.

4.1.2 Organization

Section 4.2 investigates the plastic behavior of curved members through extensive analytical studies. The fundamental properties of curved members, including plastic strengths, initial elastic stiffness, and the influence of slenderness and compactness, are examined through parametric studies with various configurations and geometric dimensions using numerical models and finite element methods. The findings contribute to developing estimation equations that accurately predict curved member behaviors.

The impact of boundary constraint conditions on curved member behavior is evaluated, and an adequate boundary constraint condition is identified to control the yielding mechanism and ensure the expected plastic behavior in Section 4.3.

Section 4.4 investigates the bidirectional effect of the curved member retrofit system under multi-directional earthquake loading to assess its performance under realistic seismic conditions.

4.2 Curved member behavior and estimation equations

Curved members in this study, which refer to steel members formed through bending processes, have gained attention in architectural and structural design because of their aesthetics, structural efficiency, and functionality. Steel has been used for curved members for a long time due to its great workability. In architecture, curved members are commonly used for canopies, spiral staircases, pedestrian pathways, etc. Roof members of station buildings, stadiums, and other large-span structures are often composed of curved members. As nonstructural elements, curved members are frequently employed in noise barriers and other auxiliary structures, demonstrating their versatility. Overall curved form is also achieved by assembling straight members with angular connections, which create an overall curved form by connecting straight members at specific angles.

Curved members are typically fabricated using cold bending or hot bending processes. An alternative approach is to assemble a curved member by welding curved-shaped web plates and bent plates. Cold bending is a method of processing mainly at room temperature to 720 degrees Celsius, with the feature of densifying the metallographic structure of steel. Cold bending is usually more economical than hot bending or induction-heating bending, but it will lead to residual stresses, initial imperfections, and increased yielding stress due to strain hardening [4-3, 4-4, 4-5]. Hot bending, particularly induction-heating bending, can mitigate these issues but requires strength verification after processing, especially in the application of plastic deformation regions.

Vertically curved members in structures are subjected to combined axial compression and in-plane flexural loads, while horizontally curved members need to resist both flexural and torsional moments [4-3, 4-6]. The second-order effects and the span angle of curved members affect the strength and the basic behavior [4-7, 4-8]. The initial eccentricity provided by the curvature of the member provides flexibility in design regarding the combination of stiffness and strength. Straight members, which are placed diagonally in structures, serve as

braces or retrofit members, such as concentrically braced frames (CBFs) and buckling-restrained braced (BRB) frames. These are efficient structural systems but have limited flexibility in selecting the combination of stiffness and strength. Braces designed by force demand often lead to significant stiffness addition, which increases the seismic acceleration demand to buildings and, therefore, the base shear.

Many studies described below have verified the benefits of intended eccentricity provided by the member. For instance, intended eccentricity can provide a characteristic of lateral stiffness loosely coupled with the yield strength and the ability to effectively fulfill multiple seismic design objectives regarding stiffness, strength, and ductility [4-9]. Applications of eccentricity in braces, known as crescent-shaped and curved knee braces, have been proposed to enhance seismic performance through their unique geometric configuration [4-10, 4-11, 4-12]. The intended initial eccentricity also controls the buckling direction, avoids the severe loss of strength due to buckling, and places the member into post-buckling behavior [4-13]. A novel brace featuring a curved geometry and partial strengthening through induction heating has been proposed. The curved deformation along the brace length has been proven to enhance the stability of its compressive behavior [4-14].

4.2.1 Strength and stiffness

The curved member has a yield mechanism different from the straight member due to the intended initial eccentricity and the resulting additional end moments. In this study, curved members are defined as the arc of a circle, characterized by the height (H), width (L), central angle (θ_L), and radius (R), and the curvature is assumed to be constant along its length, as defined in Figure 4-2. The resulting initial eccentricity (e) is introduced by curvature and serves as a key design parameter, which is defined as the maximum vertical distance between the arc and the chord, representing the inherent initial curvature of the curved geometry. In the derivation of equations, the internal forces were obtained from the case where a downward force is applied to the midpoint of two arches, as illustrated in Figure 4-1. In contrast, when applying the curved members as seismic retrofit elements, a horizontal load Q is assumed to apply at the upper end of the curved member supported at the lower end, as shown in Figure 4-2. Both pinned and fixed boundary conditions are considered in the study. Figure 4-2 shows a considered alignment and nomenclature of a curved member used as a diagonal member subjected to axial force pointing from one end to the other. The internal forces of the curved member, including bending moment, shear force, and axial force, $M(\theta)$, $Q(\theta)$, and $N(\theta)$, can be calculated by Eqs. (4-1) to (4-3).

$$Q(\theta) = Q \sin \theta - N \sin \theta \quad (4-1)$$

$$N(\theta) = -Q \sin \theta - N \cos \theta \quad (4-2)$$

$$M(\theta) = RQ \sin \theta - RN(1 - \cos \theta) \quad (4-3)$$

where Q is the external force applied to the top end of the curved member; θ is the angle used to express the specific location on the curved member. The radius R and the central angle θ_L of the curved member can be determined directly from the geometric parameter height H and width L , as shown in Figure 4-2. The relation between R , H , and L is given by Eq. (4-4) while the central angle θ_L can be expressed as Eq. (4-5), which is derived from the right-triangle geometry defined by H and L . Notably, the curved member is assumed to be

tangent to the vertical line at its upper endpoint. Under this assumption, the geometry of the curved member is uniquely defined by the ratio H/L , and there is only one corresponding value of eccentricity e .

$$R = \sqrt{L^2 + H^2} / \left(2 \sin \frac{\theta_L}{2} \right) \quad (4-4)$$

$$\theta_L = 2 \tan^{-1}(L/H) \quad (4-5)$$

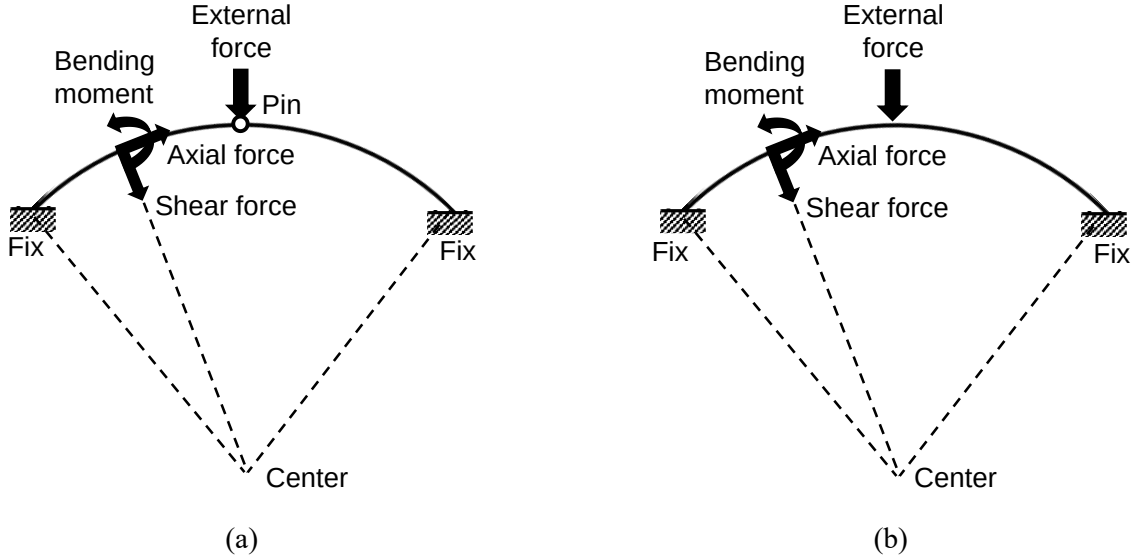


Figure 4-1 Schematic of (a) the arch with a hinge at the center and (b) a typical arch.

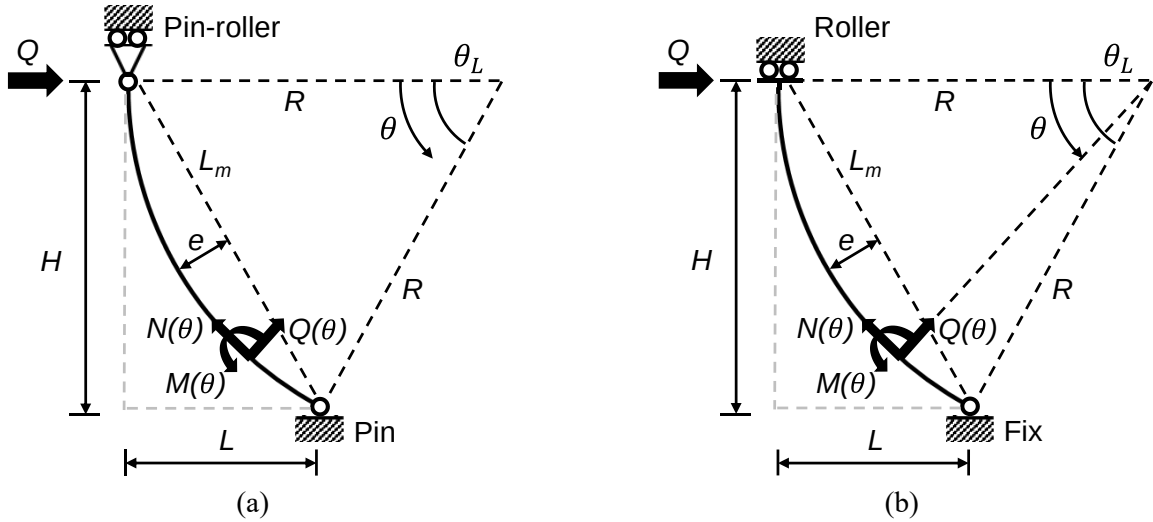


Figure 4-2 Schematic and nomenclature for a curved member used as a diagonal member with boundary conditions of (a) pin-roller to pin and (b) roller to fix.

The initial eccentricity within the curved member reduces the elastic stiffness and the yield strength in both horizontal and longitudinal directions. A recent study presented that introducing initial curvature can reduce the ultimate strength of horizontally curved members by up to 50% [4-15]. For pinned constraint conditions at the

ends, the simple virtual work principle derives the initial elastic stiffness in the horizontal direction, K_e , as Eq. (4-6). Similarly, the estimation of the yield strength, Q_y , in the horizontal direction becomes Eq. (4-7).

$$K_{e,pin} = 1/\left[1 + \left(\frac{H}{L}\right)^2\right] \left[\frac{2R^3}{EI} \left(\frac{1}{4} \sin \frac{\theta_L}{2} + \frac{\theta_L}{4} - 2 \cos \frac{\theta_L}{2} \sin \frac{\theta_L}{2} + \frac{\theta_L}{2} \cos^2 \frac{\theta_L}{2} \right) + \frac{2R}{EA} \left(\frac{1}{4} \sin \theta_L + \frac{\theta_L}{4} \right) \right] \quad (4-6)$$

$$Q_{y,pin} = \sigma_y / \left[\frac{\sqrt{(H/L)^2 + 1}}{A} + \frac{L[(H/L)^2 + 1]}{2S} \left(\sqrt{(H/L)^2 + 1} - \frac{H}{L} \right) \right] \quad (4-7)$$

where E and σ_y are the elastic modulus and yield stress of the steel material; H and L are the height and the width of the curved member; A is the cross-sectional area; S and I are the section modulus; and the moment of inertia of the cross-section.

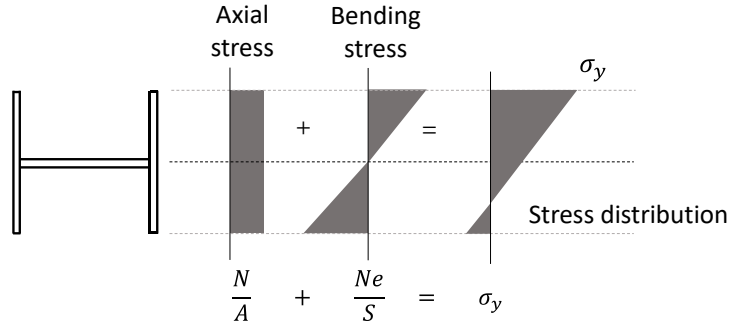


Figure 4-3 The stress distributions on a curved member.

Alternatively, equations to evaluate the initial elastic stiffness and yield strength in the horizontal direction were established using the elastic beam-column theory as Eqs. (4-8) and (4-9). The maximum bending moment occurs at the middle of the curved member. Eq. (4-9) is the yield strength evaluated through the stress distribution over the cross-section under the combined axial and bending stress, as shown in Figure 4-3. Additionally, the yield strength ($N_{y,pin}$) in the direction of the line (L_m) connecting the two ends of the curved member can be calculated by Eq. (10).

$$K_{e,pin} = \frac{(EA/\sqrt{H^2 + L^2})}{[1 + (0.73e/r)^2]} \times \cos^2(\tan^{-1} H/L) \quad (4-8)$$

$$Q_{y,pin} = \left[\frac{\sigma_y A}{1 + (eA/S)} \right] \times \cos(\tan^{-1} H/L) \quad (4-9)$$

$$N_{y,pin} = \frac{\sigma_y A}{1 + (eA/S)} \quad (4-10)$$

where e is the intended initial eccentricity, r is the radius of gyration of the cross section. The initial eccentricity varies with the height, width, and central angle of the curved members, and it can be calculated using the following Eq. (4-11). The coefficient of 0.73 in Eq. (4-8) represents the effect of moment distribution along the

member, i.e., the ratio of the maximum moment to the average moment on the curved member, based on the parametric analysis results described later.

$$e = \frac{\sqrt{L^2 + H^2}}{2 \sin \frac{\theta_L}{2}} \left(1 - \cos \frac{\theta_L}{2} \right) \quad (4-11)$$

To develop the estimation method of the elastic stiffness and the plastic strength for the curved member with fixed constraint conditions, a parametric analytical study was conducted using a numerical model of curved members developed using the OpenSees framework [4-16]. The curved members were modeled with fiber-type force-based nonlinear beam-column elements. The P-delta effect was included and the uniaxial material model of Steel02 in OpenSees was assigned to each element with nominal properties of SS400 steel. Based on a sensitivity study and in accordance with AISC specification [4-17], each curved segment was discretized into ten elements to ensure model robustness, as illustrated in Figure 4-4. A monotonic loading was applied for all cases to a drift ratio of 0.05 radians.

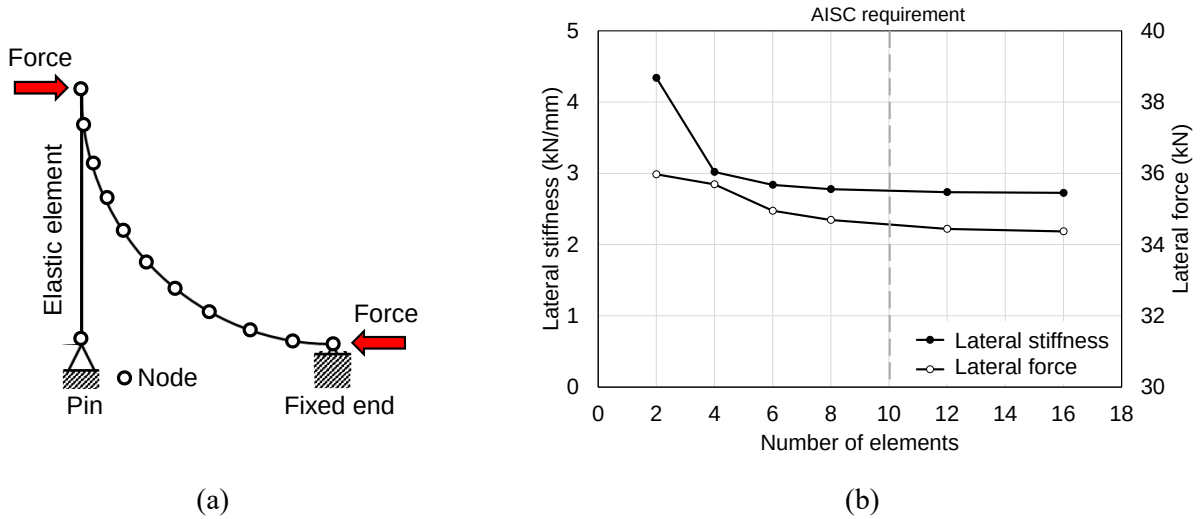


Figure 4-4 (a) Numerical model of curved member and (b) results of sensitivity analysis.

Figure 4-5 defines geometric parameters, and Table 4-1 summarizes the primary design parameters. The H over L ratio varies from 1 to 10, and the overall length (L_m), the shortest distance between the two ends of the curved member, varies from 1,005 to 1,414 mm. All the simulated curved members satisfy the seismically compact sections per specification in Japan. In total, 99 cases were analyzed for the combinations of major dimensions (d , b , t_w , and t_f), geometric parameters (H/L), and various sections, including open and closed sections.

Table 4-1 A list of major design parameters of cases

Cases	Section type	d (mm)	b (mm)	t_w (mm)	t_f (mm)	H/L	L_m
1	H section	200	200	8	12	1~10	1005~1414
2		250	175	7	11		
3		300	150	6.5	9		
4	C section	150	75	9	12.5		
5		200	80	7.5	11		
6		300	90	9	13		
7	HSS section	150	100		9		
8		200	100		9		
9		150	150		9		

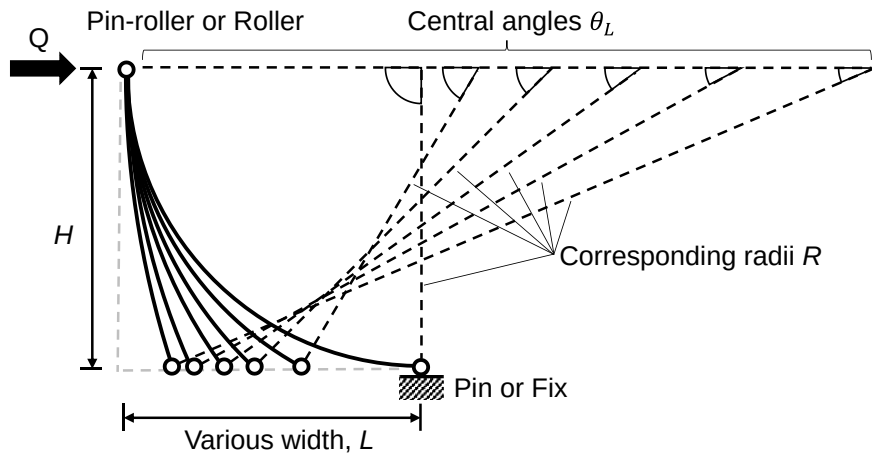


Figure 4-5 Schematic of geometric parameters of curved members for parametric analysis.

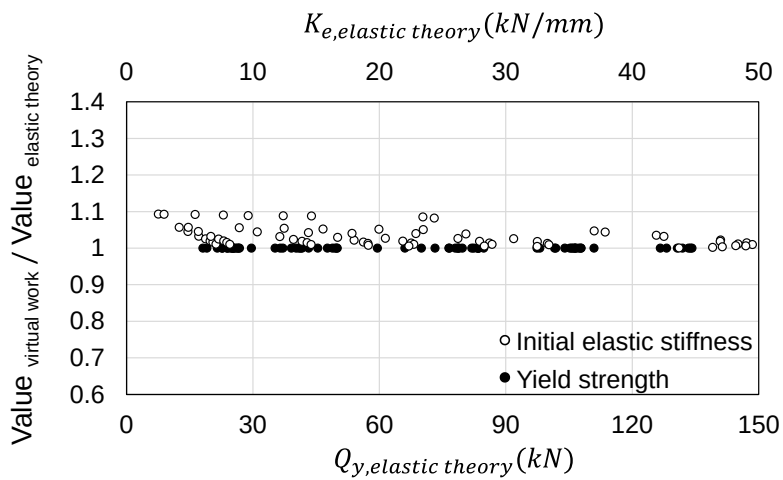


Figure 4-6 Comparisons of initial elastic stiffness and yield strength calculated by different methods.

The difference by estimation method was first checked for the 99 models. Figure 4-6 compares the results obtained by the two estimation methods. The horizontal axis represents the calculated stiffness and yield strength obtained from the elastic beam-column theory (Eq. (4-8) for stiffness and Eq. (4-9) for yield strength), while the vertical axis shows the ratio of the virtual work method to the elastic beam-column theory. The yield strength calculated by the virtual work principle, Eq. (4-7), and by the elastic beam-column theory, Eq. (4-9), are identical, and the initial elastic stiffness calculated by Eq. (4-6) and Eq. (4-8) differ by less than 10%.

The behaviors under monotonic loadings were analyzed in each curved member model with pinned and fixed end conditions in the OpenSees framework [4-16]. The analytical strength, $Q_{y,ana}$, was obtained at the first change of the elastic stiffness, $K_{e,ana}$, in the strength-displacement relationship curve.

In real applications, the behavior of curved members with fixed-end connections is important. However, establishing equations for estimating the yield strength and initial elastic stiffness is relatively tricky due to the intricate deformed shape and mechanism with fixed constraint conditions. Figure 4-7 shows the relationship between the geometric parameter (H/L) and the initial elastic stiffness, $K_{e,ana}$, and yield strength, $Q_{y,ana}$, of all the cases in the parametric analysis with fixed-end boundary conditions, where the top and bottom ends of the curved member are roller and fixed, respectively. The yield strength increases gradually as the H/L ranges from 1.0 to 2.5, attributed to the reduction in eccentricity. When the H/L exceeds 3.0, it decreases due to changes in the moment distribution along the curved members, as illustrated in Figure 4-8. The multiple lines in Figure 4-8 correspond to curved members with different cross-sections but the same H/L ratio used in the parametric analysis. The numerical results indicate that an H/L ratio of 1.0 produces a symmetric moment distribution, which aligns with the desired behavior. As the H/L ratio increases, the moment distribution changes, ultimately forming a double-curvature-like behavior when the H/L ratio reaches 10. Based on the analytical results, the range of H/L ratios between 1.0 and 3.0 is identified as the applicable range to achieve the desired behavior by having the initial plastic hinge at the middle of the curved members and stable behavior.

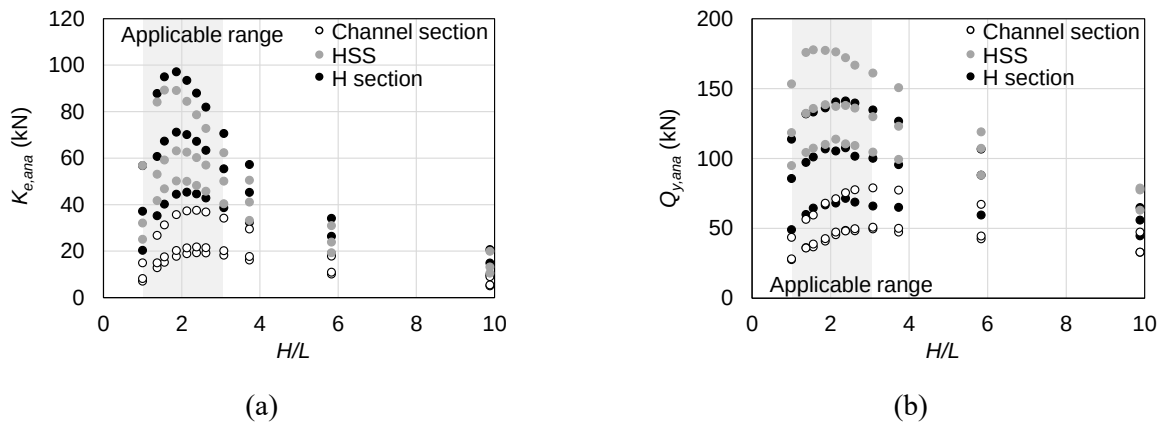


Figure 4-7 Relationship between the difference in H/L ratios and the (a) initial elastic stiffness and (b) yield strengths with fixed-end connections.

Based on the overall parametric analysis results, regression analyses were performed to develop the estimation equation of the curved members with fixed constraint conditions upon the analytical responses in the

parametric study. In the regression equation, an empirical factor (α) is used, which represents the ratio of the bending moment at the midspan of a curved member to the bending moment at the member ends and is expressed by the height (H) and the width (L) of curved members. The resulting regression equations from the parametric study are Eqs. (4-12) to (4-13). Additionally, the yield strength ($N_{y,fix}$) in the direction of the line (L_m) connecting the two ends of the curved member can be calculated by Eq. (4-15).

$$K_{e,fix} = \frac{(EA/\sqrt{H^2+L^2})}{[1+(0.73e/(1+\alpha)r)^2]} \times \cos^2(\tan^{-1} H/L) \quad (4-12)$$

$$Q_{y,fix} = \left[\frac{\sigma_y A}{1+(eA/(1+\alpha)S)} \right] \times \cos(\tan^{-1} H/L) \quad (4-13)$$

$$\alpha = 0.66(H/L)^{-0.17} \quad (4-14)$$

$$N_{y,fix} = \frac{\sigma_y A}{1+(eA/(1+\alpha)S)} \quad (4-15)$$

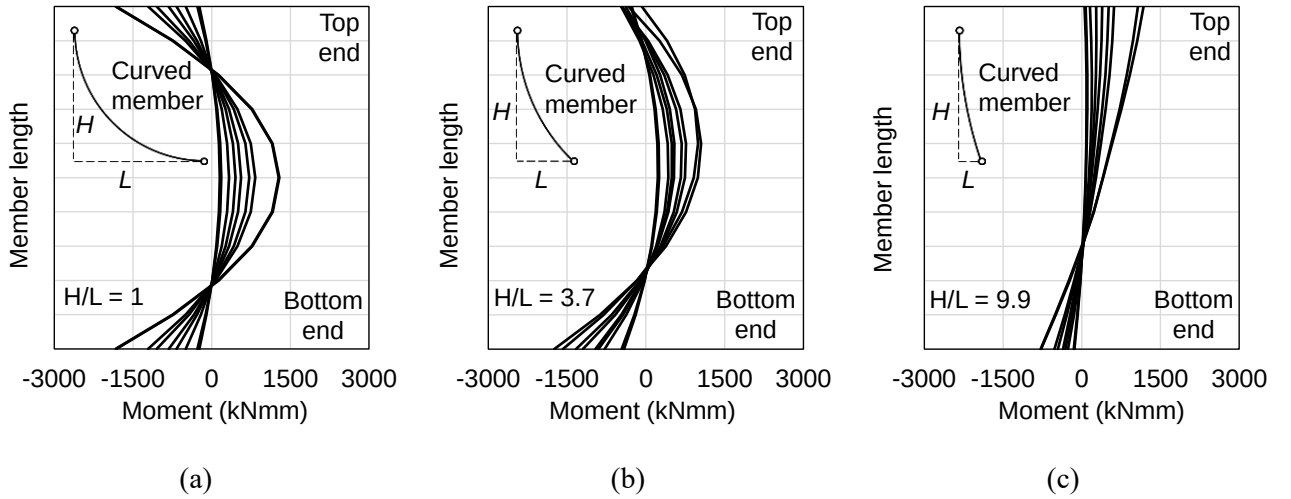


Figure 4-8 Moment distribution along the curved members with different H/L ratio.

Figure 4-9 compares $K_{e,ana}$ and $Q_{y,ana}$ with the results calculated by Eqs. (4-8) and (4-9) for pinned constraint condition and Eqs. (4-12) and (4-13) for fixed constraint condition, denoted as $K_{e,cal}$ and $Q_{y,cal}$. For pinned constraint conditions, the analytical and calculated results demonstrate good agreement, with errors confined to within $\pm 10\%$. Also, for fixed constraint conditions, the calculated and analytical values agreed well with less than 15% errors. Thus, the derived equations provide good accuracy in estimating the initial elastic stiffness and yield strength of the curved members with pinned constraint conditions.

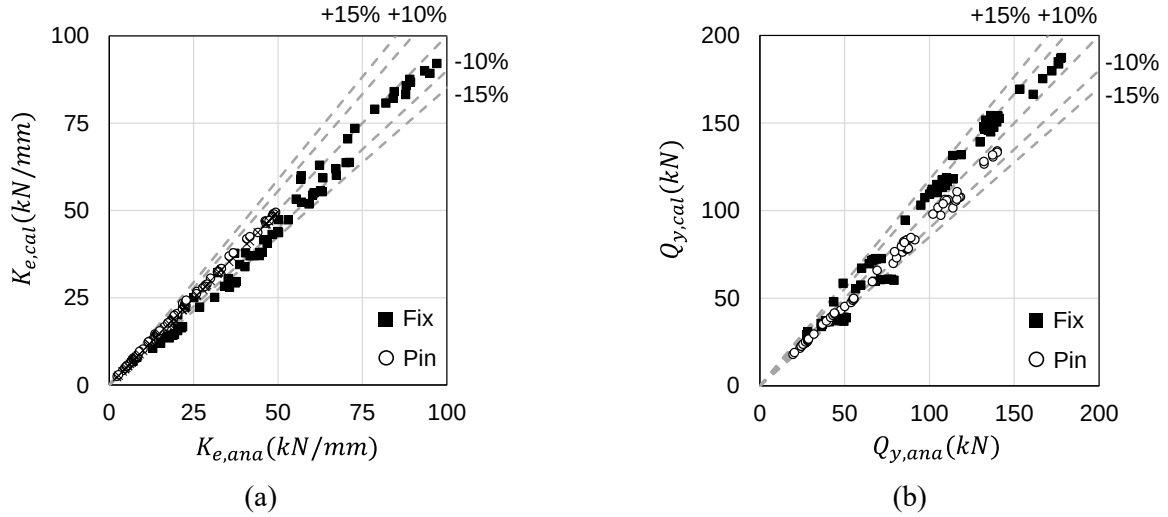


Figure 4-9 Comparisons of calculated and analytical (a) initial elastic stiffness and (b) yield strengths with different boundary conditions.

4.2.2 Slenderness effect

From a mechanics perspective, curved members can be regarded as bending elements and do not exhibit classical Euler-type instability in the same manner as straight compression members. However, when adopted as diagonal retrofit elements, they are primarily subjected to axial force along the member, and the curvature inherently introduces eccentricity, which produces additional bending moments and alters the internal force distribution. For members with relatively small eccentricity, buckling occurs in a manner similar to straight members, and the strength suddenly decreases after instability. In contrast, members with larger eccentricity exhibit a gradual strength reduction governed by inelastic buckling, where nonlinear axial-bending interaction controls the post-yield behavior. This distinction between small- and large-eccentricity cases is critical for developing estimation equations. Accordingly, the buckling strength of curved members in this study is defined as the maximum strength reached before a significant loss of load-carrying capacity, obtained from nonlinear static analysis with intended eccentricity. The following analyses clarify the influence of slenderness on buckling strength and provide the basis for the proposed equations.

The slenderness of a structural member is a critical factor influencing its stability and buckling behavior [4-18], making it essential to estimate buckling strength accurately. Slenderness is characterized by the slenderness ratio, defined as the effective length of the member divided by its radius of gyration, and it directly impacts the member's resistance to lateral deformation under axial compression. However, the buckling strength of curved members is a more complex consideration than straight members due to intended curvature and the associated changes in load distribution and deformation patterns. The initial eccentricity, which is inherently present due to the curvature, introduces a non-uniform distribution of internal forces, leading to a combination of axial compression and bending that can cause buckling modes different from straight members. The initial eccentricity can alter the buckling orientation of the curved member from the major to the minor axis, depending on the extent of the curvature radius.

In the previous section, the behavior of curved members was primarily discussed in terms of the height-to-

width ratio (H/L), which defines the global geometry of the arc. Since the geometry is assumed to be tangent to the vertical line at the upper endpoint, H/L uniquely determines the eccentricity e . In the following analyses, e is therefore used as an equivalent parameter to organize the results, as it provides a more direct representation of the initial curvature and its influence on buckling behavior.

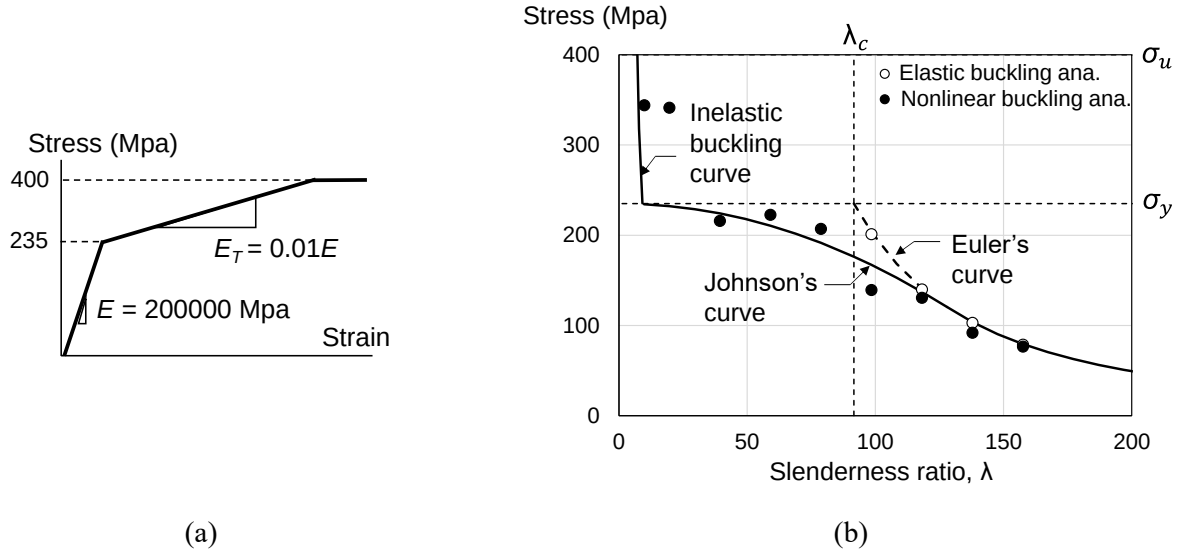


Figure 4-10(a) Material properties adopted for FEA; (b) comparisons of analytical and theoretical results of buckling strengths.

For the estimation equations for buckling strength to be evaluated, a series of finite element analyses (FEA) were performed using Abaqus [4-19, 4-20], a software application used for modeling and analyzing mechanical components. The FEA in the study is divided into two steps: the first is linear eigenvalue buckling analysis, and the second is inelastic buckling analysis. The former method estimates the critical buckling load and associated mode shapes by solving the eigenvalue problem, and the latter captures the effects of geometric nonlinearity, material nonlinearity, and initial imperfections for post-buckling behavior. For the model validation, the finite element models of straight members were first established with the boundary condition of pin-roller constraints and the member lengths ranging from 500 mm to 8,000 mm. Steel material (SS400) characterized by a trilinear stress-strain curve with a tangent modulus of 0.01 times the elastic modulus, H-shaped section ($200 \times 200 \times 8 \times 12$), and eight-node solid elements with reduced integration, which were used for meshing the members, were employed in the FEA, as shown in Figure 4-10(a). Following the elastic buckling analysis, the first buckling mode was extracted and scaled to introduce initial imperfections for inelastic buckling analysis. The analytical results obtained from both the elastic buckling analysis and the inelastic buckling analysis were compared with theoretical results, which were calculated using Euler's equation for elastic buckling, Johnson's equation for intermediate member which in the transition range between yielding and buckling, and the inelastic buckling curve for inelastic buckling strength, as shown in Figure 4-10(b). The analytical results showed good agreement with the theoretical curves, and the simulation method was subsequently applied to curved members for a parametric study.

In the initial buckling analysis for curved members, the member length of four cases varied from 2,000 mm to 8,000 mm, while the eccentricity was consistently maintained at 250 mm for all cases. Figure 4-11(a) shows the buckling curve of straight members in solid lines and the yield and buckling strength of curved members as circular markers. The curved members have yield and buckling strengths much smaller than those of straight members. Figure 4-11(b) shows the stress and displacement relationship of curved members, and the circular markers are the yield strength obtained from the analysis. The curved members exhibit nonlinear behavior before the yielding strength, but the maximum strength determined by inelastic buckling exceeds the yield strength. The analytical results indicate that the buckling strength exceeds the yield strength when the curved member has a smaller slenderness ratio. However, as the slenderness ratio increases, the buckling strength gradually converges toward the yield strength. Therefore, reasonable equations estimating the buckling strength are necessary for the design.

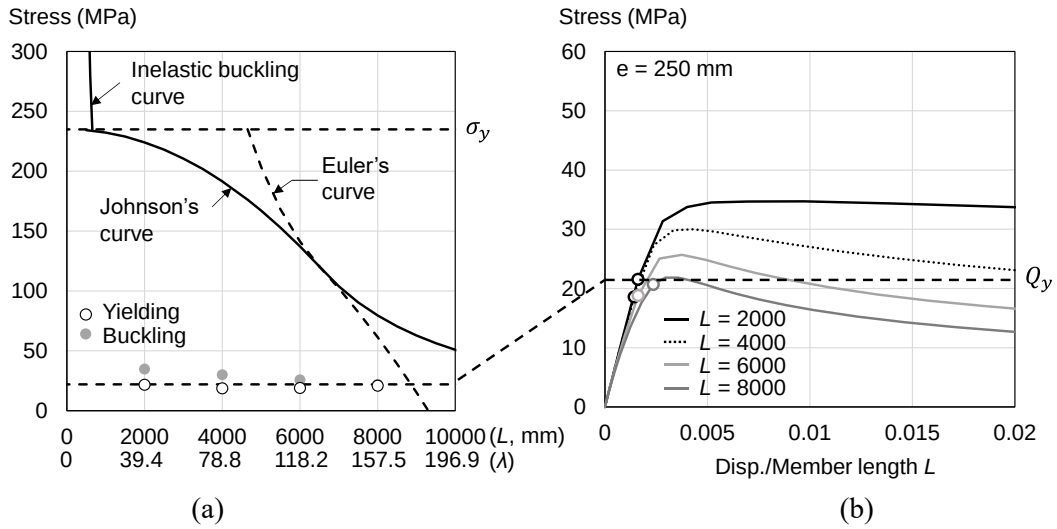


Figure 4-11(a) Comparisons of yielding and buckling strengths of curved and straight members; (b) differences between yielding and buckling strength of curved members.

The buckling strength of curved members is derived. The effect of initial imperfections, such as the initial eccentricity of a curved member, can be expressed by the following Eq. (16) and shown in Figure 4-12(a). The figure also presents a comparison between the sine curve and the circular arc curve, the latter of which was adopted in the parametric study in Section 4.2.1. The coefficient of determination, $R^2 = 0.98985$, demonstrates a great correlation, indicating the sine curve represents the arc curve shape with high accuracy.

$$u_0 = e \sin \frac{\pi z}{L_m} \quad (4-16)$$

where the initial deflection at the center of the member is e . The lateral deflection under an applied load, N , denoted by u can be expressed by Eq. (4-17). Using the principle of virtual work, the general solution to Eq. (4-17) is given by Eq. (4-18), where N_E is Euler's buckling load. The maximum lateral deflection, u_c , at the center

of the member can be calculated by Eq. (4-19).

$$EI \frac{d^2(u-u_0)}{dz^2} + Nu = 0 \quad (4-17)$$

$$u = \frac{e}{1-N/N_E} \sin \frac{\pi z}{L_m} \quad (4-18)$$

$$u_c = \frac{e}{1-N/N_E} \quad (4-19)$$

Figure 4-12(b) illustrates the relationship described in Eq. (4-18), with the initial eccentricity of the member serving as a parameter, and compared to analytical results. Without the initial eccentricity, lateral deflection occurs abruptly when the applied load reaches Euler's buckling load, indicating the critical point at which instability initiates. However, when an initial eccentricity is present, the deflection increases gradually as the load is applied, reflecting a more gradual buckling behavior. As load N approaches the critical buckling load, the deflection accelerates significantly. The analytical results show excellent agreement with the theoretical values, validating the accuracy of the models. The figure also demonstrates the inelastic buckling behavior of curved members. The yield stress of curved members illustrated in Figure 4-11(b) can be estimated using the same concept in Section 4.2.1, where stress is evaluated through the stress distribution of combined axial and bending stress over the cross-section by the following Eq. (4-20) and shown in Figure 4-13(a). Therefore, the yield stress of curved members is calculated by multiplying the yield stress σ_y of the straight member by a reduction coefficient ξ in Eq. (4-21).

$$\frac{N}{A} + \frac{Ne}{S} = \sigma_y \quad (4-20)$$

$$\frac{N}{A} = \sigma_y \xi \quad \text{where} \quad \xi = \left[\frac{1}{1+(eA/S)} \right] \quad (4-21)$$

For the inelastic buckling strength of curved members, the concept of residual stress on buckling behavior that flanges of the cross-section partially reach the plastic state with the equivalent moment of inertia was assumed based on the results of the buckling analyses. Figure 4-13 shows the stress distribution at the buckling state, and the estimation equations were divided into two conditions, single-side and double-side yielding of the flange, where the gray segment is the plastic region and the white segment is the elastic region.

Under the condition of single-side yielding of the flange, as depicted in Figure 4-13(b), two unknowns, including the width of the flange (B_p) that reaches a fully plastic state and the ratio (a) of yield stress to the sum of the combined axial and bending stress. The symbols with upper script of cr , cr' represent the stresses at the buckling state for the two sides of flanges, and the subscripts of N , M represent the stresses obtained from the axial force and bending moment, respectively, as shown in Figure 4-13.

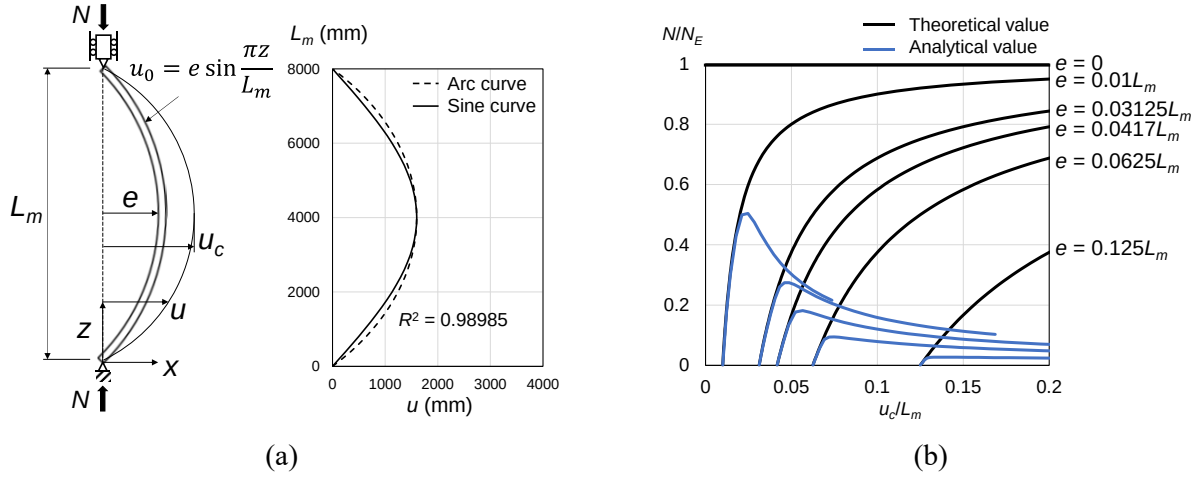


Figure 4-12 (a) Schematic of parameters of the curved member and comparison with the arc curve (b) Relationship between the buckling loads and maximum lateral deflection u_c at the center of curved members.

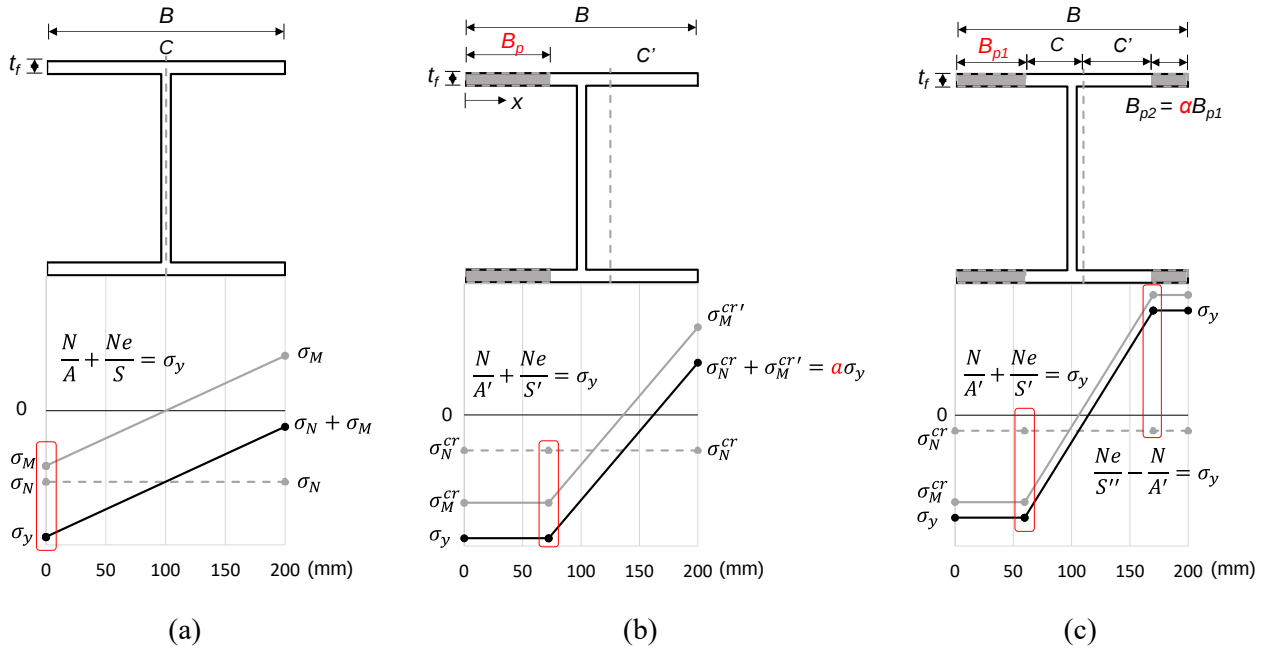


Figure 4-13 Stress distribution when (a) initial yielding occurs, (b) buckling occurs under the single-side flange yielding condition, (c) buckling occurs under the double-side flanges yielding condition

Buckling load (N) can be calculated by integrating stresses over the cross-sectional area using Eq. (4-24), which needs to satisfy two conditions: 1) the buckling load must be greater than or equal to the critical buckling load (N'_{cr}) using an equivalent moment of inertia of the cross-section of the elastic region and 2) the bending moment calculated by area integral equals the buckling load multiplied by the eccentricity (e) of curved members, as shown in Eqs. (4-22) and (4-23), respectively.

$$N \geq N'_{cr} = \frac{\pi^2 EI'_y}{(kL_m)^2} \quad (4-22)$$

$$M_{cr} = \int_A x \cdot \sigma_M(x) dA = eN \quad (4-23)$$

$$N = \int_{B_p}^B \sigma(x) dA + \int_0^{B_p} \sigma_y dA \quad (4-24)$$

$$\sigma(x) = \frac{(1+a)\sigma_y}{B-B_p} x - \frac{(1+a)\sigma_y B_p}{B-B_p} - \sigma_y \quad (4-25)$$

where E and σ_y are the elastic modulus and yield stress of the material; I'_y is the equivalent moment of inertia of the cross-section in the elastic region; k is the effective length factor of the member (k equals 1.0 for pinned constraint); B and L are the widths of the cross-section and length of the curved member; and $\sigma(x)$ is the stresses over the cross-sectional area of curved members, as shown in Eq. (4-25).

Under the condition of yielding both flanges, the buckling load, N , can be calculated using the same method, while the unknowns are the width of the flange on both sides, B_{p1} and B_{p2} , that reach the fully plastic state, as shown in Figure 4-13(c). Buckling load can be calculated by integrating stresses over the cross-sectional area using Eq. (4-27) and needs to satisfy the two conditional equations Eqs. (4-22) and (4-23) mentioned above and an additional conditional equation Eq. (4-26), which represents the equilibrium of compressive and tensile stress on both sides of the flange of curved members.

$$\frac{N}{A'} + \frac{Ne}{S'} = \frac{N}{A'} - \frac{Ne}{S''} \quad (4-26)$$

$$N = \int_{B_p}^{B-B_{p2}} \sigma(x) dA + \int_0^{B_{p1}} \sigma_y dA + \int_{B-B_{p2}}^{B_{p2}} \sigma_y dA \quad (4-27)$$

$$\sigma(x) = \frac{2\sigma_y}{B-B_{p1}-B_{p2}} x - \frac{2\sigma_y B_{p1}}{B-B_{p1}-B_{p2}} - \sigma_y \quad (4-28)$$

where $\sigma(x)$ is the sum of the stresses over the cross-sectional area of curved members, as shown in Eq. (4-28). S' and S'' are the section modulus of the cross-section of an elastic region, which can be calculated by the moment of inertia divided by different distances away from the neutral axis, and A and A' are the cross-sectional area and the cross-sectional area of the elastic region, respectively.

Figures 4-14 to 4-15 compare the calculated and analytical results from inelastic buckling analysis using Eqs. (4-22) and (4-25), and ABAQUS with various lengths (L_m) and the eccentricity (e) of curved members. In two cases where single-sided yielding of the flange occurs, as shown in Figure 4-14(a) and (b), the length (L_m) of curved members is 2,000 mm and 4,000 mm, respectively, with the same ratio of e/L_m equal to 0.01. Figure 4-14(a) and (b) also illustrate the stress distributions visualized from FEM simulations at the middle of the curved members, where the maximum stress forms when buckling occurs, which aligns closely with the results calculated by the proposed estimation method. In the other two cases where double-sided yielding of the flange occurs, both the length (L_m) of curved members is 2,000 mm, and the ratio of e/L_m is equal to 0.05 and 0.1,

respectively. Figure 4-15 shows the stress distributions visualized from FEM simulations in the middle of the curved members, where the maximum stress forms when buckling occurs, and it matches well with the results calculated by the proposed estimation method. Additionally, analytical results indicate that the strain-hardening effect derived from the material properties in ABAQUS is negligible.

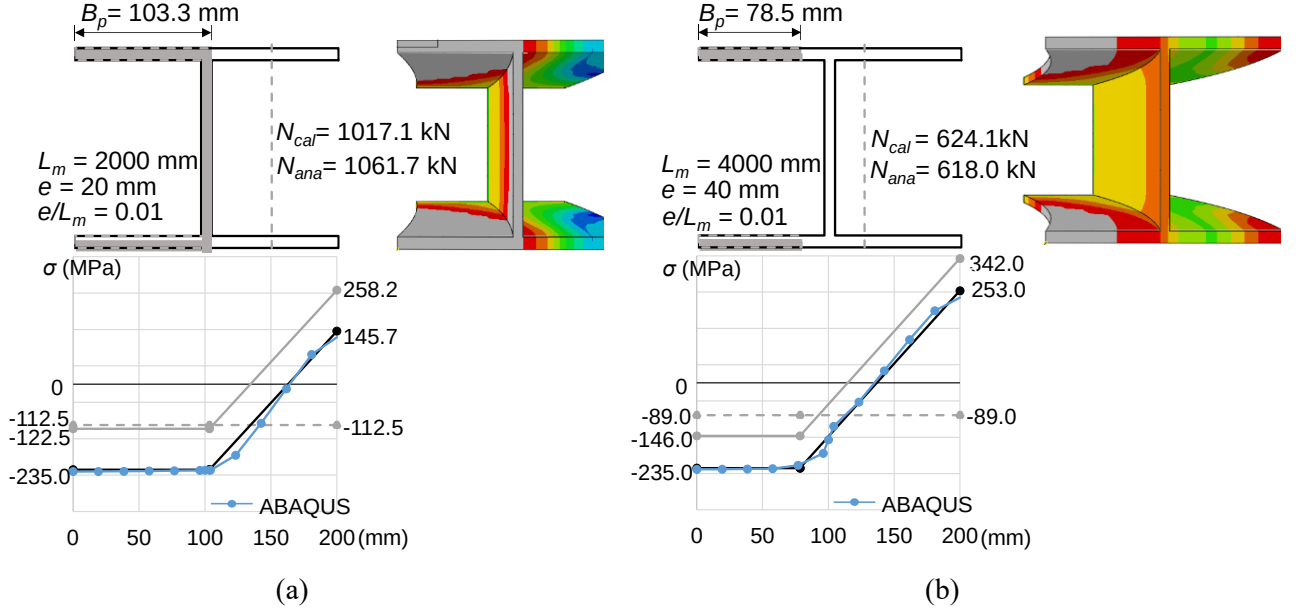


Figure 4-14 Validation with FEM models under the single-side flange yielding condition:(a) length L_m of 2000 mm; (b) length L_m of 4000 mm.

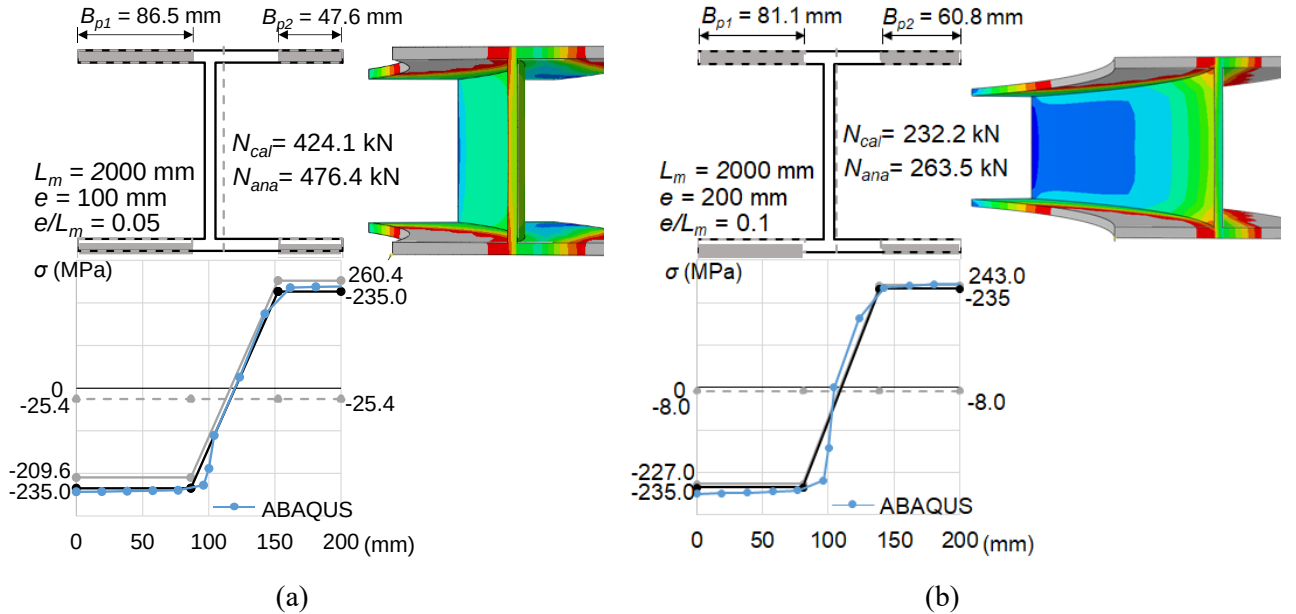


Figure 4-15 Validation with FEM models under the double-side flanges yielding condition: (a) e/L_m ratio of 0.05; (b) e/L_m ratio of 0.1.

Table 4-2 A list of the major design parameters of cases in the parametric study

Cases	L_m	e (mm)	e/L_m	H/L
1	2000	20	0.01	25.0
2		40	0.02	12.5
3		60	0.03	8.3
4		80	0.04	6.2
5		100	0.05	5.0
6		150	0.075	3.3
7		200	0.1	2.4
8		250	0.125	1.9
9		400	0.2	1.1
10		30	0.01	25.0
11	3000	150	0.05	5.0
12		300	0.1	2.4
13		600	0.2	1.1
14		40	0.01	25.0
15	4000	200	0.05	5.0
16		250	0.0625	3.9
17		400	0.1	2.4
18		800	0.2	1.1
19	6000	60	0.01	25.0
20		250	0.0417	6.0
21		300	0.05	5.0
22		600	0.1	2.4
23		1200	0.2	1.1
24	8000	80	0.01	25.0
25		250	0.0312	8.0
26		400	0.05	5.0
27		800	0.1	2.4
28		1600	0.2	1.1

A parametric analysis was conducted to verify which conditional equations to adopt in which case and applicable ranges to estimate the buckling strength of curved members. This parametric analysis considers two parameters of the curved member, including the length (L_m), eccentricity (e), and the corresponding ratio of e/L . A total of 28 cases with a length varying from 2,000 mm to 8,000 mm, and the ratio of e/L_m varies from 0.01 to 0.2, representing different slenderness ratios and curvatures, as listed in Table 4-2. The parametric analysis included more cases with $L_m = 2000$ mm to examine the transition range between single flange yielding and both flange yielding conditions. Based on the FEM simulations, the condition of single flange yielding holds if the ratio of e/L_m is less than or equal to 0.02, and the condition of both flange yielding holds for the rest.

Figure 4-16(a) presents the overall results of buckling strength for curved members with pinned constraints in the parametric analysis. The analytical results indicate that the buckling strength exceeds the yield strength when the curved member has a smaller slenderness ratio. However, as the slenderness ratio increases, the buckling strength gradually converges toward the yield strength. Consequently, the range of slenderness ratio from 20 to the intersection of Euler's curve and Johnson's curve (slenderness ratio of 120) is applicable for using Eq. (4-22) to Eq. (4-28) to estimate the buckling strength of curved members. In contrast, the slenderness ratios greater than the intersection are applicable for using Eq. (4-21), where a reduction coefficient, ζ , is used to estimate the yield strength. The parametric analysis was conducted following the same FEA method mentioned previously for the curved members with fixed constraint conditions. A similar trend of buckling strength was observed for the curved members with pinned and fixed constraint conditions. Thus, the results can be estimated by Eqs. (4-22) to (4-28) using the effective length factor (k) of 0.5. Figure 4-16(b) presents the accuracy of the calculated results of the buckling strength compared to the analytical results. It is confirmed that Eqs. (4-22) to (4-28) accurately estimate the buckling strength with errors within $\pm 15\%$.

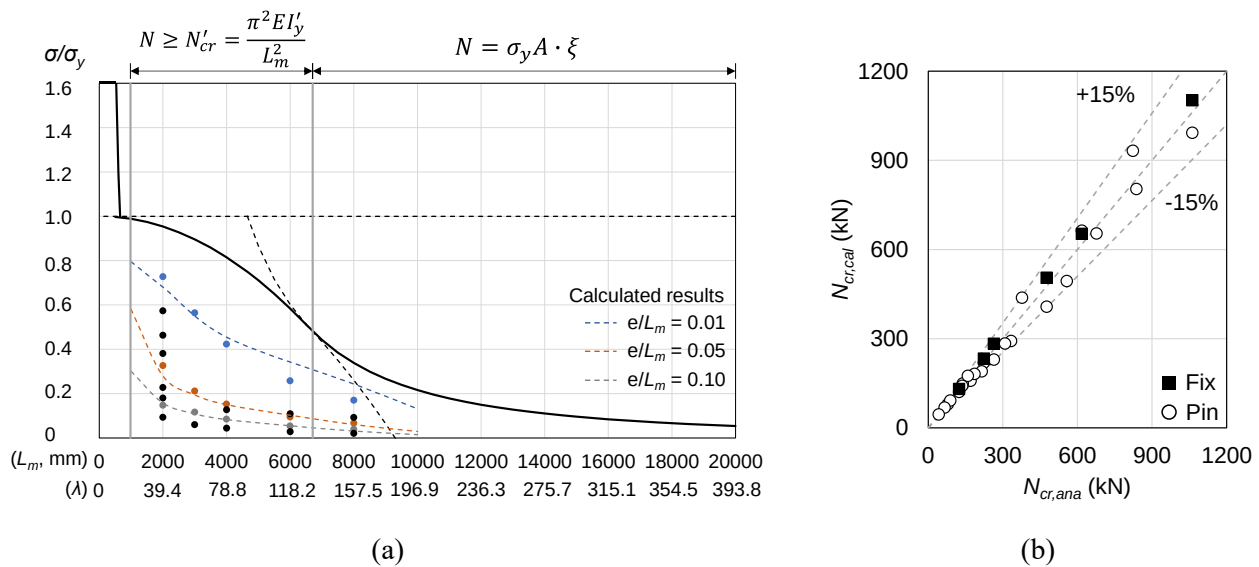


Figure 4-16(a) Relationship and comparisons of calculated and analytical results of the buckling strengths;
(b) comparisons of calculated and analytical results of the buckling strengths.

4.2.3 Section compactness effect

Section compactness is a critical factor in the design of structural members, particularly for curved members that are often subjected to complex stress distributions. While section compactness in straight members primarily addresses the prevention of local buckling and the ability to develop full plastic strength, curved members introduce additional complexities due to geometric effects and non-uniform stresses along the member curvature. The stresses of curved members are typically larger on the inner radius due to compressive forces and lower on the outer radius, where tensile forces dominate. This imbalance can lead to premature local buckling on the compression side, especially in noncompact or slender sections.

A parametric analysis with the same simulation approach as in Section 4.2.2 was conducted using the FEA

software ABAQUS to examine the influence of section compactness on the behavior of curved members. Nine cases had the width-to-thickness ratios (b/t_f) and eccentricity-to-length ratio (e/L_m) listed in Table 4-3. The section slenderness limits λ_p and λ_r are 11.2 and 29.5, respectively, for SS400 steel grade per AISC seismic provisions (AISC 341-22) [4-17]. The width-to-thickness ratio (b/t_f) varies from 6.25 to 34.09, representing the compact, noncompact, and slender sections, and the eccentricity-to-length ratio (e/L_m) varies from 0.01 to 0.1. The sections of curved members were selected based on the typical use of structural steel in practical applications, with a length of 3,000 mm and the boundary condition of pin-roller constraints. Steel material characterized by a trilinear stress-strain curve with a tangent modulus of 0.01 times the elastic modulus E , and eight-node solid elements with reduced integration, which were used for meshing the members, were employed in the FEA, as shown in Figure 4-10(a).

Table 4-3 A list of section properties of cases in the parametric study

Cases	H (mm)	B (mm)	t_w (mm)	t_f (mm)	b/t_f	$H-2t_f/t_w$	Section class	e/L_m
1~3	200	200	9	16	6.25	40.89	Compact	0.01, 0.05, 0.1
4~6	200	300	9	10	15.00	42.18	Noncompact	0.01, 0.05, 0.1
7~9	200	450	9	6	37.50	42.98	Slender	0.01, 0.05, 0.1

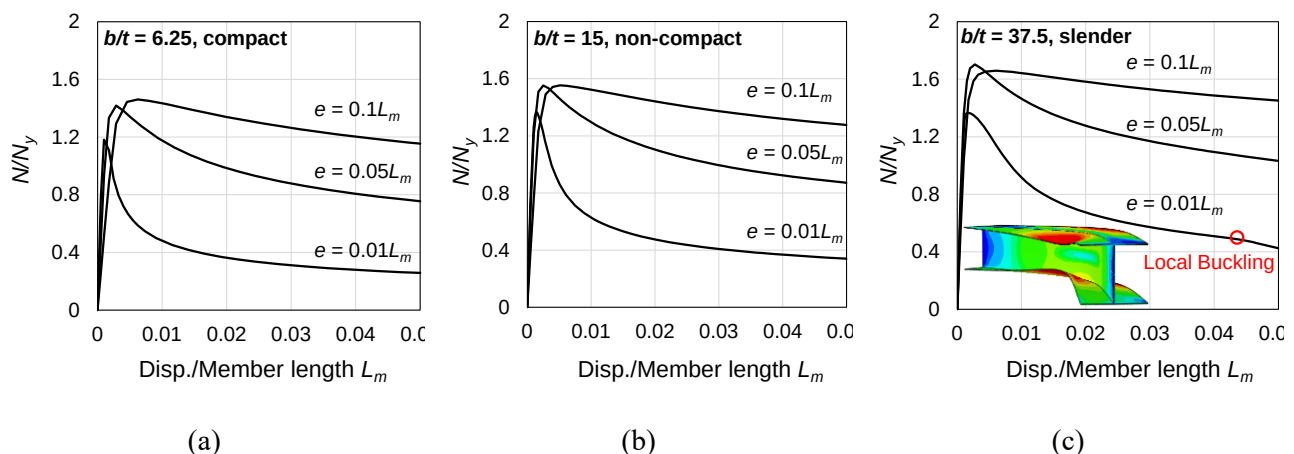


Figure 4-17 Normalized load-displacement relationships of curved members with varying section compactness (a) compact section; (b) noncompact section; (c) slender section.

Figure 4-17 illustrates the normalized load-displacement relationship for curved members with varying section compactness under different initial eccentricities ($e = 0.01L_m$, $0.05L_m$, $0.1L_m$). The behavior of curved members with compact and noncompact sections is dominated by inelastic global buckling. The axial strength gradually decreases after buckling occurs for the cases that $e = 0.05L_m$ and $0.1L_m$ due to the effect of initial eccentricity. While the cases representing relatively straight members ($e = 0.01L_m$) have a sudden decrease in strength after buckling occurs, as shown in Figures 4-17(a) and (b). For the cases with slender sections, local buckling is observed (indicated in red) in the analysis in the case that $e = 0.01L_m$, as shown in Figure 4-17(c). The analytical results demonstrate that the compact and noncompact sections of curved members prevent local

buckling. In contrast, slender sections require stricter design considerations to address local buckling, and it is recommended that the compact section be selected when adopting the curved members. Notably, as b/t increases, the cross-section's minor-axis second moment of area I_y grows appreciably, while the section modulus S increases only marginally. Consequently, the normalized axial load N/N_y rises slightly.

4.3 Effect of boundary constraint conditions

Curved members addressed in this study are placed vertically in the structure and serve as braces or retrofit members, and their behavior is closely tied to the boundary constraint conditions. Variations in the degree of fixity at the connections of the curved members can alter the force distribution and stiffness of the system. The behavior of the curved member with pinned- and fixed-end boundary conditions has been examined in Section 4-2, while the effect of the partially fixed boundary condition is also important. This section identified an effective boundary constraint condition and suggested a recommended factor that achieves the desired behavior and controls the yielding mechanism of curved members.

When adopting curved members with pinned-end boundary conditions, the maximum moment occurs at the middle of the member, while when adopting curved members with fixed-end boundary conditions, the maximum moment occurs at the middle and ends of the member. To ensure that the plastic hinge occurs first in the middle of the curved member with the fixed-end boundary condition, the degree of constraint fixity requires verification. Consider the cases where a horizontal force Q is applied at the top of the curved member, as illustrated in Figure 4-2, with two types of boundary conditions: (a) both ends are fixed, and (b) the top end is pinned, and the bottom end is fixed. The bending moment of the two cases $M_{fix}(\theta)$ and $M_{pin}(\theta)$, along the member, can be calculated by Eqs. (4-29) and (4-30) using the virtual work principle.

$$M_{fix}(\theta) = Q[R(\sin \theta_L + \lambda \cos \theta_L) + \mu - R(\sin \theta + \lambda \cos \theta)] \quad (4-29)$$

$$M_{pin}(\theta) = -QR \left[\sin \theta + \frac{-3\theta_L + 4\cos \theta_L - \cos 2\theta_L}{6\theta_L - 8\sin \theta_L + \sin 2\theta_L} (1 - \cos \theta) \right] \quad (4-30)$$

where the coefficients are listed below.

$$\begin{aligned} \lambda &= -(\alpha_1 \theta_L - \alpha_3 \beta_1) / (\alpha_2 \theta_L - \alpha_3 \beta_2) \\ \mu &= -(\alpha_2 \beta_1 - \alpha_1 \beta_2) / (\alpha_2 \theta_L - \alpha_3 \beta_2) \\ \alpha_1 &= R \left(\frac{1}{4} + \frac{\theta_L}{2} \sin 2\theta_L - \cos \theta_L + \frac{3}{4} \cos 2\theta_L \right) \\ \alpha_2 &= R \left(\frac{\theta_L}{2} + \theta_L \cos^2 \theta - \frac{3}{4} \sin 2\theta_L \right) \\ \alpha_3 &= \theta_L \cos \theta_L - \sin \theta_L \\ \beta_1 &= R(\theta_L \sin \theta_L + \cos \theta_L - 1) \\ \beta_2 &= R(\theta_L \cos \theta_L - \sin \theta_L) \end{aligned}$$

Figure 4-18(a) and (b) show the moment distribution along the curved member with pinned-fixed and fixed-fixed boundary constraint conditions, respectively. The ratio of height (H) and width (L) of curved members varied from 1 to 8. In both figures, the horizontal axis represents the angle (θ), which expresses the specific location of the curved member. The vertical axis of the bending moment is a dimensionless parameter obtained by dividing the moment by the product of the horizontal force (Q) and the radius (R). For comparing the moments at the ends and the middle of the curved members, the calculated results for the fixed-fixed boundary condition indicate that the moment has a maximum at the top end of the curved member, where the load is applied, while the moments at the bottom end and middle are nearly the same. For the pinned-fixed boundary condition, the moment is zero at the top end, reaching the maximum moment in the middle of the curved member.

To ensure that the maximum moment occurs at the middle of the curved member, Eqs. (4-29) and (4-30) are combined and modified to determine the conditions under which the plastic hinge occurs at the middle of the curved member rather than at the ends, as shown in the following Eq. (4-31).

$$M = \gamma_m M_{fix} + (1 - \gamma_m) M_{pin} \quad (4-31)$$

where γ_m ($0 \leq \gamma \leq 1$) is the factor representing the different proportions of moments for fixed-fixed and pinned-fixed boundary constraint conditions. When $\gamma_m = 0$ (pinned top, fixed bottom), the moment is maximized at the middle of the curved member, while when $\gamma_m = 1$ (both ends fixed), the moment is maximized at the top end of the curved member. An upper limit of $\gamma_{m,c}$ was determined by calculations to be the state where the moment at the top end equals the moment at the middle of the curved member. Figure 4-16(c) shows an example of moment distributions for the curved member with the ratio H/L of 1.85 when $\gamma_m = 0$ (pinned top, fixed bottom), $\gamma_m = 1$ (both ends fixed), and $\gamma_m = \gamma_{m,c}$, where the moments at the top end, bottom end, and middle of curved member are identical. The calculated results indicate that the values of $\gamma_{m,c}$ with different H/L ratios are in the range of 0.63 to 0.65, and $\gamma_{m,c}$ was set to 0.6 to ensure the plastic hinge occurs at the middle of curved members.

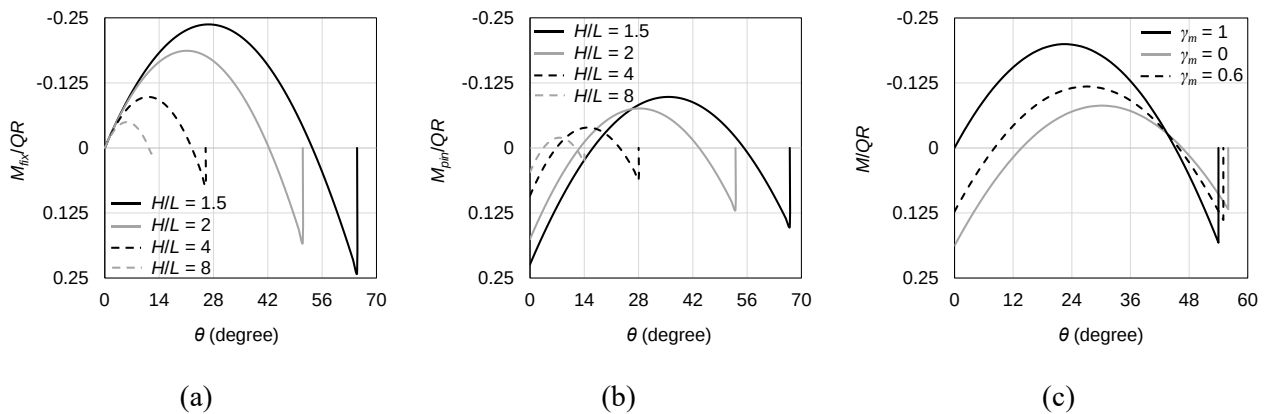


Figure 4-18 Moment distribution with (a) pinned-fixed, (b) fixed-fixed boundary constraint conditions, and (c) the effect of different γ_m factor.

4.4 Bidirectional behavior of curved member

This section investigates the bidirectional behavior of the proposed curved member retrofit technique to evaluate its effectiveness under realistic seismic loading conditions where lateral forces may act in multiple directions. While previous sections focused primarily on unidirectional loading, structures in actual earthquakes are subjected to complex multi-directional input, making it important to confirm that the retrofit system performs consistently regardless of loading direction.

To further verify the bidirectional performance of the curved member retrofit system, numerical models were developed using OpenSees, following the same modeling method described in Chapter 4.2.1, as shown in Figure 4-19. In the analysis, monotonic lateral displacements were applied separately in the in-plane and out-of-plane directions to capture the behaviors of the curved member.

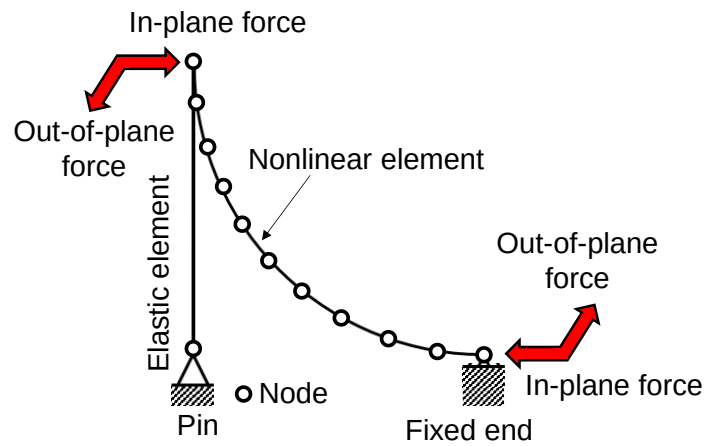


Figure 4-19 Numerical model of curved member with bidirectional loading

A parametric analysis was conducted on various height-to-width (H/L) ratios of curved members under two directional loadings. Figure 4-19 shows the variation of in-plane and out-of-plane lateral strength with respect to the H/L ratio. The in-plane yield strength increased gradually as the H/L ratio ranged from 1.0 to 2.5, attributed to the reduction in eccentricity. When the H/L exceeded 3.0, the strength decreased due to forming a double-curvature-like moment distribution along the curved members. Conversely, yield strength in the out-of-plane direction increases as the H/L ratio increases. The reason was that the effect of combined flexural and torsional stress occurred when the curved member had a large eccentricity.

The goal of the curved member retrofit technique in bidirectional loading was to ensure that the curved members provide comparable lateral strength in both the in-plane and out-of-plane directions. To achieve this behavior, the curved members were oriented such that the weak axis of the section (I_y) lay in the in-plane direction, while the strong axis (I_x) lay in the out-of-plane direction. However, the yield strength was consistent when H/L was 7, but exceeded the applicable range of 1.0 to 3.0 suggested in Section 4.2.1. To reduce the imbalance between in-plane and out-of-plane strengths, the section geometry can be modified by increasing the moment of inertia of the section about the strong axis (I_x) while keeping the weak axis (I_y) constant.

Figure 4-20 shows an example of adjusting the I_x/I_y ratio (red dots) of a H-section (H200×200×8×12) to

increase the out-of-plane strength. Starting from a reference section of H200×200×8×12 with I_x/I_y of 2.9, the section is modified by increasing the I_x while keeping the I_y constant. This adjustment resulted in a section of H400×200×8×12, where the I_x/I_y ratio increases to 13.5 and led to a significant increase in out-of-plane strength and a more balanced bidirectional behavior.

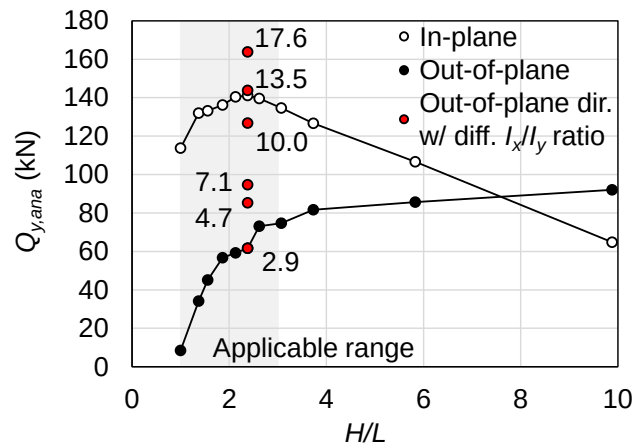


Figure 4-20 Relationship between the difference in H/L ratios and the yield strengths under bidirectional loading of the H-section member (H200×200×8×12).

4.5 Summary

Chapter 4 investigated the plastic behavior of curved members through parametric studies covering a wide variation of dimensions and boundary conditions, followed by the development of estimation equations for plastic strengths, initial elastic stiffness, and the influence of member slenderness and compactness. Two phases of parametric studies regressively quantified the effects of fixed boundary conditions and member slenderness on the behavior of curved members. Finally, this chapter evaluated the impact of boundary constraint conditions on the behavior of curved members and identified a boundary condition that effectively controls the yielding mechanism and ensures the desired plastic behavior. Based on the presented work, several remarks can be drawn as follows:

- (1) A set of equations for estimating the initial elastic stiffness and yield strength of curved members under pinned constraint conditions was developed using the virtual work principle and elastic beam-column theory, and the calculated results obtained by the two methods are consistent. The developed equations were validated through numerical modeling and demonstrated high accuracy when compared to analytical results, ensuring their applicability in practical design scenarios.
- (2) The effect of slenderness on the behavior of curved members was found that curved members with different slenderness ratios exhibit inelastic buckling behavior after yielding, and the buckling strength exceeds the yield strength when the curved member has a smaller slenderness ratio. Additionally, as the slenderness ratio increases, the buckling strength gradually converges toward the yield strength. The estimation equations were established according to a parametric study using the finite element models which the slenderness ratio range of 20 to 120 applies Eqs. (4-22) to (4-28) for estimating buckling strength,

while ratios above 120 use Eq. (4-21) with a reduction coefficient ζ for yield strength estimation.

- (3) The effect of section compactness on the behavior of curved members was verified, and compact and noncompact sections are dominated by inelastic global buckling due to initial eccentricity. For curved members with slender sections, local buckling is observed in the analysis for those with relatively small initial eccentricity. The analytical results demonstrate that the compact and noncompact sections of curved members prevent local buckling, and it is recommended that the compact section be selected when adopting the curved members.
- (4) An effective boundary constraint condition was proposed to achieve the desired behavior and control the yielding mechanism, ensuring the plastic hinge occurs first in the middle of the curved member. This was achieved by adopting a recommended factor ($\gamma_{m,c}$) of 0.6, representing the degree of fixity at the connections.
- (5) The bidirectional behavior of curved member retrofits was investigated using numerical simulations. By adjusting the ratio of I_x/I_y and locating the weak axis of the curved member section in the in-plane direction, out-of-plane strength improved, achieving more balanced performance. This confirms the importance of section shape and orientation in optimizing retrofit behavior under multi-directional seismic loads.

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CHAPTER 5

DEVELOPMENT OF COMPONENT-LEVEL RETROFIT USING CURVED MEMBERS

5.1 Introduction

5.1.1 Background

The elastic-plastic behavior of curved members, including their strength and stiffness, was clarified through analytical investigations in Chapter 4. A set of estimation equations and design recommendations was proposed for their practical use in seismic applications. These findings provided a foundation for applying curved members in seismic retrofit systems with minimal disturbance to the structures.

A significant number of low-rise industrial buildings, particularly factories employing long-span steel frames, were constructed across Japan from the late 1960s through the early 1990s. These structures, predominantly composed of steel, were designed based on outdated seismic codes and have now exceeded the legally defined service life of 34 years [5-1]. As such, many of these buildings face multiple challenges, including material deterioration, obsolete structural specifications, and inadequate seismic performance under current standards. Extending their service life and improving their seismic performance through appropriate retrofit strategies has emerged as an urgent priority for both disaster risk reduction and sustainable infrastructure management [5-2, 5-3, 5-4]. Retrofitting these aging structures is especially critical for mitigating potential economic losses and ensuring the continuity of industrial operation following major seismic events [5-5, 5-6].

Building on this foundation, Chapter 5 focuses on developing a novel retrofit method for steel column bases that leverages the advantages of curved members to enhance strength and stiffness, improve ductility, and address the limitations of conventional column retrofit methods. The design concept and basic mechanics of the curved member retrofit technique were first analytically evaluated. Four curved member specimens with varied fabrication methods and boundary conditions were tested under cyclic loadings to validate their performance. Key characteristics, including overall behavior, strength backbone curve, and deformation pattern, were examined based on measured responses and test observations. The established estimation equations of strengths of the curved member retrofit technique at various story drift levels were validated against experimental results. Finally, the numerical model presented in the preceding Chapters was refined to accurately replicate the observed responses, forming the basis for structural system-level analyses in Chapter 7.

5.1.2 Organization

Section 5.2 introduces the design concept of the curved member retrofit system, including the retrofit mechanism, preliminary modeling, prototype design process, and evaluation of the basic structural behavior to verify design targets. Section 5.3 outlines the experimental test program designed to investigate the component-level response under cyclic loading, describing the setup, specimens, and loading protocols. Section 5.4 presents the test results, focusing on observed failure modes, overall behaviors, deformation patterns, and the concentration of strain or plastic deformation. Finally, Section 5.5 discusses the development and validation of numerical models using the experimental data, establishing a reliable simulation framework for future system-level analyses.

5.2 Design and Basic Mechanism

5.2.1 Proposed retrofit technique

The proposed curved member retrofit system consists of two channels curved by an induction-heating process and lean against an existing column. The channel section was selected for its ease of installation and its effectiveness in preventing potential out-of-plane buckling. As illustrated in Figure 5-1(a), the curved members are placed in direct metal contact with the column surfaces and connected to each other by high-strength bolts and steel plates. End plates, welded to the curved channels, are tied on the foundation with anchor rods and covered by a new concrete slab. The gap between the bottom ends of curved members and the concrete slab is filled with the debonding material to ensure the independent performance of curved members.

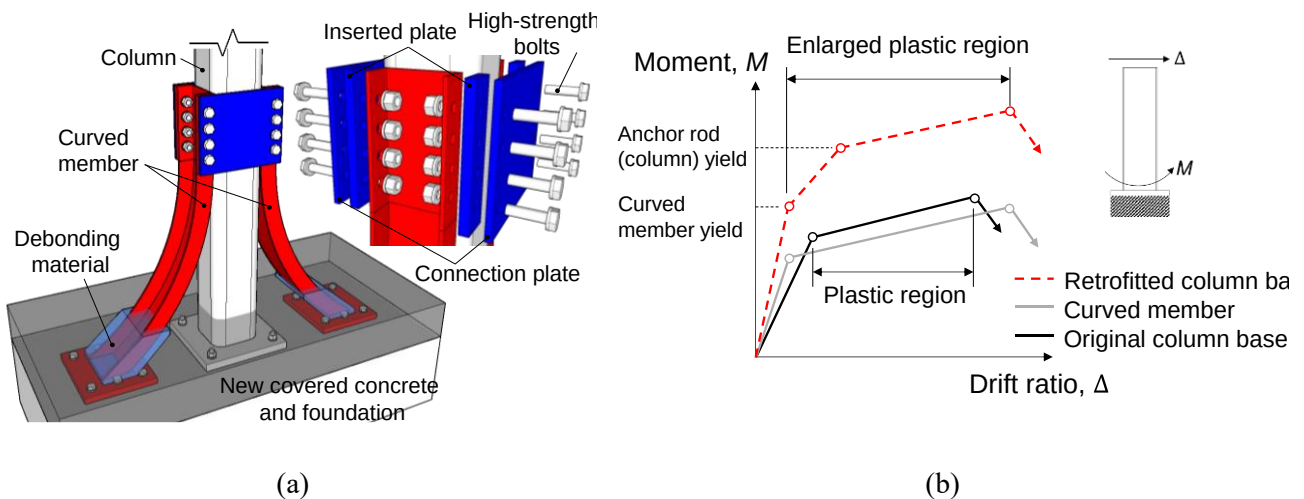


Figure 5-1 Proposed curved member retrofit system: (a) schematic and (b) strength backbone curve.

The design concept leverages the inherent characteristic of curved members to create a retrofit system with a controllable yielding region. Notably, the eccentricity provided by the curved member allows lateral stiffness to be loosely coupled with yield strength. This decoupling feature enables the strength-stiffness relationship of the retrofit system to be controlled through geometric parameters, thereby avoiding severe loss of strength due to buckling. The design targets for the retrofitted column base are as follows: (a) moderately enhanced strength,

stiffness, and improved ductility; (b) increased plastic rotation capacity to reduce the seismic demand for the column base; (c) minimal disturbance to the structure during and after adopting this retrofit method. Figure 5-1(b) compares the strength backbone curves of the original and retrofitted systems.

5.2.2 Preliminary modeling

To first examine the mechanism, performance, and design targets of the proposed curved member retrofit method, a series of pre-analyses were conducted. Numerical models of original column bases, pin-supported column bases with curved members to verify the contribution and behavior of curved members only, and retrofitted column bases representing the actual construction were developed using ABAQUS, a software application used for modeling and analysis of mechanical components and visualizing the results [5-7].

The exposed column base model in ABAQUS was composed of a base plate that was tied to a column. The FE models were built using eight-node solid elements with reduced integration (C3D8R). For simulating the complete joint penetration (CJP) welding at the curved member-to-end plate connection and column-to-base plate connection, the interactions of coupling constraints for the boundary conditions and tied contact formulation constraints were adopted as illustrated in Figure 5-2(a). Surface-to-surface contact was used to model contact interactions between the surfaces of the anchor rods and mortar. To define the tangential interactions of the surface-to-surface contacts, the penalty method was used. The friction coefficients of 0.2 and 0.7 were used for steel-to-steel and steel-to-concrete interaction, respectively. Hard contact was assumed for the normal behavior of the contact surface. ABAQUS models of column base were validated with the column base component test and presented good agreements with test results in terms of strength and stiffness [5-8], as shown in Figure 5-2(b).

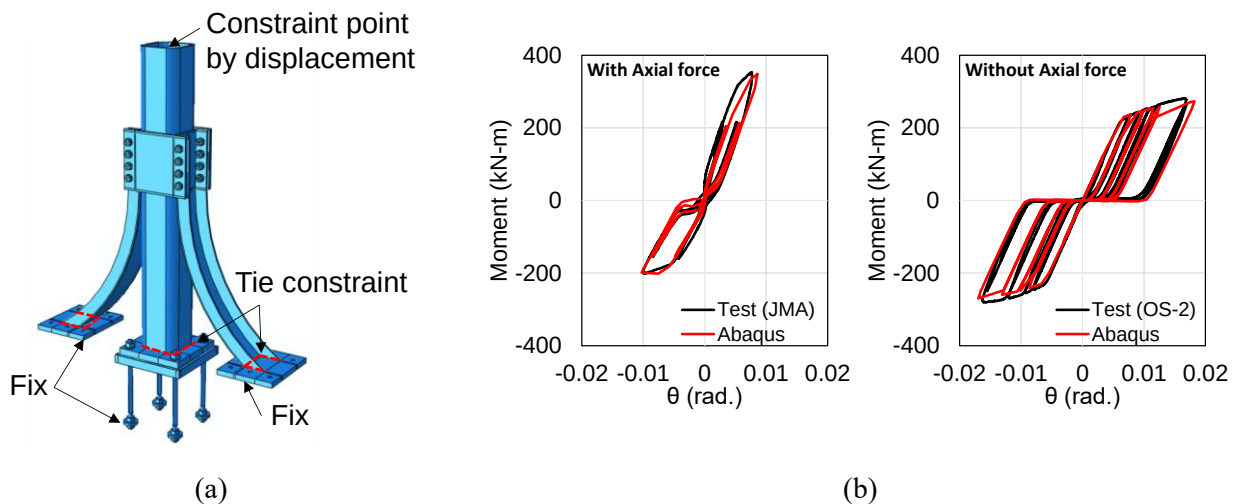


Figure 5-2 (a) Illustrations of adopted model in ABAQUS; (b) comparisons of measured and analytical hysteretic responses of column bases with axial forces, and without axial forces.

5.2.3 Prototype design process

An exposed column base, EJ250-4-30, was adopted in the pre-analysis. The adopted column base consisted

of an HSS250×250×16 column (BCR295) welded to a 410×410×40 base plate (SN490B), and the base plate was tied with four rods (30mm nominal diameter) on a grout layer.

For the adopted column base, a prototype system was designed by following the steps:

- 1) Design the curved member depth close to the adopted column depth;
- 2) Determine the height H and width L of the curved member within the range of ratio H/L as shown in Figure 4-1(a), and architectural requirements for minimizing the disturbance to structures;
- 3) Determine the material and section properties of σ_y , A , and S of the curved member by Eq. (4-11) and (4-12) to estimate the stiffness and strength of the retrofitted column base by the superposition principle;
- 4) Design connections, including connection plates and end plates of the curved member, according to the modified design methodology mentioned in Section 4.3.1;
- 5) Check the design targets of required stiffness and strength, and the moment reduction of 30% for the retrofitted column base;
- 6) Adjust the design parameters of the curved member in Steps 2 and 3 to meet the design targets. It should be noted that the numerical simulation is needed to check the behavior of curved members in step 5.

In this section, curved channel steel with a section of 200×80×7.5×11 was adopted for the prototype retrofit system, and examined in section 5.2.4.

5.2.4 Basic behavior and check of design targets

The design target for the retrofitted column base mentioned above includes having the behavior of moderate-enhanced strength, stiffness, improved ductility, and enlarged plastic rotation capacity to reduce the seismic demand for the existing column base. Moderately enhanced strength and stiffness can be achieved by controlling the geometric parameters of curved members, such as eccentricity and height. Improved ductility is achieved by the enlarged plastic region with plastic strength corresponding to the early yielding of the curved member and the delayed yielding of the anchor rods at the original column base.

Figure 5-3 shows the analytical results, including the behavior of the original column base, curved member retrofit system with a mechanical pin at the bottom of the column, and the retrofitted column base from ABAQUS. The analytical results of the retrofitted column base show that the first yielding started at the middle of the curved member at the drift angle of 0.01 radian followed by the yielding at the ends of the curved member at the drift angle of 0.016 radian, indicating the early yielding mechanism can be achieved and controlled at the middle of the curved member. The anchor rods started yielding at the drift angle of 0.014 radian for the original column base. In comparison, the yielding was delayed to the drift angle of 0.019 radian for the retrofitted column base by the curved members with reduced moment demand. The drift angle of anchor rods yielding at the retrofitted column base has been increased by 36% compared with the original column base. It indicates that the proposed retrofit system enables the enlargement of the plastic rotation capacity of the original column base and reduces its seismic demand.

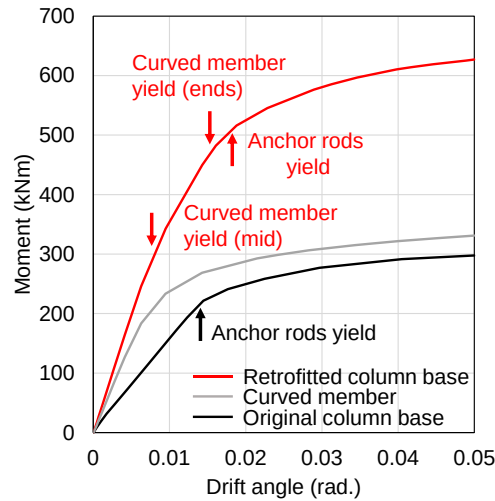


Figure 5-3 Comparison of analytical responses of curved members, original and retrofitted column base.

For comparison, an analysis focusing solely on the behavior of curved and straight members in the proposed retrofit system was conducted, revealing a significant difference. The design concept of the proposed retrofit method is to use the advantages of curved members. However, we need to verify the reasons for adopting the curved member instead of its straight counterpart. The behavior of the retrofit system adopting straight members had larger initial stiffness and yield strength, with the early occurrence of local buckling, which resulted in strength degradation. In contrast, the one adopting curved members had controllable initial stiffness and yielding strength with excellent ductility, as shown in Figure 5-4.

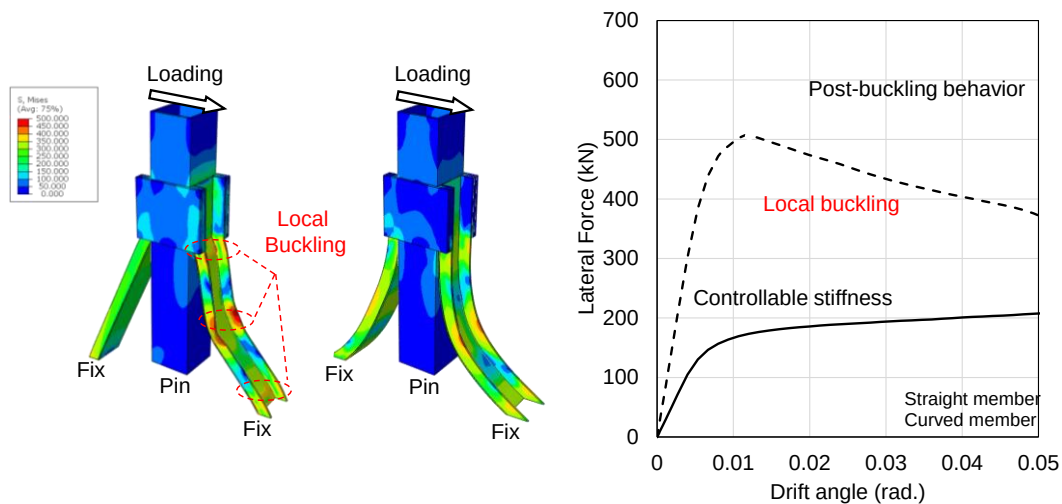


Figure 5-4 Comparison of analytical responses for the retrofit system using straight and curved members.

5.3 Test program

5.3.1 Overview of loading setup

An experimental program has been conducted to examine the proposed curved member retrofit system, focusing on the behavior of curved members and evaluating the influence of different boundary conditions and fabrication processes. Four cases of curved member retrofit systems have been tested by applying lateral cyclic loadings.

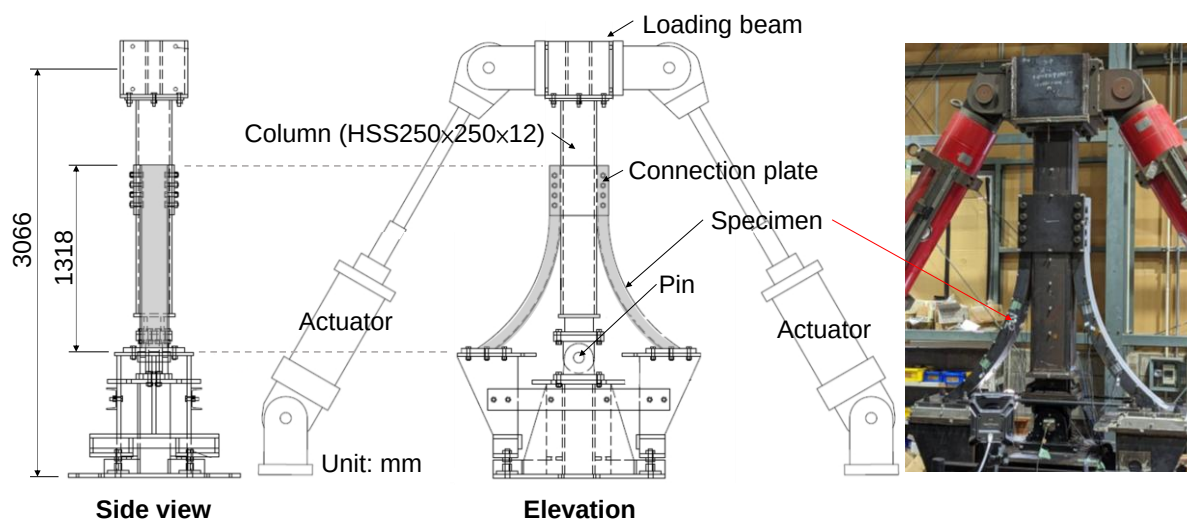


Figure 5-5 Elevations and a photo of adopted test setup in the experimental program.

Figure 5-5 shows the adopted test setup in the study with a photo of an installed specimen. Two 1,500kN hydraulic jacks applied the lateral cyclic loadings through a relatively stiff apparatus at the top. The apparatus was connected to the column (HSS 250×250×12) at the top, while the lower end was connected to a mechanical pin, representing a simple column base. Two identical specimens lean against the column and are connected by high-strength bolts and steel plates. The end plates welded to the specimens were tied with high-strength bolts on the fixture in the test. It should be noted that by adjusting the axial force of two actuators, no axial force was applied to the column in tests.

5.3.2 Specimens

All curved member specimens in the study had the same dimensions (Channel 200×80×7.5×11), including 936 mm and 350 mm in height of the curved segment and connection segment, respectively, and 1200 mm radius. At the intersection between the channel steel and the end plate, complete joint penetration (CJP) welds were used. The detailed dimensions for the specimen are shown in Figure 5-6.

A steel grade of JIS-SS400 was adopted for each specimen in the study. Table 5-1 presents the measured material properties of the adopted channel steel based on the tensile tests per ASTM E8/E8M-13a (2013) standard. Four types of coupon specimens, including standard specimens, heat-treated specimens, induction-heating bending specimens, and cold-form bending specimens, were tested to examine the effects of different fabrication processes. The coupon specimens were made from the flange and web of channel steels; therefore,

[illegible]

Fabrication process	Steel grade	Nominal plate thickness (mm)	Nominal elastic modulus E (GPa)	Measured yield strength F_y (MPa)	Measured tensile strength F_u (MPa)	Elongation (%)
Standard	SS400	7.5	200	312.0	429.0	23.4
Induction-heating w/o bending				323.0	437.2	23.4
Induction-heating bending				318.9	437.0	23.1
Cold-form bending				358.4	489.9	19.1

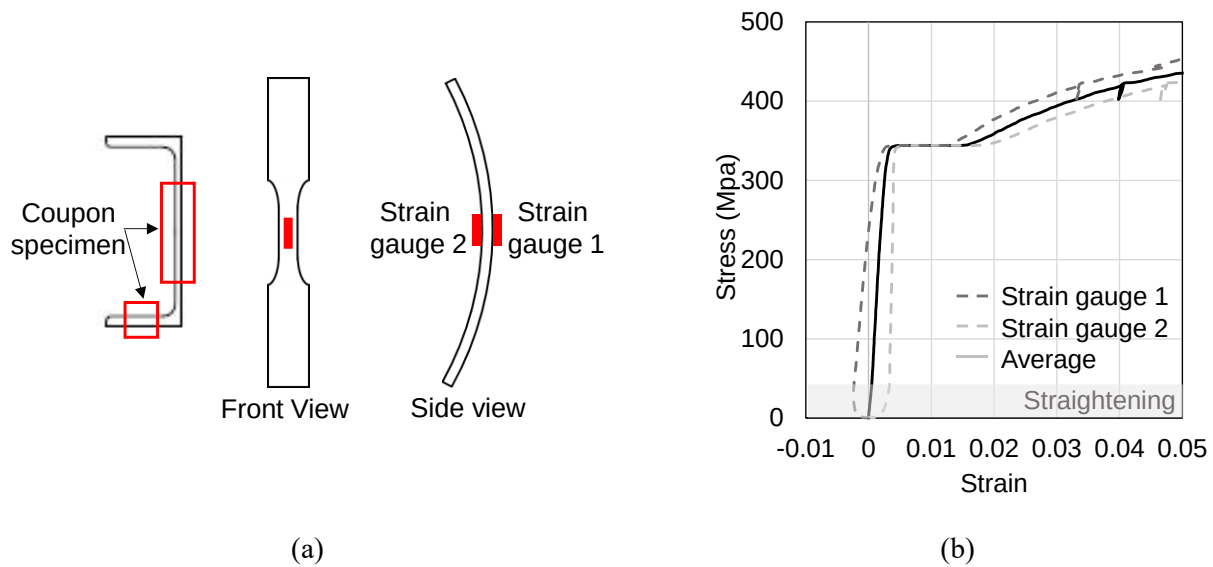


Figure 5-7 (a) locations of the coupon specimen from channel steel and (b) the stress-strain curve of the specimen with curvature.

The differences between specimens were the fabrication process and the boundary conditions. Fabrication processes for curved members can be divided into cold bending and hot bending. Cold bending is a method of processing mainly at room temperature to 720 degrees Celsius, with the feature of densifying the metallographic structure of steel, which is usually more economical than hot bending or induction heating bending. In addition, the metal is not subjected to excessive temperature by cold bending, while the residual stress is generated inside, and the yielding stress increases due to strain hardening. The effect of residual stress on structural performance has been proven not to affect the plastic strength of the member, while both the local and global buckling strength of compression members can be affected depending on external circumstances [5-9, 5-10, 5-11]. The cold-bending process requires a large force, and processing accuracy must account for material springback. The well-known cold-bending process is roll bending, in which members are bent using rollers, and was adopted for the cold-form bending specimens in the test. Induction-heating (IH) bending is an advanced hot bending process that utilizes an electric induction coil to heat a narrow band around the member circumference before it is bent by force, offering great precision and consistency. IH bending is suitable for a wide range of applications and overcomes the potential limitations of cold bending, such as distortion, wall thinning, and residual stress. The IH bending for the specimens in the test is processed by applying heat at 750 degrees Celsius and then air cooling to room temperature.

Four specimens for the curved member retrofit system have been tested. IHB presented a baseline specimen bent using induction heating, while IHB-R presented a case having the identical bending process as IHB, but with a 12 mm thickness rib at the middle of the curved member to prevent local buckling. CFB presented a counterpart specimen bent using the cold-form process to clarify the effect of different bending processes on the behavior compared to the IH bending specimen. IHB-M presented a specimen having the identical bending process as IHB, but with a 50 mm-thick mortar below the end-plate to simulate the actual construction. It should be noted that there was no pre-tension applied to the bolts connecting the end plate and the fixture to simulate

the anchor rods, and the shear force was resisted by the friction between the end plate and the mortar. The connecting rods, which connect two end plates, were used to transmit the force resisted by friction in cyclic loading.

5.3.3 Loading protocol

A cyclic loading protocol with increasing amplitudes from a lateral drift angle of 0.375 to 5.0% radians was adopted as shown in Figure 5-8(a). In addition, one extreme cycle of 10.0% drift angle was performed following the standard loading protocol to examine the failure mode of the specimens. Figure 5-8(b) shows the measuring instruments used in the test, including strain gauges to output the moment on the curved member specimen, and a load cell in the mechanical pin to measure the lateral and vertical force on the column. Two LVDT sensors were used to measure the relative displacement of the end plates and the fixture. The camera system was adopted to measure the overall and out-of-plane deformation of the specimen.

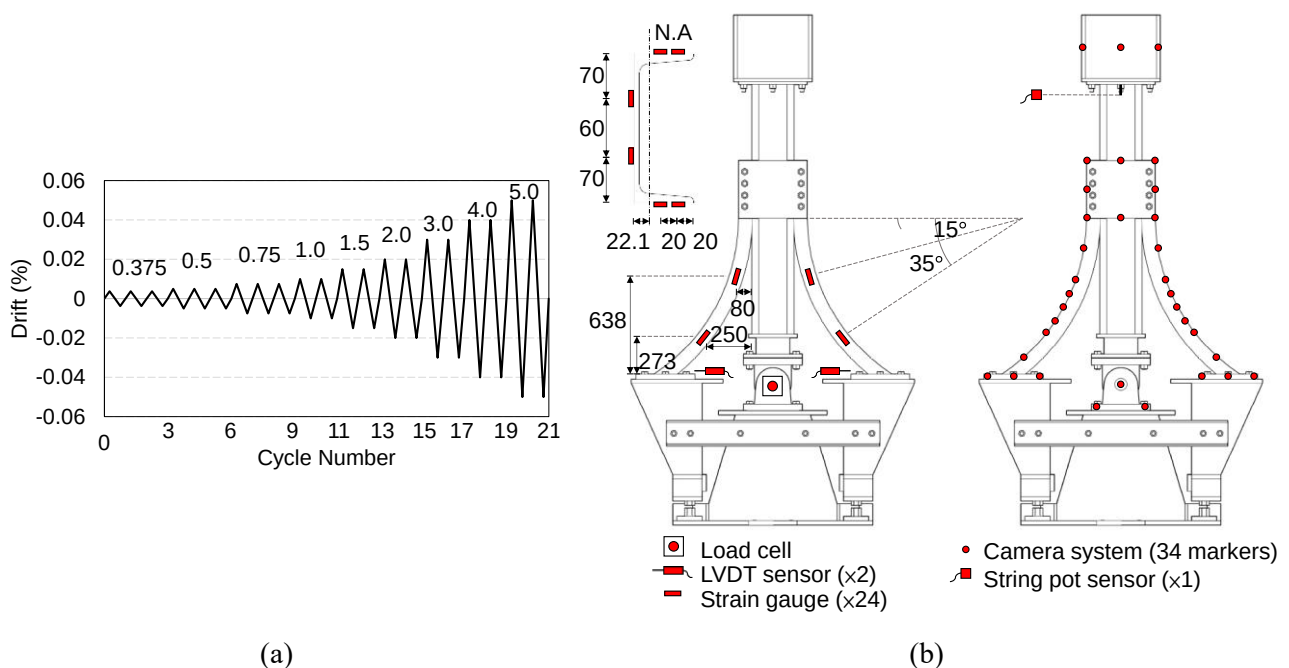


Figure 5-8 (a) Adopted cyclic loading protocol and (b) adopted measuring instruments

5.4 Measured responses and test observation

5.4.1 Overall behavior

All cases provided a stable cyclic behavior with minimal strength degradation by the end of the applied cyclic loading. Various lateral load capacities were shown among cases due to the difference in the bending processes and the boundary conditions. Upon the test observations, the curved member specimens started yielding on the flange at the middle of the curved member at the drift angle of 1.0% radians, and the end section of the curved member started yielding at the drift angle of 2.0% radians. Several damage phenomena were observed in the tests, including local buckling (LB), slipping at the bolt connection at the top of the specimen, and the end plate slipping, as shown in Figure 5-9. It should be noted that the fixture below the specimen was

observed to move during the test of IHB and IHB-R, resulting in an asymmetric initial elastic stiffness of the specimens.

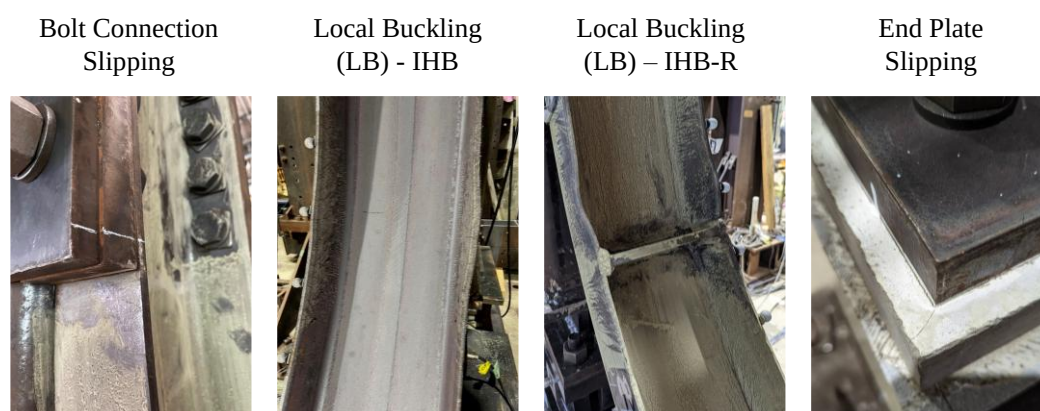


Figure 5-9 Photos of observed behaviors of the specimens.

The measured hysteretic responses of all cases are shown in Figure 5-10. Figure 5-10(a) shows the measured hysteretic responses of the specimen IHB. The IHB developed the yielding in the cycles of 1.0% drift angles. It should be noted that due to the movement of the fixture observed in the test, the local buckling of IHB actually occurred in the cycles of 4.0% drift angles instead of 5.0% drift angles as shown in Figure 5-10(a). Slipping of the bolted connection at the top of the specimen was observed in the cycles of 3.0% drift angles and resulted in some pinching behavior under large deformation. This is attributed to the stress concentration at the lower row of bolts. The measured responses of the specimen with ribs (IHB-R) at the middle of the curved member are shown in Figure 5-10(b). After eliminating the effect of fixture movement, the local buckling of IHB-R occurred earlier in the cycles of 3.0% drift angles compared to the IHB. Upon the test observations, it was shown that the specimen IHB had more distributed yielding on the flange around the middle of the curved member compared to IHB-R, where the yielding area was only concentrated on the flange above the ribs before the cycles of 4.0% drift angles. It is shown that IHB-R developed slightly higher strength than the IHB. The effect was considered related to the level of strain concentration, which increased the strain demand at the middle of the curve member specimen and thereby developed more strain hardening as well as the lateral strengths. It also implied that the rib would, however, advance the deformation concentration around the rib of the specimen and have limited improvement in preventing local buckling. The rib arrangement was not effective here.

For evaluating the effect of the cold-formed bending process on the behavior, specimen CFB, which had identical dimensions to the IHB was tested. Figure 5-10(c) shows the measured responses of CFB. The local buckling occurred in the cycles of 3.0% drift angle, and strength degradation was observed in the second cycle of 5.0% drift angle. CFB developed 1.15 times the lateral strength of IHB, corresponding to the coupon test, due to the strain-hardening effect caused by the cold-formed bending process, corresponding to the results of the material test. In addition, the initial distortion and poor precision of specimen CFB caused by the cold-form bending resulted in difficult installation in the test. It should be noted that the locations and shapes of local

buckling of CFB were observed corresponding to the initial distortion of the specimen.

The measured responses of the specimen with mortar (IHB-M) below the end plate are shown in Figure 5-10(d). It is shown that IHB-M had higher strength than the IHB due to the thickness of the mortar, and that the specimen is relatively close to the loading point. The end-plate slipping of IHB-M occurred in the cycles of 0.5% story drift but had no influence on the behavior, such as severe strength degradation and pinching. The local buckling occurred in the cycles of 4.0% drift angle as shown, no strength degradation was observed throughout the loading protocol, implying the feasibility in practical applications.

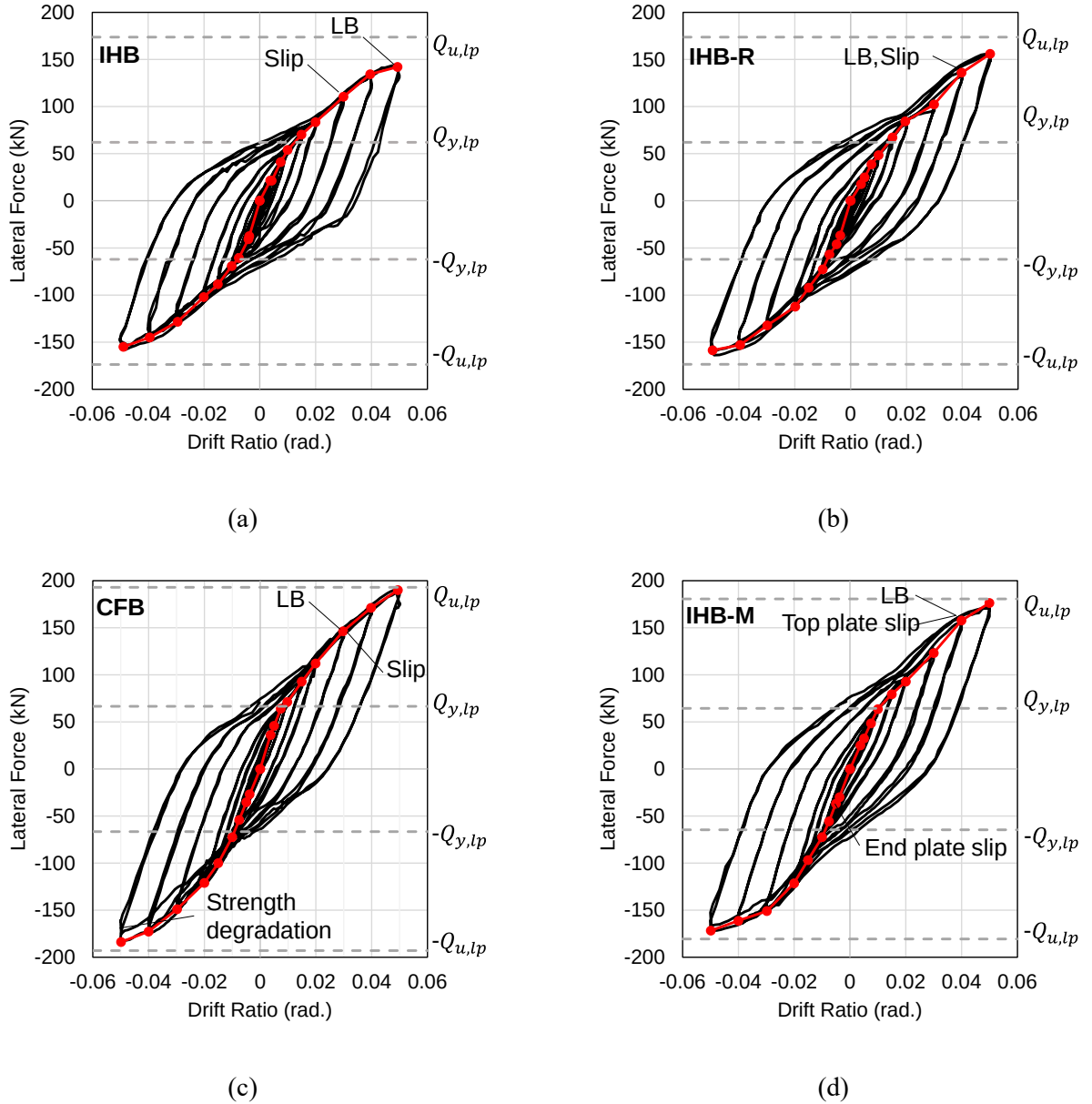


Figure 5-10 Measured hysteretic responses of specimens.

The lateral strengths at the loading point ($Q_{y,lp}$ and $Q_{u,lp}$) of the retrofit system was related to the dimensions of the existing column and the geometry of the curved member specimens. Eq. (4-15) established

in section 4.2.1 could be used to develop the estimation equations for the actual strength backbones at the loading point of the retrofit system, following Eqs. (5-1) and (5-2).

$$Q_{y,lp} = \left[2 N_{y,fix} \cos \left(\tan^{-1} \frac{H}{L} \right) H_p \right] / H_c \quad (5-1)$$

$$Q_{u,lp} = \left[(N_{y,fix} + N_{ct}) \cos \left(\tan^{-1} \frac{H}{L} \right) H_p \right] / H_c \quad (5-2)$$

where H and L are the height and the width of the curved member, respectively, H_p is the distance between the mechanical pin and the top connection of the specimen, H_c is the height of the existing column. N_{ct} is the strength of the curved member under tension that the flange partially reaches their tensile stress as shown in Eq. (5-3).

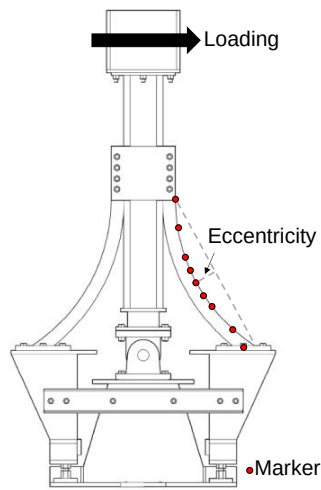
$$N_{ct} = \sigma_u A_f \quad (5-3)$$

where σ_u is the tensile stress; A_f is the area of the cross section on one side of the neutral axis that is subject to tension. The resulting value of those calculated plastic strengths at the loading point of the retrofit system are marked in Figure 5-10 to compare with the measured responses, and it is shown that the calculated results enable to represent the measured lateral plastic strengths of the test responses. Eqs. (5-1) and (5-2) were therefore verified to accurately estimate the yielding and the ultimate strength of the curved member retrofit system.

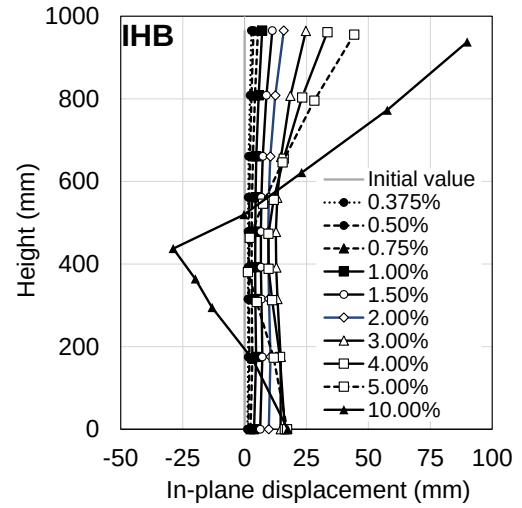
5.4.2 Deformed shape and deformation concentration

To evaluate the lateral displacement and the deformed shapes of the specimens, a motion-capture camera system was adopted in the instrumentation of the study. The markers of the camera system were attached to the flange of each specimen, as shown in Figure 5-11(a). The eccentricity of the specimen is one of the important parameters for estimating the strength capacity, and it will change with the deformation of the specimen. Upon the measured results, the in-plane displacements of IHB along the height of the right specimen at various drift angles are shown in Figure 5-11(b). The results show that the in-plane displacement had an apparent change after 4.0% drift angle. It is noted that all specimens have a similar deformed shape in the test.

The ratio of eccentricity (e) to initial eccentricity ($e_{initial}$) of IHB, IHB-R, CFB, and IHB-M at various drift angles can be calculated from the measured responses of markers, as shown in Figure 5-12(a)-(d), respectively. It should be noted that the markers on CFB fell off at the drift angle of 5.0% in the test. The results show that the eccentricities of all specimens will not exceed 1.1 times the initial value before 4.0% drift angle. It implied that the impact of eccentricity changes on strength capacity estimation was limited before large deformation. The initial eccentricities of both the right and left sides of IHB, IHB-R, and IHB-M were 128 mm, 131 mm, and 134 mm, respectively. However, the initial eccentricities of the right and left sides of CFB were 129mm and 137mm, respectively. These discrepancies were considered to be attributable to initial geometric imperfections and reduced fabrication precision of specimen CFB associated with the cold-form bending process.

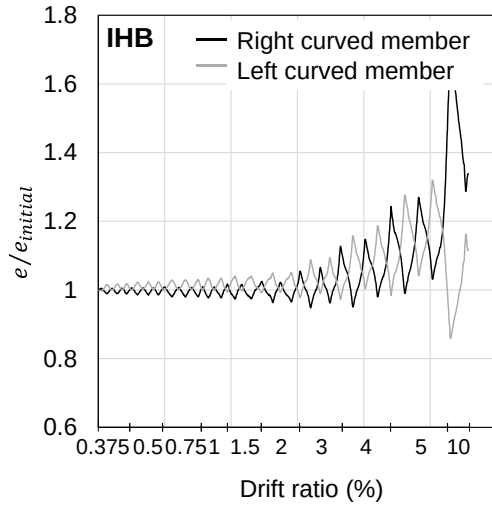


(a)

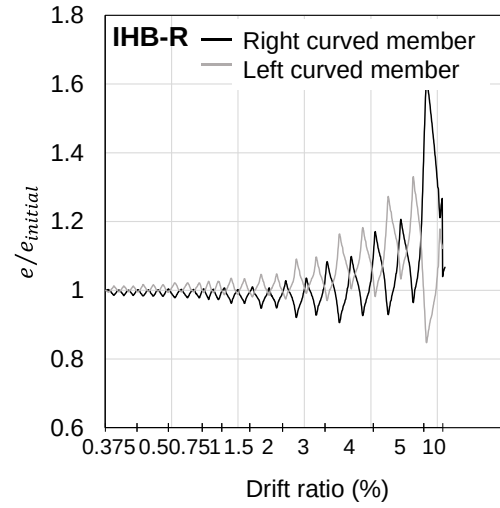


(b)

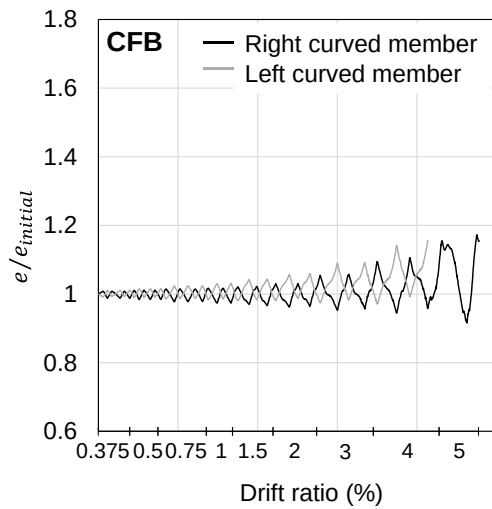
Figure 5-11 Marker placement of camera system, and deformed shape at various drift angles.



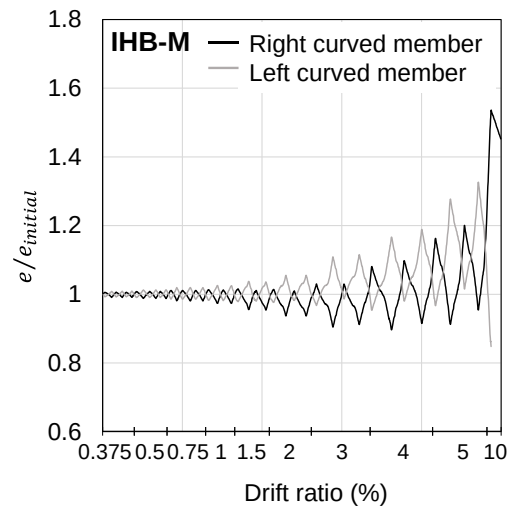
(a)



(b)



(c)



(d)

Figure 5-12 Measured eccentricity of each specimen at various drift angles.

The out-of-plane deformation of the specimens IHB, IHB-R, CFB, and IHB-M at various drift angles is shown in Figure 5-13(a) to (d), respectively. The shadow areas in Figure 5-13(b) present the location of the rib. In Figure 5-13(a) and (d), the results of IHB and IHB-M show that the flexural deformations along the member were controlled and limited around the middle of the curved member at the member height of 420 mm. Figure 5-13(b) shows the results of IHB-R where the rib at the middle of the curved member led to greater deformation concentration at the member height of 450 mm, which would result in early local buckling in the cycles of 4.0% drift angles.

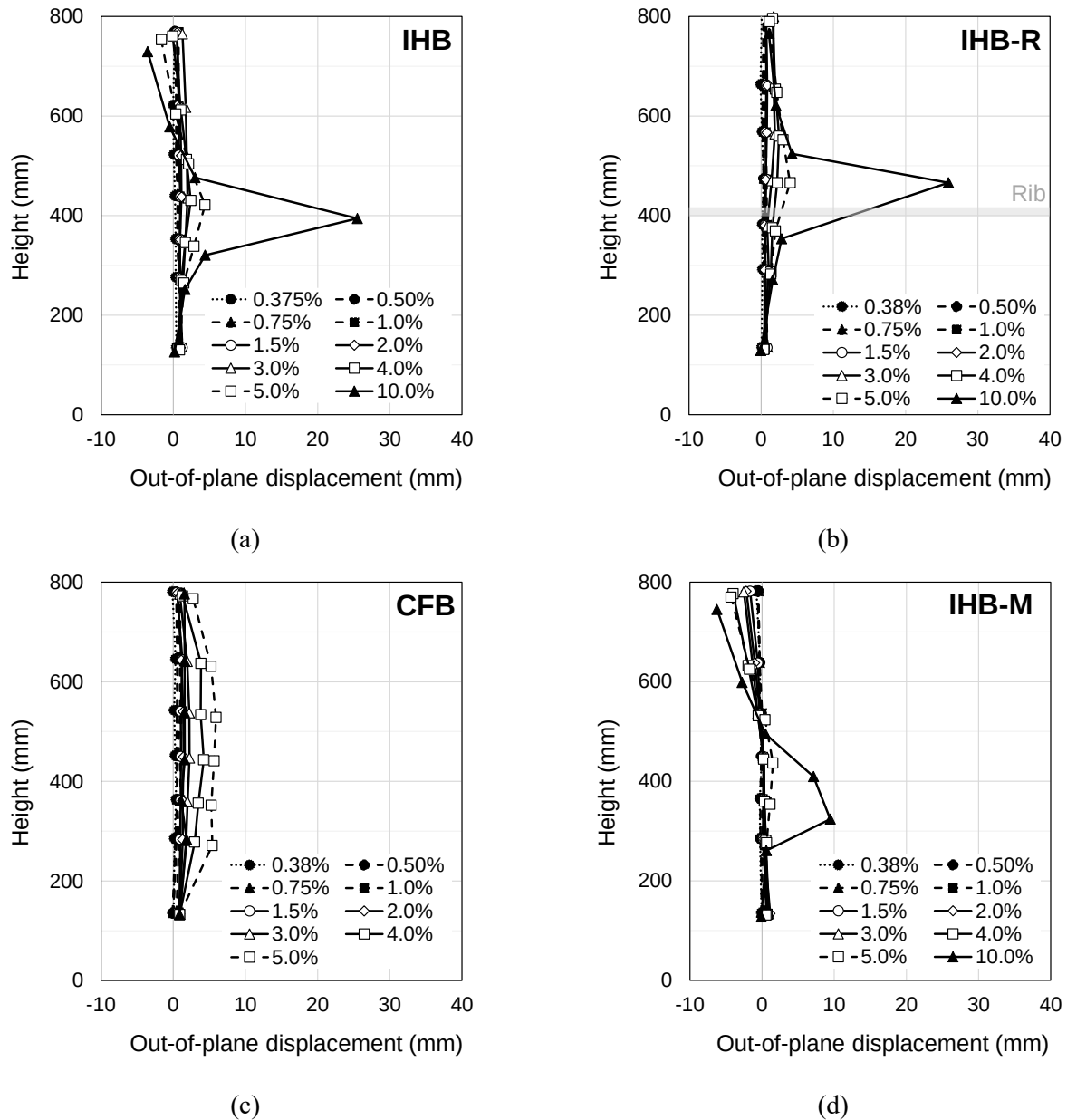


Figure 5-13 Out-of-plane displacement distributions at various drift angles.

Figure 5-14(a) shows a photograph of the deformed shape of the specimen IHB under 5.0% drift angle. It was shown that the yielding region was distributed uniformly at the middle of the curved member. The cold-

formed bending specimen CFB presented a relatively uniform out-of-plane displacement along the member height, as shown in Figure 5-13(c). However, the test observation has shown that the local buckling occurred early in the cycles of 3.0% drift angle compared to IHB and had severe deformation concentrations under 5.0% drift angle, as shown in Figure 5-14(b). In the CFB specimen, the out-of-plane deformation became notable, likely due to the initial distortion of the specimen caused by the cold-formed bending process.

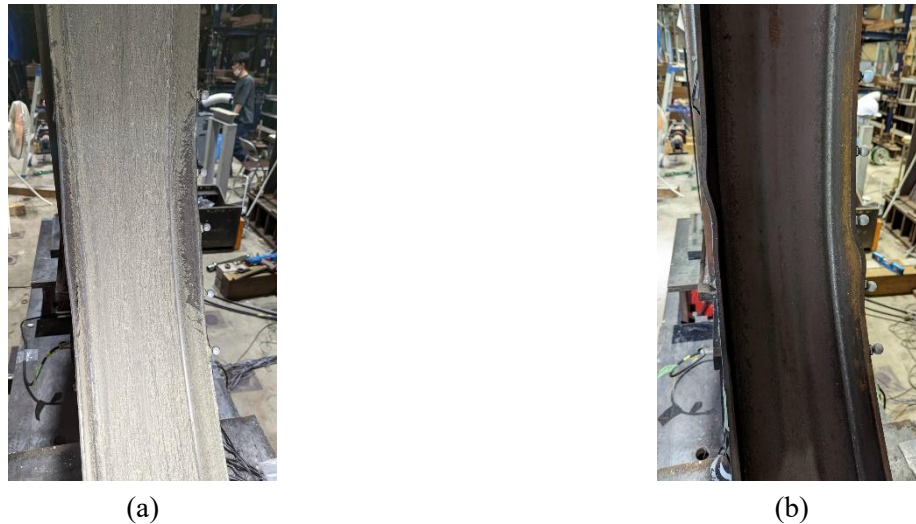


Figure 5-14 Photographs of the deformed shapes under 5.0% drift angle.

5.5 Numerical modeling and validation

To extend the test results to further analysis, a numerical model was developed in the OpenSees framework [5-12] to simulate the cyclic behavior of the curved member retrofit system. Figure 5-15 illustrates the proposed numerical model adopted for the curved member specimens and the fixtures in the test.

The simulation approach of the curved member mentioned in Section 4.2.1 was adopted in the model. The uniaxial material model of Steel02, using the yielding strength upon material test presented in Table 5-1, was applied for each fiber. In addition, two fiber-type force-based nonlinear beam-column elements with the section of connection plates were used to simulate the connection. The middle node of the column, which governed the horizontal displacement and in-plane rotational degrees of freedom, also constrained the adjacent nodes located at the sides of the connection. The nodes at the end of the specimen were mastered by the nodes at the top of the fixtures in the same degree of freedom. For the fixtures below the specimens, elastic beam-column elements were adopted with exact cross-sectional dimensions used in the test to simulate the contribution to the lateral stiffness by the fixture compared to the ideal situation with fixed ends. Fiber-type displacement-based elements with the hysteretic material were adopted for channel steel, which connects the fixtures to simulate the asymmetric behavior in the first few cycles observed in the test. The mechanical pin was simulated by zero-length elements with extremely low in-plane rotational stiffness and fixed in the rest of the degrees of freedom.

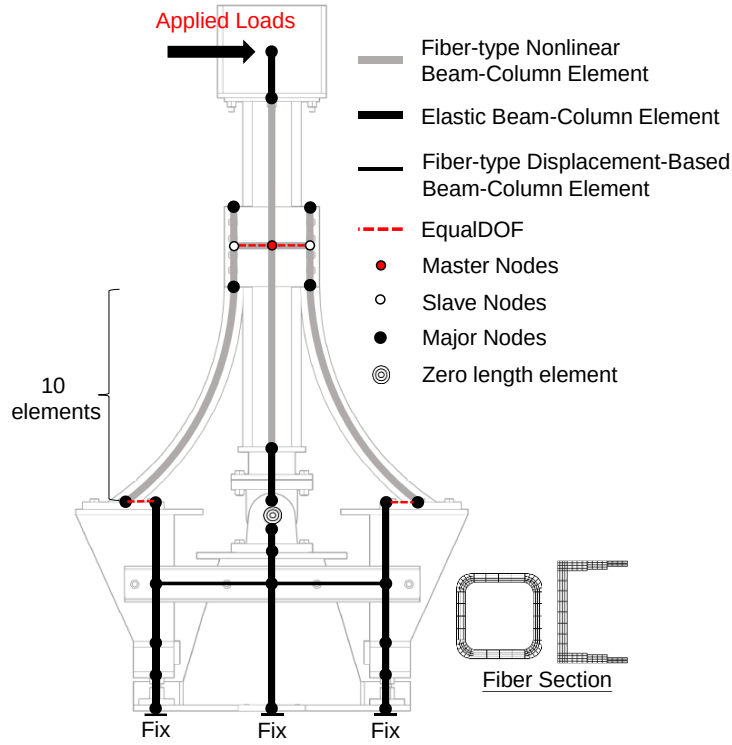


Figure 5-15 Adopted numerical model of the specimens with loading frame system.

The resulting analytical responses of the IHB and CFB are shown in Figure 5-16 compared to the measured responses. The developed numerical model generally presented high accuracy and good agreement with the test results in terms of strengths and stiffness. It should be noted that the model of fiber sections was based on the assumption of plane-section-remain-plane; however, it was unable to simulate the strength degradation caused by local buckling in CFB. In addition, the pinching behavior caused by the slipping at the bolt connection was not simulated in the model. The developed numerical model of the curved member retrofit system could generally be adopted for analytical studies at structural system levels.

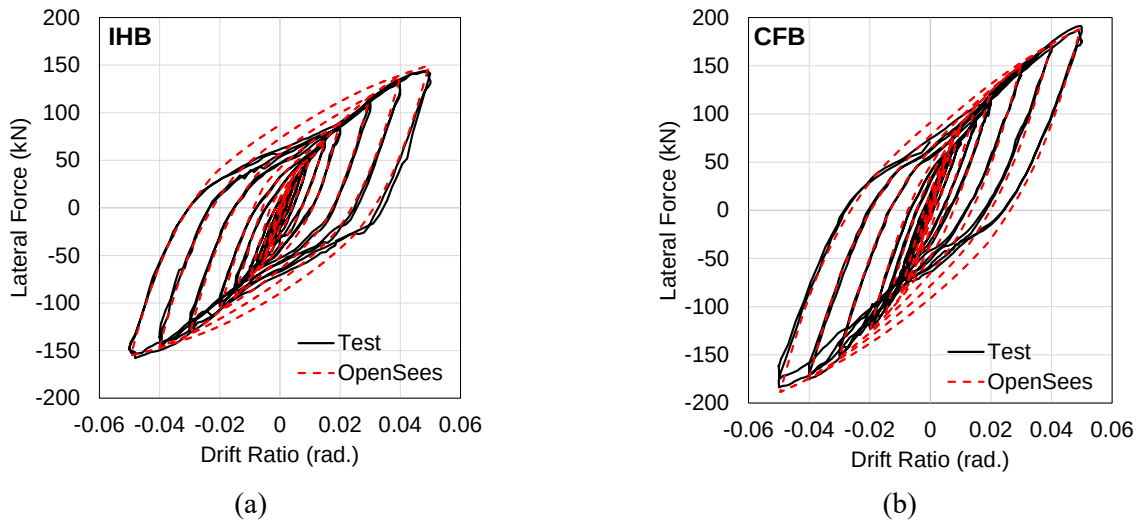


Figure 5-16 Comparisons of simulated and measured hysteretic results.

5.6 Summary

A retrofit system utilizing curved members has been proposed in this section, aiming to achieve moderately enhanced strength, stiffness, and improved ductility compared with the common retrofit method for column bases. The basic behavior, performance, and design targets of the proposed retrofit system were analytically examined by pre-analysis. Four curved member specimens have been tested, focusing on the behavior of the specimens. The effects of different fabrication processes and boundary conditions on the behavior of the specimens were investigated. Moreover, a numerical model of the curved member retrofit system was developed and validated by the experimental results. Based on the analytical and experimental investigations presented in this section, the following key findings are drawn:

- (1) The proposed curved member retrofit system was analytically verified to achieve design targets, including (a) moderate-enhanced strength, stiffness, and improved ductility; (b) increased plastic rotation capacity to reduce the seismic demand for the column base.
- (2) Numerical simulations confirmed that the behavior of the proposed curved member retrofit system exhibits improved ductility with an enlarged plastic region, where the plastic strength corresponds to the early yielding of the curved member and the delayed yielding of the anchor rods at the original column base. The drift angle of anchor rods yielding at the retrofitted column base was increased by 36% compared with the original column base, indicating that the proposed retrofit system effectively enlarges the plastic rotation capacity of the original column base and reduces its seismic demand.
- (3) A set of equations for estimating the lateral load capacities of curved member retrofit systems was established based on the theoretical equations in Section 4.2. These equations were validated through numerical simulations, demonstrating good agreement with experimental results and accurately capturing the observed behavior.
- (4) A prototype design procedure for the proposed curved member retrofit system was developed based on the validated numerical model. The design process was structured to ensure that the retrofit system achieves the targeted performance objectives.
- (5) The mechanical performance of curved channel members fabricated by induction heating and cold-form bending was experimentally investigated. Induction-heated specimens exhibited stable cyclic behavior with limited strength degradation and controllable yielding region distributed around the middle of the curved member. However, the initial imperfection and poor precision of the cold-formed specimen in this test likely resulted in early local buckling, strength degradation, and installation challenges. However, given sufficient fabrication accuracy, cold-formed bending members may still be suitable for use in the proposed retrofit system.
- (6) For IHB-M specimen, minor end-plate slipping occurred in the cycles of 0.5% story drift but did not influence the behavior, such as severe strength degradation and pinching. The local buckling occurred in the cycles of 4.0% drift angle, no strength degradation was observed throughout the loading protocol, implying the feasibility in practical applications.

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CHAPTER 6

DEVELOPMENT OF SUBASSEMBLY RETROFIT USING CURVED MEMBERS

6.1 Introduction

6.1.1 Background

In past seismic events, steel industrial buildings have exhibited various forms of damage concentrated in structurally vulnerable or brittle components. Notably, damage has been frequently observed at column bases, beam-to-column connections, and bracing joints. Non-ductile fractures in welded joints, anchor bolt failure, local buckling of slender members, and loss of bearing capacity in panel zones have all been documented as contributing factors to structural degradation or collapse [6-1, 6-2]. Insufficient detailing, such as poor weld access, low-ductility bolts, and insufficient confinement in the base plate region, can lead to premature failure even under moderate seismic excitation [6-3]. The presence of such deficiencies compromises the structural load path, reduces energy dissipation capacity, and increases the risk of functional disruption.

To address these issues, Chapter 5 proposed a design for the curved member retrofit technique at column bases, which involves arranging two curved members symmetrically against the column and connecting them with high-strength bolts and steel plates, as shown in Figure 6-1(a). This system was validated experimentally and analytically to mitigate the limitations of conventional retrofit approaches or brace systems.

To expand the application of the proposed curved member retrofit technique from just column bases to other brittle or older structural components in buildings, this chapter presents a series of experimental and analytical investigations. A series of sub-assembly experiments was conducted to test assemblies of curved members with column bases and beam-column connections. This chapter first presents the design concept and mechanism of the retrofit technique, followed by the design of the test specimens and observations of experimental results. Two half-scale specimens, including the original steel frame and the same steel frame after the application of curved member retrofit, were comparatively investigated in the study. The improvements of the post-retrofit frame, such as overall and local behavior, moment distributions, reparability, and ductility limits of frames, are then discussed in sequence based on the measured responses. A general numerical model of the post-retrofit frame using a discrete modeling approach was developed and validated by the measured results to capture the yielding mechanisms of the frames and the connections.

6.1.2 Organization

Section 6.2 introduces the design concept and mechanical behavior of the proposed curved member retrofit system when applied to structural sub-assemblies. Section 6.3 describes the experimental program conducted to investigate the cyclic behavior of steel frame sub-assemblies with and without the curved member retrofit. This section details the specimen design, material properties, boundary conditions, and loading protocols adopted for the tests. Section 6.4 presents the measured responses and key observations from the tests. Emphasis is placed on evaluating overall structural behavior, validating the retrofit design concept, examining component-level responses, and assessing energy dissipation capacity and ductility enhancement provided by the retrofit system. Finally, Section 6.5 discusses the development and validation of a numerical model simulating the behavior of the retrofitted frames.

6.2 Design and basic mechanism

6.2.1 Proposed retrofit technique

The curved member retrofit technique was developed based on two key concepts: (1) the introduction of an intended initial eccentricity through the inherent curvature of the retrofit curved member, and (2) the minimalization of disturbance to the structural behaviors of the existing buildings. This approach aims to moderately enhanced strength, stiffness, and improve ductility for structures, while reducing the seismic demand on specific weak/nonductile components. As discussed in Chapter 4, the intended initial eccentricity in curved members serves multiple purposes by helping control the buckling direction, mitigate strength degradation due to buckling, and facilitating stable post-buckling behavior.

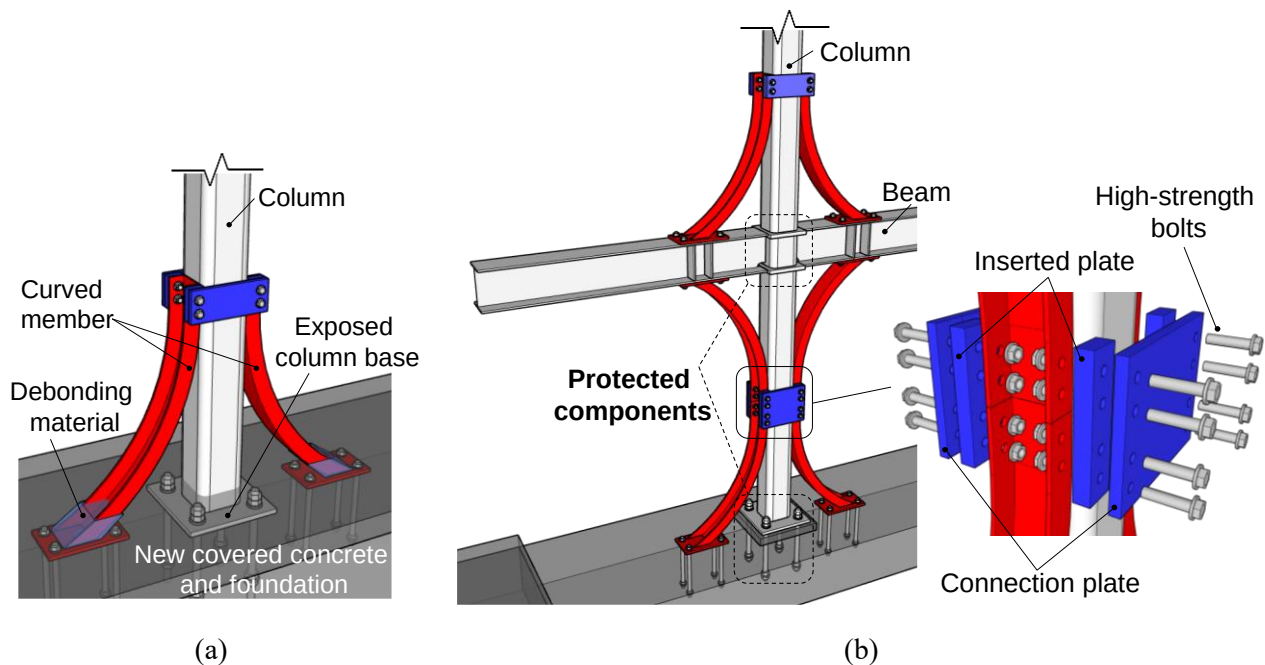


Figure 6-1 Schematic of the proposed curved member retrofit technique for (a) column bases and (b) structural components in frame system.

This chapter extends the application of this technique from a single component, specifically column bases, to the structural level, aiming to rehabilitate low-ductility or aging structural components. As illustrated in Figure 6-1(b), the enhanced retrofit configuration applies the curved member technique to both the beam-column connection and the column base within a steel moment-resisting frame, employing six curved members. These curved members are symmetrically arranged and make direct contact with the existing column. They are connected at the top ends by front and rear steel connection plates using high-strength bolts. Acting as modular plug-in devices like diagonal braces, the curved members are designed for rapid external installation with minimal disruption to existing structural components.

6.2.2 Design approach

The primary design objectives for the retrofitted frame are: (1) moderately increasing strength and stiffness; (2) enhancing plastic deformation capacity by reducing demand on non-ductile elements in older or damaged structures; (3) improving reparability and overall ductility limit. While simultaneously achieving all three objectives is ideal, implementation typically proceeds in stages: first meet strength targets, then assess and refine both local and global ductility.

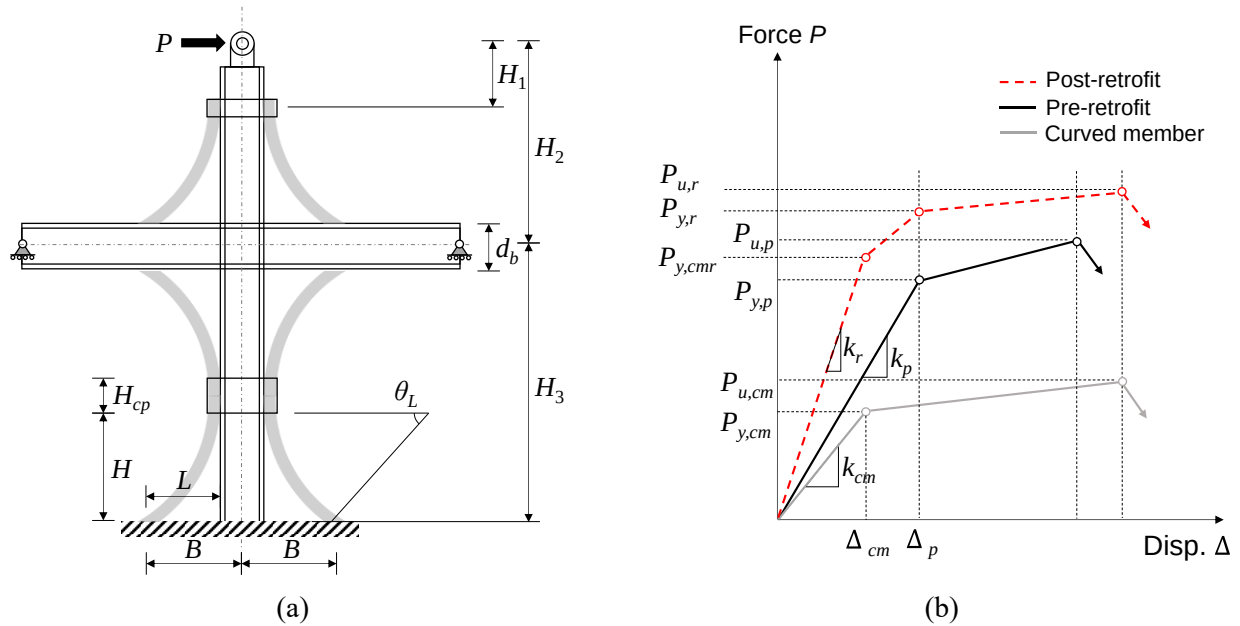


Figure 6-2 (a) illustration of design parameters and (b) comparison of strength backbone curves of pre- and post-retrofit frames.

The design of the curved member retrofit technique adopts a force-based design approach, wherein the retrofit system is proportioned to achieve a specified increase in lateral strength as illustrated in Figure 6-2. The design target strength for the retrofitted frame is defined based on project-specific requirements, such as increasing the story shear capacity by a specified margin beyond the pre-retrofit condition through the additional resistance provided by the curved members. This enhancement is achieved using the superposition principle, wherein the lateral strength contributed by the curved members is added to that of the existing frame. For the subassembly illustrated in Figure 6-2, the lateral strengths of the pre-retrofit frame, $P_{y,p}$ and $P_{u,p}$, are assumed

to be capped by the formation of plastic hinges at the beam ends and column base, representing a collapse mechanism under lateral loading. To estimate the plastic strengths, the Floor Moment Distribution Method is employed, and panel zone deformation is neglected for simplicity, as shown in Eqs. (6-1) and (6-2). The lateral stiffness of the pre-retrofit frame, k_p , can be calculated using Eq. (6-3) based on the typical beam theory.

$$P_{y,p} = (2M_{y,b} + M_{y,cb})/(H_2 + H_3) \quad (6-1)$$

$$P_{u,p} = (2M_{u,b} + M_{u,cb})/(H_2 + H_3) \quad (6-2)$$

$$k_p = 12EI/(4H_2^3 + H_3^3) \quad (6-3)$$

where $M_{y,b}$ and $M_{u,b}$ are the yield and plastic moment of the beam, respectively. H_2 and H_3 represent the distances from the beam centerline to the top loading point and bottom column ends, respectively. E is the elastic modulus and I is the second moment of area of the column. The $M_{y,cb}$ and $M_{u,cb}$ are the plastic bending capacities of the exposed column base according to the design guideline [6-4].

The lateral strength contribution of the curved member retrofit ($P_{y,cm}$ and $P_{u,cm}$), are estimated at the point where the curved members begin to yield and are fully yielded. Notably, the linear deformation of the column is assumed. The column and beams do not provide any lateral strength resistance; all strength is attributed solely to the curved members. The contribution can be calculated using Eqs. (6-4) and (6-5). While the lateral stiffness of the post-retrofit frame, k_r , is estimated by isolating the contribution of the curved member retrofit system. The existing frame is assumed to provide no lateral stiffness, in which the column base behaves as a pinned support. The post-retrofit frame in Figure 6-2(a) consists of six symmetrically arranged curved members, acting in pairs on each side of the column. Each pair functions through axial deformation, transformed into lateral resistance based on the geometric configuration. The lateral stiffness provided by one pair of curved members, k_{pair} , can be calculated by Eq. (6-6). To estimate the overall lateral stiffness of the curved member retrofit system, the pairs of curved members are considered to act in series with each other. Assuming linear elastic behavior and applying the series stiffness principle, the equivalent lateral stiffness of the frame system, k_{cm} , can be estimated by Eq. (6-7).

$$P_{y,cm} = \frac{2(H_1 + H_{cp} + d_b)N_{y,fix} \cos(\tan^{-1} H/L) + 2BN_{y,fix} \sin(\tan^{-1} H/L)}{H_2 + H_3} \quad (6-4)$$

$$P_{u,cm} = \frac{(N_{y,fix} + N_{ct})(H_1 + H_{cp} + d_b) \cos(\tan^{-1} H/L) + (N_{y,fix} + N_{ct})B \sin(\tan^{-1} H/L)}{H_2 + H_3} \quad (6-5)$$

$$k_{pair} = \frac{2(EA/\sqrt{H^2 + L^2})}{[1 + (0.73e/(1 + \alpha)r)^2]} \times \cos^2(\tan^{-1} H/L) \quad (6-6)$$

$$k_{cm} = (\sum_{i=1}^n 1/k_{pair,i})^{-1} \quad (6-7)$$

where H_1 is the distance from the loading point to the center of the top connection plate, H_{cp} is the length of the bottom connection plate, d_b is the depth of the beam, B is the distance between the column centerline and the curved member ends, e is the intended initial eccentricity which varies with the height, width, and central angle

θ of the curved members and can be calculated using the following Eq. (6-8), r is the radius of gyration of the cross section. The coefficient of 0.73 in Eq. (6-6) represents the effect of moment distribution along the member, i.e., the ratio of the maximum moment to the average moment on the curved member, based on the results from Section 4.2.1. In Eqs. (6-4) and (6-5), the plastic strengths ($N_{y,fix}$ and N_{ct}), of the curved member in both tension and compression was primarily determined by the cross-sectional properties and geometries using Eq. (5-3) based on the elastic beam-column theory and an empirical factor α as shown in Eq. (4-15). Where σ_y is the yield stress of the steel material, and e denotes the intended initial eccentricity. Additionally, A is the cross-sectional area. H and L are the height and width of a single curved member.

$$e = \frac{\sqrt{L^2 + H^2}}{2 \sin \frac{\theta_L}{2}} \left(1 - \cos \frac{\theta_L}{2} \right) \quad (6-8)$$

The total lateral strengths ($P_{y,r}$ and $P_{u,r}$) and the stiffness (k_r) of the post-retrofit frame are then given by Eqs. (6-9) to (6-11). The yielding strength of curved members in the post-retrofit frame ($P_{y,cmr}$) can be estimated using the superposition principle as shown in Eq. (6-12).

$$P_{y,r} = P_{y,p} + P_{y,cm} \quad (6-9)$$

$$P_{u,r} = P_{u,p} + P_{u,cm} \quad (6-10)$$

$$k_r = k_p + k_{cm} \quad (6-11)$$

$$P_{y,cmr} = \Delta_{cm}(k_{cm} + k_p) \quad (6-12)$$

A curved member retrofit design process is outlined in the flowchart (Figure 6-3) based on the developed estimation equations for sizing the curved member sections and is summarized as follows.

1. Initial strength calculation: calculate the story shear capacity, $P_{y,p}$, of the pre-retrofit frame based on the formation of plastic hinges on the beams.
2. Define retrofit target: set the target story shear capacity, $P_{y,r}$, of the post-retrofit frame on the required increase over the pre-retrofit capacity.
3. Curved member design: select the depth of the curved members to closely match the adopted column depth to facilitate alignment and ease of installation. Determine the height (H) and width (L) of the curved members within the applicable range of ratio H/L from 1.0 to 3.0 (Section 4.2.1) and the architectural constraints. Design the curved members' section properties and material strength to meet the required lateral force contribution according to Eq. (6-4).
4. Strength verification: evaluation of whether the combined lateral strength of the pre-retrofit frame and the curved member retrofit system satisfies the required story shear demand using Eq. (6-9). If the required strength is not met, design parameters should be adjusted and the procedure repeated.
5. Connection check: verify that the moment induced in the beam by the curved members remains below the beam's yield moment capacity to ensure elastic behavior at the connections using Eq. (6-13). Additionally, check the bolted connection at the connection plates according to the specification.

$$\sum M_y > M_{cm} \quad (6-13)$$

where M_{cm} is the moment induced in the beam-to-curved member connection by the bending moments and shear forces of curved members. M_y is the plastic moment capacity of the beam.

6. Adjust the geometry, section, or material, and repeat the above steps until all checks are satisfied, if strength or connection requirements are not met.

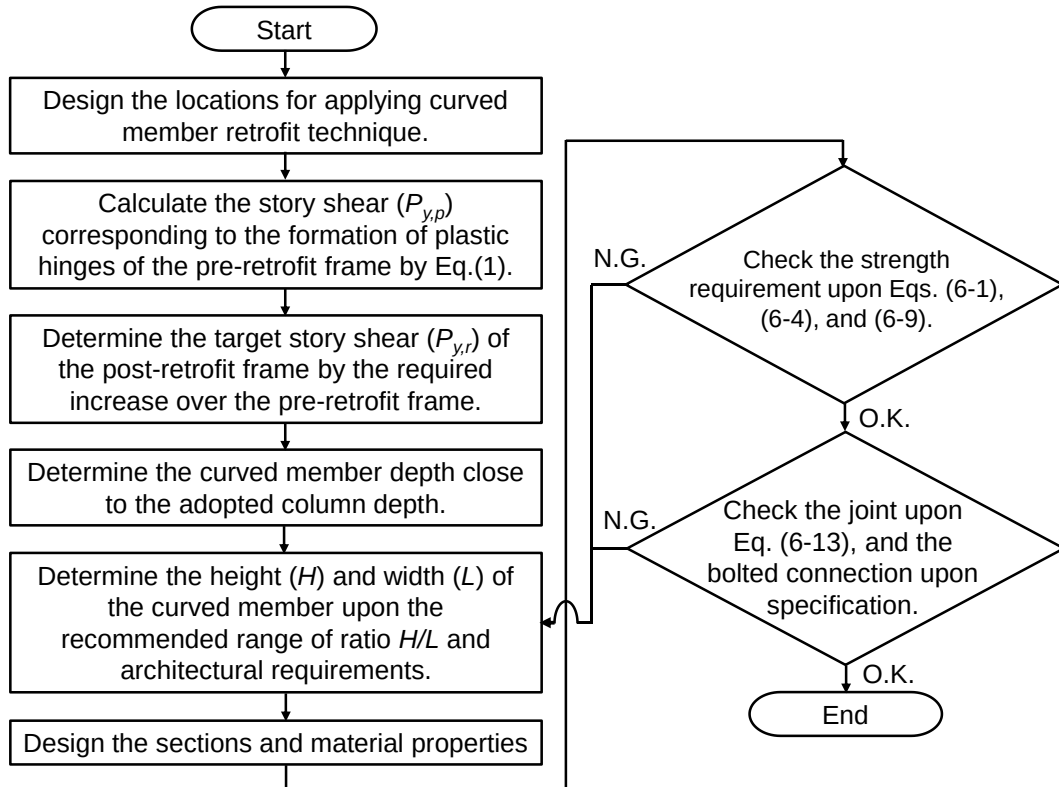


Figure 6-3 Design flow chart for the curved member retrofit system.

6.3 Test program

6.3.1 Design of frame and curved member retrofit

An experimental program was conducted to verify the effectiveness and practical feasibility of the proposed curved member retrofit technique applied to beam-column and column base connections. The program also aimed to evaluate where the technique met the design objectives established in the study and to provide a basis for validating a discrete numerical simulation model, which was also used to extend the findings further.

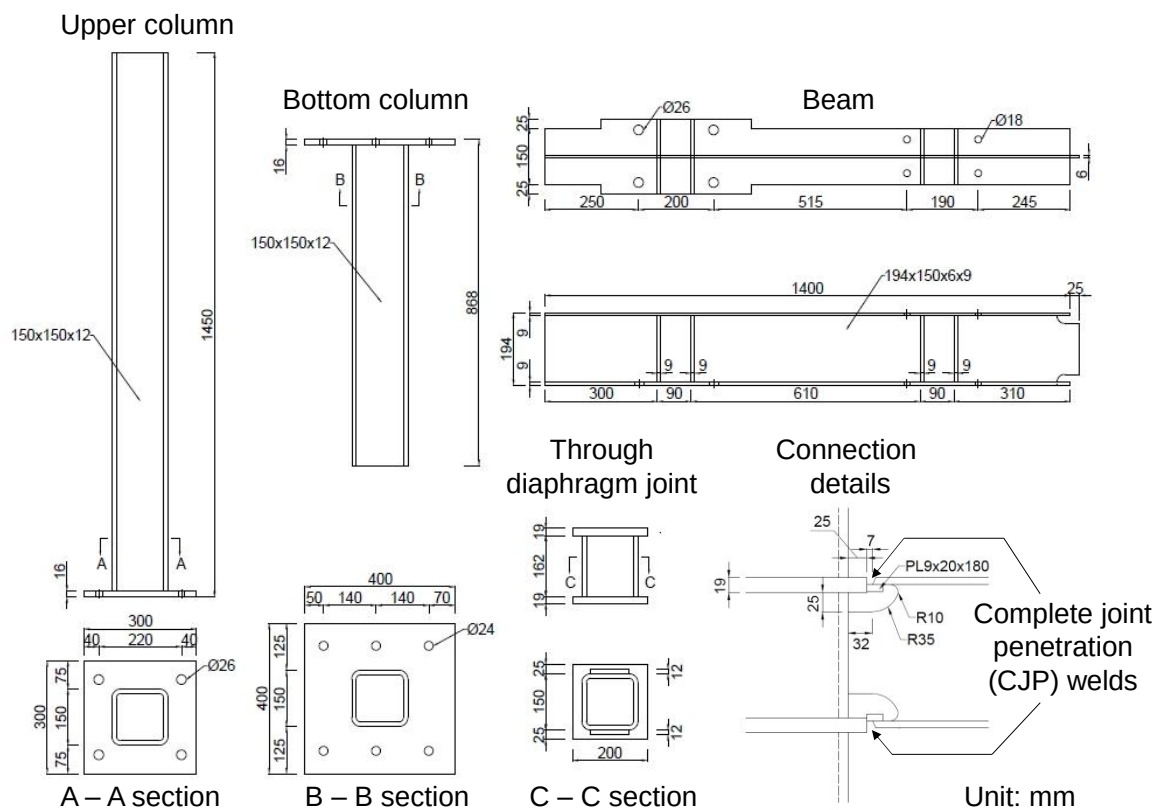
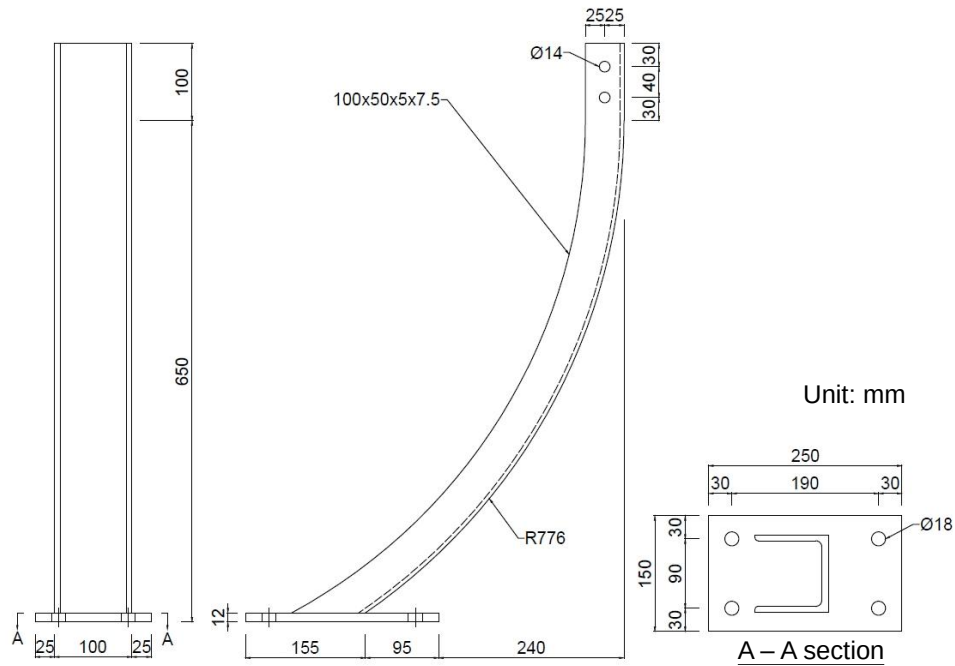
characteristics. The scaling followed the similitude theory to ensure that the test results could be meaningfully interpreted in the context of full-scale behavior. Appropriate application of geometric, kinematic, and dynamic similitude is essential when performing reduced-scale structural experiments [6-8]. Accordingly, the dimensional properties of all components were scaled by a factor of $\lambda = 0.5$, with corresponding force and displacement scaling factors of $\lambda^2 = 0.25$ and $\lambda = 0.5$, respectively. The same structural steel strength grades were used. SS400 and BCR295 steel grades were consistently used for the beams and columns, respectively, as SN490C steel was used for the base plates. Eventually, the half-scale subassembly consisted of the HSS column (HSS 150×150×12), H-section beam (H 200×150×6×9), and the beam-to-column connection adopted a through diaphragm type with PL 200×200×19. The exposed type column base consisted of a base plate (PL 300×300×16) on a 50 mm-thick mortar and four ductile anchor rods (M24, AMB490), designed to have a weak column base mechanism. Doubler plates were used to prevent panel zone yielding.

The curved member retrofit system implemented in the experimental program was designed following the design procedure proposed in Section 6.2.2, targeting a 40% increase in lateral strength over the pre-retrofit frame. This ensured that all strength criteria were satisfied in accordance with the target requirements. Channel steel (C100×50×5×7.5) with steel grade of SS400 was used for the retrofit, and a H/L ratio of 1.86 was selected. Complete joint penetration (CJP) welds connected the channel steel and the end plates.

Table 6-1 presents the measured material properties of the adopted steel frame and curved members based on the tensile tests per ASTM E8/E8M-13a (2013) standard. It is noted that the yield stress of anchor rods was calculated from the minimum cross-sectional area of the threaded region. All detailed dimensions of the steel frame and curved members are provided in Figures 6-5 and 6-6.

Table 6-1 Measured material properties of the coupon tests

Part	Steel grade	Nominal plate thickness (mm)	Nominal elastic modulus E (GPa)	Measured yield stress F_y (MPa)	Measured tensile stress F_u (MPa)
Column	BCR295	12	200	451.8	506.7
Beam flange	SS400	9		327.4	461.1
Beam web		6		321.2	459.8
Channel flange		7.5		302.6	438.4
Channel web		5		297.9	433.0
Anchor rod	ABR490			395.4	598.9



6.3.2 Loading Protocol

An increasing amplitude cyclic loading protocol, as specified in the AISC seismic provisions (ANSI/AISC 341-22), was adopted [Figure 6-7]. In accordance with the standard, the loading was applied with incremental drift increases of 0.02 radians, with two loading cycles at each drift level, to investigate the failure modes of the test specimens thoroughly.

Figure 6-4(b) illustrates the instrumentation setup used in the test. Strain gauges were installed on the curved members to output the moment, and LVDT sensors were used to measure the relative displacement of the end plates and the fixture. The motion-capture camera system was utilized to measure the overall and out-of-plane deformation of the specimen.

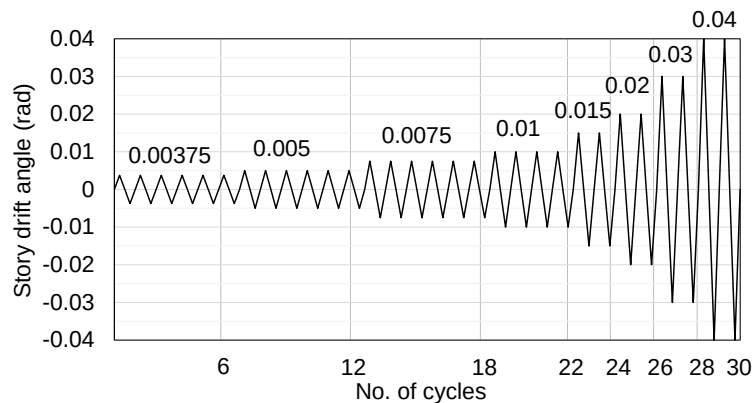


Figure 6-7 Adopted cyclic loading protocol.

6.4 Measured responses and test observation

This section summarizes the measured responses obtained from the two half-scale steel moment-resisting frame specimens, focusing on the effects of the curved member retrofit system. Comparisons are drawn between the post-retrofit and pre-retrofit specimens to evaluate improvements in strength, deformation behavior, ductility, and failure mechanisms.

6.4.1 Overall behavior and design approach validation

Figure 6-8 shows the hysteretic responses of the pre- and post-retrofit frames under cyclic loading. The proposed curved member retrofit significantly affects the responses of the frame, resulting in a 44% increase in yield strength, a 59% increase in ultimate strength, and an 81% increase in initial stiffness calculated by proposed estimation equations with nominal material properties, compared to the pre-retrofit frame.

In the pre-retrofit frame (Figure 6-8(a)), the hysteretic responses exhibit pinching behavior after 3% story drift, primarily attributed to shear key slip at the column base. The major yield mechanism of the pre-retrofit frame was the beam end flexural yielding at the flanges of the H-section and anchor rod yielding at the column base, as shown in Figure 6-9(a). The yielding led to the peeling of the whitewash, which was observed to concentrate at the beam ends, during cycles of 1.0% story drift. The local buckling (LB) of the beam was triggered by the large inelastic compressive strains at the story drift of 4.0%, resulting in strength degradation

for the following cycles, as illustrated in Figures 6-8 and 6-9(b). The maximum strength was reached at 4.0% story drift. Subsequently, beam end fracture (FR) occurred during cycles of -6.0% story drift, leading to a strength degradation to less than 80% of the maximum strength, as shown in Figure 6-9(c). As a result, the test was terminated.

In the post-retrofit frame (Figure 6-8(b)), curved member yielding (Figure 6-9(e)) first occurred at a story drift of 0.75%, and the beam ends and anchor rods yielding occurred at 1.5% and 1.3%, respectively. The apparent local buckling (LB) of the beam ends occurred during the cycles of 4.0% story drift. However, hysteretic behavior remained stable without strength reduction and reached the maximum strength at 6%, compared to the pre-retrofit frame. The beam ends fracture (FR) occurred at the second cycle of +6.0% story drift and led to the beam flange tearing (FT) at -6.0%, as shown in Figure 6-9(d). The local buckling of the curved member (Figure 6-9(f)), occurring during cycles of 6.0% story drift, remained effective in resisting lateral forces. Notably, the beam flange tearing did not result in significant strength loss or negative stiffness, as the force was effectively redistributed to the curved members, which continued to carry the load, maintaining a strength level greater than 80% of the maximum strength until the third cycle, and the test was therefore terminated.

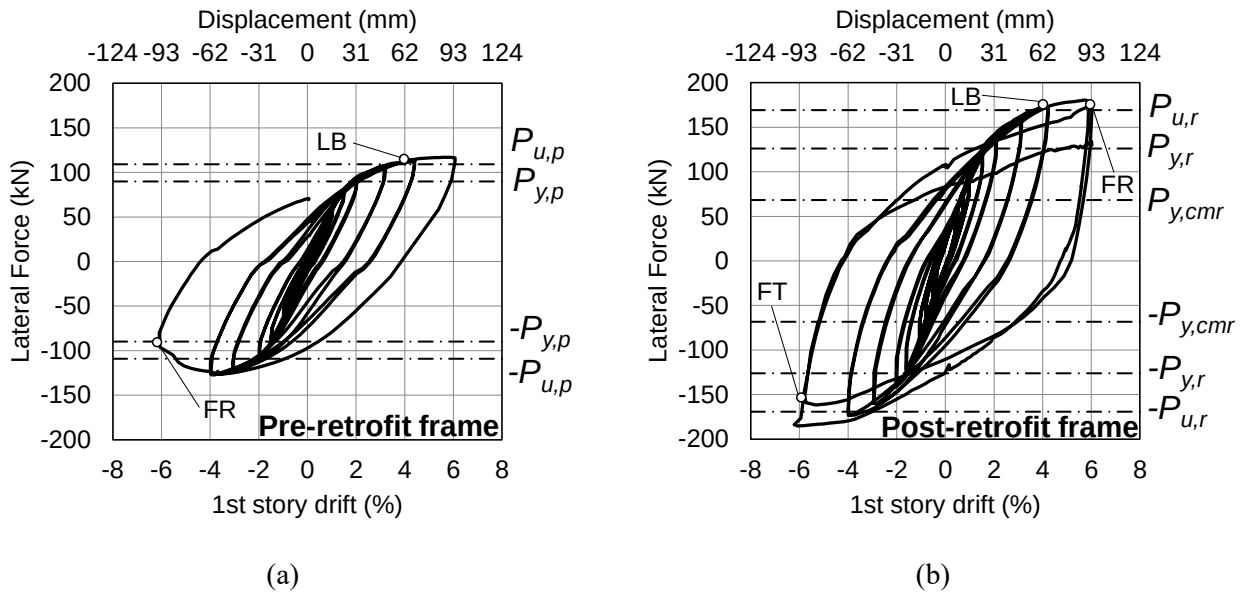


Figure 6-8 Measured hysteretic responses of (a) pre-retrofit and (b) post-retrofit frame.

The resulting values of the calculated strengths based on the design equations with nominal material properties in Section 6.2.2 are shown in Table 6-2 and marked in Figure 6-8 to compare with the measured hysteretic responses. Table 6-2 also summarizes the initial stiffness and resulting story drift levels corresponding to the first yield (Δ_y), local buckling (Δ_{LB}), and frame failure (Δ_{fa}) of the two specimens. The value of ultimate strength ($P_{u,p}$ and $P_{u,r}$) for both pre- and post-retrofit frames were slightly lower than the shown measured ultimate strengths, which reflects the effect of strain hardening. The calculated yield strength ($P_{y,cmr}$) contributed by the curved members, based on Eq. (6-12), corresponds to the initial yielding limited to the surface

of the flanges of the curved channel steel. Based on the test observations and measured results in the study, it is apparent that calculated results enable the representation of the measured lateral strengths of the test responses. Eqs. (6-9), (6-10), and (6-12) were therefore verified to accurately estimate the yield and the ultimate strengths of the post-retrofit frame.

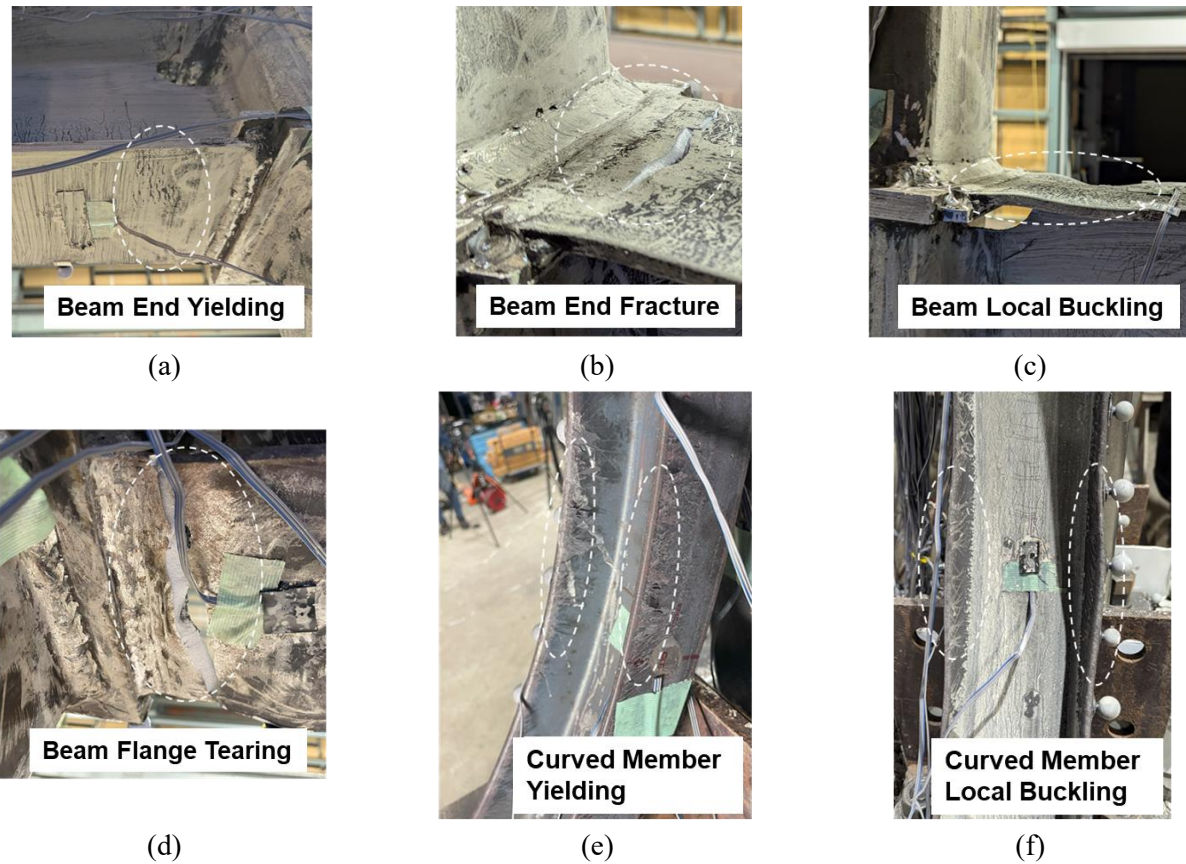


Figure 6-9 Major yield mechanisms and failure modes of the specimens.

Table 6-2 A summary of story drift levels and calculated stiffness and strengths of specimens.

Specimen	Story Drift Levels (%)			Calculated/measured stiffness (kN/mm)	Calculated Strengths (kN)		
	Δ_y	Δ_{LB}	Δ_{fa}		$P_{y,i}$	$P_{u,i}$	$P_{y,cmr}$
Pre-retrofit	1.0	4.0	4.0	5.0/3.9	88.1	107.3	N/A
Post-retrofit	0.75	4.0	6.0	7.1/10.8	126.8	171.0	72.5

6.4.2 Component-level behavior

The component-level behavior was evaluated based on the test observations, instrumentation data, including strain and displacement measurements at the column base, beam-column joint, and curved members.

The column base behavior was monitored using strain gauges and a motion capture camera system for evaluating the bending moment (M_{cb}) and column rotation angle (θ). The column base moment was calculated using the strain gauge values at the column section, and the column rotation angle was calculated using the

markers of the motion capture camera system as the lateral displacement divided by the height. Figure 6-10 shows the hysteretic responses of the column bases, compared to the calculated yield moment ($M_{y,cb}$) and ultimate moment ($M_{u,cb}$) based on the design guideline for steel structure connections [6-4]. Both specimens displayed a typical slip-type hysteretic response commonly observed in exposed column bases. Anchor rod yielding occurred at around a rotation angle of 1.0%, and cracking in the mortar was observed at 2.0%. No anchor rod fracture occurred during the test. The measured values of yield moment closely matched the prediction ($M_{y,cb}$). However, the value of the calculated ultimate moment ($M_{u,cb}$) was apparently lower than the measured moment of column base in the pre-retrofit frame, as shown in Figure 6-10(a). This was likely due to the shear key movement causing the top of the anchor rod to be constrained, and the anchor rods continue to contribute compressive forces after yielding at the large rotation angles. Additionally, base plate yielding was observed at the large rotation angles in the test.

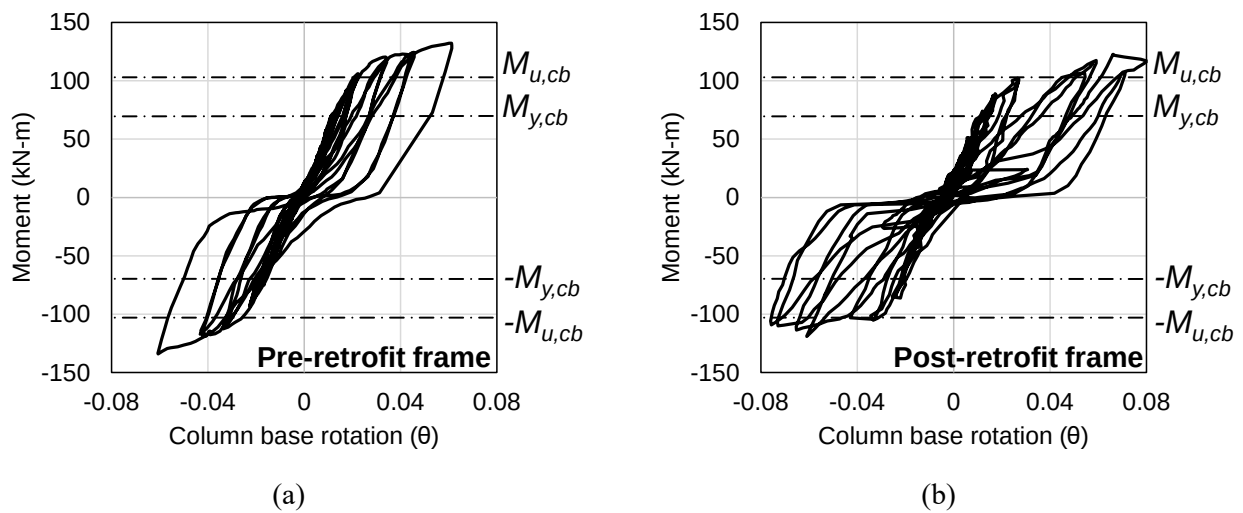


Figure 6-10 Measured hysteretic responses of column bases of (a) pre-retrofit and (b) post-retrofit frame.

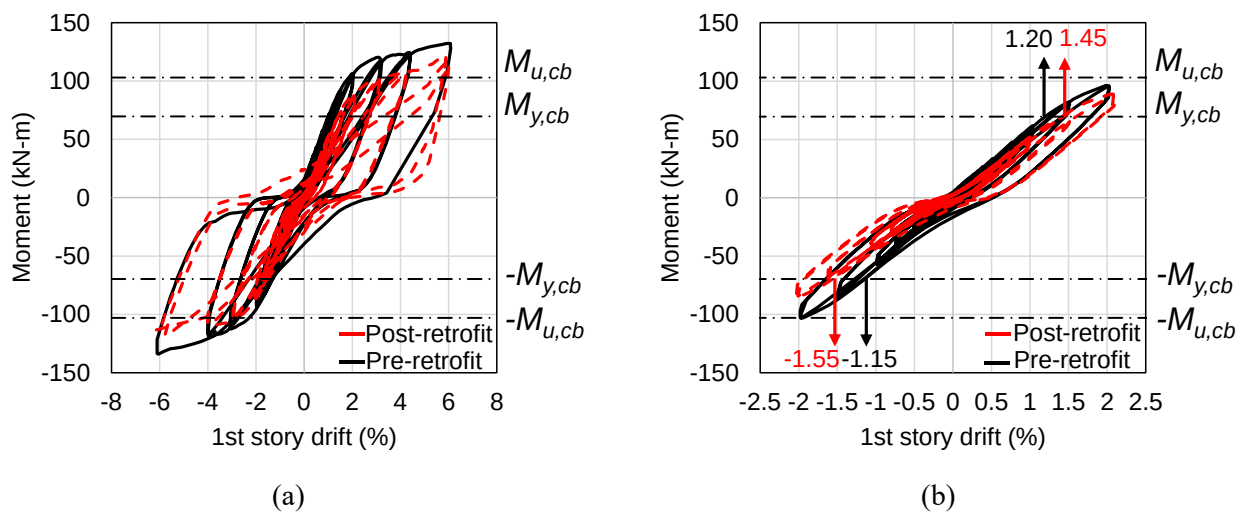


Figure 6-11 Comparison of hysteretic responses of column bases for pre- and post-retrofit: (a) overall; (b) within 2% story drift range.

For the retrofitted column base, the response exhibited a lower bending moment compared to the pre-retrofit column base at the same story drift level. Figure 6-11(b) shows that the pre-retrofit column base reached its yielding moment at positive and negative story drifts of 1.2 and -1.15%, respectively. The retrofitted column base reached its yielding moment at positive and negative story drifts of 1.45 and -1.55%, respectively. Accordingly, the yielding of the column base was delayed by the retrofit with a reduced rotation demand by an average of 28%. It was verified that the curved member retrofit significantly protected the column base by reducing the rotational demand and, therefore, limiting the damage. This behavior ensures the post-earthquake reparability of the column bases.

The behavior of beam-column connections was evaluated using moment distribution along the beam length, as shown in Figure 6-12. Strain gauges placed at multiple locations along the beam flanges enabled calculating the bending moments and comparing the pre- and post-retrofit conditions. Figure 6-12 presents the moment distribution along the beam at the elastic state (0.75% story drift) and plastic state (1.50% story drift) compared to the calculated yield moment ($M_{y,b}$). The points in the figure represent the locations of strain gauge measurements, except for the bending moments at the column center and the column surface, which were calculated using the linear interpolation method.

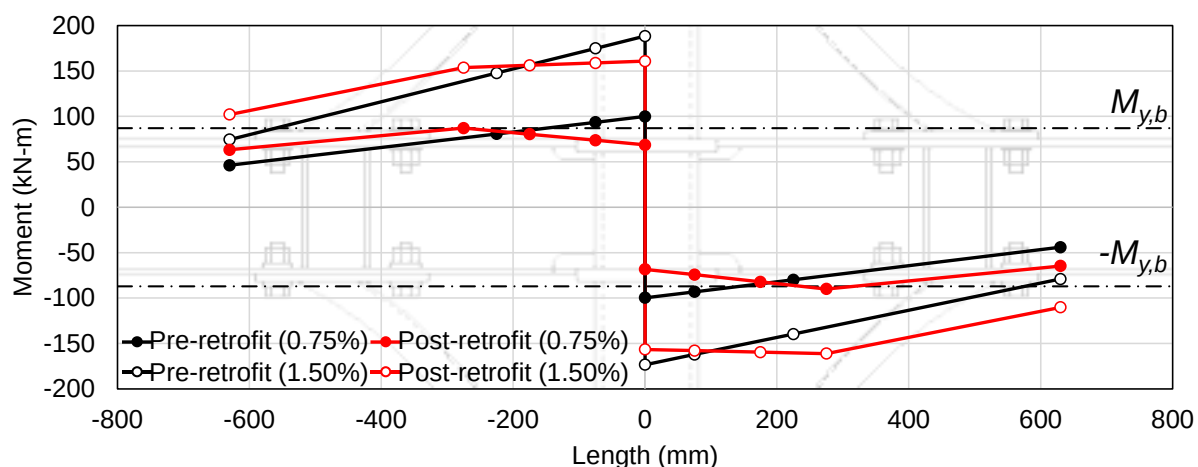


Figure 6-12 Comparisons of moment distributions at the beam-column connection of pre- and post-retrofit frame.

In the pre-retrofit frame, the moment distribution indicated the concentration at the beam ends, indicating that the plastic hinge mechanism would be localized and leading to local buckling at the beam flanges, which was consistent with test observations. In contrast, in the post-retrofit frame, the beam end moments were notably reduced and the yielding region was distributed uniformly, as shown in Figure 6-13. The beam end moments in the post-retrofit frame decreased by an average of 18.8% compared to the pre-retrofit condition. This redistribution indicates that the curved member retrofit shifted the plastic hinges away from the beam ends and reduced the flexural demand.

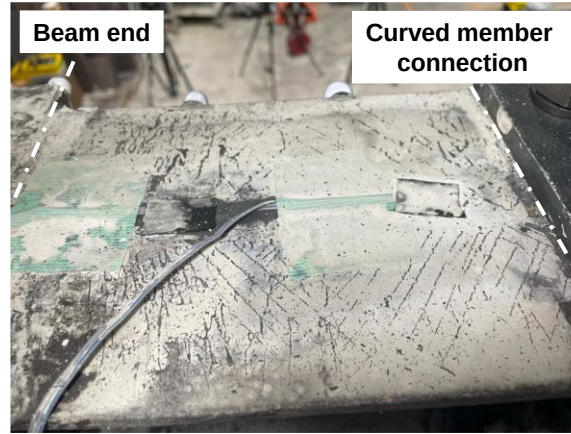


Figure 6-13 The yielding at the beam flange of post-retrofit frame.

The curved members in the post-retrofit frame were instrumented with strain gauges at multiple heights to monitor the behavior and bending moment development throughout cyclic loading. Each curved member is named according to its location within the frame and the specific position of strain gauges used to obtain the bending moments, as shown in Figure 6-14(a). The naming format is structured as [Side, Frame level] _ [Strain gauge level], where [Side] indicates the horizontal position relative to the centerline of the frame (e.g., *L* and *R* represent the left and right side of the column, respectively). [Level] refers to the vertical position of curved member within the frame and the vertical position of strain gauges on the curved member (e.g., *T*, *M*, and *B* represent the top, middle, and bottom levels, respectively). For instance, *RT_T* refers to the right-side curved member at the top level of the frame, and the strain gauge is located at the top level of this curved member, as shown in Figure 6-14(a).

The moment distribution diagrams show the development of bending moments along the height of the curved members under positive and negative loading direction compared to the calculated yield moment capacities ($M_{y,cm}$) of curved members, as shown in Figure 6-14(b) and (c). Upon the measured results, the moment development is shown from story drift of 0.375% to 1.0%. It is shown that the measured results have consistent and symmetric responses between the left and right sides of the frame.

All the curved members developed the maximum moments at the middle of the member, and reached their yielding strength at 0.75% to 1.0% story drift. The curved members at the bottom level of the frame have developed relatively larger bending moments at their middle and bottom levels (e.g., *LB_M*, *RM_M*, *LB_B*, *RB_B*) compared to other levels of the frame, which can be attributed to the rigid connections at the ends to the floor beam. The connection plate slipping was observed in the cycles of 3% story drift, as shown in Figure 6-15. However, it should be noted that the strain measurements from *RT_T*, *RT_M*, and *RT_B* exhibited unstable values from the beginning of the experiment, likely due to instrumentation error. Therefore, this set of data was excluded from the evaluation in the moment distribution.

Overall, the curved members exhibited a stable behavior even after the local buckling occurred at 6.0% story drift; the test observation and measured moment distributions were in line with expectations based on the design, verifying the effectiveness of the proposed retrofit technique.

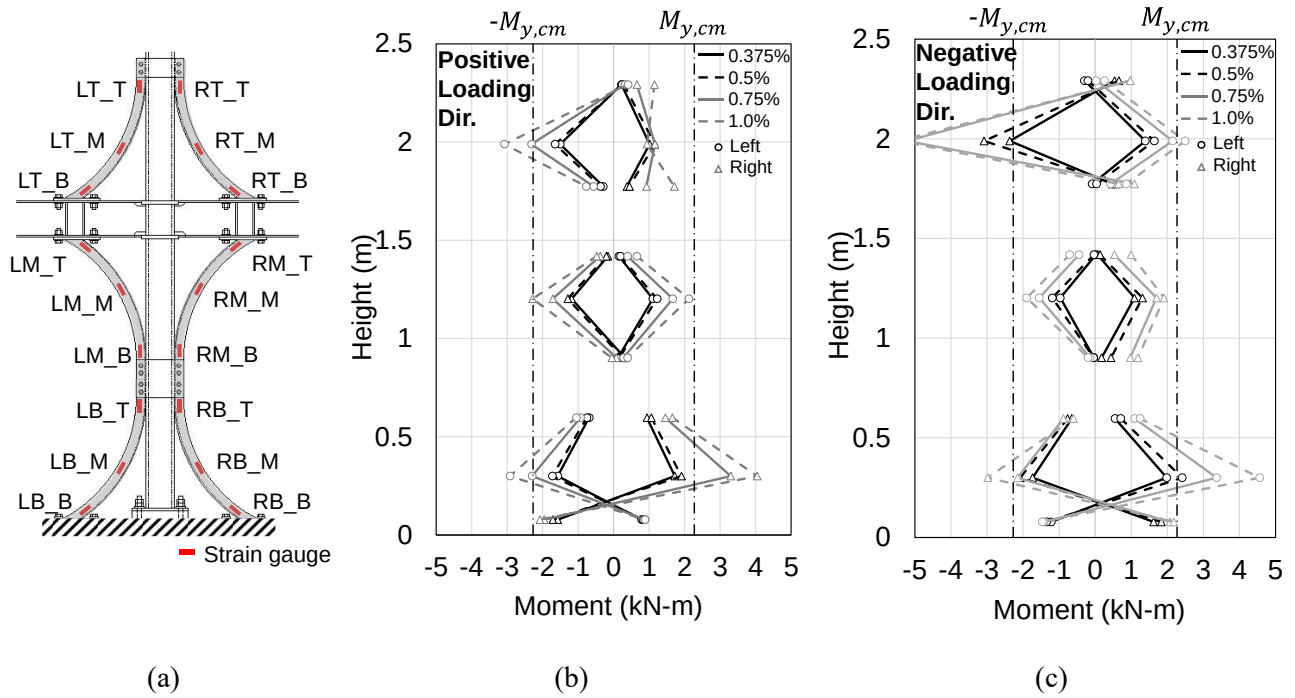


Figure 6-14 (a) Strain gauge locations, and measured moment distribution under (b) positive and (c) negative loading direction.



Figure 6-15 Photos of connection plate slipping at (a) middle and (b) top of the frame.

6.4.3 Energy dissipation

The resulting cumulative energy dissipation and cumulative plastic deformation (CPD) values at each story drift level, calculated from the measured hysteretic responses, are shown in Figure 6-16(a) and (b), respectively. In terms of the energy dissipations, the post-retrofit frame showed an early energy dissipation due to the yielding of curved members. In contrast, the pre-retrofit frame dissipated significantly less energy beyond a 4.0% story drift due to the occurrence of local buckling and the subsequent strength degradation. Additionally, the post-retrofit frame exhibited CPD (144.7 kN·mm) 1.95 times higher than that of the pre-retrofit frame (74.2 kN·mm).

These results verified the effectiveness of the proposed curved member retrofit technique and presented an enhanced energy dissipation capacity compared to the pre-retrofit frame.

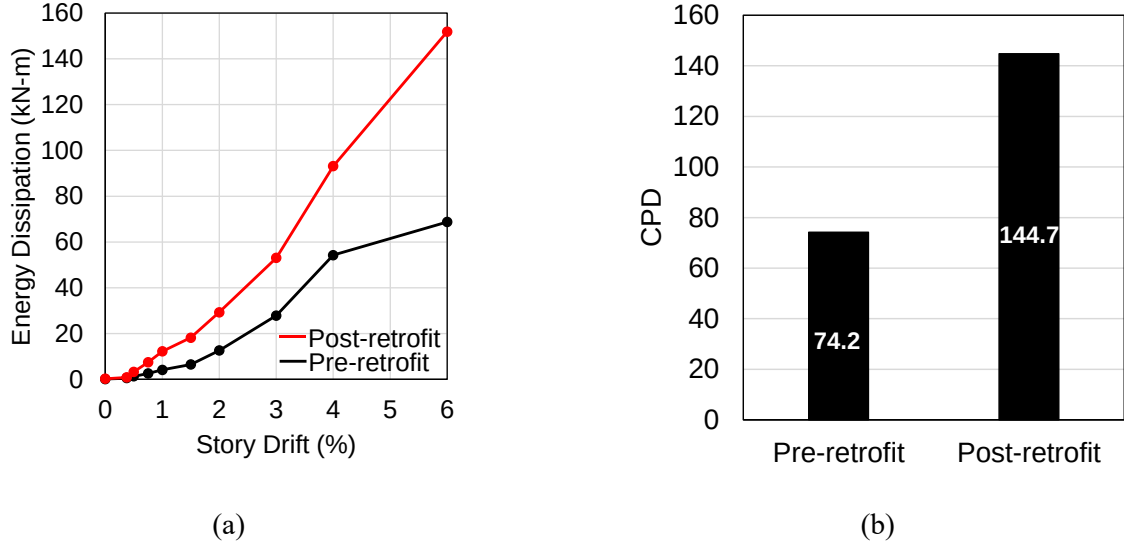


Figure 6-16 Comparisons of (a) cumulative energy dissipation and (b) cumulative plastic deformation (CPD) values among specimens.

6.4.4 Structural ductility evaluation

The retrofit enhanced the structural ductility of the subassembly by achieving the early yielding of the curved members while delaying the yielding of critical components and subsequent strength degradation. Figure 6-17 illustrates the definition of multiple limit states for evaluating the structural ductility. The structural ductility is expressed as Eq. (6-14).

$$\mu = \theta_u / \theta_y \quad (6-14)$$

where $\theta_y = \min(\theta_{y,cm}, \theta_{y,b}, \theta_{y,cb})$ represents the story drift corresponding to the first yielding of components; $\theta_u = (\theta_{u,b}, \theta_{u,cb}, \theta_{u,max})$, is the story drift corresponding to the ultimate states for column bases and beam-column connections, defined based on the deformation capacity of the components. The superscript of 'r' is for the post-retrofit frame.

Table 6-3 summarizes the story drift ratio at various yielding states and the ultimate states for both the pre- and post-retrofit frames. Here, the ultimate state is chosen as the state in which the frame reaches its maximum strength. In the pre-retrofit frame, the ultimate state was governed by beam fracture at a story drift ratio of 0.04 radians, resulting in a ductility of 4.0. In contrast, the post-retrofit frame reached an ultimate state at a story drift ratio of 0.06 radians without experiencing strength degradation, achieving a ductility of 8.0. Thus, the retrofit improved the deformation capacity.

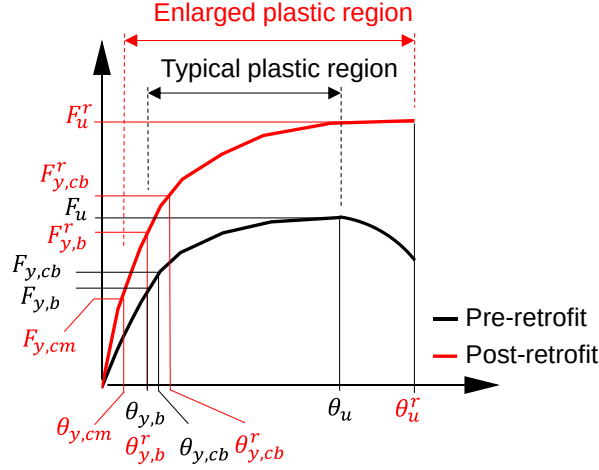


Figure 6-17 illustration of the strength backbones of pre- and post-retrofit subassemblies.

Table 3 Comparison of drift ratios for different states.

State	Component	Pre-retrofit	Post-retrofit
		1 st Story drift ratio (θ)	
Yield state	Curved member ($\theta_{y,cm}$)	N/A	0.0075
	Beam end ($\theta_{y,b}$)	0.01	0.01
	Column base ($\theta_{y,cb}$)	0.0118	0.015
Ultimate state	Frame ($\theta_{u,max}$)	0.04	0.06

Alternatively, when no plastic deformation capacity is expected for the column base or anchor rods, the ductility in terms of the first yielding to the column base yielding becomes 1.0 and 2.0, respectively. When ductility is evaluated specifically for the beam-column connections, which is defined as the first yielding to the ultimate state of the beam ends, the ductility becomes 4.0 and 6.0 for pre- and post-retrofit frames, respectively.

6.5 Numerical modeling and validation

To further extend the experimental results, numerical models of the curved member retrofit system were developed within the OpenSees (Open System for Earthquake Engineering Simulation) framework [6-9] to simulate cyclic responses of specimens in the study accurately. Test specimens, along with the loading frame system, were simulated as illustrated in Figure 6-18

Force-based nonlinear beam-column elements and fiber sections that reflect the cross-sectional geometries were primarily used to simulate the plastic behavior of the beams, columns, and curved members. The uniaxial material model of Steel02, using the yield strength obtained from material tests presented in Table 6-1, was applied for each fiber. Ten elements were used for simulating curved members as recommended by a sensitivity test (Section 4.2.1) and the AISC specification [6-10]. For the loading frame system, elastic beam-column elements were used for the loading beams and columns, with equivalent cross-sectional dimensions to those used in the test. To simulate the rigidity at the panel zone and the connections of the frame, a material with an elastic modulus E was used for the elastic beam-column elements, as illustrated by the bold lines in Figure 6-

18(a). Four pinned joints of the loading frame were simulated by zero-length elements with extremely low in-plane rotational stiffness and fixed in the remaining degrees of freedom.

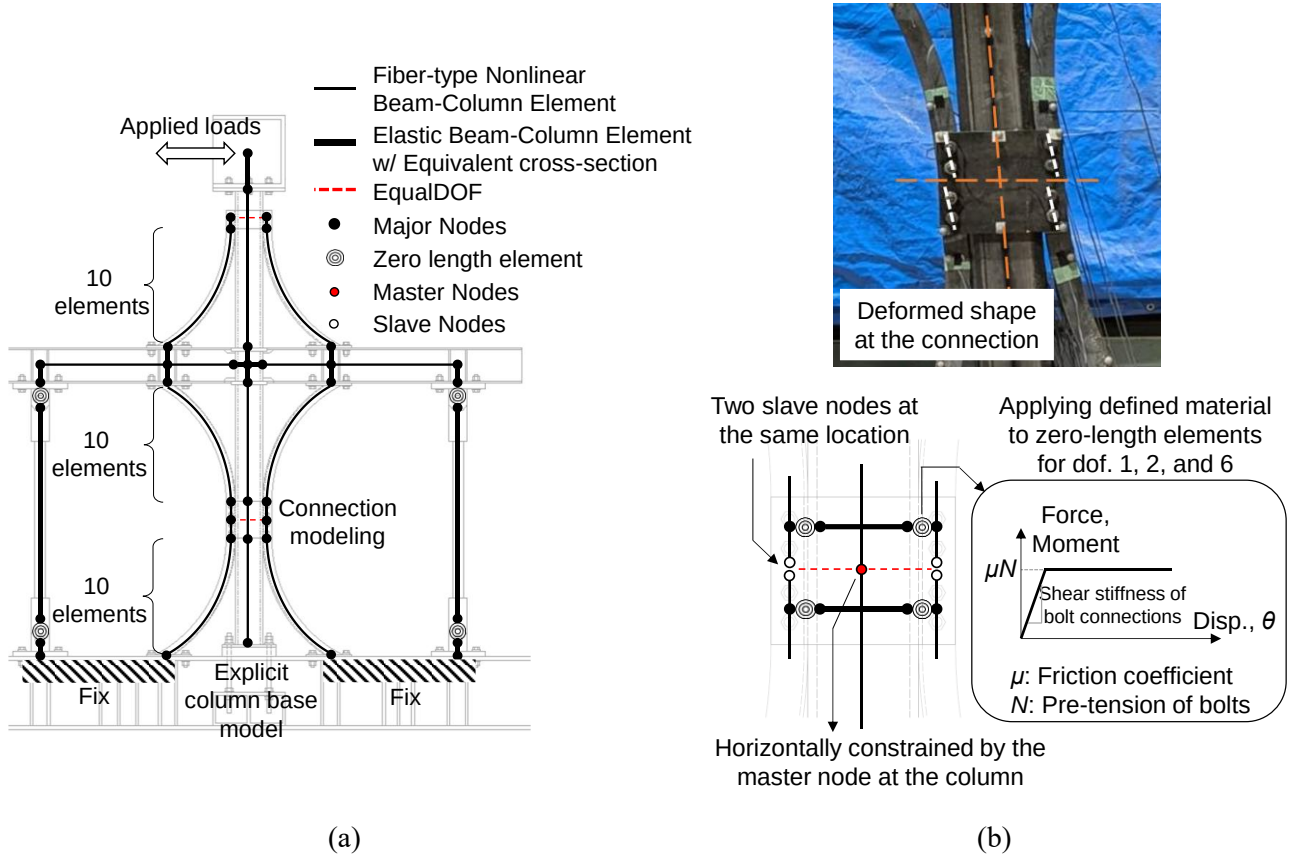


Figure 6-18 (a) Proposed discrete model of specimens along with (b) a connection simulation approach.

The explicit column base model was composed of a base plate and anchor rods [6-7]. The base plate was modeled with a fiber section assigned to nonlinear elements outside the column depth and elastic elements with 10 times the elastic modulus inside the column depth. The anchor rod was modeled with a circular fiber section assigned to a force-based nonlinear element. The material property of the anchor rods was a Hysteretic material in series with the rigid elastic no-compression material to simulate the slip-type behavior.

The development of a numerical model, experimentally validated by the measured results in the study, is intended to provide a general modeling approach for the curved member retrofit technique that can be applied to future research with various arrangements. Figure 6-18(b) illustrates the modeling scheme for the curved member connection, utilizing a nonlinear rotational spring model with defined properties to simulate the connection's rotational behavior. The connection region is modeled using two secondary nodes at the exact location, which are horizontally constrained to a primary node located at the centerline of the column. The connection plate was connected by the rotational springs, where the shear stiffness of the bolt group and the frictional resistance were implemented. Sliding initiates once the limit defined by frictional resistance is reached, where N is the pre-tension of bolts and μ is the friction coefficient of 0.2 for steel-to-steel interaction. Notably, the photo in Figure 18(b) shows the experimentally observed deformation shape at the connection, which

supports the requirement to simulate the connection flexibility rather than assuming a fully fixed connection.

Figure 19 compares the analytical and experimental results of the specimens. To exclude the behavior beyond local buckling, which a fiber-type discrete model cannot capture, only the hysteretic responses within 0.0-4.0% first-story drift angle are compared. The developed numerical model accurately simulated the measured hysteretic behavior, including yield strength, stiffness, and unloading behavior.

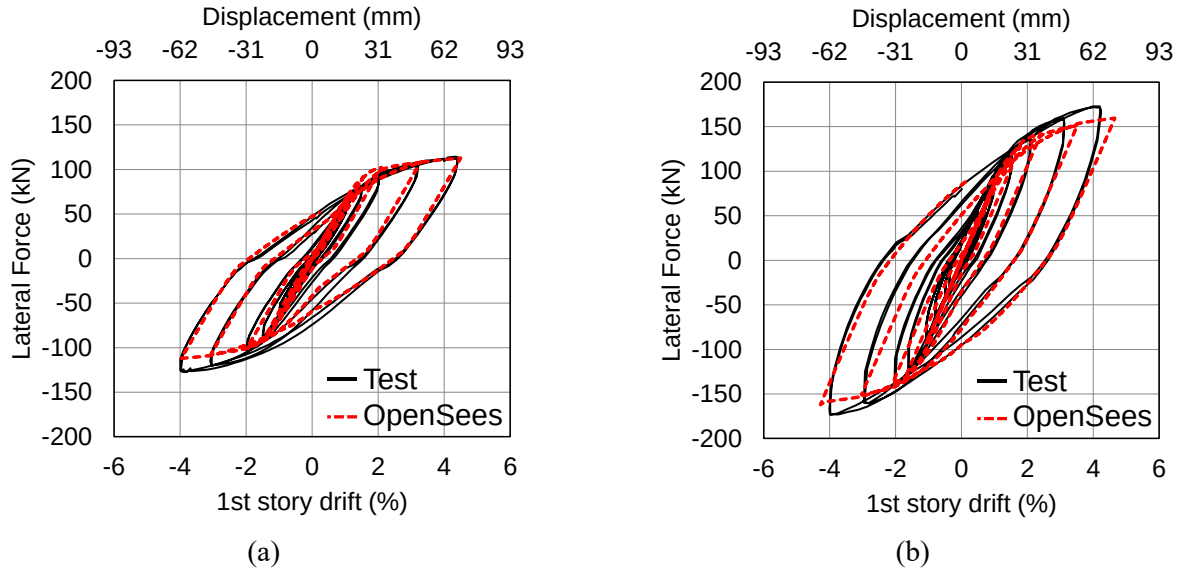


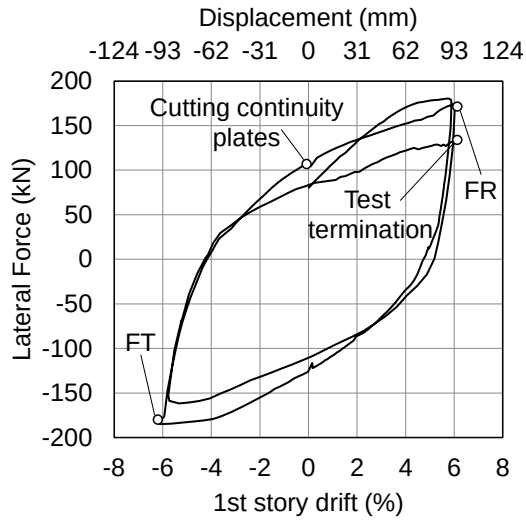
Figure 6-19 (a) Comparisons of experimental and analytical hysteretic responses of (a) pre-retrofit frame; (b) post-retrofit frame.

6.5.1 Continuity plates necessity

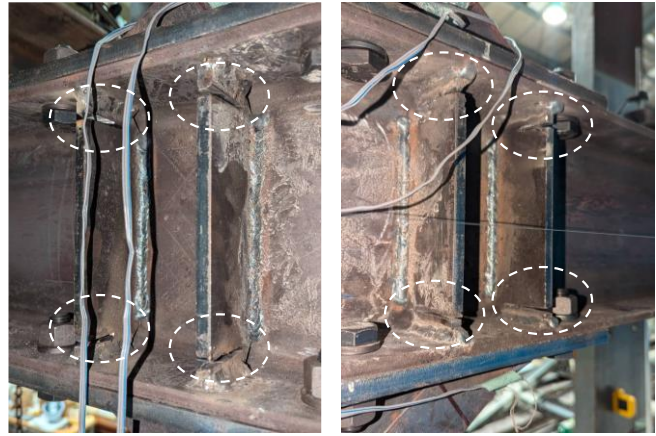
In brace frames (SCBFs) and moment frames (SMFs), brace-to-beam or beam-to-column connections require continuity plates to ensure proper load transfer across the flanges and webs of the connected members. These continuity plates prevent local deformations, flange bending, or web crippling. According to AISC Seismic Provisions (ANSI/AISC 341-22), continuity plates are critical when significant axial or eccentric forces are present at the beam flanges by braces or other members for maintaining force equilibrium at the connection.

The continuity plates were installed in the test mentioned above, yet this section examines the necessity of such continuity plates. Unlike conventional braces, curved members primarily engage through bending rather than axial force due to initial eccentricity, resulting in a more gradual distribution of stresses at the member ends. To examine the effect of continuity plates at the curved member-to-beam connection, an additional experimental investigation was conducted.

After the first cycle at a story drift of 6%, continuity plates were cut through from the beam flange of the post-retrofit frame. Figure 6-20 illustrates the observed failure modes and the photos. Even after the cutting, the post-retrofit frame continued to exhibit a stable hysteretic behavior. No vertical deformation of the beam flange at the connection was observed during the subsequent cycles. Eventually, the subassembly exhibited strength degradation due to the flange tearing at the beam ends.



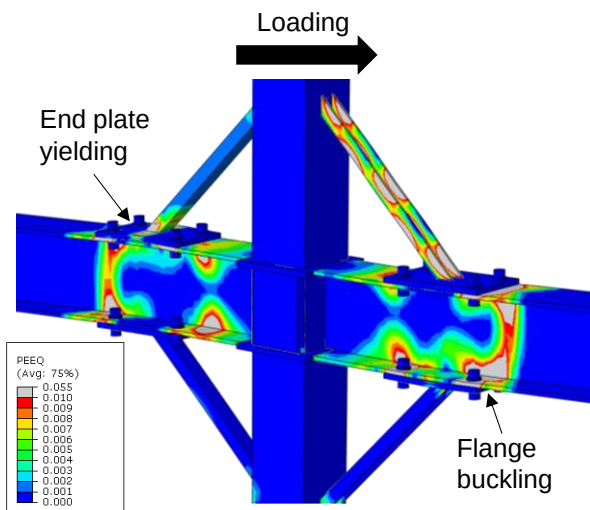
(a)



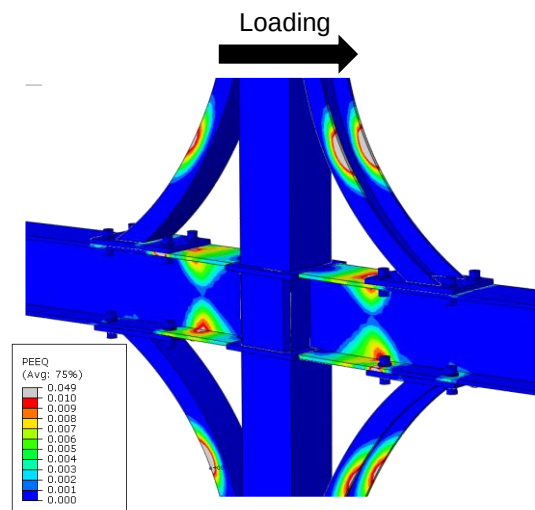
(b)

Figure 6-20 Behavior after continuity plate cutting: (a) measured responses and (b) photos.

To further evaluate the effect of continuity plates at the curved member-to-beam connections, finite element models of the specimen were established using the ABAQUS software [6-11] based on experimental simulations. The FE model was built using an eight-node solid element with reduced integration (C3D8R), along with the interactions of coupling constraints for the conditions and tied contact formulation constraints for simulating the welding. To define the tangential interactions of the surface-to-surface contacts for the bolt connection, the penalty method is used. The friction coefficient of 0.2 was used for steel-to-steel interaction. Hard contact was assumed for the normal behavior of the contact surface.



(a)



(b)

Figure 6-21 Simulation results regarding strain distribution at beams and retrofit members: (a) equivalent straight members; (b) curved members.

Figure 6-21 presents the strain distribution at a 2% story drift ratio for the model with equivalent straight members (C41×30×5×5, SS400) compared to the model adopting curved members (C100×50×5×7.5, SS400).

The results of the model using straight members exhibited significant end plate yielding and flange buckling near the beam connection. In contrast, the results of the model using curved members demonstrated an even stress distribution on the beam, with no significant local yielding or flange instability observed at the connection region without continuity plates. The analytical results are consistent with experimental observations, verifying that the continuity plates are potentially not required in the curved member retrofit technique.

6.6 Summary

This chapter presented a comprehensive experimental and analytical evaluation of a seismic retrofit technique employing curved members at the column base and beam-column joint in steel moment-resisting frames. A force-based design method was developed to determine the required retrofit capacities, and its applicability was verified through quasi-static testing. Furthermore, a general numerical modeling approach for curved member retrofit was validated against the experimental results, and the necessity of continuity plates was discussed. Key findings and conclusions are summarized as follows:

- (1) The post-retrofit frame in the study was experimentally verified to exhibit stable and ductile hysteretic behavior, with early yielding of curved members and no strength degradation up to a 6% story drift. Although local buckling occurred under cycles of 4% story drifts, it did not result in post-yield stiffness deterioration, due to the effectively redistributed forces of curved members, even after the beam ends fractured.
- (2) The test results showed that the curved member retrofit delayed yielding of the column base by reducing the rotation demand by 28%. Moreover, the retrofit reduced beam end moments demand by an average of 19%, and the yielding region along the beams formed a uniform stress distribution compared to the pre-retrofit frame. The curved member retrofit successfully shifted plastic hinge locations away from the beam ends.
- (3) A force-based design methodology was established using estimation equations for lateral strength and stiffness and validated against the experimental results. The target retrofit strength was defined as a 40% increase over the pre-retrofit frame in the study, determined through the superposition principle. The post-retrofit frame achieved a 44% increase in yield strength, a 59% increase in ultimate strength, and a 81% increase in initial stiffness, compared to the pre-retrofit frame.
- (4) The structural ductility at multiple limit states was evaluated. The ductility factor μ , defined as the ratio of ultimate to yield drift, increased from 4.0 to 8.0 for the overall frame system. At the component level, ductility ratios for the column base and beam-column connections improved from 1.0 to 1.7 and from 4.0 to 6.0, respectively.
- (5) A discrete modelling approach was developed to effectively and accurately represent the responses of the retrofitted frame before local buckling, which could be used for performing analysis on various retrofit configurations.
- (6) Finally, finite element analyses and test observations demonstrated that continuity plates at the curved

member-to-beam connections were potentially not essential. Even after removal, the connection exhibited no vertical deformation or strain concentration and maintained stable behavior at cycles of 6% story drifts.

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CHAPTER 7

APPLICATION OF PROPOSED METHODOLOGY

7.1 Introduction

7.1.1 Background

In the preceding chapters, the effectiveness of repair and retrofit strategies for steel column bases and curved member retrofit systems was examined through analytical and experimental investigations, and proposed decision-making and design methodologies. Chapter 2 demonstrated the effectiveness of a simple repair technique involving the re-tightening of anchor rods, particularly in recovering structural performance rapidly after seismic events. Chapter 3 proposed a decision-making methodology that accounts for residual capacity and structural reliability, offering a framework for post-earthquake repair and retrofit planning. Chapters 4 to 6 focused on the development and validation of a novel curved member retrofit technique, which was shown to enhance the seismic performance while minimizing structural disturbances.

Building upon these findings, this chapter presents a case study to demonstrate the practical application of the proposed methodologies while examining their effectiveness in minimizing disturbances to the existing structure. A numerical evaluation was conducted on a four-story steel moment-resisting frame to investigate the seismic performance of the structure after applying a suitable repair or retrofit technique. Both the simple repair method and the curved member retrofit system were examined to assess their respective capabilities under different seismic hazard levels. The results serve to illustrate how the strategies developed in earlier chapters can be integrated into realistic seismic response mitigation practices for damaged steel buildings. These investigations aimed to validate the practical applicability and adaptability of the proposed retrofit technique for steel buildings. The findings in previous chapters further contributed to developing a connection design methodology that enhances material efficiency while preventing overly conservative designs.

7.1.2 Organization

Section 7.2.1 introduces the case study model, including the structural configuration, damage assumptions, and the implementation of the repair and retrofit techniques. Section 7.2.2 presents the performance evaluation of the building, focusing on the effectiveness of each method in restoring or enhancing seismic performance. To further explore the adaptability of the proposed strategies, additional analyses are conducted. Section 7.3 presents a connection design methodology based on the established estimation equations and numerical results. This section introduces a modified connection design philosophy and explains the detailed design of the joints

and intersection areas between the curved members and boundary beams to prevent over-conservative connection designs that do not efficiently utilize material capacity.

7.2 Application of repair or retrofit for damaged buildings

7.2.1 Case study

To demonstrate the practical use of the proposed repair and retrofit strategies, this section applies the methods to a representative damaged steel building and evaluates their effectiveness through numerical simulations. The four-story steel moment-resisting frame (SMRF) introduced in Chapter 2 is used as the benchmark building, representing typical structural configurations and boundary conditions of low-rise buildings, as shown in Figure 2-3. The member sizes of beams and columns of the four-story SMRF building are given in Table 2-5.

The assumed damage condition was introduced through a two-stage dynamic analysis. First, the structure was subjected to initial ground motions scaled to specific intensity levels, simulating realistic earthquake-induced damage, particularly at the column bases. Following this, the appropriate repair or retrofit method was applied, and the structure was reanalyzed using separate performance assessment ground motions to evaluate the post-intervention seismic behavior, as shown in Figure 7-1.

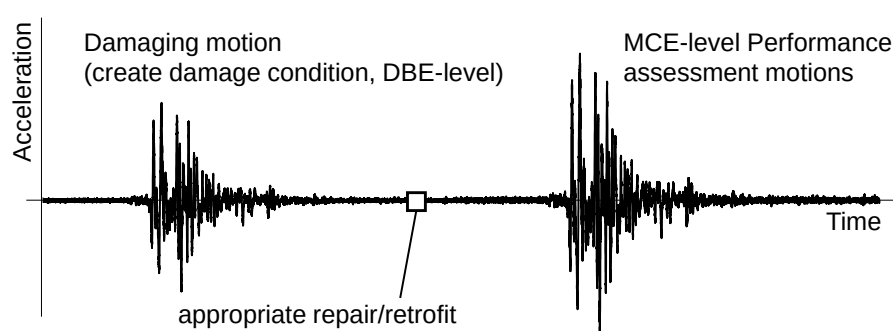


Figure 7-1 Illustrations of sequential ground motions

The numerical model and simulation approach adopted in this chapter follow the same methodology described in Chapter 2.4.2, as illustrated in Figure 2.5. The ground motion measured at the Japan Meteorological Agency (JMA) Kobe station (the NS direction) during the Hyogo-ken Nanbu earthquake in 1995 was used. These ground motions were scaled to the DBE- and MCE-level (1.5 times the design level) for the dynamic analysis, as shown in Table 7-1.

Table 7-1 Details of the ground motions

ID	Ground Motion	Reference
A1	JMA Kobe NS dir. 50%	DBE-level
A2	JMA Kobe NS dir. 75%	MCE-level

For the simple repair strategy, the decision to proceed with re-tightening of anchor rods was based on the reparability assessment flowchart presented in Chapter 3, as illustrated in Figure 3-1. The flowchart incorporated residual capacity criteria to determine whether the column bases remain functionally reliable after damage. If the damage level remained within the acceptable repairable range defined by the probability of failure of anchor rods, the simple re-tightening approach was considered feasible. The corresponding simulation results were used to validate the effectiveness of this method in restoring column base performance without requiring extensive reconstruction.

In the first simulation series, the remaining ductility of anchor rods was evaluated after each ground motion. The remaining ductility and probability of failure (P_f) of anchor rods in the four-story SMRF is shown in Figure 7-2, where the mean value of the remaining ductility factor of anchor rods as well as the required ductility were compared. Notably, the anchor rod used in the four-story SMRF has an initial ductility factor of 7.5 calculated from the material test. The results show that the probability of failure of anchor rods was 0.0 after the ground motion of A1, which was less than the limit value of P_f of 0.05, indicating that the simple repair method of re-tightening process is feasible. Thus, tightening the permanently deformed anchor rods could rapidly restore the overall stiffness of column bases with sufficient deformation capacity. However, the probability of failure of anchor rods for the four-story SMRF after the ground motion of A2 was 0.397, and it exceeded the limit value of 0.05, indicating that the ductility of anchor rods will be insufficient during the next seismic event and may result in severe damage to the structure. Therefore, other retrofit methods were necessary to ensure the required capacity of column bases. The curved member retrofit technique was adopted as another retrofit method in this case study.

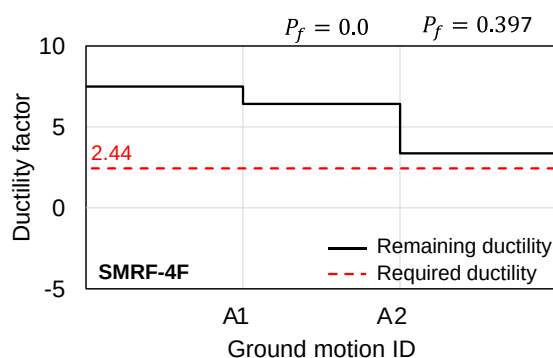


Figure 7-2 Average remaining ductility of anchor rods in four-story SMRF

The adopted curved member retrofit technique for the damaged columns was determined by the design procedure that follows the estimation equations and retrofit design flowchart established from Chapters 4 to 6. The design procedure can be summarized into the following steps: 1) determine the locations of the column bases for the seismic retrofit; 2) for the new building, determine the target moment capacity of the column bases due to the seismic demand. For the damaged building, determine the target moment capacity of the damaged column to 90% of the pre-earthquake condition that provides an acceptable level of risk for long-term occupancy per FEMA-352 [7-1]; 3) design the curved member depth close to the adopted column depth; 4) determine the

height H and width L of the curved member upon the recommended range of ratio H/L and the architectural requirements; 5) determine the material and section properties; 6) check the ultimate moment capacity upon Eq. (5-2) and adjust the design parameters from step 3 to 5 to meet the requirements; 7) design connection details based on modified design methodology for curved member connections described in Chapter 4.3.1; 8) check the design targets including strength, stiffness, and reduced demand of retrofitted column bases. It should be noted that the analysis is needed to check the behavior of the curved member retrofit technique in step 8.

Curved channel steel with a depth of 200 mm was chosen to fit the existing column. The moment capacity (M_u) of the existing column base, EJ250-4-30, can be calculated according to the equations provided by AISC Recommendations for Design of Connections in Steel Structures [7-2], which was 226 kN-m. The curved channel steel with section 200×80×7.5×11 and height (H) and width (L) of 1000 mm and 330 mm was adopted. The resulting space occupancy rate for the building that adopted the retrofit system was 13.2% in a single span on the first floor compared to the original building, as shown in Figure 7-3(b). Steel grades of SS400 with a yield stress of 245 MPa were used for channel steels, and the moment capacity of 207 kN-m can be calculated by Eq. (5-2) in step 6, which satisfies the requirement of reaching 90% of the initial moment capacity of the column base. It should be stated that the design of the curved member retrofit system assumes the behavior of the damaged column base as a pin, and the separation moment of the column base due to axial forces was ignored in this case. All the column bases were retrofitted and examined in one direction only in this section, as shown in Figure 7-3(a). After retrofitting with curved members, the damaged building was then subjected to the same MCE-level ground motion to evaluate the overall seismic performance of the building.

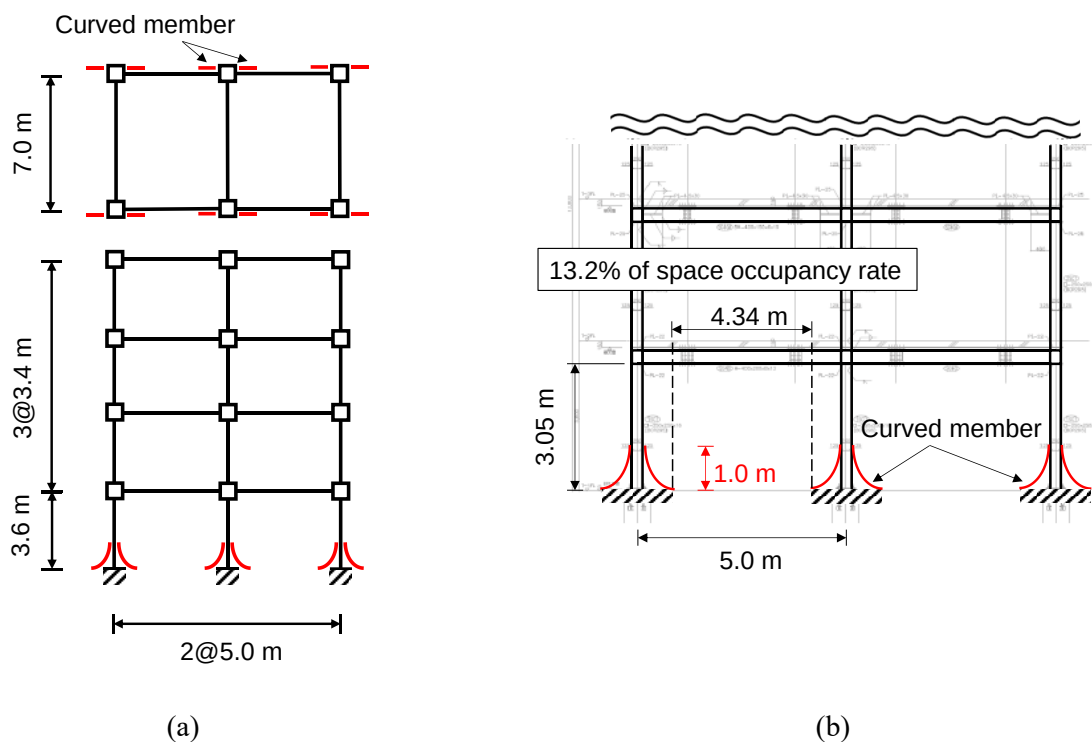


Figure 7-3 (a) Plans, elevations, and (b) space occupancy of the four-story SMRF with curved member retrofit.

7.2.2 Performance evaluation

To evaluate the disturbance impact relative to the pre-retrofit frame, the static pushover analysis that used the equivalent lateral force (ELF) recommended by FEMA 273[7-3] was performed for the four-story benchmark building (SMRF-4F) and the building adopting the curved member retrofit technique (SMRF-CM). The analytical results show that the natural period of the four-story SMRF with curved member retrofit had a 6% reduction, and the stiffness and strength are increased by about 19% compared to the original structure, as shown in Figure 7-4. It implies that the proposed retrofit system cannot effectively increase the seismic demand on the overall frame system.

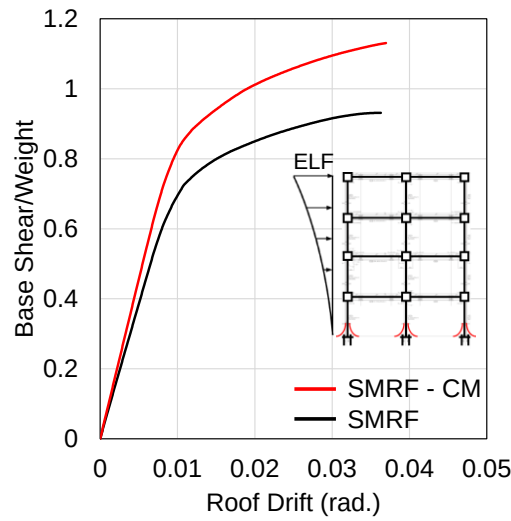


Figure 7-4 Pushover curve of the adopted four-story SMRF with curved member retrofit system

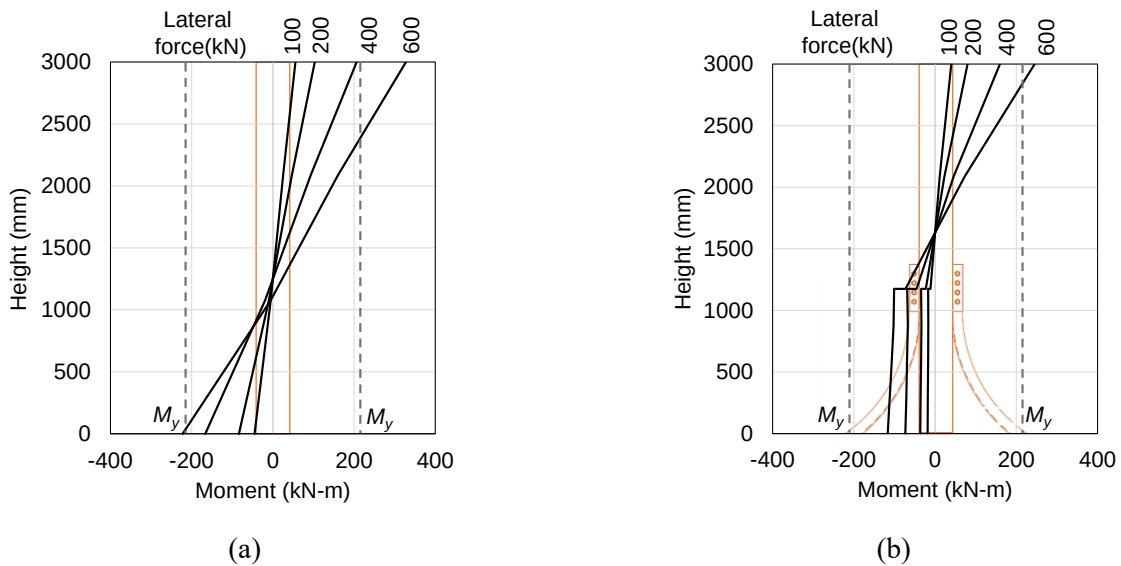


Figure 7-5 Moment distribution of the column in the frame system with (a) original column bases and (b) retrofitted column base.

Figure 7-5 shows the moment distributions on the first-floor column of the four-story frame systems (SMRF-CM and SMRF) for comparing the difference between the retrofitted and the original column bases under various lateral force levels. The analytical results show that the moments at the column base and the beam connection of the retrofitted column base are smaller than those in the frame without retrofit. The result indicated that this retrofit method was not only effective in the frame subjected to horizontal forces but also reduced the demand on the original column base.

Seismic performance evaluations of the four-story benchmark building were performed through dynamic analysis under sequential and single ground motions. The dynamic results were evaluated and discussed through the structural responses. The analytical results of the four-story building under the damaging ground motion of A1 show that the damages were concentrated at the column bases and second-floor beam ends, limiting the responses of the structure within life safety level based on FEMA365 [7-4], based on the maximum inter-story drift.

After inputting the ground motion of A1, a simple repair method of re-tightening anchor rods was adopted while ensuring that the anchor rods have sufficient deformation capacity, and then inputting a subsequent MCE-level ground motion for evaluation. The results demonstrated that the maximum inter-story drift of the four-story building with simple repair was 3.6% at the ground motion of A2, as shown in Figure 7-6(a), indicating the structure remains within the performance level of collapse prevention. An additional single ground motion of A2 was input to represent the responses of an undamaged building subjected to an MCE-level earthquake and compared with the seismic responses adopting the re-tightening process of anchor rods at sequential ground motions. The results of single ground motion were slightly smaller than the responses of sequential ground motion with re-tightened anchor rods, showing that the structural components other than column bases accumulated damage from previous ground motions, and the simple repair enables rapid recovery of the overall stiffness of the column bases to withstand aftershocks.

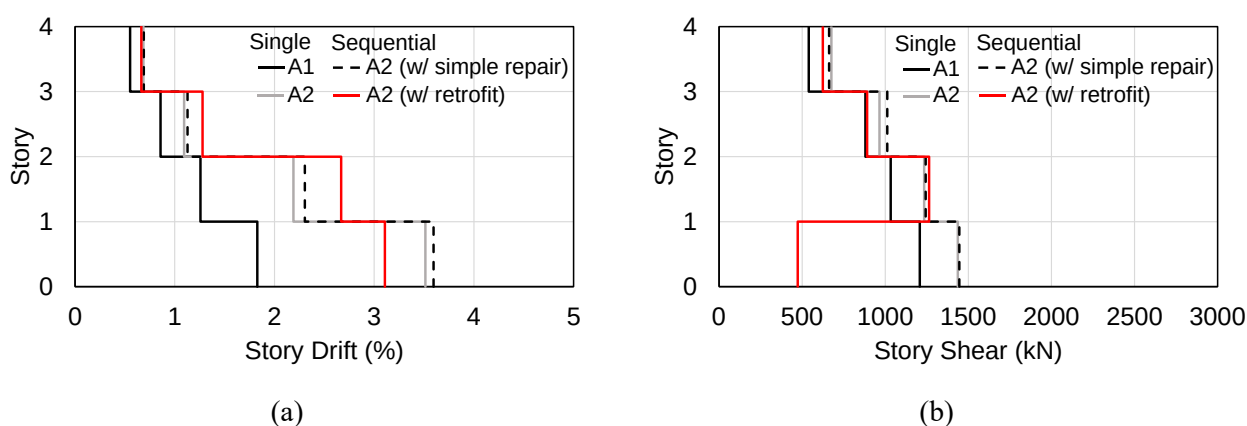


Figure 7-6 The dynamic responses of four-story SMRF: (a) story drift and (b) story shear.

After inputting the ground motion of A2, the re-tightening anchor rods option cannot be adopted due to insufficient remaining ductility ($P_f = 0.397$) described in Section 7.2.1. Therefore, the curved member retrofit

system was selected. The same MCE-level ground motion A2 was input after the damaged building was retrofitted to evaluate the effectiveness and performance of the structure. The analytical results show that the maximum inter-story drift at the first story was 3.11% and was reduced by 11.4% compared to the responses of the initial undamaged building (gray solid line), as shown in Figure 7-6(a). The maximum inter-story drift from the second to the third story was greater than the responses of the initial undamaged building, which was thought to be due to changes in story stiffness and the accumulated structural damage at the beam ends caused by previous ground motions. For maximum story shear, the response of the retrofitted column base was 33% of that before the retrofit, indicating that the curved members subjected 67% of the story shear, and the demand of the original column base was reduced, as shown in Figure 7-6(b).

7.3 Proposed design methods for connections

This section presents the proposed design methodology for curved member connections based on the estimation equations and numerical results in Section 4.2 and the detailed design of the joints and intersection areas between the curved members and boundary beams, including welding details, connection plates, stiffeners, and lateral supports.

7.3.1 Modified design methodology for curved member connections

The common seismic design approach for lateral force-resisting systems (such as MRF, CBF, etc.) is primarily driven by Capacity Design principles, with specified requirements to define the inelastic system mechanism and to ensure a preferred hierarchy of strength is achieved to activate the desired plastic hinge regions of the structure. This means that capacity design forces are determined from an assumed plastic hinge forming at the ductile components such as beam ends and braces. Regarding requirements for connection design, the specification emphasizes that the load path and strength hierarchy within the connection must be designed to prevent inelastic demand from being concentrated in the connectors or connection components.

When applied to curved member connections, design methodology becomes particularly critical due to the additional complexities introduced by the curvatures of the members. Curved members addressed in this paper are placed vertically in the structure and serve as braces or retrofit members that experience a combination of axial, bending, and shear strength that can significantly influence the distribution of forces at the connections and surrounding members. For instance, the connection design approach of its straight counterpart, such as braces, is designed to withstand maximum forces and deformations imposed by braces during cyclic yielding or buckling, per AISC [19], and presents as following Eq. (7-1).

$$\Phi R_n \geq \gamma Q_u \quad (7-1)$$

where R_n is the nominal strength of the connection component, Φ is the resistance factor, Q_u is the required strength of the member, and γ is the overstrength factor to ensure adequate resistance.

In curved member design, connections must be capable of transferring forces efficiently while accommodating the strength imposed by the initial eccentricity of the member. However, designing connections

for curved members requires a more refined approach, as commonly used methods developed for straight members can be overly conservative. This conservatism arises because the ultimate strength (such as tensile strength, $N_{str,u}$, in tension and buckling strength, $N_{str,cr}$, in compression) assumed for straight members is significantly higher than that of curved members, leading to over-designed connections that do not efficiently utilize material capacity. As illustrated in Figure 7-7, the load-deformation behavior of curved members differs from that of their straight counterparts, with lower strengths due to initial eccentricity.

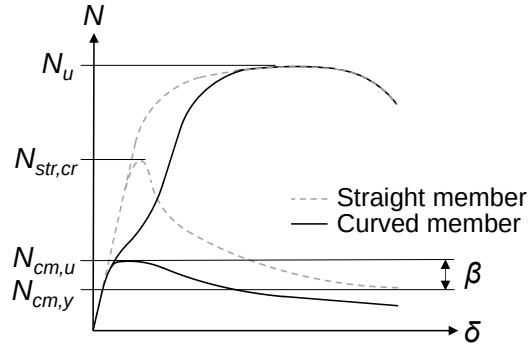


Figure 7-7 The strength capacity of straight and curved members.

To develop a more effective design methodology for connections, it is important to account for the inelastic behavior and ultimate strength of curved members. A modified design methodology has been proposed, incorporating a strength amplification factor (β) based on estimation equations developed in section 2 for the yield and ultimate strength of curved members, as shown in Eq. (7-2).

$$N_u \geq \beta N_{y,i} \quad (7-2)$$

where N_u is the required maximum strength of the connection. $N_{y,i}$ is the yield strength of the curved member, where i denotes the two boundary conditions of the curved member, e.g., pin ($N_{y,pin}$), fix ($N_{y,fix}$), and can be calculated by Eqs. (4-10) and (4-15).

The amplification factor (β) is the ratio of the ultimate and yield strength of curved members reflecting the effects of slenderness and buckling behavior and could be calculated upon Eqs. (4-22) to (4-28). These developed equations, derived from theoretical analysis and numerical simulation results, provide a more rational and efficient framework for connection design. By applying this amplification factor (β), the proposed modified design methodology ensures that the force is transmitted reliably and no local deformation occurs around the connection while avoiding unnecessary conservatism in the design.

For the design of curved-member connections, the required strengths are evaluated in terms of tensile, compressive, and flexural strengths. These demands are expressed as functions of the expected strength of the curved member. The governing design equations are summarized as follows:

$$N_c = R_y \beta N_{y,i} \quad (7-3)$$

$$N_t = R_t F_u A_t \quad (7-4)$$

$$M_{u,cm} = R_y \beta N_{y,i} e \quad (7-5)$$

where N_t and N_c are the required tensile and compressive strength of curved member connections of the member. A_t is the cross-sectional area on one side of the neutral axis that is subjected to tension. The equation for the required tensile strength (N_t) based on the experimental finding in Section 5 that the member has partially reached its tensile strength at the ultimate state. $M_{u,cm}$ is the required flexural strength of curved member connections of the member. R_y and R_t are the overstrength factors for yield and tensile strength, respectively. e is the eccentricity of the curved member.

7.3.2 Connection details

This chapter explains the design process related to connections associated with curved members in a steel structure. In the case of curved member connections, the required strength shall be the greater of the following: (1) the ultimate strength of the curved member; (2) the maximum load effect indicated by analysis that can be transferred to the curved member by the structural system. Considering the load path through the connection region, the Uniform Force Method, which assumes that the distribution of axial forces along the length of a member is uniform, can be used to determine internal forces and check the codified requirements at the curved member connection, as shown in Figure 7-8(a). The required flexural strength of curved member connections can be obtained by multiplying the eccentricity (e) and the required maximum strength ($N_{c,u}$). The θ is the angle between the shortest distance between the two ends of the curved member and the boundary beam at the ultimate state, and the direction of the force acting on the connection changes as the curved member deforms.

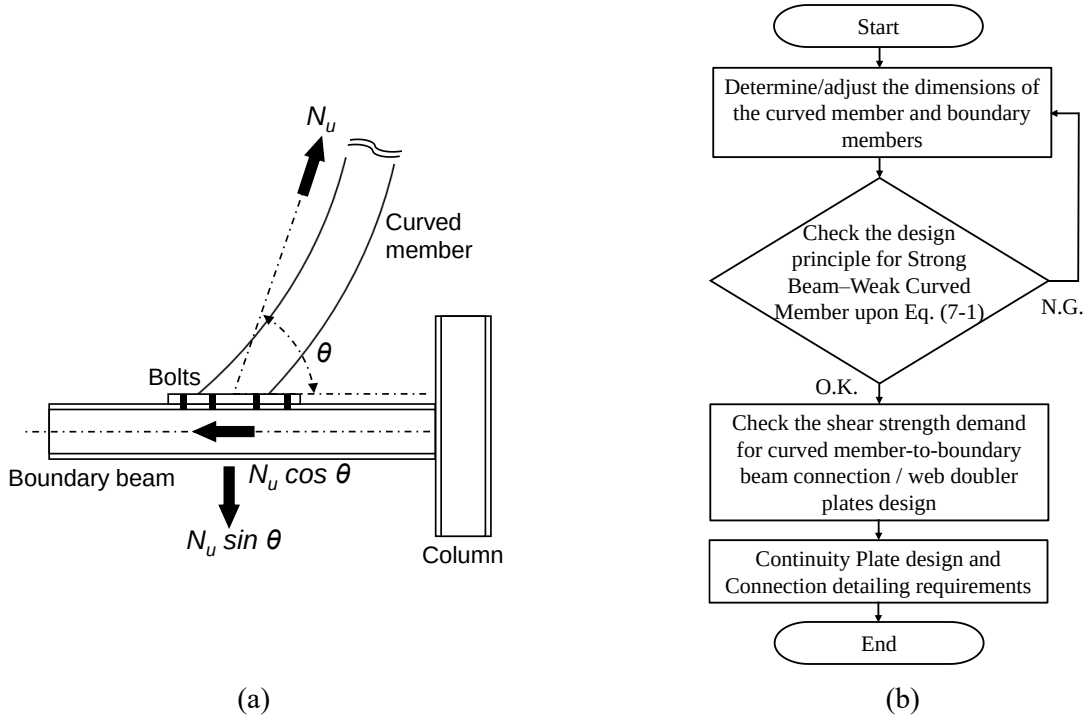


Figure 7-8 (a) The illustration of load path through curved member connection and (b) design flowchart.

The connection design flowchart is shown in Figure 7-8(b). The overall design process begins with determining the number and placement of curved members. Subsequently, the curved member and boundary beam member sizes are checked and designed sequentially. The process then proceeds to the detailed design of the connections and intersection areas between the curved members and boundary beams, including bolt details, continuity plates, and web doubler plates. For the detailed design of curved member connections, the AISC specification [7-4] for steel connections can be referred to and listed as follows.

- a. Proposed design principle for Strong Beam–Weak Curved Member in Curved Member-to-Boundary Beam Connections: when both ends of the boundary beam (near the column face) and the curved member reach plastic hinges, it must be ensured that the boundary beam on the side adjacent to the curved member remains elastic by using the proposed estimation equations and uniform force method.
- b. Shear Strength Demand for Curved Member-to-Boundary Beam Connections: The panel zone of the curved member and boundary beam connection must have sufficient shear strength to resist the required shear force, which arises from the moment in the boundary beam and the formation of plastic hinges in the curved member under load combinations. The required shear force should be calculated based on the total moment transferred to the boundary beam face when plastic hinges form in the adjacent curved members. If the shear strength of the panel zone is inadequate, web doubler plates are required.
- c. Connection Detailing Requirements for Curved Members and Boundary Beams: Several failure modes of the bolted connection need to be checked, including cross-sectional yield, net section fracture, and block shear rupture, to ensure the strength and stability of the connection.
- d. Continuity Plate Requirements at Curved Member–Boundary Beam Connections: The connection of the curved member and the boundary beam should be equipped with continuity plates at the extension position of the curved member flange when the thickness of the beam flange is less than either of the two following Eqs. (7-6) and (7-7):

$$t_{bf} < 0.4 \sqrt{1.8 b_{cmf} t_{cmf} \frac{F_{y,cm}}{F_{y,b}}} \quad (7-6)$$

$$t_{bf} < \frac{b_{cmf}}{6} \quad (7-7)$$

where t_{cmf} and t_{bf} are curved member and beam flange thickness respectively, b_{cmf} is curved member flange width, $F_{y,cm}$ and $F_{y,b}$ are the specified yield strength of the curved member and beam. Equations (7-6) and (7-7) are adapted from the AISC Seismic Provisions for continuity plate requirements. The same force-transfer concept was adopted for the curved-member connection by substituting the effective flange width and thickness at the curved member connection for the beam-flange terms, while retaining the AISC material factors.

- e. The foundation-side connection of the curved member is realized using an end plate anchored to the concrete foundation, which is designed refer to the AIJ recommendations for design of connections in

steel structures for exposed column bases. The required strengths including axial force, shear force, and end moment can be calculated by Equations (7-3) to (7-5). Notably, the foundation concrete shall have adequate strength to avoid failure by crushing or splitting of the concrete at the strength limit state of the column base. Anchor bolts shall be adequately anchored to the foundation concrete to prevent failure by pullout.

7.3.3 Comparison of required connection strengths

To demonstrate the applicability of the proposed design methodology, a worked example is presented comparing the required strength of a curved-member connection with that of a conventional capacity design approach for bracing connections.

The curved member has the geometric properties of $L_m = 6000$ mm, eccentricity $e = 600$ mm, and $H/L = 2.4$. The boundary constraint corresponds to the fixed end condition. The cross-section selected is an H-shaped section (200×200×8×12), which is seismically compact, with a yield strength of $F_y = 235$ MPa. Notably, the curved members were oriented that the weak axis of the section lay in the in-plane direction, while the strong axis lay in the out-of-plane direction.

According to AISC Seismic Provisions [19], the required strength of brace connections is based on the expected brace strength divided by α_s , where $\alpha_s = 1.0$ for LRFD and 1.5 for ASD. LRFD was adopted in this study. The governing expressions are given as follows.

$$P_t = R_y F_y A_g \quad (7-8)$$

$$P_c = 1.1 R_y F_{cr} A_g \quad (7-9)$$

$$M_{u,b} = 1.1 R_y F_y Z \quad (7-10)$$

where P_t and P_c are the required tensile and compressive strength of bracing connections of the members. $M_{u,b}$ is the required flexural strength of bracing connections of the member. A_g is the cross-sectional area of the brace member. F_{cr} is the buckling strength of the brace member. The overstrength factor of $R_y = 1.5$ was used, which is often treated as the closest ASTM counterpart (A36) to SS400 in yield level. Z is the plastic modulus of the cross-section. The calculated values in Table 4 represent the required connection demands under the AISC capacity design approach, which is intended to ensure that the connection is stronger than the member itself.

In the proposed method, the governing strengths of the curved member including required tensile and compressive strengths, and the required moment of curved-member connections were evaluated using Eqs (7-3), (7-4), and (7-5) and presented in Table 7-2. These equations incorporate the eccentricity effect and the redistribution of internal forces due to curvature.

As shown in Table 7-2, the proposed curved-member design substantially reduces the required connection strengths compared with the conventional AISC capacity design. The tensile demand decreases from 2188.3 kN to 1489.9 kN (a reduction of 32%), while the compressive demand decreases from 1448.2 kN to 120.5 kN (a reduction of 92%). The required moment is also reduced, from 94.2 kN·m to 72.3 kN·m (a reduction of 23%).

These results confirm that curved members can achieve significant reductions in connection demand, particularly in compression, due to the redistribution of forces and the effect of eccentricity discussed in Section 4. From a practical design standpoint, such reductions potentially translate into more economical connection details, including reduced bolt forces and lighter stiffening requirements.

Table 7-2 Comparison of required connection strengths between capacity design and proposed curved-member design.

	Tensile strength (kN)	Compressive strength (kN)	Moment (kN-m)
Capacity design (AISC)	2188.3	1448.2	94.2
Curved-member design	1489.9	120.5	72.3

7.4 Summary

This chapter presented a case study to evaluate the effectiveness of proposed repair and retrofit strategies for damaged steel structures, focusing on a four-story steel moment-resisting frame introduced in Chapter 2. A two-stage analysis procedure was adopted, beginning with static pushover analysis to assess the overall performance of the structure, followed by dynamic analysis using sequential and single ground motions to simulate earthquake-induced damage and assess post-intervention responses. Moreover, a modified connection design concept of curved members that avoids overly conservative connection designs to make efficient use of material capacity was suggested using established equations.

- (1) The first technique examined was a simple repair method involving the re-tightening of anchor rods. Based on the repairability flowchart in Chapter 3 and the remaining ductility capacity of the anchor rods, the simple repair was applicable for damage induced by DBE-level ground motions. The results showed that the simple repair method could rapidly restore the overall stiffness of the column bases. However, for more severe damage caused by MCE-level ground motions, the probability of failure of anchor rods exceeded the allowable limit, and the curved member retrofit was adopted.
- (2) The curved member retrofit system was applied following the design procedure established in Chapters 4 to 6 and was designed to restore at least 90% of the original column base moment capacity, and was implemented when a simple repair method is not feasible. Static pushover analysis showed that the retrofit improved the building's lateral stiffness and strength by approximately 19% and reduced the natural period by 6%, indicating enhanced structural performance without significantly increasing seismic demand on the main frame. Moreover, the space occupancy rate of the curved member retrofit system was only 13.2%, confirming its practicality as a low-disturbance retrofit solution.
- (3) Sequential time history analysis confirmed the effectiveness of the proposed retrofit technique. The first-story drift was reduced by 11.4% compared to the undamaged structure when subjected to MCE-level ground motion, and the story shear demand on the original column base was reduced by 67% due to load transfer to the curved members. These reductions demonstrate the retrofit's ability to reduce plastic deformation at critical components.

- (4) A modified connection design for curved members was suggested based on the developed estimation equations, incorporating a strength amplification factor (β) in the study to provide a rational and efficient approach for connection design while avoiding unnecessary conservatism.

Reference

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- [7-2] Architectural Institute of Japan. AIJ Recommendations for Design of Connections in Steel Structures. Tokyo: Maruzen; 2021. (in Japanese)
- [7-3] FEMA, “FEMA 273 NEHRP Guidelines for the Seismic Rehabilitation of Buildings” 1997.
- [7-4] AISC 360-16 (2016), Specification for Structural Steel Buildings, American Institute of Steel Construction; Chicago, Illinois, USA.

CHAPTER 8

CONCLUSIONS AND FUTURE STUDIES

8.1 Conclusions

This dissertation developed and validated low-disturbance strategies for the post-earthquake repair and retrofit of exposed-type steel column bases in steel structures. Beginning with a clarification of the current seismic vulnerabilities and limitations in conventional retrofit practices, the study first demonstrated that re-tightening of yielded anchor rods is a rapid repair strategy for structures adopting weak column bases. To examine the applicability of this simple repair method, the tensile tests of anchor rods and a series of dynamic analyses were performed, demonstrating its effectiveness in recovering the overall stiffness of damaged column bases and reducing inter-story drift demands in low-rise buildings after moderate damage. To support repair or retrofit decision-making, a residual performance evaluation method was proposed, using ductility-based capacity checks and probability of failure assessments. In parallel, a novel retrofit technique that leverages the advantages of curved members while minimizing disturbance to the existing building system was developed. The mechanical behavior of curved steel members was analytically studied, and estimation equations were developed to predict their initial stiffness, yield strength, and buckling strength under various boundary conditions. These findings enabled the design of the curved member retrofit technique, intended to provide ductile, moderate enhancement, and minimal disturbance to the retrofitted structures. The retrofit technique was experimentally validated at the component level and extended to subassemblies, where it showed improved lateral strength, stiffness, and deformation capacity, as well as a reduction of seismic demands away from critical components. Numerical simulations also indicated that detailing connection requirements such as continuity plates could be reduced. The proposed methods were then applied to a damaged four-story steel moment-resisting frame. Dynamic analyses confirmed that while the re-tightening method was suitable for moderate damage, the curved member retrofit could effectively restore structural performance after severe damage without increasing seismic demand. These contributions offer a practical solution for enhancing the post-earthquake reparability and resilience of steel structures with minimal disturbance. In this dissertation, the major findings are summarized as follows.

Chapter 2: Effectiveness of Simple Repair Technique for Steel Column Bases

This chapter presents an investigation into a standard repair strategy for damaged exposed-type column bases. The effectiveness of re-tightening anchor rods, a simple and rapid method intended to restore the overall

stiffness of exposed column bases following an earthquake, was evaluated through experimental testing and numerical simulations. A series of dynamic analyses were conducted on four- and six- and ten-story benchmark buildings using a discrete modeling approach through OpenSees frameworks to assess the applicability and effectiveness of this simple repair method. The major findings are summarized as follows.

- (1) The repair method is not always applicable for standard anchor rods assumed for pin or fixed column bases design to remain elastic. This chapter introduced and integrated the Japanese and foreign specifications for the required deformation capacity of ductile anchor rods.
- (2) The test data for threaded anchor rod deformation capacity with different steel grades and screw threads commonly used in Japan and abroad were prepared to verify the required deformation capacity of anchor rods through the tensile test and analyses.
- (3) A simulation method of replacing anchor rod elements in the entire structure life was established using OpenSees frameworks and validated by the shaking table test. It is shown that the numerical model generally enables the capture of frame system response including maximum story drift and acceleration, and the local behavior of column base specimens and beam-column connections in terms of initial stiffness and strength backbones.
- (4) A simple repair method of the re-tightening process of anchor rods was proven to be effective for low-rise buildings through the analysis that the maximum story drift on the first floor was reduced by 9.6% and 7.7% when the structure was subjected to subsequent ground motion. While the effectiveness of this repair method for mid- and high-rise buildings is not significant. However, the effectiveness of this simple repair method will be affected by the contribution of column bases in the system, different types of exposed column bases, and the story height of the building.

Chapter 3: Proposed Retrofit/Post-Earthquake Decision Making Methodology

Chapter 3 presents a decision-making methodology for the post-earthquake repair and retrofit of exposed steel column bases. Building on the findings from Chapter 2, a performance-based framework was developed to evaluate the applicability of a simple repair method involving the re-tightening of anchor rods, aimed at rapidly restoring the overall stiffness of column bases and reduce the risk of further damage during aftershocks. The methodology was established using structural reliability theory, incorporating inspection procedures, ductility evaluation, and failure probability calculations. Dynamic analyses and experimental data were utilized to verify the feasibility of the repair method for various building scales. The major findings are summarized as follows.

- (1) A repair decision-making flowchart was proposed to evaluate the damage condition of anchor rods and determine appropriate repair or retrofit actions. The flowchart includes steps for visual inspection, evaluation of plastic deformation, estimation of required and remaining ductility, and the use of reliability theory to assess the probability of failure.
- (2) The residual deformation capacity of anchor rods after each ground motion is examined by dynamic analysis and test results to check the applicability for adopting this repair method. A reliability-based methodology was adopted to calculate the probability of anchor rod failure. When the probability of failure

was less than 0.05, the simple repair method was considered acceptable. Conversely, if the probability exceeded the limit, the anchor rods were deemed insufficient for future seismic demands.

- (3) When the simple repair method is no longer feasible due to excessive damage or low remaining ductility, other retrofit options must be considered. In this study, a retrofit technique using curved members was proposed as an alternative to ensure the required performance of damaged column bases. The details and evaluation of this retrofit system will be introduced in the next chapter.

Chapter 4: Plastic behavior and design Method for curved members in seismic applications

Chapter 4 investigates the plastic behavior of curved members through parametric studies covering a wide variation of dimensions and boundary conditions, followed by the development of estimation equations for plastic strengths, initial elastic stiffness, and the influence of member slenderness and compactness. Two phases of parametric studies regressively quantified the effects of fixed boundary conditions and member slenderness on the behavior of curved members. Finally, this chapter evaluates the impact of boundary constraint conditions on the behavior of curved members. It identifies a boundary condition that effectively controls the yielding mechanism and ensures the desired plastic behavior. Based on the analytical work in the study, several remarks can be drawn as follows:

- (1) A set of equations for estimating the initial elastic stiffness and yield strength of curved members under pinned constraint conditions was developed using the virtual work principle and elastic beam-column theory, and the calculated results obtained by the two methods are consistent. The developed equations were validated through numerical modeling and demonstrated high accuracy when compared to analytical results, ensuring their applicability in practical design scenarios.
- (2) The effect of slenderness on the behavior of curved members was found that curved members with different slenderness ratios exhibit inelastic buckling behavior after the yielding, and the buckling strength exceeds the yield strength when the curved member has a smaller slenderness ratio. Additionally, as the slenderness ratio increases, the buckling strength gradually converges toward the yield strength. The estimation equations were established according to a parametric study using the finite element models that the slenderness ratio range of 20 to 120 applies Eq. (4-20) to (4-26) for estimating buckling strength, while ratios above 120 use Eq. (4-19) with a reduction coefficient ζ for yield strength estimation.
- (3) The effect of section compactness on the behavior of curved members was verified, and compact and noncompact sections are dominated by inelastic global buckling due to initial eccentricity. For curved members with slender sections, local buckling is observed in the analysis for those with relatively small initial eccentricity. The analytical results demonstrate that the compact and noncompact sections of curved members prevent local buckling, and it is recommended that the compact section be selected when adopting the curved members.
- (4) An effective boundary constraint condition that achieves the desired behavior and controls the yielding mechanism that plastic hinge occurs first in the middle of the curved member was suggested by adopting a recommended factor ($\gamma_{m,c}$) of 0.6, which represents the degree of fixity at the connections.
- (5) The bidirectional behavior of curved member retrofits was investigated using numerical simulations. By

adjusting the ratio of I_x/I_y and locating the weak axis of the curved member section in the in-plane direction, out-of-plane strength improved, achieving more balanced performance. This confirms the importance of section shape and orientation in optimizing retrofit behavior under multidirectional seismic loads.

Chapter 5: Development of Component-Level Retrofit Using Curved Members

A retrofit system utilizing curved members has been proposed in this section, aiming to achieve moderate-enhanced strength, stiffness, and improved ductility compared with the common retrofit method for column base. The basic behavior, performance, and design targets of the proposed retrofit system were analytically examined by pre-analysis. Four curved member specimens have been tested focusing on the behavior of the specimens. The effects of different fabrication processes and boundary conditions on the behavior of the specimens were investigated. Moreover, a numerical model of the curved member retrofit system was developed and validated by the experimental results. Based on the analytical and experimental investigations presented in this section, the following key findings are drawn:

- (1) The proposed curved member retrofit system has been analytically verified to achieve design targets including (a) moderate-enhanced strength, stiffness, and improved ductility; (b) enlarged plastic rotation capacity to reduce the seismic demand for the column base.
- (2) According to numerical simulations, the behavior of the proposed curved member retrofit system was verified as follows. Improved ductility of the retrofitted column base was achieved by the enlarged plastic region with plastic strength corresponding to the early yielding of the curved member and the delayed yielding of the anchor rods at the original column base. The drift angle of anchor rods yielding at the retrofitted column base has been increased by 36% compared with the original column base. It indicates that the proposed retrofit system enables to enlarge the plastic rotation capacity of the original column base and reduces its seismic demand.
- (3) A set of equations for estimating the lateral load capacities of curved member retrofit system were established based on the theoretical equations in Section 4.2 and verified by developed numerical models with a good agreement and enabled to accurately represent the measured responses in the test.
- (4) A prototype design process of the proposed curved member retrofit system based on the numerical simulation in this section was provided to achieve the design targets.
- (5) The mechanics of induction-heating and cold-formed bent curved channel steel have been experimentally verified. It is shown that induction-heating bent specimens provided a stable cyclic behavior with limited strength degradation and controllable yielding region distributed around the middle of the curved member. However, the initial distortion and poor precision of the cold-formed bending specimen in this test likely resulted in early local buckling, strength degradation, and difficult installation. It should be noted that the fabrication accuracy depends on the fabricator and the cold-formed bending member can also be adopted in the proposed retrofit system if fabrication accuracy is met.
- (6) The end-plate slipping of IHB-M occurred in the cycles of 0.5% story drift but had no influence on the behavior such as severe strength degradation and pinching. The local buckling occurred in the cycles of 4.0% drift angle, no strength degradation was observed throughout the loading protocol, implying the

feasibility in practical applications.

Chapter 6: Development of Subassembly Retrofit Using Curved Members

This chapter presented a comprehensive experimental and analytical evaluation of a seismic retrofit technique employing curved members at the column base and beam-column joint in steel moment-resisting frames. A force-based design method was developed to determine the required retrofit capacities, and its applicability was verified through quasi-static testing. Furthermore, a general numerical modeling approach for curved member retrofit was validated against the experimental results, and the necessity of continuity plates was discussed. Key findings and conclusions are summarized as follows:

- (1) The post-retrofit frame in the study was experimentally verified to exhibit stable and ductile hysteretic behavior, with early yielding of curved members and no strength degradation up to a 6% story drift. Local buckling occurred under cycles of 4% story drifts, and did not lead to post-yield stiffness of the specimen, due to the effectively redistributed forces of curved members, even after the beam ends fractured.
- (2) The test results showed that the curved member retrofit delayed yielding of the column base by reducing the rotation demand by 28%. Additionally, the retrofit reduced beam end moments demand by an average of 19%, and the yielding region along the beams formed a uniform stress distribution compared to the pre-retrofit frame. The curved member retrofit successfully shifted plastic hinge locations away from the beam ends.
- (3) A force-based design methodology was established using estimation equations for lateral strength and stiffness and validated against the experimental results. The target retrofit strength was defined as a 40% increase over the pre-retrofit frame in the study, determined through the superposition principle. The post-retrofit frame achieved a 44% in yield strength, a 59% increase in ultimate strength, and an 81% increase in initial stiffness, compared to the pre-retrofit frame.
- (4) The structural ductility at multiple limit states was evaluated. The ductility factor μ , defined as the ratio of ultimate to yield drift, increased from 4.0 to 8.0 for the overall frame system. At the component level, ductility ratios for the column base and beam-column connections improved from 1.0 to 1.7 and from 4.0 to 6.0, respectively.
- (5) A discrete modelling approach was developed to effectively and accurately represent the responses of the retrofitted frame before local buckling, which could be used for performing analysis on various retrofit configurations.
- (6) The finite element analyses and test observations demonstrated that continuity plates at the curved member-to-beam connections were potentially not essential; after removal, the connection exhibited no vertical deformation or strain concentration and maintained stable behavior at cycles of 6% story drifts.

Chapter 7: Application of Proposed Methodology

This chapter presented a case study to evaluate the effectiveness of proposed repair and retrofit strategies for damaged steel structures, focusing on a four-story steel moment-resisting frame introduced in Chapter 2. A two-stage analysis procedure was adopted, beginning with static pushover analysis to assess the overall

performance of the structure, followed by dynamic analysis using sequential and single ground motions to simulate earthquake-induced damage and assess post-intervention responses. Moreover, a modified connection design concept of curved members that avoids overly conservative connection designs to make efficient use of material capacity was suggested using established equations.

- (1) The first technique examined was a simple repair method involving the re-tightening of anchor rods. Based on the repairability flowchart in Chapter 3 and the remaining ductility capacity of the anchor rods, it was determined that this approach is appropriate for damage induced by DBE-level ground motions. The results showed that the simple repair method could rapidly restore the overall stiffness of the column bases. However, for more severe damage caused by MCE-level ground motions, the probability of failure of anchor rods exceeded the allowable limit, and the curved member retrofit was adopted.
- (2) The curved member retrofit system was applied following the design procedure established in Chapters 4 to 6 and was designed to restore at least 90% of the original column base moment capacity, and was implemented when a simple repair method is not feasible. Static pushover analysis showed that the retrofit improved the building's lateral stiffness and strength by approximately 19% and reduced the natural period by 6%, indicating enhanced structural performance without significantly increasing seismic demand on the main frame. In addition, the space occupancy rate of the curved member retrofit system was only 13.2%, confirming its practicality as a low-disturbance retrofit solution.
- (3) Sequential time history analysis confirmed the effectiveness of the proposed retrofit technique. The first-story drift was reduced by 11.4% compared to the undamaged structure when subjected to MCE-level ground motion, and the story shear demand on the original column base was reduced by 67% due to load transfer to the curved members. These reductions demonstrate the retrofit's ability to reduce plastic deformation at critical components.
- (4) A modified connection design for curved members was suggested based on the developed estimation equations incorporating a strength amplification factor (β) in the study to provide a rational and efficient approach for connection design while avoiding unnecessary conservatism.

8.2 Future studies

Several promising research directions emerged from this study and can be further explored in future work. While the proposed repair and retrofit strategies were validated through experimental testing and numerical simulations, additional investigations are needed to broaden their applicability and support practical implementation.

(1) Influence of Combined Axial Force and Bending Moment on Anchor Rod Ductility

In the present study the anchor rods were tested under monotonic tension only. We recognize that bending caused by base plate rotation can affect behavior and ductility at the large column base rotation. The data of anchor rods under combined tension and bending with cyclic loading is needed for the ultimate capacity reduction and fracture assessment.

(2) Assessment Guideline to Reduce Demolition

Develop a practical assessment guideline that reduces demolition by prioritizing simple repair and targeted retrofit when safety and function can be restored. The guideline will provide a clear workflow from rapid triage to detailed evaluation, list repair and retrofit options including curved member retrofits, and apply a reliability-based decision process with target performance.

(3) Effects of Retrofit Arrangement and Concentration retrofit

The impact of different retrofit configurations, including the number and placement of curved members, should be further examined to better understand their effect on the global structural response. Optimizing the distribution of retrofit components may enhance structural efficiency while reducing construction complexity and minimizing spatial intrusion.

(4) Concentrated Retrofit Strategy

An alternative approach worth exploring is the use of concentrated retrofit strategies, where only selected columns are retrofitted using relatively strong curved members. This targeted reinforcement could offer construction and cost efficiency but may alter the failure mechanism of the system. Such changes could shift plastic hinge formation to other members or influence the sequence of damage. Therefore, a new design methodology tailored to concentrated retrofit conditions is required, including criteria for selecting retrofit locations and ensuring system-wide ductile behavior.

(5) Bidirectional and Multi-Directional Seismic Effects

Although initial analyses have addressed the bidirectional performance of the curved member retrofit system, further studies are needed to evaluate its effectiveness under simultaneous multi-directional seismic inputs. These include horizontal components in different directions as well as vertical motions and torsional effects.

(6) Connection Design for Practical Applications

The foundation side connection of curved members remains unclear. Future research should experimentally investigate the local behavior of curved member connections and optimize their detailing with practical conditions. Priorities include cyclic testing of endplate/anchor assemblies, assessment of anchor and concrete limit states under combined axial force, shear force, and bending moment, and verification of grout and bearing performance, with results used to calibrate and refine the proposed design method.

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