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Large-Deformation Behavior and Failure Process of Steel Moment-Resisting Frames Examined by a Shake-Table Test

by

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ABSTRACT

Steel moment-resisting frames are popular structural systems in seismic areas. However, seismic excitations produce large deformations that eventually lead to member damage, degradation of strength and stiffness, and collapse. The structural damage changes the vibration properties and therefore the response structural response. Very few studies have been conducted in the past to examine the behavior of steel moment-resisting frames to very large deformations, i.e., story drift ratios greater than 0.10 rad. Furthermore, ductile moment-resisting beam-to-column connections are important to achieve good performance against earthquakes. This research studies the behavior of steel moment-resisting frames through cyclic static tests on beam-to column moment connection subassemblages (component-level response) and dynamic shake-table tests of a scaled moment-resisting frame (system-level response). The beam-to-column connections of the subassemblages and shake-table tests use the current detailing of the Japanese construction practice.

Six beam-to-column moment connections that used an I-section column were tested under cyclic loading. I-section columns were chosen instead of the typical square box-section columns to examine the effect of the current Japanese welding details and fabrication in moment connections to the flange and to the web of the column. The T-configuration of the specimen represented an external beam-to-column subassembly of two to four-story steel moment frame buildings in Japan. The specimens to cyclic loading according to the protocol specified in Section K2 of AISC Seismic Provisions. Three specimens were for moment connection to the column flange and three for moment connection to the column web. Complete Joint Penetration (CJP) groove welds were used to connect the beam flanges to the column. In all cases, the CJP welds were completed with no weld access holes. The tests showed that beam-to-column web connections were prone to fracture of continuity plates initiating at the corner between the beam flange and the continuity plate. This failure mode was promoted by yielding in the continuity plates. The shop-welded, no-access-hole detail for beam-to-column moment-resisting connections, which is commonly adopted in Japan, is a detail not included in the ANSI/AISC 358 but met ductility requirements for the highly ductile Special Moment Frames per the AISC Seismic Provisions. Furthermore, the quality of the welding process, in combination with having the beam web directly welded to the column web and the use of a stronger continuity plate were important factors in achieving a good seismic performance of the minor-axis moment connections even when continuity plates flush to the column flanges were used. The same no-weld access hole detail was used in the shake-table test specimen, but with the connection to square-HSS columns.

Shake-table tests of a steel moment-resisting frame were conducted to obtain detailed data on the occurrence and propagation of damage leading to the collapse of steel structures. A 2/5-scale, 4-story, 2-bay, planar specimen was subjected to in-plane, unidirectional ground motions. The specimen was intended to sustain large deformations, or story drift ratio greater than 0.10 rad, and avoid a first-story collapse mechanism. The specimen comprised I-section beams and square-HSS columns. The CJP groove welds between the beam flanges and column diaphragm plates were completed with no weld-access-hole. The ground motions included the JMA Kobe NS record scaled from 10 to 100%, and white noise excitations to trace the change in vibration properties between primary excitations. The 100%-scale record was repeated four times to produce as much deformation and damage as possible. The frame deformation response, change in internal load distribution, and acceleration were measured with a combination of displacement transducers, strain gages and accelerometers, respectively. Each of the 100% motions induced significant yielding of the beams, columns, and some column panels, which resulted in story drift of +0.08 and -0.02 rad, and added a residual story drift of +0.02 rad. As expected by design, a sidesway mechanism involving the first three stories formed during the first 100% excitation. Beam fracture occurred at two locations on the second floor, and the maximum story drift reached +0.15 rad during the last 100% excitation. The fundamental period remained constant until beam fracture occurred. Despite the widespread damage and fracture of the second-floor beams, the specimen did not collapse due to the high strength of the columns, which had 1.8 times the nominal strength.

Three supplementary cyclic-loading tests were conducted on beam-column subassemblages constructed

from the same materials and fabrication process as the shake-table specimen. The subassemblages represented one interior and two exterior connections of the shake-table test specimen. The loading protocol followed the AISC loading protocol for beam-to-column moment connections up to 0.05-rad story drift, but it was changed to a ratchetting protocol up to 0.125 rad afterward to emulate the observed beam-to-column large deformation response in the shake-table tests. However, the subassemblages exhibited more rapid development of local buckling deformation and associated strength degradation.

Comparison of the shake-table and component test results showed good agreement in strength capacity at the beam-to-column connection level but differed in terms of degradation. The difference was attributed to axial deformation restraint in the shake-table test beams, evidenced by smaller beam contractions than in the component test beams. Additionally, the inertia forces imposed on the beams of the shake-table test specimen axial loads that were not present in the component tests. The axial loads synchronized with the bending moments to increase the demands in the beam bottom flange. For this reason, fracture occurred in the bottom flange of the shake-table test beams. The subassemblage test beams, which were subject to axial-free bending moment, fractured at either the top or bottom flange.

The size effect of the shake-table test was examined by comparing the subassemblage test beam response with full-scale beam-to-column connection tests that followed a similar fabrication practice and load protocol. The weld detail and fabrication used in this study was found to not affect the test outcome. The larger length-to-depth ratio of the scaled specimen, however, reduced the maximum bending strength of the beam with respect to full-scale specimens.

The major contributions of this research are: first, the obtention of seismic response data of a steel moment-resisting frame that experienced deformations up to 0.15-rad, without premature collapse. The data can be used to develop analysis schemes for large-deformation behavior prediction. Second, the comparison with subassemblage tests showed that in order to predict the global behavior with component tests, the axial restraint and moment redistribution present in the steel moment frame must be addressed. Third, even though small cracks are likely to happen, the current Japanese fabrication of welded beam-to-column moment connections ensures ductile behavior in either beam-and-column subassemblages or complete moment-resisting frames.

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1. INTRODUCTION

1.1 GENERAL

Moment-resisting frames are popular as lateral-load resisting systems in seismic areas due to their high ductility and the possibility to have unobstructed interior spaces in the buildings. Figure 1.1 shows an example of the construction of an office steel building that used moment-resisting frames. The beams of steel moment-resisting frames are generally I-section members. The columns may be I-section, as typically in the U.S., or square-HSS tubes, as in Japan.

The most critical part of moment-resisting frames is the beam-to-column connection. Studies during the 70's and 80's promoted the use of welded beam-to-column connections to ensure good performance during earthquakes. However, the generalized damage to welded moment connections during the 1994 Northridge and the 1995 Kobe earthquake motivated an extensive research program to examine the cause of the observed failures and to improve the connection details for seismic applications. Because of those studies, the design specifications for steel moment-resisting frames were updated in the US. Improved moment-resistant connection details like the Reduced-Beam-Section (RBS) were introduced to move the formation of plastic hinges away from the face of the column, therefore reducing the demands to the welds and bolts. Also, the quality of the weld was improved and stricter requirements in toughness were enforced. In Japan, improved geometry of the weld access hole (known as “scallop” in the Japanese literature) and even the absence of such access hole was proposed after numerous tests on beam-to-column moment connections. Figure 1.2 shows an example of a typical beam-to-column moment connections in Japan, used in low- and mid-rise buildings. This connection configuration is known as “column-tree”.

By the same time, the increasing interest on Performance-Based Design encouraged the study of the inelastic behavior of moment frames subjected to strong ground motions. This methodology, however, requires extensive experimental data to calibrate the numerical models used to predict the inelastic response of a wide variety of moment frames and the causes and processes leading to the collapse of steel moment-resisting frame buildings. Given the difficulty of conducting large-scale tests of complete moment-resisting frame structures and furthermore, subjected to dynamic loading, most tests were based on simplified models subjected to static cyclic loading. Nevertheless, such studies helped to identify key factors leading to the collapse of steel moment-resisting frames. The inauguration of the E-Defense shake table in Hyogo Prefecture, Japan, opened the possibility to conduct studies on full-scale building systems subjected to earthquake loading. International collaborations have used the facilities to obtain valuable data to investigate the collapse of different kind of structures and systems, including some steel moment-resisting buildings. However, few tests were conducted to very large deformations, or story drift ratio of about 0.10-rad, before entering incipient collapse.

This dissertation presents the experimental program and results of two main issues. Because of the absence of research on moment connections to the web of an I-section column, and noting the importance of the steel moment-to column connections in the overall performance of steel moment-resisting frames, the first issue studied is the seismic performance of steel beam-to-column moment connections connected to the flange or web an I-section column. The second and principal issue corresponds to shake-table tests of a scaled steel moment-resisting frame conducted to obtain detailed data on the occurrence and propagation of damage leading to the collapse of steel structures. The specimen was intended to sustain large deformations, or story drift ratio greater than 0.10 rad, and avoid a first-story collapse mechanism. All the test specimens of both issues were fabricated according to the current Japanese practice in design, fabrication, and detailing.



Figure 1.1 A steel moment-resisting frame recently built in Sapporo, Japan



Figure 1.2 Example of beam-to-column connection in a small building.

1.2 OBJECTIVES

The primary objectives of the first issue presented in this research were:

- To understand the seismic performance of the Japanese welded moment connections when an I-section column is used.
- To fill the knowledge gap in the seismic performance of steel beam-to-column moment connections either to the flange or web of an I-section column detailed and constructed according to the current Japanese practice.

The primary objectives of the second issue were:

- To obtain detailed data on the seismic response of steel moment-resisting frames to large deformations, or story drift ratios greater than 0.10 rad, without failing prematurely by a single-story collapse mechanism.
- To trace the effects of initiation and propagation of structural damage into the global seismic response of the structure.
- To verify the seismic performance of the welded moment connection using the no-weld-access-hole detail within a complete moment frame system.
- To complement the dynamic inelastic behavior of a steel moment-resisting frame with cyclic-loading tests on beam-column subassemblages representative of the moment frame system.

1.3 OUTLINE OF THE DISSERTATION

In order to address the research objectives, this dissertation describes the experimental procedure, data processing and analysis of the results divided in the following chapters. The first issue of this study, mentioned in Sec. 1.1, is covered in Chapter 3, while the second issue in Chapters 4 to 7.

Chapter 2 summarizes the historical development of steel moment-resisting frames in the US and Japan. Particular attention is given to the beam-to-column moment connection detailing. The second part of the chapter presents the experimental studies conducted in the last two decades on the seismic performance of steel moment-resisting frames.

Chapter 3 describes the experimental and numerical study of six beam-to-column connection tests to an I-section column. Emphasis is placed on the connection details and the description of the fabrication details because they are regarded as key factors affecting the seismic performance of the connections. The direct tests observations of the tests are supplemented by photos and the response plots from measured data. The chapter also discusses the experimental and numerical results, comparing the observed response with previous tests in the literature.

Chapter 4 details the experimental program of the shake-table tests. A description of the design, detailing, fabrication and expected performance of the Specimen is presented. The loading sequence and selection of ground motion are also described. The measurement plan and the equations used to compute the Specimen's response are presented with the adopted channel numbering and the on-site measurements.

Chapter 5 presents the shake-table test results. The verification of the measuring scheme is presented by comparing key responses measured from independent sources. The trace of the vibration properties of the system between primary loadings is also presented. The global response of the system is presented chronologically, indicating the occurrence and propagation of structural damage throughout the loading sequence. Then, the behavior and observed phenomena of the columns, beams, and panel zones are presented separately.

Chapter 6 describes the cyclic-loading tests on three beam-column components representative of the shake-table test specimen interior and exterior beam-column subassemblages. In a similar way to Chapter 4, the test setup, loading plan, measurement scheme, and expected behavior are presented. The data reduction equations that are exclusive of the component tests are described, and the ones identical to the shake-table tests are referenced to Chapter 4. The tests results are presented separately for each Specimen using the measured load-deformation response plots and photos illustrating the observed phenomena and failure mode.

Chapter 7 compares the results between the shake-table and the component tests. The differences and similarities are described, and the causes of the differences are examined. The size effect of the reduced-scale specimens is evaluated by comparison with full-scale beams of similar proportions and fabrication details. The key results the comparisons are summarized.

Chapter 8 consolidates the primary results and findings of the previous chapters. The main conclusions and recommendations are listed.

2. RESEARCH BACKGROUND

2.1 GENERAL

This chapter summarizes the historical development of steel moment-resisting frames in the U.S. and Japan. Particular attention is given to the beam-to-column moment connection detailing. The second part of the chapter presents the experimental studies conducted in the last two decades on the seismic performance of steel moment-resisting frames.

2.2 STEEL MOMENT-RESISTING FRAMES

2.2.1 Description

Moment-resisting frames are a structural system intended to provide lateral strength and resistance through an assembly of beams and columns. The system is popular choice because it does not need braces that can obstruct the view or architectural planning.

The beams are designed primarily to resist moment, but the columns for a combination of axial compression and bending moment. The beam-to-column connection must be adequate to transmit the bending moments and shear from the beam ends to the columns. For seismic applications, the beam-to-column connection must provide sufficient ductility. The U.S., for example, has several connection details that are rated for high ductility in the ANSI/AISC 358 Prequalified Connections standard [1]. Japanese examples of beam-to-column moment connection details can be found in [2].

2.2.2 Historical Development

Steel moment frames were widely used in the United States from the 1920s, and in the following five decades, moment-resisting connections were placed in both directions of the I-section column [3]. During the 1970s, steel moment frames in Japan and the US started to adopt the pre-Northridge moment connection detail. Following the detail, the flanges were joined to the column by Complete Joint Penetration (CJP) groove welds using flux core metal arc welding (FCAW), the backing bars were left in place, and the web of the beam was connected to the column with a bolted shear plate. To allow the backing bar placement, the portion of the beam web connecting to the beam flange was removed, conforming the weld access hole, as is called in the west, or “scallop”, as is known in Japan.

The construction practice of both countries started to differ from the 1980s, when moment connections were formed only with respect to the flange of the column, conforming moment frames in one direction only (as shown, for example, in Figure 3.1b), while Japan kept conforming moment frames in both directions due to the introduction of square box-section columns (Figure 3.1a). The most popular construction method for low- and mid-rise steel buildings in Japan is the “column-tree” construction, as shown in Figure 3.1a, in which the moment connections are done in the shop, connecting only one short segment of the beam to the column and then the assembly is transported to the construction site. Once in place, the moment frame is completed by connecting the remaining beam portions through bolted splices.

2.3 EXPERIMENTAL STUDIES ON BEAM-TO-COLUMN MOMENT CONNECTIONS

I-section columns are less commonly used in Japanese steel construction than square-tube (hollow structural sections or square built-up) columns because Japanese construction tends to adopt moment-resisting connections at all beam-to-column connections regardless of the shape or size of the column. As a result, there are a significant number of low-rise buildings where I-section columns comprise moment-resisting frames in both directions, having moment-resisting beam-to-column connections formed to the column web as well as the column flange. Because research in Japan has focused on moment connections to square tube columns, there are no recent studies on moment connections to I-section columns, either to the column flange or web. To the knowledge of the author, the only few studies

in Japan that used I-section columns can be seen in the references [4-8].

On the other hand, construction in the U.S. forms moment-resisting frames in one direction of the column. For this reason, I-section columns are preferred because of the better use of their stiffness and strength when the column is subjected to major-axis bending. Moment connections to the column web, subjecting the column to minor-axis bending, are less preferred and have drawn limited research interest. The limited research on moment connections to the column web was mostly conducted prior to the 1994 Northridge earthquake. Figure 2.1 illustrates the common features of the beam-to-column web connections tested in the past in the U.S.: the beam flange was connected to the column web through a continuity plate using Complete-Joint-Penetration (CJP) groove welds, while the beam web was connected to a vertical shear plate by bolts, welds, or a combination of both. As in the construction practice in the U.S., these connections were intended for field welding.

To investigate the cyclic loading response of steel beam-to-column moment connections, Popov and Pinkney [9] tested twenty-three moment connections under cyclic loading including six connections to the column web. Figure 2.1a shows an example of a beam-to-column web connection that experienced premature weld fracture. Poor weld quality and deficient workmanship were identified as the causes of such undesirable behavior. The connections that followed the detail shown in Figure 2.1b, on the other hand, sustained substantial plastic deformation. That research noted that beam-to-column web connections are more prone to fracture than beam-to-column flange connections.

Other notable early tests on beam-to-column web moment connections were conducted at Lehigh University by Rentschler et al. [10]. Four beam-to-column web connections were evaluated through monotonic-loading tests. Figure 2.1c shows an example of the specimens that fractured in the continuity plate in tension. Other two similar connections that had the beam web connected to the column through a shear plate, either by fillet welds or bolts, also showed a similar fracture pattern. The observed fracture initiated either at the termination of the fillet weld to the column flange near the edge of the column flange, or at the edge of the CJP groove weld to the beam flange. Figure 2.1d shows the fourth specimen, which used CJP groove welds to connect the beam web to the shear plate. Contrary to the first three specimens that had the beam web bolted or fillet-welded, the fourth specimen survived inelastic deformations of 0.05 rad without any visible damage. Based on the tests by Rentschler et al., Driscoll and Beedle [11] recommended continuity plates thicker than the beam flange, or the use of extended and tapered continuity plates beyond the column flange to avoid the observed brittle fracture.

Cyclic-loading tests of nineteen steel moment connections were conducted by Tsai and Popov [12], including four beam-to-column web connections. Figure 2.1e details two of the beam-to-column web connections that had two vertical stiffeners attached to the beam flanges, and the bolted web connection reinforced using fillet welds. This detail controlled the slippage of the bolted web connection and fracture of the beam flange groove welds. In consequence, the beam developed a rotation of 0.05 rad with no strength degradation.

After the 1994 Northridge earthquake, the interest in beam-to-column web connections diminished. The most recent was conducted by Gilton and Uang [13], who tested two large beam-to-column web connections that adopted the Reduced Beam Section (RBS) according to the detail shown in Figure 2.1f. The experimental results, supplemented by finite element method analysis, showed that the RBS protected the critical CJP groove welds between the beam flange and continuity plate from fracture.

In Japan, limited research data is available for moment connections to I-section columns, whether to the column flange or column web. One reason is that Japanese construction tends to make the moment connections in the fabrication shop, whereas construction in the U.S. is intended for field welding. As a result, the details of beam-to-column moment connections to an I-section column in Japan differ significantly from the prequalified connections used in the U.S. and listed in the ANSI/AISC 358 [1]. The detailing used in Japan varies from shop to shop because there are no design guidelines or research that provides an understanding of the connections to an I-section column.

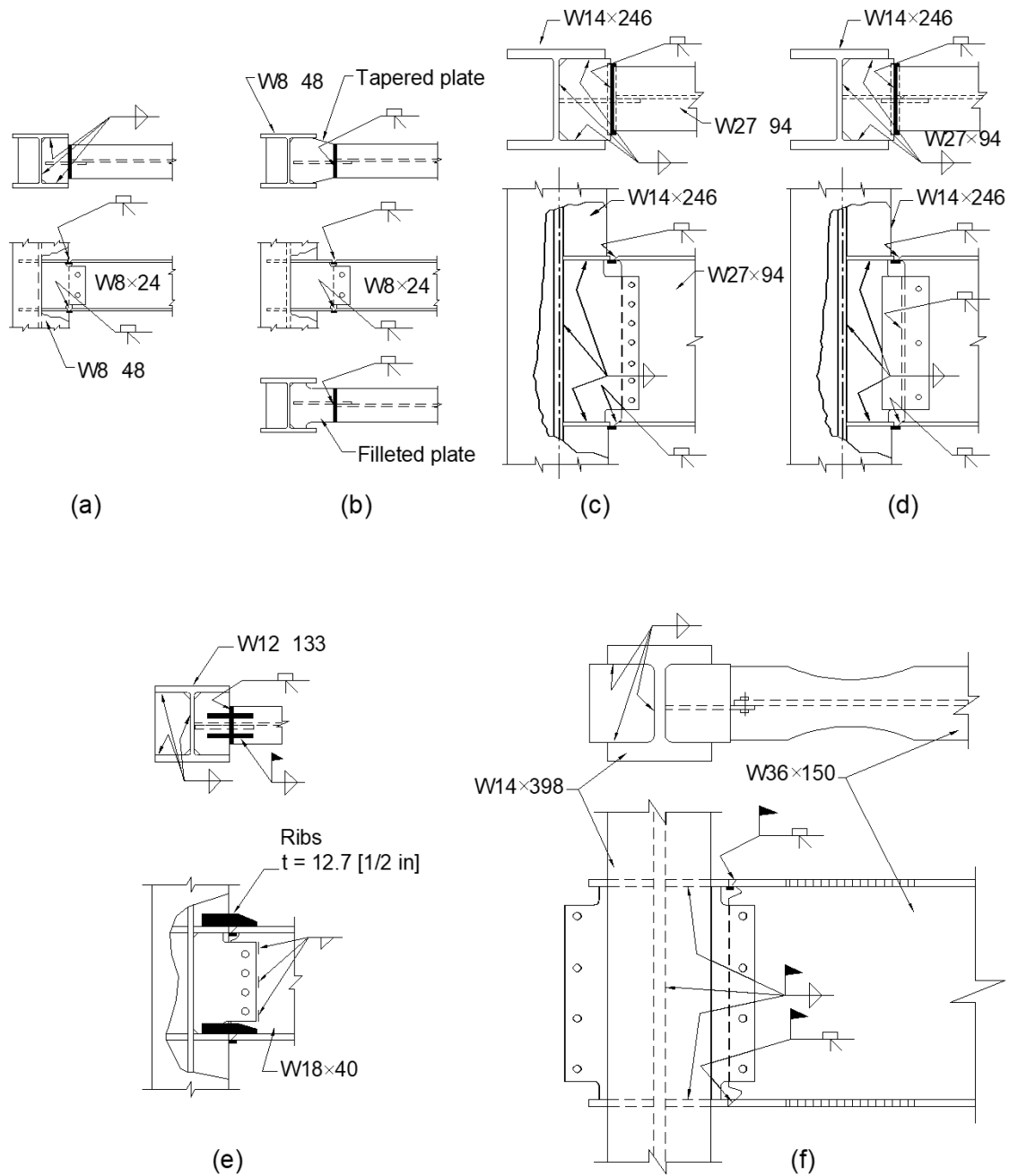


Figure 2.1 Details of beam-to-column web moment connections tested in the U.S. by a-b) Popov and Pinkney [9], c-d) Rentschler et al. [10], e) Tsai and Popov [12], and f) Gilton and Uang [13]. Drawn to scale using the information provided in the cited literature.

2.4 STUDIES ON THE COLLAPSE OF STEEL MOMENT-RESISTING FRAMES

2.4.1 Static Tests

The increasing popularity of performance-based design toward the end of the 20th and beginning of the 21st century motivated the obtention of real data on the structural behavior of structures subjected to large deformations. Since dynamic testing of real building structures is complex, costly, and rare, some researchers opted for quasi-static cyclic-loading tests. As an example, Matsumiya et al. [14] conducted cyclic-loading tests of a full-scale, 3-story, 2×1-bay steel moment-resisting building designed according to the post-1995 Kobe earthquake Japanese design and construction practice. Therefore, the columns

were hollow square-tube sections, and the beams were I-section members. The specimen had composite slabs on each floor. The specimen was subjected to a cyclic-loading protocol that included ratcheting beyond the 0.05-rad story drift cycles. The frame sustained a maximum story drift ratio of 0.06 rad in the first story before yielding and local buckling of both ends of the first story columns caused a 50% reduction in story shear. The observed collapse mode of the structure was attributed to plastic deformations of the anchor bolts [15] and to the high width-to-thickness ratios of the columns that did not satisfy the compactness requirements per AISC Seismic Provisions. The positive-moment strength of the composite beams was 30% larger than the bare-steel beam.

2.4.2 Shake-Table Tests

2.4.2.1 *Steel moment-resisting frames with concentrated plasticity*

Experimental studies of complete building structures are scarce due to the associated limitations in budget and experimental facilities. Therefore, many experimental studies of the inelastic behavior of steel moment-resisting frames used simplified models and systems.

Rodgers and Mahin [16] tested a 1/3-scale, 2-story, 1-bay planar steel moment-resisting frame on a shake-table test. The floor mass was 480 kg/m². The excitations included cosine pulses and real ground motion records representative of near-source and far-source events. The specimen had pinned column bases. The beam-to-column connection consisted of a pin capable of transferring axial and shear forces, while the moment was resisted by a couple of parallel cylindrical steel coupons mimicking the “flanges” of the beam. Some of the connections were intended for ductile behavior, while others were designed to experience brittle fracture. The tests showed that after fracture, the systems with ductile connections behaved significantly better than the specimens with brittle connections. Also, fracture induced a transient response to the base shear and acceleration response but with no visible effect on the displacement response.

Lignos et al. [17] conducted shake-table tests of two identical 1/8-scale, 4-story, 2-bay planar steel moment-resisting frames that used mechanical plastic hinge devices at the beam and column ends. The plastic hinge devices were designed to reproduce the hysteretic behavior of the full-scale building (or “prototype structure”) after which the test specimen was designed. The test setup was specially designed to isolate the P- Δ effects from the total response of the structure. Each specimen was subjected to a different series of ground motions with increasing intensities. Both specimens were proportioned according to the strong-column/weak-beam criterion but collapsed in a three-story mechanism that included plastic hinges at both ends of the second and third floors, and plastic hinges at the bases of the 1st story and the top of the 3rd story columns after sustaining a story drift ratio (SDR) of 0.15 rad. P- Δ effects were shown to be sufficiently well approximated as the product between the total structural weight and the story drift ratio of the 1st story. In the tests, the P- Δ resulted in 60% of the base shear at peak lateral strength of the system.

2.4.2.2 *Steel moment-resisting frames with realistic beam-to-column moment connections*

The test facility of E-Defense, inaugurated in 2005, facilitated the testing of notable large-scale and realistic systems of various configurations, materials, and uses in the last two decades [18].

Suita et al. [19-21] conducted shake-table tests of a full-scale, 4-story, 2×1-bay moment-resisting steel building. The test specimen was designed according to the post-Kobe Japanese practice. The structural members were proportioned with a strong-column/weak-beam ratio of 1.48. The beam-to-column moment connection adopted the no-weld-access-hole detail. Each floor had an RC slab. The specimen was subjected to bidirectional ground motions. The beams remained elastic due to the composite action, but the estimated positive-bending strength was about 30% of the bare-steel beam bending strength. After sustaining a story drift ratio of 0.025 rad at the first story, shear strength degradation due to local buckling of the first story columns led the structure to rest on a perimeter safety system at 0.16 and 0.08-rad story drift ratio in the longitudinal and transverse directions, respectively, and a total loss lateral

strength. Despite proportioning the members according to the strong column/weak-beam criterion, such collapse mode was expected because the no-weld-access-hole detail adopted in the beam-to-column connection would increase the demands in the columns [22]. Based on these tests, Lignos et al. [23] suggested that a strong-column/weak-beam ratio of about 2.0 may improve the collapse capacity and avoid a single-story mechanism. The shake-table tests by Suita et al. were supplemented by reduced-scale shake-table tests of a two-story, 1×1-bay steel moment-resisting frame and cyclic-loading tests of a column component [24]. These tests were not intended to replicate the full-scale shake-table test specimen, but to check the measurement scheme for the shake-table tests. The reduced-scale specimen resulted less stiff than the column component because its idealized conditions do not hold for the 3D frame. Furthermore, the 3D frame resulted in smaller strength than the column component due to bidirectional loading.

Kubota et al. [25, 26] tested a 1/3-scale, 18-story, 3×1-bay high-rise steel moment resisting frame. The specimen was designed according to the Japanese practice employed during the 1980s and 1990s. The CJP welds were intended for field welding. The beams and columns were proportioned to satisfy the strong-column/weak-beam criterion. RC slab on steel deck was formed on each floor. An unidirectional synthetic long-period and long-duration ground motion was applied in series of increasing intensities. The composite beams of the lower stories yielded but fractured in the bottom flange before developing substantial inelastic rotations. Beam bottom flange fractures progressed in the lower 6 stories. After reaching 0.10 rad story drift in the first story, deformation concentrated in the lower four stories until the collapse of the structure when the column bases fractured at a story drift ratio of 0.17 rad. The tests showed that the natural period of the structure and damping ratio remained constant even after fracture of the 1st story beams. Cracking of the RC slab did not have a significant effect on the natural period of the structure. The natural period and mode shapes were significantly affected when beam fracture spread to the next stories. That experimental program also included component tests of the columns [27] and beam-and-column subassemblages of variable scale [28]. The components were subjected to constant and variable amplitude cyclic loading. The component tests showed similar hysteretic response, deformation capacity, and failure mode between scaled and full-scale specimens.

2.4.2.3 Other full-scale tests of Steel Moment-Resisting Frame Systems

Shake-table tests were conducted on a full-scale building where the first four stories were designed according to the Japanese practice in the 1970s, and the remaining 17 stories were substituted by multiple concrete layers supported by isolators [29]. The specimen had RC slabs on the lower 4 stories. A 3rd-story beam fractured in the bottom flange, with fracture starting at the interface between the CJP weld and the weld access hole. The fracture occurred at a low rotation (0.007 rad). Similar crack initiation was observed in several other beams. Weld defects were detected at several locations. Therefore, the observed failure mode was similar to the reported after the 1995 Kobe earthquake.

Other shake-table tests conducted at E-Defense on steel building systems include a full-scale, 4-story, 1×1-bay steel moment-resisting frame that incorporated a spine with force-limiting connections (FLCs) [30, 31]. The spine is a relatively highly stiff vertical element that prevents the formation of story mechanisms by imposing a uniform lateral displacement profile along the building height. The FLCs are devices that limit the increased story accelerations imposed by the spine. Preliminary results showed that the spine successfully produced a uniform lateral displacement profile, and the FLC reduced the spine-induced floor accelerations.

2.4.3 Hybrid Tests

Del Carpio et al. [32, 33] studied the collapse behavior of a 1/2-scale 4-story steel moment frame building through hybrid simulation, a technique where only part of the structure is physically tested, while the rest is modeled numerically. The test specimen comprised a 1 ½-story, 1 ½-bay subassemblage of the whole four-story moment frame. The prototype building was designed as a Special Moment Frame according to the U.S. design codes. The beams adopted RBS moment connections and had a concrete slab. The hybrid system was subjected to the 1989 Loma Prieta ground motion scaled from 25 to 200%

to promote collapse. Loading was applied to the test specimen through horizontal actuators. The structure collapsed in a three-story sidesway mechanism at a 0.16-rad story drift ratio after significant degradation of the column bases. The hybrid tests demonstrated to be a more refined alternative than conventional beam-to-column subassembly tests.

2.4.4 Size Effect

The scaling or size effect is the increased likelihood of brittle fracture in steel members with thicker plates. In other words, reduced-scale specimens are less prone to brittle fracture than full-scale ones.

Oie et al. [34, 35] and Kishiki et al. [36] studied the size effect through cyclic-loading tests of built-up steel I-section beams with three different sizes. The beams were subjected to cyclic loading in three-point bending. The test results were affected by the weld-access-hole size and artificially induced weld defects.

The size effect was also examined through the beam-to-column component tests by Suita et al. [28]. The specimens consisted of two full-scale T-shaped beam-and-column subassemblages, two 1/3-scale T-shaped, and two 1/3-scale cruciform subassemblages. The reduced-scale specimens showed a better inelastic rotation performance than the full-scale specimen, which cracked and fractured at an early stage. However, the reduced-scale specimens that had the backing bars tack-welded to the beam flange developed early cracks. The tack weld was found to promote cracking of the beam flange in those reduced-scale specimens. Therefore, special attention should be paid to such small details when planning a reduced-scale specimen.

2.5 CHAPTER SUMMARY

This chapter presented a literature review of the two issues covered in this research: the beam-to-column moment connections to an I-section column and the large-deformation behavior of steel moment-resisting frames. The key lessons from the review are:

- The performance of moment connections to the column web of an I-section column is yet not well understood due to the lack of research on those connections in the last four decades.
- The poor performance of connections to the web of an I-section column in the past can be attributed to poor weld quality and connection details that were used before the Northridge and Kobe earthquakes.
- Shake-table test data of steel moment-resisting frames subjected to very large deformations is scarce.
- The use of strong-column/weak-beam ratio of about 1.5 in steel moment-resisting frames does not prevent the formation of a single-story collapse mechanism.
- Member deterioration, number of fractures, ground motion characteristics, and P- Δ effects are key factors leading to the collapse of steel moment-resisting frames.
- Reduced-scale systems tend to perform better than full-scale ones due to the size effect. However, tack welds, the size of the weld access holes, and other geometric details may potentially affect the outcome of the reduced-scale tests.

3. PERFORMANCE OF MOMENT CONNECTIONS TO AN I-SECTION COLUMN

3.1 GENERAL

Moment connections to the column flange, subjecting the column to major-axis bending, are preferred to make better use of the stiffness and strength of the I-section column. Moment connections to the column web, subjecting the column to minor-axis bending, are less preferred and consequently, not as well studied and understood. The limited research on moment connections to the column web was mostly conducted before the 1994 Northridge earthquake. Therefore, limited guidance is available in the U.S. and Japan for the design and detailing of moment connections to the web of I-section columns. Furthermore, almost no research data is available in Japan for moment connections to I-section columns, whether to the column flange or column web.

Japanese steel construction practice favors the use of square-tube (hollow structural sections or square built-up) columns over I-section columns because moment-resisting connections can be placed at any column face. Figure 3.1a illustrates a typical Japanese square-tube column that is part of two orthogonal moment resisting frames. On the other hand, the U.S. construction prefers I-section columns because the costs are lower and the fabrication easier. Typical construction of steel buildings in the U.S. comprise a moment resisting system in one direction and a braced frame in the orthogonal direction. Figure 3.1b shows an I-section column that is part of a moment-resisting frame with respect to the column's major axis, and of a braced frame about its minor axis. In the latter case, the beam-to-column connections in the braced frame are regarded as "pinned". However, in Japan there are a significant number of low-rise buildings where I-section columns comprise moment resisting systems in both directions. In such case, moment-resisting beam-to-column connections are placed to the column web as well as the column flange. Figure 3.1c shows a real example of moment connections to an I-section column in Japan. Since there is no design guidance for such detail, the connection is fabricated somewhat similar to Figure 3.1a, losing the benefits in the economy and easy fabrication mentioned about the detail in Figure 3.1b.

In recent decades, details have been studied in Japan to improve ductility of beam-to-column moment connections. One example is the elimination of the "weld-access hole" or "scallop", which is a curved cut in the web to allow the placement of the backing bars and conform the groove welds. However, the recent improvements in welding quality and details like the non-weld access hole have seldom been tested in connections to an I-section column.

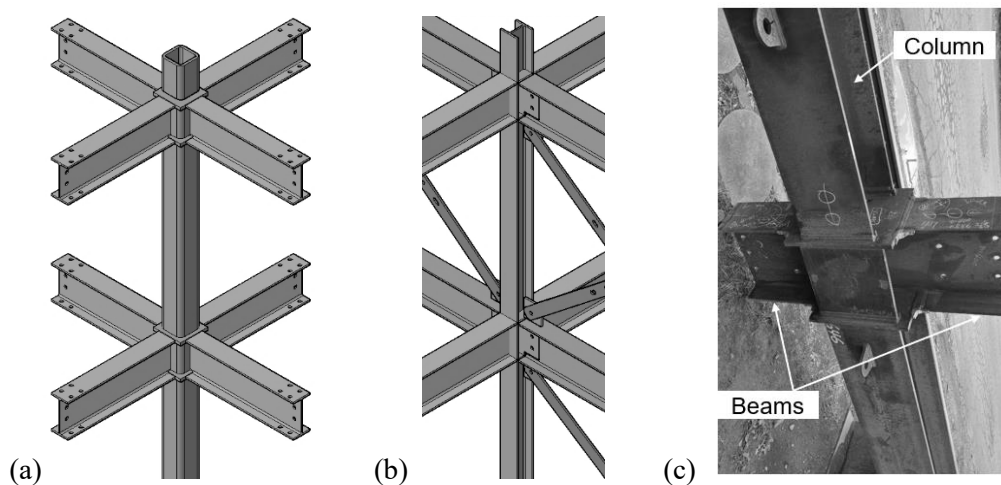


Figure 3.1 Connections to columns in moment-resisting frames: a) square-tube column in "tree-construction", b) I-section column, and c) real example of moment connections to an I-section column in Japan.

3.2 TEST PURPOSE

An experimental and numerical study was conducted on moment connections to an I-section column with the following objectives:

- To fill the knowledge gap in the seismic performance of steel beam-to-column moment connections either to the flange or web of an I-section column detailed and constructed according to the current Japanese practice through cyclic-loading tests and Finite Element Method analysis of beam-to-column subassemblies representative of typical low-rise buildings in Japan.
- To evaluate the seismic performance of the beam-to-column moment connections in terms of the deformation capacity and strength requirements listed in the U.S. Seismic Provisions.
- To present the failure modes associated with the connections to an I-shaped column and design recommendations to secure adequate cyclic-loading performance of these steel moment-resisting connections.
- To examine the influence of the current Japanese construction practice aspects, such as the no-weld access hole detail and the shop-welded connections, in the performance of the connections to an I-section column for seismic applications.
- To observe whether moment connections to the column web can achieve satisfactory performance even when the continuity plates are flush with the column flange.

3.3 TEST SETUP

The experimental program consisted of six specimens that used I-section members and tested under cyclic loading to examine the seismic performance of steel beam-to-column moment connections either to the column flange or web. Figure 3.2 shows the test setup used to subject the beam-to-column subassemblies to cyclic loading. As shown in Figure 3.3, the T-configuration of the specimen represented an external beam-to-column subassemblage of a low- to mid-rise steel moment frame buildings in Japan. Figure 3.3 also shows the definition of Story Drift Ratio, the parameter used to control displacements during the experiments. The tests considered in-plane actions only, since the beam moment connections were formed with respect to only one direction of the column.

The H-400×400×13×21 column was simply supported over a span of 2.4 m. The H-500×200×10×16 beam was loaded at the free end by a 2000-kN jack, 2.3 m away from the centerline of the column. A 0.90-m beam stub was shop welded to the column, a practice according to the column-tree construction described in Chapter 2. Then, an extension beam was spliced by twenty-four M20 F10T slip-critical bolts to complete the specimen in the test facility.

Out-of-plane support was provided to the beams at the locations indicated by “×” marks. The lateral bracing system was designed by S. Yang [37]. Rollers were attached to the flanges of the beam at the bracing points to prevent friction with the restraining system. Diagonal angles attached to the loading frame provided lateral support to prevent torsional buckling. For F1 and W1, box sections 150×100×6 were placed in both sides of the beam instead of the rollers [38].

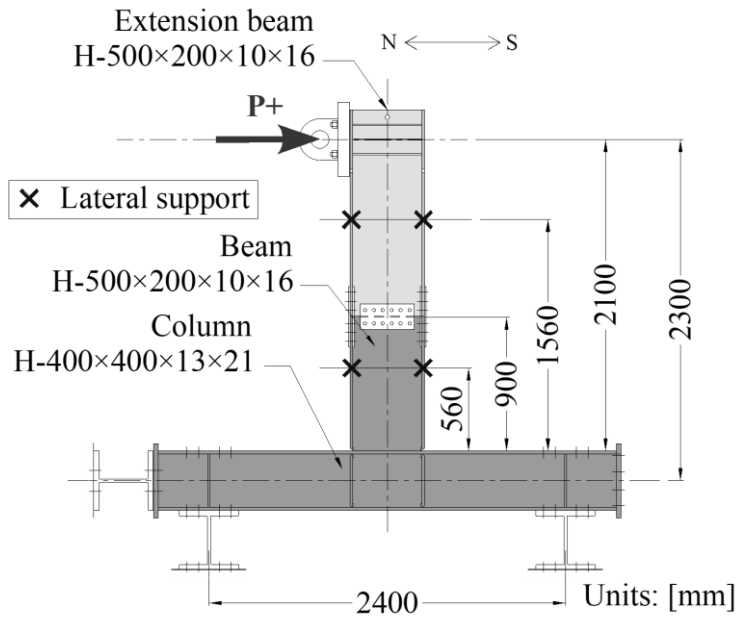


Figure 3.2 Test setup.

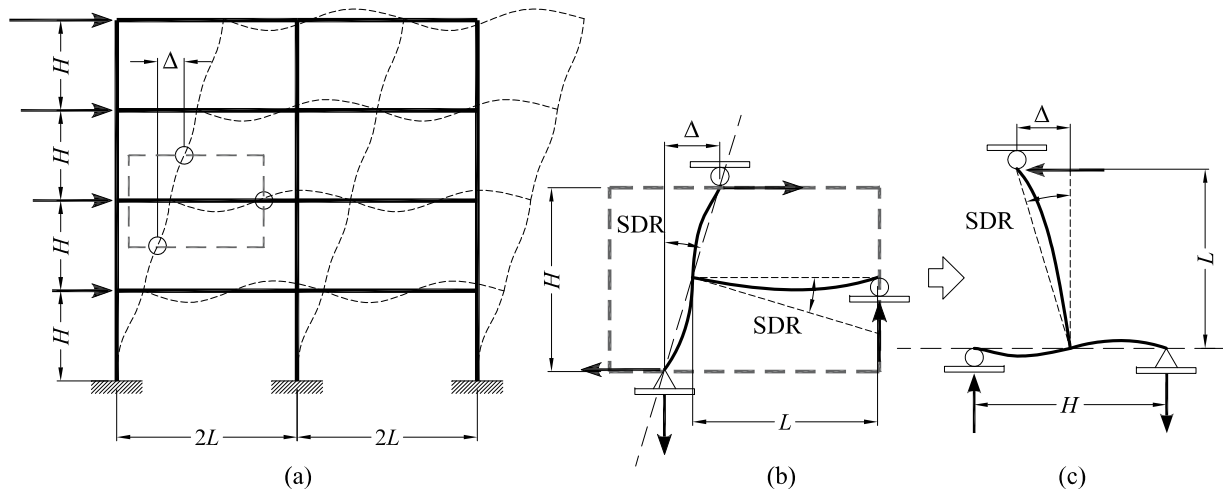


Figure 3.3 Definition of Story Drift Ratio: a) deformation of a typical 4-story building; b) beam-to-column subassembly of an exterior column; c) analogy with the test specimen setup.

3.4 TEST SPECIMENS

3.4.1 Connection details

Figure 3.4 a and b show the connection details of the three specimens with a moment connection to the column web (hereinafter referred to as W-Specimens), while Figure 3.4c to e show the three specimens with a moment connection to the column flange (F-Specimens). The details represent the common Japanese practice in design, fabrication, and construction. Specimens F1 and W1 correspond to Specimens 1 and 2, respectively, that were tested by K. Komatsu [38], who started this study on cyclic loading performance of steel beam-to-column moment connections with I-section members in the Laboratory of Structural Engineering, Hokkaido University. Therefore, the results of his work are combined and presented here with the other four specimens tested by the author.

The components of all specimens were made of JIS SN400 and SN490 steel, whose minimum specified yield strength is 235 and 295 N/mm², respectively, and are among the most common steel grades used

for building construction in Japan. Grade B is chosen for primary load-carrying members, while grade C has certified through-thickness properties. Another important property of grades B and C of SN400 and SN490 steels for cyclic-loading is their certification to a maximum yield-to-tensile strength ratio of 0.8. This limit provides the SN400 and SN490 grades B and C a better balance between strength and ductility than the high strength steels used for tall buildings in Japan [39].

Table 3.1 shows the characteristics of the six subassemblies tested in this program. All specimens used SN400B steel for the beam and SN490 steel, either grade B or C, for the column. A doubler plate was welded to the column panel of Specimens W2, F2 and F3, but not to Specimens W1, W3 and F1. It is common practice to join the doubler plate to the column by fillet welds in the plate perimeter only, since the main function of the doubler plate is to strengthen the joint panel zone in major-axis moment connections. Therefore, the connection of the beam web to a doubler plate instead of directly to the column web was cause of concern, especially due to the lack of literature available on the structural performance of the joint in such condition. In order to achieve improved integrity between the column web and doubler plate in the panel zone, Specimen F3 implemented evenly spaced plug welds per detailing requirements for structural steel tall buildings in China [40, 41].

Table 3.1 Beam-to-column connection test matrix.

Specimen	Column	Continuity plates	Doubler plate
F1	SN490C	SN400B	-
F2	SN490C	SN400B	SN490B
F3	SN490B	SN490B	SN490B
W1	SN490C	SN400B	-
W2	SN490C	SN400B	SN490B
W3	SN490B	SN490B	-

3.4.2 Continuity plates and connection of the beam web

The beam flanges were welded to the column flange for F-Specimens and to continuity plates for W-Specimens, in both cases using CJP groove welds. For all specimens, full-depth continuity plates were placed on both sides of the column web. The continuity plates were welded to the column flanges using CJP groove welds and to the column web using fillet welds. It is noted that, for beam-to-column web connection, engineers generally specify CJP groove welds between the continuity plate and column web. As shown in Figure 3.4a, fillet welds were specified instead of CJP at those locations based on the understanding that a significant portion of the tensile force delivered by the beam flanges is transferred to the column through the column flange, and hence the weld between the continuity plate and column web should not be critical. The beam web was fillet welded to the column, to the flange for F-Specimens, to the doubler plate for Specimen W2 and to the column web for Specimens W1 and W3.

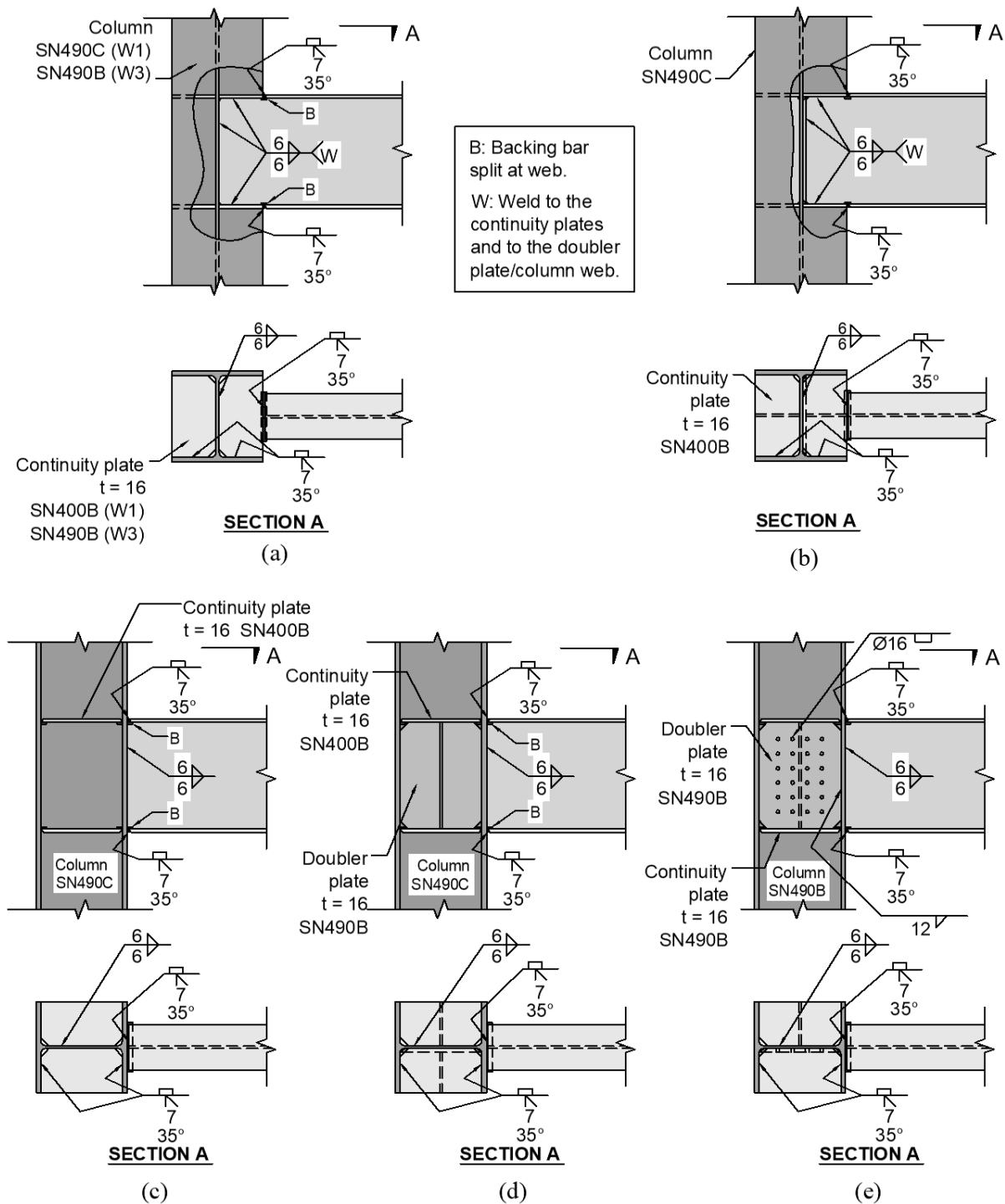


Figure 3.4 Connection details: a) Specimen W2, b) Specimen W1 and W3, c) Specimen F1, d) Specimen F2, and e) Specimen F3.

3.4.3 Fabrication process of the CJP groove welds with no-weld-access hole

The CJP groove welds were made in flat position with a gas-shielded metal arc welding (GMAW) process. Specimens W1 and F1 used the tougher and stronger 1.2-mm diameter JIS YGW18 electrodes (specified yield strength between 550 and 740 N/mm²) while the other four specimens used 1.2-mm diameter JIS YGW11 electrodes (specified yield strength between 490 and 670 N/mm²). Over the past 20 years, YGW18 have gradually replaced YGW11 as the most popular GMAW electrode used in Japan. The CJP groove welds were completed with no weld access holes, a weld detail commonly adopted in

Japan due to their superiority in strength and deformation capacity over conventional details that implement weld access holes [5].

Figure 3.5 illustrate the welding process of the specimens. For beam-to-column web connections, the beam flanges are removed and the CJP grooves are fabricated by a mill machine (see Figure 3.5a). As shown in Figure 3.5b, the no-weld-access hole detail of beam flanges calls for separate backing of the CJP groove welds, placed at each side of the web and left in place after welding. One end of the backing bar is rounded to the radius of the fillet at the junction between the beam flange and web. The beam was placed vertically against the column to allow flat-position welding of both beam flanges without repositioning see (Figure 3.5c). As in these specimens, ceramic weld tabs are preferred over steel tabs because they facilitate removal after welding. All CJP groove welds were ultrasonically tested by licensed inspectors and no defect was detected.

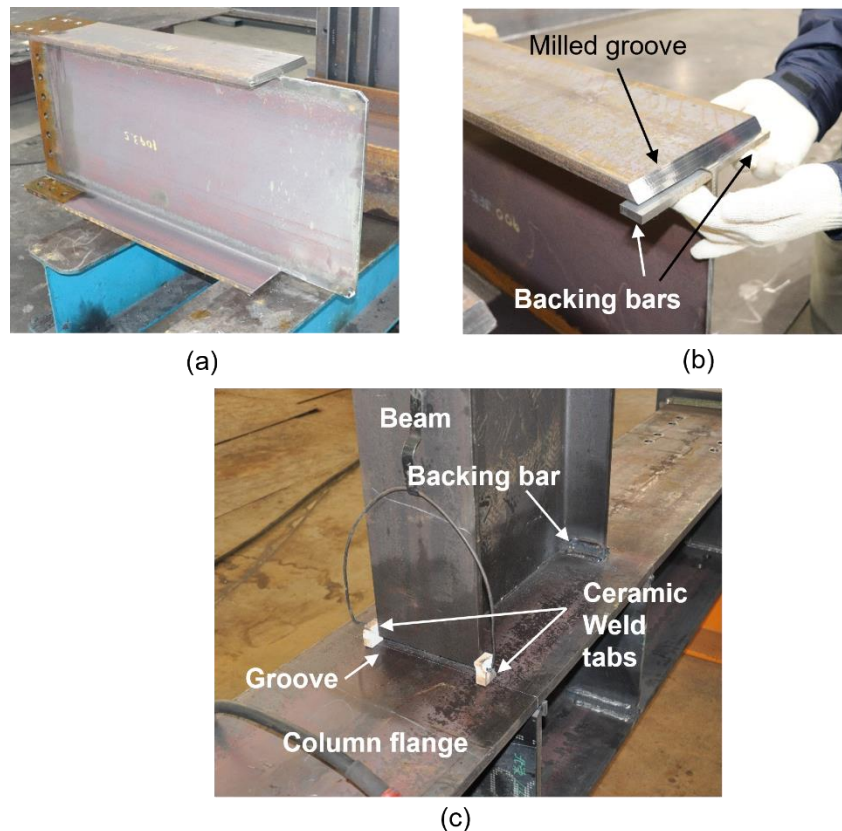


Figure 3.5 CJP weld process with no access hole: a) beam of a W-Specimen after the conformation of the groove, b) beam of an F-Specimen with the segmented steel backing bar, and c) beam of an F-Specimen with the backing bars and ceramic weld tabs, in position for groove welding.

The actual dimensions of the specimens were checked and resulted well within the tolerance requirements specified by JIS G3136 [43].

3.5 MATERIAL TESTS

3.5.1 Sampling

The material properties of the components of the test specimens were determined by tension coupon tests with designed dimensions according to JIS 1A and shown in Figure 3.6a. In order to qualify a beam-to-column moment connection for seismic applications, the AISC Seismic Provisions [44] mandates coupon tests to get the material properties of each component expected to contribute to the total inelastic rotation. However, since coupon tests from the continuity plates were not provided, the

results from mill test report were used instead.

Of the four coupon tests of the SN400B beam flange, two were designated for cyclic tests (coupons B and E in Figure 3.6b). The two cyclic coupon test data were used to calibrate the material model for the Finite Element Analysis described later in this Chapter.

The coupons of tension and cyclic tests were extracted from the locations shown in Figure 3.6b and c. All the coupons used a calibrated gauge length $L_0 = 200$ mm. The actual dimensions of each coupon were measured at four sections located at the quarters of the calibrated length. The measured values were averaged and used as the gauge length L_0 and cross-sectional area A_0 for the engineering stress and strain calculations.

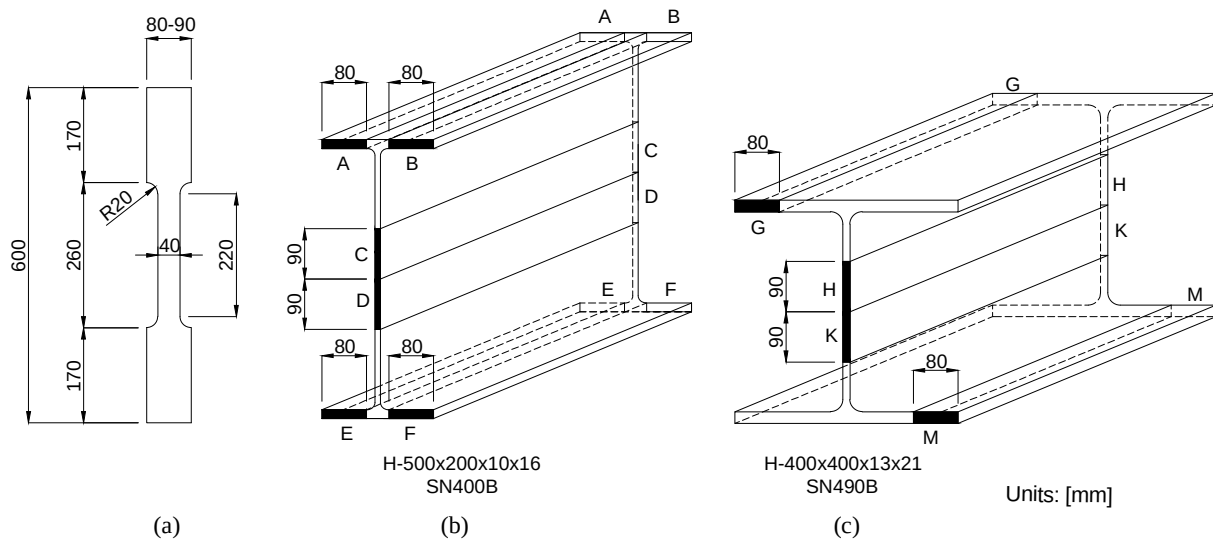


Figure 3.6 Coupons for material test: a) dimensions, b) SN400B beam, c) SN490B column.

3.5.2 Tension coupon tests

Figure 3.7 shows the typical setup used for the tension coupon tests. The samples were loaded monotonically in tension up to fracture using a 1000-kN Universal Testing Machine. The change in length was measured from the average of two CDP-25M displacement transducers, based on a 200-mm gauge length. The range of the instruments was 25 mm. Since the expected elongation was about 60 mm, it was necessary to reposition the transducers two or three times per test until the fracture of the coupon.

As a reference, two CDP-50M displacement transducers were used to measure the displacement of the crossheads of the universal testing machine.

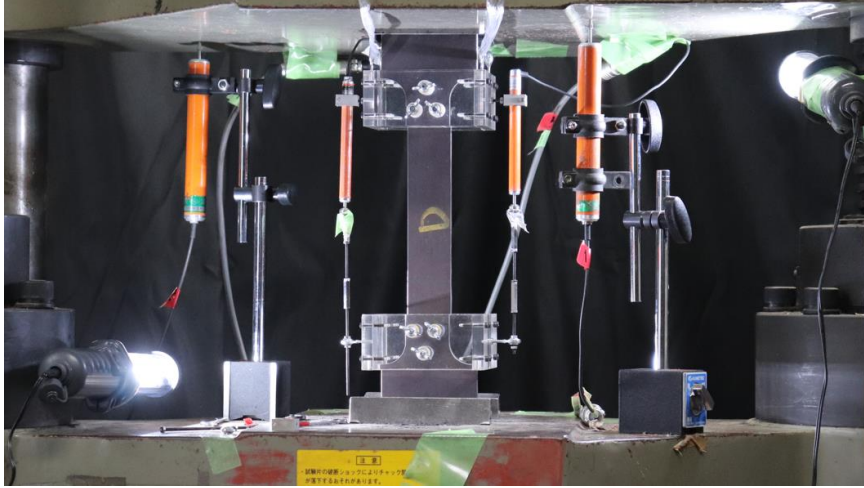


Figure 3.7 Tension coupon test setup.

Table 3.2 summarizes the material properties of every component obtained from coupon tests. Mill test report properties are listed where coupons were not provided.

Table 3.2 Mechanical properties

Element	Specimen	Grade	Component	F_y (N/mm ²)	F_u (N/mm ²)	Elong. (%)
Column	W1, W2, F1, F2	SN490C	Flange	360	532	28
			Web	400	552	23
Column	W3, F3	SN490B	Flange	368	514	33
			Web	400	529	29
Beam	W1, F1	SN400B	Flange	313	465	27
			Web	367	497	24
Beam	W2, W3, F2, F3	SN400B	Flange	321	455	32
			Web	389	481	28
Continuity plate	W3, F3	SN490B	N/A	412 ^a	538 ^a	27 ^a
Continuity plate	W1, W2, F1, F2	SN400B	N/A	293 ^a	431 ^a	29 ^a
Doubler plate	W2, F2, F3	SN490B	N/A	412 ^a	538 ^a	27 ^a

^aFrom mill test report

3.5.3 Cyclic coupon tests

Figure 3.8 shows the test setup for the cyclic tests. A 750-kN actuator was used to subject the coupons to cycles of tension and compression with increments of 0.5% strain. Each cycle amplitude was conducted two times (2 cycles of $\pm 0.5\%$, 2 cycles of $\pm 1.0\%$, 2 cycles of $\pm 1.5\%$, and so on). A buckling-restraining device was used to prevent buckling of the coupon when loaded in compression. Similarly to the tension tests, two displacement transducers were employed to measure the change in length on a 200-mm calibrated length. For these tests, however, CDP-10MT transducers, with a range of ± 5 mm in either tension or compression were used, since loading beyond strains of 0.04 mm/mm (4.0%) was not

possible due to limitations of the transducer holder device and the buckling-restrainer.

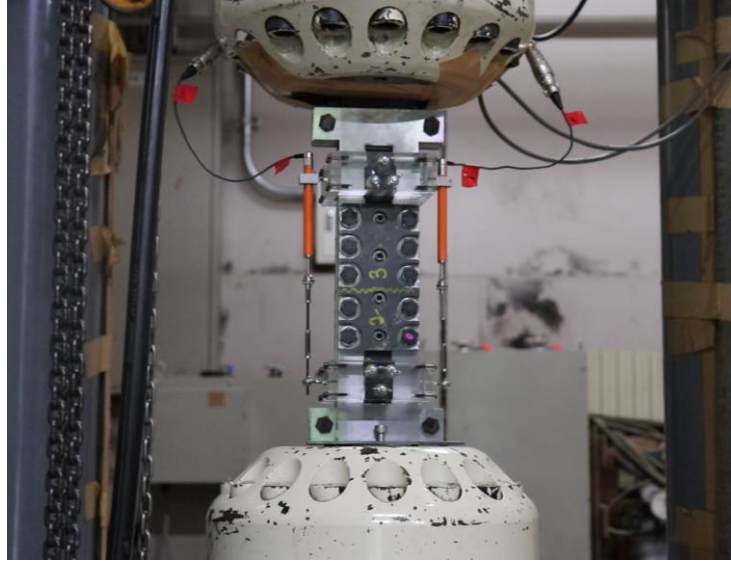


Figure 3.8 Cyclic coupon test setup.

The cyclic response of both coupons is shown in Figure 3.9. Only coupon E was able to complete the first cycle of ± 0.04 mm/mm before the restraining system was no longer strong enough to prevent the coupon's buckling.

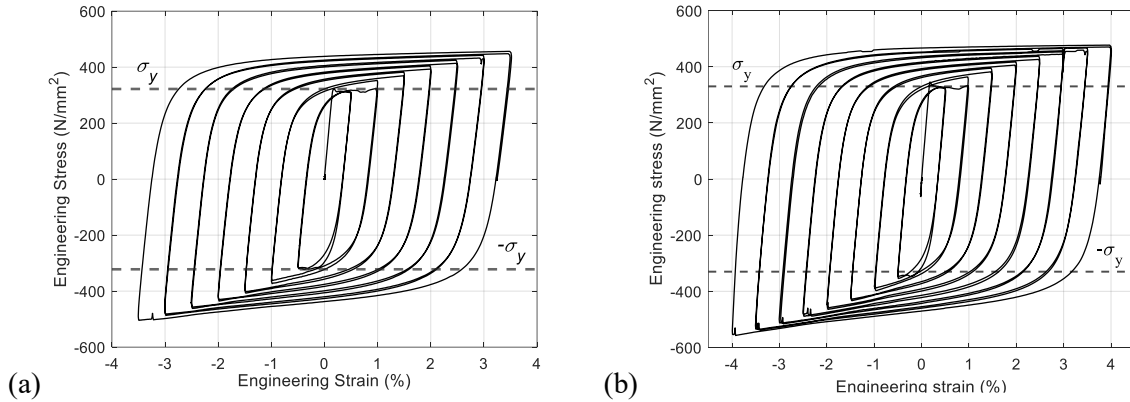


Figure 3.9 Cyclic response of SN400B: a) Coupon B, and b) Coupon E.

3.6 STRENGTH RATIOS

The plastic moment of the beam, column and panel zone were calculated using the measured yield strength of the flanges and web from Table 3.2. The equations for the I-section beam and column were derived for bending about either the major or minor axis of the cross section, considering the geometry of the flange, web and fillets in the junction between the flange and the web. The following are the equations to compute the plastic moment of the I-section members:

$$M_p = Z_f \sigma_{yf} + (Z_w + Z_r) \sigma_{yw} \quad (3.1)$$

If bending is about the major axis of an I-section member,

$$Z_f = B t_f (D - 2 t_f)$$

$$Z_w = \frac{(D - 2t_f)^2 t_w}{4} \quad (3.2)$$

$$Z_r = 0.4292R^2(D - 2t_f - 0.4467R)$$

If bending is about the minor axis of an I-section member,

$$Z_f = \frac{t_c B_c^2}{2}$$

$$Z_w = \frac{(D - 2t_f)t_w^2}{4} \quad (3.3)$$

$$Z_r = 0.4292R^2(t_w + 0.4467R)$$

Where

B : flange width

D : section depth

t_f : flange thickness

t_w : web thickness

R : fillet radius

σ_{yf} : yield strength of the flange

σ_{yw} : yield strength of the web

To compute the plastic moment strength of the Panel Zone, the AIJ [38] equation was used, considering small or negligible effect of axial load in the column. The equation is as follows:

If bending is about the major axis of an I-section column,

$$M_{ppz} = D_c D_b (t_{cw} + t_{dp}) \frac{\sigma_{ycw}}{\sqrt{3}} \quad (3.4)$$

If bending is about the minor axis of an I-section column,

$$M_{ppz} = 2 \left(1 - \frac{\sqrt{3} D_b}{B_c} \right) D_c D_b t_{cf} \frac{\sigma_{ycf}}{\sqrt{3}} \quad (3.5)$$

Where

B_c : flange width

D_b : section depth

t_{cf} : column flange thickness

t_{cw} : column web thickness

t_{dp} : doubler plate thickness (if any)

σ_{ycf} : yield strength of the column flange

σ_{ycw} : yield strength of the column web

The “strong-column/weak-beam” (SC/WB) criterion was evaluated by adding the plastic moments at the intersection of column and beam centerlines, as shown in Figure 3.10. The plastic moment of the beam and column at the joint face (M_{pb} and M_{pc} , respectively) were calculated with Eq. 3.1 and then extrapolated to the node to obtain the nodal plastic moments M_{pb}^* and M_{pc}^* . Based on Figure 3.10, the corresponding extrapolated plastics strengths for the beam, column and panel zone are:

$$M_{pb}^* = \frac{M_{pb}}{\left(1 - \frac{D_c}{2L}\right)} \quad (3.6)$$

$$M_{pc}^* = \frac{M_{pc}}{\left(1 - \frac{D_b}{H}\right)} \quad (3.7)$$

$$M_{ppz}^* = \frac{M_{ppz}}{\left(1 - \frac{D_c}{2L} - \frac{D_b}{H}\right)} \quad (3.8)$$

In ductile design of steel buildings, meeting the SC/WB results in columns that are strong enough to force a multi-story plastic mechanism, hence improving the energy dissipation characteristics of the system [13]. From Table 3.3, all the specimens satisfy the SC/WB criterion.

The panel zone plastic strength was compared in terms of its plastic shear strength V_p instead of M_{ppz} . Figure 3.11 illustrates the concept of panel zone shear. The panel zone shear strength is as follows:

$$V_p = 2 \left(1 - \frac{\sqrt{3}D_b}{B_c}\right) D_c t_{cf} \frac{\sigma_{ycf}}{\sqrt{3}} \quad (3.9)$$

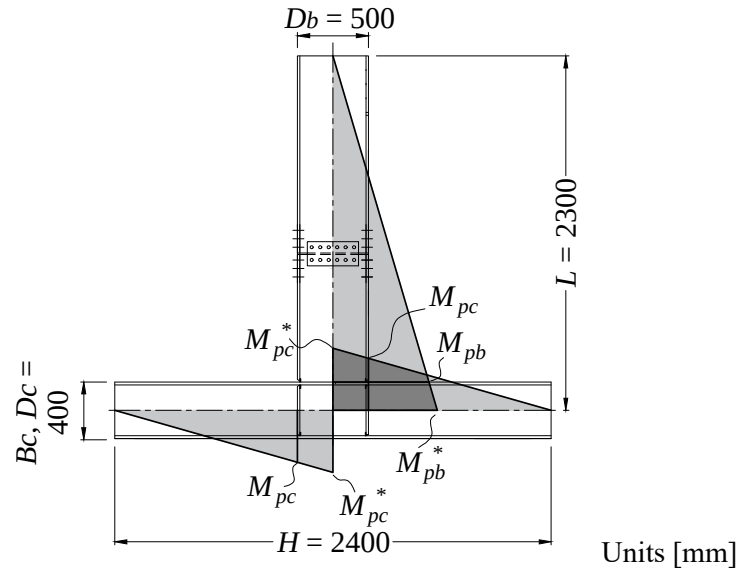


Figure 3.10 Definition of moments at the joint face and nodal moments.

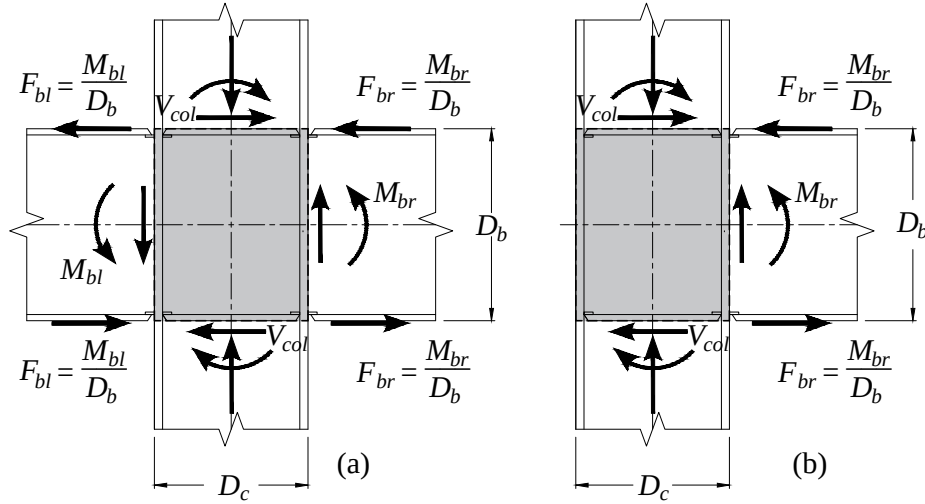


Figure 3.11 Panel zone free body diagram: a) interior connection, b) external connection.

Table 3.3 also shows the results for panel zone shear strength, V_{ppz} .

Table 3.3 Strength ratios

Specimen	Plastic moment [kN·m]		Panel zone strength [kN]	Nodal plastic moment [kN·m]			$\frac{2M_{pc}^*}{M_{pb}^*}$	$\frac{M_{ppz}^*}{M_{pb}^*}$
	M_{pb}	M_{pc}		M_{pb}^*	M_{pc}^*	M_{ppz}^*		
F1	698	1340	1200	764	1690	851	4.4	1.1
F2	723	1340	2680	792	1690	1900	4.3	2.4
F3	723	1370	2680	792	1730	1900	4.4	2.4
W1	698	613	2230	764	774	1620	2.0	2.1
W2	723	613	2230	792	774	1620	2.0	2.0
W3	723	626	2280	792	791	1650	2.0	2.1

3.7 ESTIMATION OF THE ELASTIC STIFFNESS

In order to verify the elastic stiffness of the specimens during the early loading stages of the experiments, the theoretical values were estimated from structural mechanics. To do so, firstly the horizontal displacement of the beam free end was computed considering only flexural deformations of the beam and the column and using centerline dimensions. From the virtual work method, the horizontal displacement was calculated as follows:

$$\begin{aligned}
 1 \cdot \delta &= \frac{P}{EI_b} \int_0^L (-x)^2 dx + \frac{P}{EI_c} \int_0^{H/2} \left(-\frac{L}{H}x\right)^2 dx + \frac{P}{EI_c} \int_0^{H/2} \left(\frac{L}{H}x\right)^2 dx \\
 \delta &= \frac{PL^3}{3EI_b} + \frac{PH^3}{12EI_c} \left(\frac{L}{H}\right)^2 \\
 \delta &= \frac{PL^3}{3EI_b} + \frac{PL^2H}{12EI_c}
 \end{aligned} \tag{3.10}$$

The second step was to add the contribution of the panel zone shear deformation. From the relation of angles and dimensions of Figure 3.12,

$$\begin{aligned}
 SDR &= \gamma \left(1 - \frac{D_c}{2L} - \frac{D_b}{H} \right); \quad \delta_{PZ} = L \cdot SDR = L\gamma \left(1 - \frac{D_c}{2L} - \frac{D_b}{H} \right) \\
 \tau &= G\gamma = \frac{V_p}{D_c \cdot t_w} = \frac{P \left(\frac{L_b}{D_b} - \frac{L}{H} \right)}{D_c \cdot t_w} \\
 \delta_{PZ} &= \frac{P \left(\frac{L_b}{D_b} - \frac{L}{H} \right)}{G \cdot D_c \cdot t_w} \left(1 - \frac{D_c}{2L} - \frac{D_b}{H} \right) L
 \end{aligned} \tag{3.11}$$

Hence, the lateral stiffness in [kN/m] and the rotational stiffness from the moment at the beam and column centerlines intersection vs Story Drift Ratio relationship in [kN-m/rad] are, respectively,

$$K = \frac{P}{\delta + \delta_{PZ}} = \frac{1}{\frac{L^3}{3EI_b} + \frac{L^2H}{12EI_c} + \frac{\left(\frac{L_b}{D_b} - \frac{L}{H} \right)}{G \cdot D_c \cdot t_w} \left(1 - \frac{D_c}{2L} - \frac{D_b}{H} \right) L} \tag{3.12}$$

$$K_{rot} = KL^2 \tag{3.13}$$

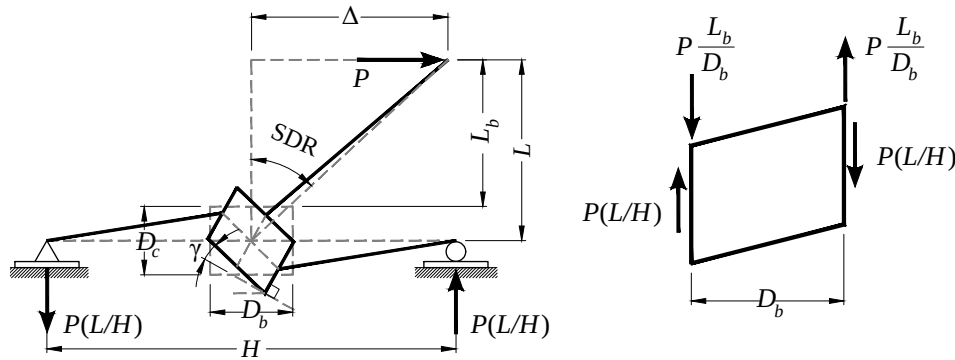


Figure 3.12 Reference dimensions for lateral stiffness calculation.

3.8 LOADING PLAN

As shown in Figure 3.13, a hydraulic jack with a capacity of ± 2000 kN and ± 150 mm stroke was used to apply horizontal loading at the beam free end. As shown in the same figure, six PC steel rods were used to connect the specimen's beam to the jack, and shim plates were also inserted between the jack and the specimen to make sure loading could start from the center of the stroke. The applied load was measured by a load cell mounted on the jack, as indicated in the Figure.

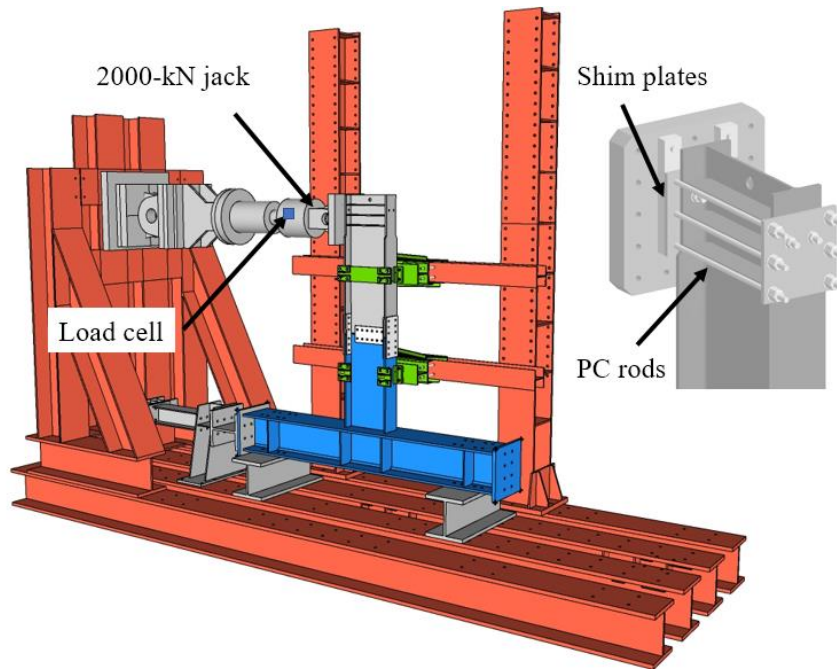


Figure 3.13 Loading system

The Story Drift Ratio was the parameter used for displacement control of the tests. The specimens were subjected to cyclic loading according to the protocol of Section K2 of AISC Seismic Provisions [44], as shown in Figure 3.14.

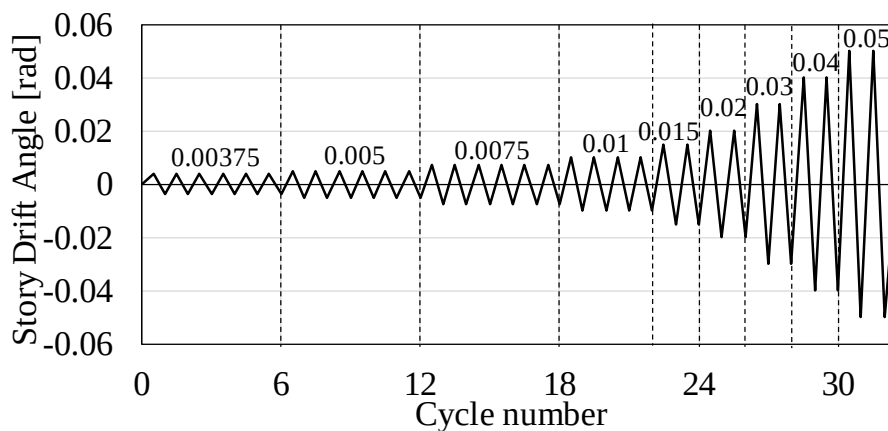


Figure 3.14 Loading protocol for beam-to-column moment connection test.

3.9 MEASUREMENT PLAN

3.9.1 Instrumentation

A combination of displacement transducers and strain gages were used to measure the global and local responses of interest in this study. Figure 3.15 shows the position of measurements, and the positive direction of each displacement-measuring instrument. Figure 3.16 shows the location and labeling of strain gages. Post-yield uniaxial strain gages, with 2-mm gage length (YEFLA-2), were used to get the strain profile of the beam in a section 75 mm away from the column face. Triaxial post-yield 2-mm gauge length rosettes (YEFRA-2) were installed in the continuity plates of W2 and W3, and in the web of the F-Specimens to evaluate the shear strains. In the case of the rosettes, it is important to identify carefully each measuring axis, as marked in Figure 3.16.

To obtain the story drift horizontal displacement, the displacement due to rigid body translation and rotation due to flexibility of the supports were subtracted from the total string pot measurement. The panel zone shear deformation was measured with two pairs of diagonally placed displacement transducers at both sides of the column, as shown in Figure 3.15. For the W-Specimens, three displacement transducers were placed beneath each continuity plate to measure its local in-plane deformation.

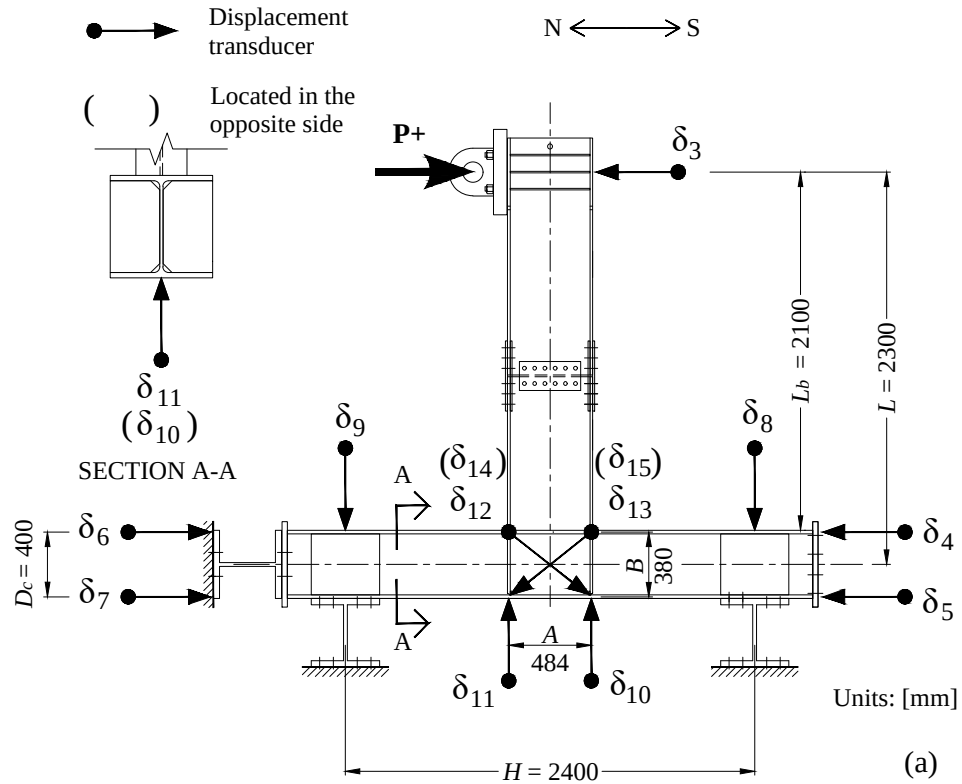


Figure 3.15 Displacement transducers on: a) F-Specimens

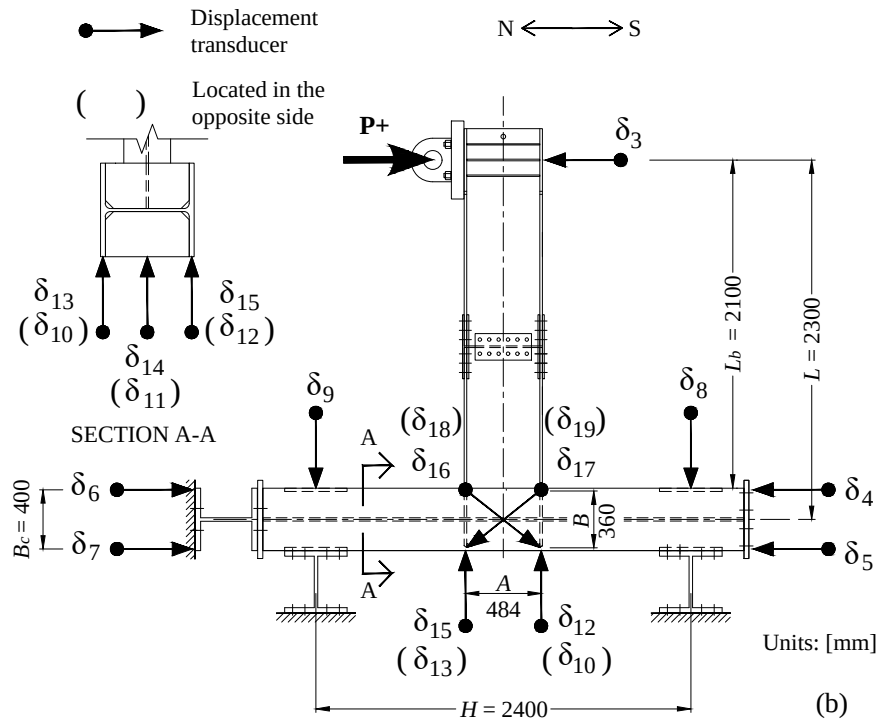


Figure 3.15(cont.) Displacement transducers on: b) W-Specimens

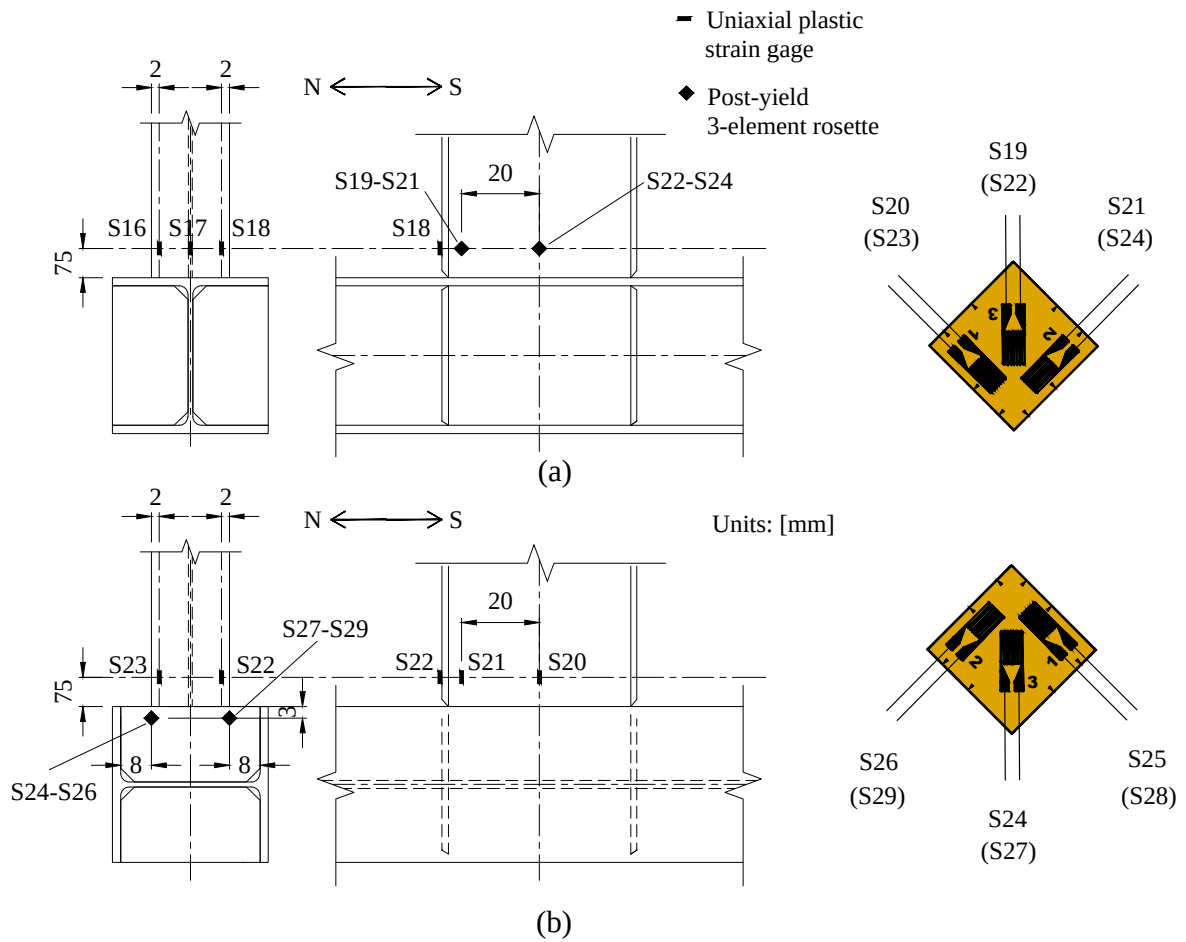


Figure 3.16 Location of strain gages and rosettes in: a) F-Specimens, and b) W-Specimens.

3.9.2 Data Reduction

As mentioned in the previous section, The story drift computation required the subtraction of the specimen's rigid body motion from the total horizontal displacement of the beam tip. All the equations were deducted assuming small deformations theory, so that $\sin \theta \approx \theta$.

Following the notation and sign convention of Figure 3.15, the total displacement of the beam tip is

$$\Delta_T = -\delta_3 \quad (3.14)$$

Horizontal rigid body displacement of the specimen due to support flexibility

$$\Delta_H = \frac{\delta_6 + \delta_7 - \delta_4 - \delta_5}{4} \quad (3.15)$$

Horizontal displacement due to rigid body rotation of the specimen due to support flexibility

$$\Delta_R = (\delta_8 - \delta_9) \frac{L}{H} \quad (3.16)$$

Story drift displacement, Δ

$$\begin{aligned} \Delta &= \Delta_T - \Delta_H - \Delta_R \\ \Delta &= -\delta_3 - \left(\frac{\delta_6 + \delta_7 - \delta_4 - \delta_5}{4} \right) - (\delta_8 - \delta_9) \frac{L}{H} \end{aligned} \quad (3.17)$$

Story Drift Ratio, SDR

$$\begin{aligned} \text{SDR} &= \frac{\Delta}{L} \\ \text{SDR} &= \frac{1}{L} \left[-\delta_3 - \left(\frac{\delta_6 + \delta_7 - \delta_4 - \delta_5}{4} \right) \right] - \frac{(\delta_8 - \delta_9)}{H} \end{aligned} \quad (3.18)$$

The global response was monitored by plotting the Story Drift Ratio SDR against the beam moment at the column face, M_b , which is the product between the applied horizontal force P and the beam clear length L_b , as follows:

$$M_b = PL_b \quad (3.19)$$

For the panel zone shear deformation, γ , Figure 3.17 shows the reference dimensions and angles used to derive the formula. The result was expressed in terms of inverse trigonometric functions, so the Maclaurin series expansion was used to simplify the expression, neglecting the terms with powers greater than 1. The resulting equation was:

$$\text{F-Specimens} \quad \gamma = \left(\frac{\delta_{12} + \delta_{14}}{2} - \frac{\delta_{13} + \delta_{15}}{2} \right) \frac{\sqrt{A^2 + B^2}}{2AB} \quad (3.20)$$

$$\text{W-Specimens} \quad \gamma = \left(\frac{\delta_{16} + \delta_{18}}{2} - \frac{\delta_{17} + \delta_{19}}{2} \right) \frac{\sqrt{A^2 + B^2}}{2AB} \quad (3.21)$$

Where A and B are the distances between targets, as Fig. 3.24 shows for each case. For the W-Specimens, the value of γ represents the average of the two panel zones characteristics of the minor axis moment connections.

The panel zone moment was calculated as the sum of the end-moments of the columns and beams connected at the joint, as was shown in Fig. 3.18(b):

$$M_{pz} = M_b - \left(\frac{V_{col,u} + V_{col,l}}{2} \right) D_b \quad (3.22)$$

The column-end shears are equal to the vertical reaction in the supports, and noting that they are equal per symmetry, the equation simplifies to

$$M_{pz} = M_b - V_{col} D_b \quad (3.23)$$

Where M_b is the moment at the column face and V_{col} is the vertical reaction at the supports:

$$V_{col} = P \left(\frac{L}{H} \right) \quad (3.24)$$

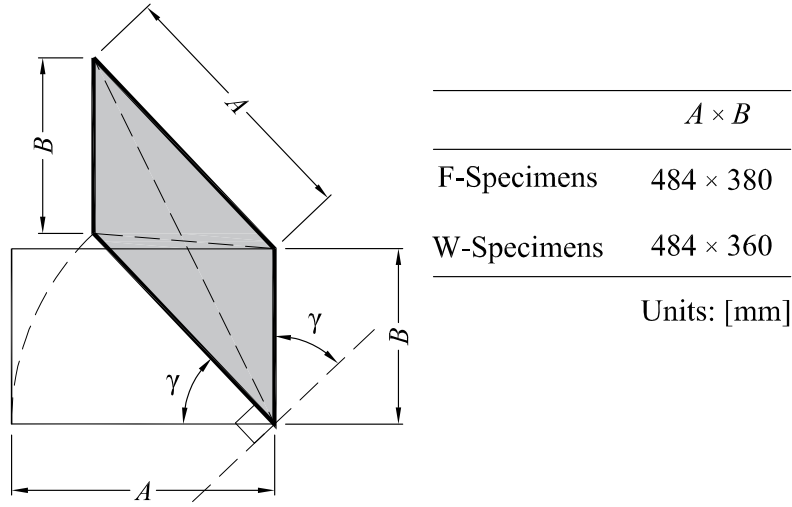


Figure 3.17 Panel zone shear deformation.

The rigid body rotation of the specimen due to support flexibility had to be deducted from the measurements to get the rotation of the joint, θ_j :

$$\text{F-Specimens} \quad \theta_j = \frac{\delta_{11} - \delta_{10}}{A} - \frac{\delta_8 - \delta_9}{H} \quad (3.25)$$

$$\text{W-Specimens} \quad \theta_j = \left(\frac{\delta_{15} + \delta_{13} - \delta_{12} - \delta_{10}}{2A} \right) - \frac{\delta_8 - \delta_9}{H} \quad (3.26)$$

The rotation of the column is the difference between the joint rotation and the panel zone deformation:

$$\theta_c = \theta_j - \gamma \quad (3.27)$$

For Specimens W2 and W3, where three transducers were placed under the bottom continuity plates, the continuity plate local deformation was measured as follows:

$$\theta_{cp} = \frac{(\delta_{14} - \delta_{11})}{A} - \theta_j \quad (3.28)$$

Finally, the beam rotation was obtained by deducing the column rotation, continuity plate deformation and panel zone shear deformation from the Story Drift Ratio. According to Figure 3.18, it is important

to note that in this case, as with the rotation of the continuity plate, the angle is measured at the column face instead of the intersection between the beam and column centerlines:

$$\theta_b = (\text{SDR} - \theta_c - \gamma) \frac{L_b}{L} - \theta_{cp} \quad (3.29)$$

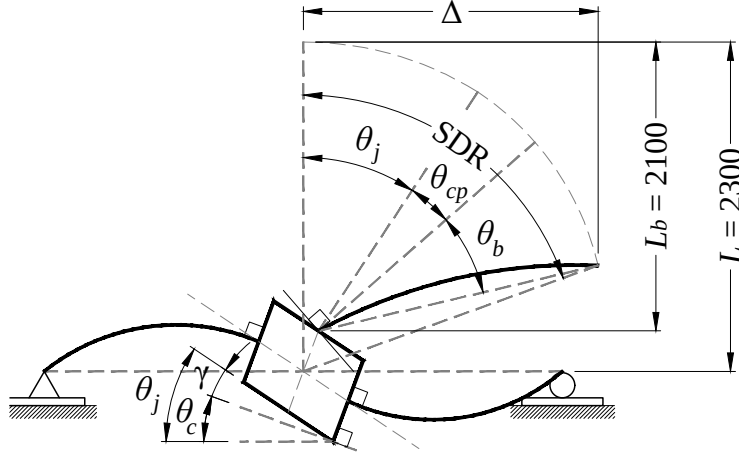


Figure 3.18 Relation of angles for computing beam rotation

The strain gage rosettes were used to compute the principal strains and shear strains at the locations in the continuity plates and beam web.

Using the notation presented previously in Figure 3.16, the horizontal and vertical strain, and the shear deformation angle were, respectively:

$$\varepsilon_x = -\varepsilon_{\theta 1} + \varepsilon_{\theta 2} + \varepsilon_{\theta 3} \quad (3.30)$$

$$\varepsilon_y = \varepsilon_{\theta 1} \quad (3.31)$$

$$\gamma_{xy} = -\varepsilon_{\theta 2} + \varepsilon_{\theta 3} \quad (3.32)$$

And the principal strain and principal directions were calculated as follows:

$$\varepsilon_{1,2} = \frac{\varepsilon_x + \varepsilon_y}{2} \pm \sqrt{\left(\frac{\varepsilon_x - \varepsilon_y}{2}\right)^2 + \left(\frac{\gamma_{xy}}{2}\right)^2} \quad (3.33)$$

$$\theta_1 = \frac{1}{2} \tan^{-1} \left(\frac{\gamma_{xy}}{\varepsilon_x - \varepsilon_y} \right) \quad (3.34)$$

3.10 TEST RESULTS

Figure 3.19 shows the moment at column face versus story drift ratio measured from the six specimens. The three F-Specimens showed identical cyclic behavior. All six specimens developed the beam plastic moment, M_{pb} , based on measured material strength. In the figures, the filled triangle indicates the peak rotation when flange local buckling became evident, and the empty triangle indicates the occurrence of fracture of a continuity plate.

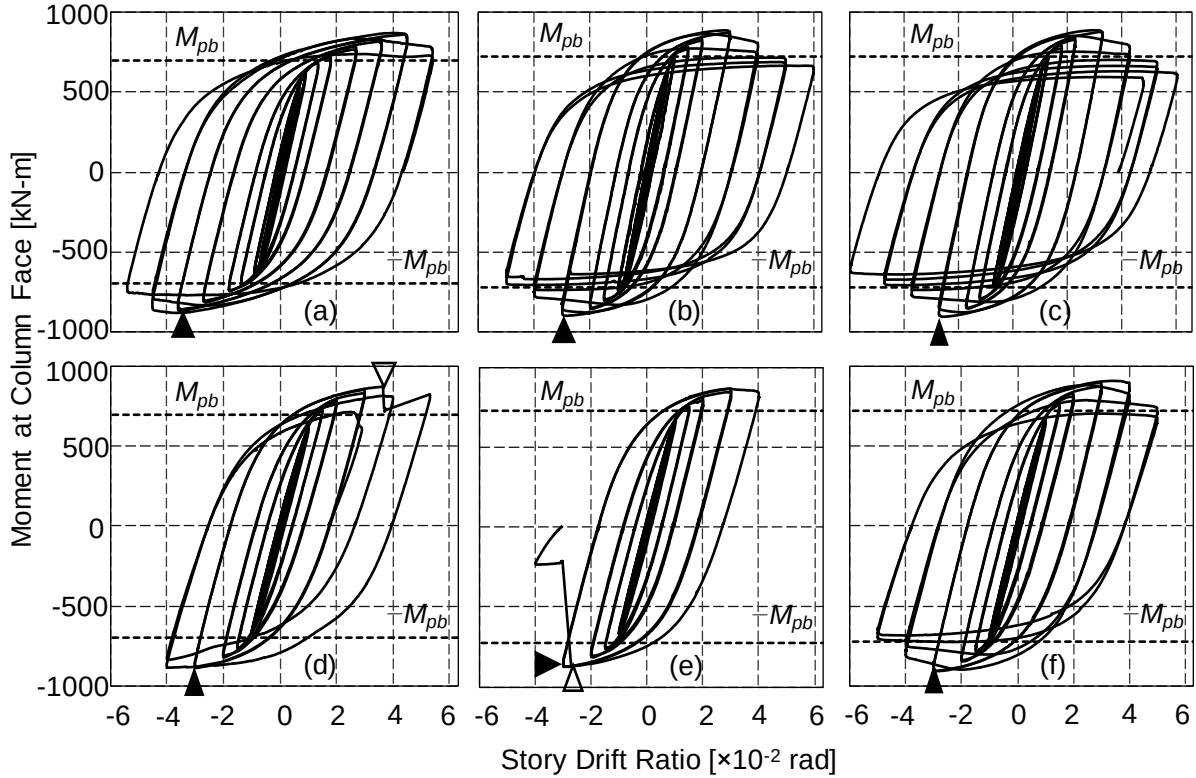


Figure 3.19 Experimental cyclic response of Specimens: a) F1, b) F2, c) F3, d) W1, e) W2, f) W3.

Except for W2, the specimens completed two ± 0.04 -rad story drift cycles while maintaining moment strength above M_{pb} . Therefore, according to the AISC Seismic Provisions [44], the five connections qualify for Special Moment Frames, or highly ductile frames. Specimen W2 could not achieve the high ductility requirement but qualified as Intermediate Moment Frame.

As shown in Figure 3.20a and b, the beams of the F-Specimens and Specimen W3 developed large flange and web local buckling. Cracks formed at the toe of the weld groove at the ends of the beam flange before the occurrence of flange local buckling. As shown in Figure 3.20c, the cracks propagated along the weld groove, but their propagation was mitigated by the strength reduction caused by local flange and web buckling deformation. The W-Specimens were prone to fracture of the continuity plate initiating at the termination of the beam flange groove weld. As shown in Figure 3.21a, the continuity plates of Specimen W1 developed cracks to about half the width of the beam flange. Specimen W2 failed by sudden and complete fracture, as shown in Figure 3.21b, after the continuity plate crack developed to about 20% of the width of the beam flange. Although Specimen W3 also formed cracks at the corner between the beam flange and continuity plate, they did not grow substantially.

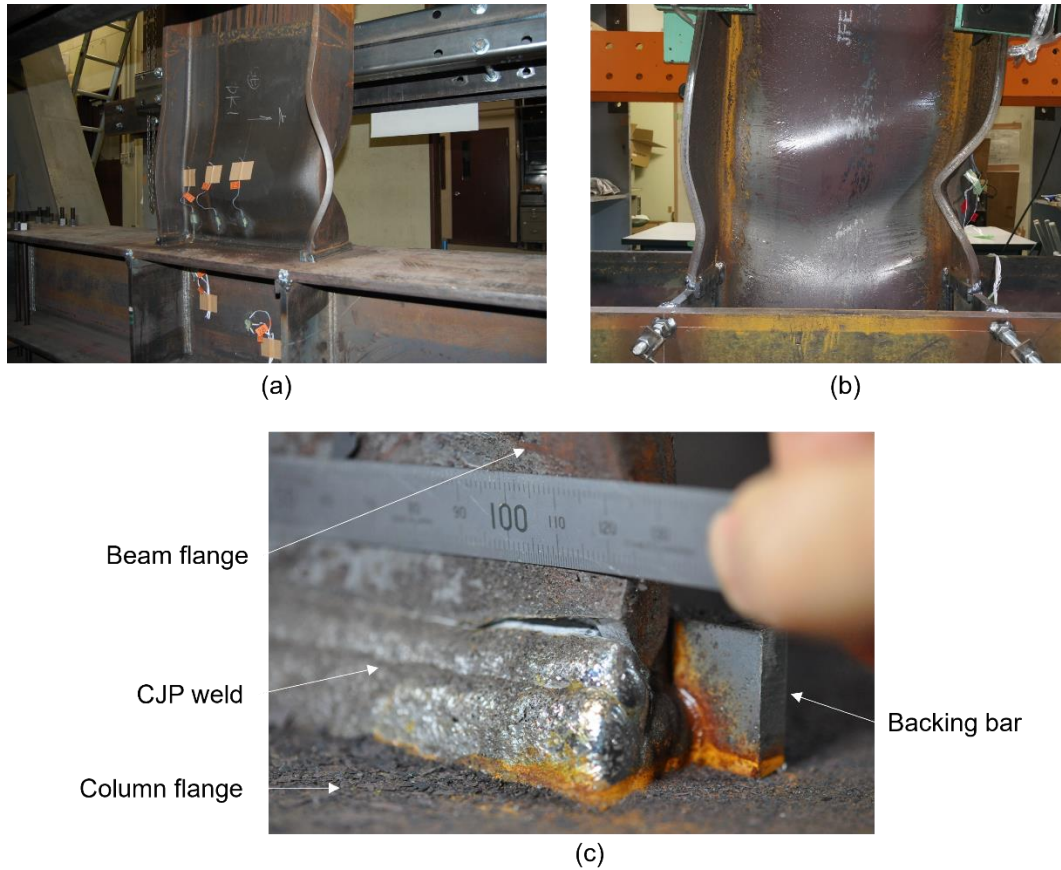


Figure 3.20 Failure mode common to the F-Specimens and Specimen W3: a) Local buckling of F1; b) Local buckling of W3, and c) Crack at the groove weld termination of F3.



Figure 3.21 Fracture in the continuity plates of Specimens: a) W1, and b) W2.

Flaking of the mill scale and notable deformation of the continuity plates of Specimens W1 and W2 suggested that the continuity plates of these Specimens, which had lower yield strength than the beam flange (293 N/mm^2 opposed to 321 N/mm^2) yielded while the continuity plates in Specimen W3, which had higher yield strength than the beam flange (412 N/mm^2 opposed to 321 N/mm^2) did not yield as substantially. Consequently, the difference in performance between the three specimens may be attributed to yielding of the continuity plates yielded during the test.

3.11 NUMERICAL SIMULATION

Finite-element-method analysis was conducted using the general-purpose analysis software ADINA Ver.9.6 [46]. Figure 3.22 shows the general features of the analysis models, including the loading and boundary conditions. The roller support allows translation in the z-direction and rotation about the x-axis, while the pin only allows rotation about the x-axis. All the other degrees of freedom were restrained at those points.

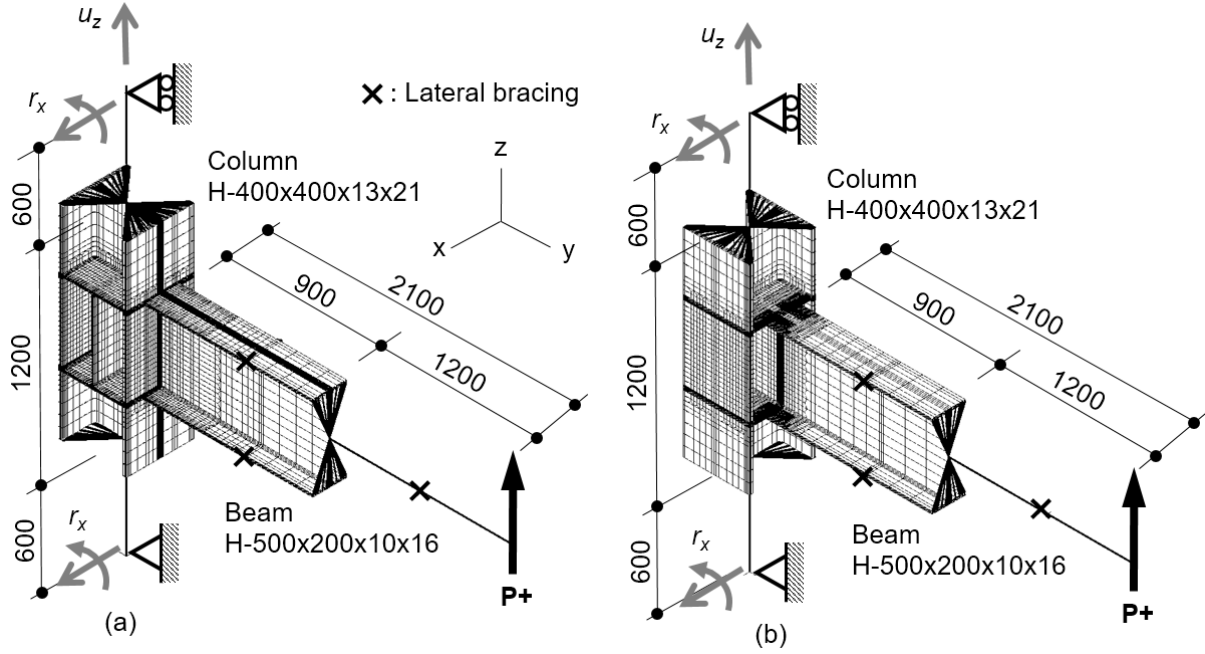


Figure 3.22 Finite element model overview: (a) F-Models; (b) W-Models.

3.11.1 Finite element models

Six models, corresponding to the F- and W- Specimens, were compared in this study. The models had the same geometry and material properties as their corresponding test specimens.

The primary components were modeled with 3-D solid 8-node isoparametric elements with large displacement formulation and $2 \times 2 \times 2$ Gauss integration points. The beam and column segments that were expected to remain elastic were modeled with 3-D 2-node beam elements, represented by single straight lines in Figure 3.22. At the boundaries between solid elements and beam elements, all degrees of freedom at the interface of the solid section were slaved to the end node of the beam element.

Figure 3.23 shows the modeling of the lateral bracing provided to the beam. Single-node linear springs were used to provide lateral bracing to the beam flange by slaving the surfaces within 25-mm width to an external master node. The spring constant was calculated according to the stiffness provided by the lateral bracing system. The combined action of both springs placed at the top and bottom flange provided torsional support when the beam cross section twisted. The lateral bracing for the line element, on the other hand, was a combined linear and torsional spring (spring constants for x-displacement and y-rotation), as shown in Figure 3.23(b). The constants used for the linear and torsional spring were $k_x = 36\,800$ N/mm and $k_{\theta y} = 6.62 \times 10^9$ N-mm/rad, respectively.

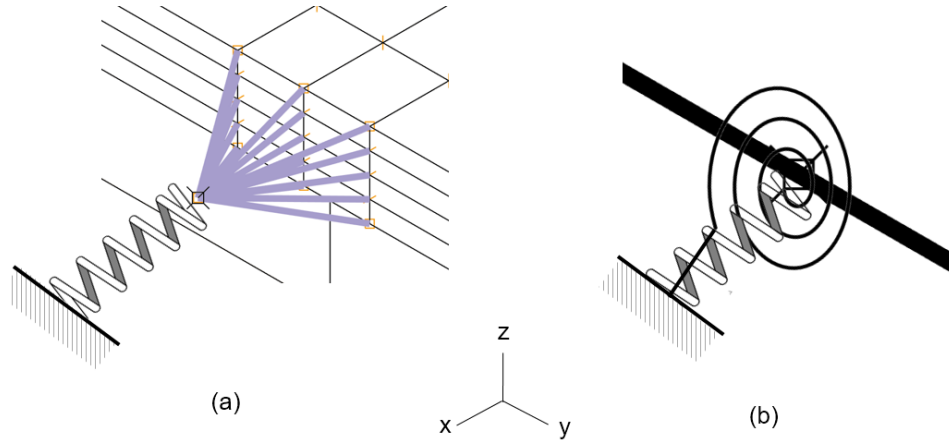


Figure 3.23 Lateral support at: a) 3D-beam flange elements, and b) linear beam element

3.11.2 Material model

A plastic-cyclic material model combining a bilinear isotropic model and a two-term Armstrong Frederick kinematic hardening model was assigned to the 3-D solid elements. The material parameters were calibrated against monotonic and cyclic coupon test data and summarized in Table 3.4. The 2-node beam elements were linear elastic, so the linear elastic properties sufficed for them.

Table 3.4 Plastic-cyclic material model parameters

Isotropic (bilinear)				Kinematic (Armstrong-Frederick)		
Rule No.	Material	σ_{y0} (N/mm ²)	E_p (N/mm ²)	Rule No.	h_i	ζ_i
1	SN490C	270	90	1	30 000	300
2	SN490C	295	70		2000	15
3	SN400B	200	70			
4	SN400B	260	80			
5	SN490B	250	90	2	60 000	400
6	SN490B	275	40		2000	25
7	SN400B	195	70			
8	SN400B	240	10			
9	SN400B	200	70			
Poisson's ratio = 0.30			Young's modulus = 205 000 N/mm ²			

All the combination of hardening rules performed in ADINA to model the material of each component of the analytical models are listed in Table 3.5.

Table 3.5 Combination of hardening rules for each component modeled in ADINA

Component	F1		F2		F3		W1		W2		W3	
	Iso.	Kin.	Iso.	Kin.	Iso.	Kin.	Iso.	Kin.	Iso.	Kin.	Iso.	Kin.
Column Flange	1	1	1	1	5	1	1	1	1	1	5	1
Column Web	2	1	2	1	6	1	2	1	2	1	6	1
Beam Flange	3	1	9	2	9	2	3	1	9	2	9	2
Beam Web	4	1	8	1	8	1	4	1	8	1	8	1
Continuity PL	3	1	3	1	5	1	3	1	3	1	5	1
Doubler PL	5	1	5	1	5	1	5	1	5	1	5	1

The monotonic tests presented earlier in this Chapter were used to calibrate the parameters for most components, and the two cyclic coupon tests to perform cyclic calibration for the beam flange material of Models W2, W3, F2 and F3. It is important to mention that due to the issues related to buckling, cyclic coupon tests are seldom available for conducting the calibration, or are based on very short coupons. The restrainer device used for the cyclic coupon tests of this study permitted us to obtain valuable cyclic data up to strain amplitudes of ± 0.04 mm/mm.

3.11.3 Initial imperfection

In the numerical models, the consideration of initial imperfections is crucial to reproduce the flange and web local buckling observed in the experiments. Therefore, a two-stage analysis was performed in ADINA to consider initial imperfection. The first analysis, starting from zero initial conditions, is performed to obtain the first buckling mode of the structure through a Linearized Buckling Analysis [46]. Once ADINA determines the buckling shape, the node with maximum displacement is considered as the reference node for the second stage.

The second stage starts by applying to every node a stress-free change of coordinates to match the buckling mode shape. The maximum displacement is then scaled to a desired value. In this study, the imperfection was scaled so that the maximum imperfection is 2.1 mm, or 1/1000 of the length of the beam. This value was chosen because it represents the out-of-straightness tolerance according to ASTM A6.

Figure 3.24 shows an example of the plan view first buckled shape of an F-Model.

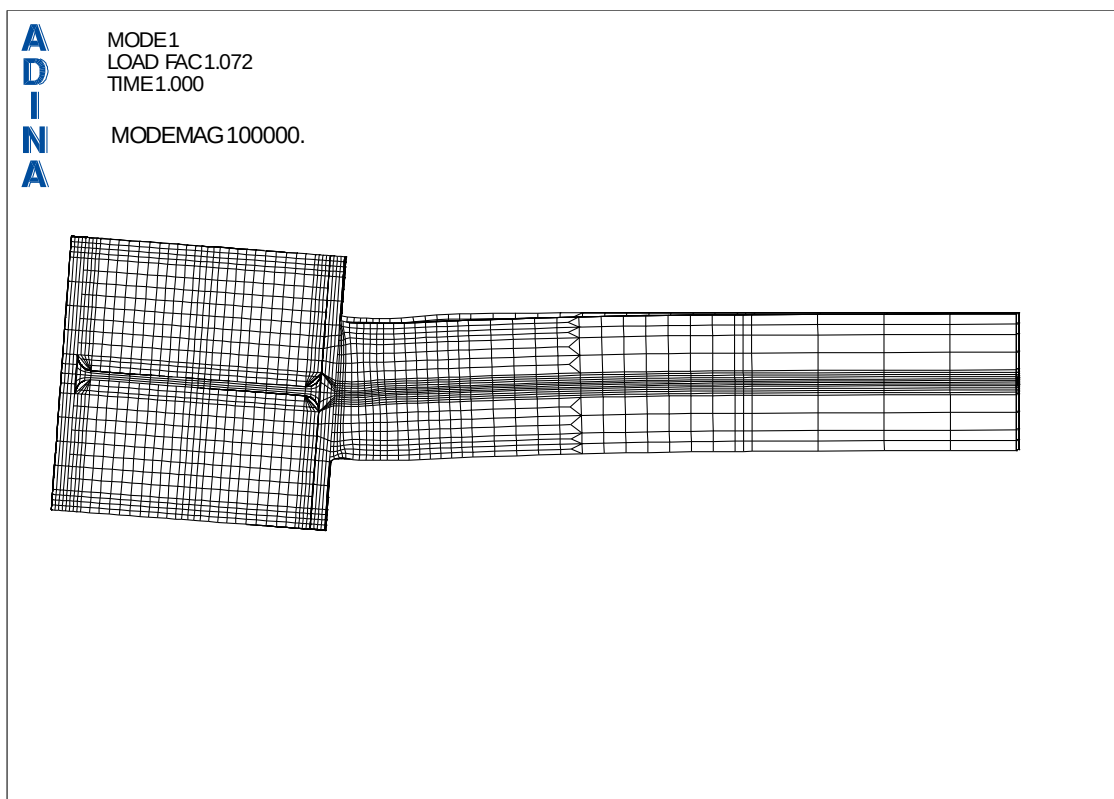


Figure 3.24 First buckling mode of Model F1.

3.11.4 Validation

Figure 3.25 shows the global response of the three analytical web models superimposed to the corresponding experimental results. The models agreed well with the experiments up to the 0.03-rad cycles, before the occurrence of strength and stiffness degradation induced by fracture and local

buckling deformation.

Figure 3.26 shows the global response of the three analytical flange models superimposed to the corresponding experimental results. The models agreed could capture the strength degradation beyond the 0.04-rad cycles, but after the first cycle of that amplitude was completed.

However, in all cases, the range of agreement is acceptable for the purposes of this study.

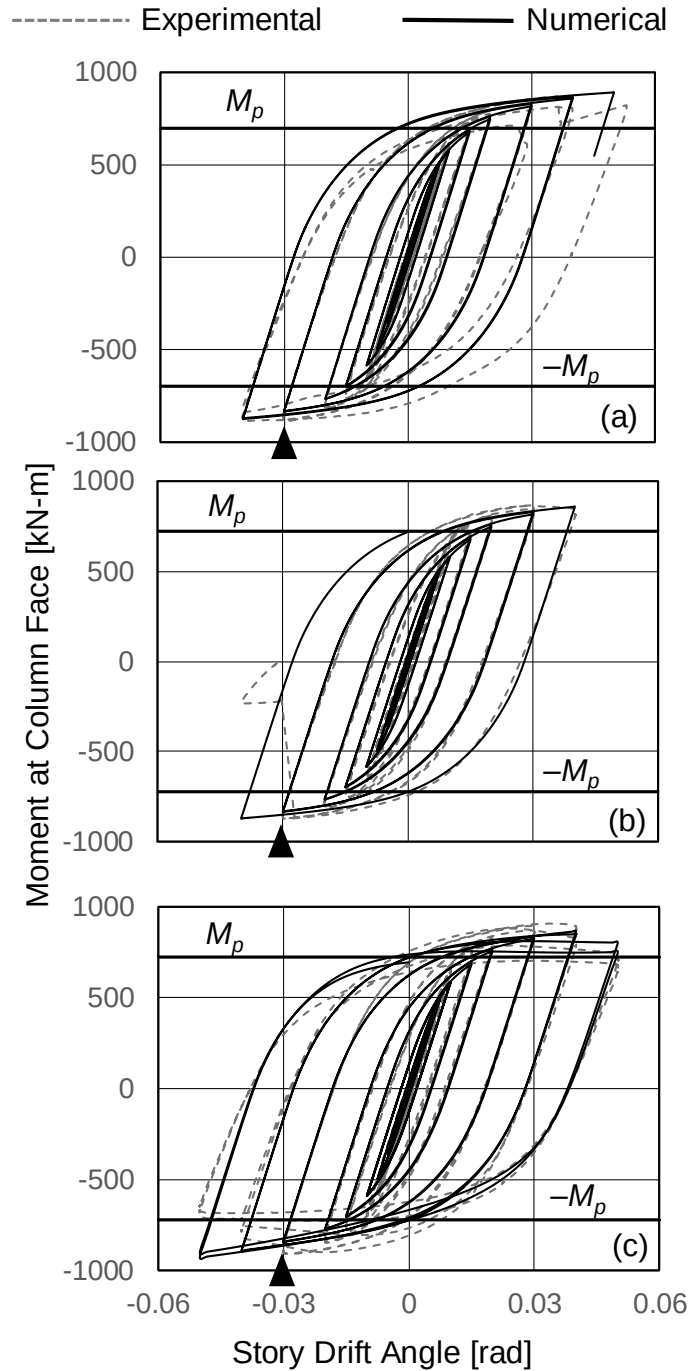


Figure 3.25 Response validation of: a) Model W1, b) Model W2, and c) Model W3.

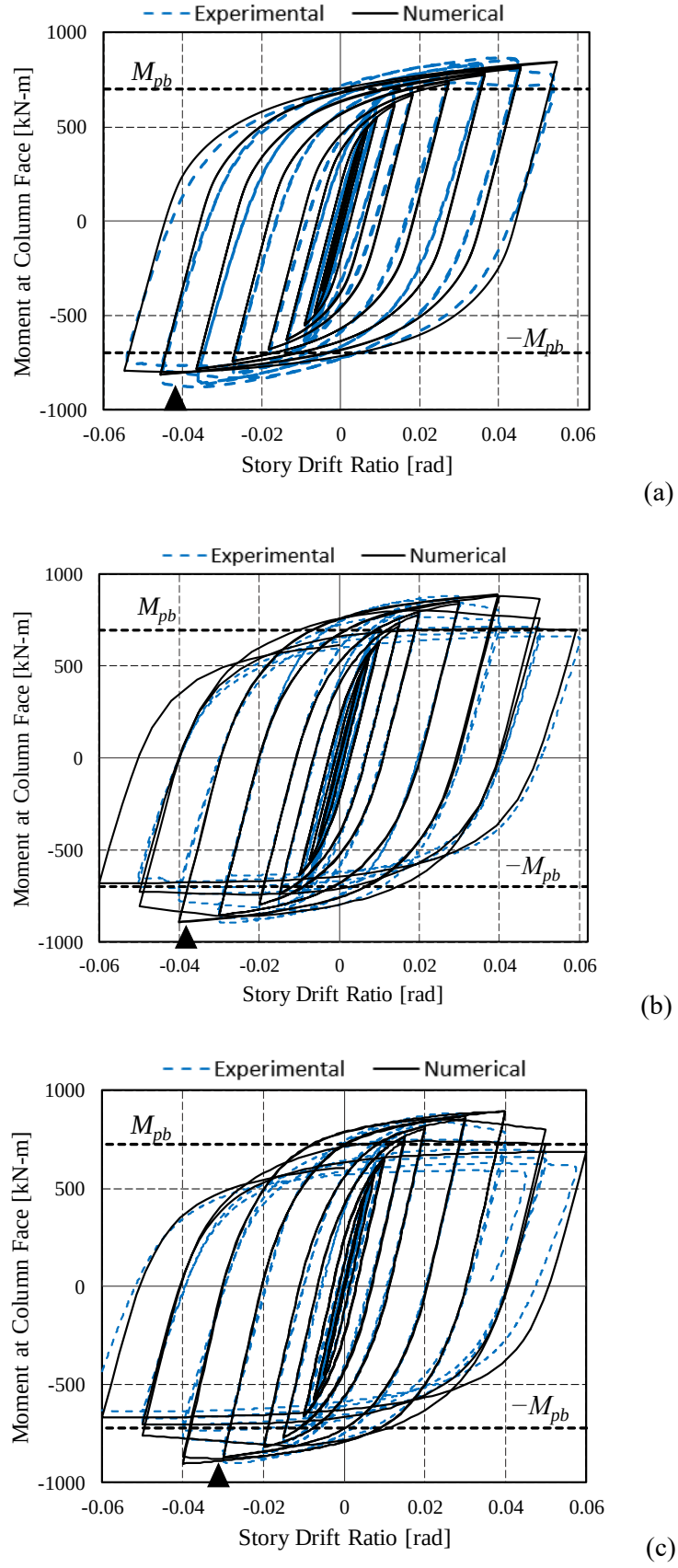


Figure 3.26 Response validation of: a) F1, b) F2, and c) F3

3.12 DISCUSSION OF THE EXPERIMENTAL AND NUMERICAL RESULTS

3.12.1 Strength and Deformation Capacity

Table 3.6 summarizes key results regarding strength and deformation capacity, and the evaluation according to the AISC Seismic Provisions [44]. The bending moment at the column face divided by the M_{pb} based on the properties listed in Table 1 is listed for key stages: the maximum value and at first peak positive and negative story drift at amplitudes ± 0.02 and ± 0.04 rad. Except for Specimen W2, which failed due to fracture of the continuity plate before completing the first -0.04 -rad excursion, all specimens developed similar strength. The maximum measured bending moment was $1.21M_{pb}$ for Specimen W2 and between 1.21 and $1.28M_{pb}$ for the other five specimens. The specimens other than Specimen W2 maintained a bending moment above M_{pb} at the end of the second cycle of ± 0.04 rad. Consequently, according to the acceptance criteria specified in the AISC Seismic Provisions for Structural Steel Buildings, Specimen W2 may qualify for Intermediate Moment Frames (IMFs) while the other five specimens may qualify for Special Moment Frames (SMFs), which is the system with highest ductility among the steel moment resisting frames.

Table 3.6 Key test results and observations

Spec.	Failure mode	M/M_{pb}			Completed amplitude [rad]	Potential AISC 341 classification
		Max.	± 0.02 rad	± 0.04 rad		
W1	Fracture of continuity plate	1.28	1.11/-1.18	1.16/-1.27	0.05	SMF
W2	Fracture of continuity plate	1.21	1.09/-1.11	1.15/N.A.	0.03	IMF
W3	Flange and web local buckling	1.25	1.12/-1.17	1.23/-1.13	0.05 min.	SMF
F1	Flange and web local buckling	1.26	1.01/-1.05	1.18/-1.21	0.05 min.	SMF
F2	Flange and web local buckling	1.24	1.13/-1.17	1.11/-1.24	0.05 min.	SMF
F3	Flange and web local buckling	1.25	1.15/-1.16	1.08/-1.06	0.05 min.	SMF

Comparing the global responses of the F-Specimens in Figure 3.19a-d, strength degradation of Specimens F2 and F3, started one loading amplitude earlier than F1, which was the only major-axis connection with panel zone not reinforced with a doubler plate. Roeder reported that improved ductility may be obtained from combined beam flexural yielding and panel zone shear deformation, provided the former precedes the latter and the amount of panel zone inelastic strain is controlled [47].

3.12.2 Stiffness

The reference angles used to compute the flexural responses of the total subassembly, the beam, the panel zone, and the beam-to-column joint were shown in Figure 3.18. Table 3.7 lists the stiffness values derived from each force-rotation response in the elastic range. The estimated global stiffness of the specimens was calculated as detailed in Sec. 3.7. The stiffness of the panel zone of W-Specimens was represented by the average obtained from the two-column flanges. Also presented in the table are the estimated total stiffness that assumed pure flexural deformation of the beam and column, and pure shear deformation of the panel zone. The difference in estimated total stiffness between the F-specimens and W-specimens was small despite the 3-times difference in moment of inertia of the column because the estimated stiffness was dictated by the beam and panel zone, and because the panel zone of W-Specimens had 1.6 times the stiffness of the F-specimens. For all specimens, the estimated global stiffness agree very well with the measured global stiffness during the elastic cycles in the tests.

Among the three F-Specimens, Specimens F2 and F3, which had doubler plates welded to the panel zone, was 1.5 times stiffer in the panel zone compared to Specimen F1, which did not have a doubler plate. The plug-welds in the panel zone, which were made for Specimen F3 but not for F2, had negligible effect on the stiffness of the panel zone. In the case of W-Specimens, the presence of a doubler plate had negligible effect on the stiffness of any component.

Table 3.7 Stiffness of each component [$\times 10^3$ kN-m/rad]

Specimen	F1	F2	F3	W1	W2	W3
Total (estimated)	83	93	93	76	76	76
Total	80	86	93	72	78	77
Beam	116	127	125	115	125	123
Panel zone	228	353	368	469	392	483
Joint	117	231	271	186	172	174

3.12.3 Failure mode

Based on the response described in Sec. 3.10, the observed failure modes presented in that section are regarded as the primary cause of strength reduction. Strength reduction was observed when crack propagation or excessive local buckling happened. The W-Specimens were prone to fracture of the continuity plate initiating at the termination of the beam flange groove weld. The same failure mode was reported by Popov and Pinkney [9] and Tsai and Popov [12] for beam-to-column web connection whose continuity plates ended near the edge of the column flanges. The fracture reported in those researches propagated into the beam flanges. Somewhat unlike past studies, the W-Specimens exhibited gradual crack growth into the continuity plate at an inclination of about 60° with respect to the beam axis. Specimen W2 failed by sudden and complete fracture after that crack developed to about 20% of the width of the beam flange. Specimen W3 formed a very similar crack at the corner between the beam flange and continuity plate, but the cracks did not grow to a substantial size. Specimen W3 formed cracks along the groove of the beam flange, very much like the F-Specimens, which grew to a larger size than the cracks into the continuity plate. The presence of a vertical stiffener between continuity plates in the opposite side of the web-moment connections did not affect the performance of the connection, but the grade of steel of the continuity plates proved to be significant: the two specimens with weaker continuity plate exhibited either extreme crack growth or complete fracture. The difference in steel grade of the column had no effect.

The three F-Specimens and Specimen W3 exhibited large local buckling deformation of the beam flanges and web. Prior to local buckling, cracks were detected in the groove weld joining the beam flanges to the column or the continuity plates. As shown in Figure 3.20c, the cracks formed at the toe of the weld groove at the ends of the beam flange. In Specimen F1, the only F-Specimen without doubler plate, the cracks hardly grew. In Specimens F2, F3 and W3, the cracks propagated straight along the weld groove, or the interface between the CJP groove weld and beam flange. Fracture initiating at the toe of the weld groove at the ends of the beam flange is a very typical failure mode for welded beam-to-column moment connections, particularly when the no weld access hole details is adopted [4]. It is believed that the strength reduction caused by local buckling was the primary reason why these cracks did not propagate in the specimens examined in this program. Specimens F2 and F3, whose panel zones were prevented from yielding, experienced larger crack growth compared to Specimen F1, which yielded in both the beam and panel zone.

3.12.4 Fracture of the continuity plate

The beam-to-column web connections were prone to fracture initiating at the CJP groove weld between the beam flange and continuity plate. As suggested by Driscoll and Beedle [11], such failure may be

controlled by extending and tapering the continuity plate beyond the column flanges. However, such solution is unfavorable to the current Japanese practice, where the beam flanges are milled for the same length as the continuity plate.

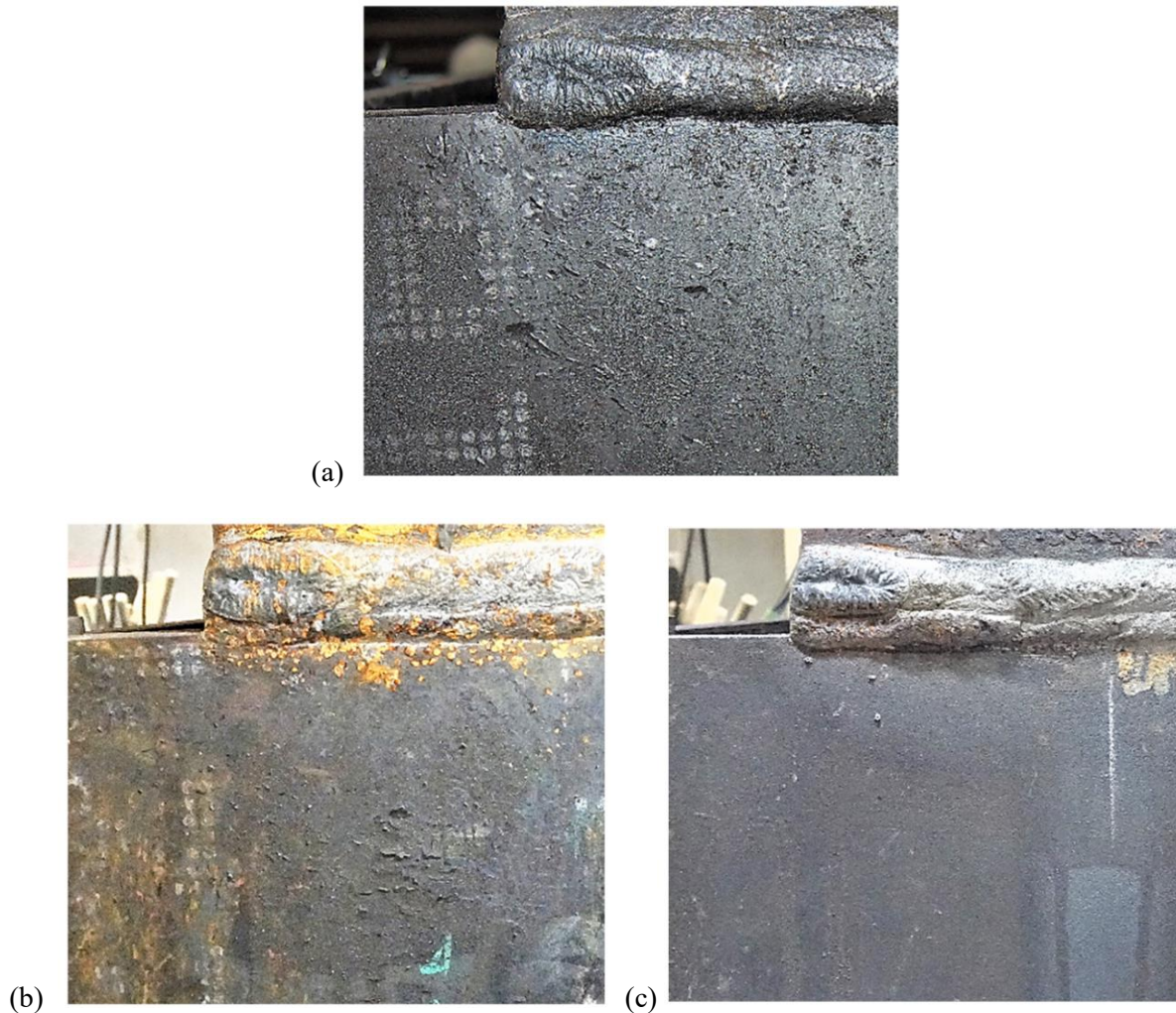


Figure 3.27 Continuity plates of the W-Specimens at 0.03 rad story drift: a) W1, b) W2, and c) W3

Figure 3.28 compares the principal strains in the continuity plates measured by strain gauges of specimens W2 and W3, 30 mm away from the groove weld connection. The primary difference between the two specimens was the steel grade used for the continuity plates: Specimen W2 used SN400B steel (same as W1), while Specimen W3 used SN490B steel. Similar to the visual evidence in Figure 3.27b and c, Figure 3.28 shows that the continuity plates in Specimen W2 experienced larger inelastic deformations than in Specimen W3. At ± 0.02 -rad and beyond, the strain at the sampled location grew much more substantially in Specimen W2, particularly in the diagonal direction to the beam axis. These results suggest that the right corner between the beam flange and continuity plate, in combination with the cracks formed at this location, acts as a notch placed orthogonal to such principal strain.

The continuity plates and beam flanges had the same thickness. Therefore, it is believed that the continuity plates in Specimen W2, which had lower yield strength than the beam flange (293 N/mm^2 against 321 N/mm^2) yielded while the continuity plates in Specimen W3, which had higher yield strength than the beam flange (412 N/mm^2 against 321 N/mm^2) did not yield as substantially. In consequence, the significantly different performance between Specimens W2 and W3 may be attributed to whether the continuity plates yielded during the test.

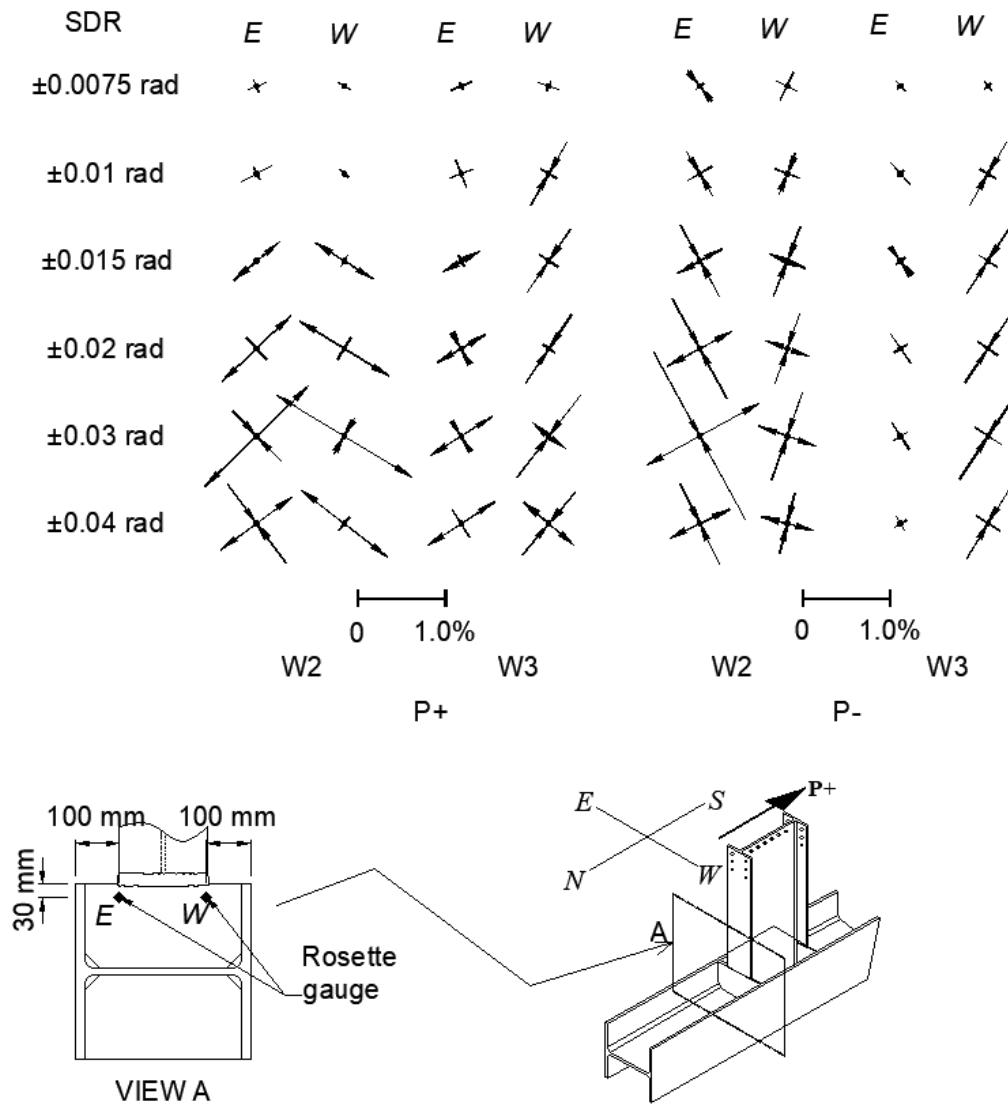


Figure 3.28 Measured principal strains in the continuity plates of W1 and W2.

The comparison between Specimens W2 and W3 suggests that beam-to-column web connections should be provided with continuity plates that are stronger than the beam flanges. If the above requirement is met, satisfactory performance may be achieved even when the continuity plates are flush with the column flanges, as shown by Specimen W3.

Figure 3.29 shows the principal stresses near the critical regions of the W-models, when the beam flange is subject to tension at the first excursion of the ± 0.03 -rad cycle. At this stage, marked by “▲” in Figure 3.22, the cracks described previously for Specimens W1 and W2 propagated rapidly into the continuity plates.

For all three numerical models, notably high stress was produced at the corner between the beam flange and continuity plate, at the same location where all three specimens formed a crack. It is also noted that the large principal stress acted perpendicular to the corner, to open any crack formed at this location, as reported for Specimens W1 and W2. Although little difference in stress distribution was confirmed between the three models, the maximum principal stress was higher in Model W2 (793 N/mm²) than in Models W1 and W3 (772 and 770 N/mm², respectively).

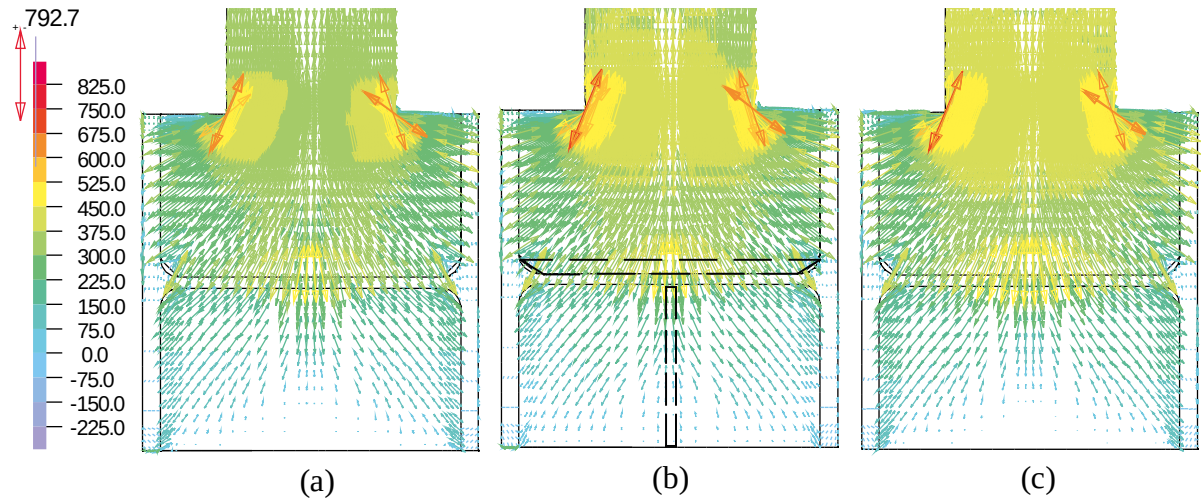


Figure 3.29 Principal stress distribution at 0.03-rad story drift: a) W1; b) W2; and c) W3

Figure 3.30 shows the distribution of principal stresses in the F-Models at 0.03-rad story drift. The level of stress is lower than the W- models, and the stresses tend to flow toward the middle of the beam flange. It is important to note, though, that the magnitude of the principal stresses remain high along the full length of the groove weld.

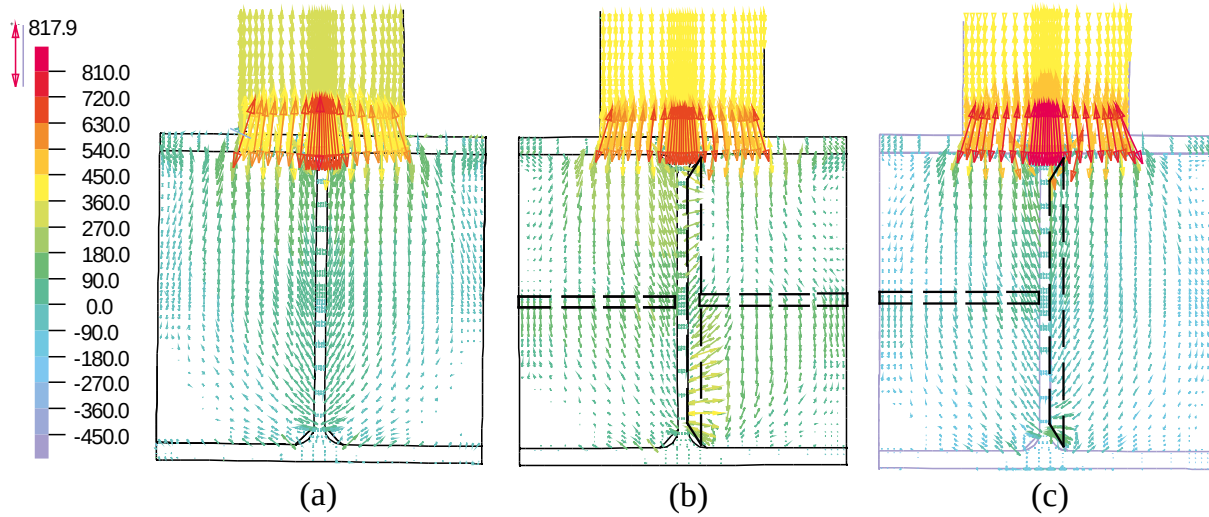


Figure 3.30 Principal stress distribution at 0.03-rad story drift: a) F1; b) F2; and c) F3

3.13 CHAPTER SUMMARY

Six beam-to-column subassemblies with dimensions typical of a 4-story steel building in Japan were tested to examine their cyclic-loading performance and evaluated according to the AISC Seismic Provisions. The specimens consisted in three connections to the column flange and three to the column web, constructed and detailed according to the current Japanese practice. The most relevant features of the Japanese construction practice incorporated in the specimens were the shop-welded connections and the no-weld-access hole detail.

The specimens exhibited good strength and ductility, except for one connection to the column web that fractured completely in the continuity plate.

Finite Element Method analysis was conducted to further examine the causes of the observed failure and possible ways to mitigate it. An analytical model was constructed for each specimen in ADINA v.9.6, using material models with parameters calibrated from monotonic and cyclic coupon tests. The Finite Element models captured well the measured response of the tests specimens up to 0.04-rad cycles.

The main findings of this study are presented below:

- When brittle fracture of the continuity plates was avoided, the beam-to-column web connections were as ductile as the beam-to-column flange connections and satisfied the high ductility requirements presented in the AISC Seismic Provisions.
- Beam-to-column web connections were prone to fracture of continuity plates initiating at the corner between the beam flange and the continuity plate. This failure mode was promoted by yielding of the continuity plates. Therefore, yielding of the continuity plates should be controlled by either adopting for the continuity plates a stronger material and/or thicker plate than the beam flanges.
- Beam flange and web local buckling were the predominant failure modes of the beam-to-column flange connections and one web connection. The beam mechanism formed in those specimens was according to the expected failure mode determined by the design plastic strength ratios. Although cracks appeared in the termination of the groove welds of the specimens with local buckling failure mode, the connection was protected because the inelastic rotations were limited to the beam plastic hinge.
- The beam-to-column flange moment connection with inelastic rotation demands shared between beam flexural and panel zone shear yielding experienced strength degradation later than the connections with beam flexural yielding only. The panel zone was, however, strong enough to force the beam to reach its plastic strength prior to fully plasticization of the panel zone.
- The shop-welded, no-access-hole detail for beam-to-column moment-resisting connections, which are commonly adopted in Japan, is a detail not included in the prequalified connections of ANSI/AISC 358 but satisfied the high ductility requirements presented in the AISC Seismic Provisions.
- Little difference in stiffness was measured between specimens with beam-to-column web connections or beam-to-column flange connections because the stiffness was dictated by the beam and panel zone rather than the 3-times difference in the moment of inertia of the column.
- The observed behavior of Specimen W3 suggests that the quality of the welding process, in combination with having the beam web directly welded to the column web and the use of a stronger continuity plate were important factors in achieving a good seismic performance of the minor-axis moment connections even when continuity plates flush to the column flanges were used.

4. SHAKE-TABLE COLLAPSE TESTS: TEST PLAN

4.1 INTRODUCTION

Shake-table tests of a steel moment-resisting frame were conducted to obtain detailed data on the occurrence and propagation of damage leading to the collapse of steel structures. A 2/5-scale, 4-story, 2-bay, planar specimen was subjected to in-plane, unidirectional ground motions. The specimen was intended to: a) sustain large deformations, or story drift ratio greater than 0.10 rad, b) follow the strong-column/weak-beam criterion, and c) avoid a first-story collapse mechanism. The structural analysis and design calculations are presented in Appendix A.

The specimen comprised I-section beams and square-HSS columns. As in typical Japanese construction, the Complete Joint Penetration (CJP) groove welds between the beam flanges and column diaphragm plates were completed with no weld-access-hole. The ground motions included the JMA Kobe NS record scaled from 10 to 100% and white noise excitations. The 100%-scale record was applied four times.

4.2 TEST SPECIMEN

4.2.1 Specimen Description

4.2.1.1 Main frame

The specimen was a 4-story, 2-bay planar steel moment-resisting frame that comprised I-section beams and square-HSS columns. Figure 4.1 shows the elevation and key dimensions of the moment-resisting frame. Table 4.1 lists the member schedule, steel grade, and minimal specified mechanical properties. Similarly to the current Japanese construction practice of low and mid-rise steel buildings, the frame was constructed following the “column-tree” configuration. In this construction practice, beam stubs are welded to the columns in the shop and then transported to the site, where the frame is completed by splicing the remaining 1500-mm beam segment with high-strength slip-critical bolts.

The bases of the columns were designed as fixed. Each column was connected to the foundation beams using 6 high-strength M22 bolts. The four corner bolts were fastened to the foundation beam. However, inserting bolts vertically into the foundation beam web was not possible. Therefore, the two inner bolts were screwed into tap holes of a 32-mm plate welded to the foundation beam. The two fillet welds used to connect the plate to the foundation beam were perpendicular to the loading direction to prevent slippage.

To prevent twisting of the columns on each floor, the top through diaphragm at each column joint was connected to the top flange of the transverse beams using two M12 bolts.

4.2.1.2 Beam-to-column moment connection

Figure 4.3 shows the beam-to-column moment connection detail used in all joints of the moment frame specimen. The beam web was welded to the columns through fillet welds and the flanges joined to the column through-diaphragm using Complete Joint Penetration (CJP) welds. To complete the CJP welds with no weld-access-holes, the backing bar was divided in two parts at the beam web.

4.2.1.3 Gravity frames

Two frames (lines A and C) were constructed parallel to the main frame (line B) to support the concrete weights. The frames were intended to support gravity load only and to have no participation in the system's lateral resistance. To achieve this, the bases of the columns were pinned to the foundation beams with clevises that allowed rotation in the plane of the frame. In addition, the beams were provided with slotted holes that allowed slanting of the columns.

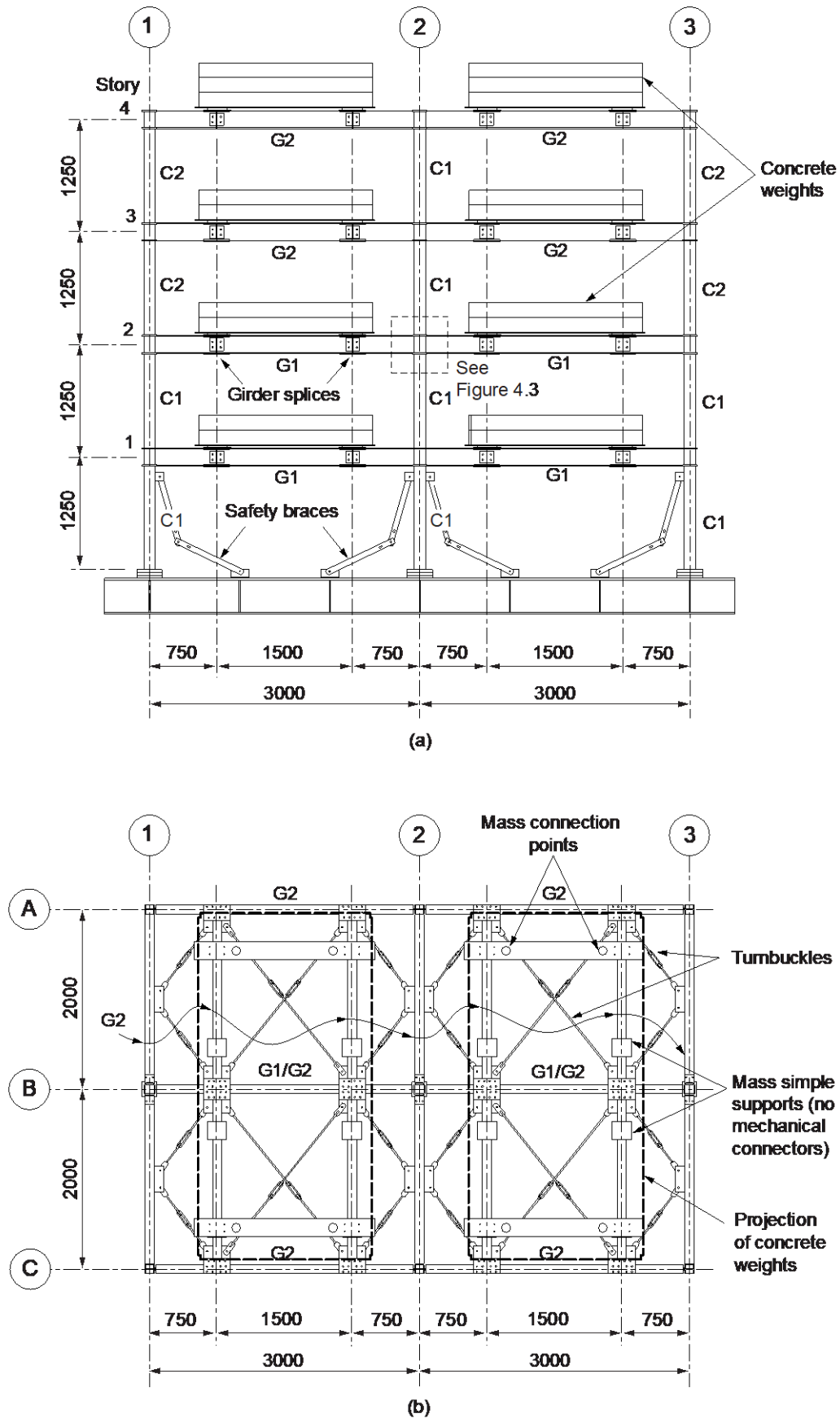


Figure 4.1 Test specimen: a) elevation of the main frame, b) typical plan view

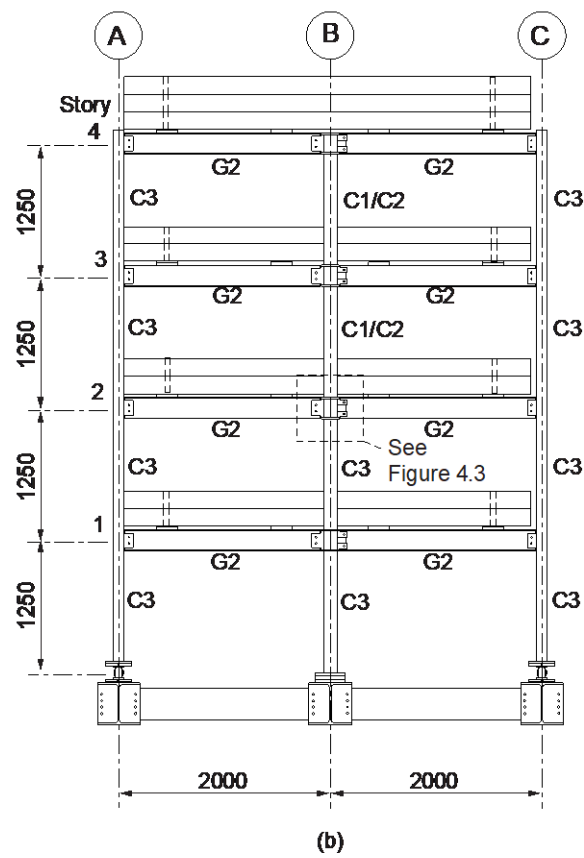
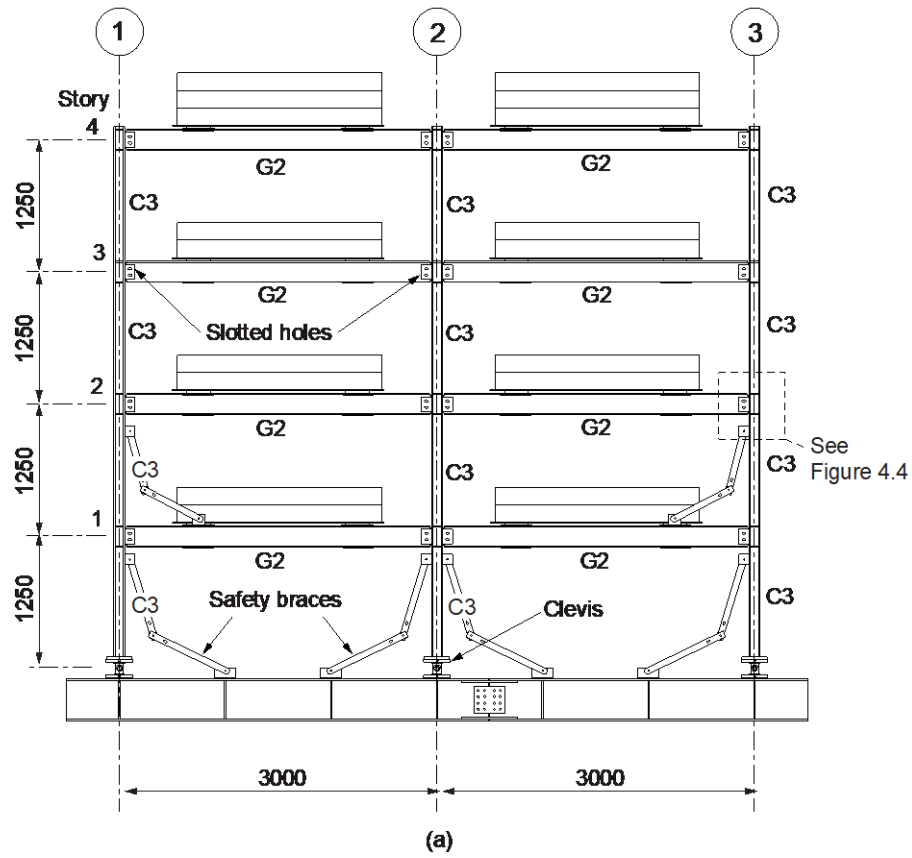


Figure 4.2 Elevation of a) gravity frames A and C, and b) transverse beams connecting the gravity and the main frame.

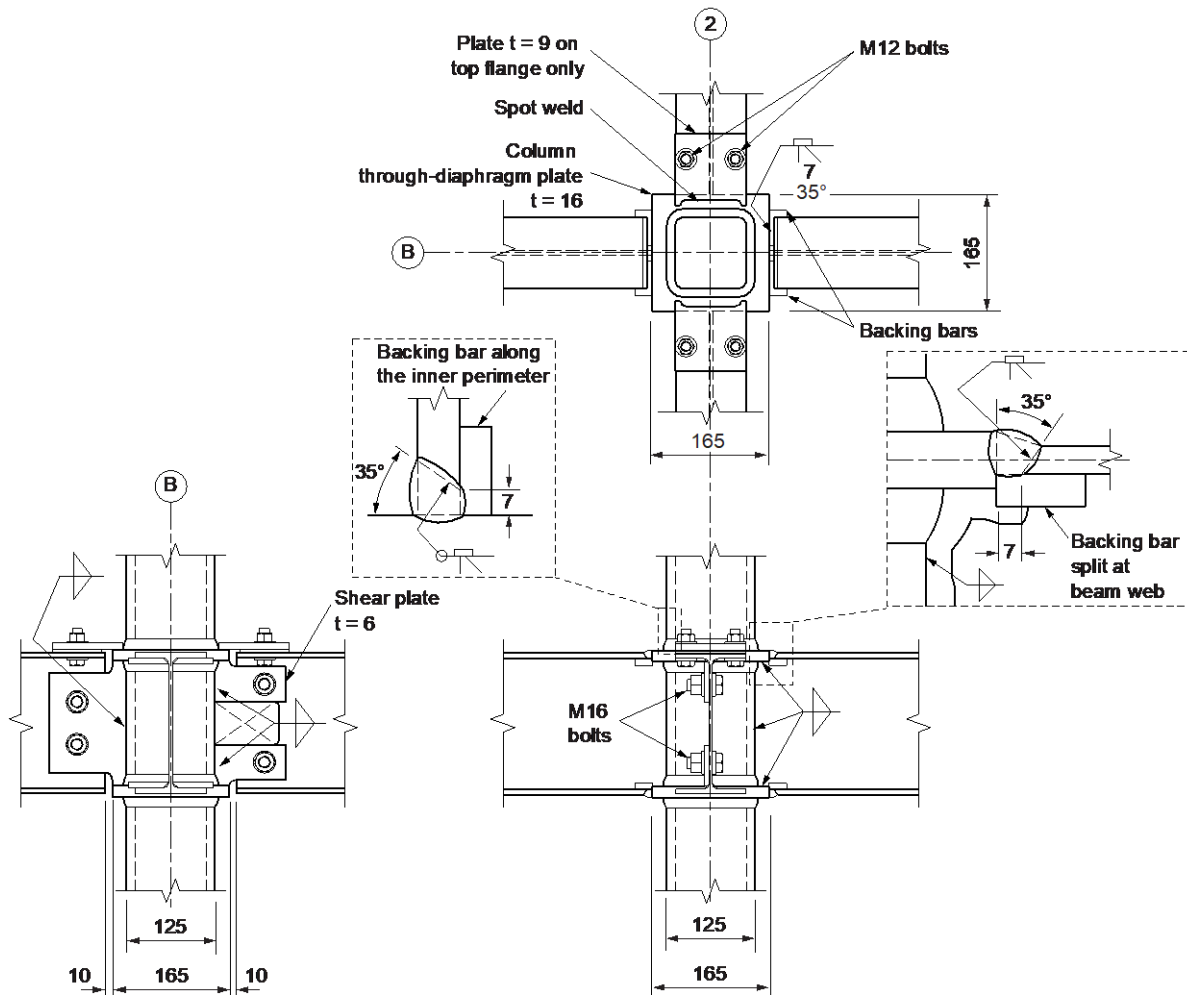


Figure 4.3 Beam-to-column connections in the main frame.

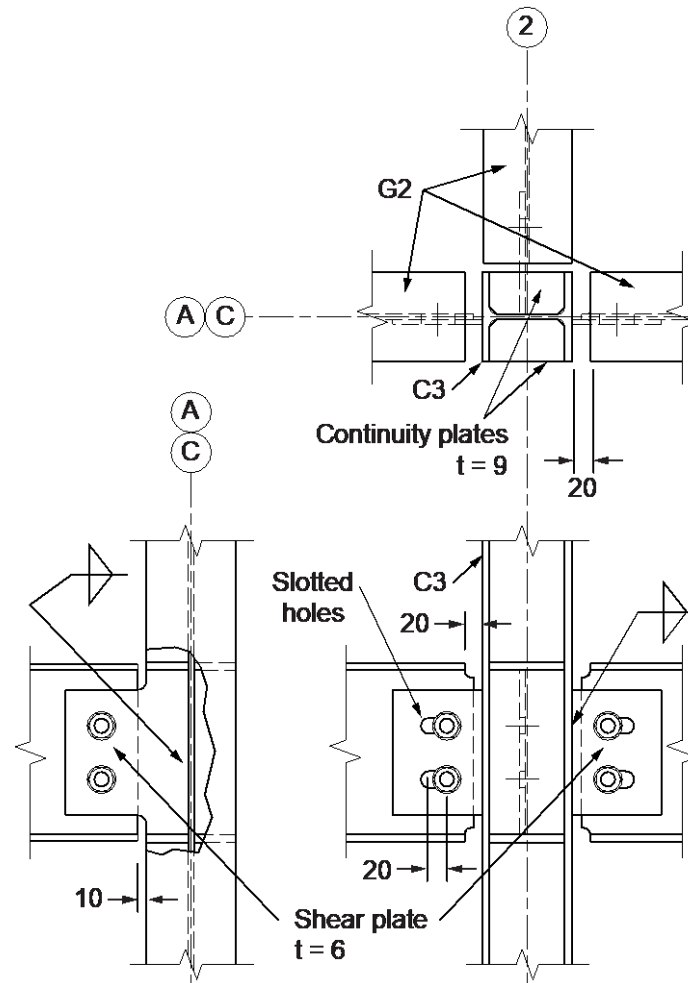


Figure 4.4 Beam-to-column connections in the gravity frames.

Table 4.1 Member schedule and minimum specified mechanical properties.

Member	Dimensions	Grade	F_y [N/mm ²]	F_u [N/mm ²]
C1	HSS-125×125×12	STKR400	245	400
C2	HSS-125×125×9	STKR400	245	400
C3	H-100×100×6×8	SS400	245	400
G1	H-200×100×5.5×8	SS400	245	400
G2	H-198×99×4.5×7	SS400	245	400

4.2.1.4 Mass

The mass of the system comprised concrete weights of 2.8 ton each, totaling 50 ton for the entire specimen. The weights were stacked as shown in Fig. Four weights were placed on each floor except the roof floor, which had six weights to maximize the inertia forces.

The dimensions and weight of each concrete plank are summarized in Table. As shown in Fig. the stacked weights were fastened to the floor beams in four locations using Ø17-mm PC rods with a 70-kN tensile strength each. In the other four locations, the weights rested on bearing plates with no mechanical connection between the concrete and the steel. Shim plates were inserted between the plates and concrete weights to close the gaps in those simple support locations where required.

Table 4.2 Story mass

Story	Mass [ton]			Weight [kN]
	Concrete	Steel	Total	
4	16.8	1.0	17.8	174
3	10.9	1.1	12.0	118
2	11.0	1.1	12.1	119
1	11.0	1.2	12.1	119
Total	49.6	4.4	54.0	529

4.2.1.5 Floor diaphragm

Fig. shows the horizontal diaphragm formed on each floor to transfer all the inertia to the main frame. Turnbuckles were used as tension-only braces in X configuration, and were connected only to the top flange of the floor beams, as shown in Figure 4.5.

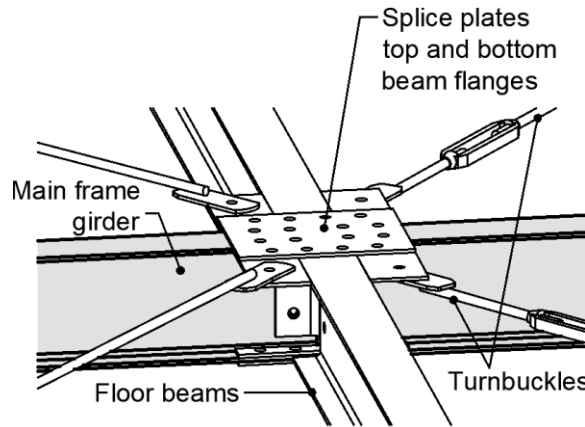


Figure 4.5 Floor diaphragm connection.

4.2.1.6 Safety braces

Sagging braces were placed in the first and second-story columns of the three parallel frames to protect the shake table from the collapse of the test specimen. On the second floor, the braces were attached only to the exterior columns of the gravity frames to allow room for the concrete weights. Additionally, the braces served as erection bracing of the columns during the frame construction. To accomplish this double purpose, the pinned joint between both brace arms was provided with slotted holes, so that the brace was straight at 45° during the construction of the frame, and the joint could be released to the slack configuration for the shake-table tests. At the released configuration, the braces were designed to engage at a Story Drift Ratio of 0.18 rad [10°].

4.2.1.7 Overall view of the Specimen

As mentioned above, the column trees and beam segments were transported to the test facility and then mounted on the shake table. Fig shows two general views of the test specimen before the tests.



Figure 4.6 Elevations of the completed Specimen: a) front, b) side.

4.2.2 Mechanical Properties

4.2.2.1 Coupon sampling

The mechanical properties of the test specimen members were determined from tension coupon tests. Figure 4.7a and b show that two samples were collected from each of the two types of square-HSS columns, and from the flange and web of the two types of beams. Similarly, two coupons from the column through diaphragm were also extracted.

Additionally, Figure 4.7d shows that four coupons sampled the CJP connection between the beam flange and the column through-diaphragm. The objective of the weld coupons was to examine the effect of the tack weld of the backing bar in the failure mode of the connection. As mentioned in Chapter 2, tack welds may affect the outcome of reduced-scale tests. Two of the coupons used the tack weld inside the CJP groove, the same detailing used in the fabrication of the shake-table test specimen. The other two

coupons had the backing bars with external tack weld, as in the common construction practice.

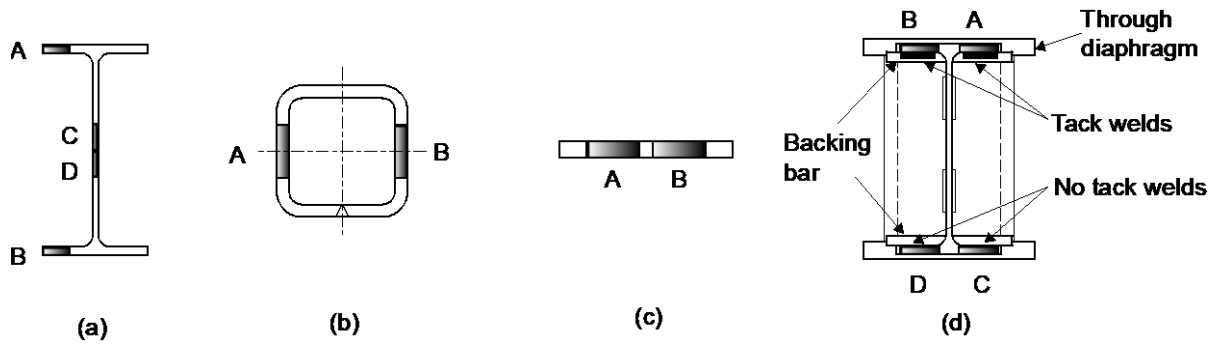


Figure 4.7 Coupon sampling: a) beams, b) columns, c) through diaphragm, d) CJP beam-to-diaphragm connection.

4.2.2.2 Tension coupon test setup

The coupons were tested in a 1000-kN universal testing machine. Figure 4.8 shows the test setup. The elongation was measured over a 50-mm calibrated length using a 10-mm range extensometer. However, the expected gauge deformation at fracture was expected to exceed the extensometer range by almost twice. Therefore, the displacements beyond 10 mm were estimated from the universal testing machine crosshead displacement. The crosshead displacement was related to the 50-mm gauge deformation during strain hardening with a polynomial curve fitting. Then, the force-displacement curve was completed using the estimated values from the fitted crosshead displacements.

Since the extensometer did not fit into the welded connection coupons, a transducer holder device was used to measure the displacement over a 75-mm gauge, as shown in Figure 4.8. Two 25-mm displacement transducers measured the elongation of the specimen.

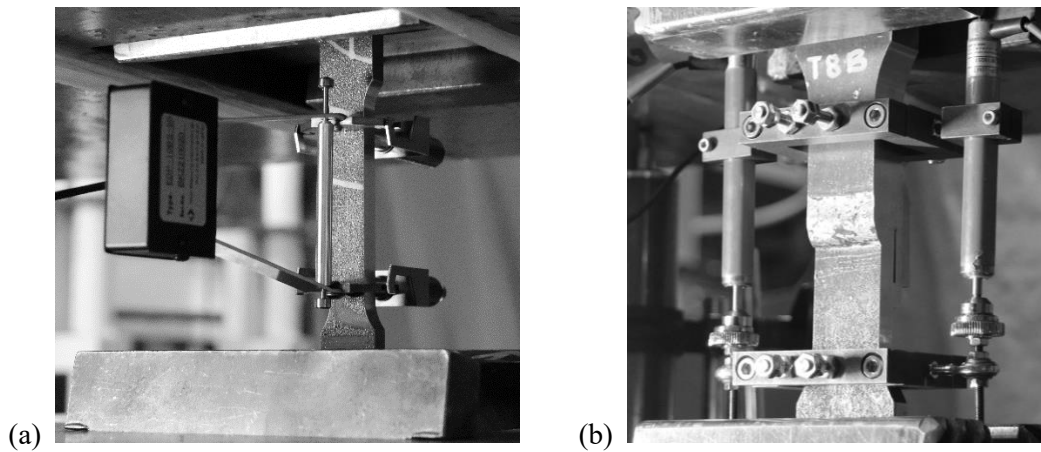


Figure 4.8 Coupon test setup: a) with extensometer, b) with transducer holder device.

4.2.2.3 Tension coupon test results

Table 4.3 summarizes the mechanical properties obtained from the tension coupon tests. Figures showing the engineering stress-strain relationships from the tests are shown in Appendix E. The minimal specified properties are also listed as a reference. The strain at failure was corroborated from direct measurement of the fractured specimen after removing it from the testing machine.

It is worth noting that the measured yield strength of the HSS columns were almost twice the nominal properties, and the yield strength of the beam webs 10-20% larger than the yield strength of the beam

flanges.

The coupons extracted from the HSS columns were bent longitudinally due to the residual stresses inherent to cold-formed tubular sections. For that reason, the stress-strain curves did not show a yield plateau. Therefore, 2% strain offset method was used to obtain the yield strength.

There were no coupon tests for the I-section columns (C3) because they were part of the gravity frames and intended not to participate in the lateral resistance of the system. Therefore, the mill test report values are listed instead.

Table 4.3 Specified minimal and measured mechanical properties.

Member	Section	Grade	Location	Minimal specified		Measured		Elongation [%]
				F_y [N/mm ²]	F_u [N/mm ²]	F_y [N/mm ²]	F_u [N/mm ²]	
C1	HSS-125×125×12	STKR400	-	245	400	441	484	32
C2	HSS-125×125×9	STKR400	-	245	400	406	443	37
C3	H-100×100×6×8	SS400	-	245	400	336 ^a	461 ^a	29 ^a
G1	H-200×100×5.5×8	SS400	Flange	245	400	317	447	38
			Web			349	463	35
G2	H-198×99×4.5×7	SS400	Flange	245	400	315	435	34
			Web			380	473	27

^aFrom mill test reports

Table 4.4 lists the results of the weld coupon tests. In coupon T8A, the fracture surface coincided with the position of the bottom transducer-holder device. Therefore, the calibrated length of the subsequent coupons was increased to 75 mm.

Table 4.4 Weld coupon test results.

Coupon	Tack weld	Dimensions	Grade	Location	F_y	F_u	Elongation
					[N/mm ²]	[N/mm ²]	[%]
T8A	Yes	25.1×7.5	SS400	Beam flange	332	474	20 ^a
T8B	Yes	25.2×7.4			327	472	12.6
T8C	No	25.2×7.5			318	461	19.6
T8D	No	25.1×7.5			334	467	18.9

^aUsed a calibrated length of 50 mm

4.2.3 Design Aspects

4.2.3.1 Member plastic strength

Table 4.5 lists the bending plastic strengths for each member constituting the test specimen and gravity frames. The strengths computed using nominal and mill test properties are listed as a reference. In the case of the beams, since the flange and web have different yield strength, a more detailed calculation was necessary to obtain the plastic strength. The calculations are presented in Appendix B.

Table 4.5 Member plastic strength

Member	Section	Grade	Plastic strength [kN-m]		
			Nominal	Mill test	Coupon test
C1	□-125×125×12	STKR400	51.0	93.6	91.7
C2	□-125×125×9	STKR400	41.4	69.6	68.6
C3	H-100×100×6×8	SS400	20.3	29.0	-
G1	H-200×100×5.5×8	SS400	50.2	73.9	66.7
G2	H-198×99×4.5×7	SS400	42.9	67.2	68.0
PZ $t = 12$	□-125×125×12	STKR400	74.0	135	132.6
PZ2 $t = 9$	□-125×125×9	STKR400	56.0	95.0	93.5

4.2.3.2 Strength ratios

The members of the moment-resisting frame were proportioned to meet the strong-column/weak-beam criterion. As previous research noted, a SC/WB factor greater than two may be required to avoid first-story collapse mechanism. Figure 4.9 shows the column-to-beam and panel-to-beam ratios computed with the plastic strength projected to the nodes. The measured mechanical properties were used in the computation of the member strengths. The nodal strengths were calculated with equations 3.6 to 3.8, with $L = 1500$, $H = 1250$, $D_c = 1250$, and $D_b = 500$, all in [mm]. The figure also shows the ratio per story of plastic strengths of the beams or panel zones, whichever is smaller, to the plastic strength of the columns.

Story	$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} / \frac{\sum M_{ppz}^*}{\sum M_{pb}^*}$		$\frac{\sum M_{pc}}{\min \sum (M_{pb}, M_{ppz})}$	Per story
4th	1.1 / 1.5	0.8 / 1.1	1.1 / 1.5	1.60
3rd	2.3 / 1.6	1.5 / 1.1	2.3 / 1.6	1.60
2nd	2.4 / 2.1	1.4 / 1.0	2.4 / 2.1	2.00
1st	2.8 / 2.1	1.4 / 1.0	2.8 / 2.1	2.00

Figure 4.9 Column-to-beam/panel-to-beam nodal strength ratios from measured properties, and minimum of beam or panel-to-column strength ratio per story.

4.2.3.3 Expected gravity effect

The gravity effect was calculated from the load transferred from the concrete weights to the steel elements by tributary area. The verification of the assumption that the gravity load was distributed by floor tributary area is presented in Chapter 5.

4.2.3.4 Expected story strength and stiffness

Using the proportional lateral distribution pattern described earlier, a plastic analysis of the frame was performed. The member plastic strengths M_p were calculated using measured mechanical properties. The plastic strength of each member was normalized by the smallest M_p , which in this case was the corresponding to the second and third story beams. Figure 4.10 shows key geometry and loads used for the plastic analysis. A sidesway collapse mechanism involving the first three stories was expected.

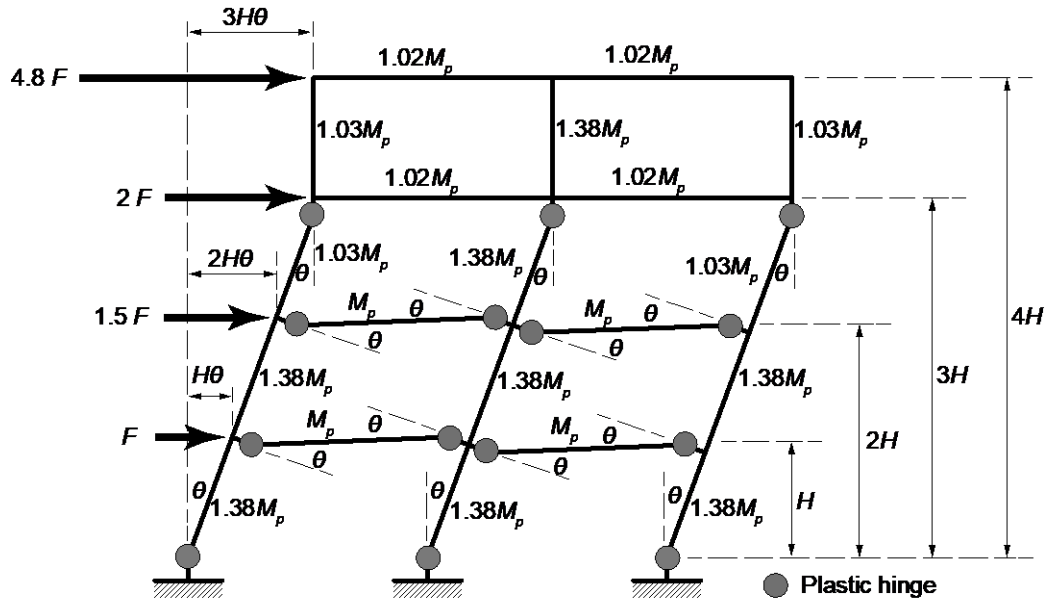


Figure 4.10 Expected plastic mechanism

The expected plastic strength of the system was 341 kN, 1.7 times higher than the plastic strength computed from nominal properties. Nevertheless, the mechanism was the same regardless of nominal or measured properties. Table 4.6 summarizes the expected plastic strength of each story, corresponding to the story shear at each level using the proportional load $F = 34.5$ kN.

The table also shows, as a reference, the expected lateral stiffness of each story, computed from elastic analysis using the same lateral load pattern than the plastic analysis. The stiffness was the quotient between the story shear Q and the story relative displacement δ . Since the global response is usually plotted in terms of story shear and story drift ratio (SDR), the lateral stiffness based on SDR is also provided.

Table 4.6 Expected story strength and stiffness

Story	Plastic strength Q_p [kN]	Stiffness Q/δ [kN/m]	Stiffness Q/SDR [kN/rad]
4	166	11 840	14 800
3	235	11 880	14 900
2	286	13 260	16 600
1	320	20 800	26 000

4.2.4 Placement on the Shake Table

The Large-scale Earthquake Simulator of the NIED (National Research Institute for Earth Science and Disaster Resilience) is a 15×14.5 m shake table constructed in 1970 as the first research facility in the planned Tsukuba Research City, Ibaraki, Japan. Several notable experiments have been conducted ever since.

The foundation for the test specimen consisted of three parallel H-400×400×13×21 beams, laterally braced at the quarters by C-300×90×9×13 single channels. The three beams were bolted to the M60 holes on the shake table, as illustrated in Fig. The specimen was placed as centered as possible within the shake table.

4.2.5 Fabrication

Figure 4.11 illustrates the fabrication process of the specimen. The Japanese construction practice of low and mid-rise buildings follows the same procedure.

The HSS square-tube columns were sliced at the column panel boundaries. Then the panel segment of the column was beveled all around both ends to allow for the CJP weld. 9-mm backing bars were inserted in the inner perimeter of both column panel ends, fixed in position with tack welds, and then welded to the 16-mm diaphragm plates. The CJP welds were made in the flat position with a gas-shielded metal arc welding (GMAW) process.

Once the column panel is completed, the beam flanges are milled for the CJP weld. Then, the beams are fixed to the column panels with tack welds. The beam web was tack-welded to the column. The tack welds of the backing bars for the flange connection are usually placed outside the CJP groove. Because of the reduced scale of this specimen, to avoid any alteration of the test results due to the flange tack weld, it was placed inside the groove and then filled with the CJP weld.

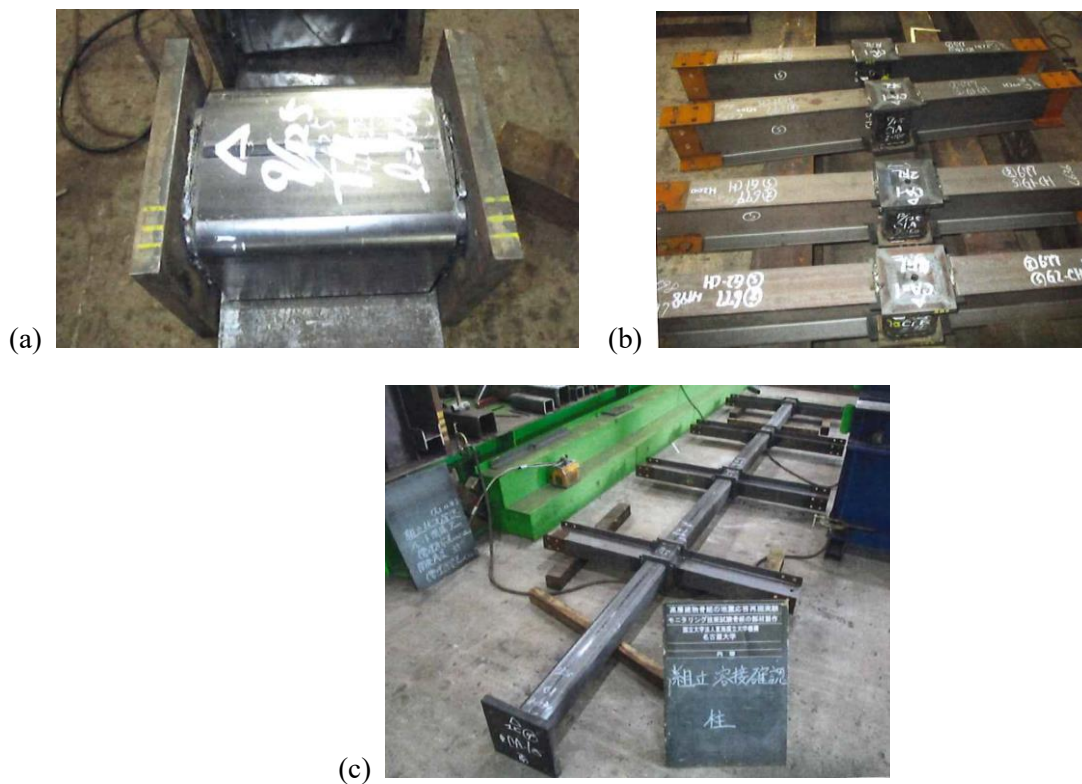


Figure 4.11 Fabrication of the shake table test specimen: a) column panel, b) beam stub-column panel assembly, c) column-tree assembly.

Once the beam and column panel joints were completed, the remaining column segments were connected following the same procedure. Then the welds were ultrasonically tested, and no defects were detected. Finally, the resulting column-trees were transported to the experiment facility.

4.2.6 Scaling Rule

Table 4.7 shows the scale factors used to convert the physical quantities from the model (specimen) to a real building domain. The factors were deduced from scaled distance and mass, therefore considering time unscaled. To convert to real building quantities, distance, mass, velocity, and acceleration are multiplied by 2.5; area and force by 6.25, while time, stress, and strain are unscaled.

Table 4.7 Scaling factors

Parameters		Scaling Factor	$\lambda = 2.5$
Distance	L	$1 / \lambda$	$1 / 2.5$
Mass	M	$1 / \lambda$	$1 / 2.5$
Area	S	$1 / \lambda^2$	$1 / 6.25$
Time	T	1	1
Velocity	$V = L / T$	$1 / \lambda$	$1 / 2.5$
Acceleration	$A = L / T^2$	$1 / \lambda$	$1 / 2.5$
Force	$F = M \times A$	$1 / \lambda^2$	$1 / 6.25$
Stress	$\sigma = F / L^2$	1	1
Strain	$\varepsilon = \sigma \times L^2 / F$	1	1

4.3 LOADING PLAN

4.3.1 Selection of Ground Motion

One ground motion was chosen between two options. Two records were evaluated: the JMA Kobe NS record of the 1995 Kobe earthquake and the Canoga Park EW record of the 1994 Northridge earthquake. One of the two records was chosen based on two criteria: a) to make the specimen yield at a low acceleration, and b) to produce significant inelastic deformations.

The evaluation was performed with the elastic spectral acceleration – spectral displacement (SA-SD) curves of the two records, as shown in Figure 4.12. The structure was converted to an equivalent SDOF system, which center of mass was located 3.33 m above the base. The calculation of the lateral displacement of the center of mass corresponding to the plastic strength of the system d_p and the expected natural period T_n are detailed in Appendix A. The acceleration a_p at which the system was expected to reach its plastic strength was 11 m/s^2 , computed with the following equation.

$$a_p = d_p \left(\frac{2\pi}{T_n} \right)^2 \quad (4.1)$$

Where $d_y = 0.02 \text{ m}$ and $T_n = 0.56 \text{ s}$.

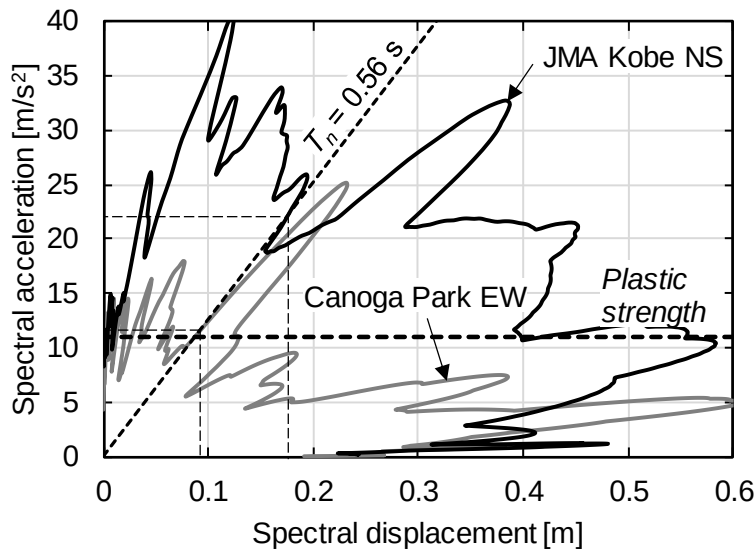


Figure 4.12 SA-SD curve for JMA Kobe NS and Canoga Park EW records.

The equivalent SDOF would be pushed to 0.10 m ($5d_y$) and 0.18 m ($9d_y$) for the JMA Kobe NS and Canoga Park EW records, respectively, for the expected natural period $T_n = 0.56$ s. Since the JMA Kobe NS record would induce almost twice the deformations induced by the Canoga Park record, the former was chosen for the tests.

4.3.2 Loading Protocol

The JMA Kobe NS record was applied in the longitudinal direction of the specimen in a sequence of increasing amplitude scales. Table 4.8 summarizes the applied motion scales and target Peak Ground Acceleration (PGA). The 100% excitation was applied four times. Herein after in this document we will refer to the primary excitations as 10%, 25%, 50%, 100%-1, 100%-2, 100%-3 and 100%-4. White noise was applied between the primary excitations to trace the change in vibration properties of the system.

The tests were conducted on two consecutive days. On the first day, the 10% to the 50% were applied, and in the second day all the four 100% excitations.

Table 4.8 Loading protocol

Day	Test No.	Scale	Target PGA [m/s^2]
1	W.N.	-	-
	1	10%	0.8
	W.N.	-	-
	2	25%	2.0
	W.N.	-	-
	3	50%	4.1
2	W.N.	-	-
	4	100%	8.2
	5	100%	8.2
	6	100%	8.2
	7	100%	8.2

4.4 INSTRUMENTATION

Data was acquired from a combination of strain gages, displacement transducers, string pots and accelerometers, in a total of 264 data channels. The data was sampled at 100 Hz (100 measurements per second) using 6 data loggers model DS-50A, each with capacity of 50 channels. Uniaxial elastic strain gages were placed on multiple sections of each beam and column to enable sampling of internal force distribution. Strain gages were also placed on the first and second-story columns of the gravity frames. Displacement transducers were placed at each story between the concrete mass or foundation beam and upper beam to measure relative story drift. Displacement transducers were used to measure beam-end rotation, panel-zone shear deformation, and slippage of the column base plates. Uniaxial accelerometers were attached to the concrete masses at each story.

The rotation of the beams was measured from a pair of displacement transducers with targets fixed at the column diaphragm plate and on both beam flanges 490-mm away from the column. The panel zone deformation was measured at one face of the columns, with a pair of displacement transducers placed diagonally between the column diaphragm plates.

The story lateral displacement at the first story was measured with two laser transducers mounted on a steel tower with bases fixed to the foundation beams. The targets for the lasers were glued to the mid-

depth of the second-story beams' web. The measurements for stories 2 to 4 were rather indirect since the mounting towers were glued to the top surface of the concrete weights. To overcome this problem, string-pots were placed in the gravity frames (Lines A and C). Additionally, independent optical sensor measurements were available at the second-floor masses and beams.

4.5 DATA REDUCTION

4.5.1 Story Lateral Displacements

4.5.1.1 Gravity frames (Frames A and C)

The distances B and H_s were measured on the site before the tests. H_s was equal to the story height H in stories 2-4 but differed in the first story. Therefore, the measured lateral displacement of story 1 D_1 was corrected using similar triangles as shown in Figure 4.13b. For the dimensions of this specimen, when the change in length of the string δ is less than 50 mm (or a SDR = 0.05 rad) equations (4.2 to (4.5 deduced from Figure 4.13c give a sufficiently good approximation of the lateral displacement D . For larger deformations, the difference with the nonlinear equations increases more than 1%. The approximate equations overestimate the lateral displacement beyond this point. Therefore, the SDR of frames A and C will be computed with equations (4.6) to (4.13, deduced from Figure 4.13c considering H constant.

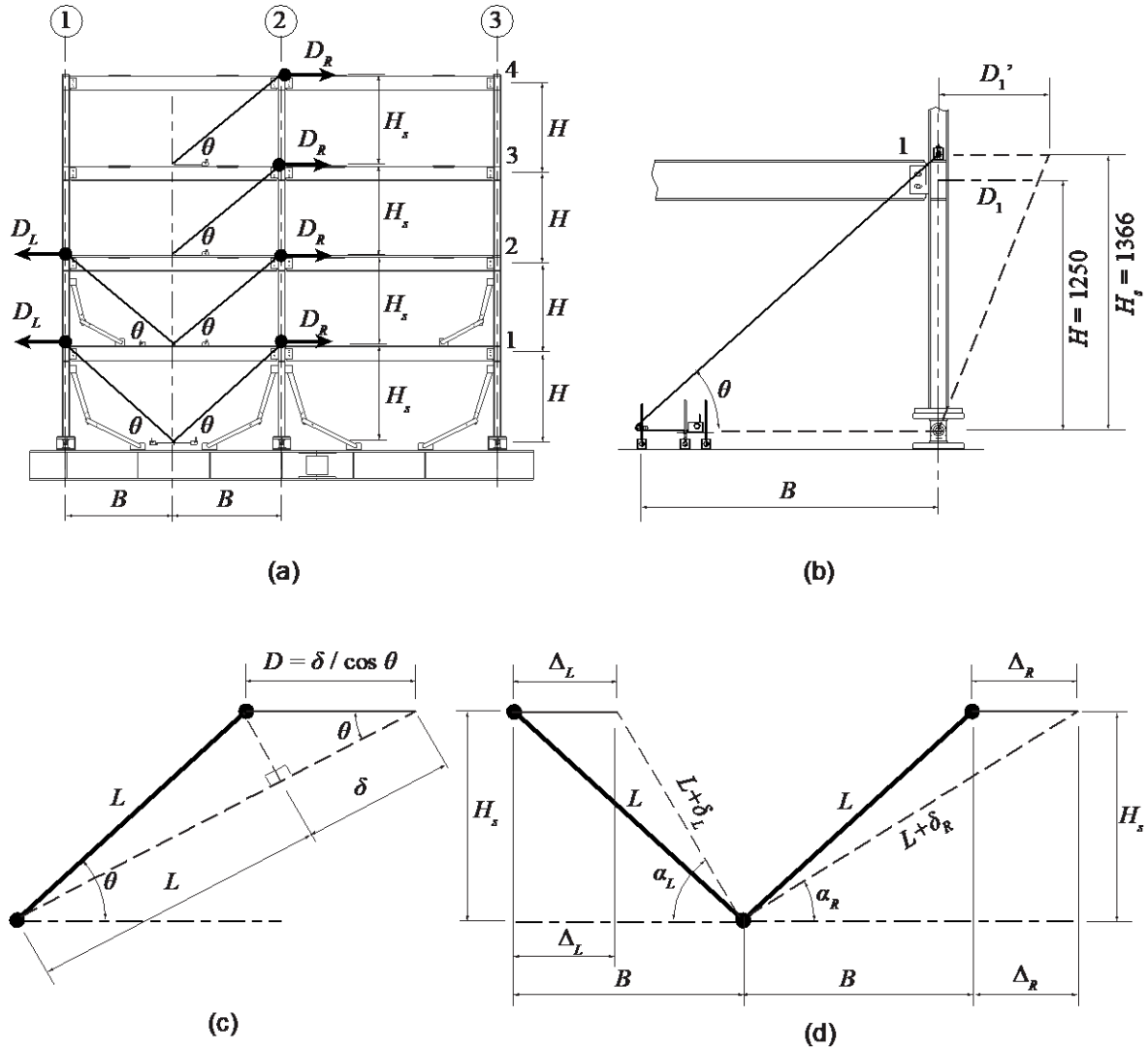


Figure 4.13 Displacements of gravity frames A and C: a) instruments, b) enlarged view of the first

story, c) simplified relationship, d) nonlinear relationship.

$$\text{Story 1} \quad D_{A,C1} = 0.915 \frac{(\delta_{R1} - \delta_{L1})}{2 \cos \theta} \quad (4.2)$$

$$\text{Story 2} \quad D_{A,C2} = D_{A,C1} + \frac{(\delta_{R2} - \delta_{L2})}{2 \cos \theta} \quad (4.3)$$

$$\text{Story 3} \quad D_{A,C3} = D_{A,C2} + \frac{\delta_{R3}}{\cos \theta} \quad (4.4)$$

$$\text{Story 4} \quad D_{A,C4} = D_{A,C3} + \frac{\delta_{R4}}{\cos \theta} \quad (4.5)$$

Table 4.9 Story displacement channels and measurements, Frame A

Frame	Story	δ_L	δ_R	θ [°]	H_s [mm]	B [mm]
A	4	-	CH 185	40	1250	1500
	3	-	CH 184	40	1250	1500
	2	CH 182	CH 183	40	1250	1500
	1	CH 180	CH 181	42	1336	1500

Table 4.10 Story displacement channels and measurements, Frame C

Frame	Story	δ_L	δ_R	θ [°]	H_s [mm]	B [mm]
C	4	-	CH 191	40	1250	1500
	3	-	CH 190	40	1250	1500
	2	CH 188	CH 189	40	1250	1500
	1	CH 187	CH 186	42	1366	1500

$$L = \sqrt{B^2 + H_s^2} \quad (4.6)$$

$$\alpha_{L,R} = \sin^{-1} \left(\frac{H}{L + \delta_{L,R}} \right) \quad (4.7)$$

$$\Delta_L = B - (L + \delta_L) \cos \alpha_L \quad (4.8)$$

$$\Delta_R = (L + \delta_R) \cos \alpha_R - B \quad (4.9)$$

$$\text{Story 1} \quad D_{A,C1} = 0.915 \left(\frac{\Delta_{R1} + \Delta_{L1}}{2} \right) = 0.915 \left[\frac{(L + \delta_{R1}) \cos \alpha_{R1} - (L + \delta_{L1}) \cos \alpha_{L1}}{2} \right] \quad (4.10)$$

$$\text{Story 2} \quad D_{A,C2} = D_{A,C1} + \frac{\Delta_{R2} + \Delta_{L2}}{2} = D_{A,C1} + \left[\frac{(L + \delta_{R2}) \cos \alpha_{R2} - (L + \delta_{L2}) \cos \alpha_{L2}}{2} \right] \quad (4.11)$$

$$\text{Story 3} \quad D_{A,C3} = D_{A,C2} + \Delta_R = D_{A,C2} + (L + \delta_{R3}) \cos \alpha_{R3} - B \quad (4.12)$$

$$\text{Story 4} \quad D_{A,C4} = D_{A,C3} + \Delta_{R4} = D_{A,C3} + (L + \delta_{R4}) \cos \alpha_{R4} - B \quad (4.13)$$

4.5.1.2 Displacement of story 2 in Frame C by optical sensors

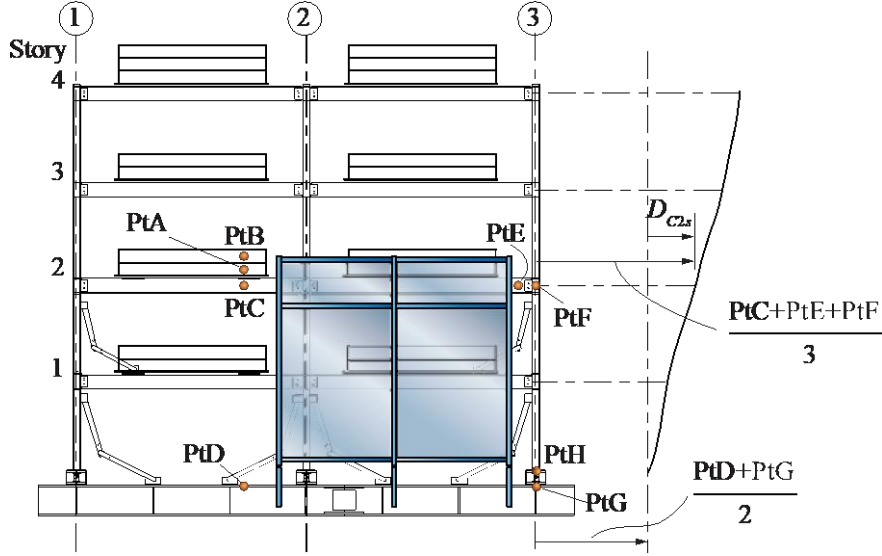


Figure 4.14 Optical sensors in Frame C.

$$\text{Story 2} \quad D_{C2s} = \left(\frac{PtC + PtE + PtF}{3} \right) - \left(\frac{PtD + PtG}{2} \right) \quad (4.14)$$

$$\text{Slippage of the concrete} \quad S_2 = PtB - PtC \quad (4.15)$$

4.5.1.3 Main frame (Frame B)

The “auxiliary” relative displacements δ_B of frame B are the relative displacement of each story to the concrete weight on the immediate inferior story. However, there was relative displacement between the concrete and frame B beams. The lateral displacement of the concrete was estimated as the average of the lateral displacement of the gravity frames A and C because the connection points were closer to those frames and there was no direct contact between the concrete and the beams of Frame B.

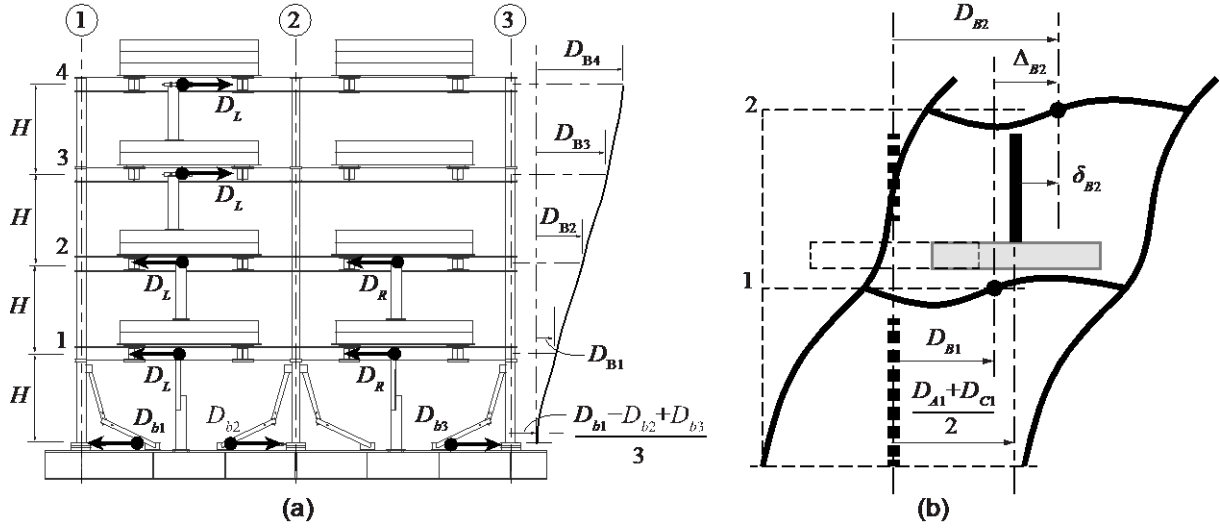


Figure 4.15 Displacements Frame B: a) Instrument schedule, b) correction of displacement.

$$\text{Story 1} \quad \delta_{B1} = -\left(\frac{D_{L1} + D_{R1}}{2} - \frac{D_{b1} - D_{b2} - D_{b3}}{3}\right) \quad (4.16)$$

$$\text{Story 2} \quad \delta_{B2} = -\left(\frac{D_{L2} + D_{R2}}{2}\right) \quad (4.17)$$

$$\text{Story 3} \quad \delta_{B3} = D_{L3} \quad (4.18)$$

$$\text{Story 4} \quad \delta_{B4} = D_{L4} \quad (4.19)$$

Therefore, the story lateral displacements of Frame B are:

$$\text{Story 1} \quad D_{B1} = \delta_{B1} \quad (4.20)$$

$$\text{Story } i = 2, 3, 4 \quad D_{Bi} = \left(\frac{D_A + D_C}{2}\right)_{i-1} + \delta_{Bi} \quad (4.21)$$

Table 4.11 Story displacement channels and measurements, Frame B

Story	D_L	D_R	D_{b1}	D_{b2}	D_{b3}	H [mm]
4	CH 119	-	-	-	-	1250
3	CH 118	-	-	-	-	1250
2	CH 263	CH 264	-	-	-	1250
1	CH 261	CH 262	CH 175	CH 176	CH 177	1250

4.5.2 Story Drift Ratio

$$\text{Story 1} \quad \text{SDR}_1 = \frac{D_{B1}}{H} = \frac{\Delta_{B1}}{H} \quad (4.22)$$

$$\text{Story } i = 2, 3, 4 \quad \text{SDR}_i = \frac{D_{Bi} - D_{Bi-1}}{H} = \frac{\Delta_{Bi}}{H} \quad (4.23)$$

4.5.3 Beam Rotation

The rotation of the beams, relative to the column face measured over a fixed 490-mm length as shown in Figure 4.16. The instrument holder plates were glued to either the beam flange or the column through-diaphragm ensuring the 490-mm length between targets in every location. The distance h was measured on the site before the tests. The sign convention is analogous to the internal moment convention: positive rotation means beam concave deformation, while negative moment corresponds to convex deformation. All the beams had instruments to measure the right-end rotation. However, only beams 1-6 had instruments on the left end because of availability limitations. Nevertheless, the external joints were expected to develop the most critical rotations. The beam rotation was calculated using (4.24) and the expected flexural stiffness using (4.25).

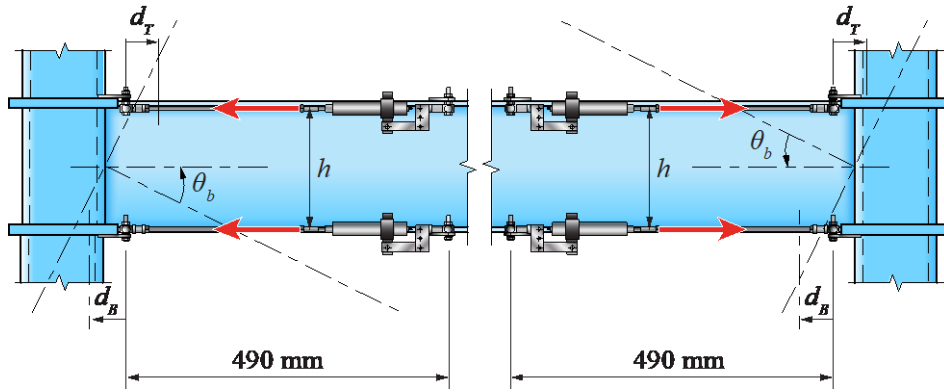


Figure 4.16 Beam rotation measurement scheme

$$\theta_b = \frac{d_B - d_T}{h} \quad (4.24)$$

$$k_b = \frac{3EI_b}{L_0} \quad (4.25)$$

Where L_0 is the distance from the face of the column to the beam inflection point. Generally, the inflection point happened in the beam centerline.

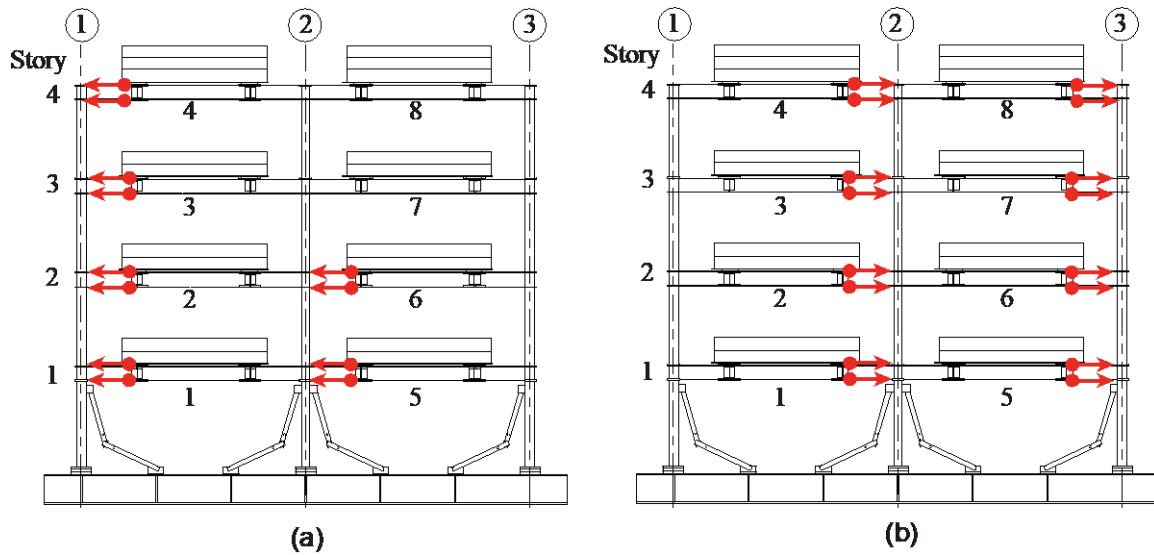


Figure 4.17 Instrument location for beam rotation measurement in the shake-table test specimen: a) left end rotation, b) right end rotation.

Table 4.12 Beam rotation measurements and channels

Beam No.	Left end			Right end		
	h [mm]	d_T	d_B	h [mm]	d_T	d_B
1	190	CH 120	CH 121	195	CH 122	CH 123
2	195	CH 128	CH 129	191	CH 130	CH 131
3	186	CH 136	CH 137	182	CH 138	CH 139
4	190	CH 142	CH 143	195	CH 144	CH 145
5	190	CH 124	CH 125	193	CH 126	CH 127
6	188	CH 132	CH 133	190	CH 134	CH 135
7	-	-	-	186	CH 140	CH 141
8	-	-	-	190	CH 146	CH 147

4.5.4 Panel Zone Forces and Deformation

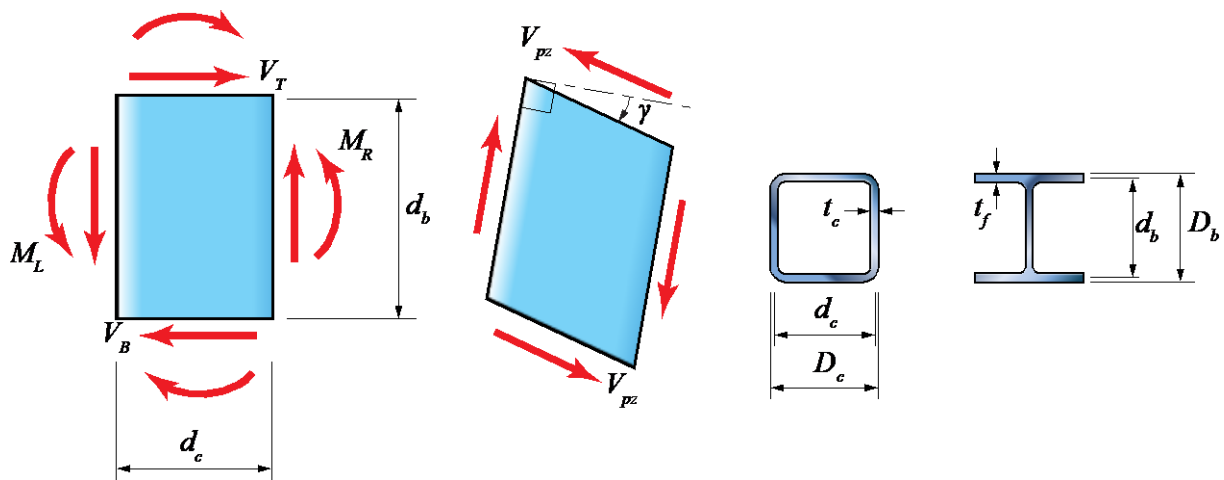


Figure 4.18 Definition of panel shear and deformation.

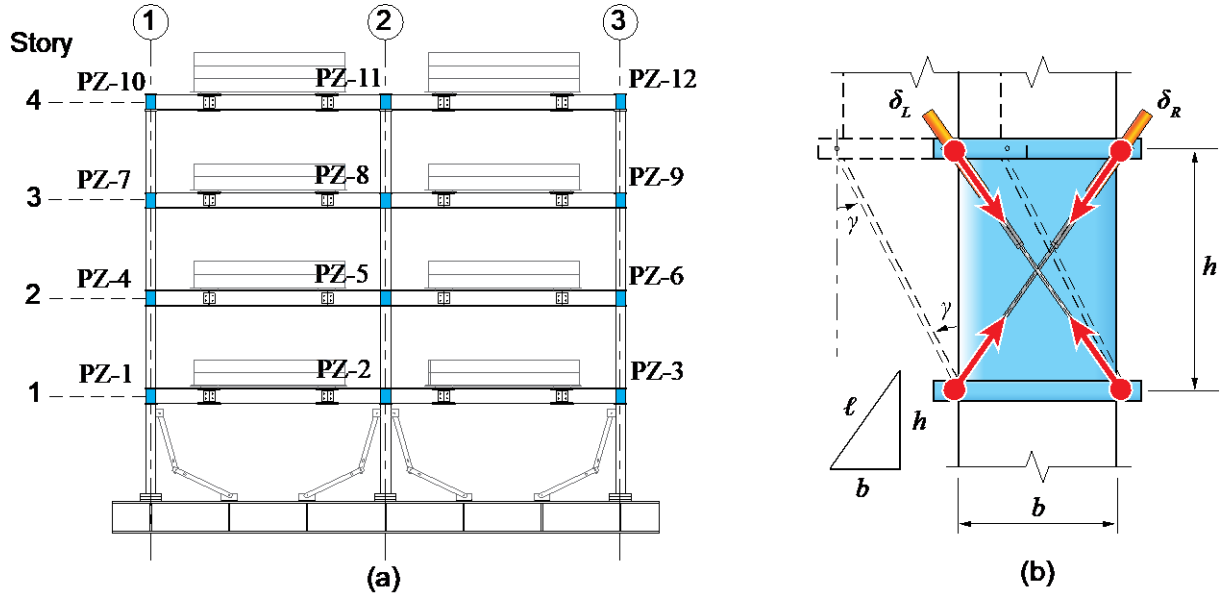


Figure 4.19 Panel zone: a) numbering, b) measurement of shear deformation.

Panel shear
$$V_{pz} = \left(\frac{M_R + M_L}{d_b} \right) - \frac{V_B + V_T}{2} \quad (4.26)$$

Panel shear deformation
$$\gamma = \frac{\sqrt{b^2 + h^2}}{2bh} (\delta_L - \delta_R) \quad (4.27)$$

(with left instrument only)
$$\gamma = \sin^{-1} \left[\frac{(\ell + \delta_L)^2 - \ell^2}{2bh} \right] \quad (4.28)$$

(with right instrument only)
$$\gamma = \sin^{-1} \left[\frac{\ell^2 - (\ell + \delta_R)^2}{2bh} \right] \quad (4.29)$$

Table 4.13 Panel zone measurements and channels

Story	Label	D_L [mm]	D_R [mm]	b [mm]	h [mm]
4	PZ-10	CH 169	CH 170	135	193
	PZ-11	CH 171	CH 172	134	193
	PZ-12	CH 173	CH 174	135	192
3	PZ-7	CH 163	CH 164	136	193
	PZ-8	CH 165	CH 166	134	195
	PZ-9	CH 167	CH 168	135	194
2	PZ-4	CH 157	CH 158	135	195
	PZ-5	CH 159	CH 160	134	194
	PZ-6	CH 161	CH 162	136	194
1	PZ-1	CH 151	CH 152	136	195
	PZ-2	CH 153	CH 154	134	194
	PZ-3	CH 155	CH 156	136	194

The panel zone yield and plastic strengths were calculated with the AIJ equation

$$\text{Panel moment yield strength} \quad M_{yp} = \frac{8}{9} V_e \sqrt{1 - n^2} \left(\frac{\sigma_{yc}}{\sqrt{3}} \right) \quad (4.30)$$

$$\text{Panel moment plastic strength} \quad M_{pp} = \alpha_n V_e \left(\frac{\sigma_{yc}}{\sqrt{3}} \right) \quad (4.31)$$

$$\text{Panel shear yield strength} \quad V_{yp} = \frac{M_{ypz}}{d_b} \quad (4.32)$$

$$\text{Panel shear plastic strength} \quad V_{pp} = \frac{M_{ppz}}{d_b} \quad (4.33)$$

Where

σ_{yc} : yield strength of the column material

$$n = \frac{\text{average}(N_{cT}, N_{cB})}{\sigma_{yc} A_c}$$

$$\alpha_n = \begin{cases} 1, & n \leq 0.5 \\ 2\sqrt{n(n-1)}, & n > 0.5 \end{cases}$$

In all cases n was between 0.01 and 0.16, thus $\alpha_n = 1$.

$$V_e = 2d_c d_b t_c$$

A_c : cross section area of the column

The expected elastic panel zone stiffness was obtained by:

$$\gamma = \frac{V_{pz}}{2t_c d_c G} \rightarrow \frac{V_{pz}}{\gamma} = k_{pz} = 2t_c d_c G$$

$$k_{pz} = 2G(D_c - d_c)t_c \quad (4.34)$$

Where

$$G = \frac{E}{2(1 + \nu)}$$

$$E = 205\,000 \text{ N/mm}^2$$

$$\nu = 0.3$$

4.5.5 Column Internal Forces and Moments

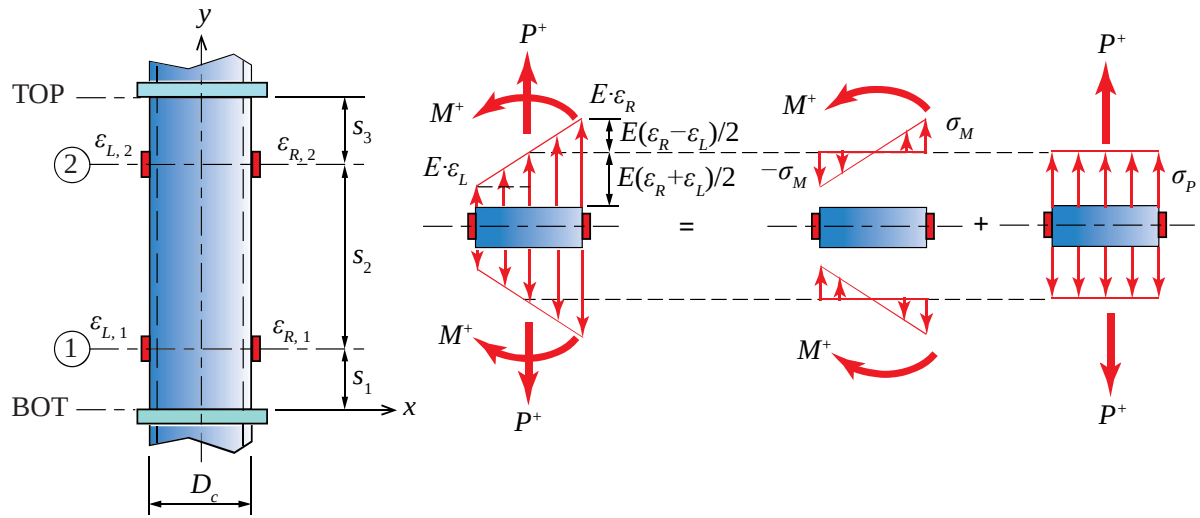


Figure 4.20 Internal forces and moments from strain.

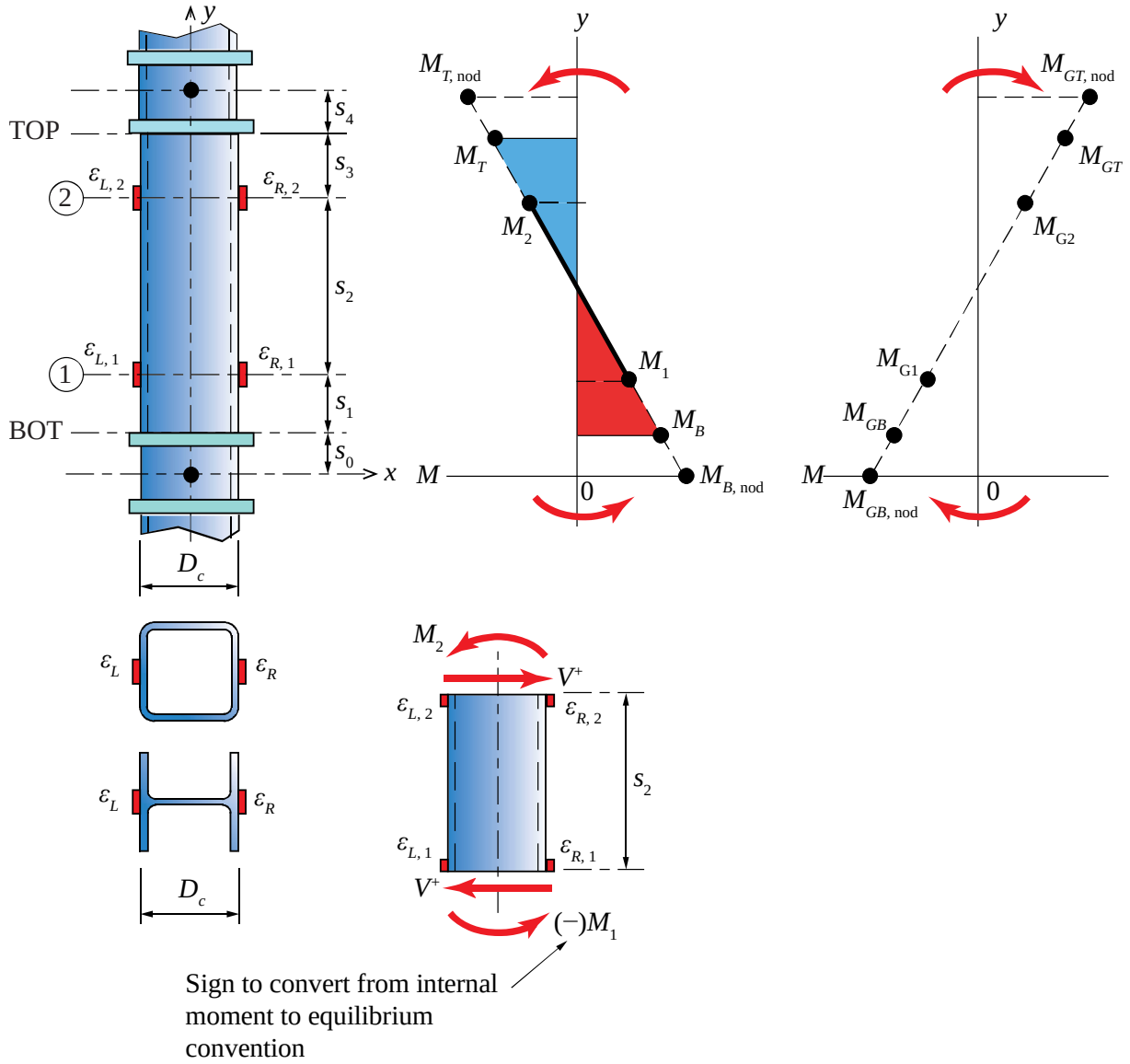


Figure 4.21 Column moments and shear.

Since all the measurements were initialized at 0 before the first test, the seismic load effect can be computed with the following formulas.

$$\text{Bending moment} \quad M_c = \frac{EI_c}{D_c} (\varepsilon_R - \varepsilon_L) \quad (4.35)$$

$$\text{Axial} \quad P_c = EA_c \left(\frac{\varepsilon_R + \varepsilon_L}{2} \right) \quad (4.36)$$

$$\text{Shear} \quad V_c = \frac{M_{c2} - M_{c1}}{s_{c2}} \quad (4.37)$$

Extrapolation of bending moments assuming linear bending moment diagram (lateral load effect)

$$\text{Moment at column top} \quad M_{cT} = M_{c1} + \left(\frac{M_{c2} - M_{c1}}{s_{c2}} \right) (s_{c2} + s_{c3}) \quad (4.38)$$

$$\text{Moment at column bottom} \quad M_{cB} = M_{c1} + \left(\frac{M_{c2} - M_{c1}}{s_{c2}} \right) (s_{c2} + s_{c3}) \quad (4.39)$$

$$\text{Moment at column top node} \quad M_{cTn} = M_{c1} + \left(\frac{M_{c2} - M_{c1}}{s_{c2}} \right) (s_{c2} + s_{c3} + s_{c4}) \quad (4.40)$$

$$\text{Moment at column bottom node} \quad M_{cBn} = M_{c1} - \left(\frac{M_{c2} - M_{c1}}{s_{c2}} \right) (s_{c0} + s_{c1}) \quad (4.41)$$

Moments due to gravity loads

$$\text{Gravity moment at column top} \quad M_{GcT} = M_{GcTn} - \left(\frac{M_{GcTn} - M_{GcBn}}{H} \right) (s_{c4}) \quad (4.42)$$

$$\text{Gravity moment at column bottom} \quad M_{GcB} = M_{GcBn} + \left(\frac{M_{GcTn} - M_{GcBn}}{H} \right) (s_{c0}) \quad (4.43)$$

$$\text{Gravity moment at column section 1} \quad M_{Gc1} = M_{GcBn} + \left(\frac{M_{GcTn} - M_{GcBn}}{H} \right) (s_{c0} + s_{c1}) \quad (4.44)$$

$$\text{Gravity moment at column section 2} \quad M_{Gc2} = M_{GcTn} - \left(\frac{M_{GcTn} - M_{GcBn}}{H} \right) (s_{c3} + s_{c4}) \quad (4.45)$$

Expected plastic moment for square hollow section columns considering the reduction by axial load, from AIJ.

$$M_{pcn} = m_{pn} M_{pc} \quad (4.46)$$

Where

$$n = \frac{P_c}{P_{yc}}, \quad P_{yc} = A_c \sigma_{yc}$$

$$m_{pn} = \begin{cases} 1 - \frac{4}{3}n^2, & n \leq 0.5 \\ \frac{4}{3}(1 - n), & n > 0.5 \end{cases}$$

Table 4.14 Column measured distances and gravity moments

Column	s_{c1} [mm]	s_{c2} [mm]	s_{c3} [mm]	M_{GcBn} [kN-m]	M_{GcTn} [kN-m]	P_G [kN]
1	350	446	350	1.23	-2.46	-58.38
2	352	444	349	0.00	0.00	-135.24
3	351	445	349	-1.23	2.46	-58.38
4	303	438	300	3.62	-3.55	-45.28
5	302	440	300	0.00	0.00	-105.44
6	301	440	300	-3.62	3.55	-45.28
7	300	443	299	2.63	-2.33	-32.13
8	300	439	303	0.00	0.00	-75.74
9	299	442	300	-2.63	2.33	-32.13

10	300	441	301	4.55	-7.07	-18.63
11	303	441	298	0.00	0.00	-46.74
12	301	442	299	-4.55	7.07	-18.63
13	454	342	454	0.00	0.00	-31.00
14	459	341	451	0.00	0.00	-62.00
15	460	340	450	0.00	0.00	-31.00
16	402	447	402	0.00	0.00	-24.00
17	404	449	398	0.00	0.00	-48.00
18	400	450	401	0.00	0.00	-24.00
19	460	341	450	0.00	0.00	-17.00
20	462	339	450	0.00	0.00	-34.00
21	461	339	451	0.00	0.00	-17.00
22	399	447	405	0.00	0.00	-10.00
23	399	450	402	0.00	0.00	-20.00
24	399	448	404	0.00	0.00	-10.00

4.5.6 Beam Internal Forces and Moments

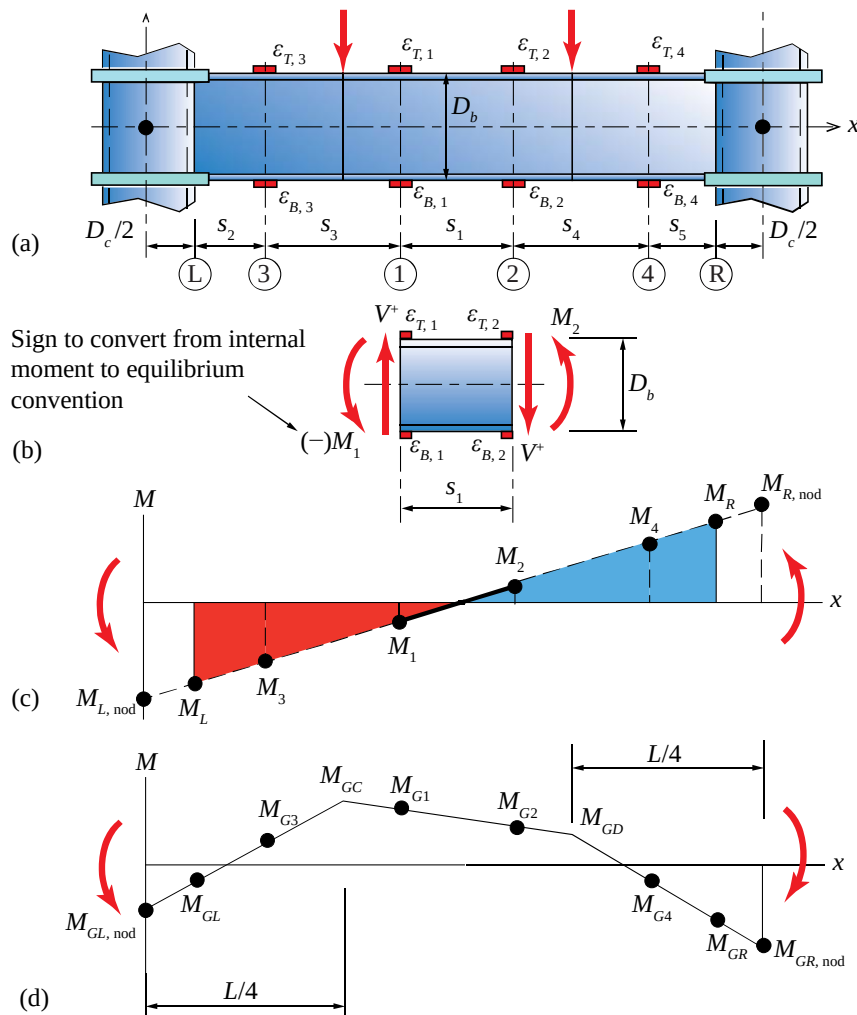


Figure 4.22 Beam moments and shear.

$$\text{Bending moment} \quad M_b = \frac{EI_b}{D_b} (\varepsilon_B - \varepsilon_T) \quad (4.47)$$

$$\text{Axial} \quad P_b = EA_b \left(\frac{\varepsilon_B + \varepsilon_T}{2} \right) \quad (4.48)$$

$$\text{Shear} \quad V_b = \frac{M_{b2} + (-M_{b1})}{s_{b1}} = \frac{M_{b2} - M_{b1}}{s_{b1}} \quad (4.49)$$

$$\text{Moment at beam left end} \quad M_{bL} = M_{b1} - \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) (s_{b2} + s_{b3}) \quad (4.50)$$

$$\text{Moment at beam right end} \quad M_{bR} = M_{b1} + \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) (s_{b1} + s_{b4} + s_{b5}) \quad (4.51)$$

$$\text{Moment at beam left node} \quad M_{bLn} = M_{b1} - \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) \left(\frac{D_c}{2} + s_{b2} + s_{b3} \right) \quad (4.52)$$

$$\text{Moment at beam right node} \quad M_{bRn} = M_{b1} + \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) \left(s_{b1} + s_{b4} + s_{b5} + \frac{D_c}{2} \right) \quad (4.53)$$

$$\text{Moment at beam section 3} \quad M_{b3} = M_{b1} - \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) s_{b3} \quad (4.54)$$

$$\text{Moment at beam section 4} \quad M_{b4} = M_{b1} + \left(\frac{M_{b2} - M_{b1}}{s_{b1}} \right) (s_{b1} + s_{b4}) \quad (4.55)$$

Moments due to gravity loads. Computed using as a reference the moments at the nodes and at sections C and D.

$$\text{Gravity moment at beam left end} \quad M_{GbL} = M_{GbLn} + \left(\frac{M_{GbC} - M_{GbLn}}{L/4} \right) \frac{D_c}{2} \quad (4.56)$$

$$\text{Gravity moment at beam right end} \quad M_{GbR} = M_{GbRn} - \left(\frac{M_{GbRn} - M_{GbD}}{L/4} \right) \frac{D_c}{2} \quad (4.57)$$

$$\text{Gravity moments at beam section 1} \quad M_{Gb1} = M_{GbC} + \left(\frac{M_{GbD} - M_{GbC}}{L/2} \right) \left(s_{b2} + s_{b3} + \frac{D_c}{2} - \frac{L}{4} \right) \quad (4.58)$$

$$\text{Gravity moments at beam section 2} \quad M_{Gb2} = M_{GbC} + \left(\frac{M_{GbD} - M_{GbC}}{L/2} \right) \left(s_{b1} + s_{b2} + s_{b3} + \frac{D_c}{2} - \frac{L}{4} \right) \quad (4.59)$$

$$\text{Gravity moments at beam section 3} \quad M_{Gb3} = M_{GbLn} + \left(\frac{M_{GbC} - M_{GbLn}}{L/4} \right) \left(s_{b2} + \frac{D_c}{2} \right) \quad (4.60)$$

$$\text{Gravity moments at beam section 4} \quad M_{Gb4} = M_{GbRn} - \left(\frac{M_{GbRn} - M_{GbD}}{L/4} \right) \left(s_{b5} + \frac{D_c}{2} \right) \quad (4.61)$$

$$\frac{D_c}{2} = 62.5 \text{ mm}, \quad \frac{L}{2} = 1500 \text{ mm}, \quad \frac{L}{4} = 750 \text{ mm}$$

Table 4.15 Beam measured distances and gravity moments

Beam	D_b	S_{b1} [mm]	S_{b2} [mm]	S_{b3} [mm]	S_{b4} [mm]	S_{b5} [mm]	M_{GbLn} [kN-m]	M_{GbRn} [kN-m]	M_{GbC} [kN-m]	M_{GbD} [kN-m]
1	200	912	380	603	600	380	-6.08	-8.77	3.74	2.39
2	200	913	379	598	601	384	-6.18	-8.72	3.68	2.41
3	198	917	382	594	596	386	-6.87	-8.38	3.26	2.51
4	198	915	380	601	599	380	-7.07	-14.18	6.90	3.35
5	200	914	490 ^a	490	491 ^b	491	-8.77	-6.08	2.39	3.74
6	200	917	489 ^a	489	490 ^b	490	-8.72	-6.18	2.41	3.68
7	198	915	488 ^a	488	493 ^b	493	-8.38	-6.87	2.51	3.26
8	198	915	489 ^a	489	491 ^b	491	-14.18	-7.07	3.35	6.90

^a Measured distance split evenly between s_{b2} and s_{b3} ^b Measured distance split evenly between s_{b4} and s_{b5}

4.5.7 Story Shears

Story shears in the main frame by column forces:

$$\text{Story 1} \quad Q_1 = V_{c1} + V_{c2} + V_{c3} \quad (4.62)$$

$$\text{Story 2} \quad Q_2 = V_{c4} + V_{c5} + V_{c6} \quad (4.63)$$

$$\text{Story 3} \quad Q_3 = V_{c7} + V_{c8} + V_{c9} \quad (4.64)$$

$$\text{Story 4} \quad Q_4 = V_{c10} + V_{c11} + V_{c12} \quad (4.65)$$

Story shears in frame A by column forces:

$$\text{Story 1} \quad Q_{A1} = V_{c13} + V_{c14} + V_{c15} \quad (4.66)$$

$$\text{Story 2} \quad Q_{A2} = V_{c16} + V_{c17} + V_{c18} \quad (4.67)$$

Story shears in frame C by column forces:

$$\text{Story 1} \quad Q_{C1} = V_{c19} + V_{c20} + V_{c21} \quad (4.68)$$

$$\text{Story 2} \quad Q_{C2} = V_{c22} + V_{c23} + V_{c24} \quad (4.69)$$

Story shears by acceleration:

$$\text{Story 1 } Q_{ac1} = -m_4 \left(\frac{a_{4W} + a_{4E}}{2} \right) - m_3 \left(\frac{a_{3W} + a_{3E}}{2} \right) - m_2 \left(\frac{a_{2W} + a_{2E}}{2} \right) - m_1 \left(\frac{a_{1W} + a_{1E}}{2} \right) \quad (4.70)$$

$$\text{Story 2 } Q_{ac2} = -m_4 \left(\frac{a_{4W} + a_{4E}}{2} \right) - m_3 \left(\frac{a_{3W} + a_{3E}}{2} \right) - m_2 \left(\frac{a_{2W} + a_{2E}}{2} \right) \quad (4.71)$$

$$\text{Story 3 } Q_{ac3} = -m_4 \left(\frac{a_{4W} + a_{4E}}{2} \right) - m_3 \left(\frac{a_{3W} + a_{3E}}{2} \right) \quad (4.72)$$

$$\text{Story 4 } Q_{ac4} = -m_4 \left(\frac{a_{4W} + a_{4E}}{2} \right) \quad (4.73)$$

Table 4.16 Story mass and accelerometers

Story	mass [ton]	a_W [m/s ²]	a_E [m/s ²]
1	12.1	CH 105	CH 106
2	12.1	CH 107	CH 108
3	12.0	CH 109	CH 110
4	17.8	CH 112	CH 115

4.5.8 Out-of-plane displacement of the first story

Lateral displacement of the Second Floor, measured from the top of the foundation beam

$$\Delta_{2F} = - \left(\frac{\delta_{CH\ 261} + \delta_{CH\ 262}}{2} \right) \quad (4.74)$$

Change in length due to the lateral displacement of 1st story

$$\Delta_{lat} = \sqrt{L_0^2 + \Delta_{2F}^2} - L_0 \quad (4.75)$$

Displacement in the NS direction

$$\Delta_{NS} = \frac{\delta - \Delta_{lat}}{\cos \theta} \quad (4.76)$$

$$\delta = \begin{cases} \delta_{CH\ 49} \text{ (Line 1)} \\ \delta_{CH\ 50} \text{ (Line 3)} \end{cases}, \quad L_0 = 1925 \text{ mm}, \quad \theta = 38^\circ$$

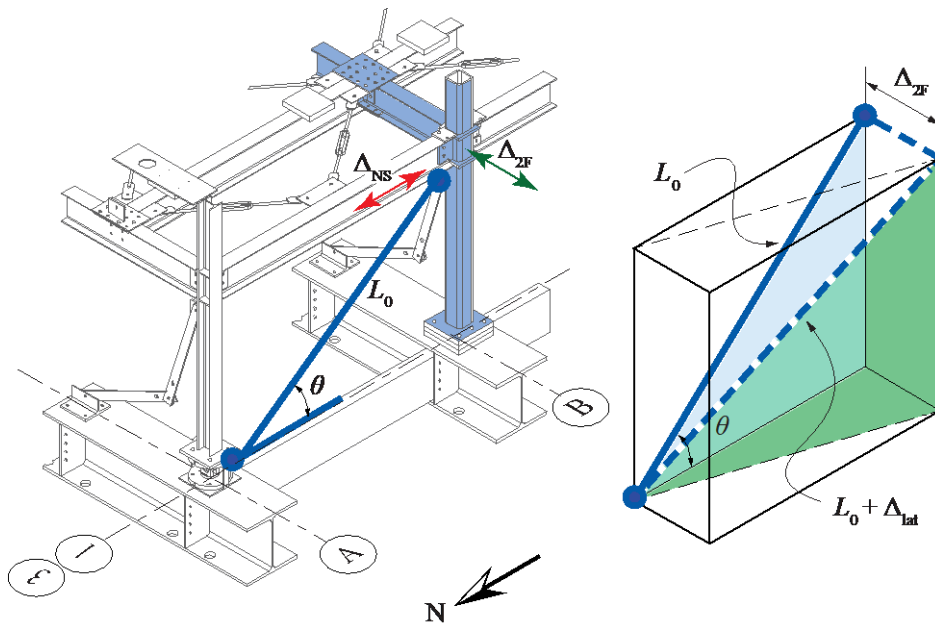


Figure 4.23 Out-of-plane displacement of the first story.

4.5.9 Contraction of the Interior Column

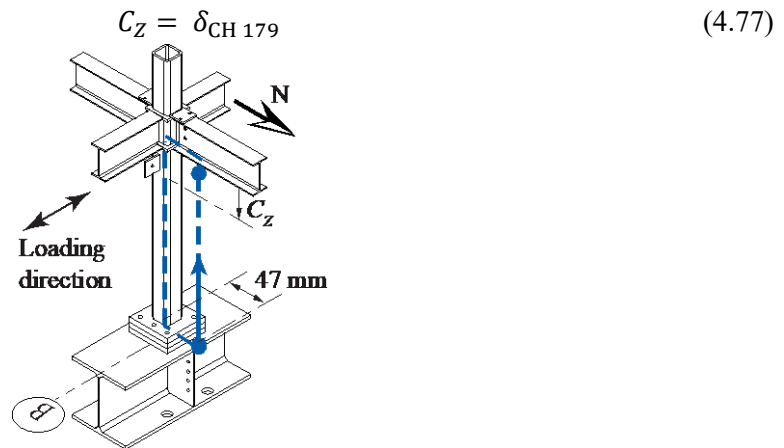


Figure 4.24 Contraction of the interior column.

5. SHAKE-TABLE COLLAPSE TESTS: TEST RESULTS

5.1 VERIFICATION OF THE DATA REDUCTION SCHEME

Figure 5.1 plots the story shear measured from the main frame columns vs. story shear obtained from acceleration response during the 100%-1 excitation. Both measurements show good agreement since the plots are very close to the 45° line. This also indicates that the inertia forces were transferred to the main frame as intended.

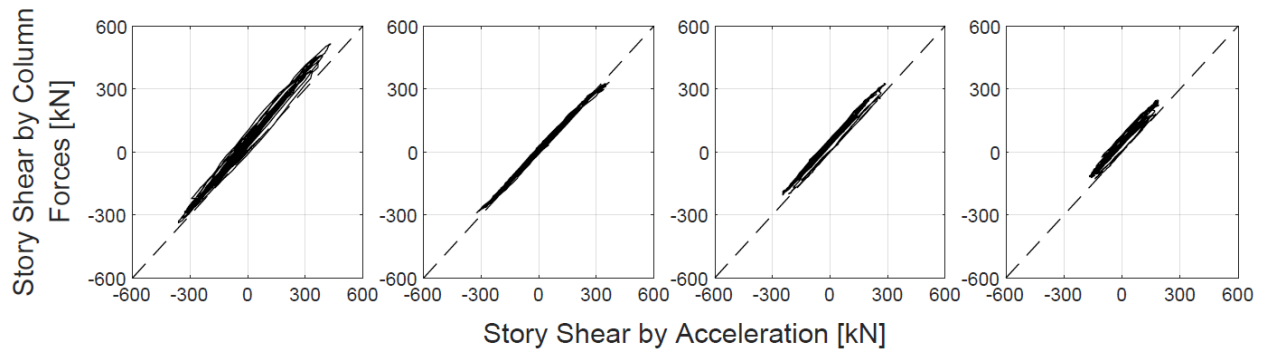


Figure 5.1 Comparison of story shear from column internal forces and from acceleration, 100%-1 excitation, at the: a) 1st, b) 2nd, c) 3rd, and d) 4th stories

The elastic stiffness of beams, panel zones, and stories computed using measured material properties match the measured initial stiffness of those components in the shake table tests. String pot measurements of the 2nd story beam displacements at the external frame C match those obtained from optical sensors. From optical sensors placed on the 2nd story concrete mass and 2nd story beams, no slippage was observed. Only the slippage at 3rd story could not be verified by independent measurements.

5.2 SLIPPAGE OF THE CONCRETE MASS ON STORY 2

Because the instruments to measure story lateral displacements in stories 2 to 4 were mounted on concrete weights and not directly into the lower story's steel beam, it is necessary to check any slippage between the concrete and the steel.

To do so, the measurements on the concrete weight and steel beam on the second story, obtained by optical sensors, are used as independent measurements. In the first place, since the sensors were installed on the Frame C, the 2nd story displacement measured from string pot was compared to the displacement of the same story obtained by optical sensors. Figure 5.2 shows the result of the comparison for the last excitation. Both measurements match well. Therefore, the string pot measurements were verified.

Referring to Figure 5.3, the next step is to obtain the slippage of the mass as the difference between the lateral displacement of points PtA and PtB (on the concrete weights) and PtC (on the steel beam). The comparison of the displacements of the three points are very similar, with a difference less than 1 mm. This check was also conducted for the last excitation. Therefore, it was verified that there was no appreciable slippage of the 2nd story mass. Unfortunately, there was no independent measurement to verify the slippage of the concrete masses on the 3rd story.

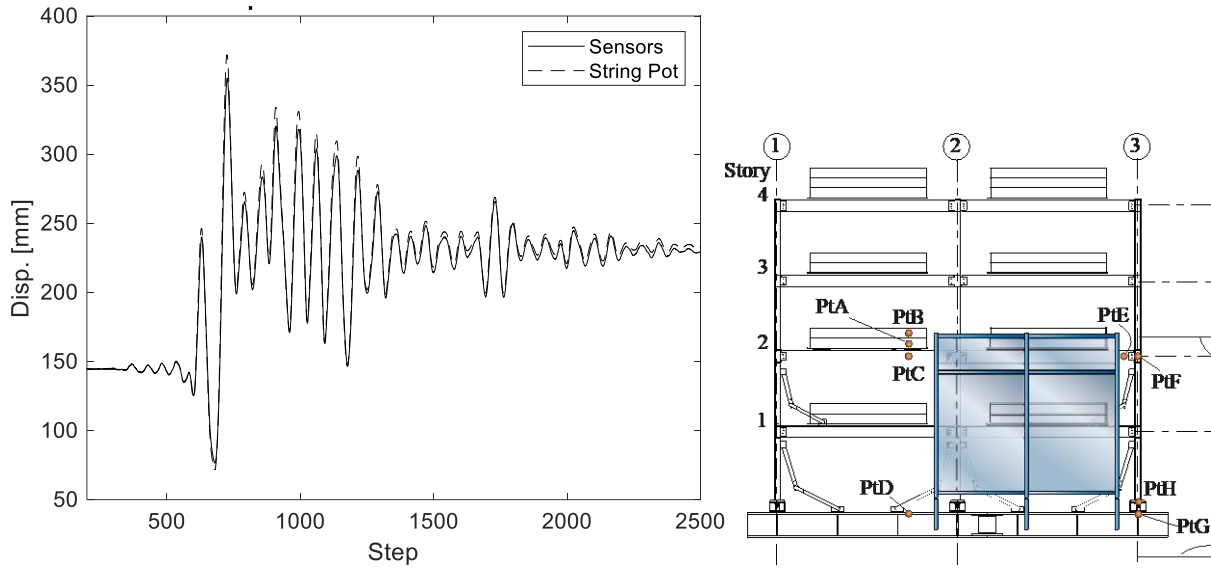


Figure 5.2 Verification of 2nd story lateral displacement by optical sensors and string pot, 100%-4.

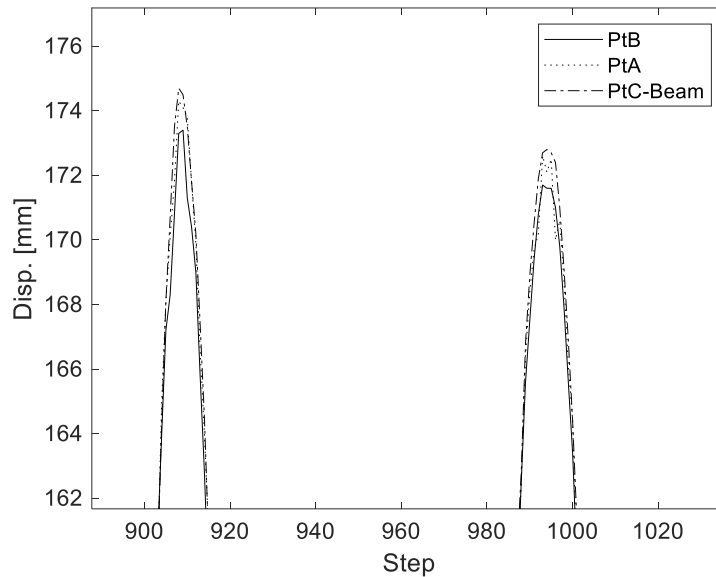


Figure 5.3 Comparison of displacement of the concrete weight and steel beam on the 2nd story.

5.3 LOAD TRANSFER

Figure 5.4 shows the Free-Body Diagram used to estimate the axial loads P_L and P_R in the outer beam segments of the shake-table test Specimen. The axial force at the central beam segment, P_C , was computed from strain gage measurements. P_L is the difference between column shears above and below the joint. By equilibrium, the inertia force I_F is the difference between P_R and P_L . Figure 5.5a shows that P_R computed from equilibrium match the axial loads computed from strain gage measurements at the outer beam segments during elastic response. Also, Figure 5.5b shows that the jumps in the axial load diagram match the inertia loads transferred to the beam. Therefore, all the inertia force was delivered through the main frame beams. Figure 5.4 also shows the bending moment diagram distribution. The central portion of the diagram was estimated from strain gage measurements. Since the strain gages on the outer beam segments yielded, those readings are valid during elastic response only. As mentioned above, the mass-support beams are connected to the floor diaphragm and the main frame beams at the top flange. Therefore, the inertia force induced jumps to the bending moment diagram, as shown in the figure. Figure 5.5c indicates that the concentrated moment due to the transferred inertia forces to the beam closely matches the jump in the beam bending moment diagram. Therefore, the inertia loads were

transferred to the main frame as expected.

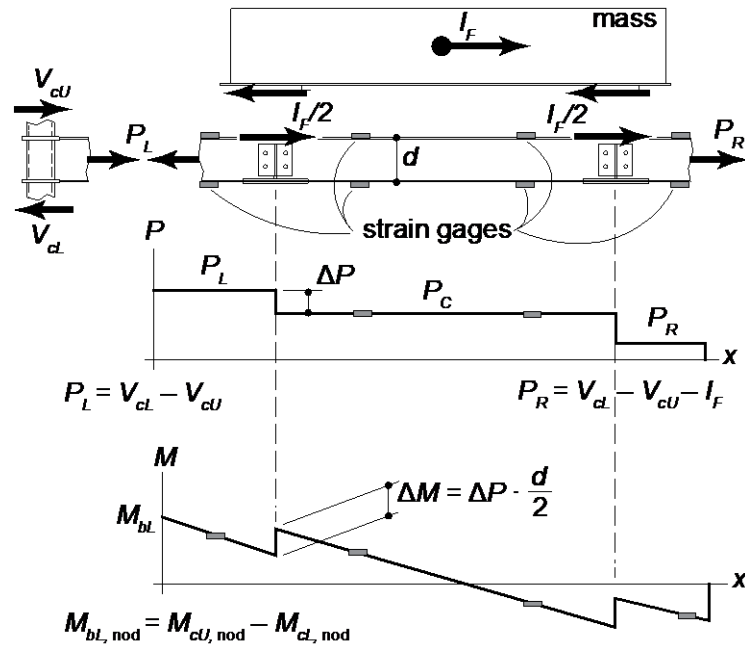


Figure 5.4 Equilibrium of horizontal forces and internal moment distribution in the shake-table test beams. Only horizontal forces are shown for clarity.

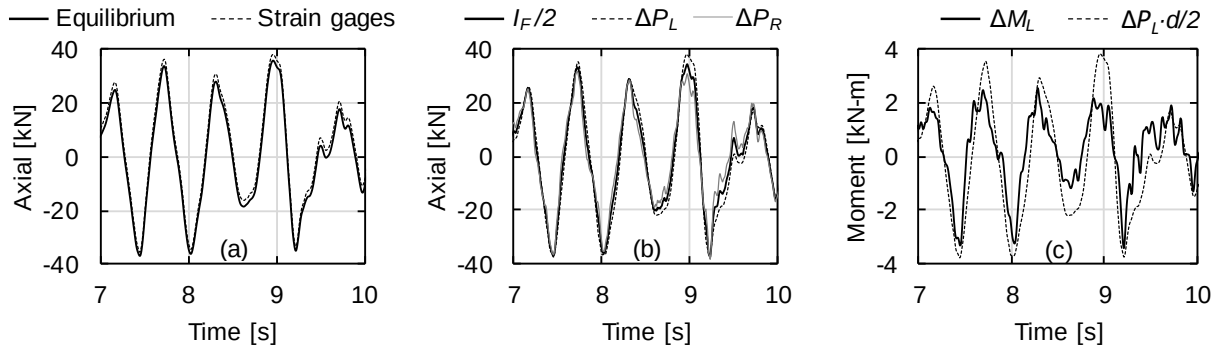


Figure 5.5 Load delivery checks: a) axial force in the outer beam segment, b) concentrated axial, and c) concentrated moments.

5.4 LOAD TAKEN BY THE GRAVITY FRAMES

Figure 5.6 compares the story shear of the main and gravity frames for the first two stories. The figure indicates that the gravity frames developed minimal lateral-load resistance, as intended. However, during the 100% excitations, the gravity frame columns of the second story took lateral resistance of 10% the total story shear to accommodate the displacements of the main frame. The column ends developed bending moments of $0.70M_p$.

The participation of the gravity frames in the lateral resistance of the third and fourth stories was estimated from the difference between the story shear by inertia forces and the story shear of the main frame, assuming that the damping force is negligible. The estimations showed that the shear taken by the gravity columns in the upper stories did not exceed the shear taken at the 2nd story during the 50%, 100%-1, and 100%-4 excitations. The plots of the first two stories also show the lateral resistance of the gravity frames from strain gage measurements. The estimated and measured values agree well. Therefore, the estimation of lateral resistance in the upper stories is reliable. The agreement also indicates that the damping force was small.

The slotted hole shear connections allowed slanting of the gravity columns, but the column continuity provided lateral resistance when subjected to large deformations. This behavior was contrary to the design intention. One possible way to avoid this problem in future experiments may be hinging the column ends at each story.

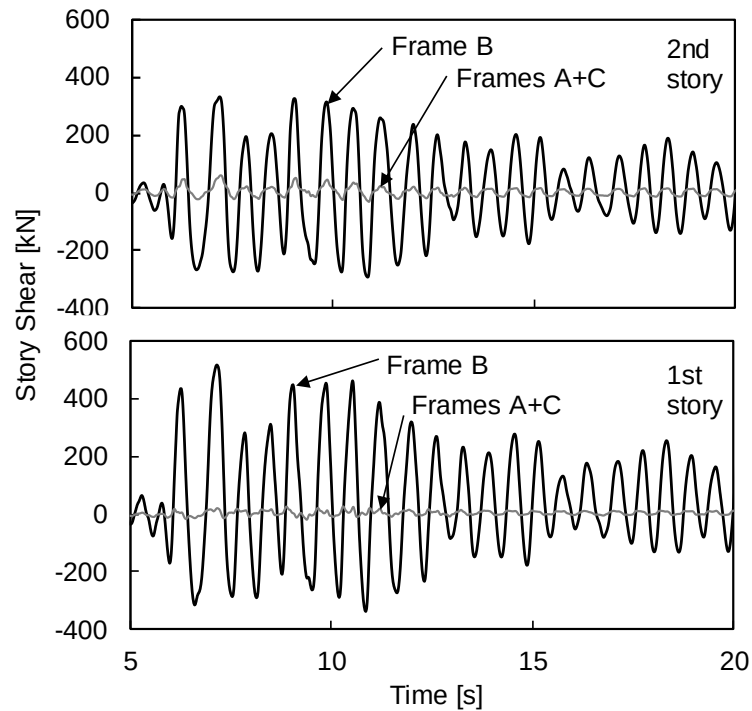


Figure 5.6 History of the story shear resisted by each parallel plane frame, 100%-1 excitation

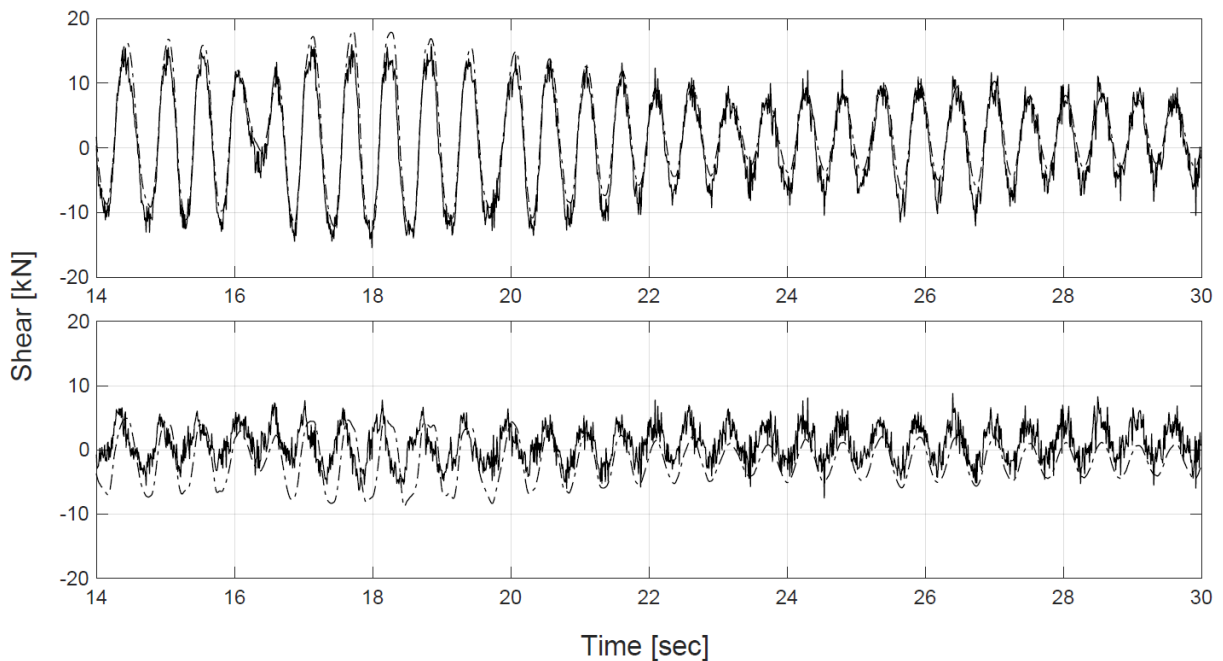


Figure 5.7 History of story shear resisted by the two gravity frames from strain gages and from the difference between inertia and main frame shear.

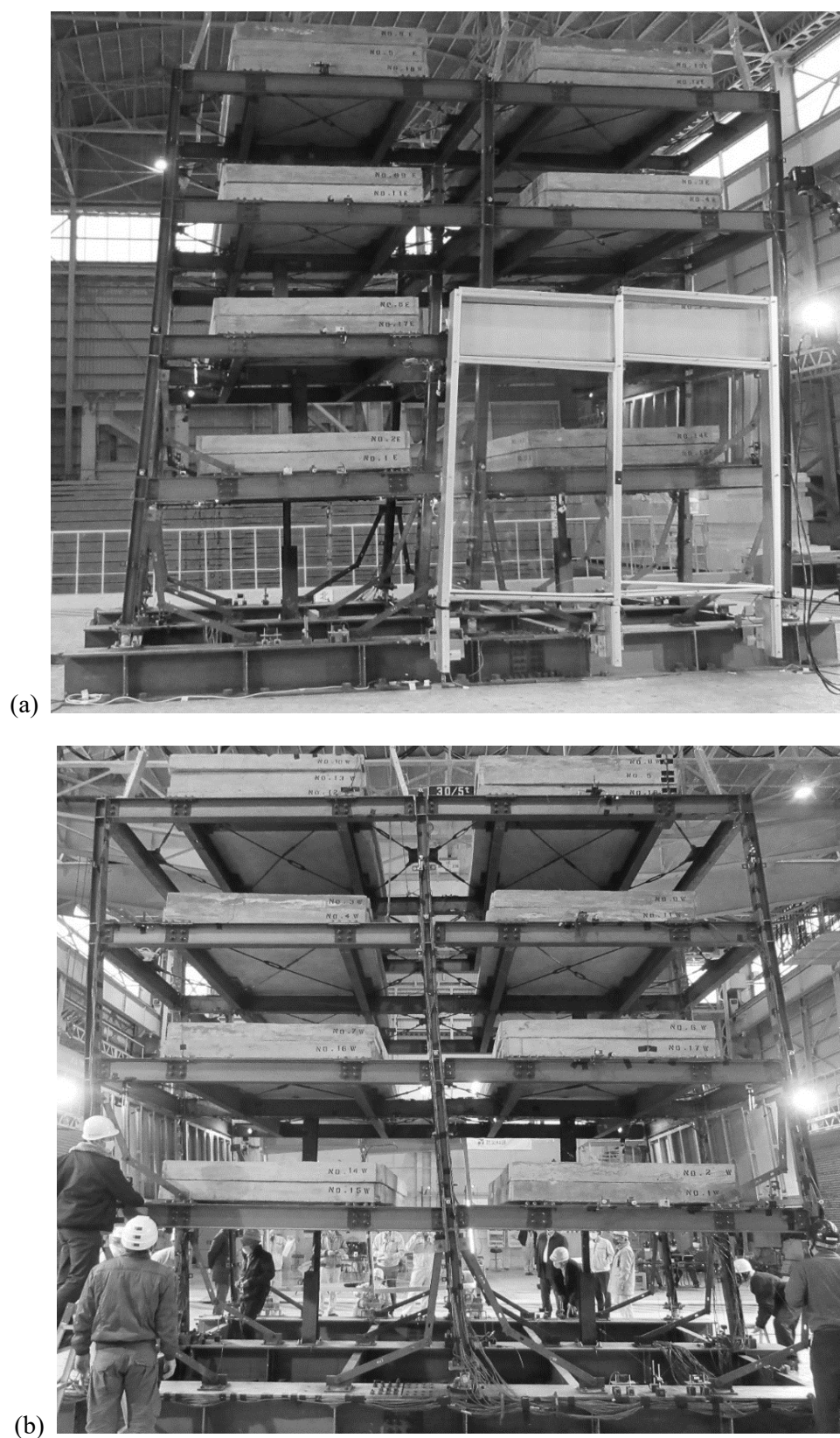


Figure 5.8 Deformed configuration of the gravity frames after the tests: a) Frame C, b) Frame A.

5.5 EFFECT OF GRAVITY

The effect of gravity was measured with the strain gages on the columns. Unfortunately the strain gages on the beams were not initialized to read the change in strain during the construction stage. Therefore, the gravity moments were estimated from the comparison with the gravity moments estimated by tributary area. Figure 5.9 shows the comparison between the tributary area moments (Figure 5.9a) and the moments obtained during the construction stage (Figure 5.9b), when all the masses were already in place. Figure 5.9c shows the history of strain of the first story left column during the last stage of construction. The data was very noisy due to the construction, a time instant when fewer people were working on the specimen was chosen. At that time instant, only two men were working on the 4th story west side masses. The data was smoothed with a 100-second average moving window.

The comparison shows good agreement between the tributary load estimation and the strain gage measurement in the columns. Therefore, the gravity moments on each element were computed using tributary area load.

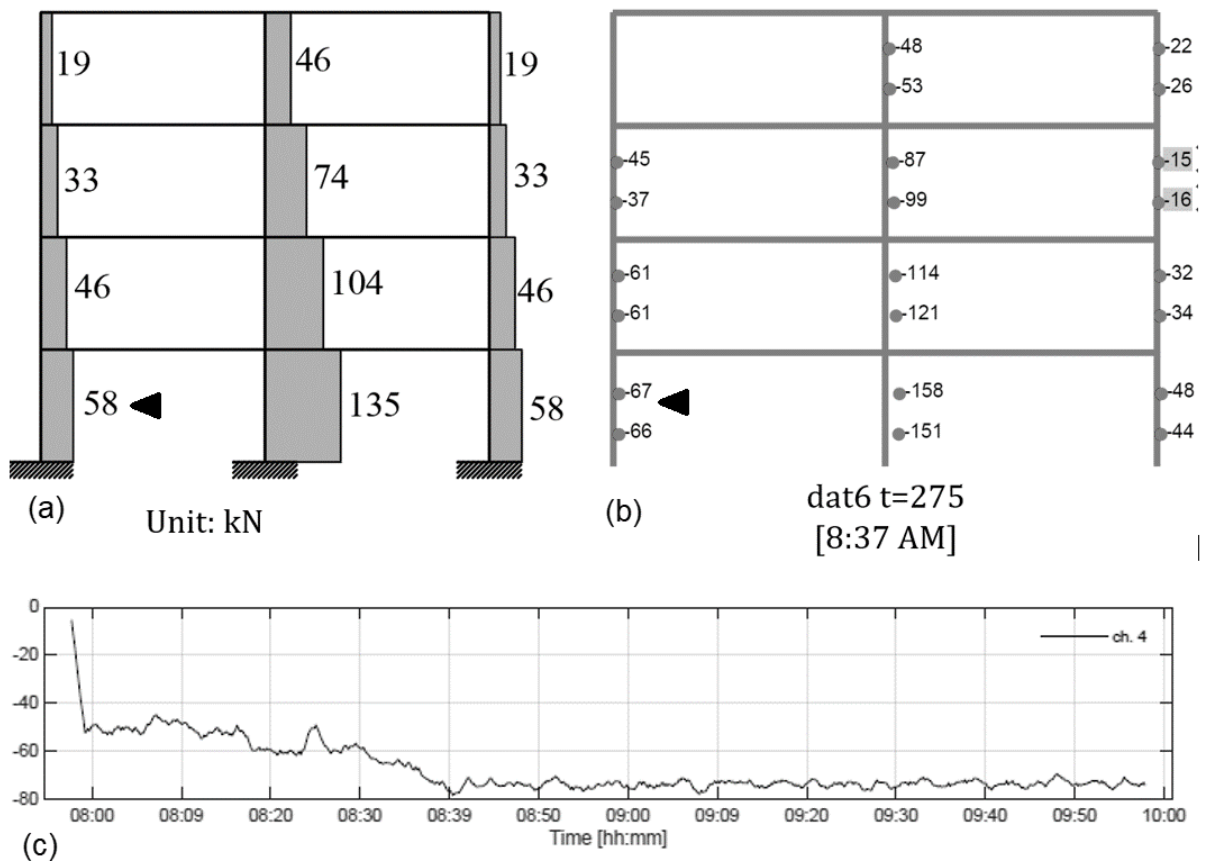


Figure 5.9 Estimation of gravity effect: a) Moments in columns by tributary area, b) moments in columns by strain gages during construction, c) example of smoothed strain history in the first-story left column.

5.6 ELASTIC VIBRATION PERIOD

Figure 5.10 shows the smoothed transfer function from the five white-noise excitations conducted to identify the system's natural period. The natural frequencies correspond to the frequency at each peak in the plot. Figure 5.11 shows the transition of the specimen's first and second natural vibration periods identified from white noise excitations conducted between primary excitations. However, since no white noise was conducted beyond the first 100% excitation, the variation of the period to the end of the 100% tests was completed using the transfer function of the last 10-sec segment of the input acceleration and roof-floor acceleration response. The acceleration history was not filtered. The expected periods,

computed from elastic 2-D frame analysis are shown in dashed lines. At the measured first natural period, the spectral acceleration of 24 m/s^2 would push the specimen to 0.05-rad story drift. This acceleration is 2.2 times the acceleration corresponding to the expected plastic strength of the specimen based on mill test report values. The period remained practically constant even after the 50% excitation when the second and third-floor beams and the column bases yielded substantially as will be mentioned later. This result suggests that steel systems can retain their elastic properties even after experiencing significant yielding of primary components. Past research noted the same behavior. The natural period elongated to 0.64 s after the 4th 100% excitation due to the fracture of the second-floor beams.

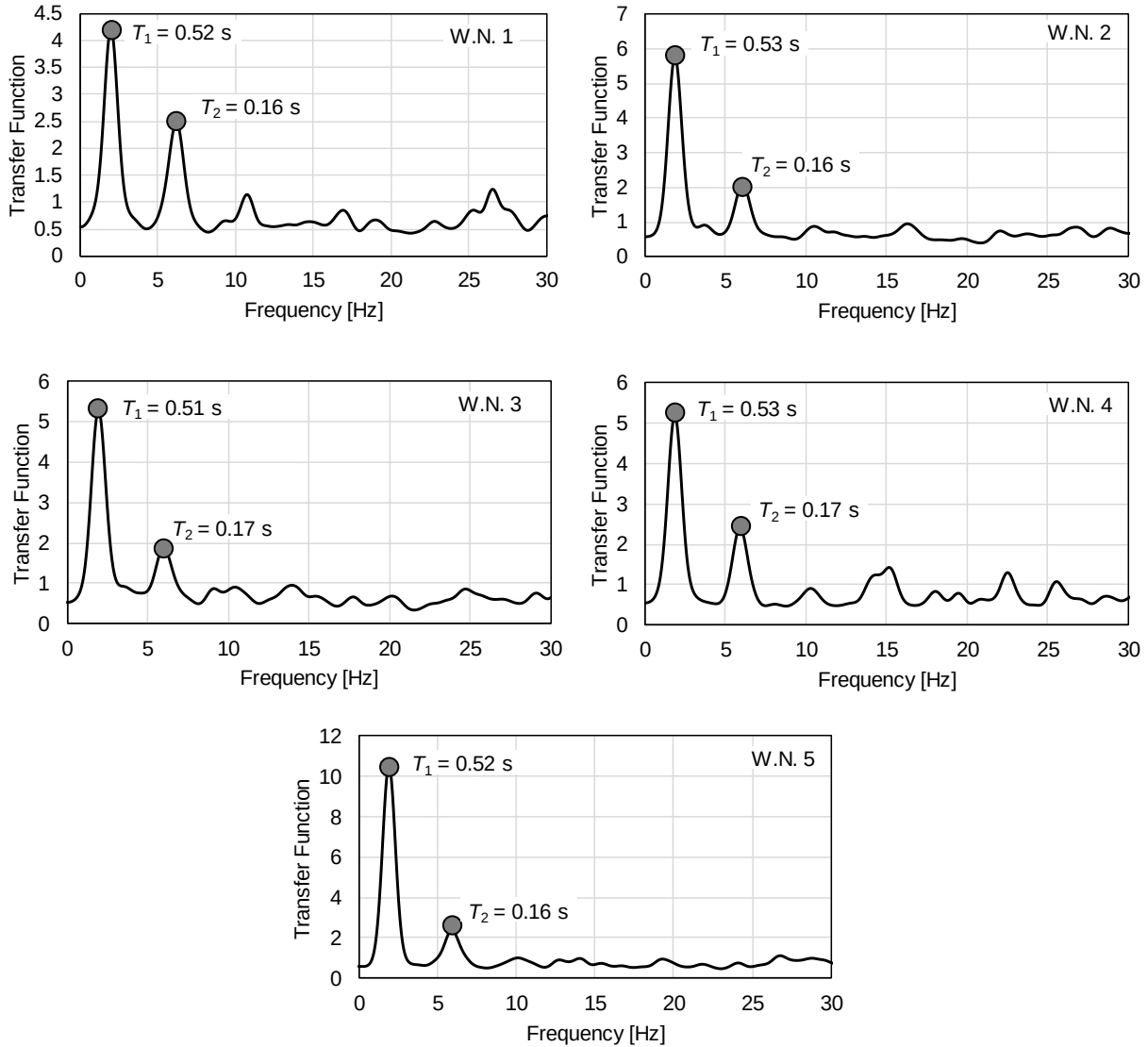


Figure 5.10 Transfer function to identify the natural period from white noise excitations.

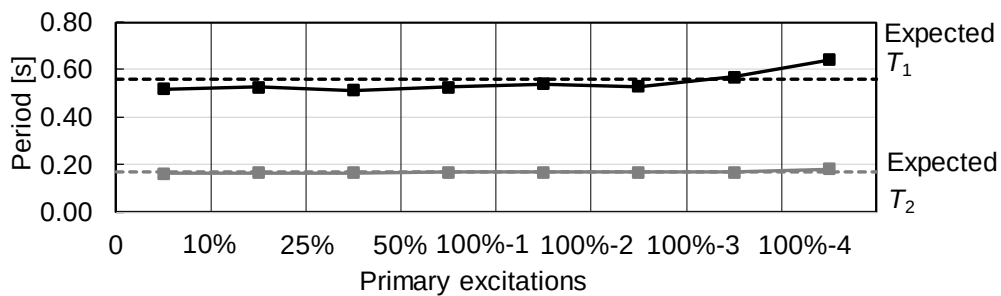


Figure 5.11 Variation of the natural period between excitations.

5.7 GLOBAL BEHAVIOR

Figure 5.12 shows the distribution of the maximum story drift ratio and floor acceleration. The story drift was larger in the first two stories, a tendency that grew larger after each 100% excitation. Each of the 100% motions induced story drift of +0.08 and -0.02 rad, and added a residual story drift of +0.02 rad. A maximum story drift of 0.15 rad was measured at the first two stories during the last excitation, leaving a residual story drift of 0.09 rad. The maximum acceleration exceeded 1g during the 100% excitations and was rather consistent over the height.

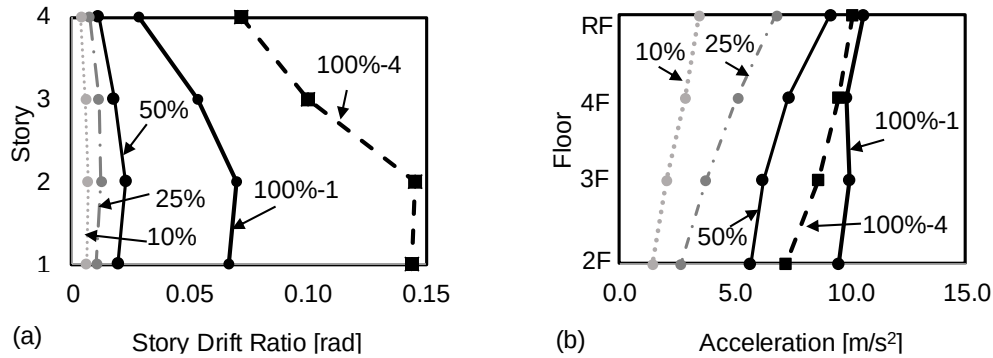


Figure 5.12 Peak global response: a) Story drift ratio, and b) floor acceleration.

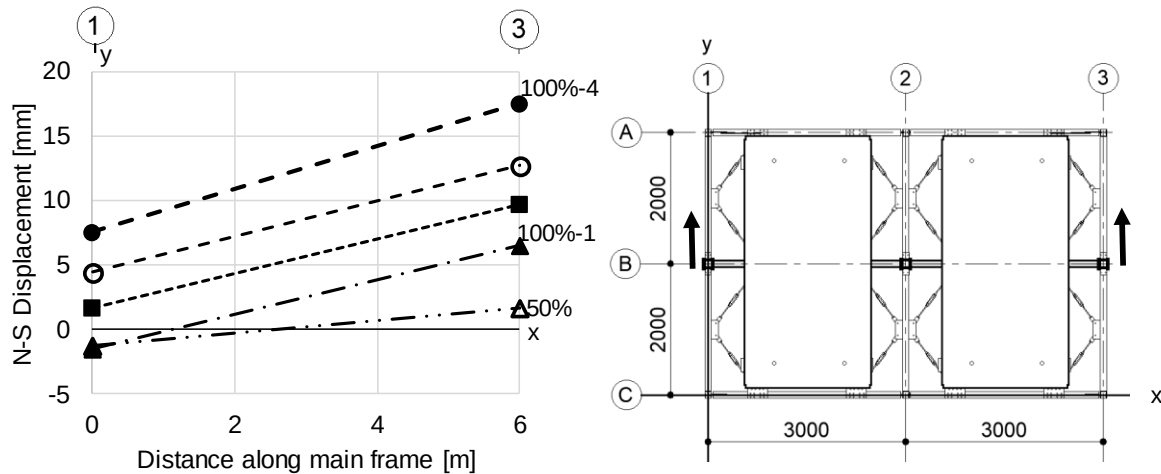


Figure 5.13 Twisting of the first story.

Figure 5.14 presents the bending moment distribution normalized by the plastic strength M_p based on strain gauge measurements sampled from the 50%, 1st 100%, 3rd 100%, and 4th 100% excitations at peak deformation. The figure indicates the zone where the plastic strength was exceeded, as well as the locations where the fracture occurred. During the 50% excitation, the second and third-floor beams developed plastic hinges at both ends and the first-story columns yielded at the bases. During the 1st 100% excitation, the system formed a sidesway mechanism involving the lower three stories. Video recording captured local buckling of the beam flanges at the exterior joints of the second floor. Yielding continued to spread during the subsequent 100% motions until the bottom flange of the external joint beams on the second floor fractured.

Fracture of the exterior second-floor beams drew the column inflection point of the external columns toward the connection, forming two-story tall columns. The gravity loads, which were overproportioned for this scaled test, shifted the beam bending moment toward the lateral displacement direction. The inflection points in the internal connection were more or less centered.

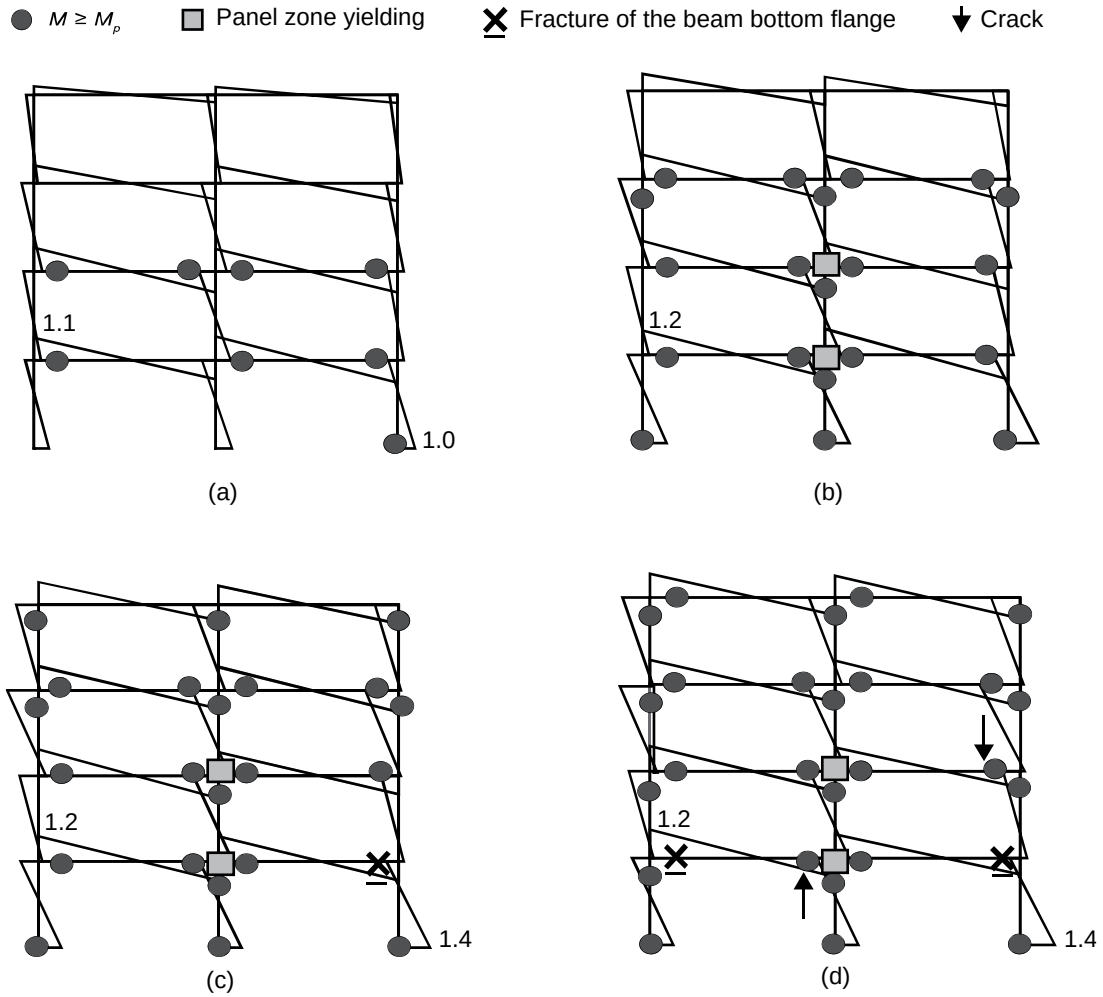


Figure 5.14 Normalized bending moment diagram at maximum deformation and damage propagation at the excitations: a) 50%, b) 1st 100%, c) 3rd 100%, and d) 4th 100%.

5.8 BEAM BEHAVIOR

Figure 5.16 and Figure 5.16 show the beam-end moment versus rotation response measured at the beam-to-column connections along the height of the specimen during the four 100% excitations.

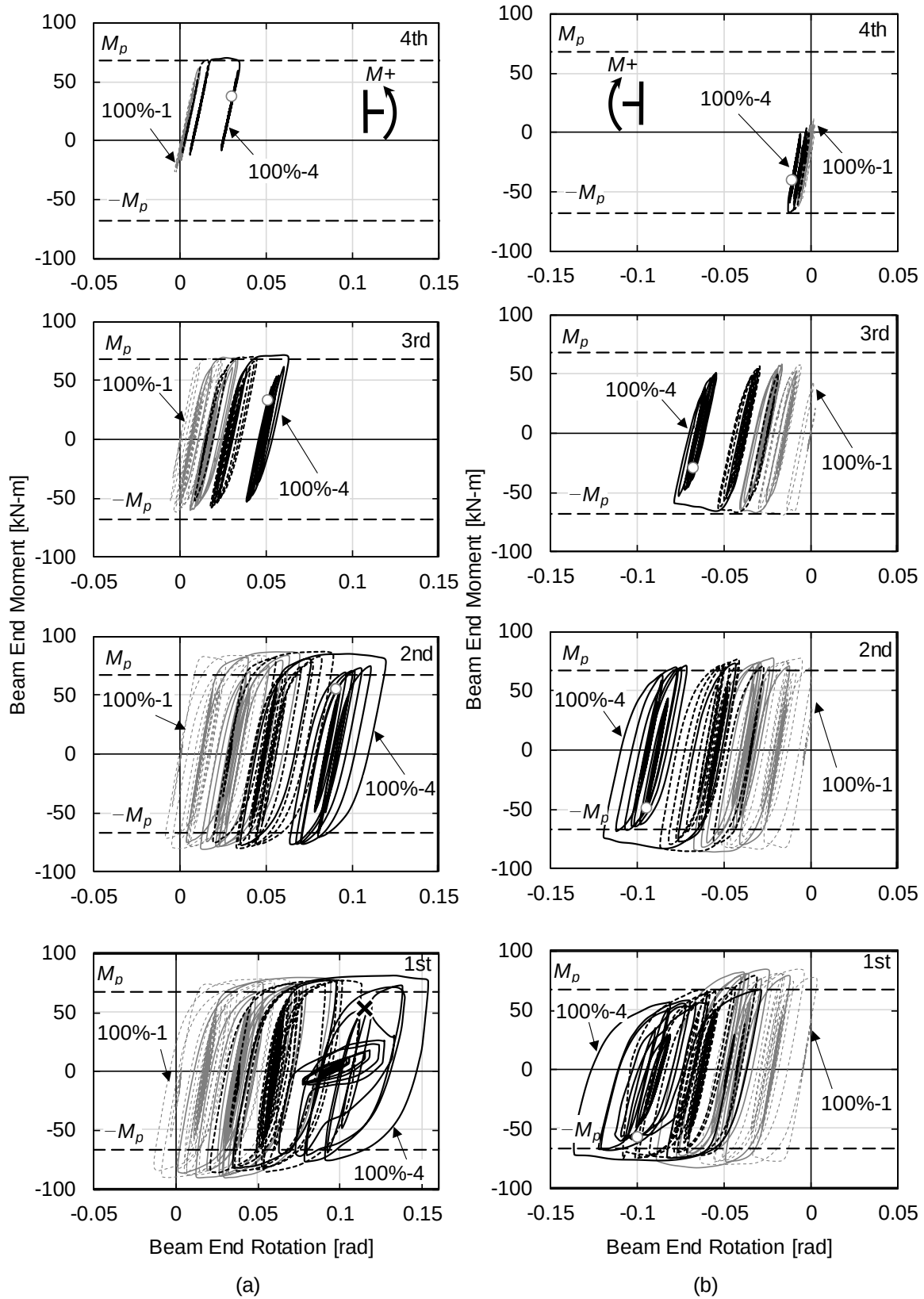


Figure 5.15 Beam end moment vs. rotation of the left-side beams at each story: a) connection to the left exterior column, b) connection to the interior column.

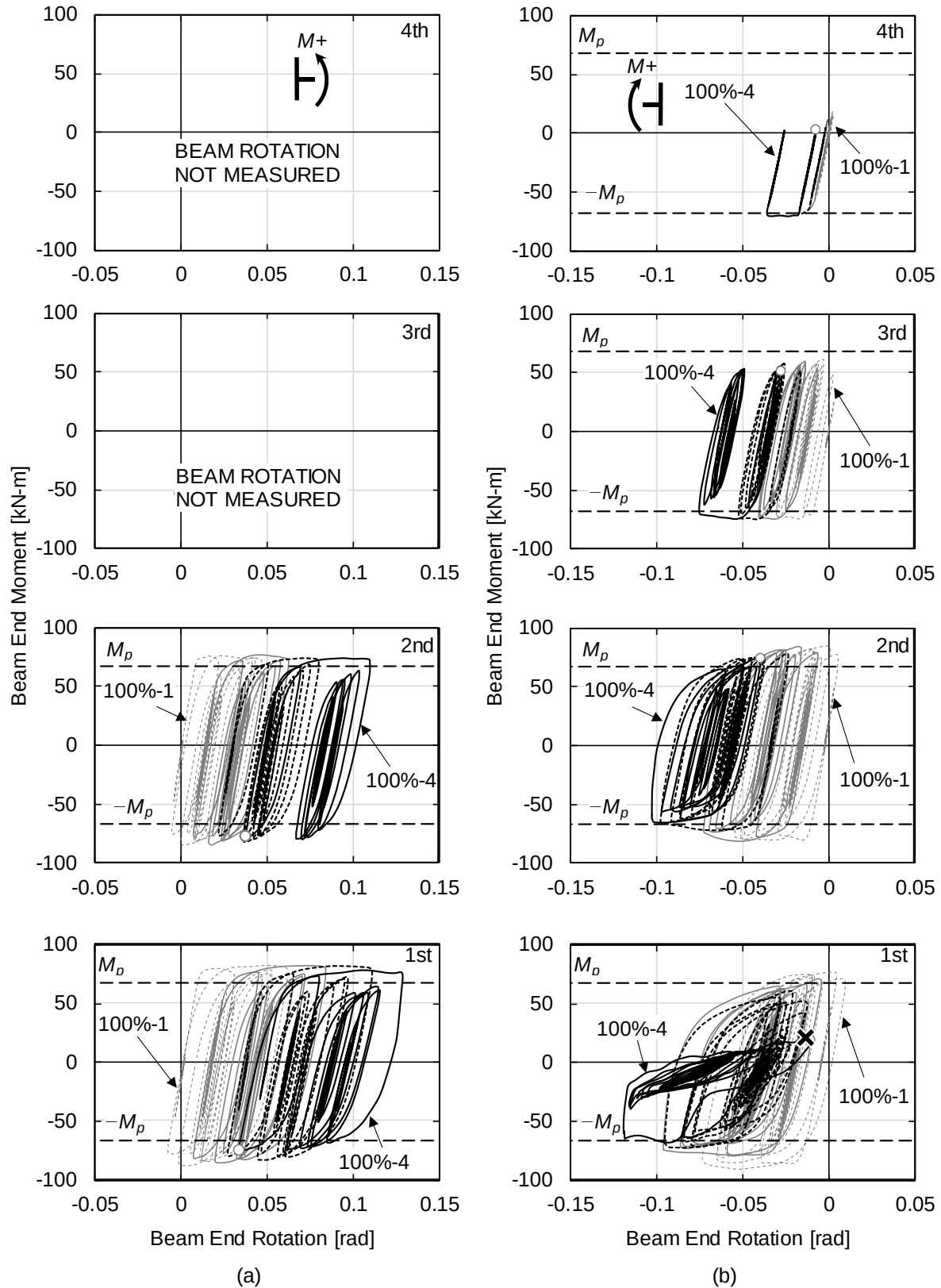


Figure 5.16 Beam end moment vs. rotation of the right-side beams at each story: a) connection to the interior column, b) connection to the right exterior column.

During the 2nd 100% excitation, the second-floor beam degraded in strength due to local buckling of the bottom flange. During the 3rd 100% excitation, a crack formed at the termination of the CJP groove weld, at the location indicated in Figure 5.17, then propagated along the weld bevel until it rested 25

mm (12% of the beam depth) into the web. During the 3rd 100% excitation, the beam bottom flange completely fractured to the state shown in Figure 5.17b. The instant when the bottom flange fractured is marked by “x” in Figure 5.16a. This beam maintained appreciable negative strength because the fracture did not travel the entire beam depth. As strength deteriorated in the second-floor beam, the moment demands in the third-floor beam decreased. The fourth-floor beam yielded but to a smaller extent than at the second and third floors. The roof beam yielded during the last 100% excitation.

Figure 5.18 and Figure 5.19 show another 1st story beam that followed a similar fracture pattern. The bottom flange fractured from instants E to F of the 4th 100% excitation with no significant effect on either the beam (Figure 5.19a) or the story (Figure 5.19b). In the positive excursion of the next cycle, the crack propagated into the beam web very rapidly (instants G to H), producing an evident negative stiffness of the beam and the end moment dropped by 33% (Fig. 14a) but the story maintained its shear strength well above the structure’s plastic strength Q_p ($= 320$ kN, Figure 5.19b). Despite the severe strength and stiffness degradation, the beam kept negative moment strength due to partial fracture.

5.9 COLUMN BASE BEHAVIOR

Figure 5.20a plots the base moment at the central column vs. story drift response of the first story during the four 100% excitations. Despite the large story drift of +0.08 and -0.02 rad induced by each of the 100% excitations, no strength degradation occurred in this column. The normalized P-M interaction in Figure 5.20b indicates no significant variation of column axial load for the interior column (-0.07, -0.06 P_y), whose initial axial due to gravity loads was 6% of its axial yield strength. Bending moments were dominant and plasticity was developed at the negative moment side (towards the positive loading direction). Figure 5.20c shows the deformed state of that column after the test. Perhaps in part owing to the small width-to-thickness ratio of 10, which was easily within the limit $33 \sqrt{235/F_y} = 24$ specified for hollow square columns by AIJ, the columns exhibited very ductile behavior showing no sign of local buckling and developing no sign of fracture in the CJP groove welds to the column base plate.

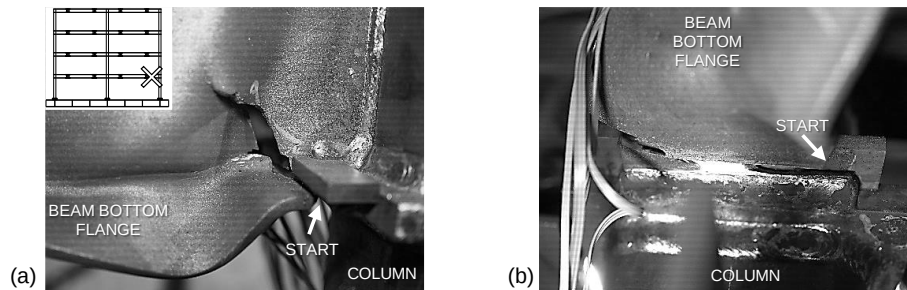


Figure 5.17 Fracture of 1st story beam at the end of the tests: a) front view, and b) bottom view.

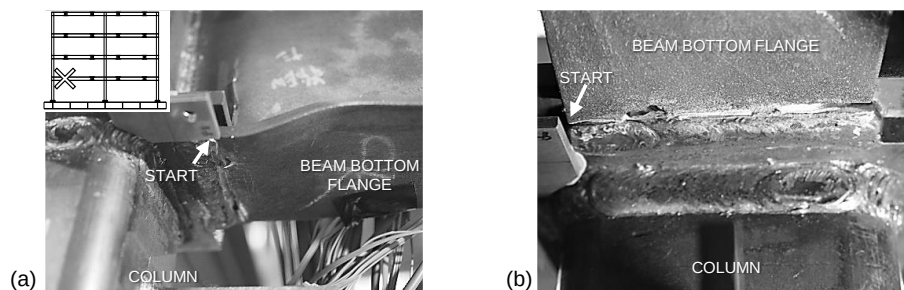


Figure 5.18 Fracture of 1st story beam during 4th 100%: a) front view, and b) bottom view.

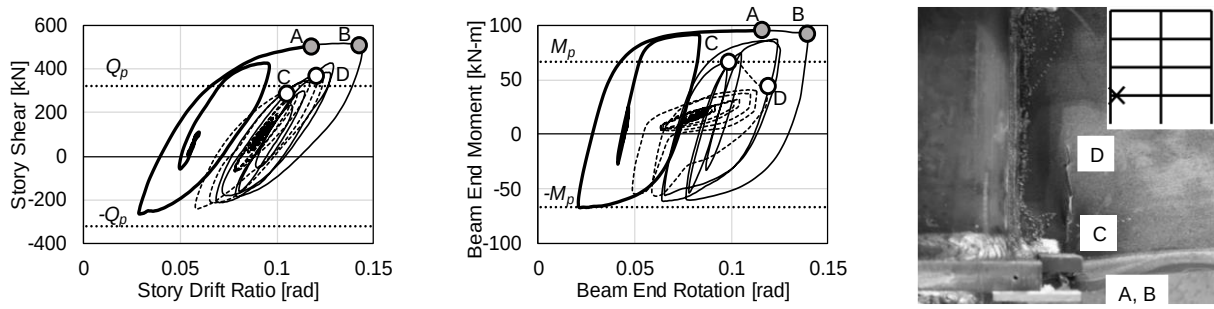


Figure 5.19 Response during 100%-4: a) story shear versus story drift of 1st story, b) moment versus rotation of the 1st story beam, and c) beam fracture at the end of the test.

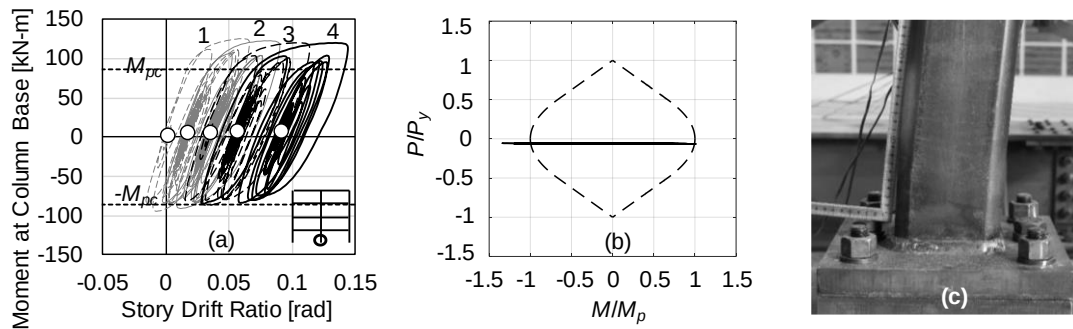


Figure 5.20 Response of the 1st story and 2nd story interior panel zone during the a) 1st 100%, b) 3rd 100%, and c) 4th 100%.

5.10 PANEL ZONE

Figure 5.21 shows the shear force versus shear deformation of the panel zones in the exterior and interior columns on the second and third floors for the 1st and 4th 100% excitations. The panel zones yielded at both floors during the 1st 100% excitation. Afterwards, the residual deformation of the third-floor panel zone grew larger than that of the second-floor. This indicates that moments were redistributed to the third floor after degradation and fracture of the second-floor beams. Fracture of the second-story beams during the 4th 100% excitations resulted in larger deformations in the third-floor panel zone, while shear demands and deformation in the second-floor panel decreased. The panel zones of the exterior columns, not shown in the plots, remained elastic to the end of the tests because they had 70% of the strength but less than 50% of the demands of the interior column panel zones.

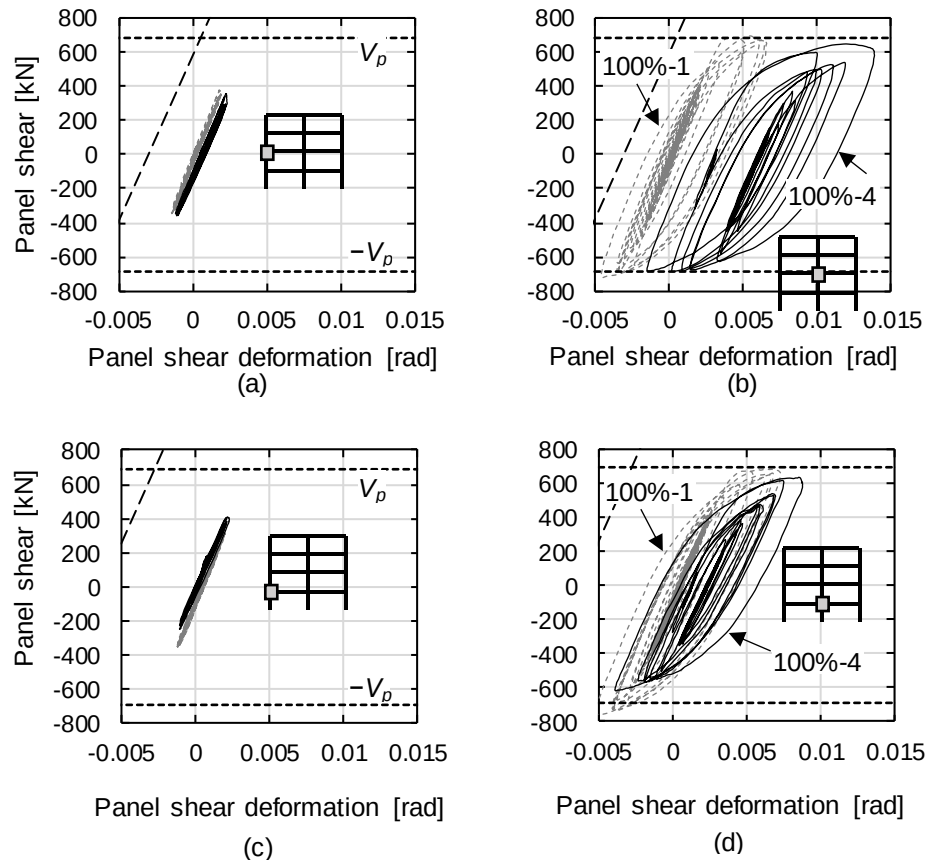


Figure 5.21 Panel zone shear versus shear deformation at the first two floors during the 100%-1 and 100%-4 at the: a) 3F left exterior, b) 3F interior, c) 2F exterior, and d) 2F interior joints.

6. COMPONENT TESTS

6.1 INTRODUCTION

Three beam-and-column assemblages were constructed simultaneously with the shake-table specimen using the same heat of steel, design, and detailing. The subassemblages represented two external and one internal joint extracted at the middle of the beams and columns of the shake-table test moment-resisting frame. The objective of the component tests was to reproduce the behavior of the entire moment-resisting frame system by beam-column subassemblages. Therefore, the tests were planned to simulate the lateral bracing and boundary conditions of the shake-table test beam-and-column joints as much as possible. This chapter presents the test plan and results of the three component tests.

6.2 TEST PLAN

6.2.1 Test Setup

Figure 6.1 shows a general view of the test setup used for the three component tests. The test specimen, shown in black, was placed on three parallel deep beams. The reaction wall consisted on three steel columns that were braced at the top to provide sufficient stiffness against the jack reaction. A 250-kN hydraulic jack was installed on the top of the three reaction columns to subject the test specimen to cyclic deformations.

The lateral support system consisted of 8 H-175 columns located at the beam ends and midspan, to provide lateral restraint to the specimens. The four columns located at the beam midspan were taller to allow the lateral support of the column top. Horizontal C-150 channels were used to connect the columns along the specimen's longitudinal direction. The lower channel was aligned with the top flange of the specimen's beams. The upper channel was aligned with the jack centreline. Both pairs of tall columns were braced with turnbuckles to provide adequate out-of-plane stiffness. The shorter columns, located at the beam ends, were not braced since the out-of-plane force at those locations was expected to be minimal.

Bearing rollers were used at each lateral support location to restraint out-of-plane displacements but allow in-plane motions. This was one key difference with the beam lateral bracing condition in the shake-table tests because in that case a horizontal diaphragm was connected to the beam top flange at the bracing locations. The rollers were intended to bear against thick reaction plates. The gap between the rollers and the plates was set to 1 mm before the tests.

The column base was laterally supported by two angles that bore against reaction plates. In this case, since the bracing was plate-to-plate, the contacting faces of both the angles and the plates had Teflon sheets glued to minimize the friction.

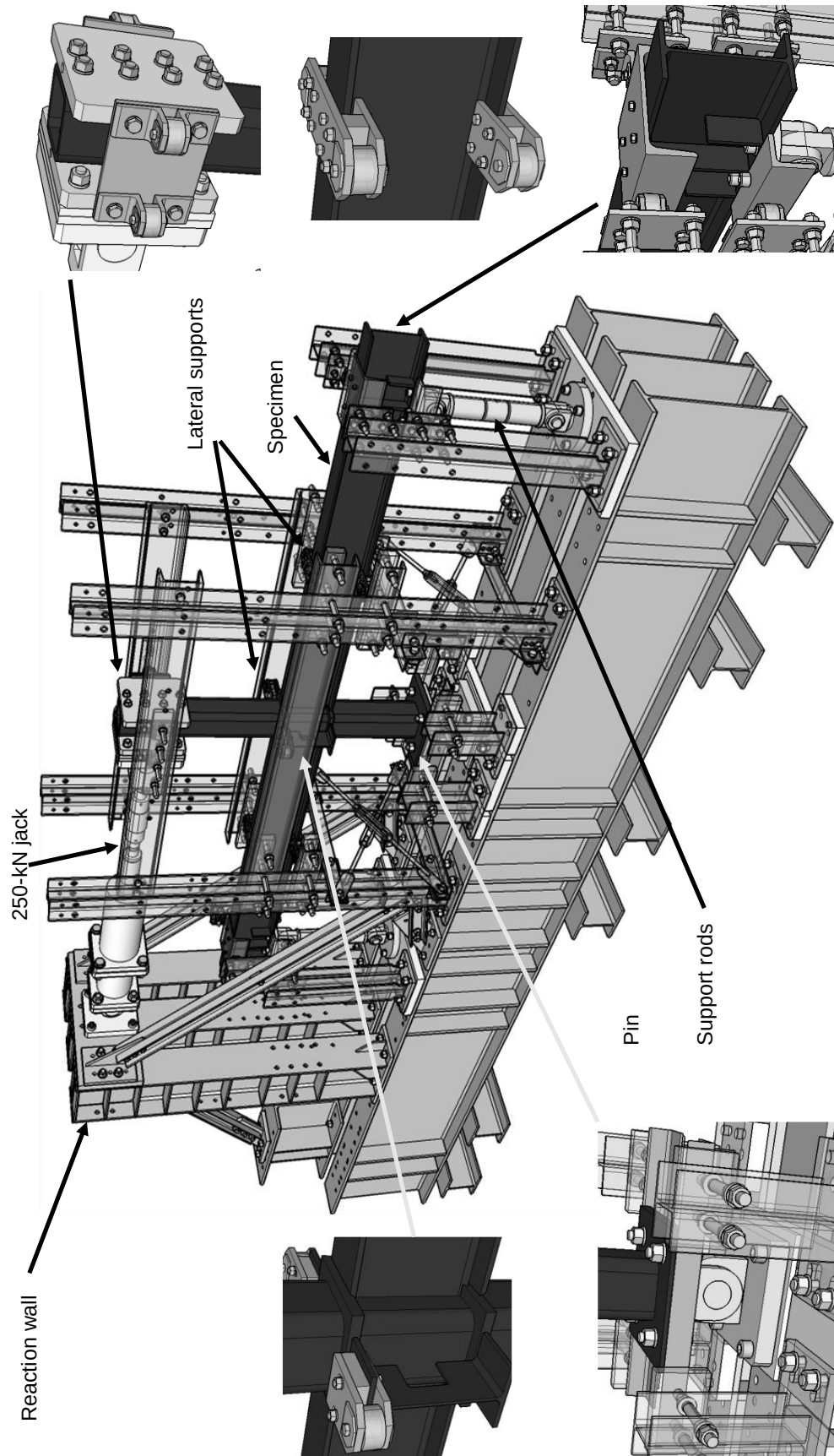


Figure 6.1 Component test setup.

6.2.2 Test Specimen

The three beam-and-column assemblages were constructed simultaneously with the shake-table specimen using the same heat of steel, design, and detailing. Therefore, the same mechanical properties of the shake-table test specimen also apply to the component test specimens.

As shown in Figure 6.2, the subassemblages represented two external joints (T1 and T2) and one internal joint (T3) extracted at the middle of the beams and columns, assuming that the inflection points due to lateral loads of the moment frame occur at those locations. During the tests, the subassemblages were lateral braced at the same locations where the moment frame of the shake-table test specimen was braced by orthogonal beams. The beam end was supported by a $\varnothing 75$ rod segmented in three pieces to allow a ± 8 -mm height adjustment. Two short beam segments were tack welded to both sides of the panel zone to provide lateral support at such locations in a similar way the shake-table test specimen was laterally braced. The load was applied to the column top, at 1250 mm from the base pin. The 1250-mm distance corresponds to the column inflection points in the shake table test columns, assuming they happen at the mid-height.

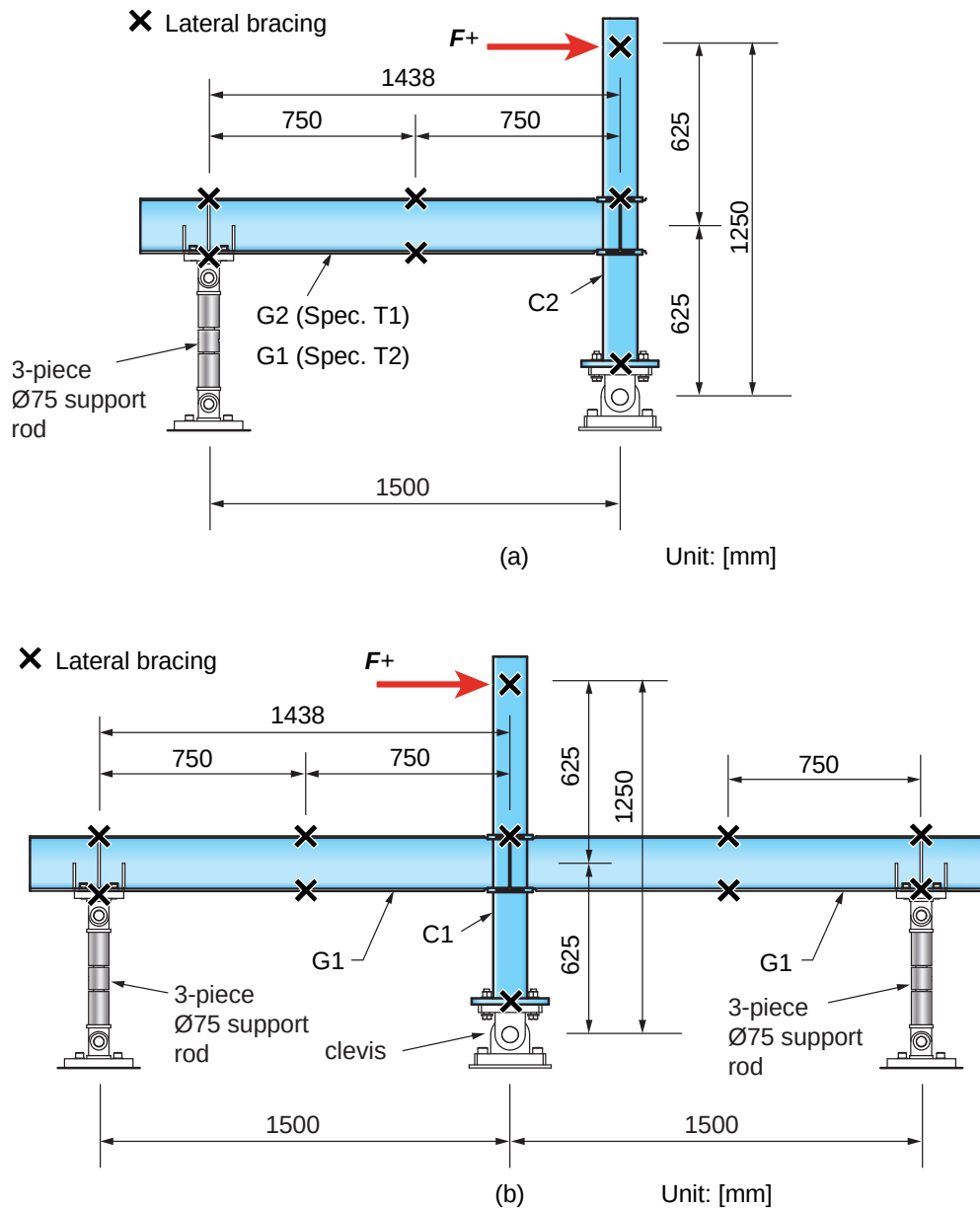


Figure 6.2 Subassemblages: a) T1 and T2, b) T3

6.2.3 Expected Strength and Stiffness

The estimation of the elastic stiffness of the subassemblages was computed from basic structural mechanics, dividing the applied horizontal force by the horizontal displacement at the top of the column. Figure 6.1 shows the structural modelling of the subassemblages. This calculations assumed axially rigid members. The panel zone was omitted since the members were modelled as straight lines meeting at a rigid joint. The boundary conditions were assumed as follows: beams with simply supported end, column base pinned and column top free. The expected stiffness values are very important to check during the initial loading stages of the tests if the measurement scheme is reliable the results are summarized in Table 6.1.

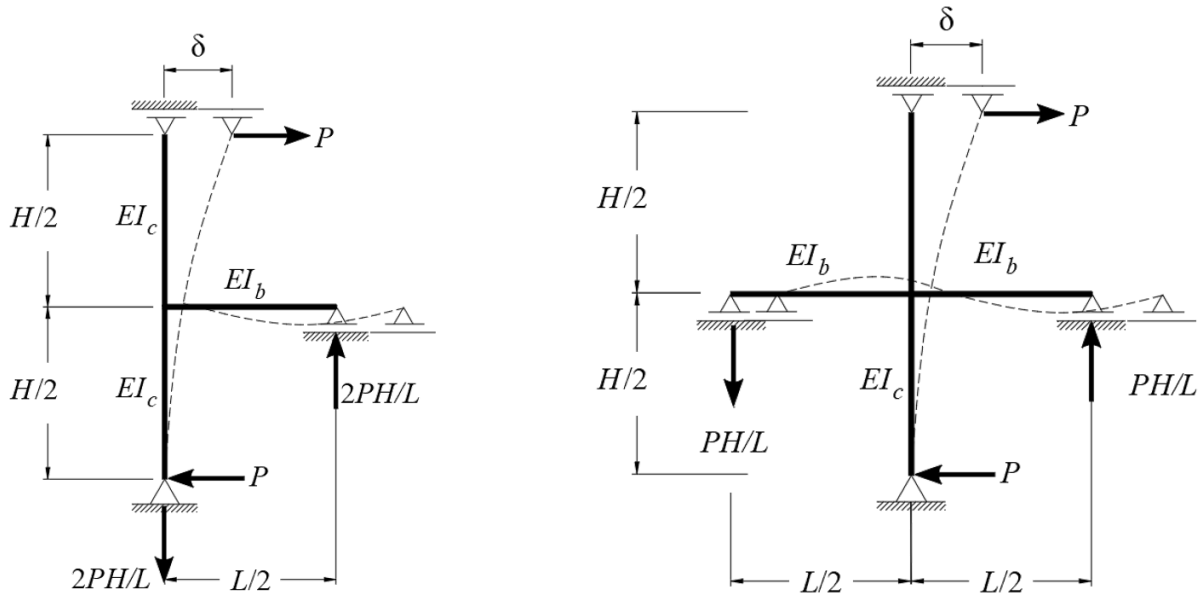


Figure 6.3 Calculation of the lateral stiffness of the subassemblages.

Table 6.1 Expected lateral stiffness and plastic strength of the subassemblages

Specimen	Lateral stiffness		Plastic strength
	F/Δ [kN/m]	F/SDR [kN/rad]	F_p [kN]
T1	2950	3688	49
T2	3300	4108	56
T3	5480	6828	111

6.2.4 Loading Plan

The subassemblages were subjected to cyclic loading to story drift ratio of ± 0.05 rad according to the loading protocol for beam-to-column connections defined by the AISC Seismic Provisions Sect. K2, as shown in Figure 6.4. Afterward, the loading cycles were skewed to one direction to a maximum of -0.125 rad. The ratchetting part of the loading protocol was determined from the observed peak story drift ratios observed during the four 100% excitations in the shake-table tests. In average, each 100% excitation induced a 0.10 -rad story drift ratio span between the negative and positive loading directions. The upper SDR was limited by the stroke of the hydraulic jack, and the negative SDR of -0.125 rad was chosen due to space limitation between the left beams and the reaction wall.

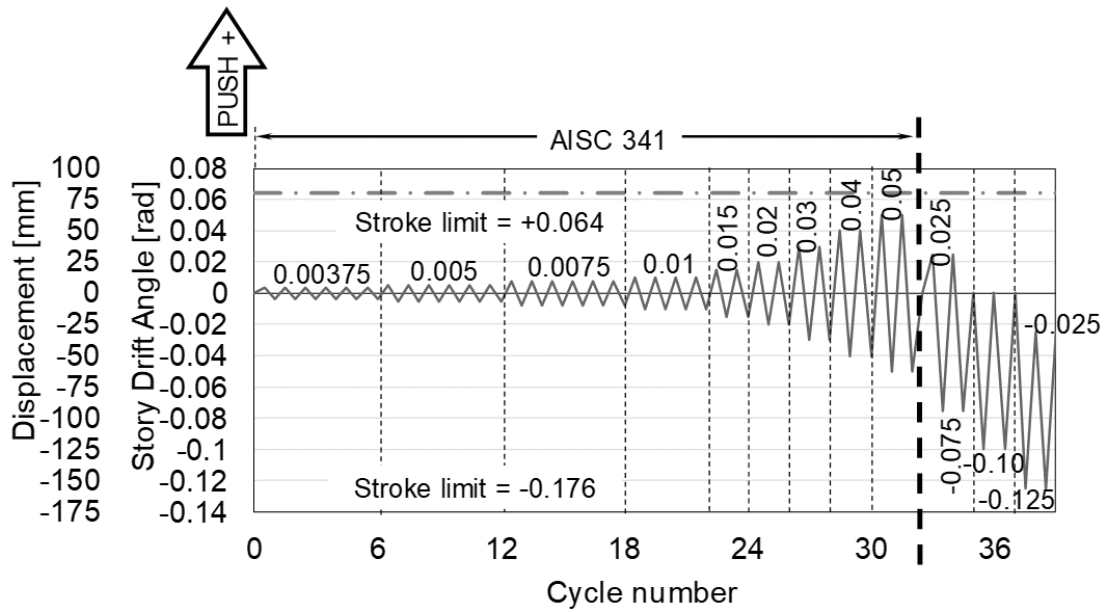


Figure 6.4 Loading protocol.

6.2.5 Measurement Plan

A measurement scheme similar to the one used in the shake-table tests was adopted. Figure 6.5 to Figure 6.7 show the instrumentation schemes of the three specimens, showing the displacement transducer and strain gage plan separately, as well as the numbering used. Each number corresponds to the channel number of the data logger where the instruments were connected to. Also, because the data logger had a capacity of 30 channels, only the displacement transducers were connected to the data logger, while the strain gages were connected to a 30-channel switch box. The figures also indicate the key distances that are necessary for data reduction. Those distances were measured on the site right before the tests.

The lateral displacement of the column top was measured with three instruments (1, 2, and 24) targeting the column connection plate. Transducers 1 and 2 were ± 200 -mm range transducers, consequently having poor resolution for the small displacements expected during the elastic cycles of 0.00375 to 0.0075 rad SDR (5 to 9 mm, respectively). Therefore, a ± 50 -mm range transducer was placed in the middle of the plate (24) to get good resolution data during those cycles. That instrument was removed after the ± 0.02 -rad cycles, when the specimens experienced displacements that could be measured with instruments 1 and 2 with good resolution.

The panel zone deformation and beam rotation used exactly the same scheme than the shake-table test specimen, but this time such responses were measured on both sides (front and back). A pair of transducers (8 and 20) were placed horizontally in the column base pin to measure the slippage of the clevis. Another pair of transducers (7 and 19) measured the vertical displacement of the column base.

Four transducers were installed on the beam free end to measure the rotation and vertical displacement of the beam support. At least two vertical instruments were needed (22 and 21 for T1 and T2, and also 23 and 27 for T3) because a single-point measurement alone could not trace the rotation of the rod support.

Two elastic strain gages, one on each beam flange, were attached at two beam sections to obtain the beam internal forces and moments. Both sections corresponded to the locations where strain gages were attached in the shake-table test specimen beams. Additionally, inelastic strain gages were attached to the beam flanges and web of a section 50 mm away from the column face. The objective of those strain gages was to identify the effect of fracture in the beam. No strain gages were attached to the columns because the column forces could be estimated from statics.

A pair of elastic strain gages were also attached to each beam support rods to estimate the beam reaction. The force obtained from those gages was calibrated and corroborated with the reaction estimated from two independent sources: by equilibrium of the system and by the beam shear. This calibration will be explained in the following section.

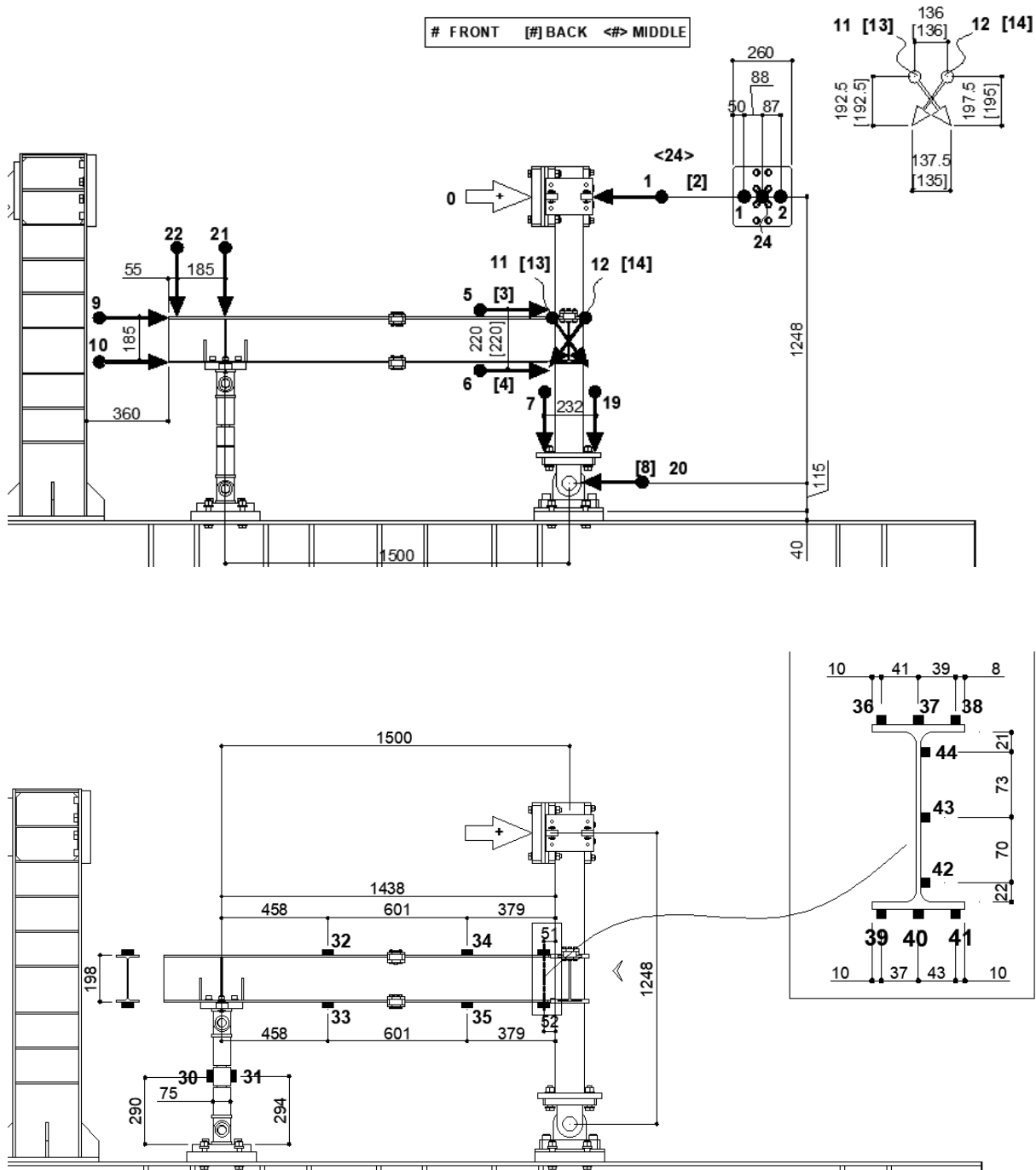


Figure 6.5 Instrumentation scheme of the Subassemblage T1: a) displacement transducers, b) strain gages.

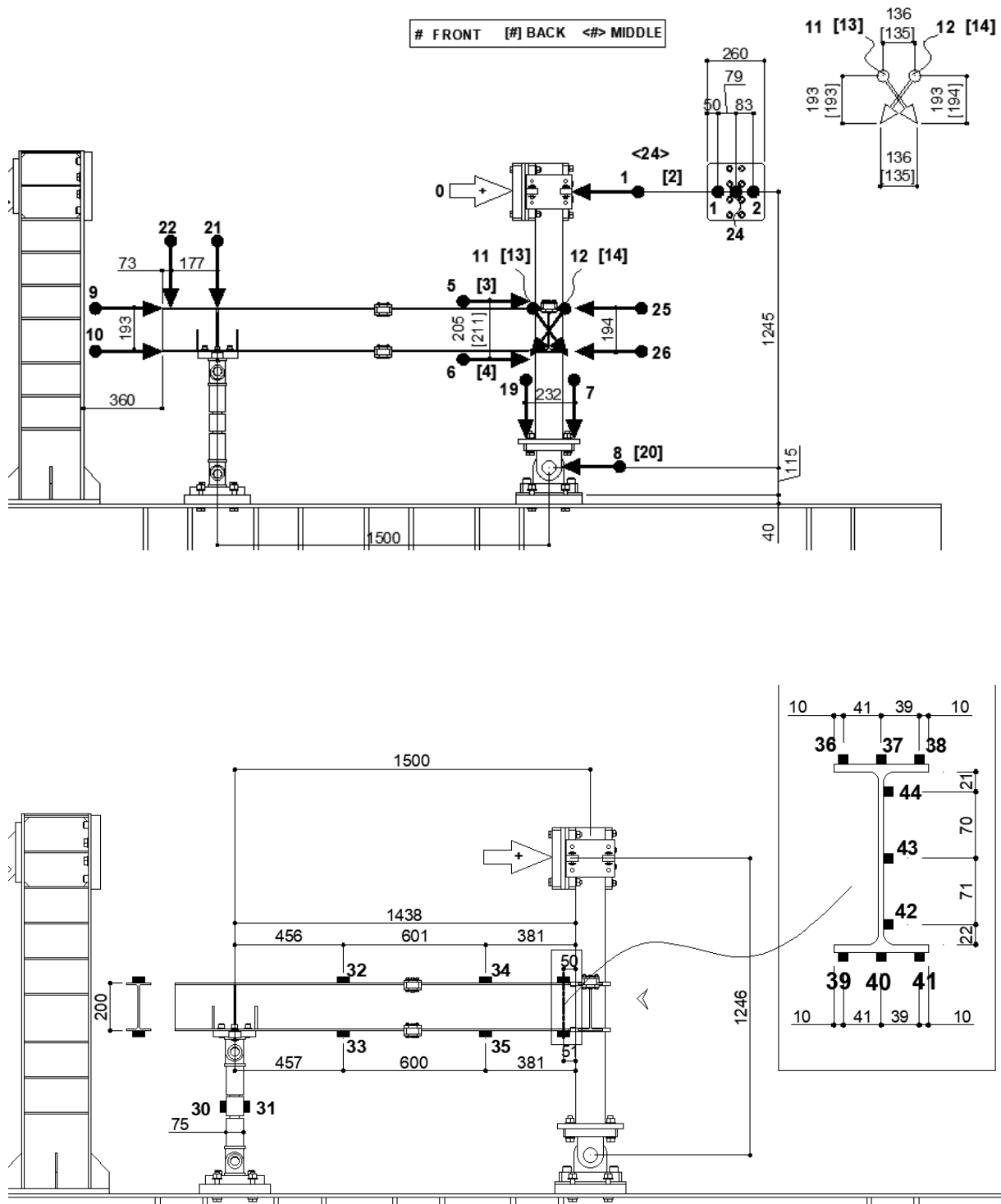


Figure 6.6 Instrumentation scheme of the Subassembly T2: a) displacement transducers, b) strain gages.

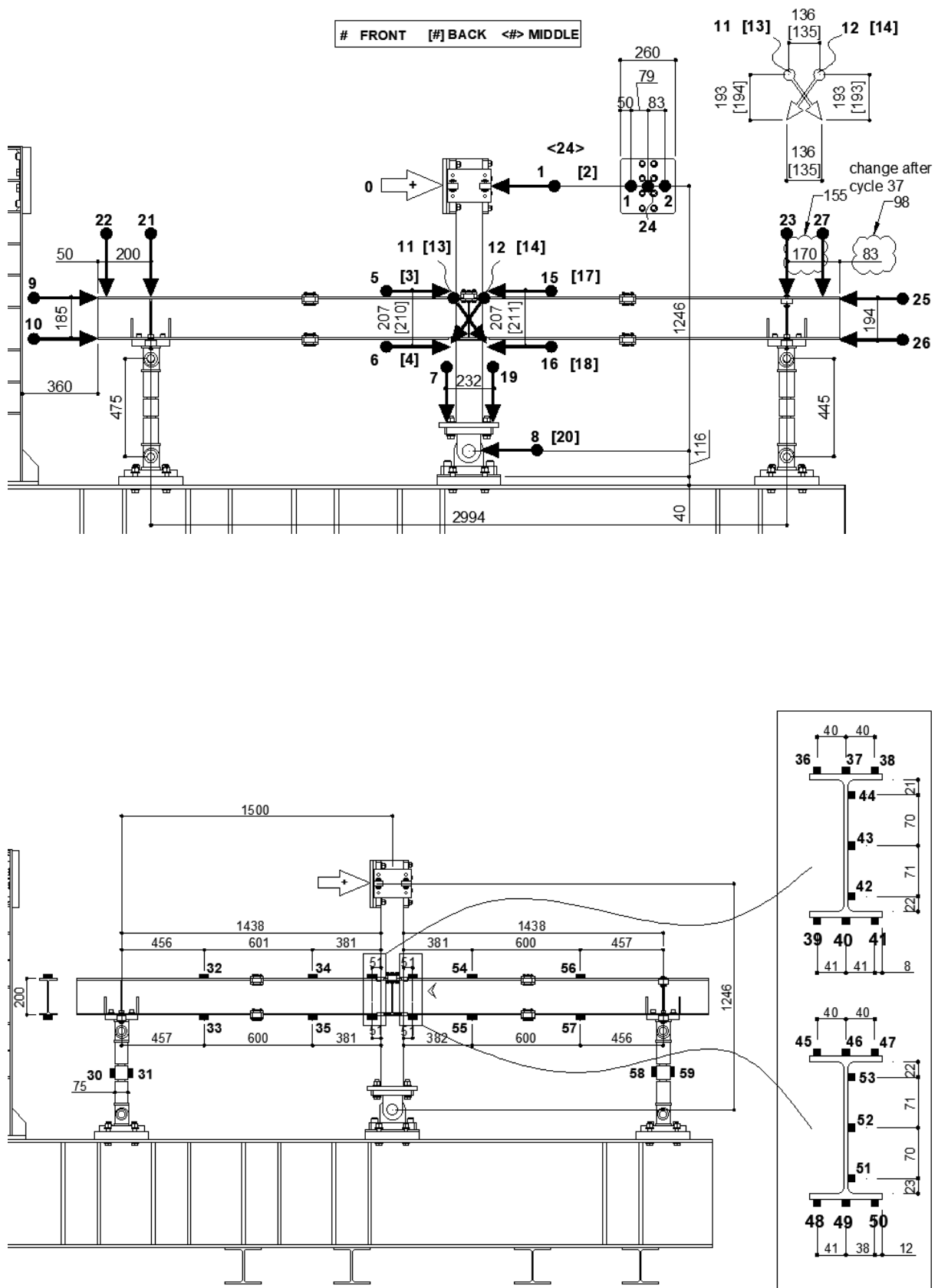


Figure 6.7 Instrumentation scheme of the Subassemblage T3: a) displacement transducers, b) strain gages.

6.3 DATA REDUCTION

6.3.1 Story Drift Ratio

The total displacement at the column top is:

$$d_T = -\left(\frac{d_1 + d_2}{2}\right) \quad (6.1)$$

The rotation due to flexibility of beam end support rod is calculated with the following formulas. The traced vertical displacement of the supports are:

$$r_L = -\left[d_{21} + \left(\frac{d_{21} - d_{22}}{S_L}\right)d_9\right], \quad r_R = -\left[d_{23} + \left(\frac{d_{23} - d_{27}}{S_R}\right)d_{25}\right] \quad (6.2)$$

The horizontal displacement at the column top due to r is:

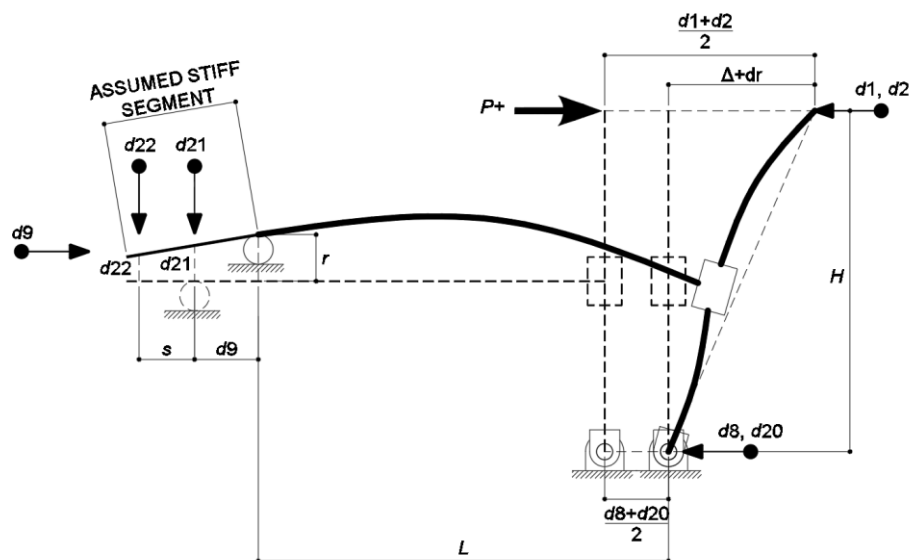
$$d_r = (r_R - r_L) \left(\frac{H}{L} \right) \quad (6.3)$$

The horizontal displacement for SDR calculation is:

$$\Delta = d_T - d_p - d_r \quad (6.4)$$

Finally, the SDR can be computed as:

$$\text{SDR} = \frac{\Delta}{H} \quad (6.5)$$



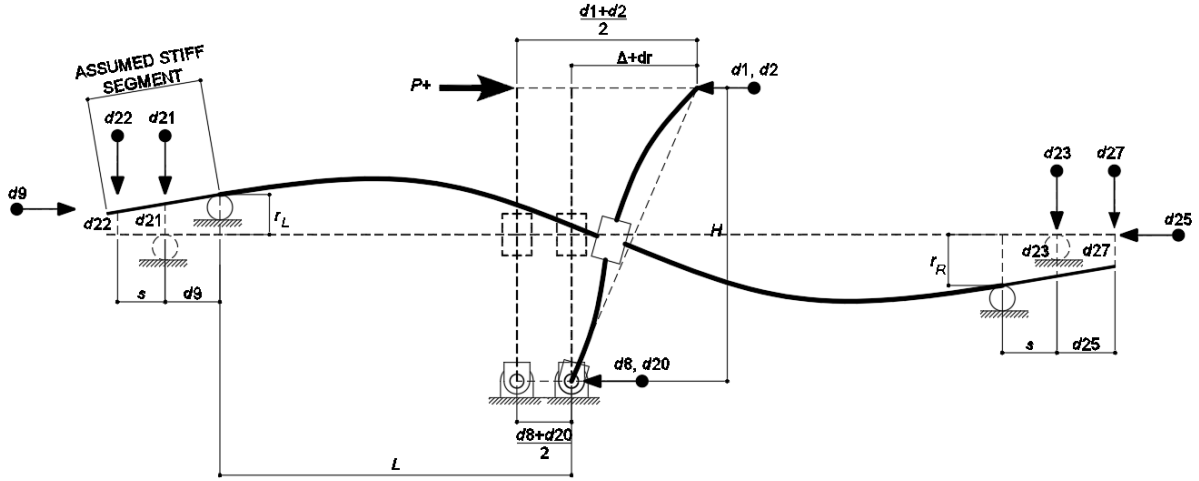


Figure 6.8 Deduction of Story Drift Ratio: a) Specimens T1 and T2, b) Specimen T3

6.3.2 Column Shear

The column shear was computed from equilibrium of the system, as given by equation (6.6). The column shear equalled the force applied by the jack.

$$V_c = F \quad (6.6)$$

6.3.3 Beam Rotation

The same equations used in Chapter 4 were used to compute the beam rotation at each face. The beam rotation was then the average between the measurements at both faces, as given by equation (6.7).

$$\theta_b = \frac{1}{2} \left(\left. \frac{d_B - d_T}{h} \right|_{\text{Back}} + \left. \frac{d_B - d_T}{h} \right|_{\text{Front}} \right) \quad (6.7)$$

6.3.4 Beam Forces and Moments

Figure 6.9 shows the scheme for determining the bending moment distribution of the beams. The moments at the sections that had strain gages (M_2, M_3, M_6, M_7) were computed with the same equations used for the shake-table test beams (Chapter 4). M_3 and M_7 were used as the reference moments since those sections remained elastic during all the tests. Sections 2 and 6 yielded from the ± 0.02 -rad SDR cycles. Therefore, the moments at those sections were used to compute the moments at the beam support M_4 and M_8 with equations (6.8) and (6.10), and verify that they are zero. After completing the verification, it was possible to extrapolate the moments to any beam section using only the reference moments M_3 and M_7 . The moments at the column face using this method are given by equations (6.9) and (6.11).

Moments in the left beam:

$$\text{Moment at the support} \quad M_4 = M_3 + \left(\frac{M_3 - M_2}{s_3} \right) s_4 \quad (6.8)$$

$$\text{Moment at the column face} \quad M_{bL} = M_3 + \left(\frac{M_3}{s_4} \right) (s_2 + s_3) \quad (6.9)$$

Moments in the right beam:

Moment at the support

$$M_8 = M_7 + \left(\frac{M_7 - M_6}{s_7} \right) s_8 \quad (6.10)$$

Moment at the column face

$$M_{bR} = M_7 + \left(\frac{M_7}{s_8} \right) (s_6 + s_7) \quad (6.11)$$

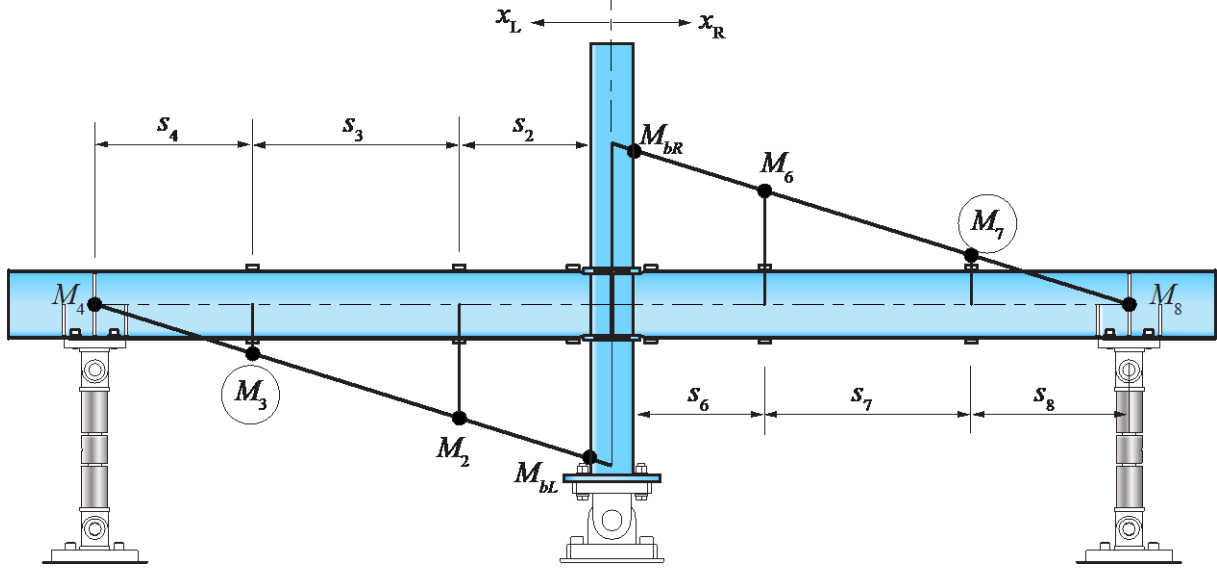


Figure 6.9 Deduction of beam internal moment distribution.

6.3.5 Beam Reaction

The forces in the beam supports were calibrated and verified using three independent measurements, as shown in Figure 6.10. The reaction obtained by beam shear (Figure 6.10b) was plotted against the rod force obtained directly from strain gages as given by Figure 6.10a, and equation (6.12) to obtain the relations shown in Figure 6.11. The parameters m and b were obtained from linear regression. Finally, the calibrated rod force is given by equation (6.13).

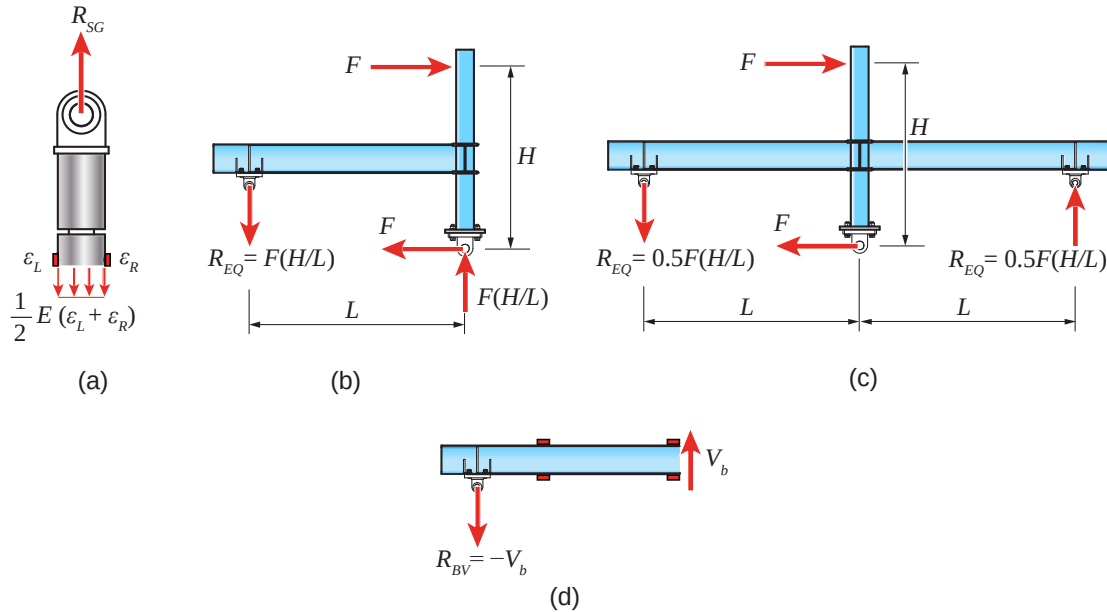


Figure 6.10 Beam reaction by different sources: a) support rod axial, b) reactions of Specimens T1 and T2, c) Reactions of Specimen T3, d) reaction by beam shear.

$$R_{SG} = EA \left(\frac{\varepsilon_L + \varepsilon_R}{2} \right) \quad (6.12)$$

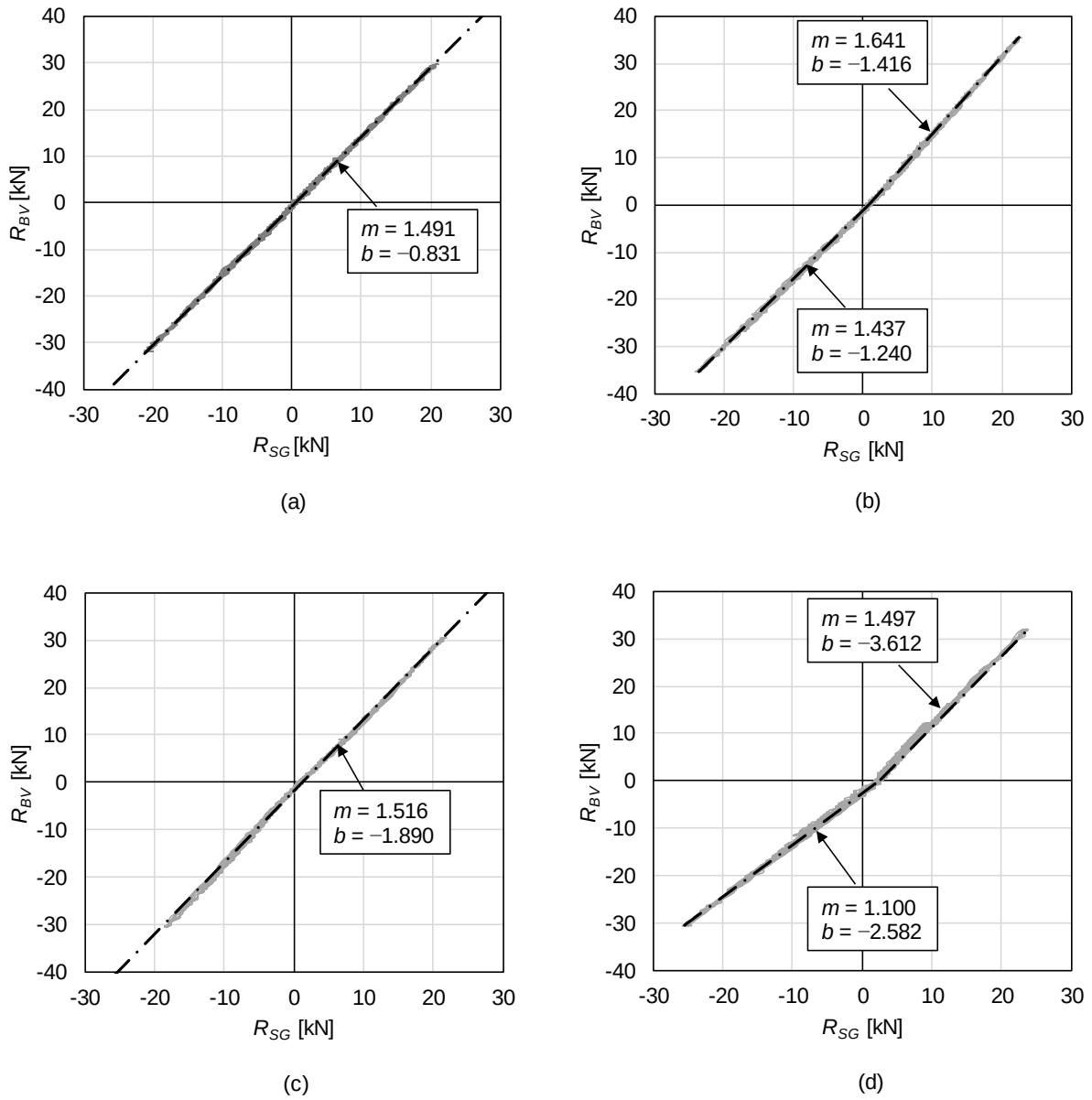


Figure 6.11 Calibration of support rod force.

$$R = mR_{SG} + b \quad (6.13)$$

6.3.6 Panel Shear Deformation

The same equations used in Chapter 4 were used to compute the panel shear deformation at each face. The vertical and horizontal distances between targets of the panel zone instruments are shown in Figure 6.5 to Figure 6.7. The distances were averaged for use in the equations. After computing the panel deformation at both faces, the average is given by equation (6.14).

$$\text{Panel shear deformation} \quad \gamma = \frac{1}{2} (\gamma_{\text{Back}} + \gamma_{\text{Front}}) \quad (6.14)$$

6.4 TEST RESULTS

6.4.1 Specimen T1

This specimen represented an external T-shape connection of the moment-resisting frame in the two upper stories. Figure 6.12 shows the three measured responses of the specimen: the applied load vs. story drift ratio, the beam end moment vs. beam rotation, and the panel zone shear vs. angular deformation. The expected elastic stiffness is shown in dashed inclined lines, and the respective plastic strength indicated by horizontal dashed lines.

The elastic stiffness of the system, measured as the slope in the positive half cycles of the 0.0375-rad SDR, was 3654 kN/rad, 99% of the expected elastic stiffness. This result is reasonable since the actual structure tends to be softer because the expected values were computed using ideal conditions. The beam yielded during the first +0.015-rad SDR incursion, corresponding to a beam rotation of similar magnitude. The maximum strength of $1.10M_p$ was reached during the cycles of 0.04 rad. At the same time, the subassembly also reached its peak lateral strength of 1.27 times the plastic strength F_p . The instant is marked by an empty triangle in the plots. During the first ± 0.05 -rad cycles, local buckling was evident in both beam flanges and the web, followed by strength degradation to $0.90M_p$. The strength and stiffness of the beam degraded with each subsequent cycle as the local buckling of the flanges progressed. The system's strength and stiffness also degraded with the beam because most of the contribution to the lateral strength of the subassembly was dictated by the beam. It was noted during the tests that a 3-mm gap between the rollers and the contacting plates in the lateral support system was likely the cause of twisting of the beam section. Originally it was believed that such twisting may have accelerated the strength and stiffness degradation. However, as will be discussed later, the other tests also showed a similar pattern of degradation even though the gap with the lateral bracing system was carefully reduced to 1-mm.

Figure 6.13 shows the cracks that formed in the specimen in the beam bottom flange during the first negative incursion of 0.05-rad SDR (Figure 6.13b, crack 1) and in the top flange during the second positive incursion of 0.05-rad SDR (Figure 6.13c, crack 2). As seen in the figures, all the cracks started from the weld termination between the CJP groove and the beam flange, in a similar way to the beam-to-column moment connections of Chapter 3. Filled triangles in Figure 6.12 mark the start of the cracking.

The cracks progressed very slowly along the groove weld bevel. As Figure 6.13d shows, crack 1 grew up to 35 mm, or 1/3 the beam top flange width. Local buckling, on the other hand, was significantly large and was present also in the beam web. In the final cycle, beam strength dropped to $2/3 M_p$, but significant inelastic deformations had been developed by this time. No further loading was attempted beyond the 0.125-rad SDR due to space limitations in the test setup.

During the last half cycle, the beam rotation instruments saturated, and unfortunately no data was recorded. Nevertheless, by this point significant crack growth and beam degradation by local buckling had been recorded.

The panel zone response is shown in Figure 6.13c. The panel zone behavior was completely elastic throughout the test since the maximum demand was about half the expected plastic strength. The inelasticity was concentrated in the beam plastic hinge and there was no moment redistribution because the T-configuration of the subassembly was a statically determinate structure.

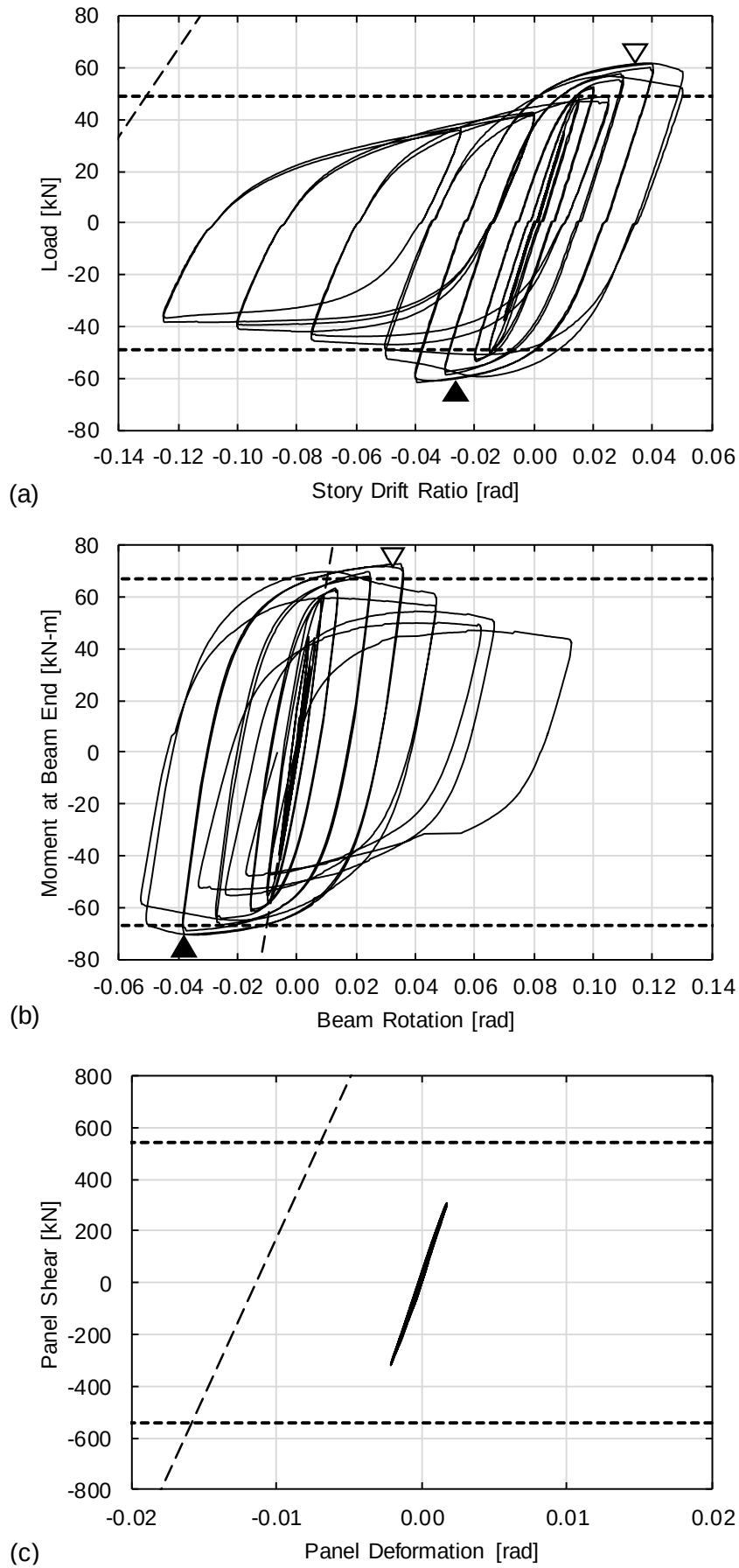
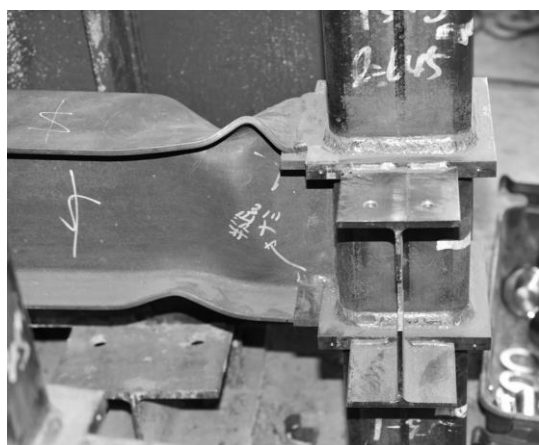
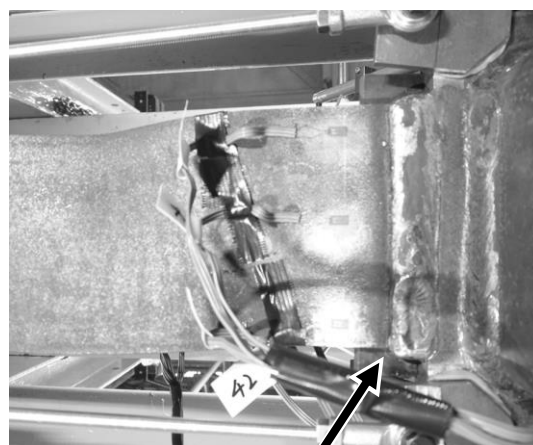


Figure 6.12 Force-deformation relationships of T1: a) Global, b) Beam, c) Panel zone.



(a)



(b)



(c)



(d)

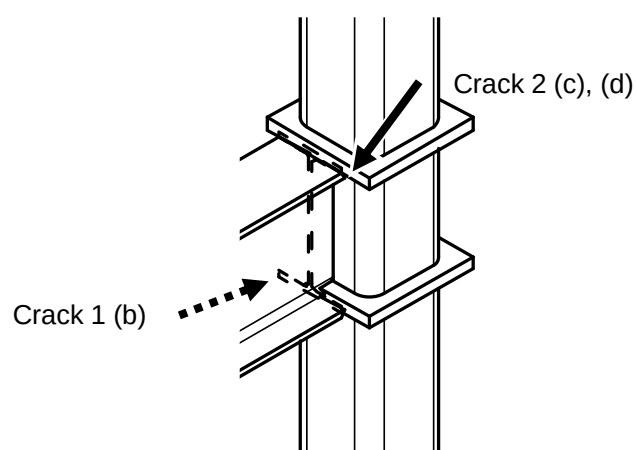


Figure 6.13 Observed phenomena in Specimen T1: a) local buckling, b) crack 1, c) crack 2, d) Propagation of the crack 2.

6.4.2 Specimen T2

Similarly to Specimen T1, this subassembly also represented an external T-shape connection of the moment-resisting frame. This case, however, had a beam representative of the first or second story. Figure 6.14 shows the three measured responses of the specimen: the applied load vs. story drift ratio, the beam end moment vs. beam rotation, and the panel zone shear vs. angular deformation.

The beam of this Specimen also yielded during the first +0.015-rad SDR incursion, corresponding to a beam rotation of similar magnitude. The maximum strength of $1.27M_p$ was reached during the cycles of 0.04 rad. The instant is marked by an empty triangle in the plots. During the second +0.05-rad cycle, local buckling was evident in the beam bottom flange and the strength degraded to $1.10M_p$. Local buckling happened in the top flange during the first -0.015-rad SDR incursion. Unlike Specimen T1, the negative moment strength of the beam, however, survived the negative 0.05-rad SDR cycles. The slightly sooner degradation of Specimen T1 may be attributed in part to the 3-mm gap with the lateral restraining system, that was reduced to 1-mm for Specimen T2, and also because the beam of Specimen 2 had a beam with a slightly larger cross section than the beam of Specimen T1. Therefore, the smaller width-to-thickness ratio of the flanges and web of the beam in Specimen T2 slightly delayed the occurrence of local buckling.

Figure 6.15 shows the final state of the local buckling and cracks that formed in the specimen. The first crack appeared in the bottom flange (Figure 6.15b) during the first -0.015-rad SDR incursion. During the second positive cycle of 0.05 rad, the second crack formed in the top flange (Figure 6.15c). All the crack formation and propagation was similar to the observed in Specimen T1. Filled triangles in Figure 6.14 mark the start of the cracking.

The crack in the beam top flange eventually exceeded the beam flange half width and entered into the beam web, as seen in Figure 6.15d. This web crack initially propagated along the beam's longitudinal direction, but then changed direction 60° toward the beam mid-height. During the last half cycle the beam top flange completely fractured at a 0.06-rad SDR and beam rotation,, as evidenced with the sudden strength drop to 1/3 the plastic strength in both the global and beam response plots of Figure 6.14a and b, respectively. The instant of fracture is indicated by a "x" mark in the plots. The strength in the fractured direction was not completely lost because the fracture did not travel the entire beam depth. In the last incursion of -0.125 rad SDR, the opposite loading direction of the fracture, the hysteresis loop shows a pinching effect because the beam bottom flange, which cracked but did not fracture, was able to maintain a $0.60M_p$ -moment strength. The fractured flange in Figure 6.15c shows the exposed separate backing bars used to fabricate the CJP weld with no weld-access-hole. As has been seen in other connections with the same detailing, the crack propagates along the groove bevel and in the beam flange once it reaches the beam web.

The panel zone response is shown in Figure 6.14c. The panel zone behavior was completely elastic throughout the test since the maximum demand was about 70% the expected plastic strength. Noting that the demand on the panel zone of Specimen T1 was 50% the expected plastic strength indicates that the panel of Specimen T2 participated slightly more.

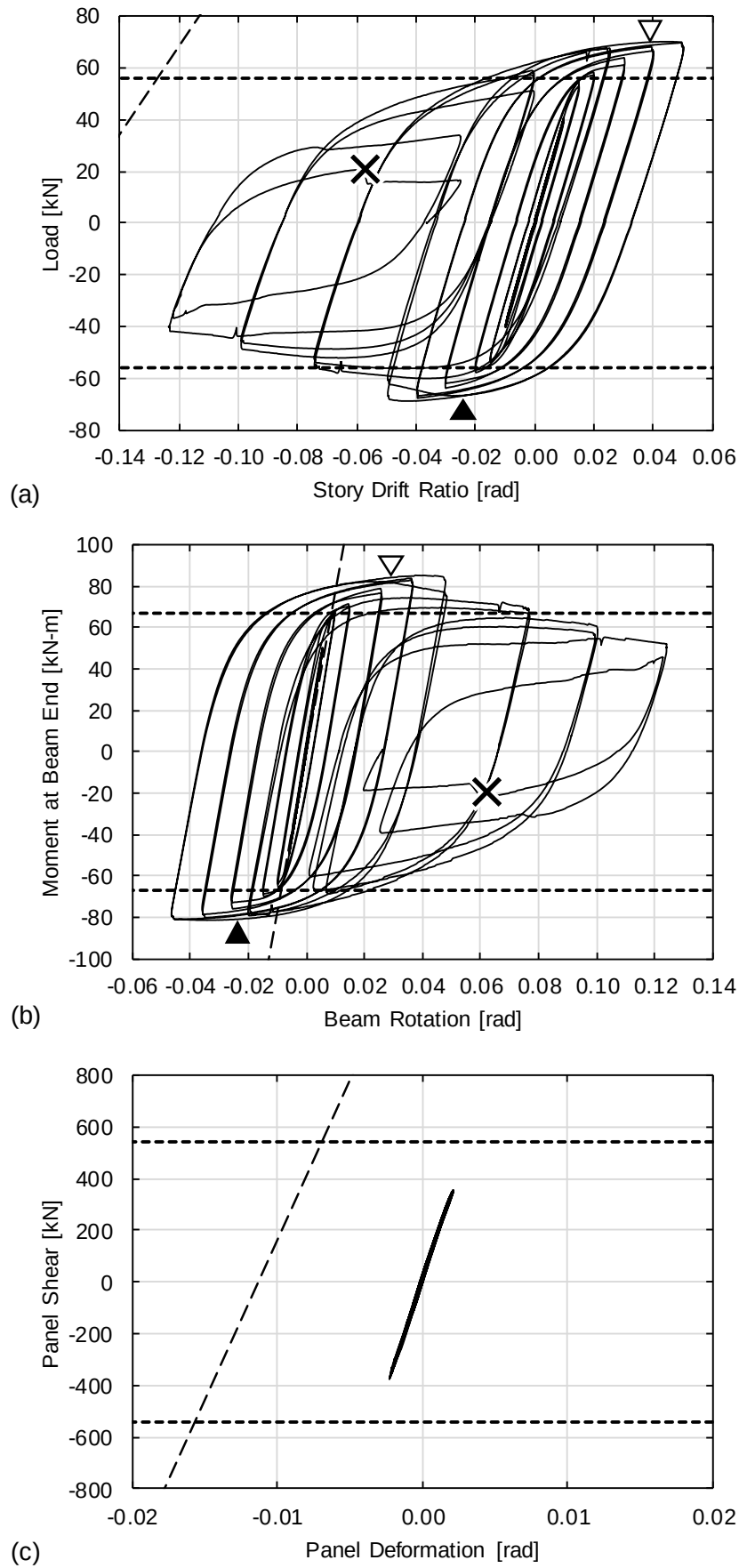
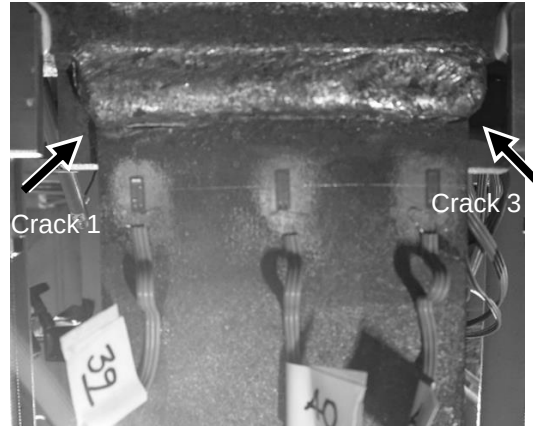


Figure 6.14 Force-deformation relationships of T2: a) Global, b) Beam, c) Panel zone.



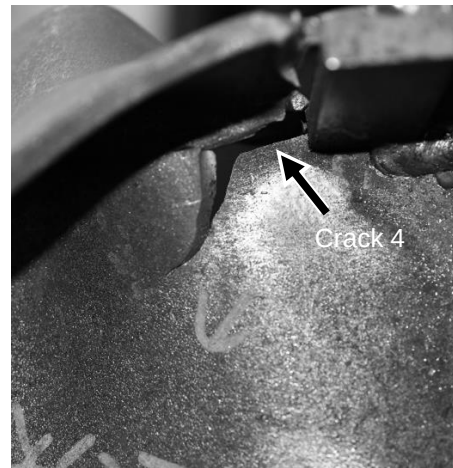
(a)



(b)



(c)



(d)

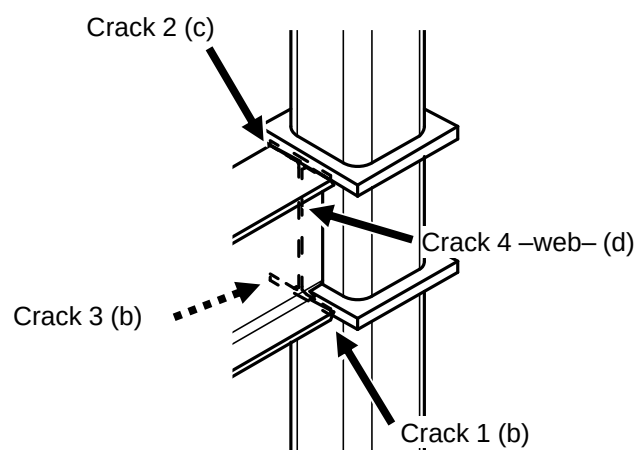


Figure 6.15 Observed phenomena in Specimen T2: a) local buckling, b) bottom flange cracks, c) fracture of the top flange, d) propagation of the fracture into the beam web.

6.4.3 Specimen T3

This specimen represented a first or second-story interior cruciform connection of the shake-table test moment-resisting frame. Figure 6.16 shows the three measured responses of the specimen: the applied load vs. story drift ratio, the end moment vs. rotation of both beams, and the panel zone shear vs. angular deformation.

The beams yielded during the first +0.015-rad SDR incursion, corresponding to a rotation of similar magnitude in both beams. The maximum strength of 1.28 and $1.31M_p$ were reached during the positive and negative half cycles of 0.04 rad SDR in the right and left beams, respectively. The instant when the left beam reached its peak strength is marked by an empty triangle in the plots. During these cycles, local buckling started in the top flange of both beams. Also, cracks formed in the top flange of the beams. Figure 6.17 shows the cracks that formed in the specimen. All the cracks started at the weld termination between the CJP groove and the beam flange, in a similar way to the beam-to-column moment connections of Chapter 3 and Specimens T1 and T2. Filled triangles mark the start of the cracking.

Local buckling occurred in both flanges of the two beams in the cycles beyond the 0.05 -rad SDR cycles. Unlike Specimens T1 and T2 which beam strength degraded below M_p at beam rotations of about 0.06 rad, the beams of Specimen T3 maintained M_p up to 0.10 -rad beam rotation. The strength and stiffness of the beam and the system degraded with each subsequent cycle as the local buckling of the flanges progressed but to a lesser extent than the previous specimens. The rate of degradation was slower because inelastic deformations were concentrated in both beams and not in a single one, like in the previous two Specimens, and because the panel zone also developed inelastic deformations.

The crack in the right beam bottom flange eventually exceeded the beam flange width and entered into the beam web, as seen in Figure 6.17d. During the last half cycle the bottom flange of the right beam completely fractured, as indicated by the sudden strength drop in the response plots of Figure 6.16a and b. The strength was not completely lost because the fracture did not travel the entire beam depth. The beam maintained a moment strength of $0.30M_p$ to the end of the test. The effect on the global behaviour was smaller, as seen in Figure 6.16a. The strength drop of the subassembly was small, only $0.06F_p$ because the moment redistributed to the left beam.

The panel zone response is shown in Figure 6.16c. In this case, the panel zone yielded and reached a maximum strength of $1.1V_{ppz}$. However, there was no visual evidence of yielding or flaking of the mill scale.

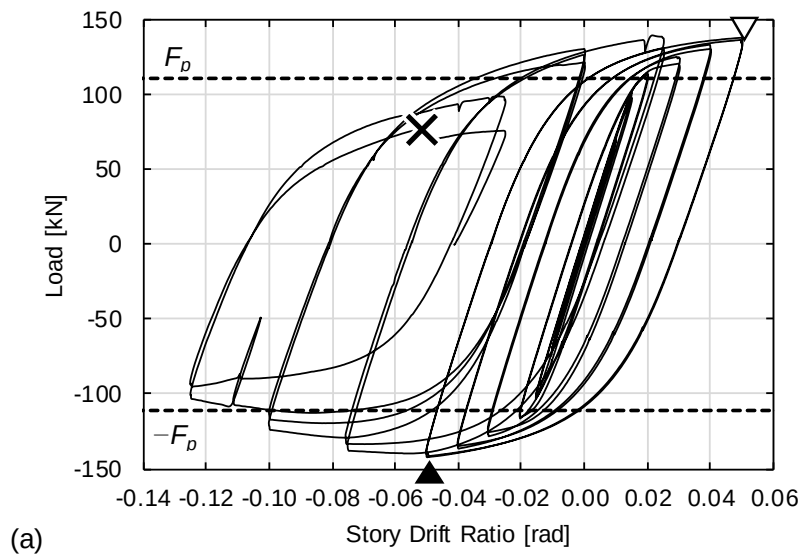


Figure 6.16 Force-deformation relationships of T2: a) Global,

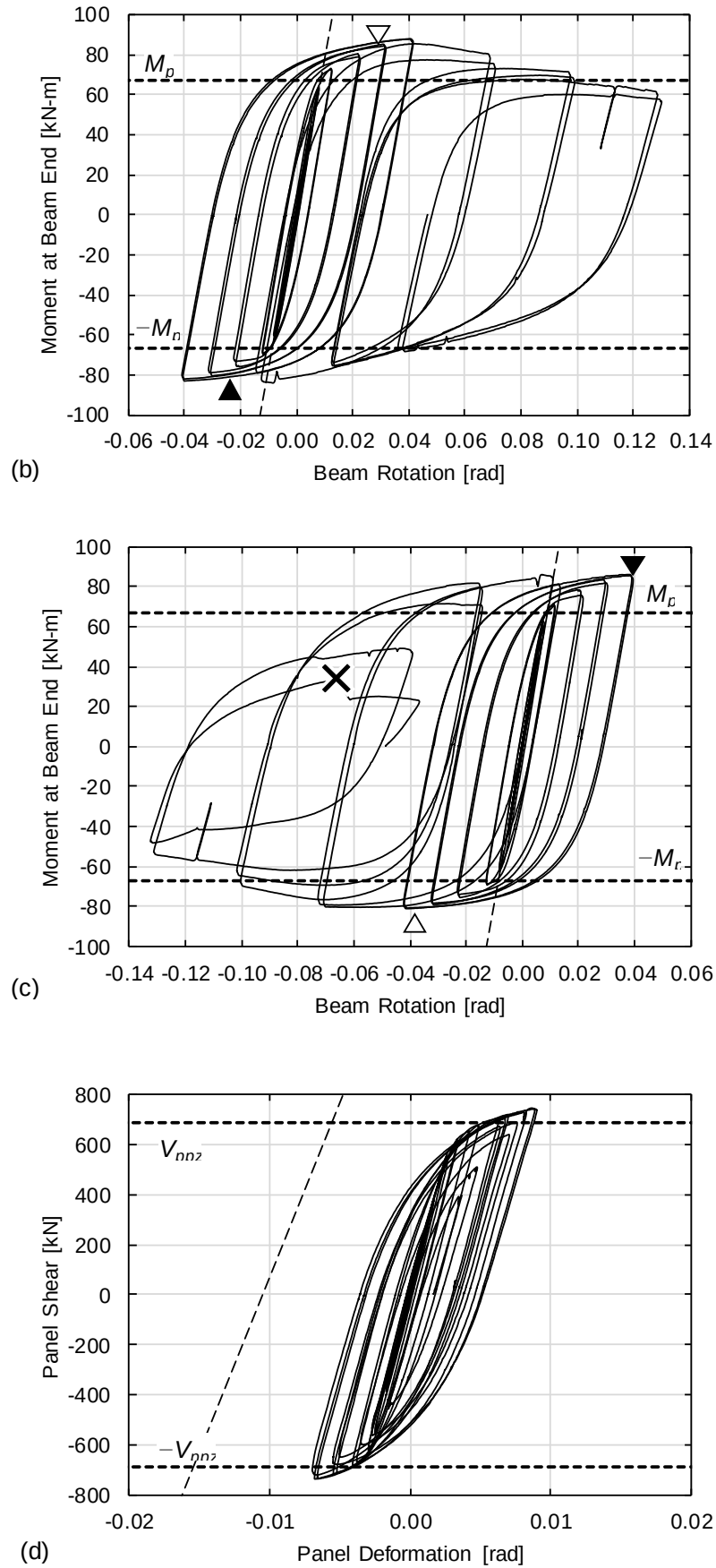


Figure 6.16 cont. Force-deformation relationships of T2: b) Left Beam, c) Right Beam d) Panel zone.

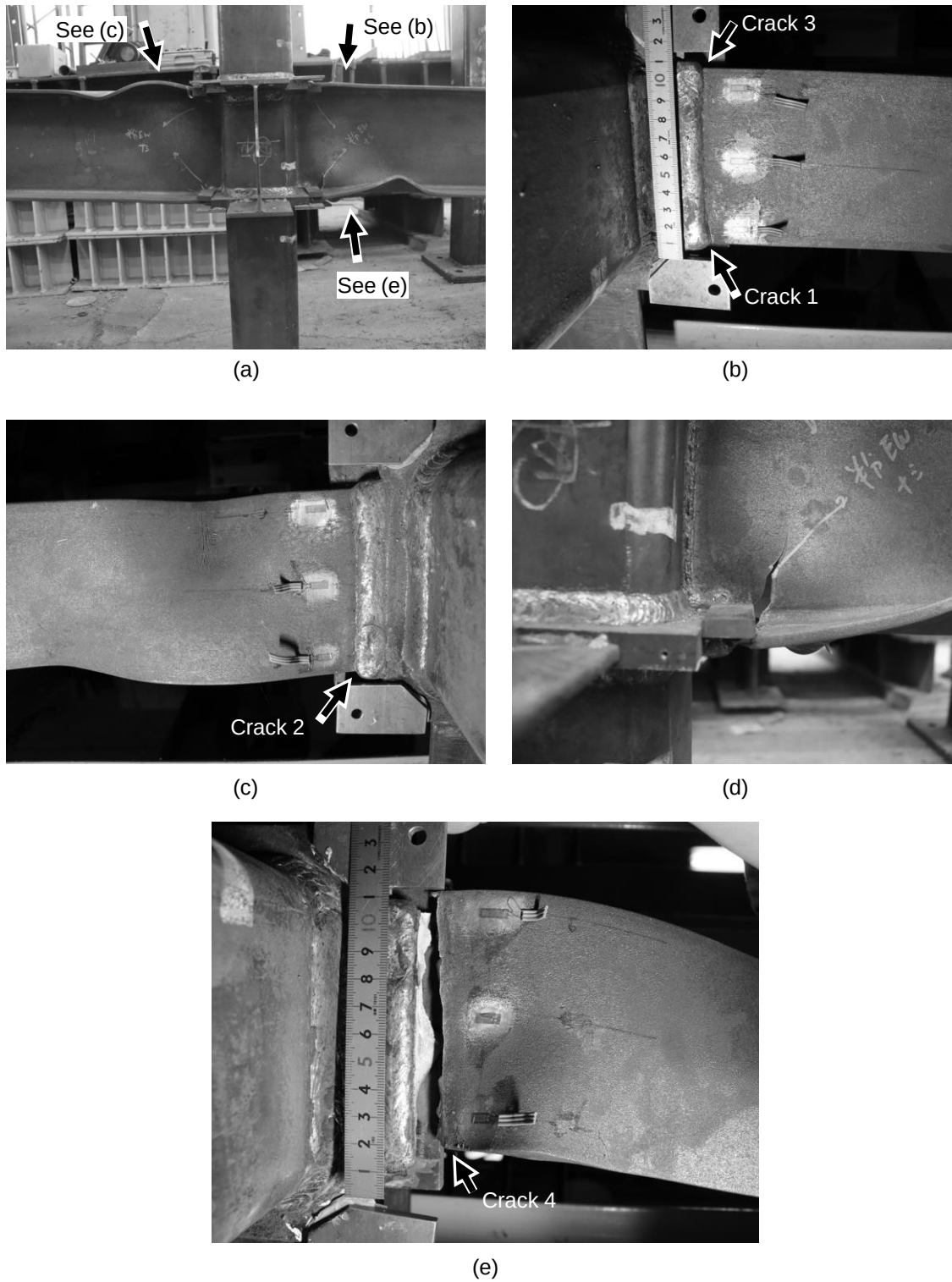


Figure 6.17 Observed phenomena in Specimen T3: a) Local buckling, b) cracks on the top flange of the right beam, c) cracks on the top flange of the left beam, d) crack from the bottom flange into the web of the right beam, e) fracture of the bottom flange.

6.5 CHAPTER SUMMARY

Three beam-and-column assemblages were constructed simultaneously with the shake-table specimen using the same heat of steel, design, and detailing. The subassemblages represented two external and one internal joint extracted at the middle of the beams and columns of the shake-table test moment-resisting frame, assuming that the inflection points in the shake-table test moment-resisting frame happened at those locations. The objective of the component tests was to reproduce the behavior of the entire moment-resisting frame system by beam-column subassemblages.

The test setup included a lateral bracing system that was a combination of rollers and bearing plates that allowed in-plane motion but restrained out-of-plane displacements. The bracing locations corresponded to the same positions in the shake-table test specimen.

The subassemblages were subjected to cyclic loading to story drift ratio of ± 0.05 rad according to the loading protocol for beam-to-column connections defined by the AISC Seismic Provisions Sect. K2. Afterward, the loading cycles were skewed to one direction to reproduce the ratchetting observed during the four 100% excitations in the shake-table tests.

The key test observations are summarized as follows:

- The subassemblages showed strength and stiffness degradation after SDR of 0.04 rad caused by local buckling in the beam flanges and web.
- All the specimens developed cracks after reaching the peak strength, that was around 1.30 times the expected plastic strength F_p . All the cracks started from the weld termination between the CJP groove and the beam flange, in a similar way to the beam-to-column moment connections of Chapter 3.
- Crack growth was slow in all cases, and propagated along the bevel of the groove weld.
- Fracture occurred in either the top or the bottom beam flange.
- The inelastic deformations were concentrated in the beam plastic hinge of the T-subassemblages. Therefore, the global response of the subassemblage was greatly affected by the beam response. In the cruciform subassemblage, on the other hand, the plastic hinge on the two beams and the panel zone contributed to the inelastic behavior.

7. COMPARISON BETWEEN SHAKE-TABLE AND COMPONENT TEST RESULTS

7.1 RESPONSE COMPARISON

Figure 7.1 shows the beam-end moment versus beam rotation (over gauge length 490 mm measured from the beam end) obtained from the three specimens. Specimen T3 is represented by the right beam noting that the left beam showed nearly identical behavior but fracture occurred only in the right beam. The three beams developed a maximum strength of 1.1, 1.2 and $1.1M_p$, respectively, during the first +0.05-rad excursion. All specimens behaved similarly. Local buckling of the beam became prevalent during the second +0.05-rad excursion (marked by $\triangle$), followed by crack initiation in the top beam flange at the termination of CJP groove weld during the subsequent half cycle ($\blacktriangle$). The “ $\times$ ” in Figure 7.1b and c indicates fracture of the beam top flange of Specimen T2 and the right beam bottom flange of Specimen T3.

Figure 7.2 shows the same relationships obtained from corresponding beams in the shake-table test during the four 100% JMA Kobe N-S excitations. Similarly to Figure 7.1, the “ $\triangle$ ”, “ $\blacktriangle$ ”, and “ $\times$ ” marks indicate local buckling, crack initiation, and fracture of the beam flange, respectively. The shake-table specimen was subjected to similarly large deformation at 1st story but substantially smaller deformation at 3rd story. The shake-table specimen developed similar flexural strength at the connections and exhibited similar damage progressed as the subassemblages. However, unlike the subassemblages, the shake-table specimen exhibited no significant degradation in strength or stiffness before fracture. This difference may be attributed to the axial restraint of the beam by surrounding members that was present in the shake-table test but not in the subassemblages.

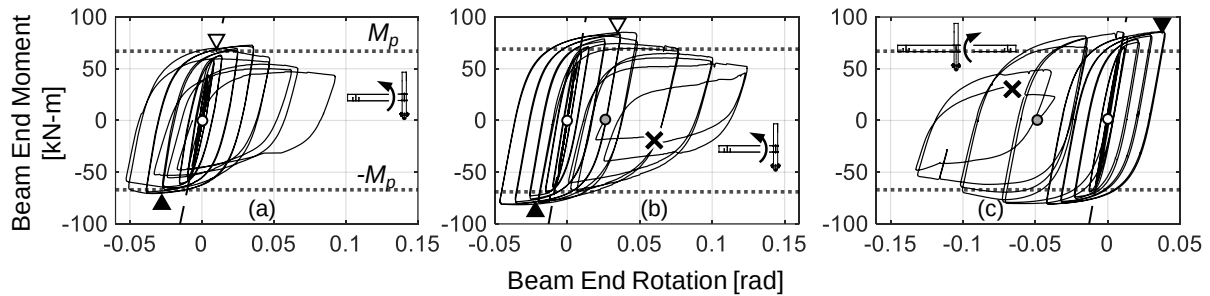


Figure 7.1 Beam-end moment vs. Beam Rotation of the subassemblages a) T1, b) T2, c) T3.

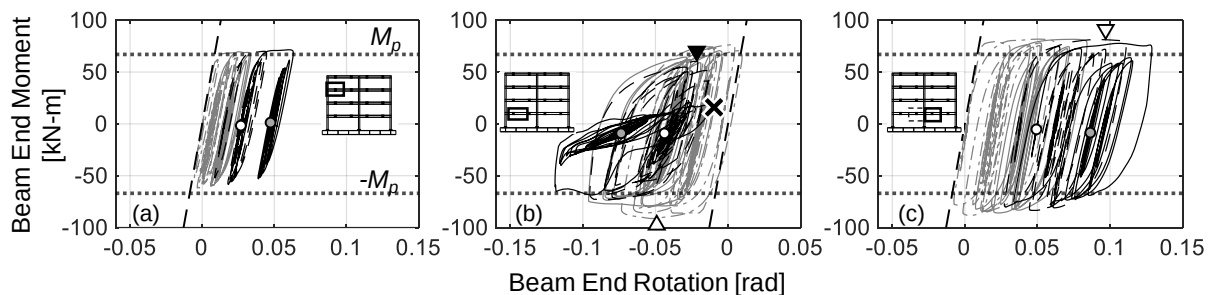


Figure 7.2 Beam-end moment vs. Beam Rotation of a) 3rd story external joint, b) 1st story external joint, and c) 1st story internal joint.

7.2 BEAM CONTRACTION AND AXIAL LOAD

Figure 7.3 shows the history of beam elongation measured over a 490-mm gauge length from the beam end. The subassemblages, which had no mechanism to develop beam axial force, contracted in the beam:

for Specimen T2, 9 mm or 1.8% of gauge length. In the shake-table tests, the beam contracted 5 mm, or half the subassembly, after experiencing similar rotation history. The contraction grew larger after complete fracture of the bottom flange indicated by “x” in Figure 7.3c, but never exceeded 7.5 mm.

Figure 7.3b and c also show the history of axial load and bending moment normalized by yield strength P_y and M_p respectively. Figure 7.3a shows that the beam of Specimen T2 contracted under positive bending moment, which is likely an outcome of local buckling occurring in the upper flange.

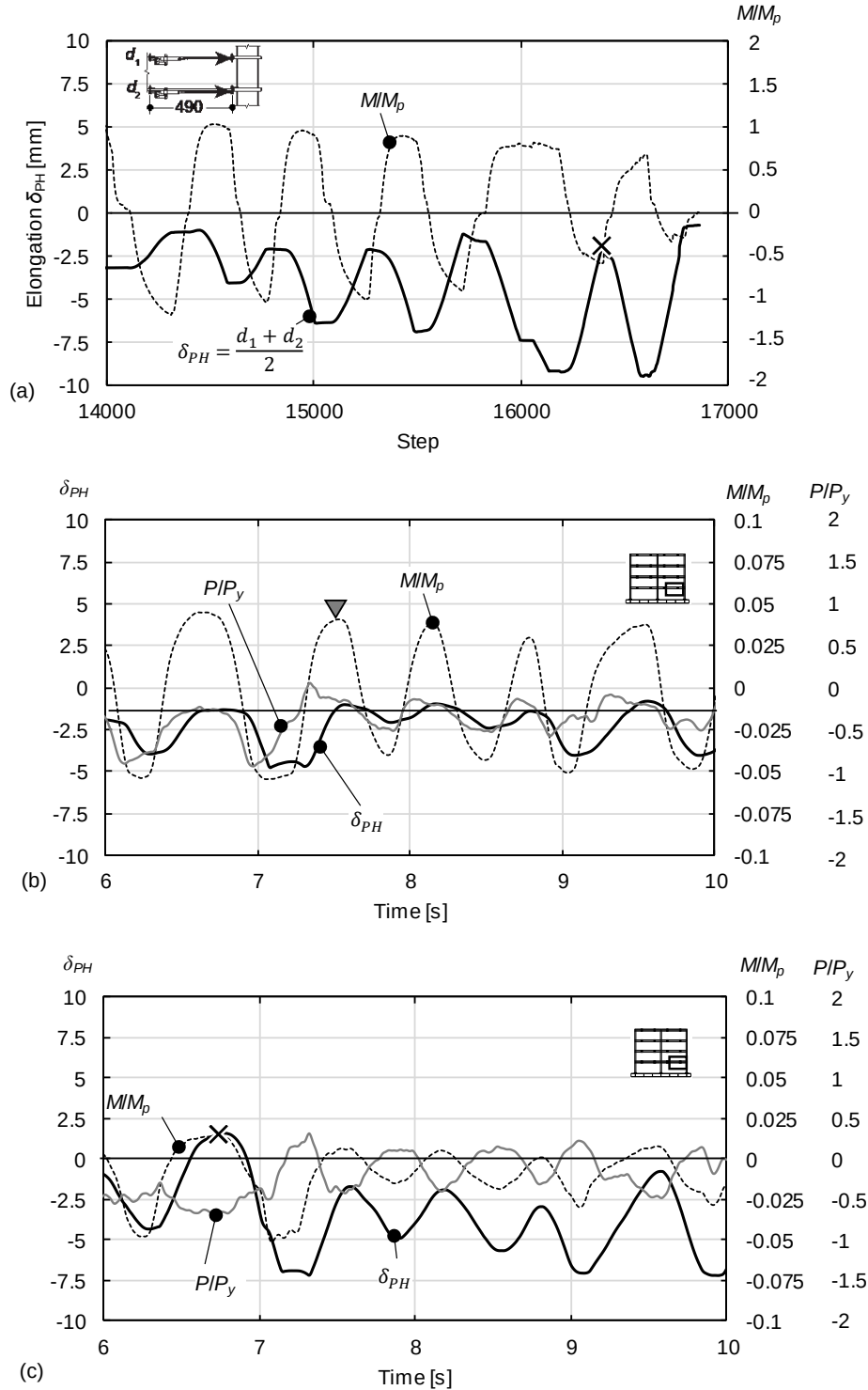


Figure 7.3 Axial deformation of the plastic hinge in a) T1, b) 1st story external joint during 3rd 100%, and c) 1st story external joint during 4th 100%.

In the shake-table tests, axial constraints imposed by the surrounding columns produced axial loads in the beams. As shown in Figure 7.3b, before fracture in the bottom flange, during 100%-3, the 1st story beams were subjected to compression. Axial force synchronized with axial contraction and compression synchronized with negative moment, which was likely the cause of local buckling occurring more substantially in the bottom flange. As shown in Figure 7.3c, after fracture, the axial force was released. Positive strength of the beam reduced to $0.20M_p$ due to fracture of the bottom flange, but negative strength was maintained at $0.50M_p$. The larger variations of beam elongation was likely due to the opening and closing of the crack.

Figure 7.4 shows the axial load distribution of the shake-table test specimen during three stages of the 100%-4 excitation. It can be noted that big variations of axial happened in the outer segments of the beams due to the load reversals imposed by the ground motion. Important tension loads were developed in many connections to the columns. The beams that were subjected to tension did not show significant degradation, but the beams subjected to compression did.

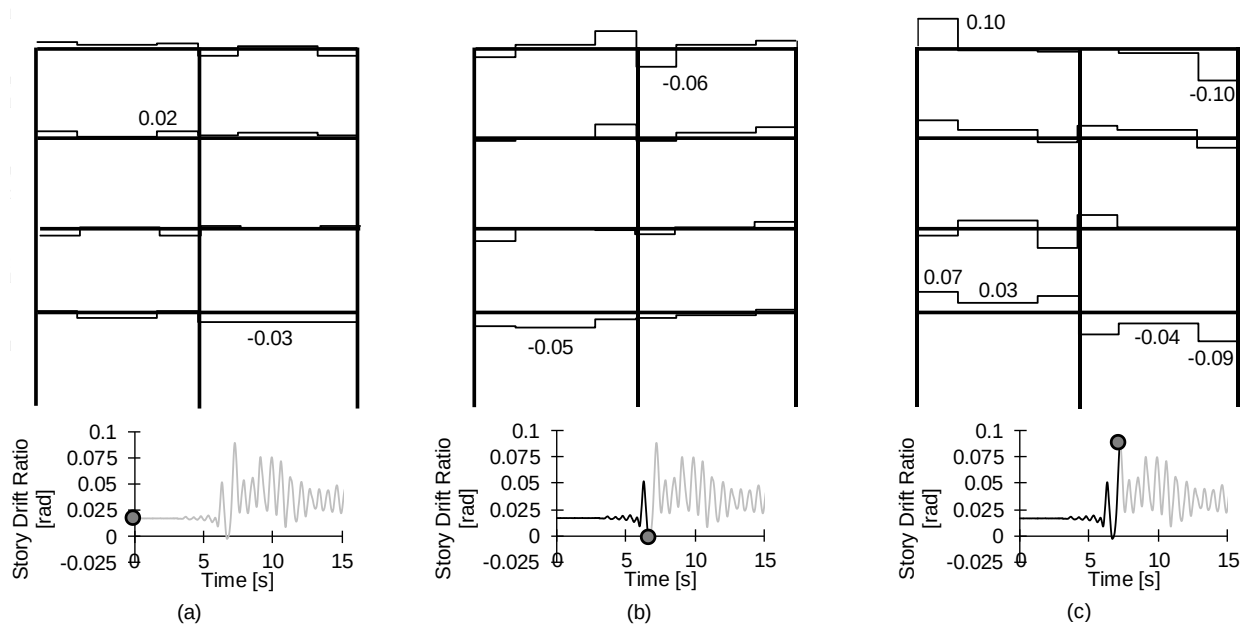


Figure 7.4 Variation of axial in the beams, 100%-4.

7.3 SHIFTING OF THE INFLECTION POINTS IN THE SHAKE-TABLE TEST SPECIMEN

Figure 7.5 shows the enlarged bending moment distribution of the frame before and after the fracture in the 2F beam occurred during the 4th 100% excitation. After fracture, the inflection points of the beams are drawn closer to the fractured end due to moment unloading of the beam. In the case of the first story exterior columns, their moment distribution tends to “merge” with the upper column, forming a two-story high column.

This change in bending moment distribution caused by the redistribution of moments invalidates the assumption of inflection points in the middle of the members. The conditions in the component tests are different to the conditions in the moment frame due to redistribution of moments that is not possible in the component tests.

After fracture, the moment in the fractured beam end unloads, and the moment increases in the column bases. No significant redistribution of moments is observed two stories beyond the fractures. Story shear of the first two stories also increased, as shown in the figure.

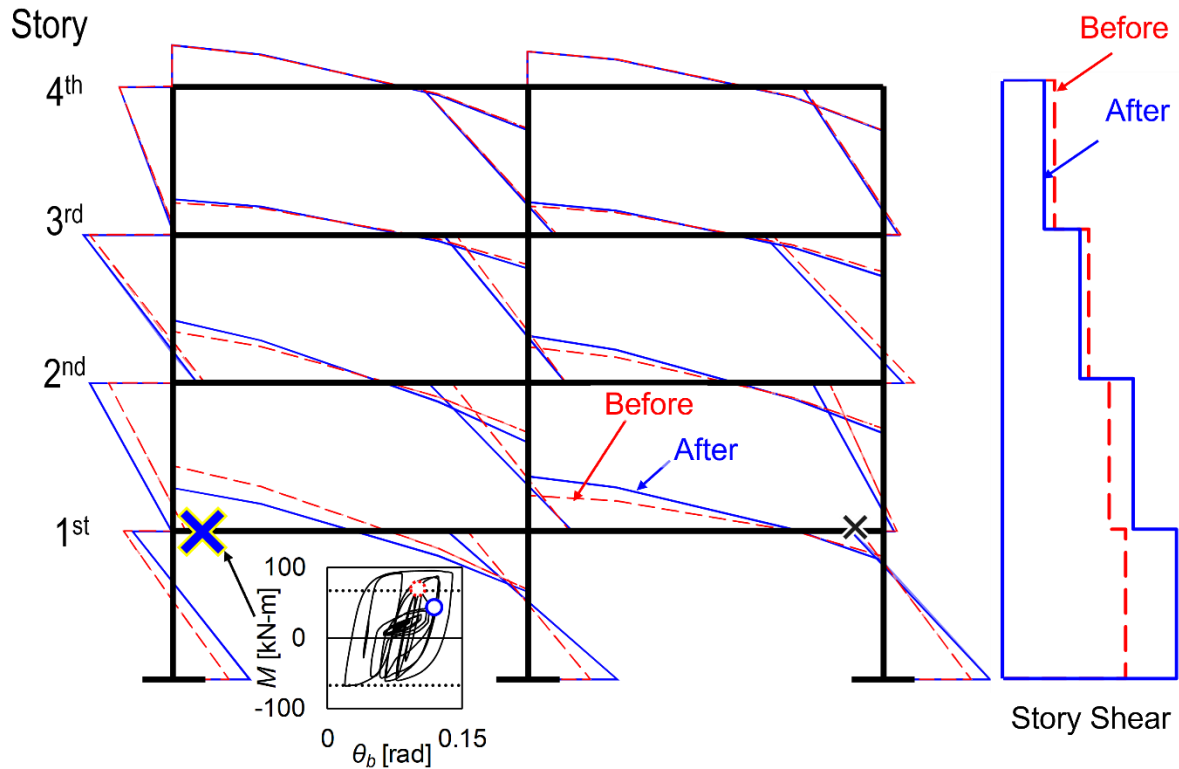


Figure 7.5 Enlarged bending moment distribution of the first two stories after beam fracture during the 100%-4 excitation.

7.4 SIZE EFFECT

The response of the subassemblages, represented by Specimen T2, was compared with full-scale beam-to-column connection tests that used a similar construction and fabrication detailing. The dimensions of the beam tested by Takahashi [48] correspond to roughly 2 times the beam of T2, while the beam by Aoyagi [49] corresponds to 2.5 times T2. The three beams were cyclically loaded under the same protocol, but the test by Aoyagi was conducted dynamically on a shake table.

Figure 7.6 shows the beam moment vs. story drift ratio response of the three beams. Table 3 lists the beam dimension parameters, steel grade, peak moment response, and failure mode. The three beams exhibited similar deformation capacity. The larger beam length to the column face over beam depth, L/d , of the beam of Specimen T2 may have caused the slightly smaller moment strength.

A detailing aspect that may potentially affect the response of reduced-scale specimens is the tack-welding of the backing bars used for the CJP groove-weld. The backing bars are tack-welded to the inner side of the beam flange and to the column through-diaphragm. In the fabrication of a reduced-scale beam-to-column connection, the size of the tack weld is limited to 4 mm, which is the minimum required by the Japanese construction practice. This means that the weld would be oversized for a real structure. To avoid scaling problems, the tack welds of the backing bars in the Shake-table test specimens and subassemblages was applied inside the CJP groove that would later be filled by the weld.

The effect of the tack welding of the backing bar on the failure mode was examined through tension-coupon tests. Figure 7.7 shows the stress-strain relationship and deformation of a coupon that used external tack welds (B) and two that did not (C and D).

The external tack weld shifted away the fracture surface by approximately 4 mm and reduced the elongation at fracture by 6.3%. Therefore, avoiding the external tack weld in the shake-table test specimen and subassemblages was a good fabrication decision.

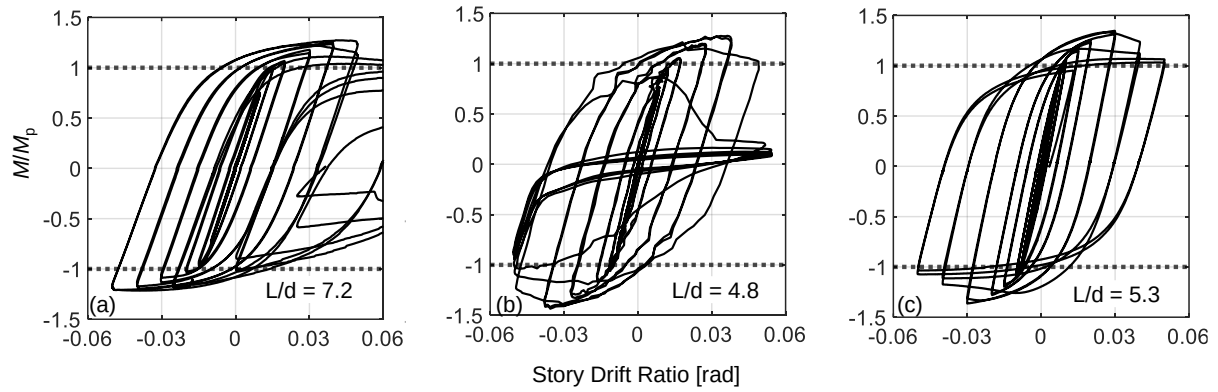


Figure 7.6 Normalized beam bending moment at column face vs. Story Drift Ratio of: a) Specimen T2, b) Aoyagi [42], and c) Takahashi [41].

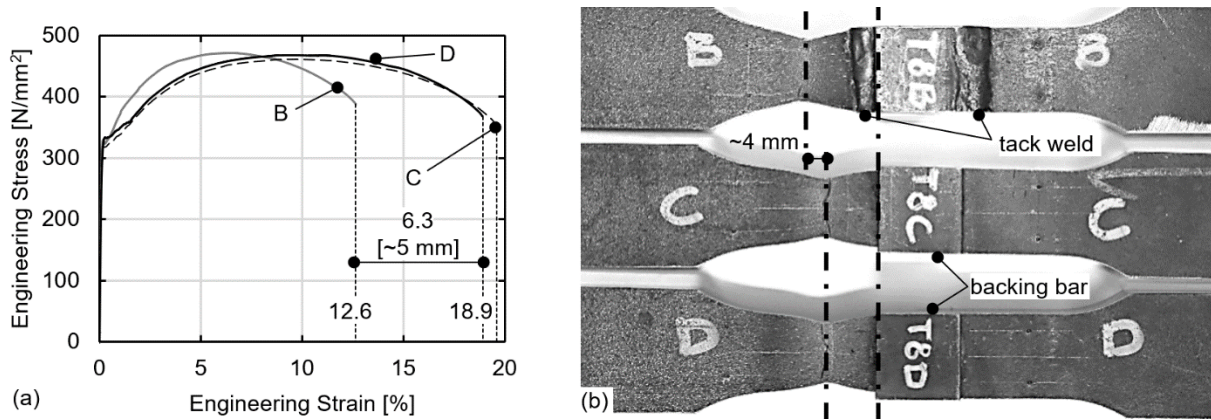


Figure 7.7 Welded coupon tests: a) stress-strain relationship, b) coupons after the tests.

Table 7.1 Comparison of beam with different scales.

Beam Section	Steel Grade	max M/M_p	$L/2d$	$b_f/2t_f$	Failure mode	Ref.
H-200×100×5.5×8	SS400	1.27 / -1.22	7.2	6.2	Top flange fracture	-
H-400×200×8×13	SN400B	1.35 / -1.36	5.3	7.7	Local buckling	3
H-500×200×10×16	SN490B	1.28 / -1.44	4.8	6.2	Bottom flange fracture	4

8. CONCLUSIONS

8.1 SUMMARY

This dissertation described an experimental program to examine the large-deformation behavior and the process leading to failure of steel moment-resisting frames. The experiments included shake-table tests of a 1:2.5-scale 4-story plane steel moment-resisting frame and beam and column subassemblages corresponding to interior and exterior joints of the moment frame. The specimens were designed and constructed using the current Japanese practice for low and mid-rise buildings, with shop-welded beam-to-column moment connections having CJP made with no weld-access-holes. Full-scale subassemblages of an I-section beam connected to an I-section column were also tested to examine the performance of the connection fabricated with the current Japanese practice. The specimens exhibited good strength and ductility, except for one connection to the column web that fractured completely in the continuity plate.

The shake-table tests provided new data that can be used to develop and calibrate analysis schemes for predicting the collapse behavior of steel moment-resisting structures. The shake-table tests were compared with the cyclic component tests. Important differences in terms of strength, degradation, and failure mode were observed between the component and system-level responses. The causes of such differences were examined.

8.2 CONCLUSIONS

8.2.1 Moment-Resisting Connections to an I-Section Column

Six beam-to-column subassemblies with dimensions typical of a 4-story steel building in Japan were tested to examine their performance and evaluated according to the AISC Seismic Provisions. The specimens consisted in three connections to the column flange and three to the column web, constructed and detailed according to the current Japanese practice. The shop-welded connections and the no-weld-access hole detail were incorporated in the specimens.

The main achievements and findings of these tests are:

- Experimental data on moment connections to the web of an I-section column was obtained.
- The failure mode of connections to the column web was promoted by yielding of the continuity plates. Therefore, yielding of the continuity plates should be controlled by either adopting for the continuity plates a stronger material and/or thicker plate than the beam flanges.
- Beam flange and web local buckling was the predominant failure mode of the beam-to-column flange connections and one web connection. The beam mechanism formed in those specimens was according to the expected failure mode determined by the design plastic strength ratios. Although cracks appeared in the termination of the groove welds of the specimens with local buckling failure mode, the connection was protected because the inelastic rotations were limited to the beam plastic hinge.
- The ductility of the shop-welded, no-access-hole detail for beam-to-column moment-resisting connections, which are commonly adopted in Japan, met the requirements for highly ductile moment frames (Special Moment Frames) established in the AISC Seismic Provisions ().
- The observed behavior of Specimen W3 suggests that the quality of the welding process, in combination with having the beam web directly welded to the column web and the use of a stronger continuity plate were important factors in achieving a good seismic performance of the minor-axis moment connections even when continuity plates flush to the column flanges were used.

8.2.2 Shake-table Collapse Tests of a 4-story 1:2.5-Scale Steel Moment-Resisting Frame

Shake-table tests of a scaled steel moment-resisting frame were conducted to obtain detailed data on the occurrence and propagation of damage leading to the collapse of steel structures. The primary achievements and findings are:

- Real data of damage initiation and propagation in a steel moment-resisting frame was obtained from shake-table and cyclic tests.
- Steel structure alone can retain its fundamental period even after the significant yielding of primary components.
- Steel moment-resisting frames have a much higher ductility than the required by design. However, the effect of scaling needs further investigation.
- The floor diaphragm in the shake-table tests worked as expected, transferring all the inertia to the main frame. The gravity frames did not take significant lateral resistance, as intended, but yielded in the 2nd story due to compatibility of displacements with the main frame.
- A strong column/weak beam ratio of 3.0 in the first story joints was adequate to form a sideway plastic mechanism involving the first three stories, as it was predicted from the design process. After forming the mechanism, the damage spread to all stories and plastic hinges formed in the 4th story beams.
- Despite sustaining large deformations after forming the mechanism, the system did not collapse due to the very high strength of the columns, which nearly doubled the nominal strength. There was no appreciable strength or stiffness degradation in the columns, and no visual signs of local buckling owing to the small width-to-thickness ratio of 10, which was easily within the limit 24 specified for hollow square columns by AIJ.
- Comparison of the component tests with the shake-table tests showed good agreement in strength capacity at the beam-to-column connection level but differed in terms of degradation. The difference was attributed to axial deformation restraint in the shake-table test beams, evidenced by smaller beam contractions than in the component test beams.
- The inertia forces imposed on the beams of the shake-table test specimen axial loads that were not present in the component tests. Those axial loads synchronized with the bending moment to produce local buckling and eventually fracture in the beam bottom flange of the shake-table test specimen.
- The global behavior of the shake-table test specimen differed from the component tests because the component tests have fixed dimensions assuming the inflection points in the mid-span and mid-height of the beams and columns, respectively, of the moment frame. In the shake-table test specimen, the inflection points varied after degradation and fracture of the beams.
- The Japanese welded connections and the no-weld-access hole behave ductile. Cracks in the weld termination are likely to form, but they propagate slowly and may cause partial fracture only after very large beam rotations, greater than 0.10 rad, or similar story drift ratio.
- The tack welds of the backing bars to the beam flanges affect the failure mode of the connection. In a scaled test specimen, the use of that external tack weld may affect the outcome of the tests. “Hiding” the tack welds inside the groove weld can avoid that problem, and does not produce any deficient performance of the CJP weld.

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APPENDIX A: SHAKE-TABLE TEST SPECIMEN DESIGN CALCULATIONS

DISTRIBUTION OF LATERAL LOADS

1. Seismic hazard zoning factor

Zone
 $Z = 1.0$

2. Critical period

Soil type
 $T_c = 0.6 \text{ s}$

3. Fundamental natural period

Use

Approximate (if calculated is not known)

Total height = 5.00 m

 $\lambda = 1$ (steel building)

 $T = 0.15 \text{ s}$

Calculated period

 $T = 0.56 \text{ s}$

Period to be used

 $T = 0.56 \text{ s}$

4. Vibration characteristic factor

 $R_t = 1.00$

T	$T < T_c$	$T_c \leq T < 2 T_c$	$T \geq 2 T_c$
R_t	1	$1 - 0.2 \left(\frac{T}{T_c} - 1 \right)^2$	$\frac{1.6 T_c}{T}$

5. Standard shear coefficient

* : R_t can also be calculated by other methods, but the calculated value shall not be less than 0.75 of the value given by this table.

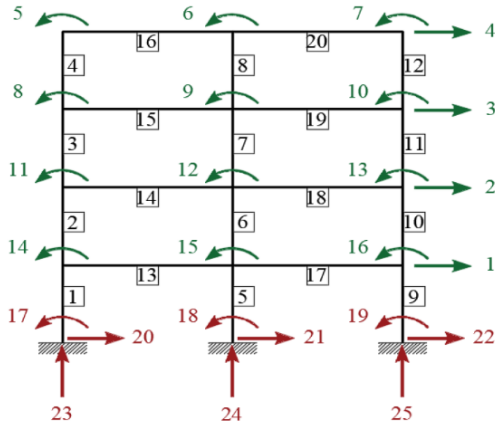
(From plastic analysis)

Ground motion
 $C_\theta = 1$

6. Lateral force distribution

 $T = 0.56 \text{ s}$
 $R_t = 1.00$
 $C_\theta = 1$
 $Z = 1.0$

Story	Wi [kN]	ΣW_i	α_i	Ai	Ci	Vi [kN]	Fi [kN]	Proportion
4	174	174	0.33	1.59	1.59	277	277	4.8
3	118	292	0.55	1.33	1.33	389	112.1	2.0
2	119	411	0.78	1.15	1.15	473	83.7	1.5
1	119	530	1.00	1.00	1.00	530	57.1	1.0
Σ	530						530	



$$\begin{aligned}
 I_c &= 0.00\text{E}+00 \text{ m}^4 \\
 H &= 0.00 \text{ m} \\
 E &= 2.05\text{E}+08 \text{ kN/m}^2 \\
 L &= 0.00 \text{ m} \\
 EI &= 0.00 \text{ kN-m}^2 \\
 F_n &= 1.76 \text{ s} \\
 T_n &= 0.57 \text{ s}
 \end{aligned}$$

Span/beam height check

$$L/d_b = 15$$

Lateral stiffness matrix [kN/m]

$$\mathbf{K} = \begin{bmatrix} 68265 & -37839 & 8010 & -1022 \\ -37839 & 56740 & -33768 & 6384 \\ 8010 & -33768 & 50970 & -23995 \\ -1022 & 6384 & -23995 & 18474 \end{bmatrix}$$

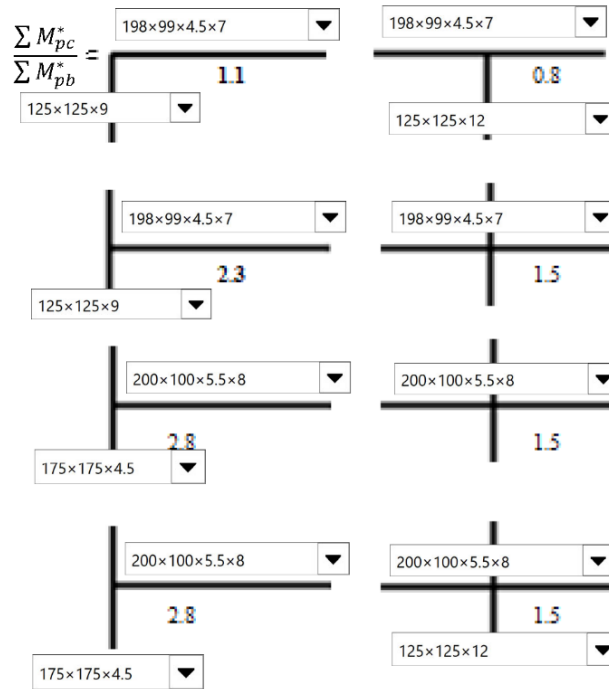
Mass matrix [ton]

$$\mathbf{M} = \begin{bmatrix} 12.1 & 0 & 0 & 0 \\ 0 & 12.1 & 0 & 0 \\ 0 & 0 & 12 & 0 \\ 0 & 0 & 0 & 17.8 \end{bmatrix}$$

$$\begin{aligned}
 \text{Guess value } w_2 &= 122.746263 \\
 \det(\mathbf{K} - w_2^2 \mathbf{M}) &= 0
 \end{aligned}$$

$$\begin{aligned}
 \text{Period} &= 0.5671 \\
 \text{Ctrl+y} &
 \end{aligned}$$

Strong column-weak beam criteria check



Element 1

M-H-V

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	17	20	14	1		End displ *(1/EI)
17	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000
20	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
14	1.9040	-4.5696	0.0000	3.8080	4.5696	-65.944
1	4.5696	-7.3114	0.0000	4.5696	7.3114	79.282
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Element 2

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	14	1	11	2		End displ *(1/EI)
14	3.8080	-4.5696	0.0000	1.9040	4.5696	-65.944
1	-4.5696	7.3114	0.0000	-4.5696	-7.3114	79.282
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11	1.9040	-4.5696	0.0000	3.8080	4.5696	-63.345
2	4.5696	-7.3114	0.0000	4.5696	7.3114	190.281
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Element 3

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.00\end{aligned}$$

	11	2	8	3		End displ *(1/EI)
11	3.2000	-3.8400	0.0000	1.6000	3.8400	-63.345
2	-3.8400	6.1440	0.0000	-3.8400	-6.1440	190.281
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
8	1.6000	-3.8400	0.0000	3.2000	3.8400	-52.979
3	3.8400	-6.1440	0.0000	3.8400	6.1440	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Element 4

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.00\end{aligned}$$

	8	3	5	4		End displ *(1/EI)
8	3.2000	-3.8400	0.0000	1.6000	3.8400	-52.979

3	-3.8400	6.1440	0.0000	-3.8400	-6.1440	0.0000	*EI	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.000
5	1.6000	-3.8400	0.0000	3.2000	3.8400	0.0000	*EI	-29.974
4	3.8400	-6.1440	0.0000	3.8400	6.1440	0.0000		363.636
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.000

Element 5

$$\begin{aligned}\theta &= 90.00^\circ & \lambda x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	18	21		15	1		End displ *(1/EI)
18	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000	0.000
21	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000	0.000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
15	1.9040	-4.5696	0.0000	3.8080	4.5696	0.0000	-45.144
1	4.5696	-7.3114	0.0000	4.5696	7.3114	0.0000	79.282
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 6

$$\begin{aligned}\theta &= 90.00^\circ & \lambda x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	15	1		12	2		End displ *(1/EI)
15	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000	-45.144
1	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000	79.282
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
12	1.9040	-4.5696	0.0000	3.8080	4.5696	0.0000	-47.438
2	4.5696	-7.3114	0.0000	4.5696	7.3114	0.0000	190.281
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 7

$$\begin{aligned}\theta &= 90.00^\circ & \lambda x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	12	2		9	3		End displ *(1/EI)
12	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000	-47.438
2	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000	190.281
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
9	1.9040	-4.5696	0.0000	3.8080	4.5696	0.0000	-43.457
3	4.5696	-7.3114	0.0000	4.5696	7.3114	0.0000	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 8

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

	9	3	6	4			End displ *(1/EI)
9	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000	-43.457
3	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
6	1.9040	-4.5696	0.0000	3.8080	4.5696	0.0000	-20.412
4	4.5696	-7.3114	0.0000	4.5696	7.3114	0.0000	363.636
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 9

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

							End displ *(1/EI)
	19	22		16	1		
19	3.8080	-4.5696	0.0000	1.9040	4.5696	0.0000	0.000
22	-4.5696	7.3114	0.0000	-4.5696	-7.3114	0.0000	0.000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
16	1.9040	-4.5696	0.0000	3.8080	4.5696	0.0000	-65.944
1	4.5696	-7.3114	0.0000	4.5696	7.3114	0.0000	79.282
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 10

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.19\end{aligned}$$

							End displ *(1/EI)
	16	1		13	2		
16	$\left[\begin{array}{cccccc} 3.8080 & -4.5696 & 0.0000 & 1.9040 & 4.5696 & 0.0000 \\ -4.5696 & 7.3114 & 0.0000 & -4.5696 & -7.3114 & 0.0000 \\ 0.0000 & 0.0000 & 0.0000 & 0.0000 & 0.0000 & 0.0000 \\ 1.9040 & -4.5696 & 0.0000 & 3.8080 & 4.5696 & 0.0000 \\ 4.5696 & -7.3114 & 0.0000 & 4.5696 & 7.3114 & 0.0000 \\ 0.0000 & 0.0000 & 0.0000 & 0.0000 & 0.0000 & 0.0000 \end{array} \right]$						
1							
13							
2							
						$*EI$	
							-65.944
							79.282
							0.000
							-63.345
							190.281
							0.000

Element 11

$$\begin{aligned}\theta &= 90.00^\circ & \lambda_x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda_y &= 1.00 \\ \text{relat. } EI &= 1.00\end{aligned}$$

							End displ *(1/EI)
	13	2	10	3			
13	3.2000	-3.8400	0.0000	1.6000	3.8400	0.0000	-63.345
2	-3.8400	6.1440	0.0000	-3.8400	-6.1440	0.0000	190.281
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

10	1.6000	-3.8400	0.0000	3.2000	3.8400	0.0000	-52.979
3	3.8400	-6.1440	0.0000	3.8400	6.1440	0.0000	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 12

$$\begin{aligned}\theta &= 90.00^\circ & \lambda x &= 0.00 \\ L &= 1.25 \text{ m} & \lambda y &= 1.00 \\ \text{relat. } EI &= 1.00\end{aligned}$$

	10	3		7	4		End displ *(1/EI)
10	3.2000	-3.8400	0.0000	1.6000	3.8400	0.0000	-52.979
3	-3.8400	6.1440	0.0000	-3.8400	-6.1440	0.0000	291.749
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
7	1.6000	-3.8400	0.0000	3.2000	3.8400	0.0000	-29.974
4	3.8400	-6.1440	0.0000	3.8400	6.1440	0.0000	363.636
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000

Element 13

$$\begin{aligned}\theta &= 0.00^\circ & \lambda x &= 1.00 \\ L &= 3.00 \text{ m} & \lambda y &= 0.00 \\ \text{relat. } EI &= 2.09\end{aligned}$$

	14		23	15		24	End displ *(1/EI)
14	2.7867	0.0000	1.3933	1.3933	0.0000	-1.3933	-65.944
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
23	1.3933	0.0000	0.9289	1.3933	0.0000	-0.9289	0.000
15	1.3933	0.0000	1.3933	2.7867	0.0000	-1.3933	-45.144
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
24	-1.3933	0.0000	-0.9289	-1.3933	0.0000	0.9289	0.000

Element 14

$$\begin{aligned}\theta &= 0.00^\circ & \lambda x &= 1.00 \\ L &= 3.00 \text{ m} & \lambda y &= 0.00 \\ \text{relat. } EI &= 2.09\end{aligned}$$

	11		23	12		24	End displ *(1/EI)
11	2.7867	0.0000	1.3933	1.3933	0.0000	-1.3933	-63.345
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
23	1.3933	0.0000	0.9289	1.3933	0.0000	-0.9289	0.000
12	1.3933	0.0000	1.3933	2.7867	0.0000	-1.3933	-47.438
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
24	-1.3933	0.0000	-0.9289	-1.3933	0.0000	0.9289	0.000

Element 15

$$\begin{aligned}\theta &= 0.00^\circ & \lambda x &= 1.00 \\ L &= 3.00 \text{ m} & \lambda y &= 0.00\end{aligned}$$

relat. $EI = 1.78$

	8	23	9	24	End displ * (1/EI)
8	2.3733	0.0000	1.1867	1.1867	-52.979
	0.0000	0.0000	0.0000	0.0000	0.000
23	1.1867	0.0000	0.7911	1.1867	0.000
9	1.1867	0.0000	1.1867	2.3733	-43.457
	0.0000	0.0000	0.0000	0.0000	0.000
24	-1.1867	0.0000	-0.7911	-1.1867	0.000
				0.7911	

Element 16

$\theta = 0.00^\circ$ $\lambda_x = 1.00$
 $L = 3.00 \text{ m}$ $\lambda_y = 0.00$
 relat. $EI = 1.78$

	5	23	6	24	End displ * (1/EI)
5	2.3733	0.0000	1.1867	1.1867	-29.974
	0.0000	0.0000	0.0000	0.0000	0.000
23	1.1867	0.0000	0.7911	1.1867	0.000
6	1.1867	0.0000	1.1867	2.3733	-20.412
	0.0000	0.0000	0.0000	0.0000	0.000
24	-1.1867	0.0000	-0.7911	-1.1867	0.000
				0.7911	

Element 17

$\theta = 0.00^\circ$ $\lambda_x = 1.00$
 $L = 3.00 \text{ m}$ $\lambda_y = 0.00$
 relat. $EI = 2.09$

	15	24	16	25	End displ * (1/EI)
15	2.7867	0.0000	1.3933	1.3933	-45.144
	0.0000	0.0000	0.0000	0.0000	0.000
24	1.3933	0.0000	0.9289	1.3933	0.000
16	1.3933	0.0000	1.3933	2.7867	-65.944
	0.0000	0.0000	0.0000	0.0000	0.000
25	-1.3933	0.0000	-0.9289	-1.3933	0.000
				0.9289	

Element 18

$\theta = 0.00^\circ$ $\lambda_x = 1.00$
 $L = 3.00 \text{ m}$ $\lambda_y = 0.00$
 relat. $EI = 2.09$

	12	24	13	25	End displ * (1/EI)
12	2.7867	0.0000	1.3933	1.3933	-47.438
	0.0000	0.0000	0.0000	0.0000	0.000
24	1.3933	0.0000	0.9289	1.3933	0.000
13	1.3933	0.0000	1.3933	2.7867	-63.345
	0.0000	0.0000	0.0000	0.0000	0.000
				0.0000	

$$25 \begin{bmatrix} -1.3933 & 0.0000 & -0.9289 & -1.3933 & 0.0000 & 0.9289 \end{bmatrix} \quad 0.000$$

Element 19

$$\begin{aligned} \theta &= 0.00^\circ & \lambda_x &= 1.00 \\ L &= 3.00 \text{ m} & \lambda_y &= 0.00 \\ \text{relat. } EI &= 1.78 \end{aligned}$$

	9	24	10	25		End displ *(1/EI)	
9	2.3733	0.0000	1.1867	1.1867	0.0000	-1.1867	-43.457
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
24	1.1867	0.0000	0.7911	1.1867	0.0000	-0.7911	0.000
10	1.1867	0.0000	1.1867	2.3733	0.0000	-1.1867	-52.979
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
25	-1.1867	0.0000	-0.7911	-1.1867	0.0000	0.7911	0.000

**EI*

Element 20

$$\begin{aligned} \theta &= 0.00^\circ & \lambda_x &= 1.00 \\ L &= 3.00 \text{ m} & \lambda_y &= 0.00 \\ \text{relat. } EI &= 1.78 \end{aligned}$$

	6	24	7	25		End displ *(1/EI)	
6	2.3733	0.0000	1.1867	1.1867	0.0000	-1.1867	-20.412
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
24	1.1867	0.0000	0.7911	1.1867	0.0000	-0.7911	0.000
7	1.1867	0.0000	1.1867	2.3733	0.0000	-1.1867	-29.974
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
25	-1.1867	0.0000	-0.7911	-1.1867	0.0000	0.7911	0.000

**EI*

APPENDIX B: STRENGTH RATIO CALCULATION

GIRDER G1

Beam SS400

$$B_b := 100 \text{ mm}$$

$$D_b := 200 \text{ mm}$$

$$t_{bf} := 8 \text{ mm}$$

$$t_{bw} := 5.5 \text{ mm}$$

$$R_b := 8 \text{ mm}$$

$$\sigma_{ybf} := 317 \frac{\text{N}}{\text{mm}^2}$$

$$\sigma_{ybw} := 349 \frac{\text{N}}{\text{mm}^2}$$

Beam plastic moment

$$Z_{bf} := B_b \cdot t_{bf} \cdot (D_b - t_{bf}) = 1.536 \cdot 10^5 \text{ mm}^3$$

$$Z_{bw} := \frac{(D_b - 2 \cdot t_{bf})^2 \cdot t_{bw}}{4} = 46552 \text{ mm}^3$$

$$Z_{br} := 0.4292 \cdot R_b^2 \cdot (D_b - 2 \cdot t_{bf} - 0.4467 \cdot R_b) = 4956.0967 \text{ mm}^3$$

$$M_{pb} := Z_{bf} \cdot \sigma_{ybf} + (Z_{bw} + Z_{br}) \cdot \sigma_{ybw} = 66.6675 \text{ kN m}$$

GIRDER G2

Beam SS400

$$B_b := 100 \text{ mm}$$

$$D_b := 200 \text{ mm}$$

$$t_{bf} := 8 \text{ mm}$$

$$t_{bw} := 5.5 \text{ mm}$$

$$R_b := 8 \text{ mm}$$

$$\sigma_{ybf} := 315 \frac{\text{N}}{\text{mm}^2}$$

$$\sigma_{ybw} := 380 \frac{\text{N}}{\text{mm}^2}$$

Beam plastic moment

$$Z_{bf} := B_b \cdot t_{bf} \cdot (D_b - t_{bf}) = 1.536 \cdot 10^5 \text{ mm}^3$$

$$Z_{bw} := \frac{(D_b - 2 \cdot t_{bf})^2 \cdot t_{bw}}{4} = 46552 \text{ mm}^3$$

$$Z_{br} := 0.4292 \cdot R_b^2 \cdot (D_b - 2 \cdot t_{bf} - 0.4467 \cdot R_b) = 4956.0967 \text{ mm}^3$$

$$M_{pb} := Z_{bf} \cdot \sigma_{ybf} + (Z_{bw} + Z_{br}) \cdot \sigma_{ybw} = 67.9571 \text{ kN m}$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-2, PZ-5

Data

Column	Beam			
$D_c := 125 \text{ mm}$	$B := 100 \text{ mm}$	$cF_y := 441 \frac{\text{N}}{\text{mm}^2}$	$cN_U := 105 \text{ kN}$	$bM_L := 405 \text{ kN m}$
$t_p := 12 \text{ mm}$	$D := 200 \text{ mm}$		$cN_L := 135 \text{ kN}$	$bM_R := 525 \text{ kN m}$
	$t_f := 8 \text{ mm}$	$bF_y := 317 \frac{\text{N}}{\text{mm}^2}$	$cM_U := 370 \text{ kN m}$	$bQ_L := 85 \text{ kN}$
$A := 50.53 \text{ cm}^2$	$t_w := 5.5 \text{ mm}$		$cM_L := 455 \text{ kN m}$	$bQ_R := 260 \text{ kN}$
	$bZ_p := 205 \text{ cm}^3$	$h_U := 1250 \text{ mm}$	$cQ_U := 270 \text{ kN}$	
	$cZ_p := 208 \text{ cm}^3$	$h_L := 1250 \text{ mm}$	$cQ_L := 310 \text{ kN}$	
$A_w := 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p$			$l_L := 3000 \text{ mm}$	$l_R := 3000 \text{ mm}$
$A_f := 0.5 \cdot A_w$			$E := 205000 \frac{\text{N}}{\text{mm}^2}$	$\nu := 0.3$
$d_c := D_c - t_p = 113 \text{ mm}$	$d_b := D - t_f = 192 \text{ mm}$		$G := \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2}$	

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.32 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0539$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 5.207 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 117.6754 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 612.893 \text{ kN}$$

Panel yield shear deformation

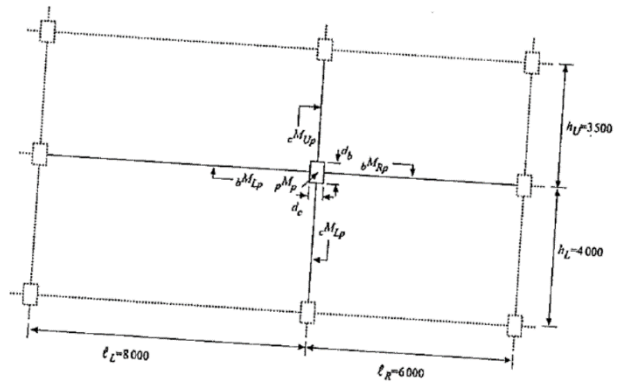
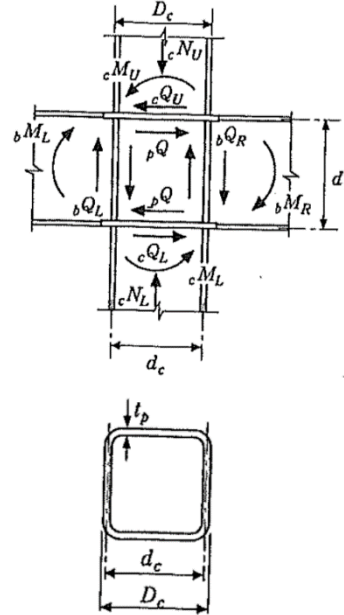
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.733 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} n \leq 0.5 & = 1 \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 132.5772 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 690.506 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{RP} := 66.7 \text{ kN m}$$

$$bM_{LP} := bM_{RP}$$

Top column plastic moment

$$n_U := \frac{cN_U}{cF_Y \cdot A} = 0.0471$$

$$cM_{Up} := \begin{cases} \text{if } n_U \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_U^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_U) \end{cases} \cdot cZ_p \cdot cF_Y = 91.4565 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_Y \cdot A} = 0.0606$$

$$cM_{LP} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_p \cdot cF_Y = 91.2791 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_P := \left(1 + \frac{d_b}{h_L}\right) \cdot cM_{LP} + \left(1 + \frac{d_b}{h_U}\right) \cdot cM_{Up} = 210.8038 \text{ kN m}$$

Beam

$$bM'_P := \left(1 + \frac{d_c}{l_R}\right) \cdot bM_{RP} + \left(1 + \frac{d_c}{l_L}\right) \cdot bM_{LP} = 138.4247 \text{ kN m}$$

Panel

$$pM'_P := \left(1 + \frac{d_c}{2 \cdot l_R} + \frac{d_c}{2 \cdot l_L} + \frac{d_b}{2 \cdot h_L} + \frac{d_b}{2 \cdot h_U}\right) \cdot pM_P = 157.9348 \text{ kN m}$$

Ratio

$$\frac{pM'_P}{bM'_P} = 1.1409$$

$$\frac{cM'_P}{bM'_P} = 1.5229$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-8

Data

$$\begin{aligned}
 D_c &:= 125 \text{ mm} & B &:= 99 \text{ mm} & cF_y &:= 441 \frac{\text{N}}{\text{mm}} & cN_U &:= 130 \text{ kN} & bM_L &:= 405 \text{ kN m} \\
 t_p &:= 12 \text{ mm} & D &:= 198 \text{ mm} & & & cN_L &:= 150 \text{ kN} & bM_R &:= 525 \text{ kN m} \\
 A &:= 50.53 \text{ cm}^2 & t_f &:= 7 \text{ mm} & cF_{yU} &:= 441 \frac{\text{N}}{\text{mm}} & cM_U &:= 370 \text{ kN m} & bQ_L &:= 85 \text{ kN} \\
 A_U &:= 50.53 \text{ cm}^2 & t_w &:= 4.5 \text{ mm} & & & cM_L &:= 455 \text{ kN m} & bQ_R &:= 260 \text{ kN} \\
 & & bZ_P &:= 175 \text{ cm}^3 & bF_y &:= 315 \frac{\text{N}}{\text{mm}} & cQ_U &:= 270 \text{ kN} & & \\
 & & cZ_P &:= 208 \text{ cm}^3 & h_U &:= 1250 \text{ mm} & cQ_L &:= 310 \text{ kN} & l_L &:= 3000 \text{ mm} \\
 & & cZ_{PU} &:= 208 \text{ cm}^3 & h_L &:= 1250 \text{ mm} & & & l_R &:= 3000 \text{ mm}
 \end{aligned}$$

$$A_w := 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p$$

$$A_f := 0.5 \cdot A_w$$

$$d_c := D_c - t_p = 113 \text{ mm}$$

$$d_b := D - t_f = 191 \text{ mm}$$

$$E := 205000 \frac{\text{N}}{\text{mm}^2} \quad \nu := 0.3$$

$$G := \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2}$$

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.61 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0628$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 5.1799 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 117.001 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 612.571 \text{ kN}$$

Panel yield shear deformation

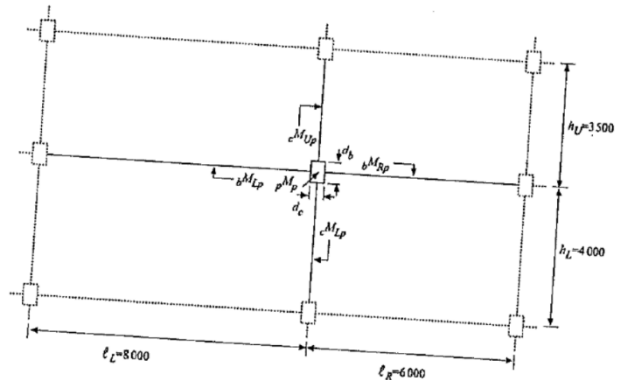
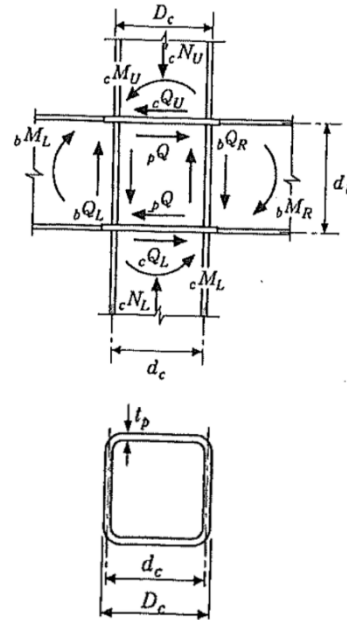
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.729 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} \text{if } n \leq 0.5 & = 1 \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 131.8867 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 690.506 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{Rp} := 68 \text{ kN m}$$

$$bM_{Lp} := bM_{Rp}$$

Top column plastic moment

$$n_U := \frac{cN_U}{cF_{yU} \cdot A_U} = 0.0583$$

$$cM_{Up} := \begin{cases} \text{if } n_U \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_U^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_U) \end{cases} \cdot cZ_{pU} \cdot cF_{yU} = 91.3118 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_y \cdot A} = 0.0673$$

$$cM_{Lp} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_p \cdot cF_y = 91.1738 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_p := \left(1 + \frac{d_b}{h_L}\right) \cdot cM_{Lp} + \left(1 + \frac{d_b}{h_U}\right) \cdot cM_{Up} = 210.3694 \text{ kN m}$$

Beam

$$bM'_p := \left(1 + \frac{d_c}{l_R}\right) \cdot bM_{Rp} + \left(1 + \frac{d_c}{l_L}\right) \cdot bM_{Lp} = 141.1227 \text{ kN m}$$

Panel

$$pM'_p := \left(1 + \frac{d_c}{2 \cdot l_R} + \frac{d_c}{2 \cdot l_L} + \frac{d_b}{2 \cdot h_L} + \frac{d_b}{2 \cdot h_U}\right) \cdot pM_p = 157.0067 \text{ kN m}$$

Ratio

$$\frac{pM'_p}{bM'_p} = 1.1126$$

$$\frac{cM'_p}{bM'_p} = 1.4907$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-11

Data

$$\begin{aligned}
 D_c &:= 125 \text{ mm} & B &:= 99 \text{ mm} & cF_y &:= 441 \frac{\text{N}}{\text{mm}^2} & cN_U &:= 0 \text{ kN} & bM_L &:= 405 \text{ kN m} \\
 t_p &:= 12 \text{ mm} & D &:= 198 \text{ mm} & & & cN_L &:= 150 \text{ kN} & bM_R &:= 525 \text{ kN m} \\
 A &:= 50.5 \text{ cm}^2 & t_f &:= 7 \text{ mm} & & & cM_U &:= 370 \text{ kN m} & bQ_L &:= 85 \text{ kN} \\
 & & t_w &:= 4.5 \text{ mm} & & & cM_L &:= 455 \text{ kN m} & bQ_R &:= 260 \text{ kN} \\
 & & bZ_p &:= 175 \text{ cm}^3 & bF_y &:= 315 \frac{\text{N}}{\text{mm}^2} & cQ_U &:= 270 \text{ kN} & & \\
 & & cZ_p &:= 208 \text{ cm}^3 & & & cQ_L &:= 310 \text{ kN} & & \\
 & & & & h_U &:= 1250 \text{ mm} & l_L &:= 3000 \text{ mm} & l_R &:= 3000 \text{ mm} \\
 & & & & h_L &:= 1250 \text{ mm} & & & & \\
 A_w &:= 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p & & & E &:= 205000 \frac{\text{N}}{\text{mm}^2} & \nu &:= 0.3 \\
 A_f &:= 0.5 \cdot A_w & & & G &:= \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2} \\
 d_c &:= D_c - t_p = 113 \text{ mm} & d_b &:= D - t_f = 191 \text{ mm} & & & & & &
 \end{aligned}$$

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.61 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0337$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 5.1799 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 117.1661 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 613.435 \text{ kN}$$

Panel yield shear deformation

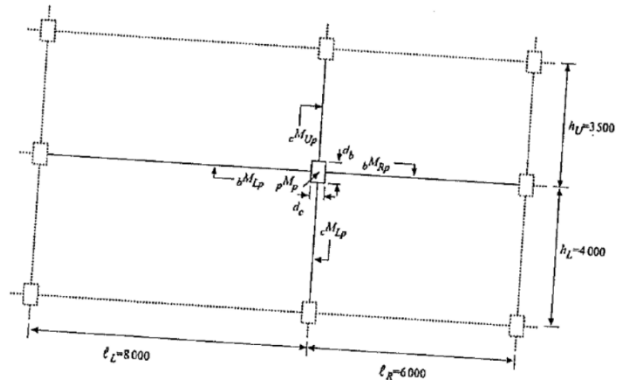
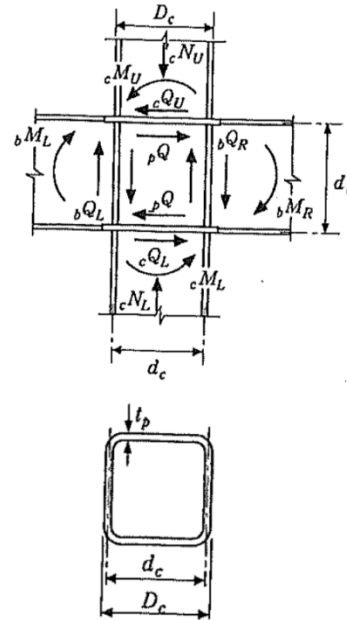
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.738 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} 1 & \text{if } n \leq 0.5 \\ 2 \cdot \sqrt{n \cdot (n - 1)} & \text{else} \end{cases} = 1$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 131.8867 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 690.506 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{RP} := 68 \text{ kN m}$$

$$bM_{LP} := bM_{RP}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_y \cdot A} = 0.0674$$

$$cM_{LP} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_p \cdot cF_y = 91.1732 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_P := \left(1 + \frac{d_b}{h_L} \right) \cdot cM_{LP} = 105.1044 \text{ kN m}$$

Beam

$$bM'_P := \left(1 + \frac{d_c}{l_R} \right) \cdot bM_{RP} + \left(1 + \frac{d_c}{l_L} \right) \cdot bM_{LP} = 141.1227 \text{ kN m}$$

Panel

$$pM'_P := \left(1 + \frac{d_c}{2 \cdot l_R} + \frac{d_c}{2 \cdot l_L} + \frac{d_b}{2 \cdot h_L} \right) \cdot pM_P = 146.9306 \text{ kN m}$$

Ratio

$$\frac{pM'_P}{bM'_P} = 1.0412$$

$$\frac{cM'_P}{bM'_P} = 0.7448$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-1, PZ-3

Data

Column	Beam			
$D_c := 125 \text{ mm}$	$B := 100 \text{ mm}$	$cF_y := 441 \frac{\text{N}}{\text{mm}^2}$	$cN_U := 129 \text{ kN}$	$bM_L := 405 \text{ kN m}$
$t_p := 12 \text{ mm}$	$D := 200 \text{ mm}$		$cN_L := 225 \text{ kN}$	$bM_R := 525 \text{ kN m}$
	$t_f := 8 \text{ mm}$	$bF_y := 315 \frac{\text{N}}{\text{mm}^2}$	$cM_U := 370 \text{ kN m}$	$bQ_L := 85 \text{ kN}$
$A := 50.53 \text{ cm}^2$	$t_w := 5.5 \text{ mm}$		$cM_L := 455 \text{ kN m}$	$bQ_R := 260 \text{ kN}$
	$bZ_p := 205 \text{ cm}^3$	$h_U := 1250 \text{ mm}$	$cQ_U := 270 \text{ kN}$	
	$cZ_p := 208 \text{ cm}^3$	$h_L := 1250 \text{ mm}$	$cQ_L := 310 \text{ kN}$	
$A_w := 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p$			$l_L := 3000 \text{ mm}$	$l_R := 3000 \text{ mm}$
$A_f := 0.5 \cdot A_w$			$E := 205000 \frac{\text{N}}{\text{mm}^2}$	$\nu := 0.3$
$d_c := D_c - t_p = 113 \text{ mm}$	$d_b := D - t_f = 192 \text{ mm}$		$G := \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2}$	

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.32 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0794$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 5.207 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 117.4741 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 611.844 \text{ kN}$$

Panel yield shear deformation

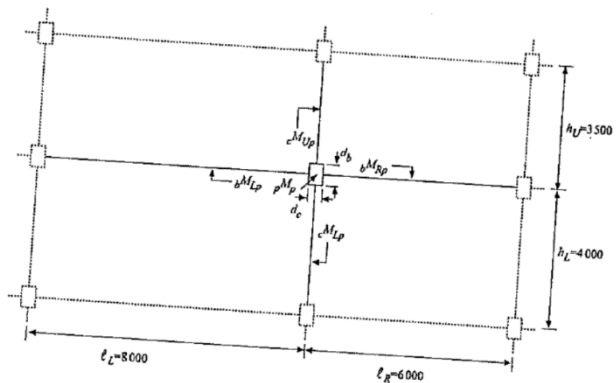
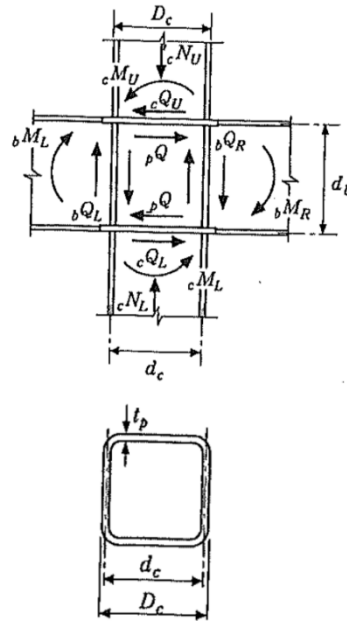
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.723 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} \text{if } n \leq 0.5 & = 1 \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 132.5772 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 690.506 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{RP} := 66.7 \text{ kN m}$$

Top column plastic moment

$$n_U := \frac{cN_U}{cF_Y \cdot A} = 0.0579$$

$$cM_{Up} := \begin{cases} \text{if } n_U \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_U^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_U) \end{cases} \cdot cZ_p \cdot cF_Y = 91.3181 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_Y \cdot A} = 0.101$$

$$cM_{Lp} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_p \cdot cF_Y = 90.4811 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_p := \left(1 + \frac{d_b}{h_L}\right) \cdot cM_{Lp} + \left(1 + \frac{d_b}{h_U}\right) \cdot cM_{Up} = 209.7236 \text{ kN m}$$

Beam

$$bM'_p := \left(1 + \frac{d_c}{I_R}\right) \cdot bM_{RP} = 69.2124 \text{ kN m}$$

Panel

$$pM'_p := \left(1 + \frac{d_c}{2 \cdot I_R} + \frac{d_b}{2 \cdot h_L} + \frac{d_b}{2 \cdot h_U}\right) \cdot pM_p = 155.4379 \text{ kN m}$$

Ratio

$$\frac{pM'_p}{bM'_p} = 2.2458$$

$$\frac{cM'_p}{bM'_p} = 3.0301$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-4, PZ-6

Data

$$\begin{aligned}
 D_c &:= 125 \text{ mm} & B &:= 100 \text{ mm} & cF_y &:= 441 \frac{\text{N}}{\text{mm}^2} & cN_U &:= 130 \text{ kN} & bM_L &:= 405 \text{ kN m} \\
 t_p &:= 12 \text{ mm} & D &:= 200 \text{ mm} & & & cN_L &:= 150 \text{ kN} & bM_R &:= 525 \text{ kN m} \\
 A &:= 50.53 \text{ cm}^2 & t_f &:= 8 \text{ mm} & cF_{yU} &:= 406 \frac{\text{N}}{\text{mm}^2} & cM_U &:= 370 \text{ kN m} & bQ_L &:= 85 \text{ kN} \\
 A_U &:= 39.7 \text{ cm}^2 & t_w &:= 5.5 \text{ mm} & & & cM_L &:= 455 \text{ kN m} & bQ_R &:= 260 \text{ kN} \\
 & & bZ_p &:= 205 \text{ cm}^3 & bF_y &:= 315 \frac{\text{N}}{\text{mm}^2} & cQ_U &:= 270 \text{ kN} & & \\
 & & cZ_p &:= 208 \text{ cm}^3 & & & cQ_L &:= 310 \text{ kN} & & \\
 & & cZ_{pU} &:= 169 \text{ cm}^3 & h_U &:= 1250 \text{ mm} & l_R &:= 3000 \text{ mm} & & \\
 & & & & h_L &:= 1250 \text{ mm} & & & & \\
 A_w &:= 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p & & & & & E &:= 205000 \frac{\text{N}}{\text{mm}^2} & \nu &:= 0.3 \\
 A_f &:= 0.5 \cdot A_w & & & & & G &:= \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2} & & \\
 d_c &:= D_c - t_p = 113 \text{ mm} & d_b &:= D - t_f = 192 \text{ mm} & & & & & &
 \end{aligned}$$

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.32 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0628$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 5.207 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 117.6136 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 612.571 \text{ kN}$$

Panel yield shear deformation

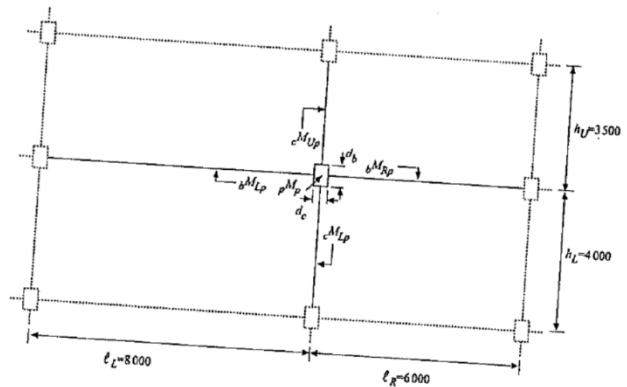
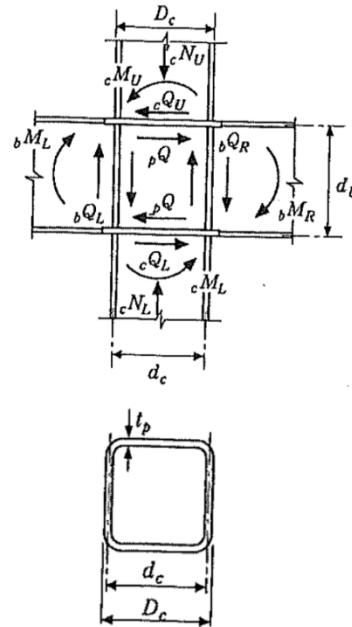
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.729 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} n \leq 0.5 & = 1 \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 132.5772 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 690.506 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{Rp} := 66.7 \text{ kN m}$$

Top column plastic moment

$$n_U := \frac{cN_U}{cF_{yU} \cdot A_U} = 0.0807$$

$$cM_{Up} := \begin{cases} \text{if } n_U \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_U^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_U) \end{cases} \cdot cZ_{pU} \cdot cF_{yU} = 68.0189 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_y \cdot A} = 0.0673$$

$$cM_{Lp} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_p \cdot cF_y = 91.1738 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_p := \left(1 + \frac{d_b}{h_L}\right) \cdot cM_{Lp} + \left(1 + \frac{d_b}{h_U}\right) \cdot cM_{Up} = 183.6447 \text{ kN m}$$

Beam

$$bM'_p := \left(1 + \frac{d_c}{I_R}\right) \cdot bM_{Rp} = 69.2124 \text{ kN m}$$

Panel

$$pM'_p := \left(1 + \frac{d_c}{2 \cdot I_R} + \frac{d_b}{2 \cdot h_L} + \frac{d_b}{2 \cdot h_U}\right) \cdot pM_p = 155.4379 \text{ kN m}$$

Ratio

$$\frac{pM'_p}{bM'_p} = 2.2458$$

$$\frac{cM'_p}{bM'_p} = 2.6534$$

PANEL ZONE CHECK - SQUARE BOX COLUMN
 PZ-7, PZ-9

Data

Column

Beam

$$\begin{aligned}
 D_c &:= 125 \text{ mm} & B &:= 99 \text{ mm} & cF_y &:= 406 \frac{\text{N}}{\text{mm}^2} & cN_U &:= 0 \text{ kN} & bM_L &:= 405 \text{ kN m} \\
 t_p &:= 9 \text{ mm} & D &:= 198 \text{ mm} & & & cN_L &:= 0 \text{ kN} & bM_R &:= 525 \text{ kN m} \\
 A &:= 39.7 \text{ cm}^2 & t_f &:= 7 \text{ mm} & bF_y &:= 315 \frac{\text{N}}{\text{mm}^2} & cM_U &:= 370 \text{ kN m} & bQ_L &:= 85 \text{ kN} \\
 & & t_w &:= 4.5 \text{ mm} & & & cM_L &:= 455 \text{ kN m} & bQ_R &:= 260 \text{ kN} \\
 & & bZ_p &:= 175 \text{ cm}^3 & h_U &:= 1250 \text{ mm} & cQ_U &:= 270 \text{ kN} & & \\
 & & cZ_p &:= 169 \text{ cm}^3 & h_L &:= 1250 \text{ mm} & cQ_L &:= 310 \text{ kN} & & \\
 A_w &:= 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p & & & l_R &:= 3000 \text{ mm} & & & & \\
 A_f &:= 0.5 \cdot A_w & & & & & E &:= 205000 \frac{\text{N}}{\text{mm}^2} & \nu &:= 0.3 \\
 d_c &:= D_c - t_p = 116 \text{ mm} & d_b &:= D - t_f = 191 \text{ mm} & & & G &:= \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}^2}
 \end{aligned}$$

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.61 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 3.9881 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 83.0954 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 435.054 \text{ kN}$$

Panel yield shear deformation

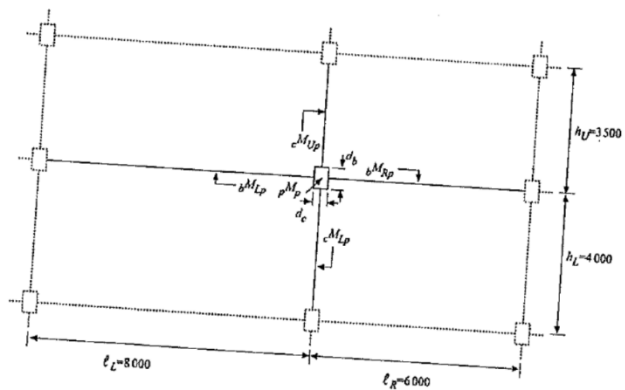
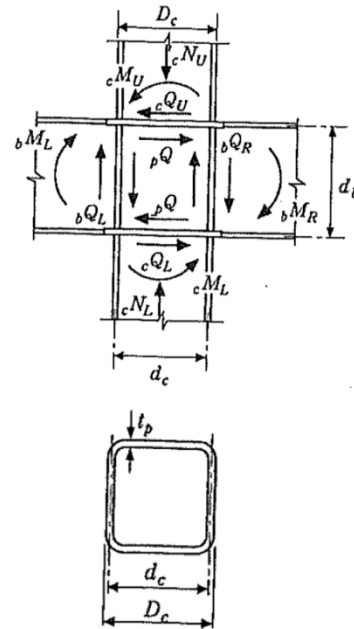
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.285 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} n \leq 0.5 & = 1 \\ 1 & \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 93.4823 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 489.436 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{RP} := 68 \text{ kN m}$$

Top column plastic moment

$$n_U := \frac{cN_U}{cF_Y \cdot A} = 0$$

$$cM_{Up} := \begin{cases} \text{if } n_U \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_U^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_U) \end{cases} \cdot cZ_P \cdot cF_Y = 68.614 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_Y \cdot A} = 0$$

$$cM_{Lp} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_P \cdot cF_Y = 68.614 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_P := \left(1 + \frac{d_b}{h_L}\right) \cdot cM_{Lp} + \left(1 + \frac{d_b}{h_U}\right) \cdot cM_{Up} = 158.1964 \text{ kN m}$$

Beam

$$bM'_P := \left(1 + \frac{d_c}{l_R}\right) \cdot bM_{RP} = 70.6293 \text{ kN m}$$

Panel

$$pM'_P := \left(1 + \frac{d_c}{2 \cdot l_R} + \frac{d_b}{2 \cdot h_L} + \frac{d_b}{2 \cdot h_U}\right) \cdot pM_P = 109.5737 \text{ kN m}$$

Ratio

$$\frac{pM'_P}{bM'_P} = 1.5514$$

$$\frac{cM'_P}{bM'_P} = 2.2398$$

PANEL ZONE CHECK - SQUARE BOX COLUMN PZ-10, PZ-12

Data

$$\begin{aligned}
 D_c &:= 125 \text{ mm} & B &:= 99 \text{ mm} & cF_y &:= 406 \frac{\text{N}}{\text{mm}} & cN_U &:= 0 \text{ kN} & bM_L &:= 405 \text{ kN m} \\
 t_p &:= 9 \text{ mm} & D &:= 198 \text{ mm} & & & cN_L &:= 150 \text{ kN} & bM_R &:= 525 \text{ kN m} \\
 & & t_f &:= 7 \text{ mm} & & & cM_U &:= 370 \text{ kN m} & bQ_L &:= 85 \text{ kN} \\
 A &:= 39.7 \text{ cm}^2 & t_w &:= 4.5 \text{ mm} & & & cM_L &:= 455 \text{ kN m} & bQ_R &:= 260 \text{ kN} \\
 & & bZ_P &:= 175 \text{ cm}^3 & bF_y &:= 315 \frac{\text{N}}{\text{mm}} & cQ_U &:= 270 \text{ kN} & & \\
 & & cZ_P &:= 169 \text{ cm}^3 & & & cQ_L &:= 310 \text{ kN} & & \\
 & & & & h_U &:= 1250 \text{ mm} & l_L &:= 3000 \text{ mm} & l_R &:= 3000 \text{ mm} \\
 & & & & h_L &:= 1250 \text{ mm} & & & & \\
 & & & & & & E &:= 205000 \frac{\text{N}}{\text{mm}} & \nu &:= 0.3 \\
 & & & & & & G &:= \frac{E}{2 \cdot (1 + \nu)} = 7.885 \cdot 10^4 \frac{\text{N}}{\text{mm}} & & \\
 A_w &:= 2 \cdot (D_c - 2 \cdot t_p) \cdot t_p & & & & & & & & \\
 A_f &:= 0.5 \cdot A_w & & & & & & & & \\
 d_c &:= D_c - t_p = 116 \text{ mm} & d_b &:= D - t_f = 191 \text{ mm} & & & & & &
 \end{aligned}$$

(1) Panel moment

$$pM := bM_R + bM_L - (cQ_U + cQ_L) \cdot \frac{d_b}{2} = 874.61 \text{ kN m}$$

(2) Yield strength

Panel axial load ratio

$$n := \frac{cN_U + cN_L}{2 \cdot cF_y \cdot A} = 0.0465$$

Panel equivalent volume

$$V_e := 2 \cdot d_c \cdot d_b \cdot t_p = 3.9881 \cdot 10^5 \text{ mm}^3$$

Panel yield strength

$$pM_y := \frac{8}{9} \cdot V_e \cdot \sqrt{1 - n^2} \cdot \frac{cF_y}{\sqrt{3}} = 83.0053 \text{ kN m}$$

Panel shear yield strength

$$pV_y := \frac{pM_y}{d_b} = 434.583 \text{ kN}$$

Panel yield shear deformation

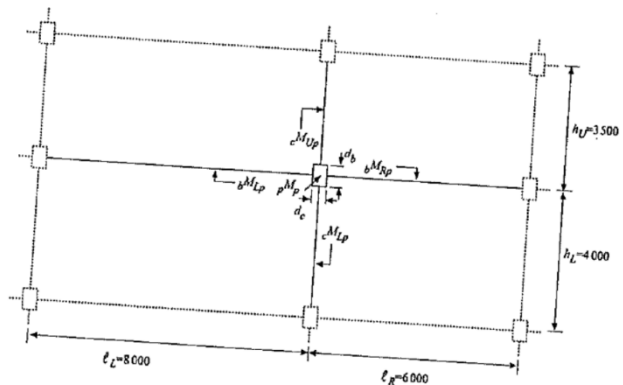
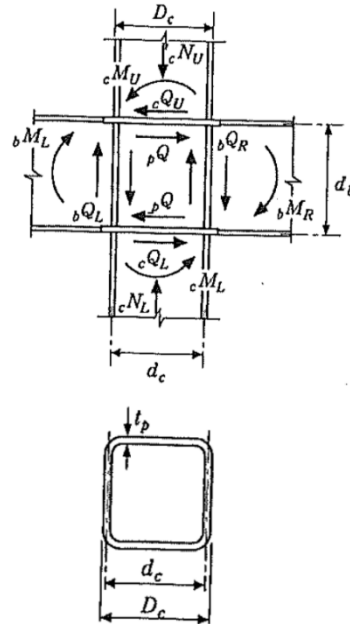
$$\gamma_y := \frac{pV_y}{t_p \cdot d_c \cdot G} = 5.279 \cdot 10^{-3} \text{ rad}$$

(3) Plastic strength

$$\alpha_n := \begin{cases} n \leq 0.5 & = 1 \\ \text{else} & 2 \cdot \sqrt{n \cdot (n - 1)} \end{cases}$$

$$pM_p := \alpha_n \cdot V_e \cdot \frac{cF_y}{\sqrt{3}} = 93.4823 \text{ kN m}$$

$$pV_p := \frac{pM_p}{d_b} = 489.436 \text{ kN}$$



(4) Strength ratio

Beam plastic moment

$$bM_{RP} := 68 \text{ kN m}$$

Bottom column plastic moment

$$n_L := \frac{cN_L}{cF_Y \cdot A} = 0.0931$$

$$cM_{LP} := \begin{cases} \text{if } n_L \leq 0.5 \\ 1 - \frac{4}{3} \cdot n_L^2 \\ \text{else} \\ \frac{4}{3} \cdot (1 - n_L) \end{cases} \cdot cZ_P \cdot cF_Y = 67.8217 \text{ kN m}$$

Nodal plastic moments

Column

$$cM'_P := \left(1 + \frac{d_b}{h_L} \right) \cdot cM_{LP} = 78.1848 \text{ kN m}$$

Beam

$$bM'_P := \left(1 + \frac{d_c}{I_R} \right) \cdot bM_{RP} = 70.6293 \text{ kN m}$$

Panel

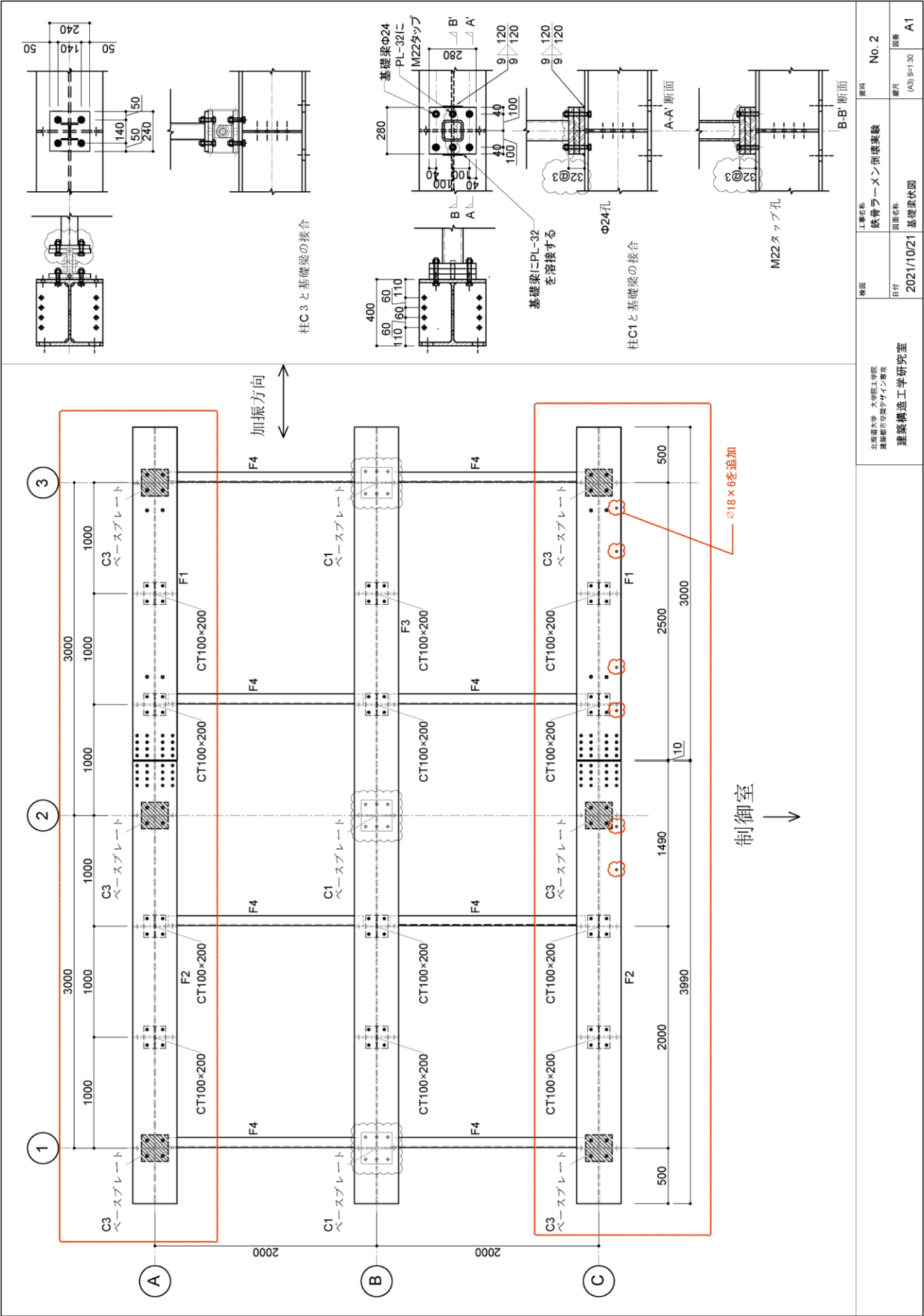
$$pM'_P := \left(1 + \frac{d_c}{2 \cdot I_R} + \frac{d_b}{2 \cdot h_L} \right) \cdot pM_P = 102.4316 \text{ kN m}$$

Ratio

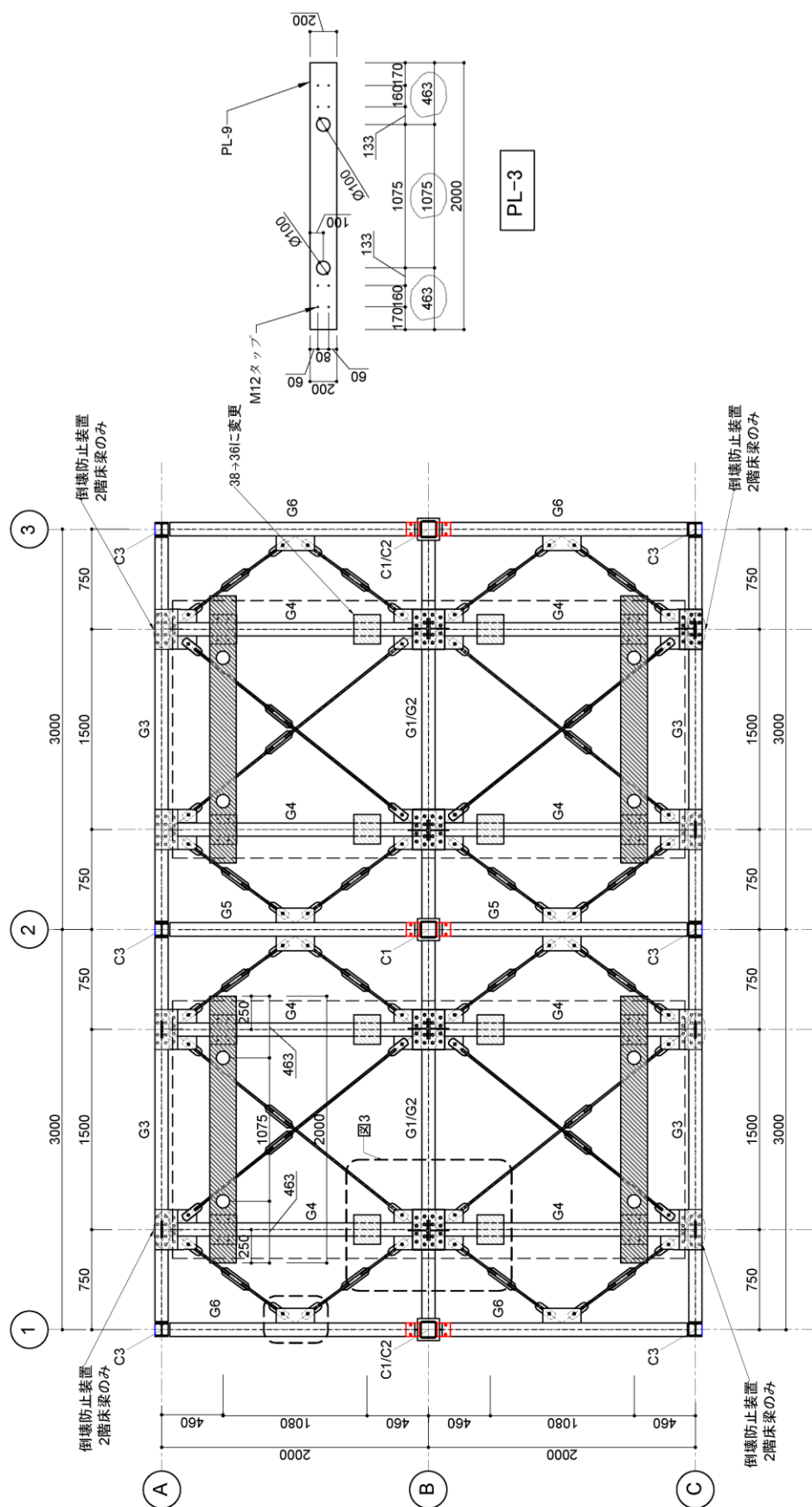
$$\frac{pM'_P}{bM'_P} = 1.4503$$

$$\frac{cM'_P}{bM'_P} = 1.107$$

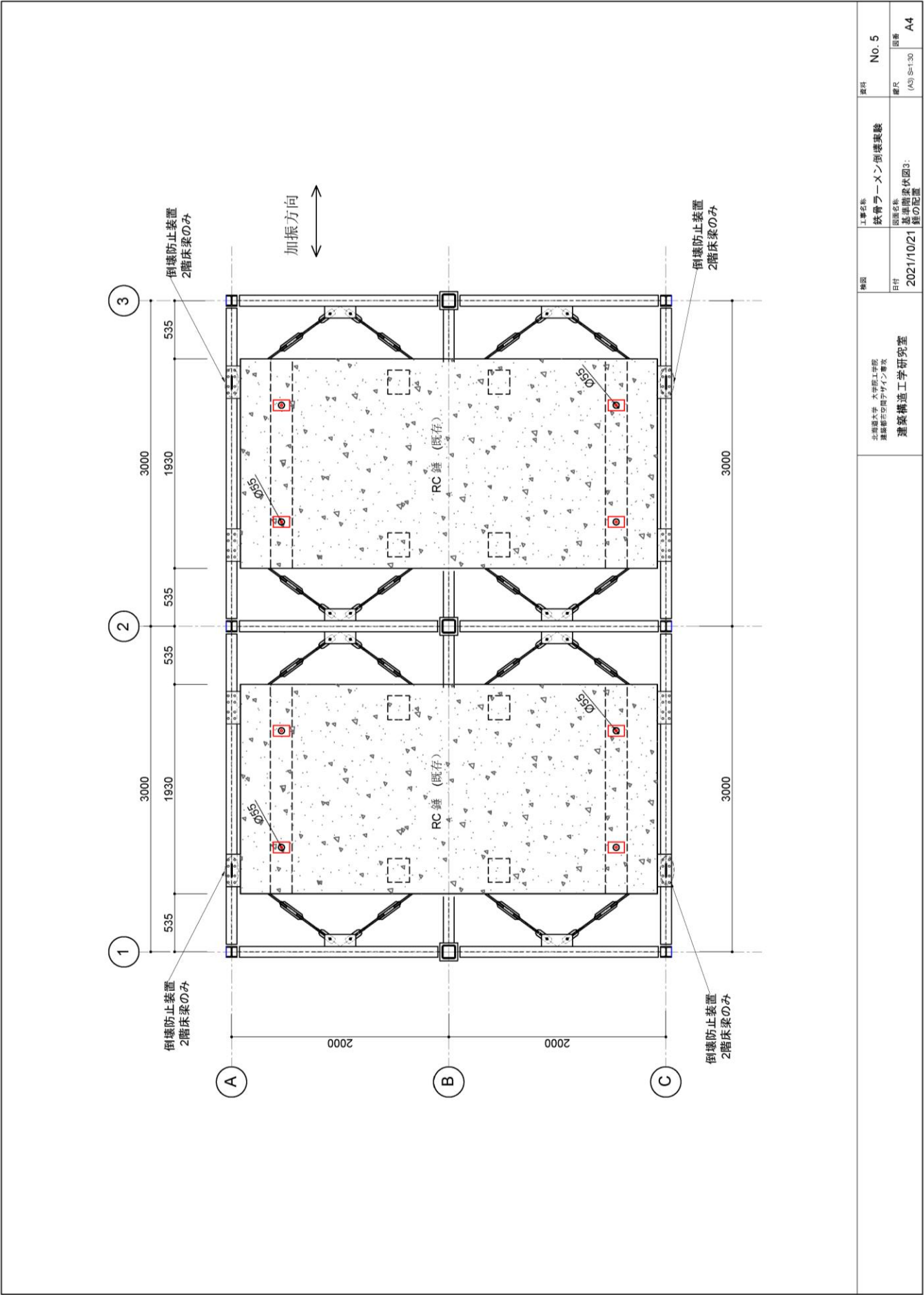
APPENDIX C: ENGINEERING DRAWINGS OF THE SHAKE-TABLE TEST SPECIMEN



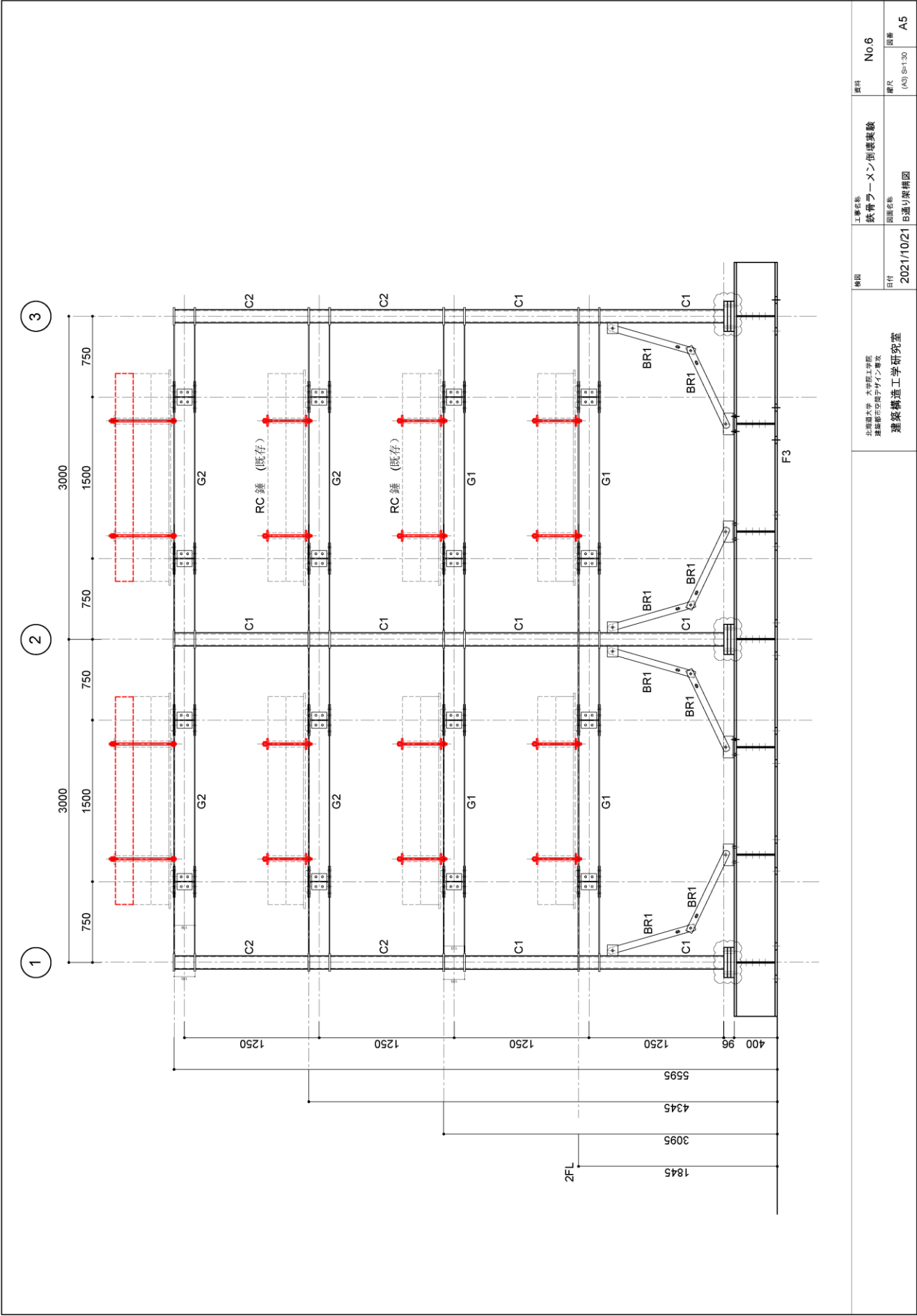


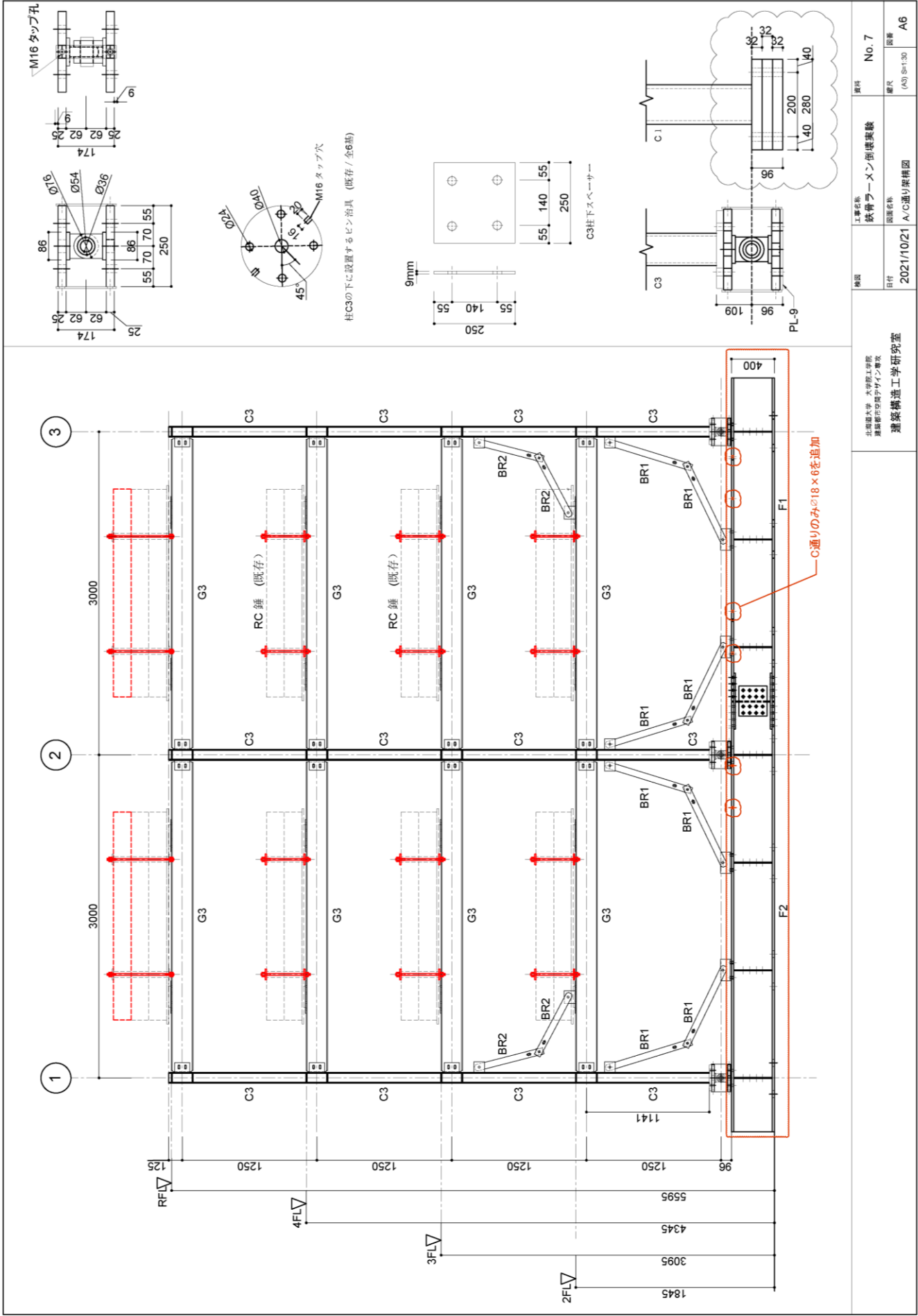


研究機関 北海道大学 大学院工学部 建築都市空間デザイン専攻 建築構造工学研究室	研究内容 鉄骨ラーメン型震害試験 図面名称 鉄骨梁柱状図2: 壁の支持面	図面番号 2021/10/21	図面 No. 4

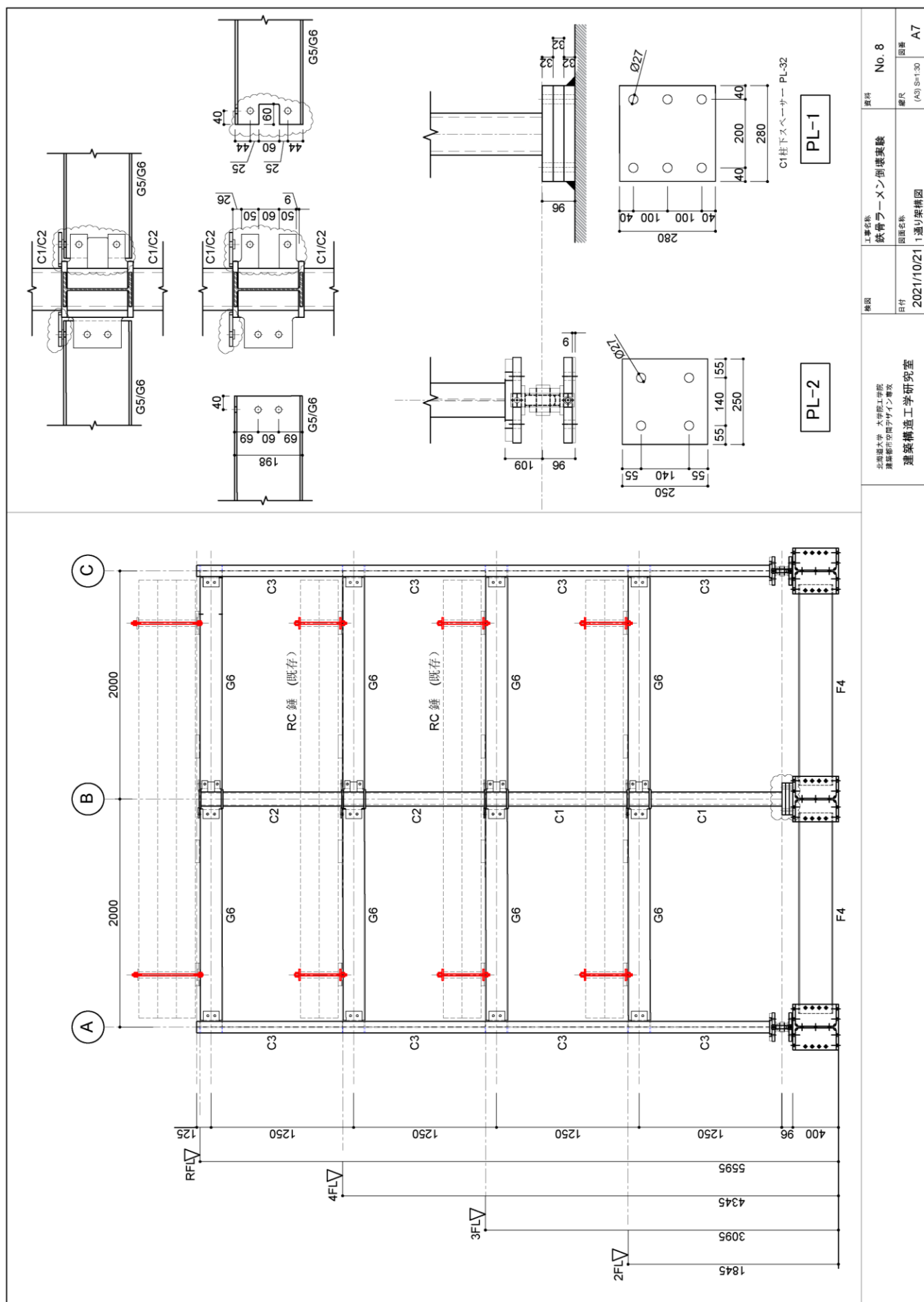


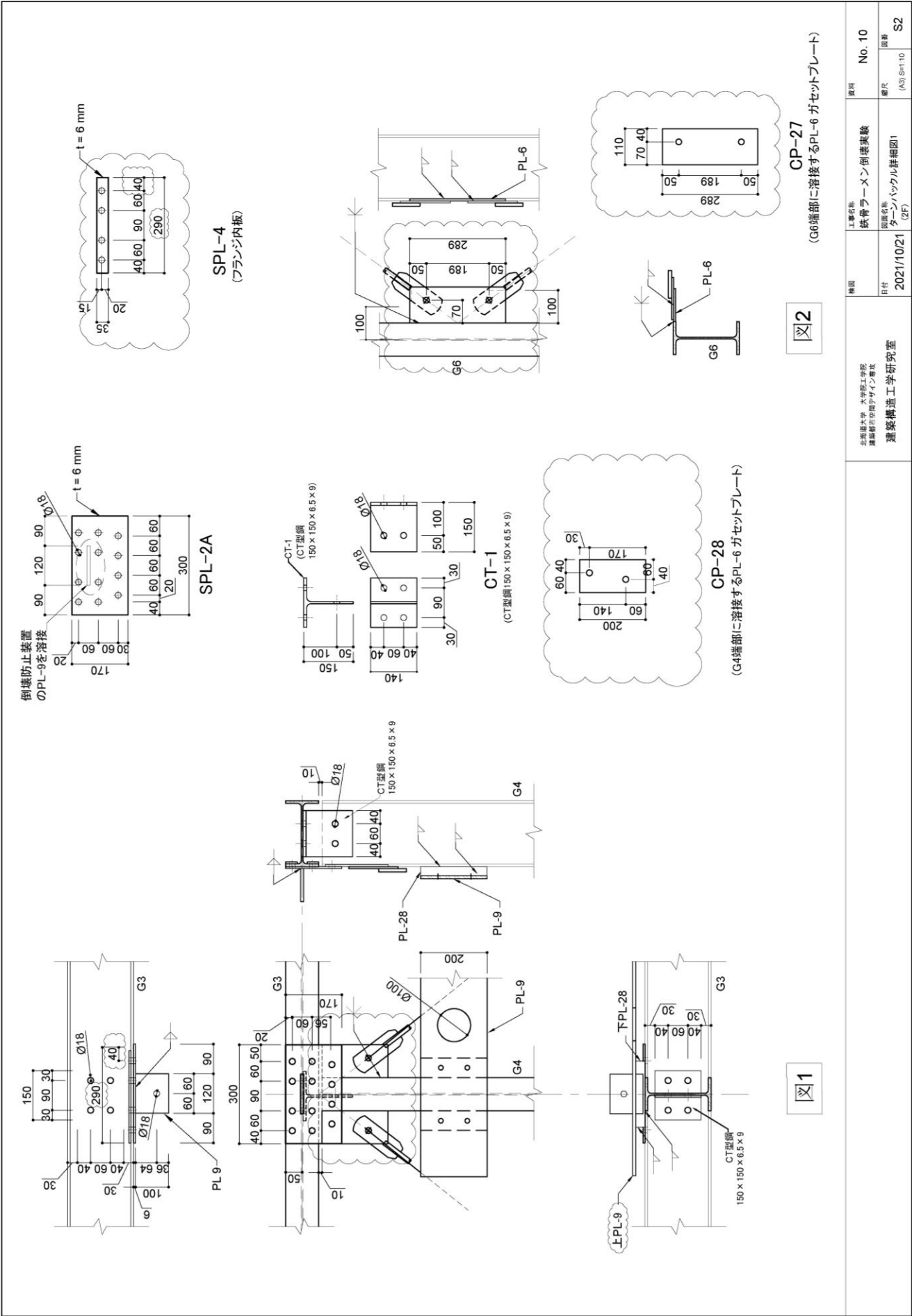
図名 鉄骨ラーメン倒壊実験 図面名 基礎構造体図3: 基礎構造体図3: 柱の配置	図面 日付 2021/10/21	設計 No. 5	図番 A4

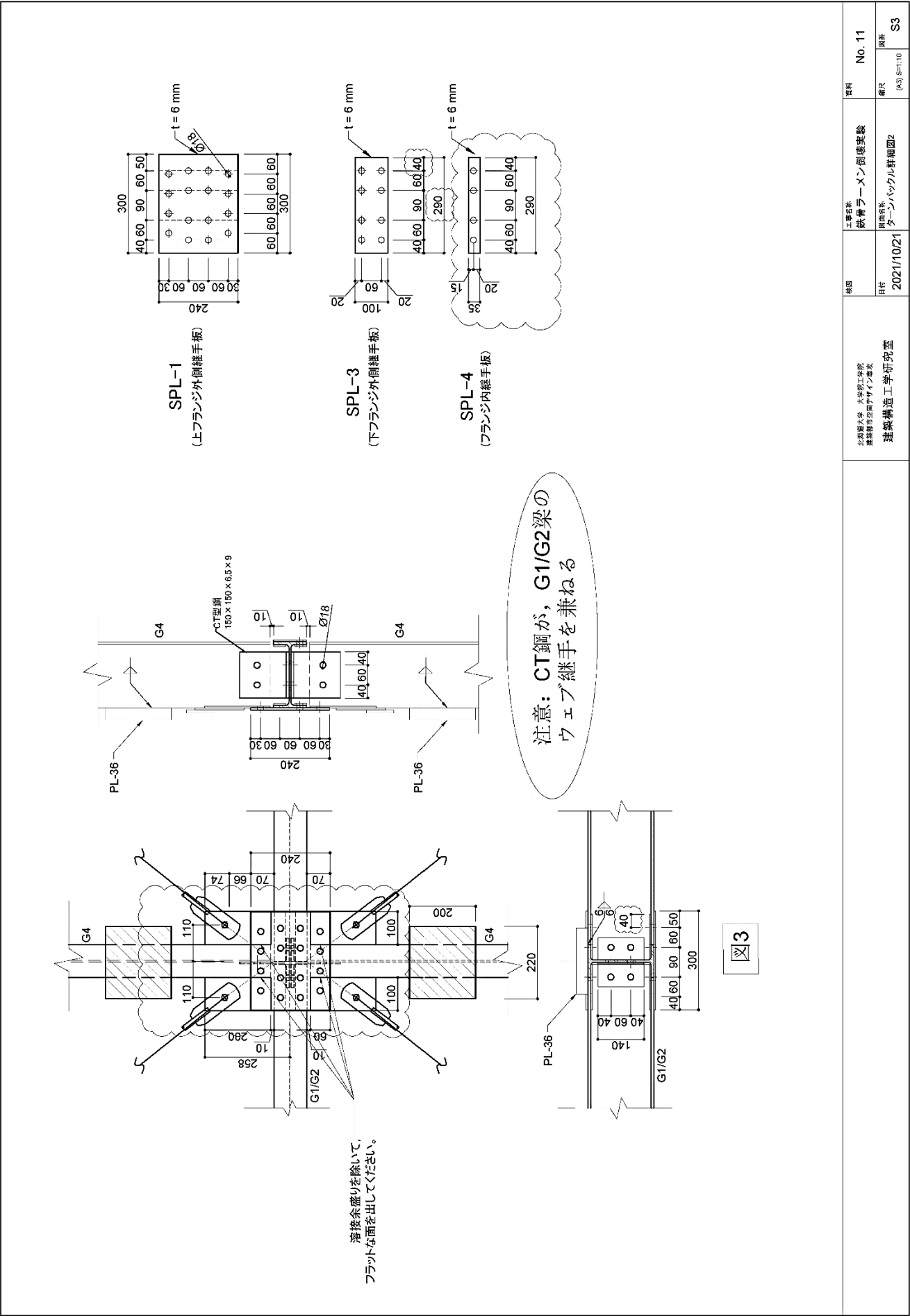




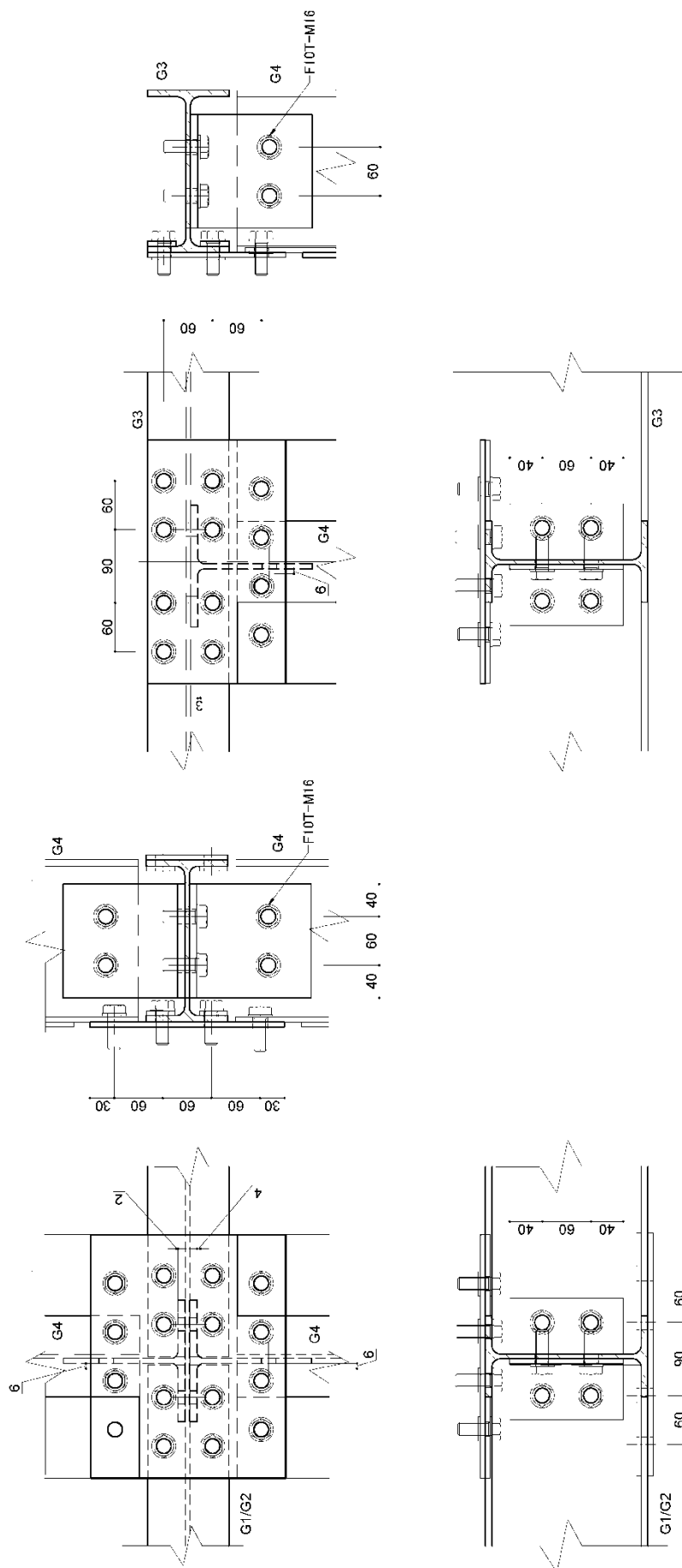
北国大学 大学院工学部 建築構造工学研究室	種類 工事名称 鉄骨ラーメン倒壊実験	図名 A/C通り架構図	図番	No. 7
			図尺 (A) S=1.30	
			A6	
2021/10/21		日付		

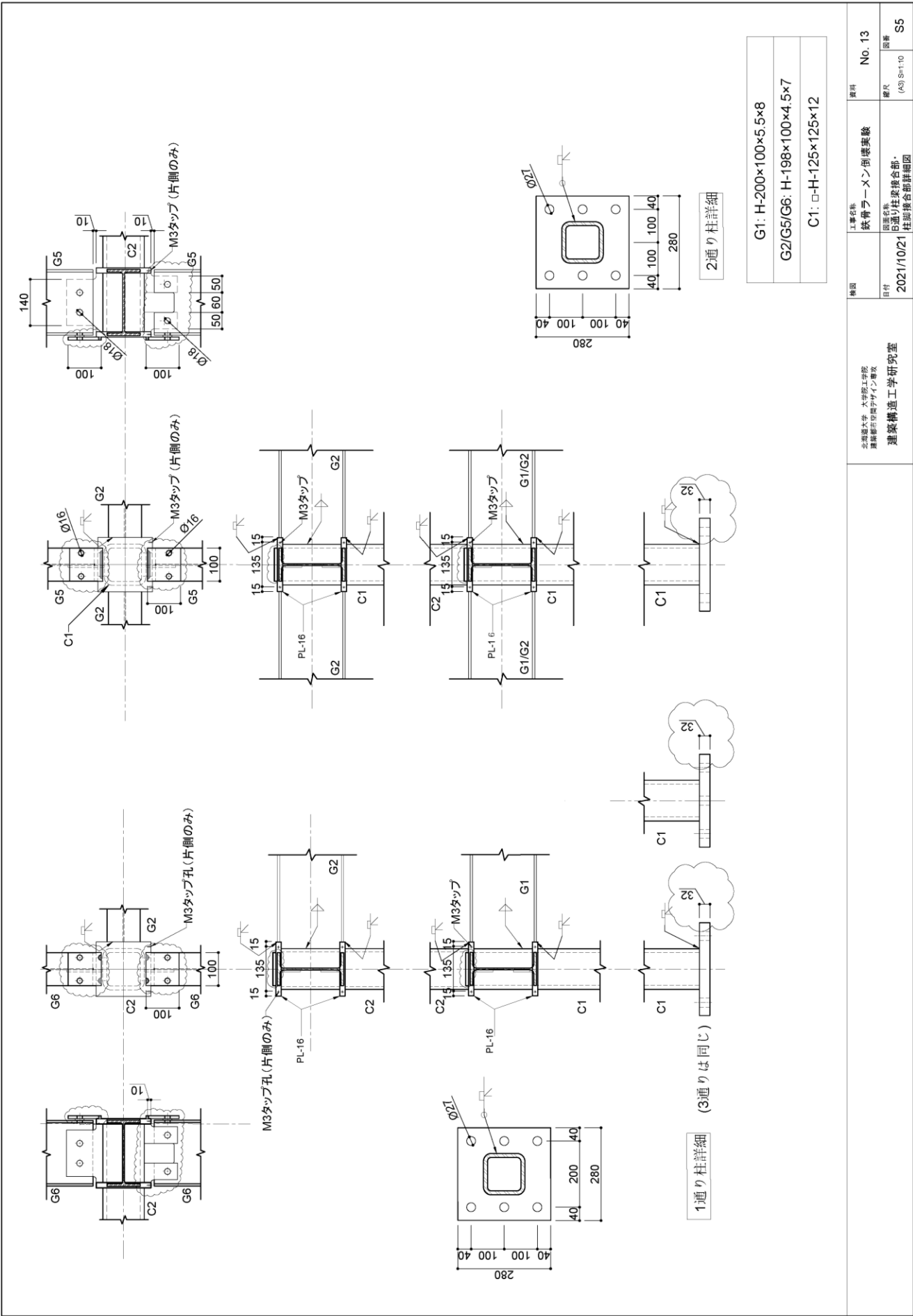


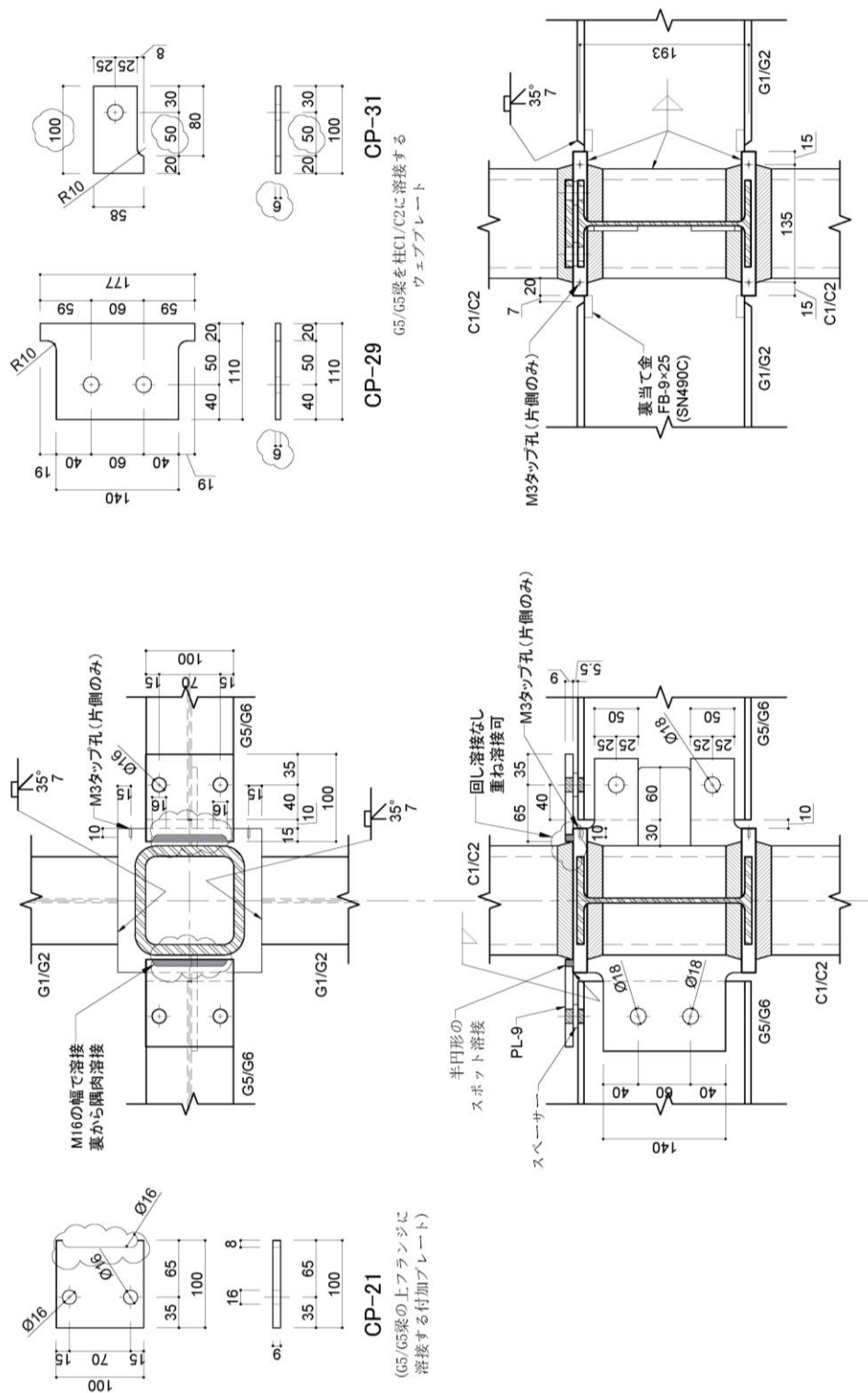




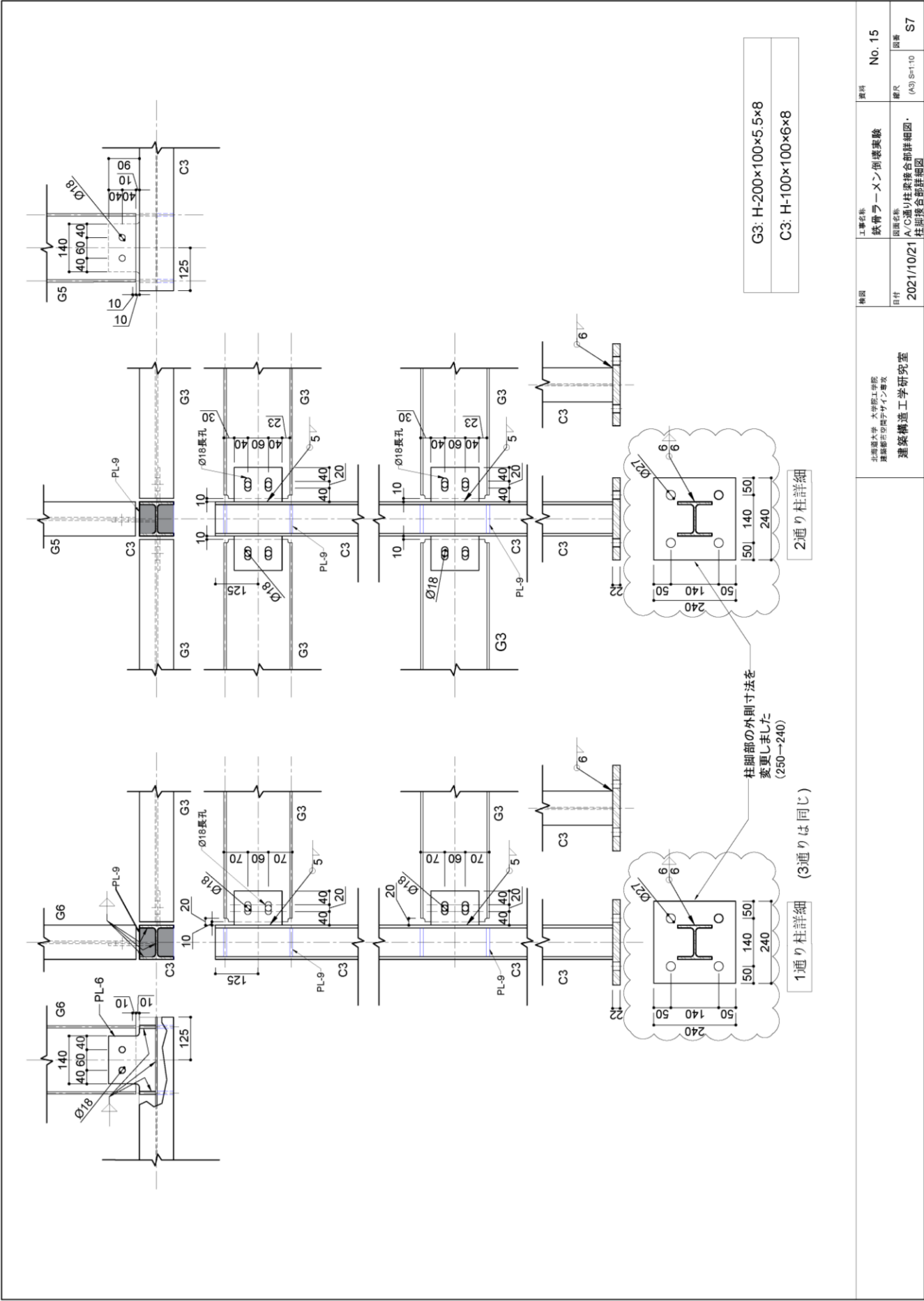
北陽大学 大学院工学部 建築都市環境デザイン専攻 建築構造工学研究室	図面 日付	2021/10/21	図面番号 ターミナル詳細図2	材料 鉄骨ラーメン剛接合部	No. 11

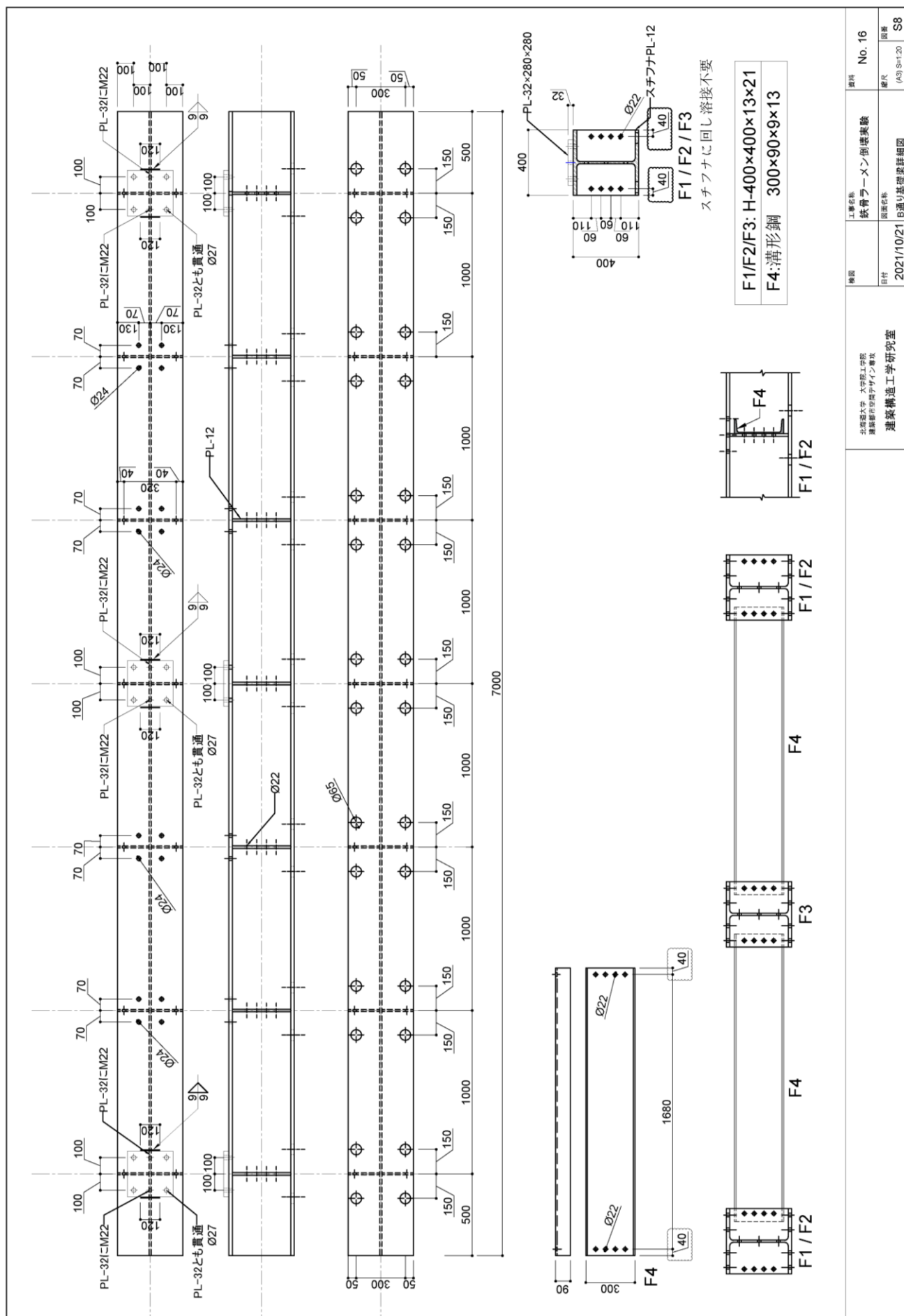
[illegible]

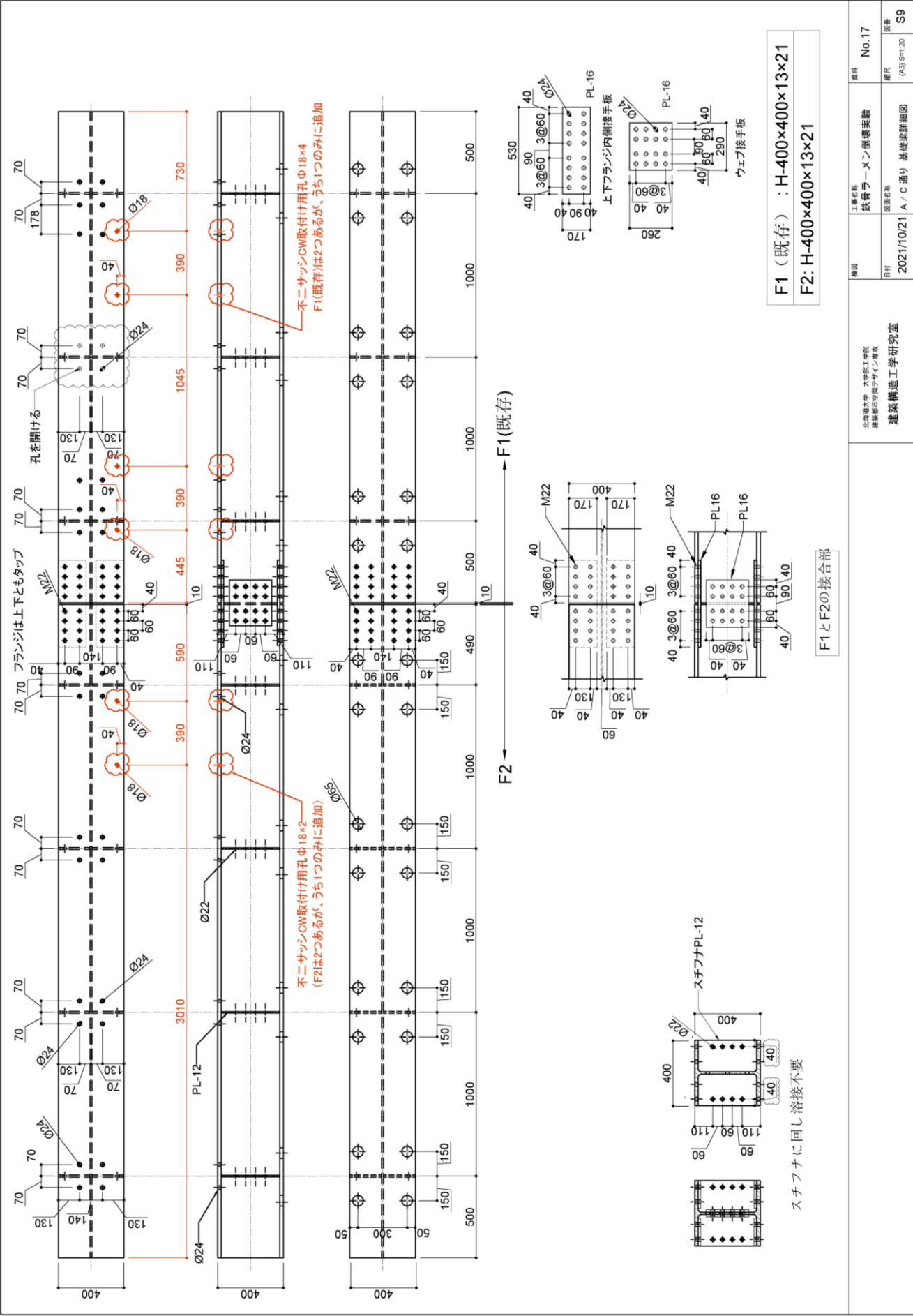


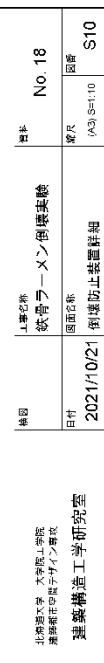


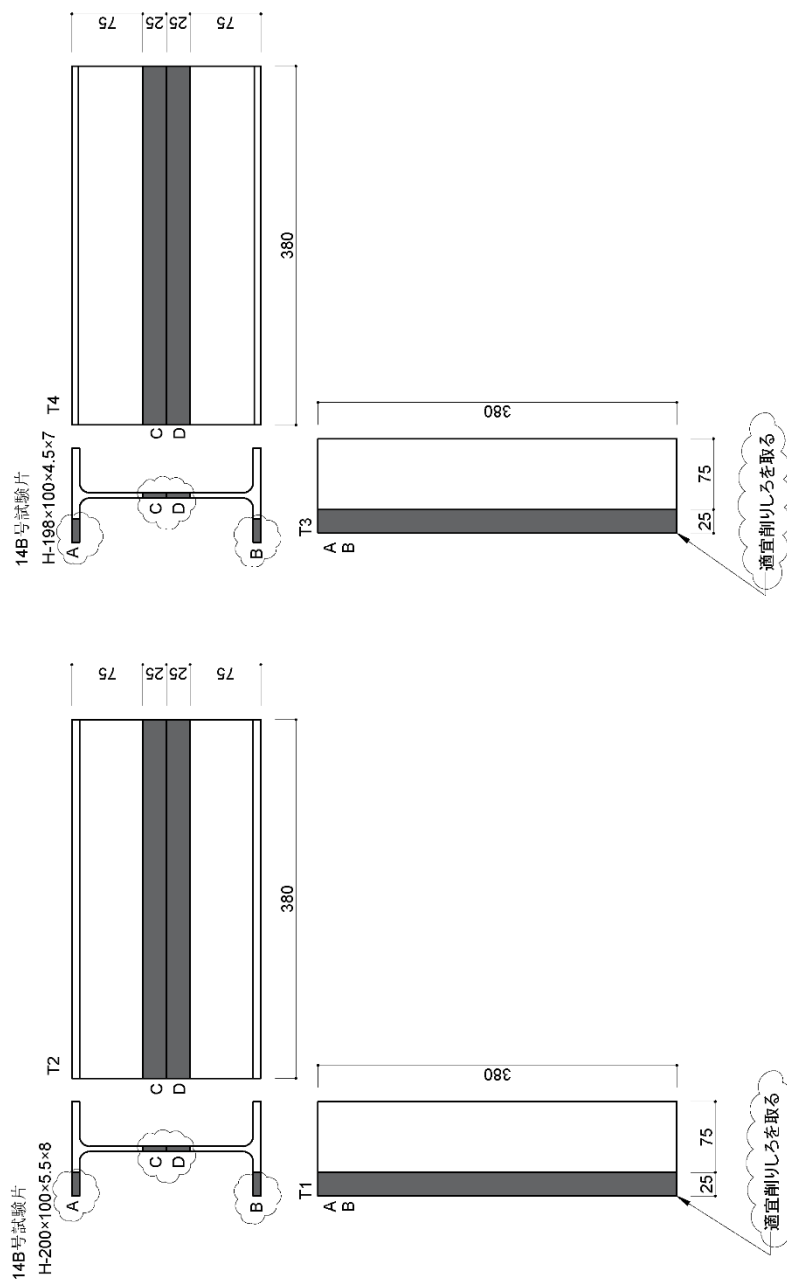
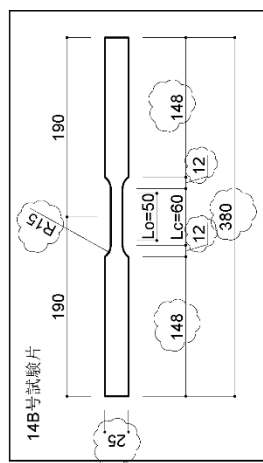
北野建設大学 大学理工学院 建築系建築環境デザイン学専攻 建築構造工学研究室	種類	工書名称 鉄骨ラ－Xハ照壁壁設計	図面
	日付 2021/10/21	図面名称 B棟リフト壁構造軸組断面図	巻数 第1巻



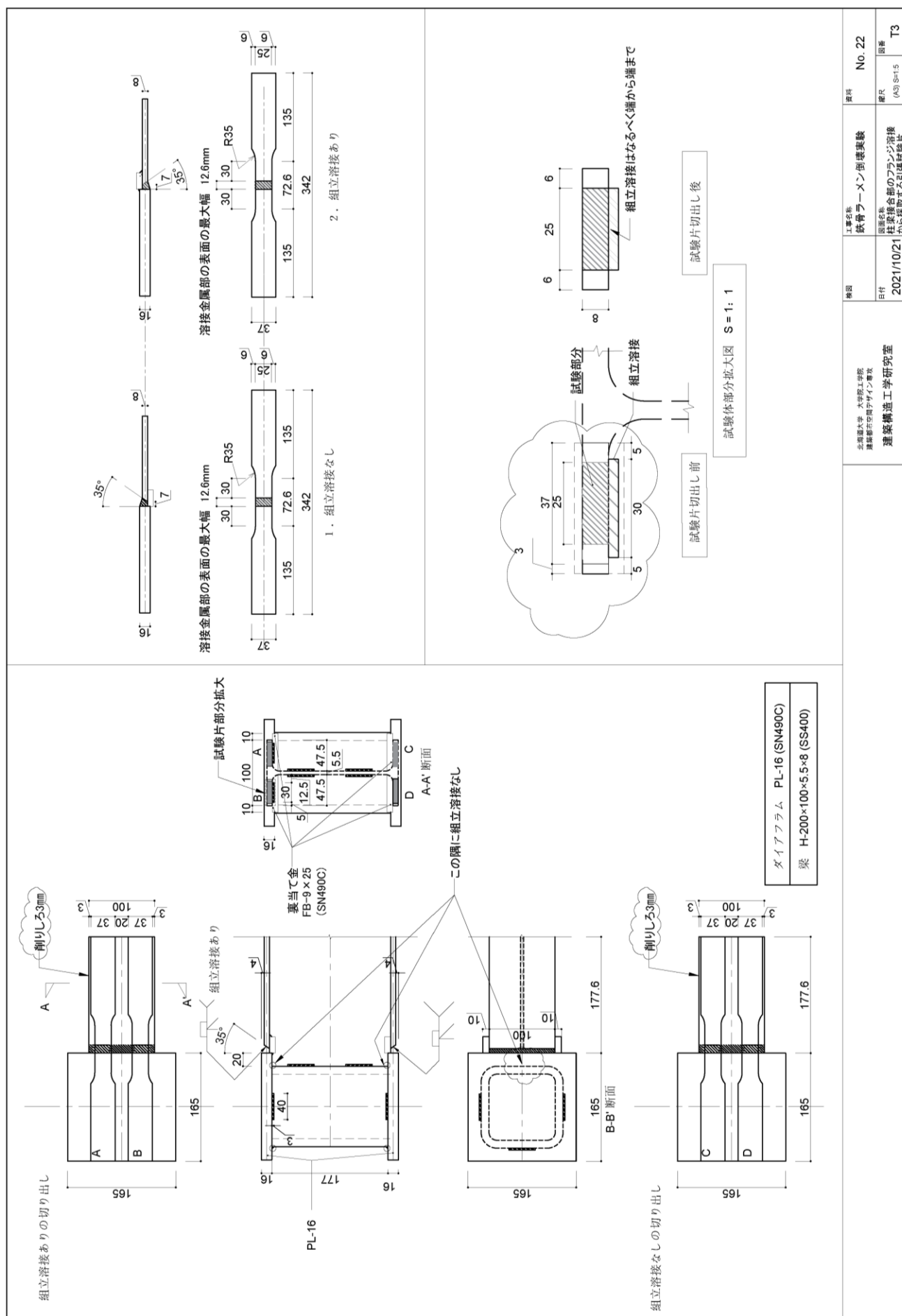




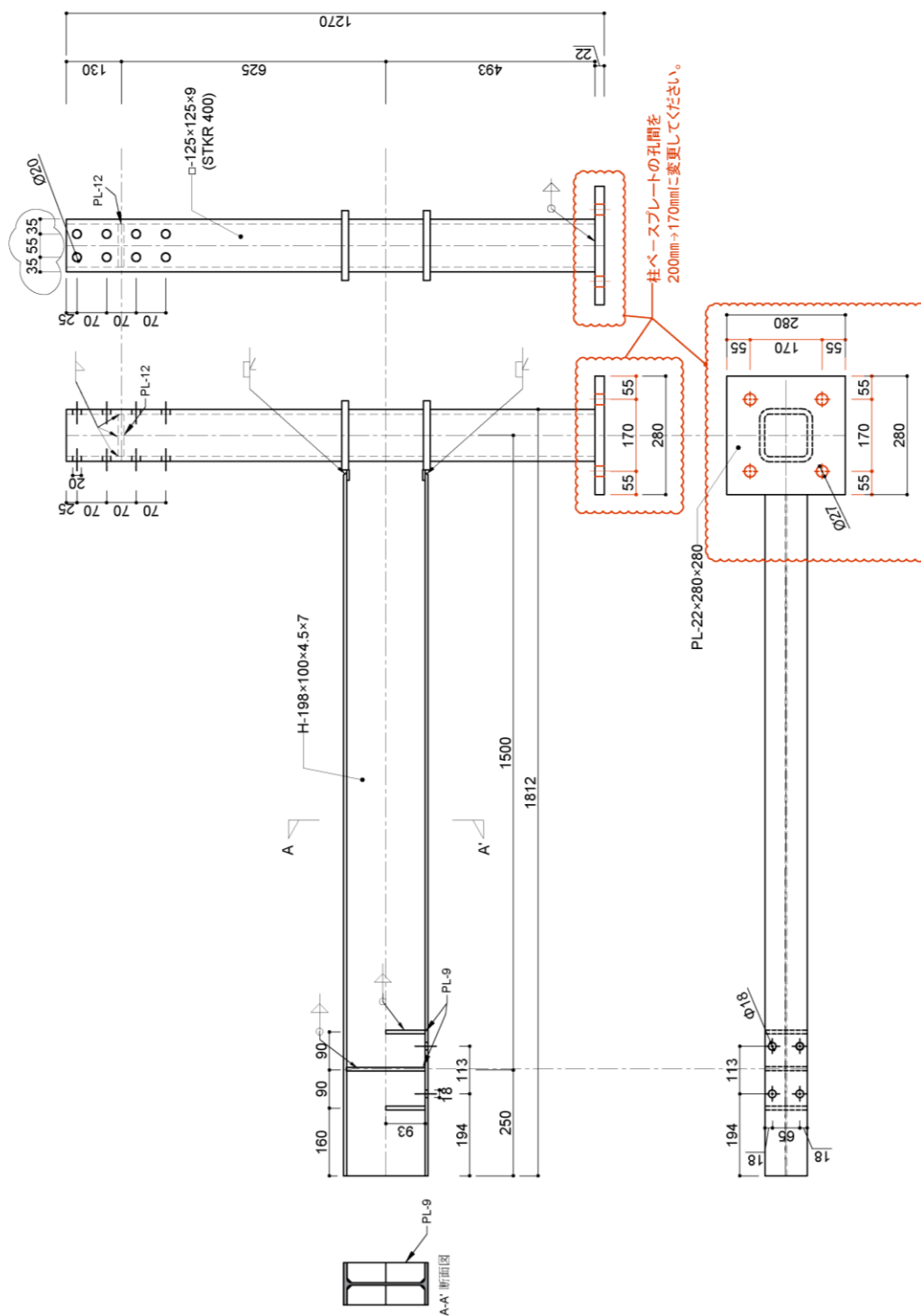




北海道大学 大学院工学院 札幌市青森町サイエンス4 建築・環境工学研究所	校名	工業系 秋田大学	学年	No. 20
	日付	図形 2021/10/21	単位	単位 1
		図形 2021/10/21	単位	単位 1







	北海道大学 大学院工学院 建築都市空間デザイン専攻	竣工名称 鉄骨ラーメン型実験	資料	No. 24
	建築構造工学研究室	図面名義 日付 2021/10/21	版尺 (A) S=10	T5



APPENDIX D: CERTIFIED MILL TEST REPORTS

INSPECTION CERTIFICATE 鋼材検査証明書

GODO STEEL, LTD. OSAKA WORKS
合同製鐵株式会社大阪製造所
1-1-2, HISHIJIMA, HISHIYODOGAWA-KU, OSAKA, JAPAN
大阪市西淀川区西島1丁目1番2号

Contract No. 10529170004
Order's No. GDY1704 573
Supplier 工工エム建材株式会社
Commodity H形鋼
Specification 規格 JIS G 3101 SS400
Customer 需要家 山本鋼業株/志度
Shipper :
Destination 場 港 :
工事名称:

JIS No. JIS3333 証書番号: QAO507002
Ship No. 船番:

Certificate No. 証明書番号: 1020210800724
Date 発行日: 2021/08/06
処理コード: 0805 30406

Size 寸法	Length 長さ	Quantity 員数	Mass 質量 kg	Charge No. 鋼番	Chemical Composition 化学成分 (%)									
					C X100	Si X100	Mn X100	P X1000 Max. 50	S X1000 Max. 50	CE X100				
200X100X5.5/8	12.000m	15	3,765	562407	10	19	55	19	15	23				
合計		15	3,765											
Size 寸法	Charge No. 鋼番	Tensile Test 引張試験 (G1=200mm)				Bend. Test 曲げ試験		Hardness 硬度						
		Y.P. 降伏点 又は0.2%耐力 N/mm ² Min.	T.S. 引張強さ N/mm ² 400 - 510	EL. 伸び %	R.A. 絞り %	Angle °								
200X100X5.5/8	562407	355	448	32										

本証明書は○印の本数のみ有効です

令和3年9月28日

香川県さぬき市志度

山本鋼業株式会社

岩本健太

CE: C14m/6+1/24H1/40HC/r/5+H0/4+V/14

Leader of Quality Control Group
品質管理グループリーダー
Shinpei Higashita
道下 天輔

WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT.
上記注文書は御指定の規格または仕様に従って製造され、その要求事項を満足していることを証明します。

Surveyor to

鋼材検査証明書

INSPECTION CERTIFICATE

GODO STEEL, LTD. OSAKA WORKS
合同製鐵株式会社大阪製造所
 1-1-2, NISHIJIMA, NISHIYODOGAWA-KU, OSAKA, JAPAN
 大阪市西淀川区西島1丁目1番2号

Certificate No. 証明番号 : 1020210901474
 Date 発行日 : 2021/09/13
 処理コード : 0910 32366

JIS No. JIS証証番号: QA0507002

Ship No. 船番 :

Contract No. 注文No.: 10529190004
 Order's No. 発注番号: GDY1904 578
 Supplier 注文者: 工工工建材株式会社
 Commodity 品名: H形鋼
 Specification 規格: JIS G 3101 SS400
 Customer 需要家: 山本鋼業(株)/志度
 Shipper 船務:
 Destination 工場名称:

Size 寸法	Length 長さ	Quantity 員数	Mass 質量 kg	Charge No. 鋼番	Chemical Composition 化学成分 (%)									
					C X100	Si X100	Mn X100	P X1000 Max. 50	S X1000 Max. 50	CE X100				
200X100X5.5/8	8.000m	9	1,503	592055	8	20	54	28	23	21				
200X100X5.5/8	8.000m	1	167	592056	10	19	55	28	23	23				
200X100X5.5/8	9.000m	15	2,820	592047	9	18	53	24	23	22				
200X100X5.5/8	11.000m	10	2,300	592056	10	19	55	28	23	23				
200X100X5.5/8	12.000m	40	10,040	582048	10	18	52	27	19	23				
200X100X5.5/8	12.000m	5	1,255	592030	8	19	54	24	20	21				
合計		80	18,085											
Size 寸法	Charge No. 鋼番	Tensile Test 引張試験 (GL=200mm)			Hardness 硬度			Impact Test 衝撃試験						
		Y.P. 降伏点 N/mm ² Min. 245	T.S. 引張強さ N/mm ² 400 - 510	El.伸び % Min. 17	Brinell HBS	Rockwell HRC	Angle °	Charpy J	Drop mm	Direction °				
200X100X5.5/8	592055	362	457	30										
200X100X5.5/8	592056	370	471	28										
200X100X5.5/8	592047	362	457	28										
200X100X5.5/8	592056	370	471	28										
200X100X5.5/8	582048	365	472	28										
200X100X5.5/8	592030	354	451	29										

本証明書は○印の本数のみ有効です

令和3年9月28日

香川県さぬき市志度5丁目5番5号

山本鋼業株式会社

岩本健太

CE: C-Mn/61Si/24Ni/40+Cr/5+Mo/4+V/14

Leader of Quality Control Group
 品質管理グループリーダー
 Deliauke Nishishita
 道下 大輔
 G0093000R002A01

WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT
 上記注文品は御指定の規格または仕様に従って製造され、その要求事項を満足していることを証明します。

INSPECTION CERTIFICATE 鋼材検査証明書

GODO STEEL, LTD. OSAKA WORKS
合同製鐵株式会社大阪製造所
1-1-2, NISHIJIMA, NISHIYODOGAWA-KU, OSAKA, JAPAN
大阪市西淀川区西島1丁目1番2号

Contract No. 10529180003
Order's No. GY1803 528
Supplier 工工工鋼材株式会社
Commodity 品名: H形鋼
Specification 規格: JIS G 3101 SS400
Customer 需要家: 山本鋼業㈱/志度
Shipper 船 港:
Destination 工事名称:

JIS No. JIS証証番号: QA0507002
Ship No. 船番:

Certificate No. 証明番番号: 1020210302034
Date 発行日: 2021/09/16
処理コード: 0915 32660

Size 寸法 mm	Length 長さ mm	Quantity 員数	Mass 質量 kg	Charge No. 鋼番	Chemical Composition 化学成分 (%)						Tensile Test 引張試験 (GL=50mm) Y.P. 降伏点 又は0.2%R.P. N/mm ² Min. 245	Elongation 伸長率 % Min. 21	Bend Test 曲げ試験 Angle °	Hardness 硬度	Impact Test 衝撃試験
					C x100	Si x100	Mn x100	P x1000 Max. 50	S x1000 Max. 50	CE x100					
198X99X4.5/7	12,000mm	② 10	2,140	592025	8	19	56	21	21	23					
合計		10	2,140												
Size 寸法 mm	Charge No. 鋼番	Tensile Test 引張試験 (GL=50mm) Y.P. 降伏点 又は0.2%R.P. N/mm ² Min. 245													
198X99X4.5/7	592025	377	459	38											

本証明書は〇印の本数のみ有効です

令和3年9月28日

香川県さぬき市志度5416

山本鋼業株式会社

岩本健太

CE: C:Mn/6+Si/24+Ni/40+Cr/5+Mo/4+V/14

Leader of Quality Control Group
品質管理グループリーダー

Osaka Works
通下 大輔

WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT.
上記の文書は御指定の規格または仕様に従って製造され、その要求事項を満足していることを証明します。

Signature to

CG09 300F000001

INSPECTION CERTIFICATE 鋼材検査証明書

GODO STEEL, LTD. OSAKA WORKS
合同製鐵株式会社大阪製造所
1-1-2, NISHIJIMA, NISHIYODOGAWA-KU, OSAKA, JAPAN
大阪市西淀川区西島1丁目1番2号

Contract No. 注文No.: 10529170004
Order's No. 社経号: GDY1704 5ZB
Supplier 工工工建材株式会社
Commodity 品名: H形鋼
Specification 規格: JIS G 3101 SS400
Customer 需要家: 山本鋼業(株)/志度
Shipper 船主:
Destination 港:
工庫名称:

JIS No. JIS認証番号: QA0507002

Certificate No. 証明番号: 1020210800930

Date 発行日: 2021/08/02

処理コード: 0730 30101

Ship No. 船番:

S i z e 寸 法 mm	Length 長 さ mm	Quantity 員 数	Mass 質 量 kg	Charge No. 鋼 番	Chemical Composition 化 学 成 分 (%)							Tensile Test 引 張 試 験 (d L = 5 0 m m)				Bend Test 曲 げ 試 験	Impact Test 撞 撃 試 験					
					C X100	Si X100	Mn X100	P X1000 Max. 50	S X1000 Max. 50	OE X100									Y.P. N/mm ² Min. 245	T.S. N/mm ² 400 - 510	EL. 伸び % Min. 21	R.A. 絞り % Min. 21
198X99X4.5/7	7.000mm	20	2,500	572375	8	19	53	27	21	21												
198X99X4.5/7	8.000mm	25	3,550	572372	8	18	52	24	20	21												
198X99X4.5/7	9.000mm	15	2,400	562396	8	19	56	19	19	22												
198X99X4.5/7	9.000mm	15	2,400	562397	8	19	53	22	19	21												
198X99X4.5/7	10.000mm	5	890	552446	8	21	54	22	23	21												
198X99X4.5/7	10.000mm	25	4,450	562393	8	20	53	25	17	23												
198X99X4.5/7	11.000mm	25	4,900	572371	8	19	54	21	23	21												
198X99X4.5/7	12.000mm	20	4,280	572375	8	19	53	27	21	21												
合計		150	25,370																			
S i z e 寸 法 mm	Charge No. 鋼 番	本証明書は○印の本数のみ有効です																			令和3年7月28日 香川県さぬき市志度5-1-5 山本鋼業株式会社 岩本健太	
198X99X4.5/7	572375																				品質管理グループリーダー 志度 大輔 D. Muro 009300FR002A01	
198X99X4.5/7	572372																					
198X99X4.5/7	562396																					
198X99X4.5/7	562397																					
198X99X4.5/7	552446																					
198X99X4.5/7	562393																					
198X99X4.5/7	572371																					
198X99X4.5/7	572375																					

CE: CHMn/6+Si/24+Ni/40+Cr/54Mo/4+V/14

WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT.
上記注文書は御指定の規格または仕様に従って製造され、その要求事項を満足していることを証明します。

Surveyor 10

INSPECTION CERTIFICATE
鋼材検査証明書

GODO STEEL, LTD. OSAKA WORKS
合同製鐵株式会社 大阪製造所
1-1-2, NISHIJIMA, NISHIYODOGAWA-KU, OSAKA, JAPAN
大阪市西淀川区西島1丁目1番2号

JIS No. JIS鋼種番号: Q4507002

Certificate No. 証明書番号: 1020210900776
Date 発行日: 2021/09/07
処理コード: 0906 32115

Ship No. 船番:

Contract No. 注文No.: 10529180003
Order's No. 発注品番: GDY1803 528
Supplier 注文者: 工入工入建材株式会社
Commodity 品名: H形鋼
Specification 規格: JIS G 3101 SS400
Customer 需要家: 山本鋼業(株)/志度
Shipper 船主:
Destination 工事名称:

S i z e 寸 法 mm	Length 長 さ	Quantity 員 数	Mass 質 量 kg	Charge No. 鋼 番	Chemical Composition 化 学 成 分 (%)						Tensile Test 引 張 試 験 (G=50mm) T.S. 引張強さ N/mm ² Min. 245	Bend Test 曲 げ 試 験 R.A. 絞り % Min. 21	Hardness 硬 度 Angle °	Impact Test 沖 撃 試 験
					C X100	Si X100	Mn X100	P X1000 Max. 50	S X1000 Max. 50	CE X100				
198X99X4.5/7	7.000m	25	3,125	572372	8	18	52	24	20	21	115	450	37	
198X99X4.5/7	8.000m	15	2,130	582419	7	19	55	23	20	20				
198X99X4.5/7	8.000m	5	710	582420	7	20	54	21	19	19				
198X99X4.5/7	9.000m	30	4,800	562394	9	20	55	21	20	23				
198X99X4.5/7	10.000m	10	1,780	562397	8	19	53	22	19	21				
198X99X4.5/7	11.000m	8	1,568	592024	8	19	58	21	20	22				
198X99X4.5/7	11.000m	2	392	592025	8	19	56	21	21	23				
198X99X4.5/7	12.000m	20	4,280	592026	8	20	55	23	19	22				
合計		115	18,785											
S i z e 寸 法 mm	Charge No. 鋼 番	Tensile Test 引 張 試 験 (G=50mm)		Bend Test 曲 げ 試 験		Hardness 硬 度		Impact Test 沖 撃 試 験						
		T.S. 引張強さ N/mm ²	EL. 伸び %	R.A. 絞り %	Rockwell HRC	Brinell HBS	Charpy J	Drop kg						
198X99X4.5/7	572372	391	450	37										
198X99X4.5/7	582419	390	460	37										
198X99X4.5/7	582420	376	455	36										
198X99X4.5/7	562394	368	449	40										
198X99X4.5/7	562397	386	457	39										
198X99X4.5/7	592024	386	464	37										
198X99X4.5/7	592025	377	459	38										
198X99X4.5/7	592026	388	466	35										

本証明書は○印の本数のみ有効です

令和3年7月28日
香川県さぬき市志度544-6
山本鋼業株式会社
岩本健太

本証明書は○印の本数のみ有効です

令和3年9月28日

香川県さぬき市志度544-16

山本鋼業株式会社

岩本健太

CE: C-Mn/6+Si/24+Ni/40+C+75+Mo/4+V/14

WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT.
上記注文書は御指定の規格または仕様に従って製造され、その要求事項を満足していることを証明します。

Supervisor No.

Leader of Quality Control Group
品質管理グループリーダー
D. Nishikawa
志度 大輔
0909300FR002401

[illegible]

鋼材検査証明書
INSPECTION CERTIFICATE

東京鐵道株式會社

TOKYO STEEL MFG. CO., LTD.

〒100-0013 東京都千代田区麹町3丁目7番1号 麹が関東ビル(15階)
 Telex Office: Kasunipasek, Tokyo, 115F, 3-7-1 Kasunipasek, Chiyoda-ku, Tokyo, 100-0013, Japan

收錄卷二第

206-207

證明書番号 Certificate No. 130346

注文者 小野建設株式会社 店員支店
Shipper

Customer
Customer

品名	Commodity
H.I.	

工 程 名 称	晓 松
Project Name	晓 松
工 程 编 号	6310122400
Project No.	6310122400

和合樓大什貨

寸法 Size	長さ Length	数量 Quantity			鋼番 Charge No.	備考 Remark
		束数 Bundles	員数 Pieces	総員数 Total Pieces		
H300X150X6.5/9	12.0M	3	15	6500	N1-1749	
S-TOTAL		3	15	6500		
G-TOTAL				12480		

引張試験 Tensile Test		衝撃試験 Impact Test		透過率試験 Through-thickness Characteristics	
試験方法 Test Method	試験結果 Test Result	試験方法 Test Method	試験結果 Test Result	試験方法 Test Method	試験結果 Test Result
引張試験 Tensile Test	引張強さ Tensile Strength N/mm ² 400 MIN-510	衝撃試験 Impact Test	衝撃強さ Impact Strength J 17 MIN	透過率試験 Through-thickness Characteristics	透過率 Through-thickness Reduction of Area % 1 2 3 Ave 1 2 3 Ave 1 2 3 Ave

化学成分 Chemical Composition		機械的性質 Mechanical Properties	
C	Si	Mn	P
0.24	0.035	0.035	0.005
S		S	
0.005	0.005	0.005	0.005

引張試験 Tensile Test		衝撃試験 Impact Test	
試験方法 Test Method	試験結果 Test Result	試験方法 Test Method	試験結果 Test Result
引張試験 Tensile Test	引張強さ Tensile Strength N/mm ² 400 MIN-510	衝撃試験 Impact Test	衝撃強さ Impact Strength J 17 MIN

検査証明書

下記注文品は検査の結果御指定の規格に合格したことを証明します。



日鉄建材株式会社

広畑製造所 管理グループ長 遠藤 誠

(1) 証明書番号

0-00993 001

(2) シリアルナンバー

東京バイパス興業株式会社

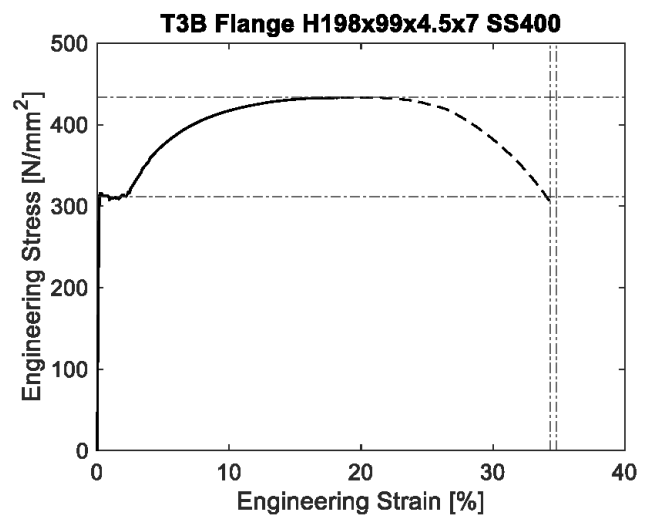
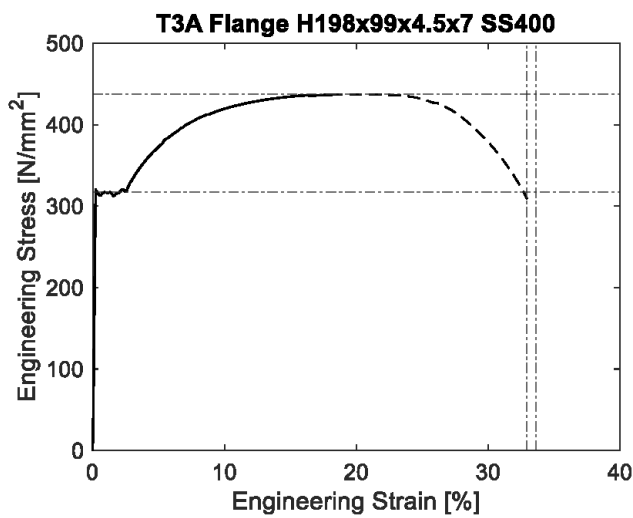
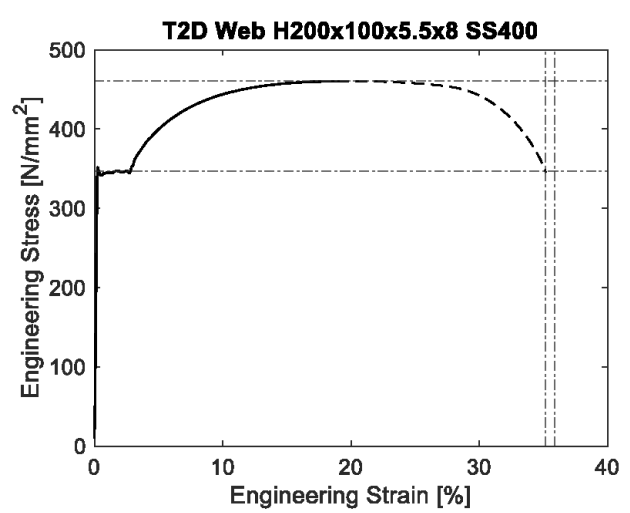
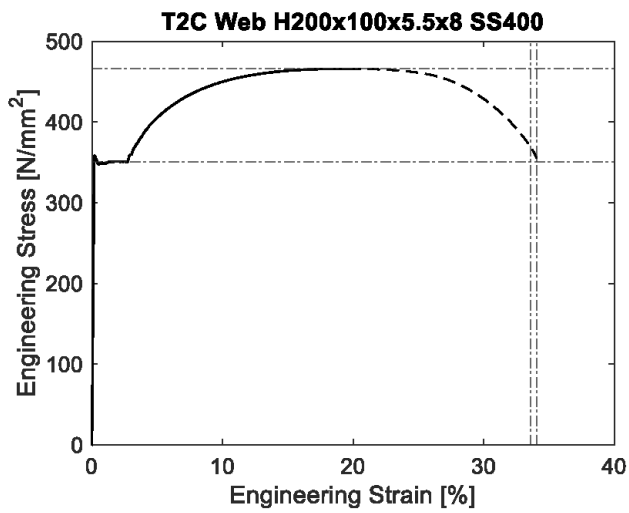
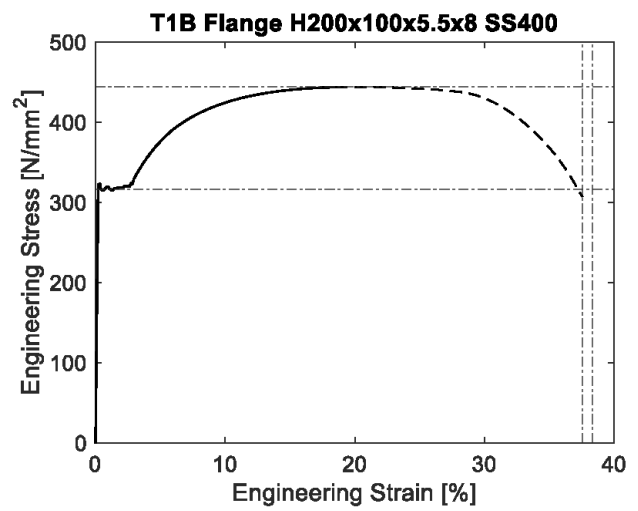
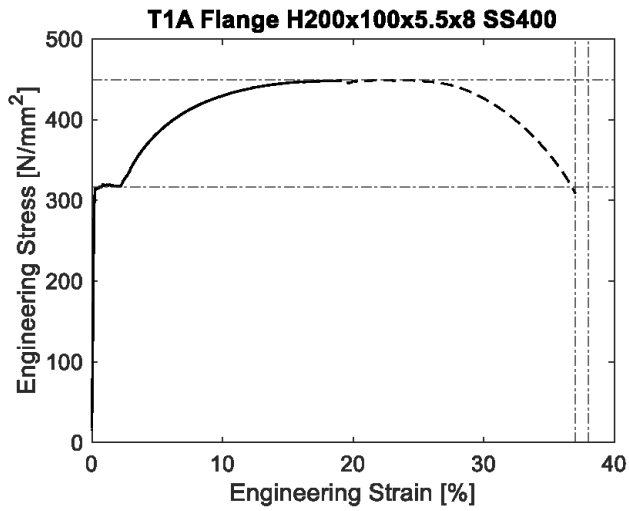
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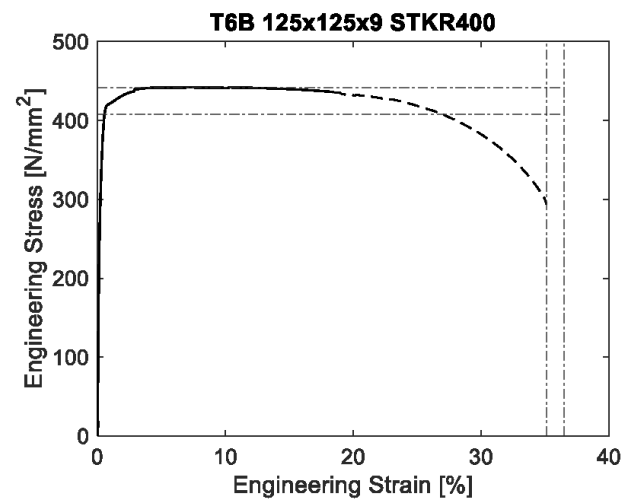
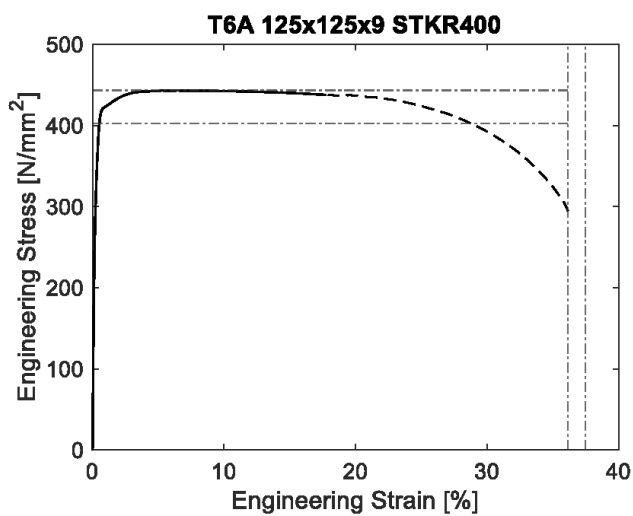
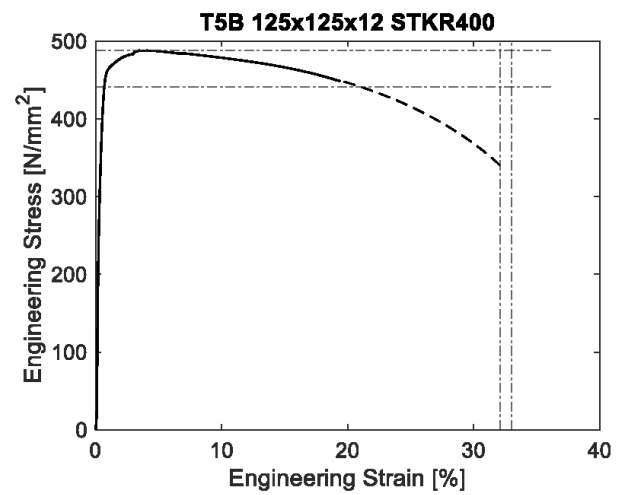
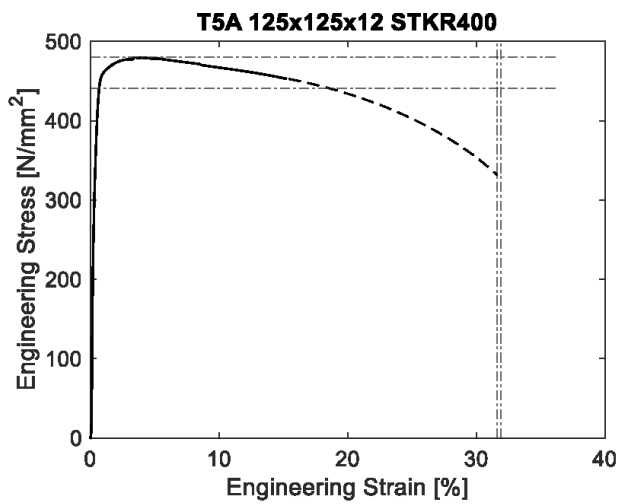
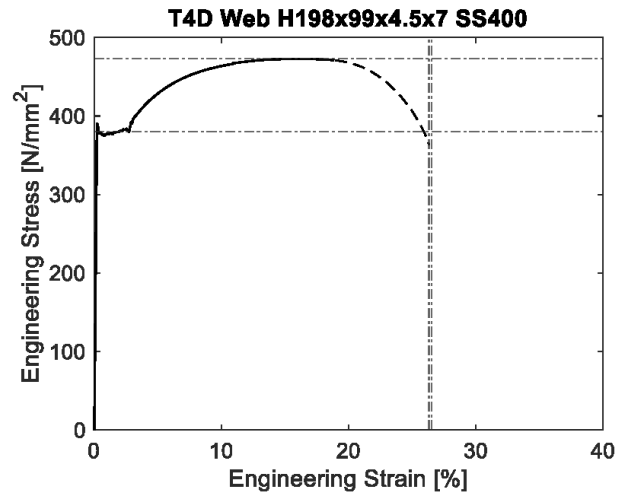
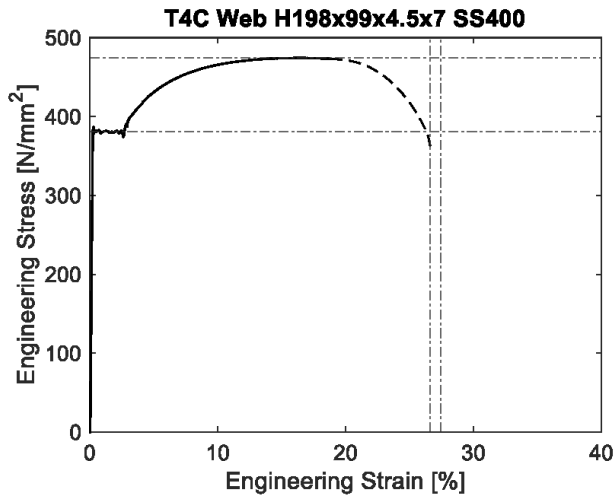
2021-10-15

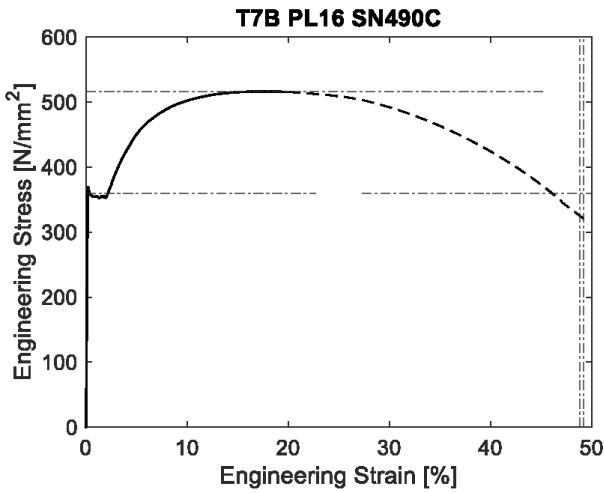
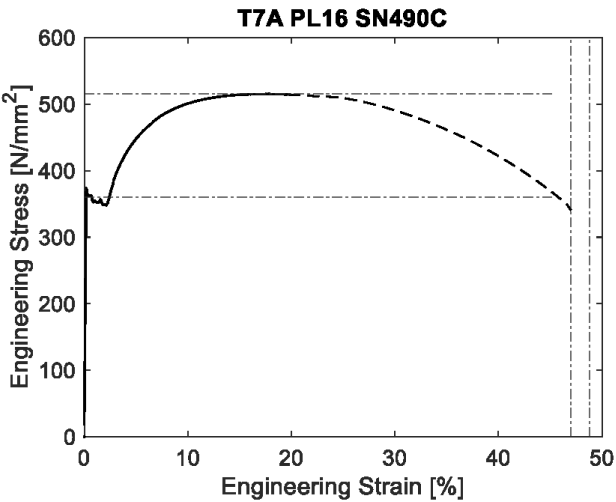
2021-10-14

1-K2-A22-1-9-SA56		日鉄物産株		御中		492		017648		角形鋼管																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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APPENDIX E: TENSION TESTS RESULTS







APPENDIX F: CERTIFIED WELD INSPECTION REPORT

超音波探傷検査 報告書

(自主検査)

(北海道大学) 鉄骨ラーメン倒壊実験

(名古屋大学) 高層建物骨組の地震応答再現実験 モニタリング技術試験

工事名： (名古屋大学) 高層建物骨組の地震応答再現実験 非構造部材試験

遠藤工業株式会社

検査技術者 服部 希



2021 年 11 月 20 日

目 次

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1	はじめに	2
2	試験箇所	3
3	斜角探傷	3
4	欠陥の評価	4
5	合否の判定	5
6	合否判定表	6
7	試験記録記号説明	7
8	超音波探傷試験箇所図	8～9
9	超音波探傷試験条件表	10
10	キーププラン	11
11	超音波探傷結果集計表	12～13
12	超音波探傷試験結果	14
13	非破壊検査技術者技量認定証明書(写し)	15

1. はじめに

本記録は、下記工事の完全溶け込み溶接部の超音波探傷試験について報告するものです。

鉄骨ラーメン倒壊実験（北海道大学）

【中間検査】

1) 試験期日 自 2021 年 11 月 9 日
 至 2021 年 11 月 17 日

2) 適用規格 日本建築学会 2018
 「鋼構造建築溶接部の超音波探傷検査基準・同解説」

3) 合否の判定
 疲労を考慮しない溶接部 — 引張応力が作用する溶接部（表 3）

3) 試験技術者

氏 名	登録番号
-----	------

福地 敏博（㈱ニチゾウテック）	N10080598 UT ベル2
-----------------	------------------

濱田 潤一（㈱ニチゾウテック）	N10182335 UT ベル2
-----------------	------------------

2. 試験箇所

工場施工の完全溶け込み溶接部の内超音波探傷試験箇所図に示す箇所とする。
(8～9頁 参照)

3. 斜角探傷

1) 探傷面及び使用屈折角

継手形状	板厚	探傷面	探傷の方法	使用屈折角
T継手及び 角継手	75mm以下	両面片側	直射法及び 1回反射法	70度
	75mmを超えるもの	両面片側	直射法及び 1回反射法	70度又は45度

2) 走査範囲

0 から 1.0 (スキップ) の範囲をジグザグ走査により探傷する。

3) 探傷感度

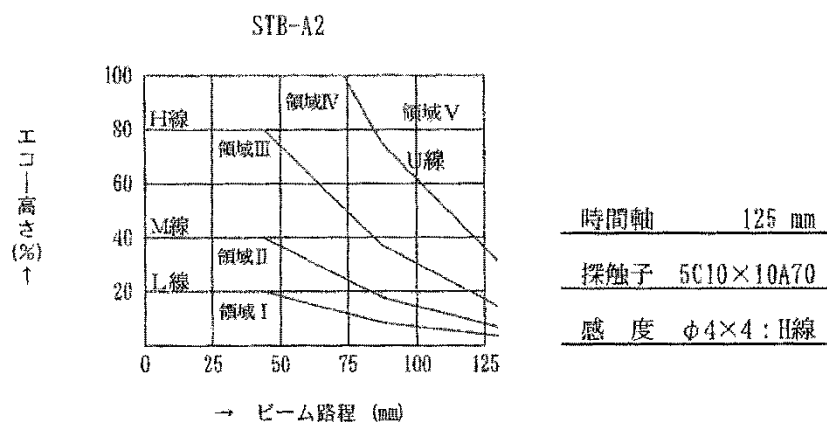
ア STB-A2による場合、

屈折角 70度を使用する場合 $\phi 4 \times 4\text{mm}$ の標準のエコ-高さが H線、屈折角 45度を使用する場合 U線に合うようにゲイン調整し、これを探傷感度とする。

イ RB-4による場合、

標準穴のエコ-高さ H線に合うようにゲイン調整し、これを探傷感度とする。

4) 距離振幅特性曲線



4. 欠陥の評価

4.1 合否の判定の対象とする欠陥

合否の判定の対象とする欠陥は、欠陥指示長さが突き合わせる母材の板厚に応じて表1に示す値以上の欠陥とする。ただし、板厚が異なる突き合わせ溶接の場合は母材の薄い方の板厚とする。

表1. 欠陥指示長さの最小値(単位:mm)

母材の板厚 (mm)	欠陥指示長さ (mm)
6以上20以下	5
20を超え48以下	$t/4$
48を超えるもの	12

4.2 欠陥評価長さ

同一断面内の欠陥群では深さ方向の位置が同一と見なされ、かつ欠陥と欠陥の間隔が長い方の欠陥指示長さ以下の場合は、同一欠陥群と見なす。その欠陥評価長さはそれら欠陥の欠陥指示長さとの和とする。

また、欠陥と欠陥の間隔が長い方の欠陥指示長さを超える場合は、それぞれ独立した欠陥と見なし、その欠陥評価長さはそれぞれの欠陥指示長さとする。

4.3 欠陥評価長さの境界値

突き合わせる母材の板厚に応じて、欠陥評価長さの境界値 S・M・ML・L及びLLは表2に示す値とする。

表2. 欠陥評価長さの境界値(単位:mm)

母材の板厚 (mm)	S	M	ML	L	LL
6以上20以下	10	15	20	30	40
20を超え48以下	$t/2$	$3/4t$	t	$3/2t$	$2t$
48を超えるもの	24	36	48	72	96

5. 合否の判定

5.1 単位溶接線

溶接長さが300mm以上の場合は、欠陥が最も密になるような連続した長さ300mmを、また、溶接線長さが300mm未満の場合には、全長を単位溶接線とする。

5.2 単位溶接線の合否

単位溶接線の合否には、溶接部に作用する応力の種類に応じて、欠陥評価長さ及びE₁₀-高さの領域を用いて判定する。ただし、単位溶接線に複数の欠陥が存在する場合は、欠陥評価長さの総和も考慮して合否の判定を行う。なお、それぞれの欠陥でE₁₀-高さ領域が相違する場合は、そのうち最も高いE₁₀-高さ領域を採用する。

疲労を考慮しない溶接部に引張応力が作用する場合、欠陥のE₁₀-高さ領域に応じて、欠陥評価長さあるいはその総和が、表3に示す境界値以上ある単位溶接線は不合格とする。

表3. 引張応力が作用する溶接部

E ₁₀ -高さの領域		欠陥評価長さ	欠陥評価長さ の 総 和
斜角一探触子法	タンデム探傷法		
II	II	L	L L
III, IV	III	M L	L
V	IV	M	M L

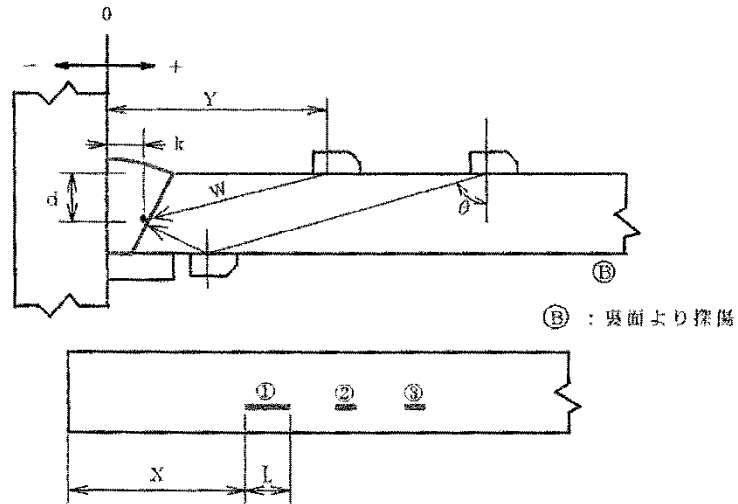
6. 合否判定表

単位：mm

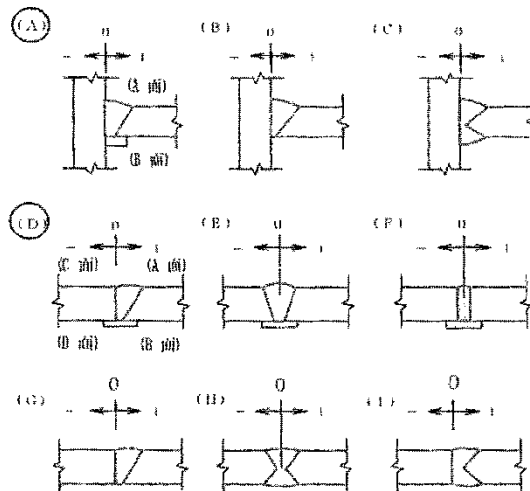
領域 板厚	Ⅱ		Ⅲ・Ⅳ		Ⅴ	
	欠陥 評価 長さ	総和	欠陥 評価 長さ	総和	欠陥 評価 長さ	総和
6 ~ 20	30	40	20	30	15	20
21	32	42	21	32	16	21
22	33	44	22	33	17	22
23	34	46	23	35	18	23
24	35	48	24	36	18	24
25	36	50	25	38	19	25
26	38	52	26	39	20	26
27	39	54	27	41	21	27
28	41	56	28	42	21	28
29	42	58	29	44	22	29
30	44	60	30	45	23	30
31	45	62	31	47	24	31
32	47	64	32	48	24	32
33	48	66	33	50	25	33
34	50	68	34	51	26	34
35	51	70	35	53	27	35
36	53	72	36	54	27	36
37	54	74	37	56	28	37
38	56	76	38	57	29	38
39	57	78	39	59	30	39
40	59	80	40	60	30	40
41	60	82	41	62	31	41
42	62	84	42	63	32	42
43	63	86	43	65	33	43
44	65	88	44	66	33	44
45	66	90	45	68	34	45
46	68	92	46	69	35	46
47	69	94	47	71	36	47
48	71	96	48	72	36	48
49 ~	72	96	48	72	36	48

※ 境界値以上は不合格

7. 試験記録記号説明



開先タイプ



X : 溶接線方向距離 (開先側から見て左から)

Y : 溶接部 0 点一探触子入射点距離

W : ビーム路程

k : 欠陥の直角方向距離

d : 開先面側からの欠陥深さ

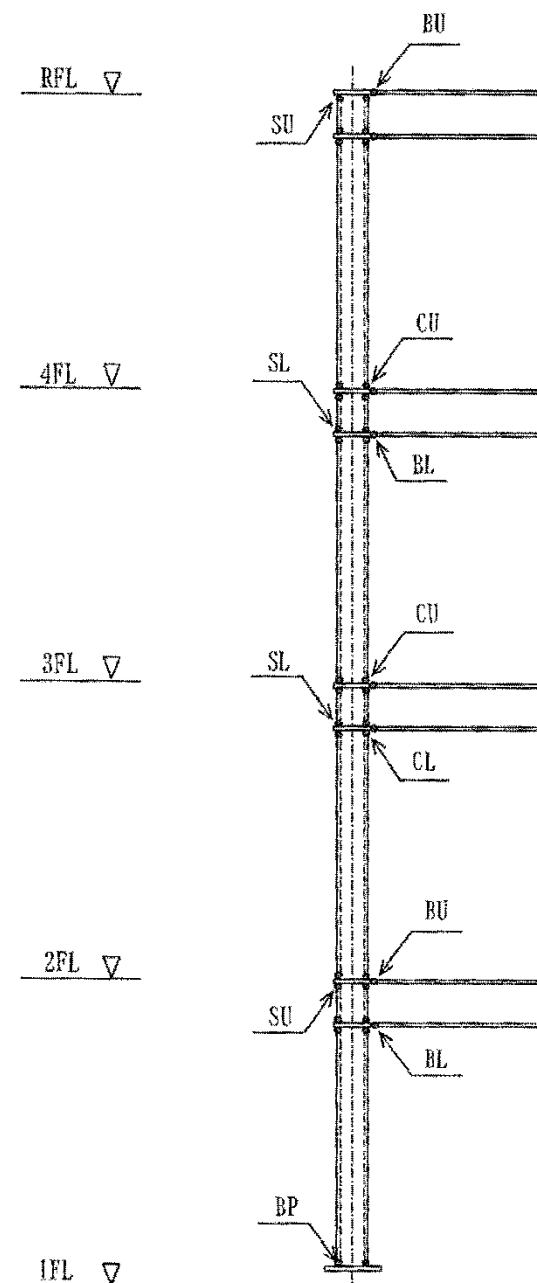
θ : STB 屈折角 (度)

L : 欠陥の指示長さ

①, ②, ③ : 欠陥 No

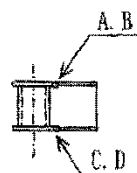
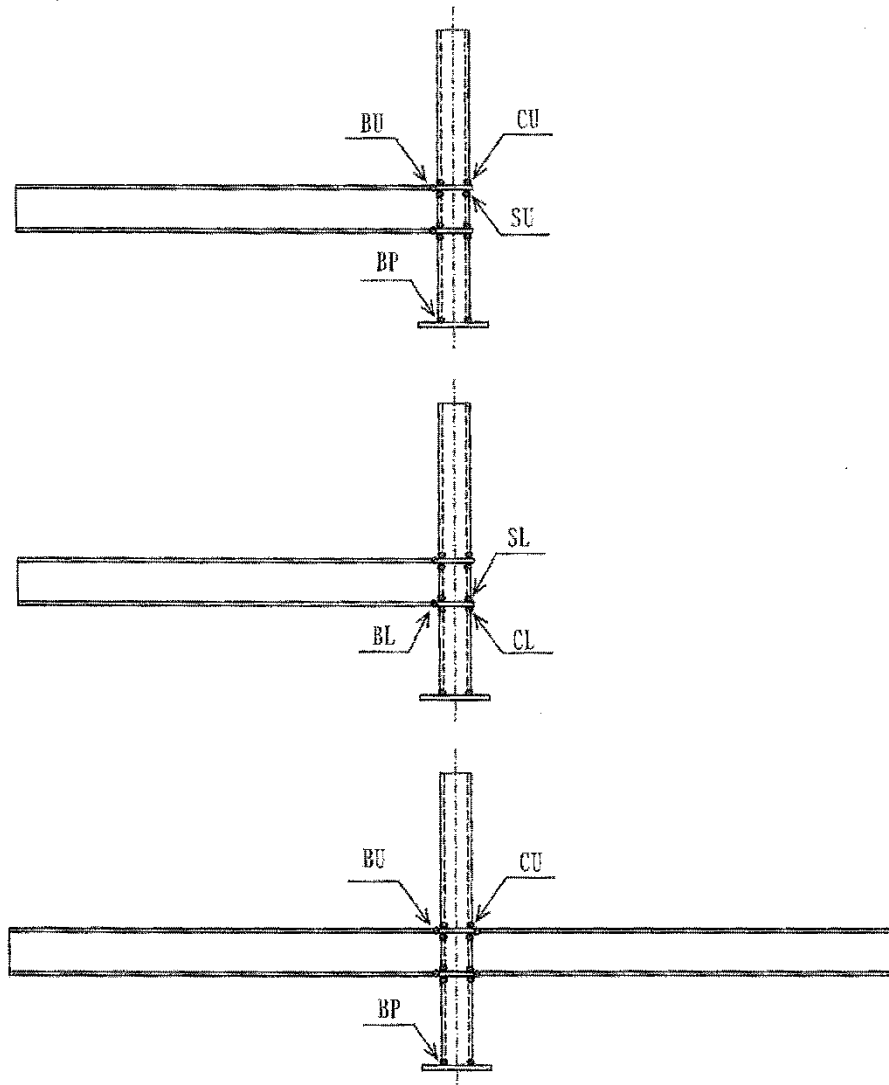
8. 超音波探傷試験箇所図 (例)

柱



8. 超音波探傷試験箇所図 (例)

柱



検査位置の表示例

部材記号	CA-1	3FL	N	CU
		階	方位	部位記号

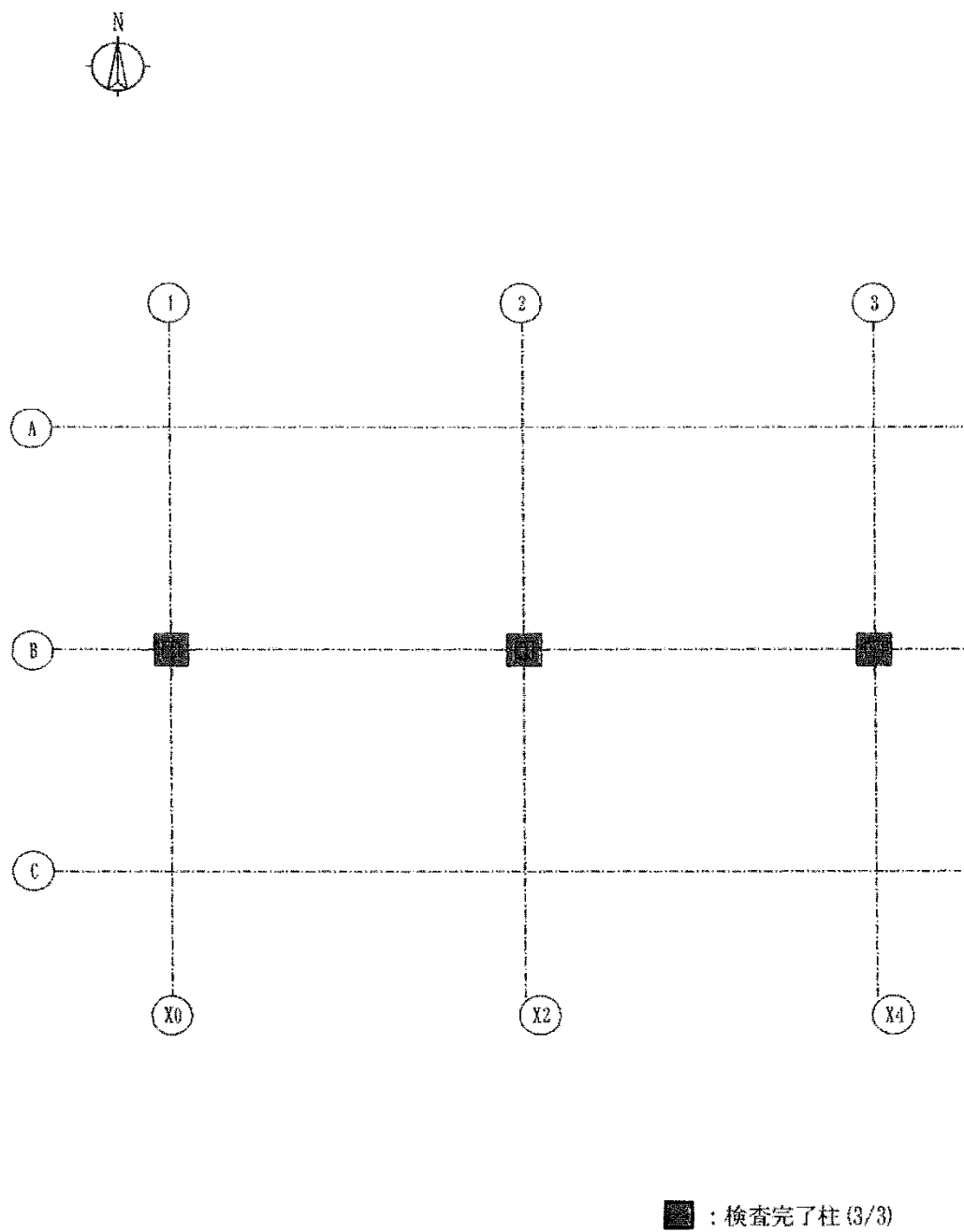
C : 柱
 BP : ベースプレート
 U : 上
 L : 下
 B : ブラケット突合せ継手
 F : ブラケットT継手
 S : スチフナ
 G : 梁突合せ継手

ULTRASONIC EXAMINATION SETUP INFORMATION

9. 超音波探傷試験条件表

工 事 名	鉄骨ラーメン倒壊実験 (北海道大学)	
品 名	柱	
検 査 場 所	遠藤工業株式会社	
検 査 年 月 日	2021年 11月 9日 ~ 2021年 11月 17日	
適 用 規 格	日本建築学会 2018「鋼構造建築溶接部の超音波探傷検査基準・同解説」	
検 査 技 術 者	濱田 潤一 (JSNDI-UT2)	
検 査 技 術 者	福地 敏博 (JSNDI-UT2)	
検 査 技 術 者	-	
使用 機名 探傷器	探 傷 器 名	UI-S9 UI-S7 α
	管 理 番 号	UT-M106 UT-M101
	製 造 者	菱電湘南エレクトロニクス株式会社 菱電湘南エレクトロニクス株式会社
	製 造 番 号	UI19AS9062 UI17ES71373
	点 検 年 月 日	2021.3.18 2021.3.18
	増幅の直線性	+1.0% -0.9% +0.5% -1.1%
	時間軸の直線性	+0.0% -0.1% +0.0% -0.0%
探傷条件	探 傷 方 法	斜角一探触子法 板 厚 7 mm ~ 12mm
	溶 接 方 法	CO2半自動 接 触 媒 質 ミタエコーS (感度補正+3dB)
	探傷面の状態	ロール肌 探 傷 時 期 溶接完了後常温時
	材 質	STKR400, SS400 感 度 試 験 片 STB-A2, STB-A3
	感 度 補 正	-
	判 定 基 準	「疲労を考慮しないで引張応力が作用する溶接部」
使用探触子	斜 角 探 触 子	
	呼 称	5C10×10A70 5C10×10A70
	製造会社名	ジャパンプロブ ジャパンプロブ
	製造番号	A0 1102257 A0 1501042
	STB 屈折角	70.5度 70.0~70.5度
	接近限界距離	11 12
	不 感 帯	0 mm 0 mm
分 解 能	4 mm 4 mm	

10. キープラン



鉄骨ラーメンの倒壊実験

データ No.	探傷器	探触子	実測入射点	STB屈折角	探傷者	X:溶接線方向距離 W:ビーム路程 d:探傷面からの深さ	実測 試験長さ (mm)	板厚 (mm)	試験日	欠陥位置 (mm)	欠陥 X	欠陥 Y	欠陥 W	欠陥 k	欠陥 d	領域 指示長さ (mm)	欠陥 総和 (mm)	判定	備考
以下余白																			
記録すべきエコー検出せず。																			

APPENDIX G: INSTRUMENTATION OF THE SHAKE-TABLE TESTS

This appendix lists the channels, ID and location of the instruments used in the shake-table tests. The tables and figures contained here serve as a reference for understanding, checking and processing the raw data obtained in the tests. Due to the large amount of channels, only some representative channels are illustrated in photos

G.1 CHANNEL LIST

The following table lists the instrumentation used in the shake-table test. Each number corresponds to the channel used in the data logger. The location of each instrument in the specimen is shown in the drawings of the following section. The calibration factors inputted in the data acquisition system to convert the voltage into a physical quantity are also listed, as well as the cable length used to connect each instrument to the data logger.

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
1	Column internal forces and moments, Frame B	1F	Column 1	W, Top			5%	0.9524	20
2				E, Top			5%	0.9524	20
3				W, Bot.			5%	0.9524	20
4				E, Bot.			5%	0.9524	20
5			Column 2	W, Top			5%	0.9524	20
6				E, Top			5%	0.9524	20
7				W, Bot.			5%	0.9524	20
8				E, Bot.			5%	0.9524	20
9			Column 3	W, Top			5%	0.9524	20
10				E, Top			5%	0.9524	20
11				W, Bot.			5%	0.9524	20
12				E, Bot.			5%	0.9524	20
13		2F	Column 4	W, Top			5%	0.9524	20
14				E, Top			5%	0.9524	20
15				W, Bot.			5%	0.9524	20
16				E, Bot.			5%	0.9524	20
17			Column 5	W, Top			5%	0.9524	20
18				E, Top			5%	0.9524	20
19				W, Bot.			5%	0.9524	20
20				E, Bot.			5%	0.9524	20
21			Column 6	W, Top			5%	0.9524	20
22				E, Top			5%	0.9524	20
23				W, Bot.			5%	0.9524	20
24				E, Bot.			5%	0.9524	20
25		3F	Column 7	W, Top			5%	0.9524	30
26				E, Top			5%	0.9524	30

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
27				W, Bot.			5%	0.9524	30
28				E, Bot.			5%	0.9524	30
29			Column 8	W, Top			5%	0.9524	30
30				E, Top			5%	0.9524	30
31				W, Bot.			5%	0.9524	30
32				E, Bot.			5%	0.9524	30
33			Column 9	W, Top			5%	0.9524	30
34				E, Top			5%	0.9524	30
35				W, Bot.			5%	0.9524	30
36				E, Bot.			5%	0.9524	30
37		4F	Column 10	W, Top			5%	0.9524	30
38				E, Top			5%	0.9524	30
39				W, Bot.			5%	0.9524	30
40				E, Bot.			5%	0.9524	30
41			Column 11	W, Top			5%	0.9524	30
42				E, Top			5%	0.9524	30
43				W, Bot.			5%	0.9524	30
44				E, Bot.			5%	0.9524	30
45			Column 12	W, Top			5%	0.9524	30
46				E, Top			5%	0.9524	30
47				W, Bot.			5%	0.9524	30
48				E, Bot.			5%	0.9524	30
49	Frame B out-of-plane displacement	1F			DP-500ES002	BDZ130116	500mm	0.05	20
50					DP-500ES002	BDZ140125	500mm	0.05	20
51	Beam internal forces and moments, Frame B	2F	Beam 1-Sec. 1	Top fl.			5%	0.9524	30
52				Bot. fl.			5%	0.9524	30
53			Beam 1-Sec. 2	Top fl.			5%	0.9524	30
54				Bot. fl.			5%	0.9524	30
55			Beam 1-Sec. 3	Top fl.			5%	0.9524	30
56				Bot. fl.			5%	0.9524	30
57			Beam 1-Sec. 4	Top fl.			5%	0.9524	30
58				Bot. fl.			5%	0.9524	30

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
59			Beam 5-Sec. 2	Top fl.			5%	0.9524	30
60				Bot. fl.			5%	0.9524	30
61			Beam 5-Sec. 3	Top fl.			5%	0.9524	30
62				Bot. fl.			5%	0.9524	30
63		3F	Beam 2-Sec. 1	Top fl.			5%	0.9524	30
64				Bot. fl.			5%	0.9524	30
65			Beam 2-Sec. 2	Top fl.			5%	0.9524	30
66				Bot. fl.			5%	0.9524	30
67			Beam 2-Sec. 3	Top fl.			5%	0.9524	30
68				Bot. fl.			5%	0.9524	30
69			Beam 2-Sec. 4	Top fl.			5%	0.9524	30
70				Bot. fl.			5%	0.9524	30
71			Beam 6-Sec. 2	Top fl.			5%	0.9524	30
72				Bot. fl.			5%	0.9524	30
73			Beam 6-Sec. 3	Top fl.			5%	0.9524	30
74				Bot. fl.			5%	0.9524	30
75		4F	Beam 3-Sec. 1	Top fl.			5%	0.9524	30
76				Bot. fl.			5%	0.9524	30
77			Beam 3-Sec. 2	Top fl.			5%	0.9524	30
78				Bot. fl.			5%	0.9524	30
79			Beam 3-Sec. 3	Top fl.			5%	0.9524	30
80				Bot. fl.			5%	0.9524	30
81			Beam 3-Sec. 4	Top fl.			5%	0.9524	30
82				Bot. fl.			5%	0.9524	30
83			Beam 7-Sec. 2	Top fl.			5%	0.9524	30
84				Bot. fl.			5%	0.9524	30
85			Beam 7-Sec. 3	Top fl.			5%	0.9524	30
86				Bot. fl.			5%	0.9524	30
87		RF	Beam 4-Sec. 1	Top fl.			5%	0.9524	30
88				Bot. fl.			5%	0.9524	30
89			Beam 4-Sec. 2	Top fl.			5%	0.9524	30
90				Bot. fl.			5%	0.9524	30

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
91			Beam 4-Sec. 3	Top fl.			5%	0.9524	30
92				Bot. fl.			5%	0.9524	30
93			Beam 4-Sec. 4	Top fl.			5%	0.9524	30
94				Bot. fl.			5%	0.9524	30
95			Beam 8-Sec. 2	Top fl.			5%	0.9524	30
96				Bot. fl.			5%	0.9524	30
97			Beam 8-Sec. 3	Top fl.			5%	0.9524	30
98				Bot. fl.			5%	0.9524	30
99									
100									
101	Shake table disp.			WE	DP-1000E	BDH150879	1000mm	0.1	10
102	Shake table acceleration			NS	ARJ-100AT	DNL200001	100m/s ²	0.0845	10
103				WE			100m/s ²	0.0845	10
104				UD			100m/s ²	0.0867	10
105	Specimen's acceleration	2F	Mass top, W	WE	ARF-100A	DJE180569	100m/s ²	0.0888	20
106			Mass top, E	WE	ARF-100A	DJE180552	100m/s ²	0.0902	20
107		3F	Mass top, W	WE	ARF-100A	DJE180565	100m/s ²	0.0887	20
108			Mass top, E	WE	ARF-100A	DJE180549	100m/s ²	0.0903	30
109		4F	Mass top, W	WE	ARF-100A	DJE180572	100m/s ²	0.0885	20
110			Mass top, E	WE	ARF-100A	DJE180570	100m/s ²	0.0897	30
111		RF	Mass top, W	NS	ARF-100AT	DNL200002	100m/s ²	0.0858	20
112				WE			100m/s ²	0.0872	20
113				UD			100m/s ²	0.0927	20
114			Mass top, E	NS	ARF-100AT		100m/s ²	0.092	30
115				WE			100m/s ²	0.0919	30
116				UD			100m/s ²	0.0894	30
117	Specimen's accel.	2F	Beam 1	Bot. fl.	ARF-100A	DJE180566	100m/s ²	0.0893	10
118	Story drift, Frame B, 3/4F	3F	W		CDP-100M	BBR11054	100mm	0.01	20
119		4F	W		CDP-100M	BBR11055	100mm	0.01	20
120	Beam rotation (Frame B)	2F	Beam 1 left end	Top fl.	CDP-25	528661	25mm	0.002	10
121			Beam 1 left end	Bot. fl.	CDP-25	525960	25mm	0.002	10
122			Beam 1 right end	Top fl.	CDP-25	528502	25mm	0.002	20

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
123			Beam 1 right end	Bot. fl.	CDP-25	522857	25mm	0.002	20
124			Beam 5 left end	Top fl.	CDP-25	526808	25mm	0.002	30
125			Beam 5 left end	Bot. fl.	CDP-25	522878	25mm	0.002	30
126			Beam 5 right end	Top fl.	CDP-25	525957	25mm	0.002	20
127			Beam 5 right end	Bot. fl.	CDP-25	525963	25mm	0.002	20
128		3F	Beam 2 left end	Top fl.	CDP-25	520540	25mm	0.002	20
129			Beam 2 left end	Bot. fl.	CDP-25	BBC03530	25mm	0.002	20
130			Beam 2 right end	Top fl.	CDP-25	527622	25mm	0.002	20
131			Beam 2 right end	Bot. fl.	CDP-25	525948	25mm	0.002	20
132			Beam 6 left end	Top fl.	CDP-25	529731	25mm	0.002	30
133			Beam 6 left end	Bot. fl.	CDP-25	525950	25mm	0.002	30
134			Beam 6 right end	Top fl.	CDP-25	BBC03551	25mm	0.002	20
135			Beam 6 right end	Bot. fl.	CDP-25	525944	25mm	0.002	20
136		4F	Beam 3 left end	Top fl.	CDP-25	525198	25mm	0.002	20
137			Beam 3 left end	Bot. fl.	CDP-25	522854	25mm	0.002	20
138			Beam 3 right end	Top fl.	CDP-25	520537	25mm	0.002	20
139			Beam 3 right end	Bot. fl.	CDP-25	525203	25mm	0.002	20
140			Beam 7 right end	Top fl.	CDP-25	520539	25mm	0.002	30
141			Beam 7 right end	Bot. fl.	CDP-25	525946	25mm	0.002	30
142		RF	Beam 4 left end	Top fl.	CDP-25	529722	25mm	0.002	20
143			Beam 4 left end	Bot. fl.	CDP-25	529728	25mm	0.002	20
144			Beam 4 right end	Top fl.	CDP-25	525945	25mm	0.002	20
145			Beam 4 right end	Bot. fl.	CDP-25	520536	25mm	0.002	20
146			Beam 8 right end	Top fl.	CDP-25	529727	25mm	0.002	30
147			Beam 8 right end	Bot. fl.	CDP-25	523857	25mm	0.002	30
148	Specimen's acceleration	3F	Beam 2	Bot. fl.	ARF-100A	DJE180568	100m/s ²	0.0873	
149		4F	Beam 3	Bot. fl.	ARF-100A	DJE180553	100m/s ²	0.0901	
150		RF	Beam 4	Bot. fl.	ARF-100A	DJE180550	100m/s ²	0.0893	
151	Panel zone shear deformation	2F	Panel zone 1	left	CDP-25MT	BCX211003	25mm	0.002	10
152			Panel zone 1	right	CDP-25MT	BCX211004	25mm	0.002	10
153			Panel zone 2	left	CDP-25MT	BCX211006	25mm	0.002	20
154			Panel zone 2	right	CDP-25MT	BCX211002	25mm	0.002	20

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
155			Panel zone 3	left	CDP-25MT	BCX211001	25mm	0.002	20
156			Panel zone 3	right	CDP-25MT	BCX211005	25mm	0.002	20
157		3F	Panel zone 4	left	CDP-25MT	BCX211009	25mm	0.002	10
158			Panel zone 4	right	CDP-25M	BBX150209	25mm	0.002	10
159			Panel zone 5	left	CDP-25M	BBX150208	25mm	0.002	20
160			Panel zone 5	right	CDP-25MT	BCX211010	25mm	0.002	20
161			Panel zone 6	left	CDP-25MT	BCX211008	25mm	0.002	20
162			Panel zone 6	right	CDP-25MT	BCX211007	25mm	0.002	20
163		4F	Panel zone 7	left	CDP-10MT	BCW190513	10mm	0.001	20
164			Panel zone 7	right	CDP-10MT	BCW190512	10mm	0.001	20
165			Panel zone 8	left	CDP-25MT	BCX200009	25mm	0.002	30
166			Panel zone 8	right	CDP-25MT	BCX200010	25mm	0.002	30
167			Panel zone 9	left	CDP-10MT	BCW190515	10mm	0.001	20
168			Panel zone 9	right	CDP-10MT	BCW170424	10mm	0.001	20
169		RF	Panel zone 10	left	CDP-10MT	BCW170393	10mm	0.001	20
170			Panel zone 10	right	CDP-10MT	BCW190514	10mm	0.001	20
171			Panel zone 11	left	CDP-10MT	BCW190516	10mm	0.001	20
172			Panel zone 11	right	CDP-10MT	BCW190510	10mm	0.001	20
173			Panel zone 12	left	CDP-10M	BBW150186	10mm	0.001	30
174			Panel zone 12	right	CDP-10MT	BCW190511	10mm	0.001	30
175	Slippage of the Frame B column bases	1F	Column 1		CDP-50	BBD09263	50mm	0.005	10
176			Column 2		CDP-50	519744	50mm	0.005	20
177			Column 3		CDP-50	519749	50mm	0.005	20
178									
179	Col. contraction	1F	Column 2	S	DP-500F	BDL190368	500mm	0.05	10
180	Story drift, frame A	1F	Column A1		DP-1000G	BDS210122	1000mm	0.1	20
181			Column A2		DP-1000G	BDS211024	1000mm	0.1	20
182		2F	Column A1		DP-1000F	BDM200399	1000mm	0.05	20
183			Column A2		DP-1000E	BDH120335	1000mm	0.05	20
184		3F	Column A1		DP-500ES002	BDZ140124	500mm	0.05	20
185		4F	Column A2		DP-500F	BDL190365	500mm	0.05	20
186	Story drift, frame C	1F	Column <u>C</u> 2		DP-1000G	BDS211025	1000mm	0.1	20

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
187			Column C1		DP-1000G	BDS211026	1000mm	0.1	20
188		2F	Column C1		DP-1000F	BDM200400	1000mm	0.05	20
189			Column C2		DP-1000E	BDH09415	1000mm	0.05	20
190		3F	Column C2		DP-500F	BDL190367	500mm	0.05	20
191		4F	Column C2		DP-500F	BDL190366	500mm	0.05	20
192	Slippage of the top mass	RF	Line A beam top fl.	W	CDP-50	BBD210242	50mm	0.005	20
193			Line C beam top fl.	W	CDP-50	BBD210241	50mm	0.005	20
194	Acceleration of the Curtain Wall					DJE180571	100m/s ²	0.0836	
195						DJE180567	100m/s ²	0.088	
196						DJE180551	100m/s ²	0.0889	
197	Strain of the Curtain Wall frame						5%	0.9524	
198							5%	0.9524	
199							5%	0.9524	
200							5%	0.9524	
201	Column internal forces and moments, Frame A	1F	Column A1	W, Top			5%	0.9524	30
202				E, Top			5%	0.9524	30
203				W, Bot.			5%	0.9524	30
204				E, Bot.			5%	0.9524	30
205			Column A2	W, Top			5%	0.9524	30
206				E, Top			5%	0.9524	30
207				W, Bot.			5%	0.9524	30
208				E, Bot.			5%	0.9524	30
209			Column A3	W, Top			5%	0.9524	30
210				E, Top			5%	0.9524	30
211				W, Bot.			5%	0.9524	30
212				E, Bot.			5%	0.9524	30
213		2F	Column A1	W, Top			5%	0.9524	30
214				E, Top			5%	0.9524	30
215				W, Bot.			5%	0.9524	30
216				E, Bot.			5%	0.9524	30
217			Column A2	W, Top			5%	0.9524	30
218				E, Top			5%	0.9524	30

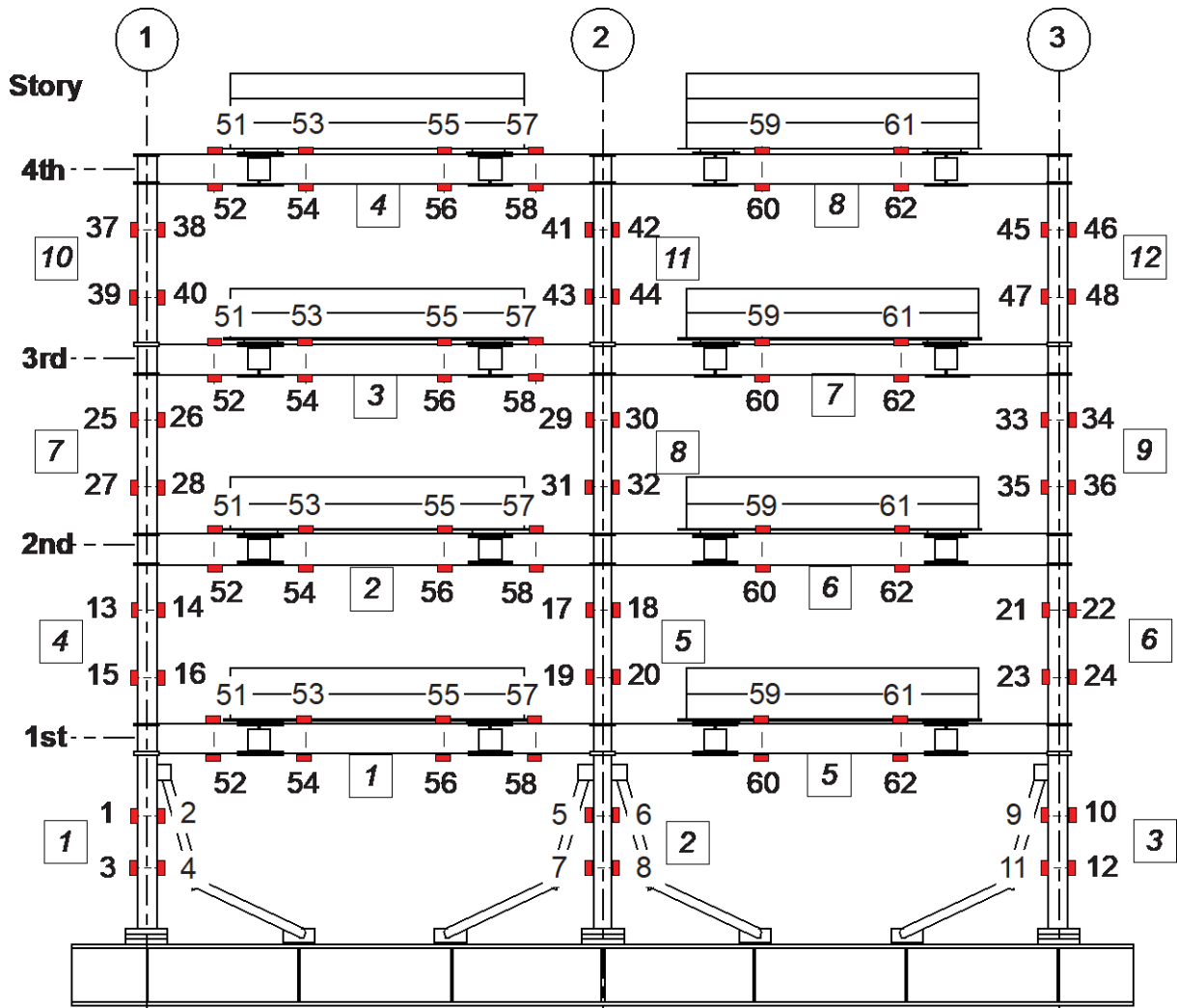
CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
219				W, Bot.			5%	0.9524	30
220				E, Bot.			5%	0.9524	30
221			Column A3	W, Top			5%	0.9524	30
222				E, Top			5%	0.9524	30
223				W, Bot.			5%	0.9524	30
224				E, Bot.			5%	0.9524	30
225	Column internal forces and moments, Frame C	1F	Column C1	W, Top			5%	0.9524	20
226				E, Top			5%	0.9524	20
227				W, Bot.			5%	0.9524	20
228				E, Bot.			5%	0.9524	20
229			Column C2	W, Top			5%	0.9524	20
230				E, Top			5%	0.9524	20
231				W, Bot.			5%	0.9524	20
232				E, Bot.			5%	0.9524	20
233			Column C3	W, Top			5%	0.9524	20
234				E, Top			5%	0.9524	20
235				W, Bot.			5%	0.9524	20
236				E, Bot.			5%	0.9524	20
237		2F	Column C1	W, Top			5%	0.9524	20
238				E, Top			5%	0.9524	20
239				W, Bot.			5%	0.9524	20
240				E, Bot.			5%	0.9524	20
241			Column C2	W, Top			5%	0.9524	20
242				E, Top			5%	0.9524	20
243				W, Bot.			5%	0.9524	20
244				E, Bot.			5%	0.9524	20
245			Column C3	W, Top			5%	0.9524	20
246				E, Top			5%	0.9524	20
247				W, Bot.			5%	0.9524	20
248				E, Bot.			5%	0.9524	20
249	Deformation of the curtain wall				DT-50EM5	YT9970009	50mm		
250					DT-50EM5	YT9970010	50mm		

CH	Measurement	Location			Instrument	Serial number	Range	Calibration factor	Cable length [m]
251	In-plane rotation of the curtain wall			Gyro Y		GX9160002	±15.71rad/s	8.032	20
252				Gyro Y		GX9160003	±15.71rad/s	7.926	20
253				Gyro Y		GX9160004	±15.71rad/s	8.011	20
254				Gyro Y		GX9160005	±15.71rad/s	8.032	20
255	Deformation of the curtain wall				CDP-50		50mm	0.005	20
256					CDP-50		50mm	0.005	20
257					CDP-50		50mm	0.005	20
258					CDP-50		50mm	0.005	20
259					CDP-50		50mm	0.005	20
260					CDP-50		50mm	0.005	20
261	Story drift, Frame B, 1/2F	1F	W		IL-600	7L120031	±400mm	80	20
262			E		IL-600	7L120036	±400mm	80	20
263		2F	W		IL-600	63718290	±400mm	80	20
264			E		IL-600	63718291	±400mm	80	20

G.2 LABELING OF INSTRUMENTS

STRAIN GAGES-FRAME B

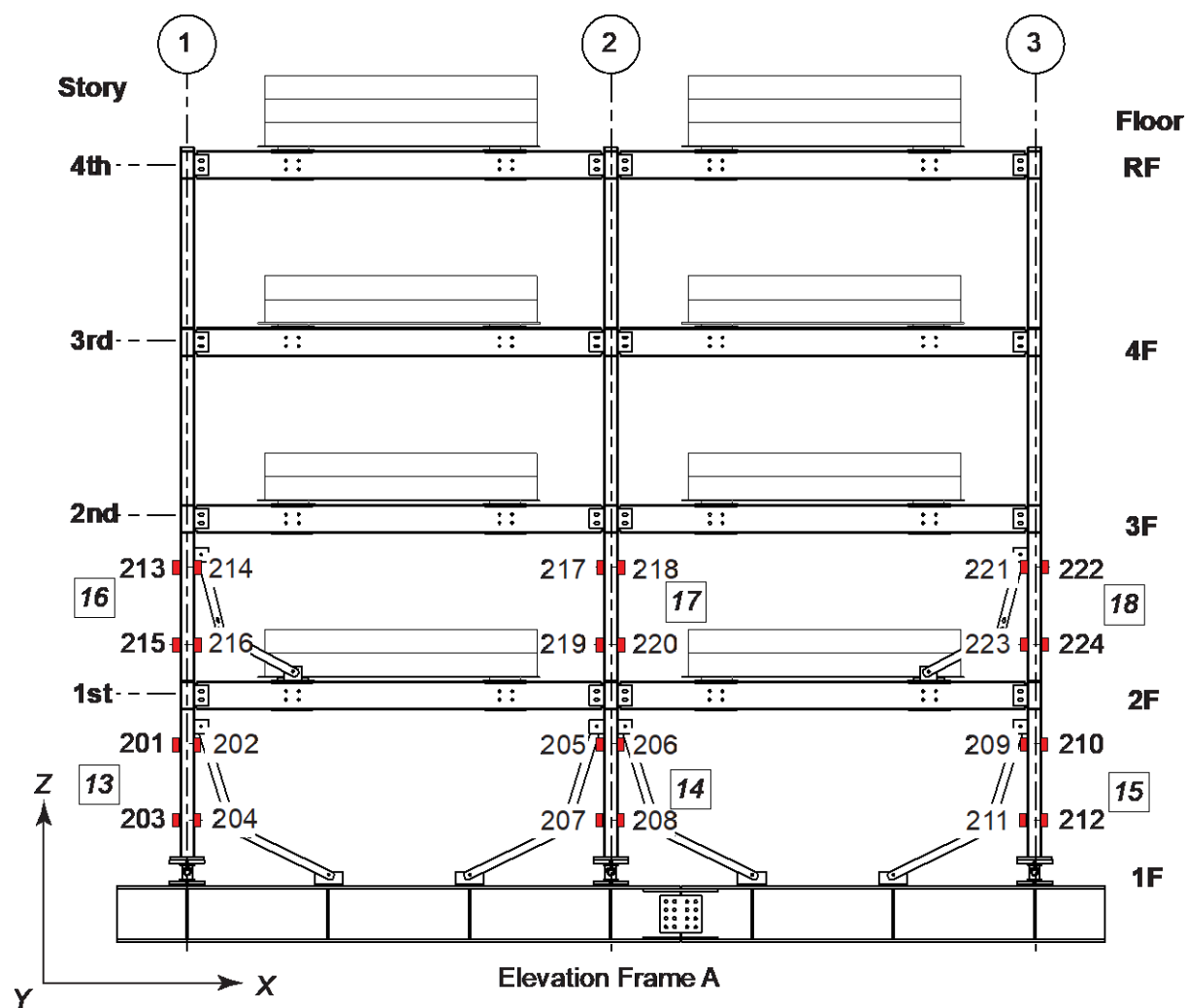
The column and beam number are shown in a square box.



Elevation Frame B

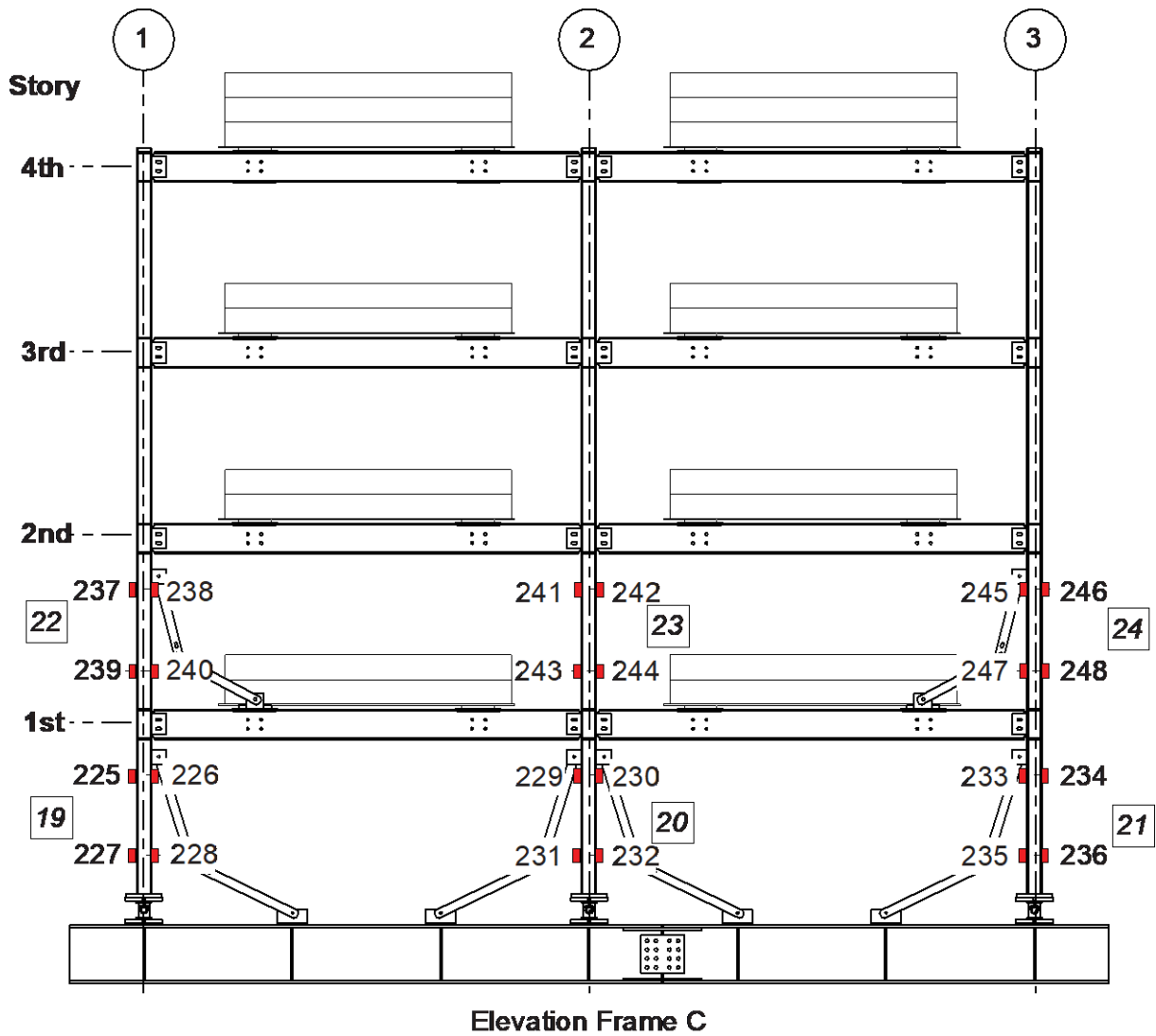
STRAIN GAGES-FRAME A

The column numbers are shown in a square box.

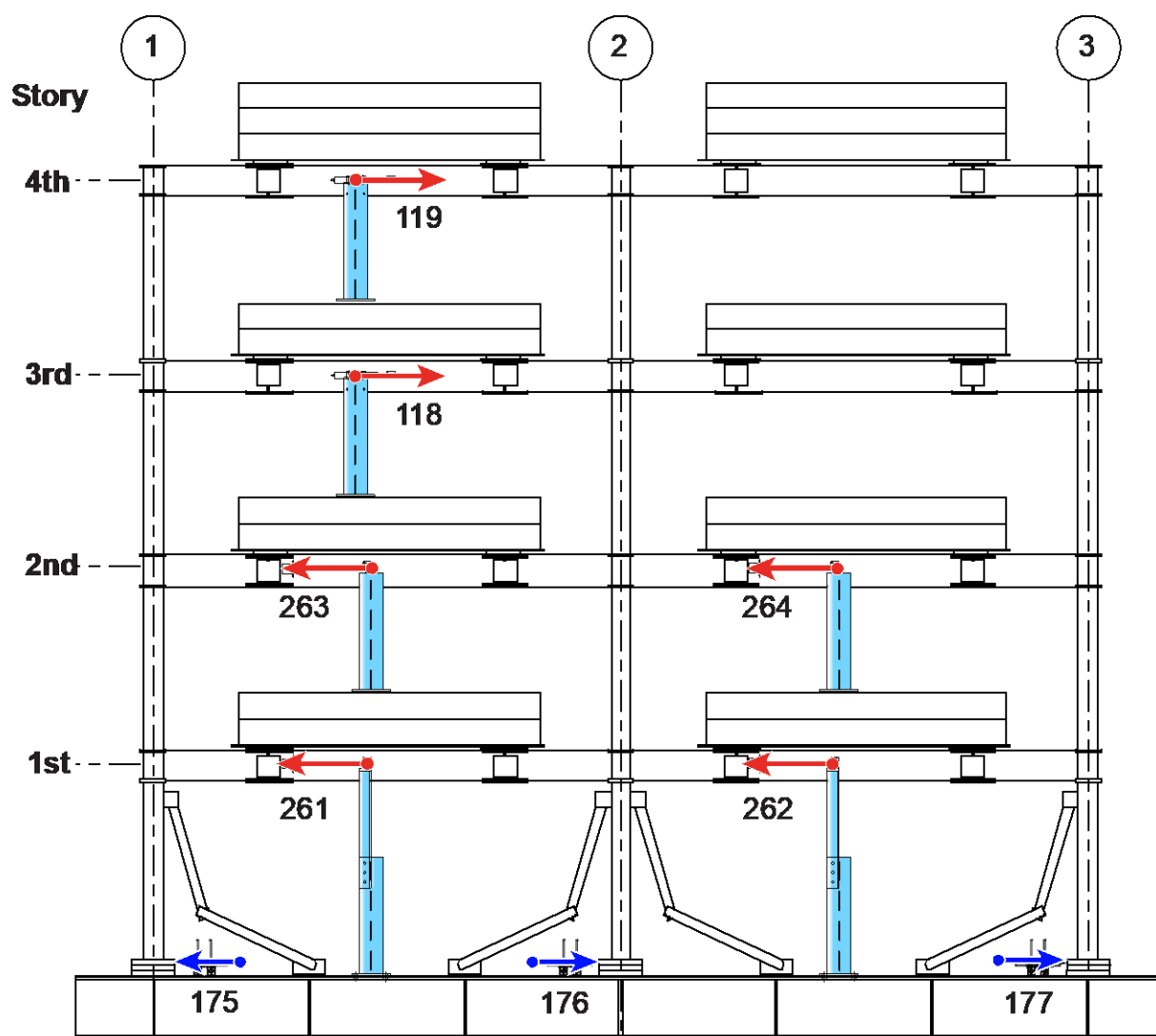


STRAIN GAGES-FRAME C

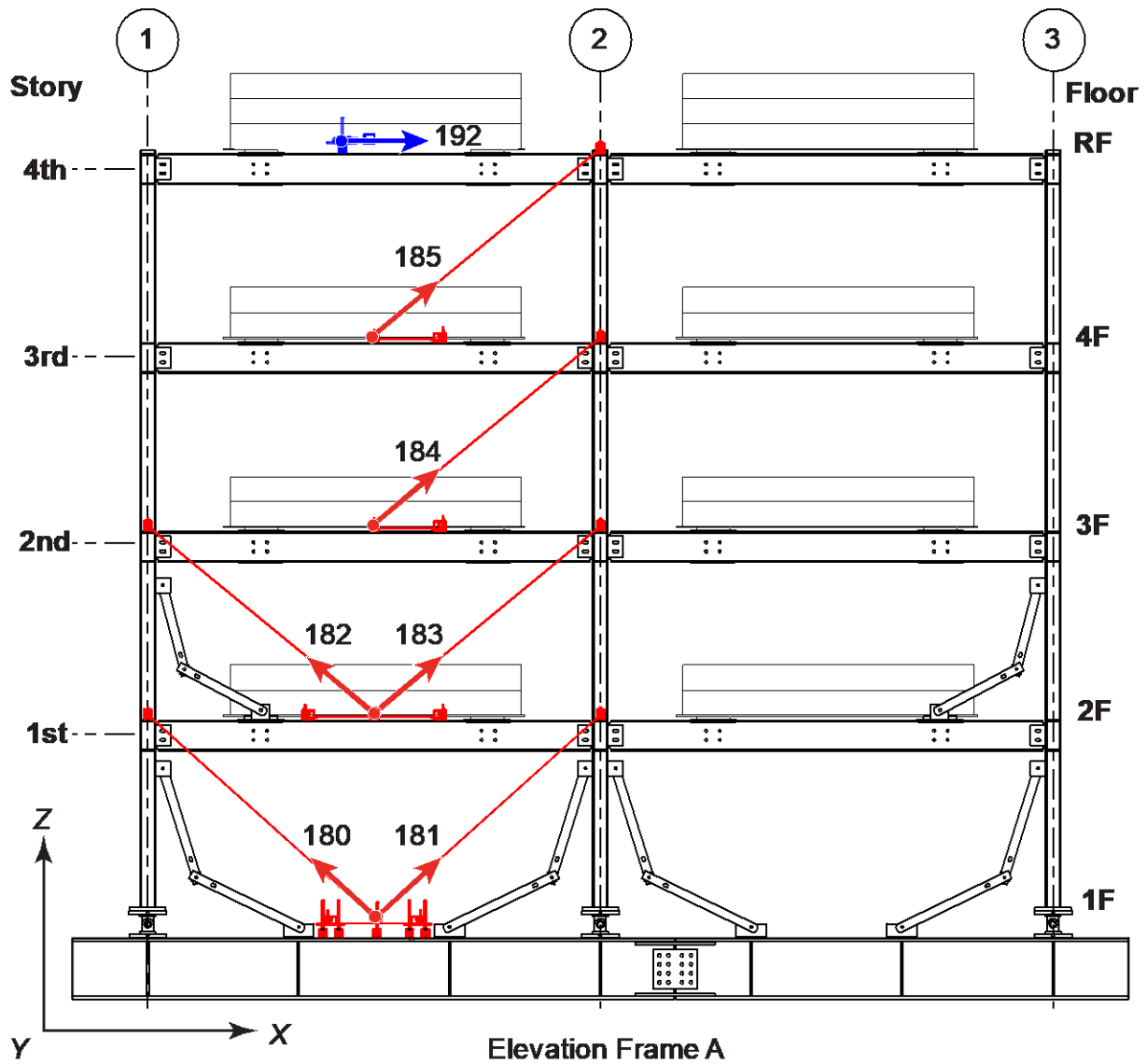
The column numbers are shown in a square box.



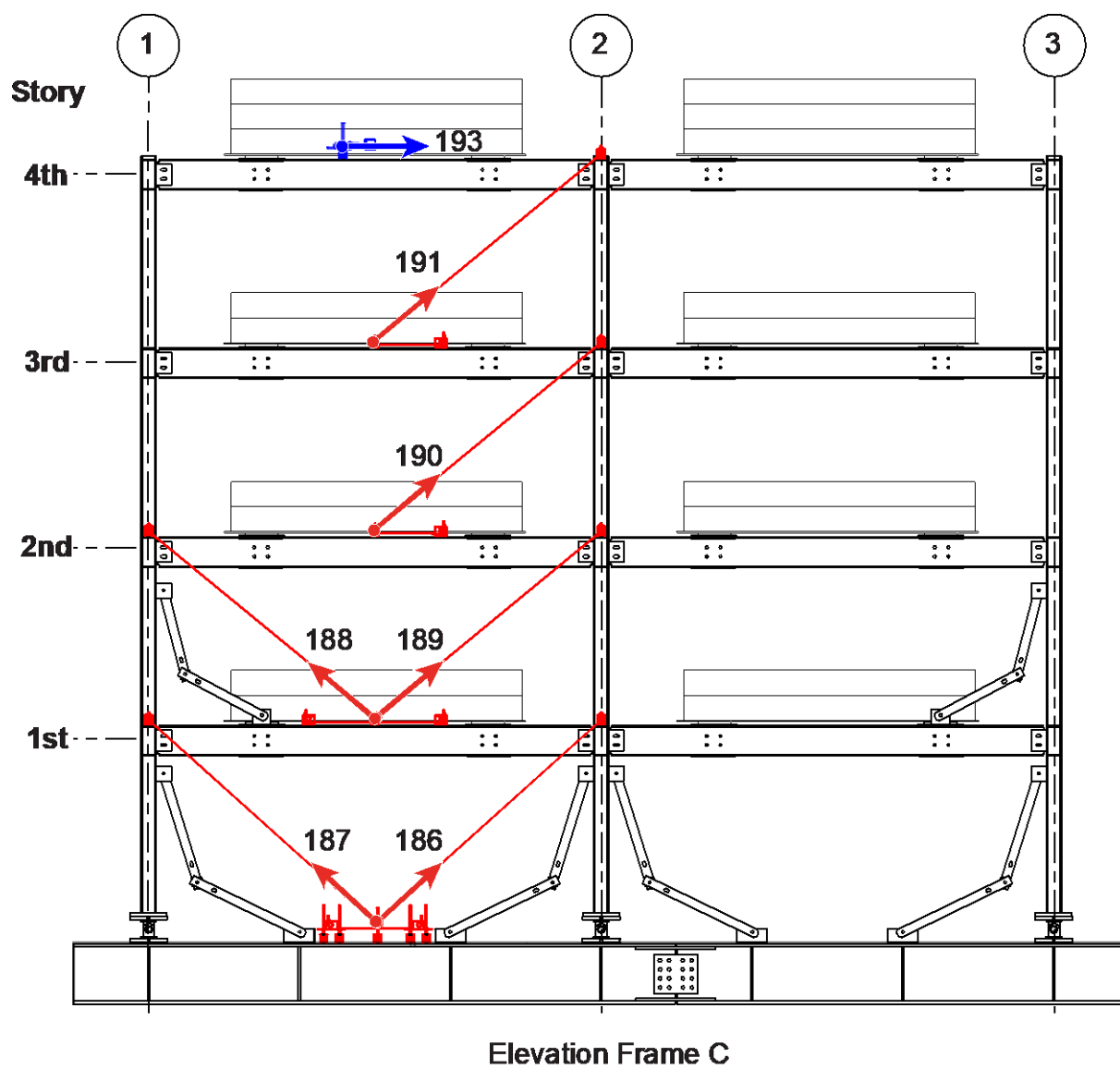
STORY DRIFT-FRAME B



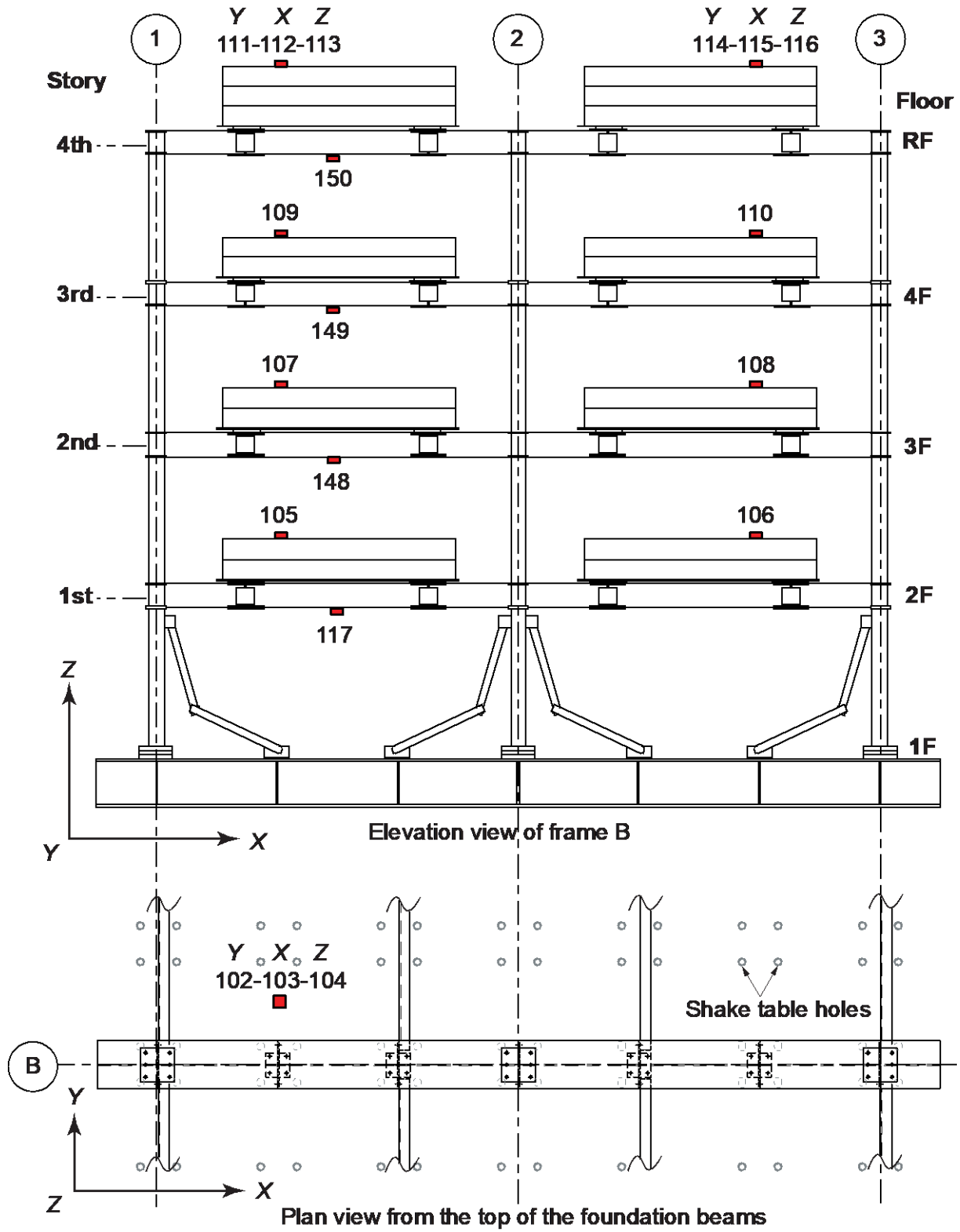
STORY DRIFT-FRAME A



STORY DRIFT-FRAME C

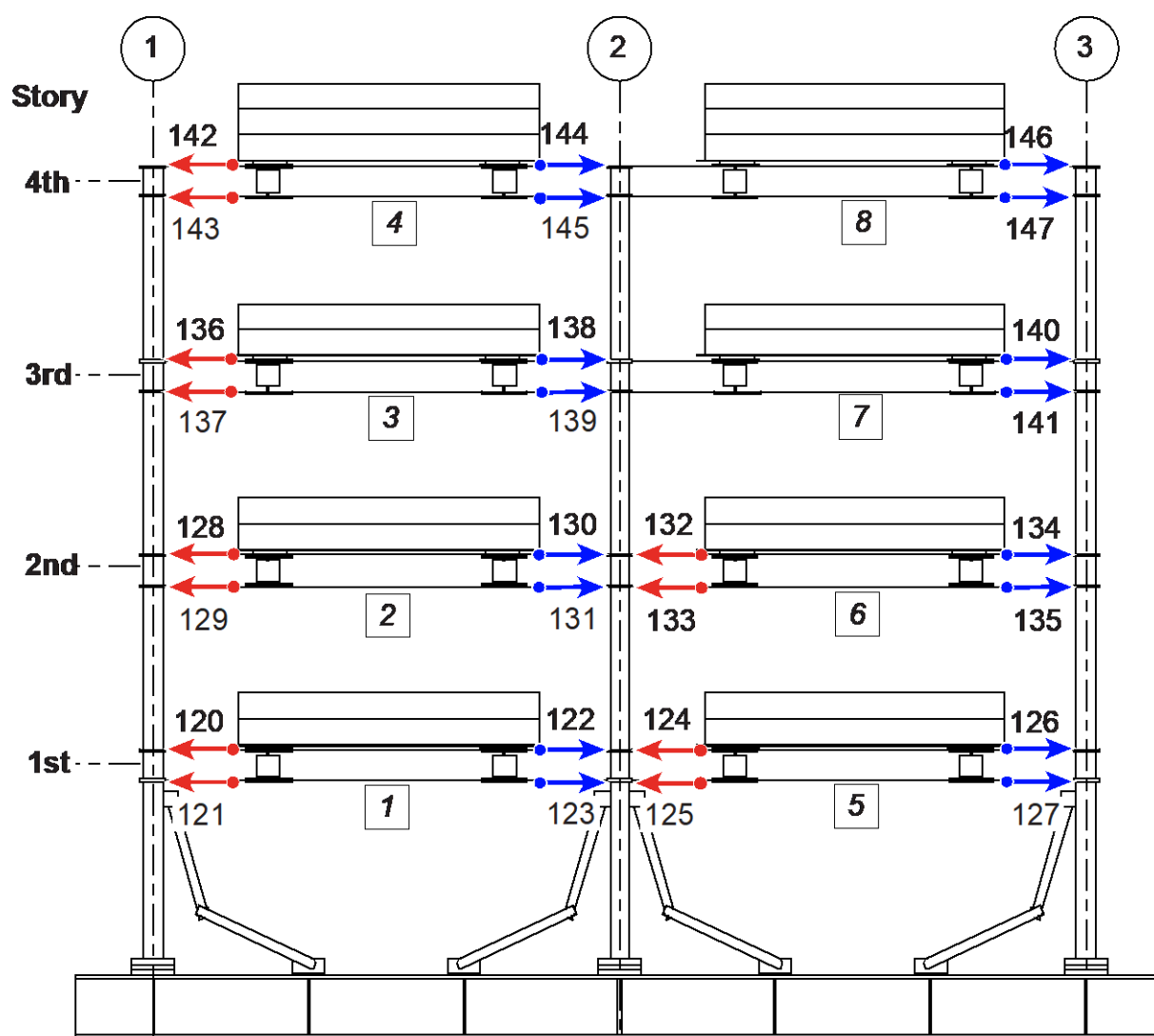


ACCELEROMETERS



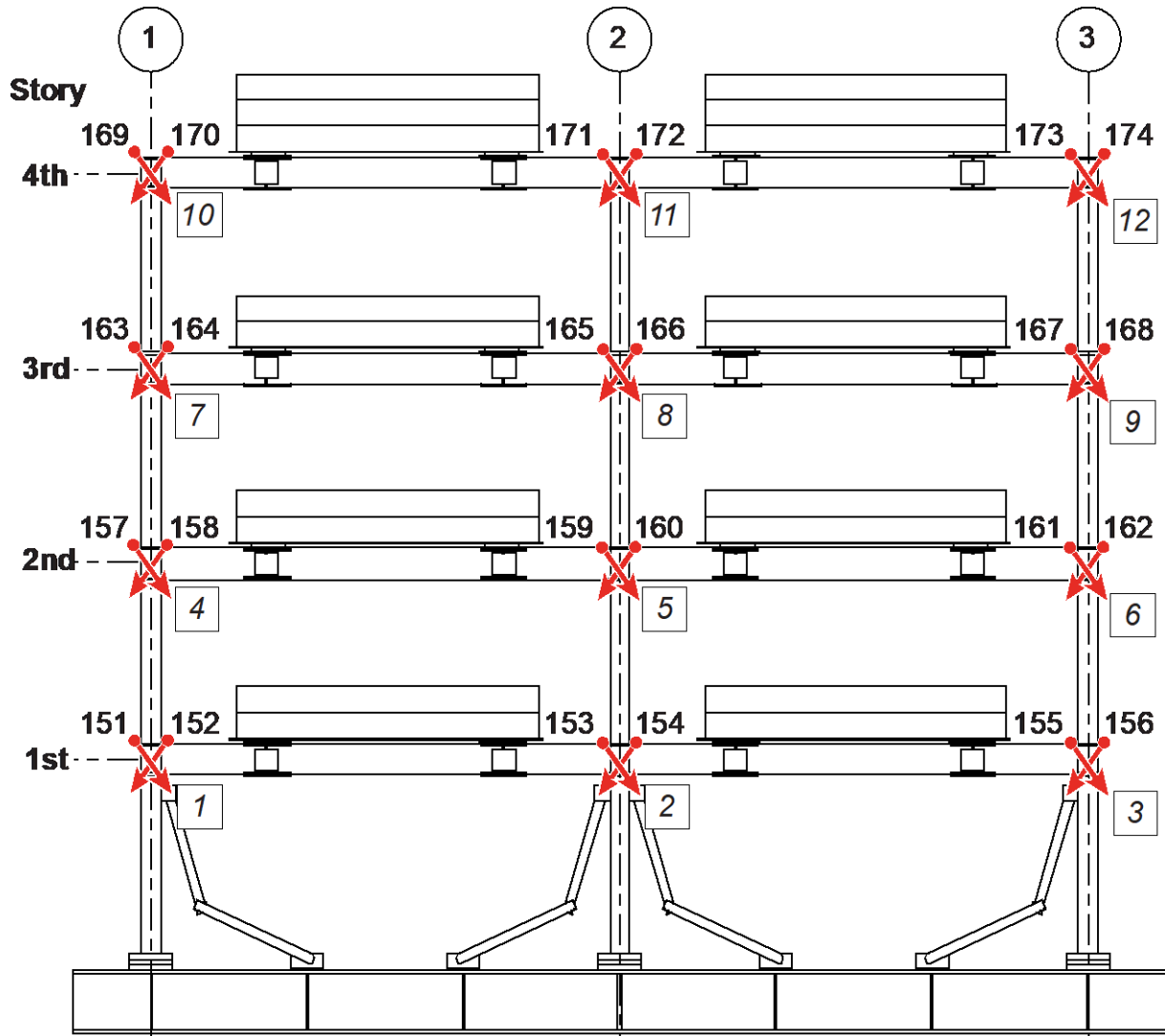
BEAM ROTATION

The beam numbers are shown in a square box.

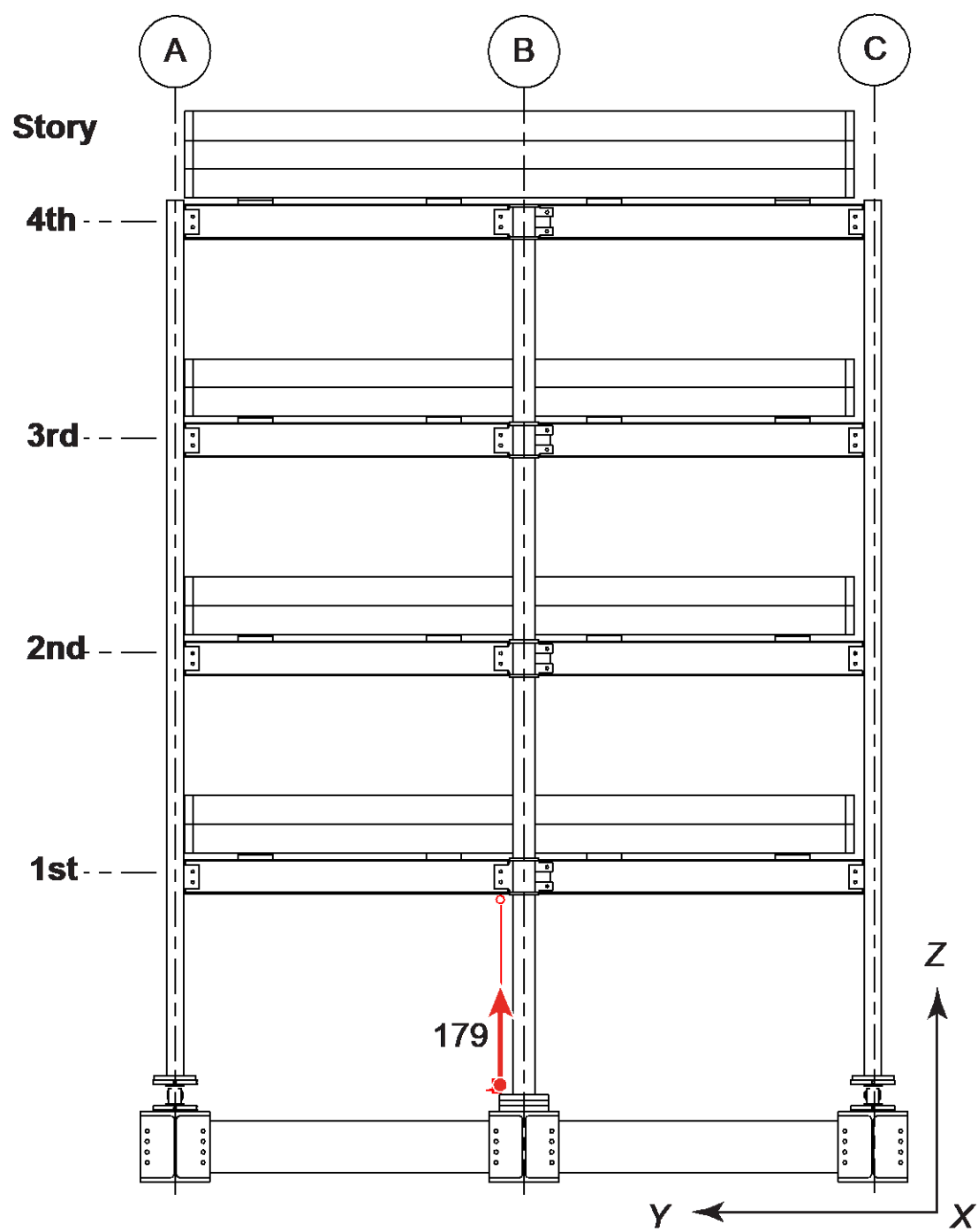


PANEL ZONE DEFORMATION

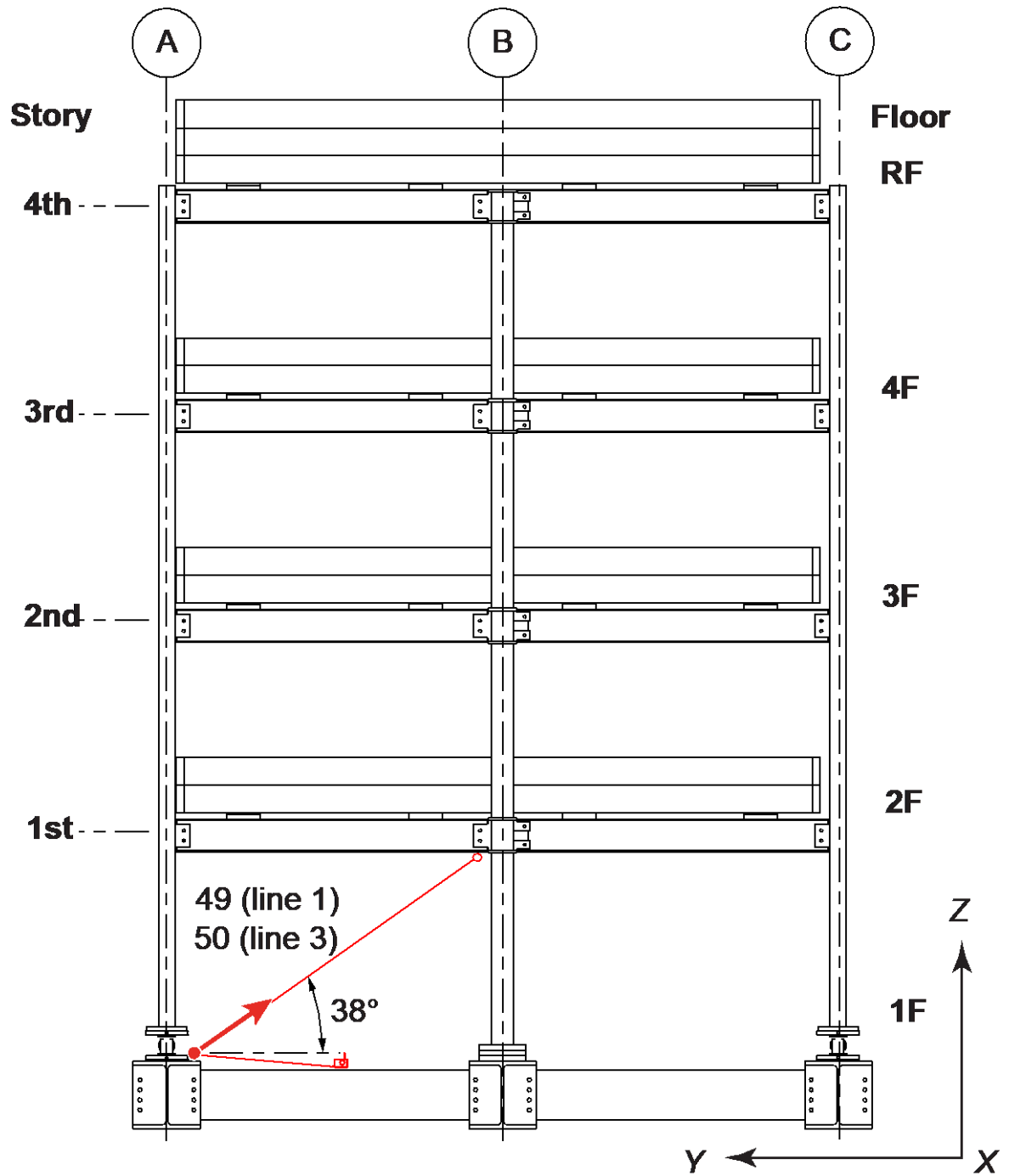
The panel zone numbers are shown in a square box.



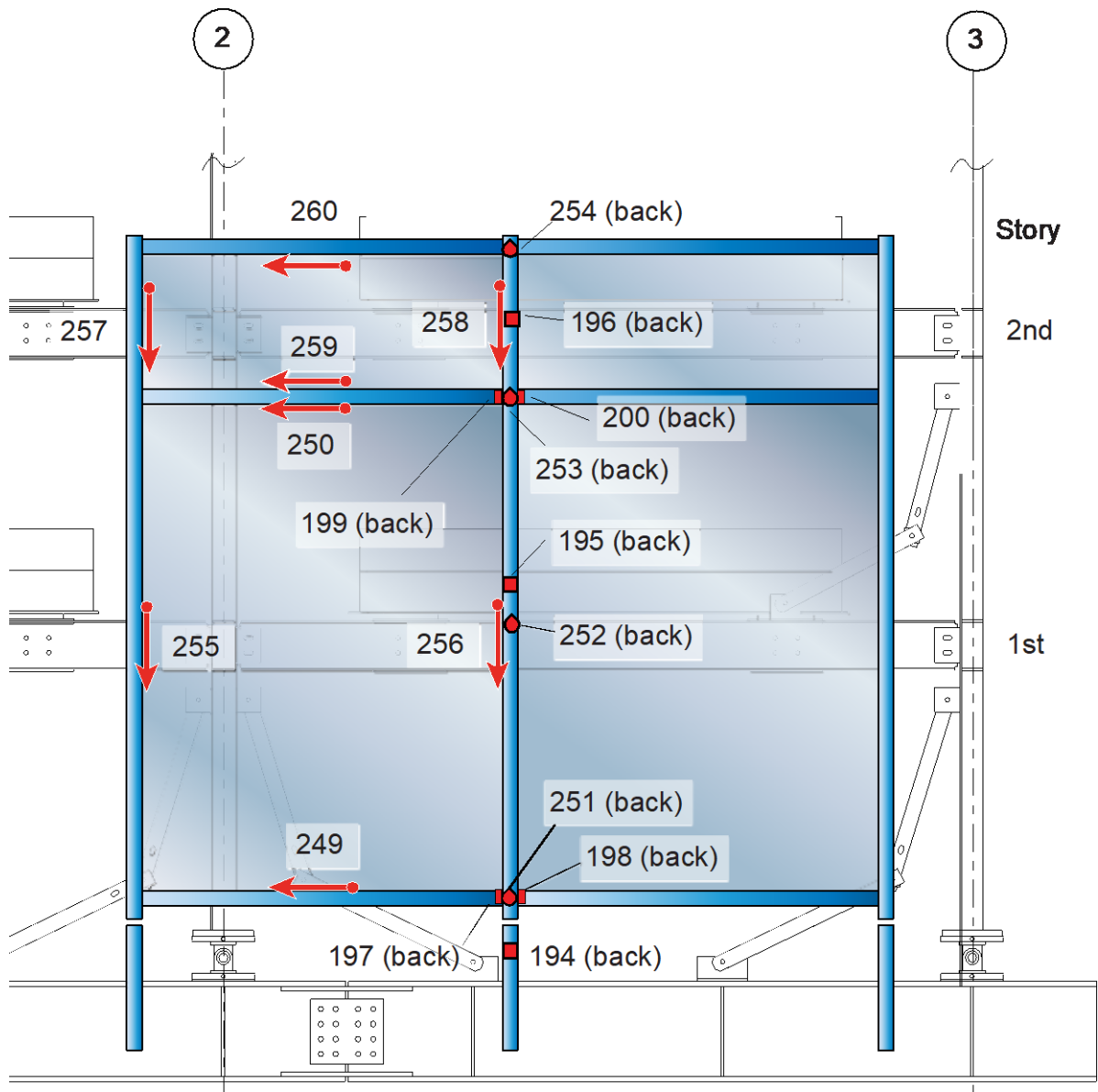
CONTRACTION OF THE CENTRAL COLUMN



Elevation Frame 2

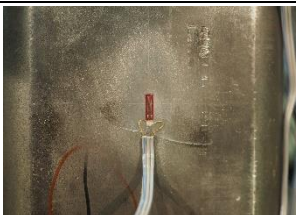
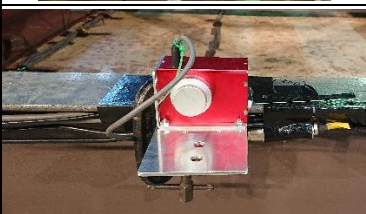
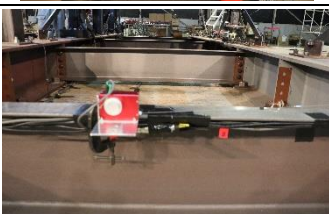
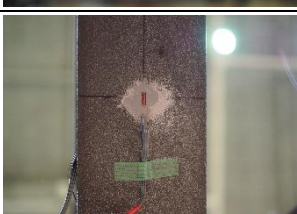
OUT-OF-PLANE DEFORMATION OF FRAME B AT THE 1ST STORY

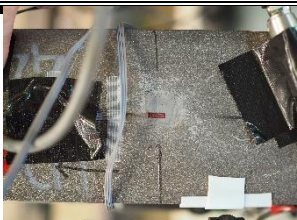
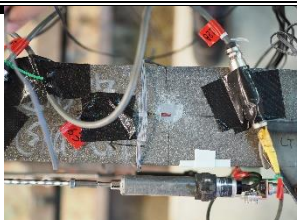
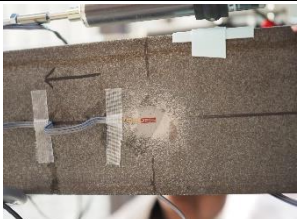
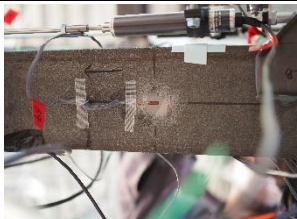
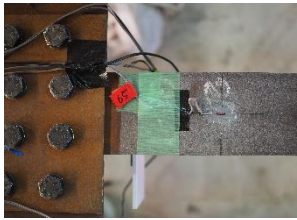
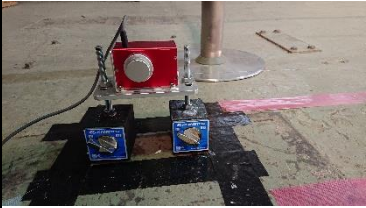
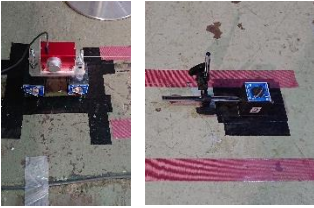
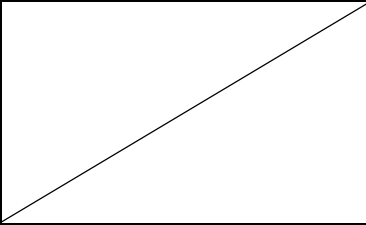
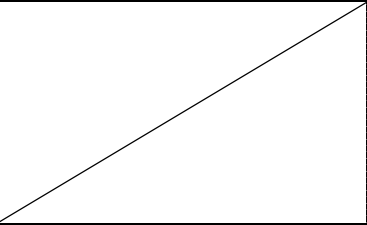
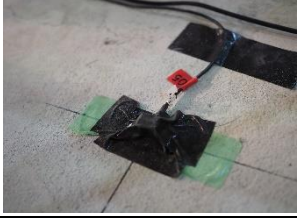
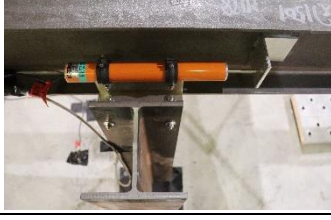
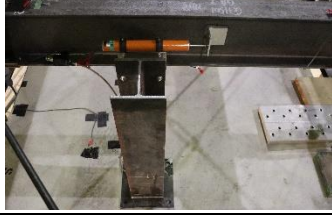
CURTAIN WALL

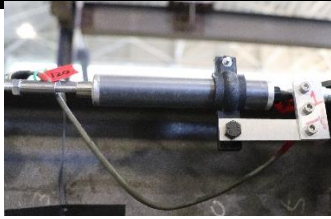
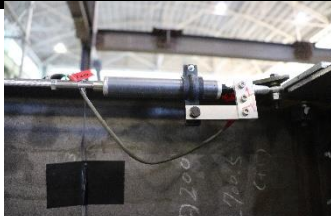
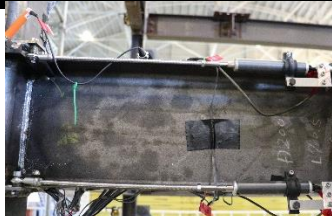
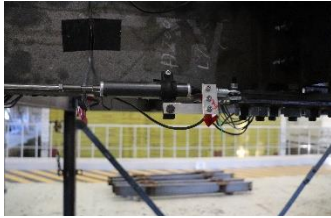
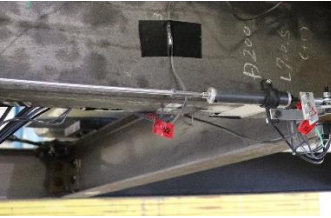
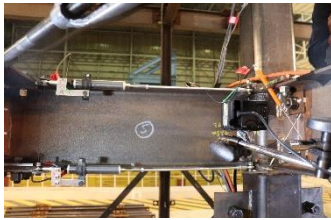
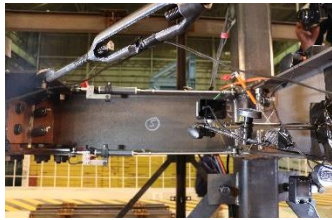
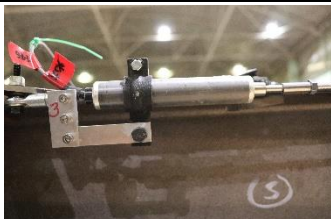
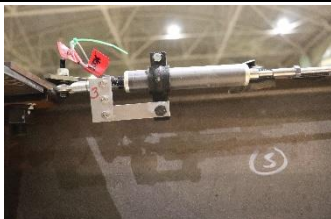
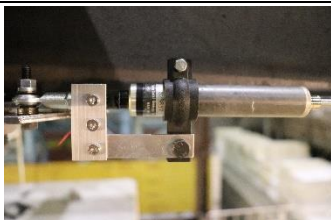


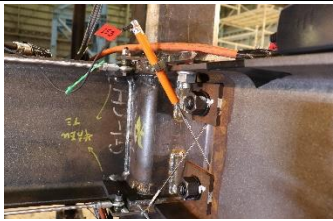
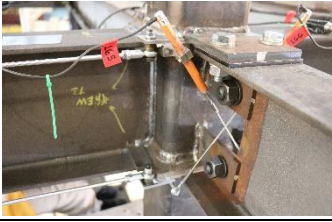
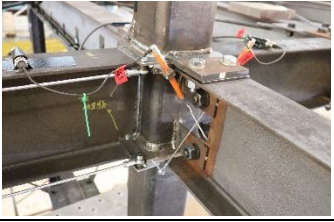
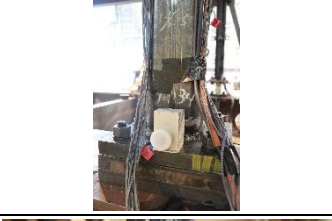
Elevation Curtain Wall

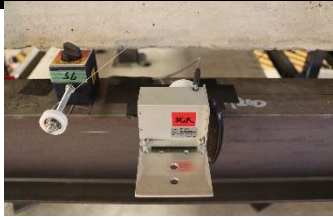
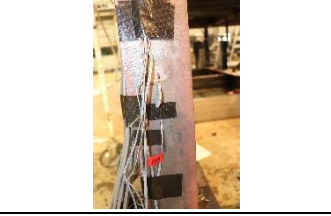
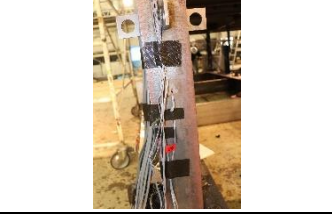
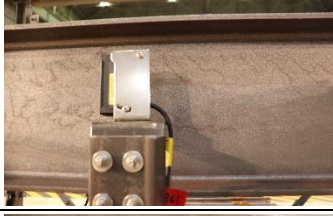
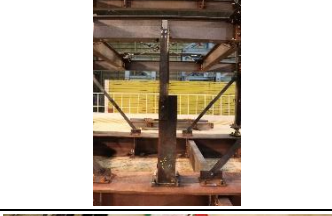
G.3 EXAMPLES OF INSTRUMENTATION

CH	Close	Medium	Far
009			
010			
011			
012			
050			
061			
062			

CH	Close	Medium	Far
063			
064			
065			
101			
102			
103			
104			
105			
114			
115			
116			
119			

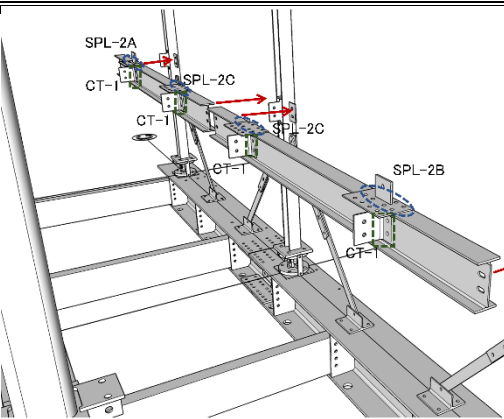
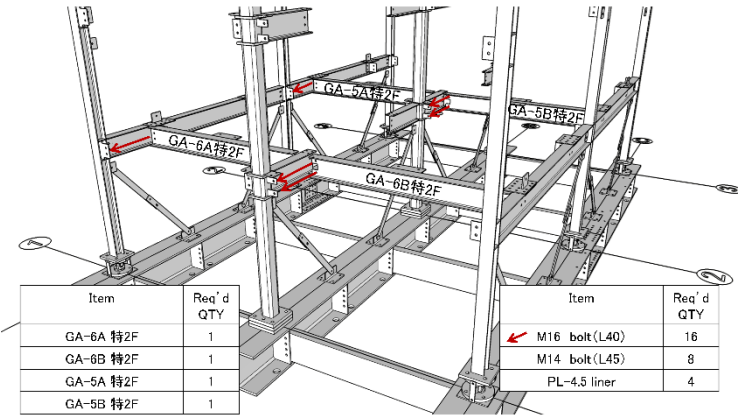
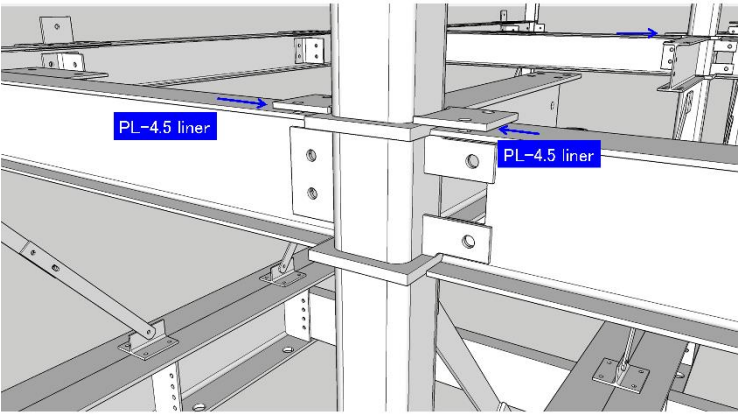
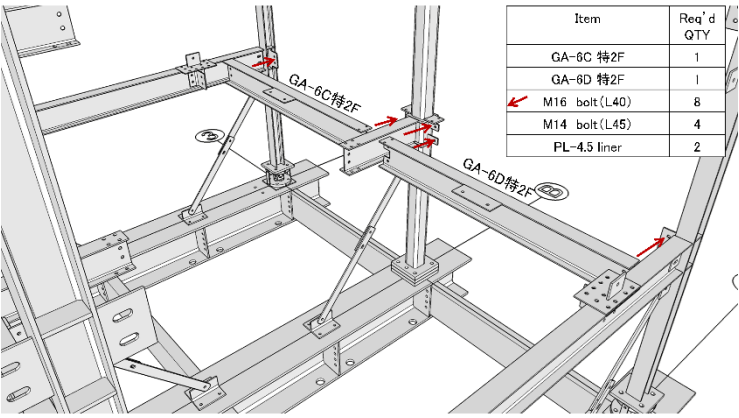
CH	Close	Medium	Far
120			
121			
122			
123			
144			
145			
146			
147			

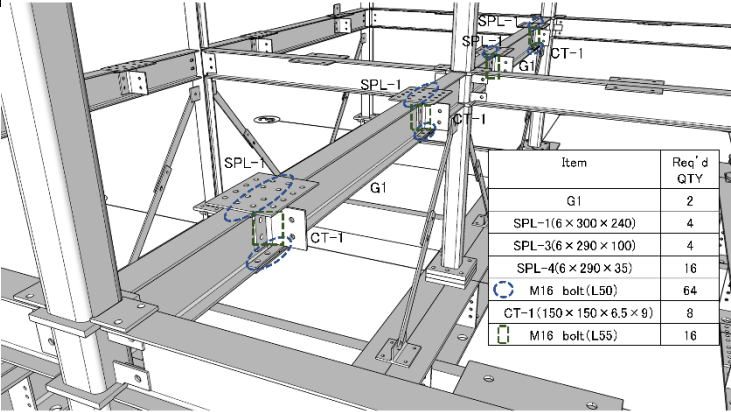
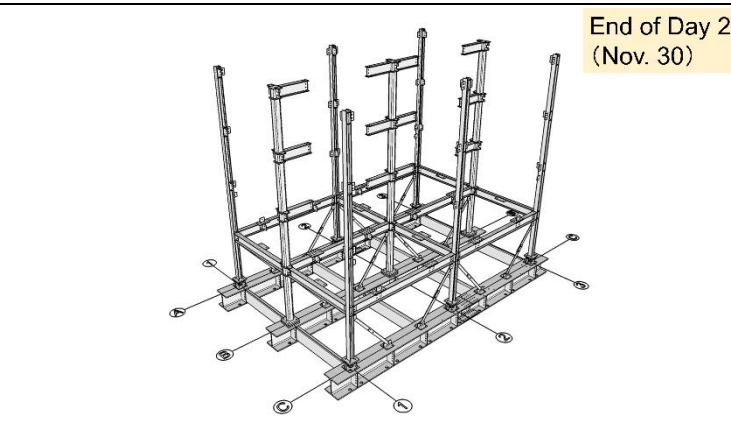
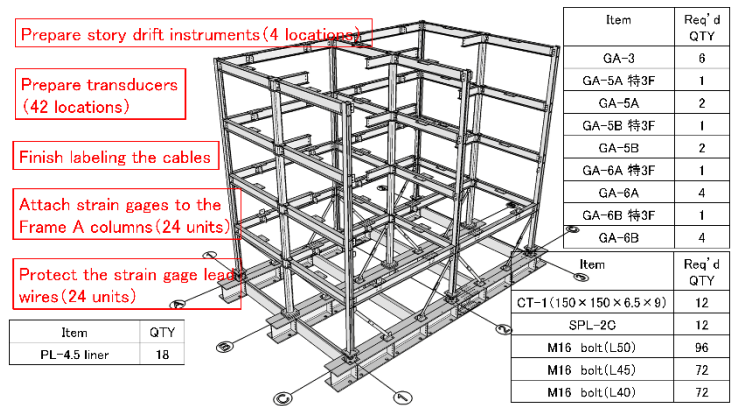
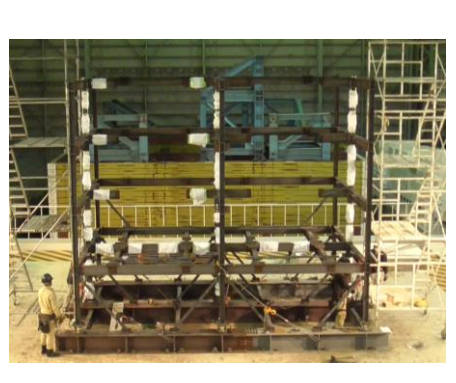
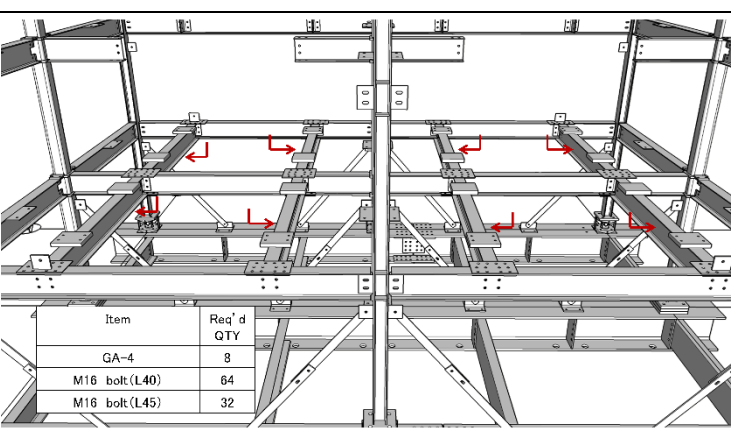
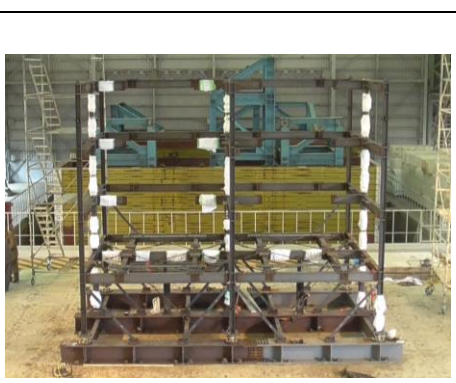
CH	Close	Medium	Far
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153			
154			
165			
166			
175			
179			
180			

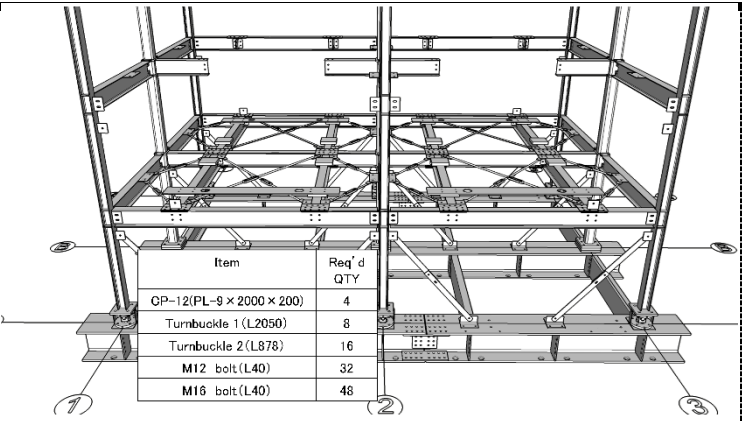
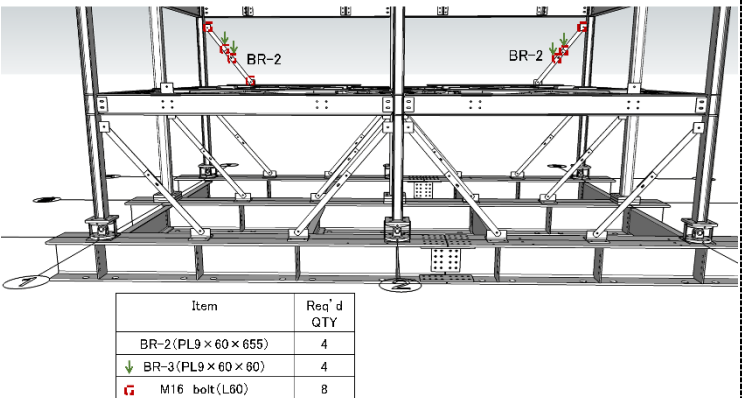
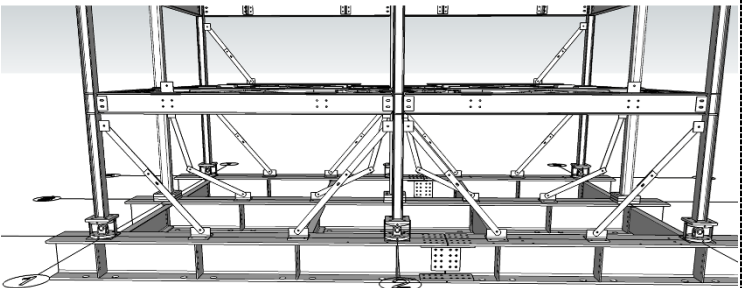
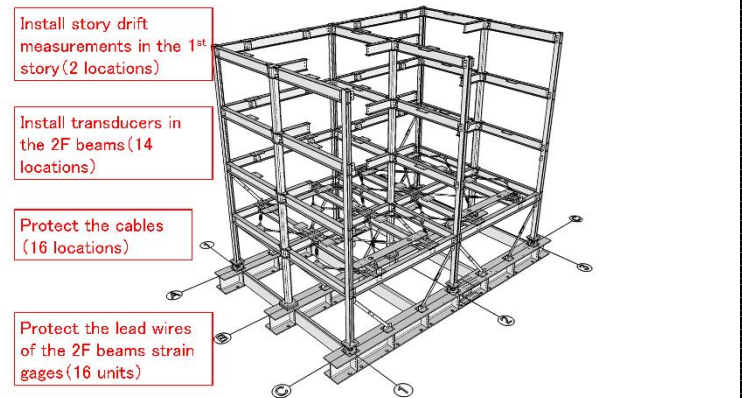
CH	Close	Medium	Far
191			
193			
201			
202			
203			
204			
261			
264			

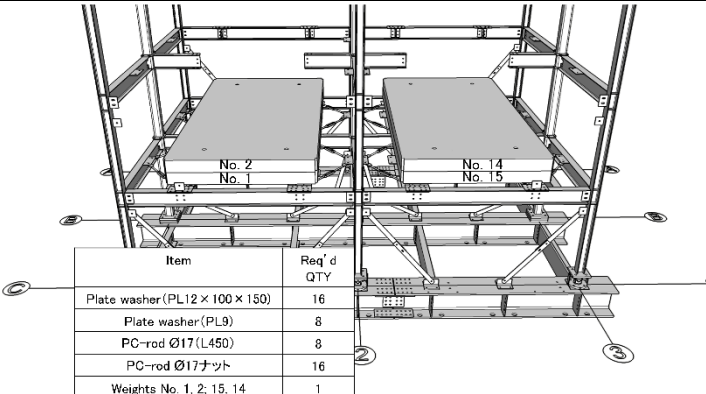
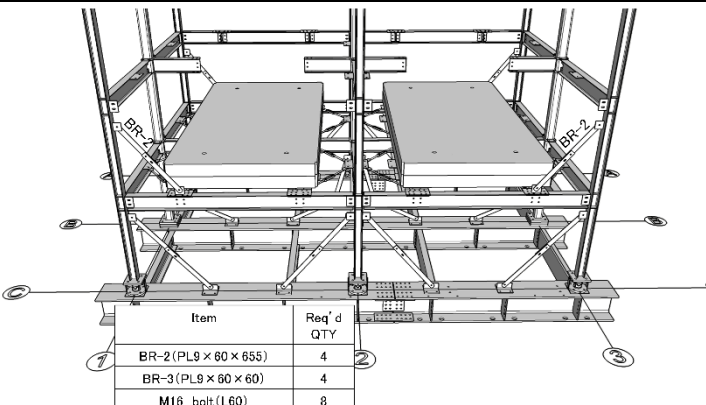
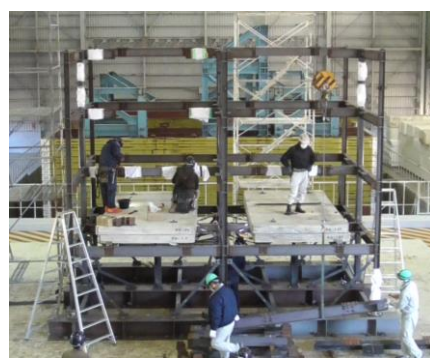
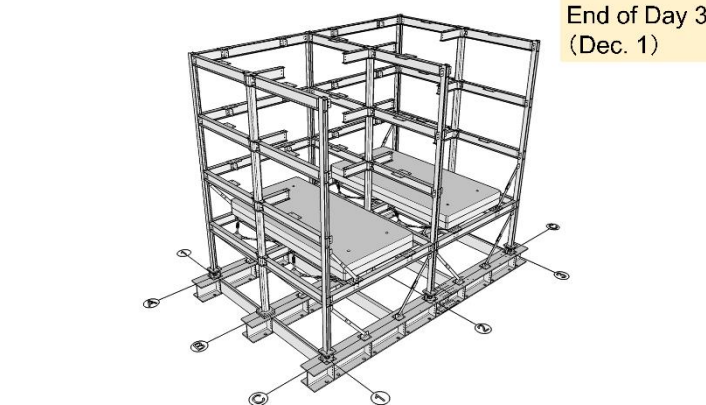
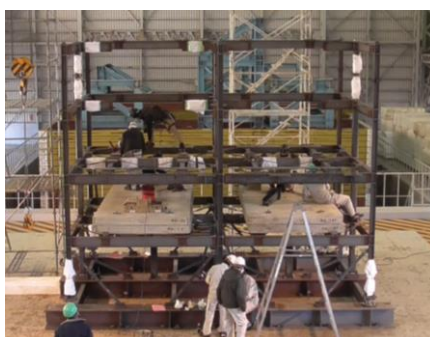
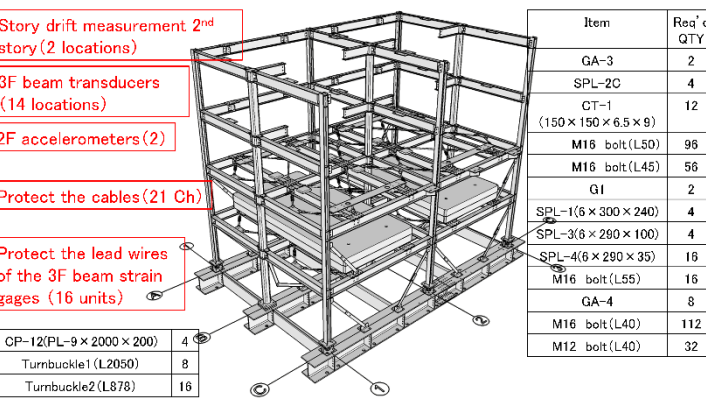
APPENDIX H: CONSTRUCTION SEQUENCE OF THE SHAKE-TABLE TESTS

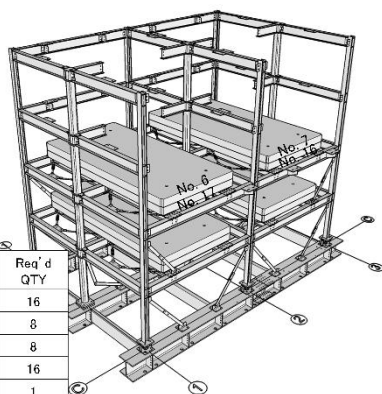
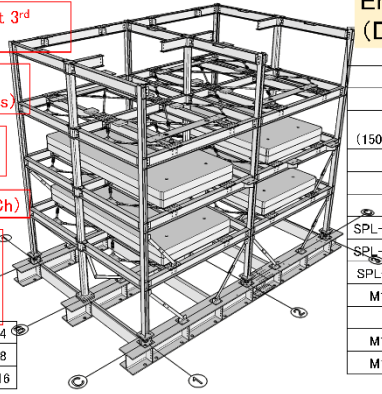
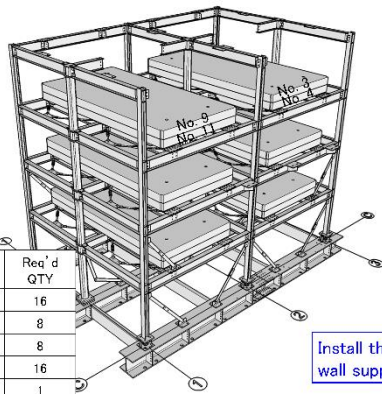
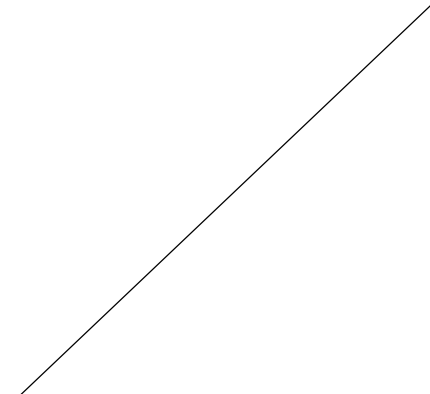
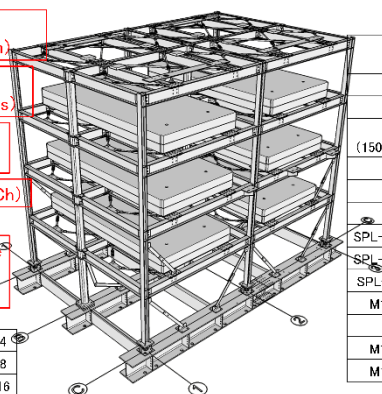
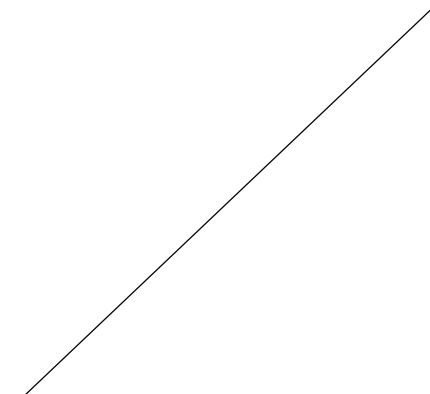
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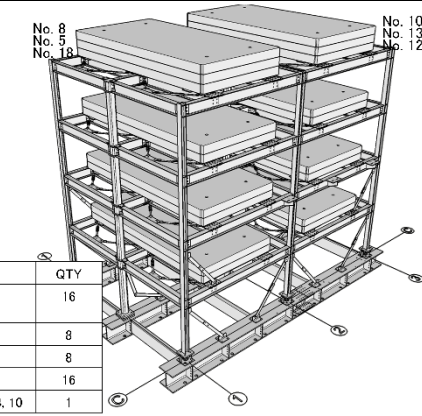
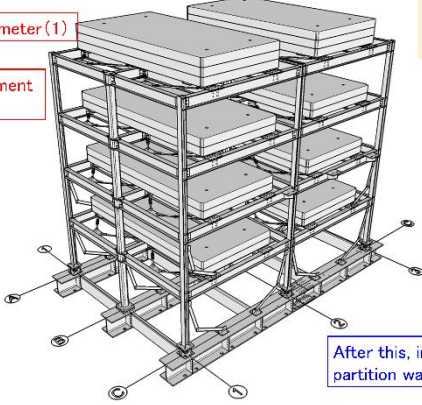
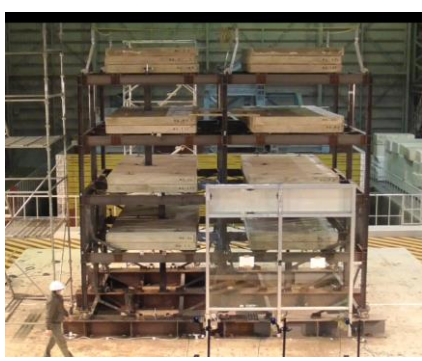
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Prepare story drift instruments(4 locations) Prepare transducers (42 locations) Finish labeling the cables Attach strain gages to the Frame A columns(24 units) Protect the strain gage lead wires(24 units) <table border="1"> <thead> <tr> <th>Item</th><th>QTY</th></tr> </thead> <tbody> <tr> <td>PL-4.5 liner</td><td>18</td></tr> </tbody> </table>  <table border="1"> <thead> <tr> <th>Item</th><th>Req'd QTY</th></tr> </thead> <tbody> <tr> <td>GA-3</td><td>8</td></tr> <tr> <td>GA-5A 特3F</td><td>1</td></tr> <tr> <td>GA-5A</td><td>2</td></tr> <tr> <td>GA-5B 特3F</td><td>1</td></tr> <tr> <td>GA-5B</td><td>2</td></tr> <tr> <td>GA-6A 特3F</td><td>1</td></tr> <tr> <td>GA-6A</td><td>4</td></tr> <tr> <td>GA-6B 特3F</td><td>1</td></tr> <tr> <td>GA-6B</td><td>4</td></tr> </tbody> </table> <table border="1"> <thead> <tr> <th>Item</th><th>Req'd QTY</th></tr> </thead> <tbody> <tr> <td>CT-1(150 × 150 × 6.5 × 9)</td><td>12</td></tr> <tr> <td>SPL-2C</td><td>12</td></tr> <tr> <td>M16 bolt(L50)</td><td>96</td></tr> <tr> <td>M16 bolt(L45)</td><td>72</td></tr> <tr> <td>M16 bolt(L40)</td><td>72</td></tr> </tbody> </table>	Item	QTY	PL-4.5 liner	18	Item	Req'd QTY	GA-3	8	GA-5A 特3F	1	GA-5A	2	GA-5B 特3F	1	GA-5B	2	GA-6A 特3F	1	GA-6A	4	GA-6B 特3F	1	GA-6B	4	Item	Req'd QTY	CT-1(150 × 150 × 6.5 × 9)	12	SPL-2C	12	M16 bolt(L50)	96	M16 bolt(L45)	72	M16 bolt(L40)	72	
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Large-Deformation Behavior and Failure Process of Steel Moment-Resisting Frames Examined by a Shake-Table Test

Martin SOMARRIBA
Candidate for the Degree of Doctor of Engineering
Supervisor: Dr. Taichiro Okazaki
Division of Architecture and Structural Design

Introduction

Steel moment-resisting frames are popular structural systems in seismic areas. However, seismic excitations produce large deformations that eventually lead to member damage, degradation of strength and stiffness, and collapse. Few studies have been conducted in the past to examine the behavior of steel moment-resisting frames to very large deformations, i.e., story drift ratios greater than 0.10 rad. Furthermore, ductile moment-resisting beam-to-column connections are important to achieve good performance against earthquakes. This research studies the behavior of steel moment-resisting frames through cyclic static tests on beam-to column moment connection subassemblages (component-level response) and dynamic shake-table tests of a scaled moment-resisting frame (system-level response). All the specimens in this study used the current detailing of the Japanese construction practice.

Research Background

Most experimental studies on the collapse of steel structures used simplified models. Rodgers et al. [1] showed that large amplitude pulses in the excitation and large number of fractured beam-to-column connections are significant factors leading to collapse. Lignos et al. [2] evaluated the reduction of collapse capacity due to P- Δ effects and component deterioration. Matsumiya et al. [3] conducted cyclic-loading tests of a three-story building using the post-Kobe Japanese design and construction practice completed with composite floor slabs. The frame sustained a maximum story drift ratio of 0.06 rad in the first story before yielding and local buckling of both ends of the first story columns caused a 50% reduction in story shear.

Large shake table tests by Suita et al. [4] subjected a four-story steel building to bidirectional ground motions. After sustaining a story drift ratio of 0.025 rad at the first story, shear strength degradation due to local buckling of the first story columns lead the structure to rest on a perimetral safety system at 0.16 and 0.08-rad story drift ratio in the longitudinal and transverse direction, respectively, and a total loss lateral strength. Despite proportioning the members with a strong column-weak beam ratio of about 1.50, such collapse mode was expected because the no-weld-access-hole detail adopted in the beam-to-column connection would increase the demands in the columns. Based on these tests, Lignos et al. [5] suggested that a strong-column/weak-beam ratio

of about 2.0 may improve the collapse capacity and avoid a single-story mechanism.

Beam-to-Column Moment Connections to an I-section Column

An experimental and analytical study was conducted on moment connections to an I-shaped column primarily to fill the gap in knowledge due to the scarce experimental data when I-section columns are used.

Fig. 1 shows the details of the six specimens: three for moment connection to the column flange (F1, F2 and F3) and three for moment connection to the column web (W1, W2, and W3).

Specimen W3 used tougher and stronger SN490B continuity plates. The Specimens were subjected to cyclic loading according to the protocol specified in Section K2 of AISC Seismic Provisions [6].

As shown in a, the F-Specimens exhibited large local buckling deformation of the beam flanges and web. Before local buckling, cracks were detected in the groove weld joining the beam flanges to the column. The cracks formed at the toe of the weld groove at the ends of the beam flange. In Specimen F1, the only F-Specimen without doubler plate, the cracks hardly grew. As shown in b, the cracks in Specimens F2 and F3 propagated straight along the interface between the CJP groove weld and beam flange.

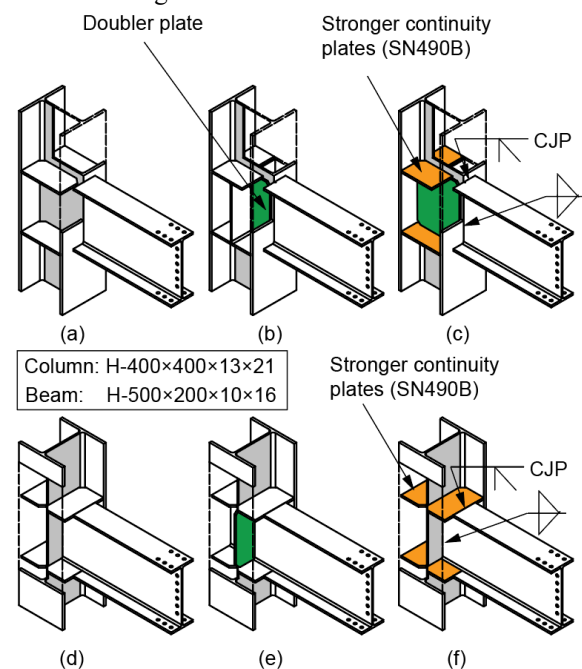


Fig. 1 Beam-to-column moment connection test specimens: a) F1, b) F2, c) F3, d) W1, e) W2, and f) W3.

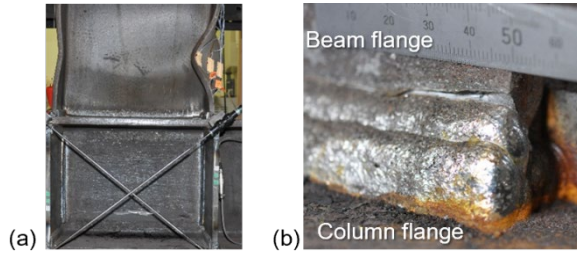


Fig. 2 Observed phenomena in the connections to the column flange: a) local buckling, and b) crack at the groove weld termination.

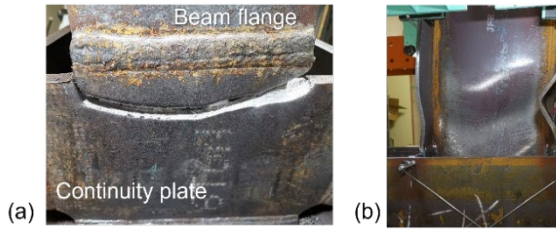


Fig. 3 Observed phenomena in the connections to the column web: a) Fracture of the continuity plate of Specimen W2, and b) local buckling of Specimen W3.

The W-Specimens were prone to fracture of the continuity plate initiating at the termination of the beam flange groove weld, as shown in Fig. 3a. Specimen W2 failed by sudden and complete fracture after that crack developed to about 20% of the width of the beam flange. Specimen W3 formed a very similar crack at the corner between the beam flange and continuity plate, but the cracks did not grow to a substantial size. Specimen W3 formed cracks along the groove of the beam flange, very much like the F-Specimens, which grew to a larger size than the cracks into the continuity plate.

Fig. 4 shows the measured moment at column face versus story drift ratio of the six specimens. Comparing Fig. 4a–c with Fig. 4f, the deformation capacity and strength of

the F-Specimens were identical to the W-Specimen which continuity plate did not fracture. Furthermore, all the Specimens except W2 survived the cycles of 0.04-rad story drift ratio while keeping the strength above $0.80M_p$. Therefore, the connections met the ductility requirements for Special Moment Frames per AISC Seismic Provisions. Following the same provisions, W2 can be regarded as Intermediate Moment Frame.

Finite-element-method analysis of each tested specimen was performed using the general-purpose analysis software ADINA Ver.9.6 to further investigate the causes of damage.

Fig. a shows the principal stresses near the critical regions of the connections to the column web, when the beam flange is subject to tension at the first excursion of the ± 0.03 -rad cycle. At this stage, marked by “▲” in Fig. d and e, the cracks observed in Specimens W1 and W2 propagated rapidly into the continuity plates.

All three web-connection numerical models showed notably high stress at the corner between the beam flange and continuity plate, at the location where all three specimens formed a crack. It is also noted that the large principal stress acted perpendicular to the corner, to open any crack formed at this location, as reported for Specimens W1 and W2. Although little difference in stress distribution was confirmed between the three models, the maximum principal stress was higher in Model W2 (803 N/mm^2) than in Models W1 and W3 (772 and 770 N/mm^2 , respectively).

The right corner between the beam flange and continuity plate, as well as cracks formed at this location, act as a notch placed perpendicular to such principal stress. The continuity plates were equal to the beam flanges in thickness. Therefore, it can be seen that the continuity plates in Specimens W1 and W2, which had lower yield strength than the beam flange (293 N/mm^2 compared to 321 N/mm^2) yielded while the continuity plates in Specimen W3, which had higher yield strength than the beam flange (412 N/mm^2 compared to 321

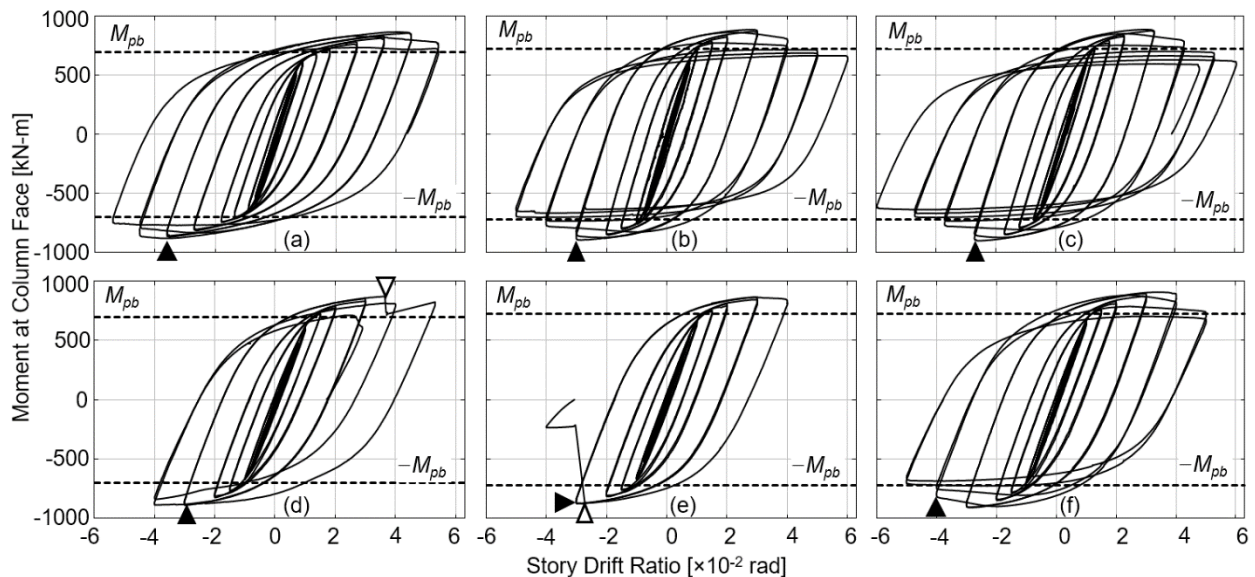


Fig. 4 Response of Specimens: a) F1, b) F2, c) F3, d) W1, e) W2, and f) W3.

N/mm²) did not yield as substantially. Consequently, the significantly different performance between Specimens W2 and W3 may be attributed to whether the continuity plates yielded during the test.

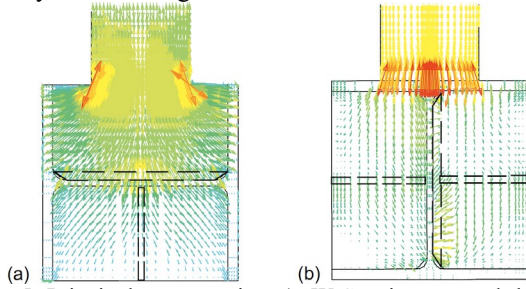


Fig. 5 Principal stresses in: a) W-Specimens, and b) F-Specimens.

Shake-Table Collapse Tests

Test Plan

Fig. 6a shows the elevation of the specimen, which was a 2/5-scale, 2-bay, 4-story steel moment-resisting frame fixed at the column bases. All stories were 1.25 m in height and 3 m in span. Table 1 lists the beams and columns. The columns used JIS (Japanese Industry Standard) STKR designation, cold-formed, square-hollow-structural sections (HSS), and the beams used JIS SS400 I-sections. To supply the targeted inertia, concrete weights, approximately 2.8 ton each, were fastened to the floor beams through high strength PC-rods. The total 50-ton mass of the entire specimen was supported by the main frame and two gravity frames aligned in parallel.

As shown in Fig. 6b, the gravity frames had pinned bases, and their beams were provided with slotted-hole shear connections to minimize lateral load resistance of the gravity frame. Fig. 6c illustrates the rigid diaphragm formed at each floor, connecting the top flange of the beams through turnbuckles, to transfer all inertia to the main frame. As shown in Figs. 6a and 6b, sagged braces were placed in the lower stories which would engage at very large story drifts to protect the shake table from collapse of the specimen. As in the Japanese construction practice of low- to mid-rise steel buildings, beam stubs were shop welded to the columns, and the frame was completed on the site by splicing the central 1500-mm beam segments through high-strength bolts tightened to slip-critical condition.

Fig. 7 details the beam-to-column moment connection adopted in the main frame. The Complete Joint Penetration (CJP) groove welds at the beam flanges did not require weld access holes. First, backing bars split at the beam web were tack-welded inside the beam flange bevel. Next, the beam web was connected to the column by fillet welds. Last, the beam flanges were connected to the diaphragm by CJP groove welds.

Table 1 lists the mechanical properties of steel established from coupon tests. The measured yield strengths were 1.8, 1.7, and 1.3 times the nominal for columns C1 and C2, and beams G1 and G2. The members were proportioned such that the ratio or sum of plastic

moments between the column and beam from nominal strength values are 2.3 and 1.1, respectively, for the exterior and interior joints.

The plastic strength between the panel zones and the sum of plastic moments of the beams were 1.7 for the external and 0.8 for the internal joints. However, due to the higher-than-expected strength of the columns, the panel zone-to-beam strength ratio from measured properties resulted in 2.3 and 1.1 for the external and internal joints, respectively.

The JMA Kobe NS record was applied in the plane of the specimen. The motion was gradually increased from 10% to 100%, and the 100% excitation was repeated 4 times.

Uniaxial elastic strain gages were placed on multiple sections of each beam and column to enable sampling of internal force distribution. Strain gages were also placed on the first and second-story columns of the gravity frames. Displacement transducers were placed at each story between the top of the concrete weights or foundation beam and upper beam to measure relative story drift. Displacement transducers were used to measure beam-end rotation, panel-zone shear deformation, and slippage of the column base plates. Uniaxial accelerometers were attached to the concrete masses at each story.

Test Results

Fig. 8 shows the transition of the specimen's first and second natural vibration periods identified from white noise excitations conducted between primary excitations and from the tail of the acceleration response when white noise was not available. The period remained practically constant even after the 50% excitation when the second and third-floor beams and the column bases yielded substantially as mentioned later. This result suggests that steel systems can retain their elastic properties even after experiencing significant yielding of primary components. The natural period elongated to 0.64 s after the 4th 100% excitation due to fracture of the second-floor beams.

Fig. 9 presents the bending moment distribution normalized by the plastic strength M_p based on strain gauge measurements sampled from the 50%, 1st 100%, 3rd 100%, and 4th 100% excitations at peak deformation. The figure indicates the zone where the plastic strength was exceeded, as well as the locations where the fracture occurred. During the 50% excitation, the second and third-floor beams developed plastic hinges at both ends and the first-story columns yielded at the bases. During the 1st 100% excitation, the system formed a sidesway mechanism involving the lower three stories. Yielding continued to spread during the subsequent 100% motions until the bottom flange of the external joint beams on the second floor fractured. During the 3rd 100% excitation, a crack formed at the termination of the CJP groove weld, at the location indicated in Fig. 10, then propagated along the weld bevel until it rested 25 mm (12% of the beam depth) into the web. During the 4th 100% excitation, the beam bottom flange completely fractured to the state shown in Fig. 8b.

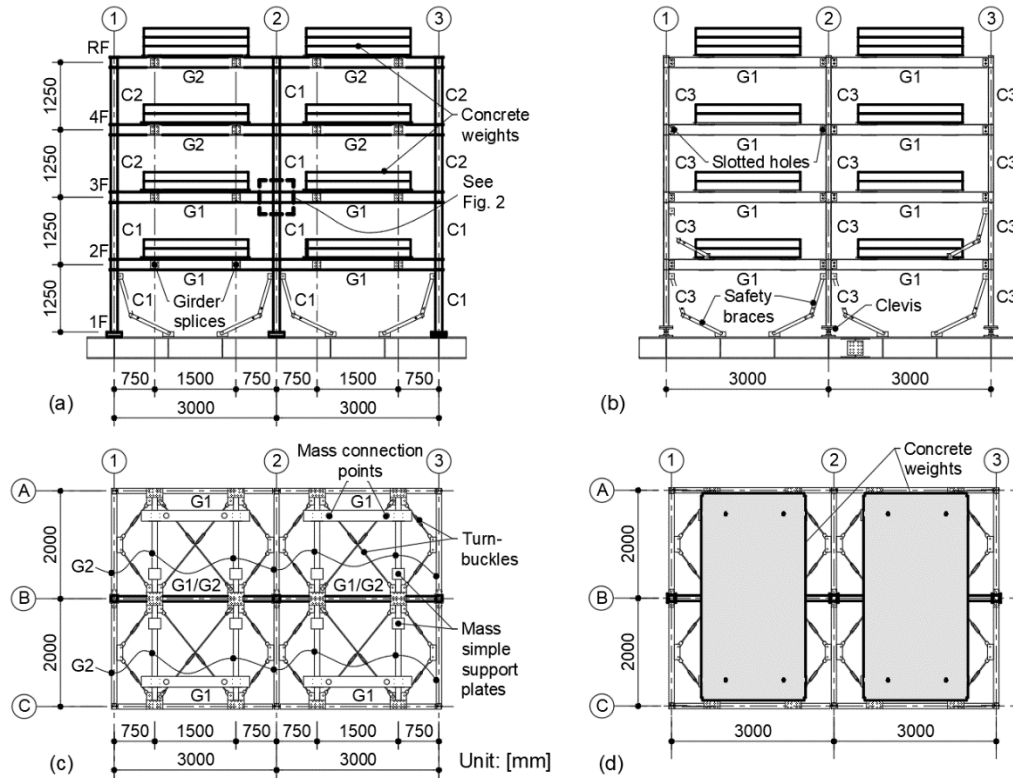


Fig. 6 Test specimen: a) Main frame elevation, b) Gravity frame elevation, c) Floor diaphragm, and d) Placement of concrete weights in plan.

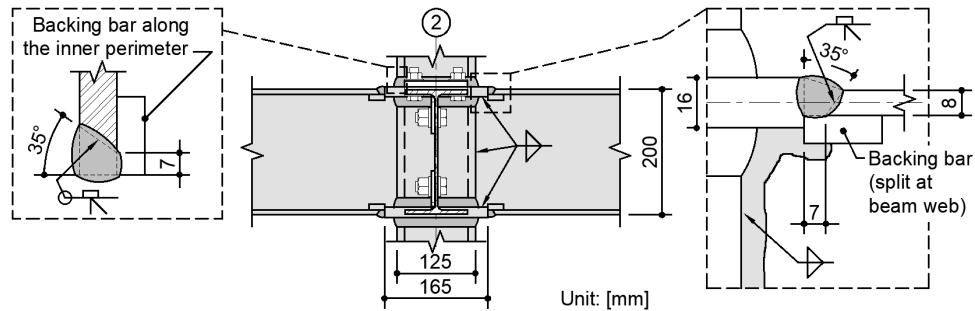


Fig. 7 Beam-to-column moment connection with no weld-access-hole at flange connection.

Table 1 Column and beam schedule and specified minimal mechanical properties.

Member	Dimensions	Grade	F_y [N/mm ²]	F_u [N/mm ²]
C1	HSS-125×125×12	STKR400	245	400
C2	HSS-125×125×9	STKR400	245	400
C3	H-100×100×6×8	SS400	245	400
G1	H-200×100×5.5×8	SS400	245	400
G2	H-198×99×4.5×7	SS400	245	400

Fig. 11a plots the base moment at the central column vs. story drift response of the first story during the four 100% excitations. Despite the large story drift of +0.08 and -0.02 rad induced by each of the 100% excitations, no strength degradation occurred in this column. The normalized P-M interaction in Fig. 11b indicates no significant variation of column axial load for the interior column ($-0.07, -0.06P_y$), whose initial axial due to gravity loads was 6% of its axial yield strength. The columns exhibited very ductile behavior showing no sign

of local buckling and developing no sign of fracture in the CJP groove welds to the column base plate.

Component Tests

Three beam-and-column subassemblages were constructed simultaneously with the shake-table specimen using the same heat of steel, design, and detailing.

As shown in Fig. 12, the subassemblages represented two external joints (T1 and T2) and one internal joint (T3). During the tests, the subassemblages were laterally braced at the same locations where the moment-resisting beams of the shake-table test specimen were braced by orthogonal beams.

The subassemblages were subjected to cyclic loading to a story drift ratio of ± 0.05 rad according to the loading protocol for beam-to-column connections defined by the AISC Seismic Provisions Sect. K2. Subsequently, the loading cycles were skewed to one direction to a maximum of -0.125 rad.

Fig. 13 shows the beam-end moment versus beam rotation obtained from the three Specimens. Specimen T3 is represented by the right beam noting that the left beam showed nearly identical behavior but fracture occurred

only in the right beam. All specimens behaved similarly. Local buckling of the beam became prevalent during the second $+0.05$ -rad excursion (marked by $\triangle$), followed by crack initiation in the top beam flange at the termination of the CJP groove weld during the subsequent half cycle ($\blacktriangle$). The “ $\times$ ” in Figs. 13b and 13c indicate fracture of the beam top flange of Specimen T2 and the bottom flange of the right beam of Specimen T3.

Fig. 14 shows the same relationships obtained from corresponding beams in the shake-table test during the four 100% JMA Kobe N-S excitations. The shake-table test specimen developed similar flexural strength at the connections and similar damage progression as the subassemblages. However, unlike the subassemblages, the shake-table test specimen exhibited no significant degradation in strength or stiffness before fracture. This

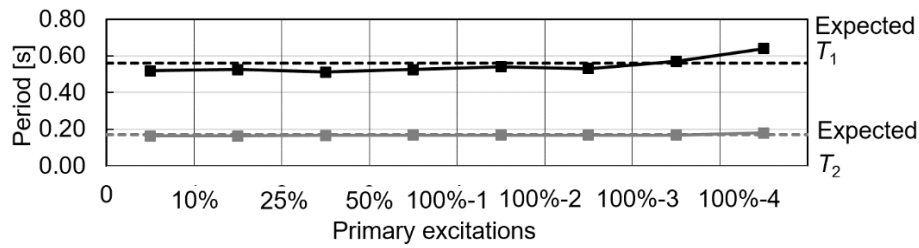


Fig. 8 Variation of the natural period between excitations.

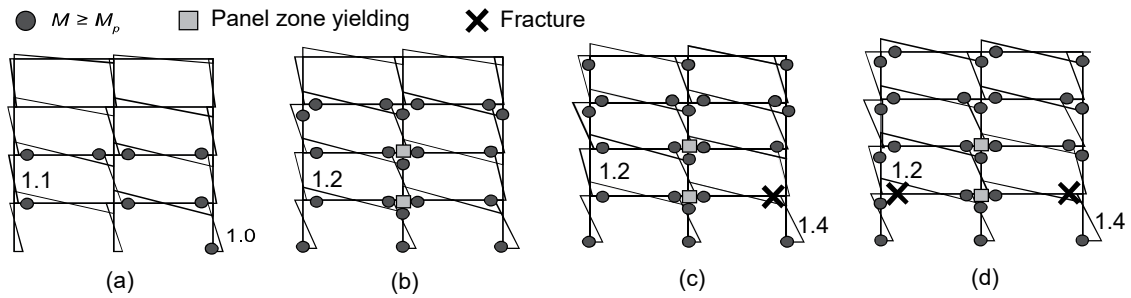


Fig. 9 Normalized bending moment diagram at maximum deformation and damage propagation at the excitations: a) 50%, b) 1st 100%, c) 3rd 100%, and d) 4th 100%.

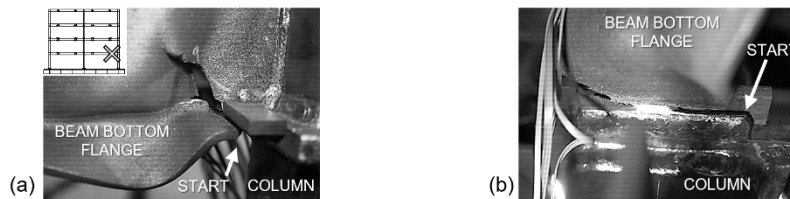


Fig. 10 Fracture of 2F beam at the end of the tests: a) front view, and b) bottom view.

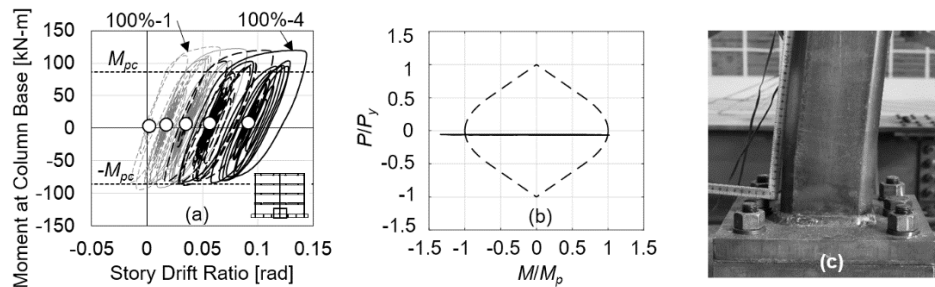


Fig. 11 Response of the interior column base during the 100% excitations: a) Moment at column base vs. story drift ratio, b) P-M interaction, and c) deformation at the end of the tests.

difference may be attributed to the axial restraint of the beam by surrounding members that was present in the shake-table tests but not in the subassemblages.

Conclusions

The conclusions drawn from the shake-table tests of a scaled steel moment-resisting frame and corresponding beam-to-column subassemblages are:

- A strong column/weak beam ratio of 3.0 in the first story joints was adequate to form a sideway plastic mechanism involving the first three stories, as it was predicted in the design process.
- Cracking at the termination of the CJP welds progressed slowly and their effect on either the beam or the system response was not evident until complete fracture of the bottom flange happened.
- Despite sustaining large deformations up to 0.15 rad story drift ratio, the system did not collapse due to the very high strength of the columns.
- The system retained its fundamental period even after the significant yielding of primary components but elongated after beam fracture.
- A comparison of the component tests with the shake-table tests showed good agreement in strength capacity at the beam-to-column connection level but differed in

terms of degradation. The difference may be attributed to axial deformation restraint in the shake-table test beams.

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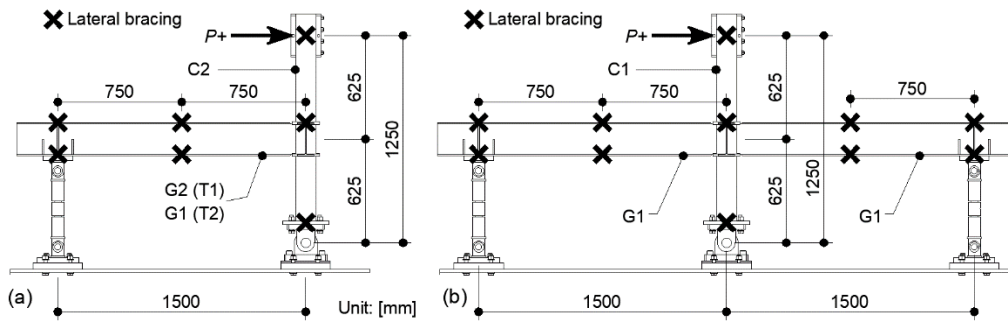


Fig. 12 Beam-to-column subassemblages: a) T1 and T2, and b) T3.

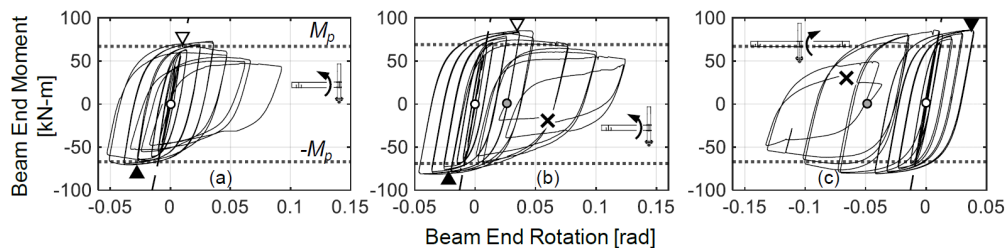


Fig. 13 Beam-end moment vs. rotation response of Specimens: a) T1, b) T2, and c) T3 (right).

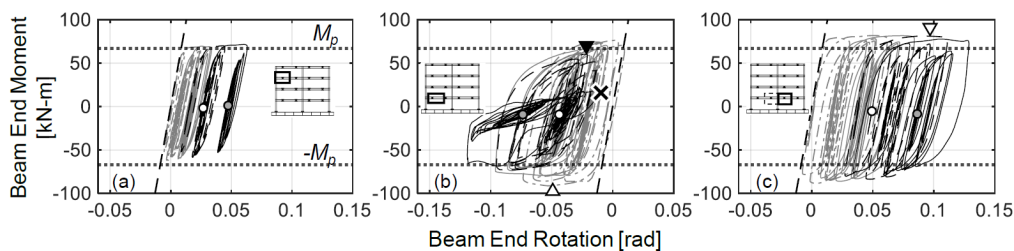


Fig. 14 Beam-end moment vs. rotation response of the shake-table test at: a) 4F external joint, b) 2F external joint, and c) 2F internal joint.