DEVELOPMENT OF A 10-ELEMENT HYBRID SIMULATION PLATFORM AND ITS APPLICATION TO SEISMIC PERFORMANCE ASSESSMENT OF MULTI-STOREY BRACED FRAMES

by

Saeid Mojiri

A thesis submitted in conformity with the requirements for the degree of Doctor of Philosophy
Graduate Department of Civil and Mineral Engineering
University of Toronto

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Saeid Mojiri
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ABSTRACT
This study presents a 10-element hybrid (experimental-numerical) simulation platform, referred to as UT10, which was developed for running pseudo-dynamic hybrid simulations of braced frames with up to 10 large-capacity physical brace specimens. This study presents the details of the development of different components of UT10 including a network interface for actuator controller, NICON-10. An adjustable brace specimen, referred to as Adjustable Yielding Brace (AYB), was designed to simulate the hysteretic response of yielding braces such as buckling-restrained braces (BRBs) thus facilitating the seismic performance evaluation of multi-storey structures with hysteretic energy dissipative braces through hybrid simulations. Also, a buckling specimen was designed to simulate the hysteretic response of conventional buckling braces. Both AYB and buckling specimens were cyclically tested in UT10. The results indicated that these specimens are capable of producing hysteretic responses with characteristics similar to BRBs and conventional braces. A five-storey buckling-restrained braced frame (BRBF) and a special concentrically braced frame (SCBF) were designed and tested, respectively, with AYB specimens and buckling specimens representing the braces. Cyclic tests of the AYB and buckling specimens, 1- and
3-element hybrid simulations of the BRBF, and 2- and 4-element hybrid simulations of the SCBF inside UT10 confirmed the functionality of UT10 for running hybrid simulations on multiple specimens. Comparison of the results of the hybrid simulations of the BRBF and SCBF with their fully numerical models showed that the modelling inaccuracies of the yielding braces could affect the global response of the multi-storey braced frames further emphasizing the need for experimental calibration or hybrid simulation for achieving more accurate response predictions. UT10 provides a simple and reconfigurable platform that can be used to achieve a realistic understanding of the seismic response of multi-storey frames with yielding braces, distinguish their modelling limitations, and improve different modelling techniques available for their seismic response prediction.
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<th>Description</th>
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<tbody>
<tr>
<td>ACTIA</td>
<td>Application for Camera Triggering and Image Acquisition</td>
</tr>
<tr>
<td>ADC</td>
<td>Analog to Digital Converter</td>
</tr>
<tr>
<td>AYB</td>
<td>Adjustable Yielding Brace</td>
</tr>
<tr>
<td>BRB</td>
<td>Buckling-Restrained Brace</td>
</tr>
<tr>
<td>BRBF</td>
<td>Buckling-Restrained Braced Frame</td>
</tr>
<tr>
<td>CBF</td>
<td>Concentrically Braced Frame</td>
</tr>
<tr>
<td>CP</td>
<td>Collapse Prevention</td>
</tr>
<tr>
<td>CV</td>
<td>Coefficient of Variation</td>
</tr>
<tr>
<td>DAC</td>
<td>Digital to Analog Converter</td>
</tr>
<tr>
<td>DBE</td>
<td>Design Basis Earthquake</td>
</tr>
<tr>
<td>DC</td>
<td>Displacement-Controlled (in actuator control)</td>
</tr>
<tr>
<td>DC-E</td>
<td>Displacement-Controlled with External signals (in actuator control)</td>
</tr>
<tr>
<td>DOFs</td>
<td>Degrees of Freedom</td>
</tr>
<tr>
<td>FE</td>
<td>Frequent Earthquake</td>
</tr>
<tr>
<td>FC</td>
<td>Force-Controlled (in actuator control)</td>
</tr>
<tr>
<td>FC-S</td>
<td>Force-Controlled with force command provided from force measurements of another actuator (in actuator control)</td>
</tr>
<tr>
<td>IO</td>
<td>Immediate Occupancy</td>
</tr>
<tr>
<td>LP</td>
<td>Linear Potentiometer</td>
</tr>
<tr>
<td>LS</td>
<td>Life Safety</td>
</tr>
<tr>
<td>MCE</td>
<td>Maximum Considered Earthquake</td>
</tr>
<tr>
<td>MRF</td>
<td>Moment Resisting Frame</td>
</tr>
<tr>
<td>NICON</td>
<td>Network Interface for Controllers</td>
</tr>
<tr>
<td>NICON-10</td>
<td>Network Interface for Controllers for 10 independent degrees of freedom</td>
</tr>
<tr>
<td>NRHA</td>
<td>Nonlinear Response History Analysis</td>
</tr>
<tr>
<td>R</td>
<td>Rigid links: Displacement-controlled actuators with a constant displacement command</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>R-C</td>
<td>Rigid link actuators that are set to be in compression</td>
</tr>
<tr>
<td>R-T</td>
<td>Rigid link actuators that are set to be in tension</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
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<tr>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>SCBF</td>
<td>Special Concentrically Braced Frame</td>
</tr>
<tr>
<td>SDR</td>
<td>Storey Drift Ratio</td>
</tr>
<tr>
<td>SET</td>
<td>Shell Element Tester</td>
</tr>
<tr>
<td>SFRS</td>
<td>Seismic Force Resisting System</td>
</tr>
<tr>
<td>UT-SIM</td>
<td>University of Toronto Simulation (UT-SIM) framework</td>
</tr>
<tr>
<td>UT10</td>
<td>University of Toronto’s 10-element hybrid simulation platform</td>
</tr>
<tr>
<td>YBS</td>
<td>Yielding Brace System</td>
</tr>
</tbody>
</table>
CHAPTER 1: INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Current seismic design standards allow structures or some of their components to be damaged to absorb and dissipate seismic energy and hence protect their occupants and contents. Performance-based seismic design of structures, which has been the preferred design methodology over the last two decades, requires a realistic evaluation of the actual nonlinear performance of different seismic force resisting systems (SFRS) under different levels of seismic intensity (ATC, 2006). This has increased the importance of experimental earthquake engineering and experimental data as the most realistic source of information in the achievement of such understanding (Filiatrault et al., 2013).

Advancement of computational technology and improvements in design and analysis guidelines has made it possible for practicing engineers to use nonlinear dynamic analysis as a more common tool for design and analysis of new and existing structures. However, simplified nonlinear models are generally used and prescribed for the nonlinear and energy dissipating elements in SFRS. Multi-storey braced frames are among the most frequently used SFRS which derive their lateral stiffness and strength from the hysteretic response of their bracing elements. Therefore, the global nonlinear response of these SFRS is predominantly affected by the nonlinear hysteretic behaviour of the braces and the inaccuracies in the numerical modelling of the braces can significantly impact the global dynamic response of the braced frames. Experimental testing methods like hybrid simulation techniques can aid in providing a realistic system- and component-level response of SFRS based on the actual response of the most important nonlinear structural elements in the system. In the context of performance-based design, these results can be used as the reference data to identify the implications of using simplified models on the global and local response of the structures. Furthermore, they can aid the research community in their ongoing investigations, like the ATC-114 project (Hamburger et al., 2018), to introduce more sophisticated and realistic models for nonlinear elements that can be used in a robust nonlinear dynamic analysis of structures.

Hybrid simulation is one of the structural testing methods in which the experimental test data from the critical elements is integrated within an entire numerical structural system thus providing more realistic predictions of structural response in comparison to a fully numerical simulation. This is achieved by applying the predicted displacements to the physical elements (experimental or physical substructures) and then integrating the resulting displacement and force feedback from the physical elements into the equations of motion in the numerical model of the rest of the structure (numerical substructure) (Saouma and Sivaselvan, 2008; McCrum and Williams, 2016). Since its early development in 1969 and the 1980’s (Hakuno et al., 1969; Mahin and Shing, 1985; Takanashi and
Chapter 1: Introduction

Nakashima, 1987) hybrid simulation techniques have been constantly improved in different aspects such as numerical integration schemes, actuator control, delay compensation, etc. and it has matured over the years such that more emphasis is currently applied on extending its application in seismic performance assessment of different structural systems (McCrum and Williams, 2016). However, the application and implementation of this technique in the laboratories are still rather restricted due to the complex nature of the simulation method. Development of a user-friendly and reconfigurable hybrid simulation platform is, therefore, one of the necessary steps towards further development, implementation, and application of the hybrid simulation method in the future (Shao and Griffith, 2013).

One of the questions that arise when performing hybrid simulation is the level of accuracy improvements that can be achieved by this method and if these improvements are large enough to justify the need for adopting this complex technique. The accuracy improvement gained through hybrid simulation highly depends on the type of SFRS. For example, for base-isolated structures the global response is dominated by the hysteretic behaviour of isolation bearings and hence physically testing the bearings in the hybrid simulation will greatly improve the accuracy of the response prediction. On the other hand, for SFRS like multi-storey braced frames, the nonlinear response is distributed along the height of the structure in the braces which collectively contribute to the strength and stiffness of SFRS. For these types of SFRS, physically testing only a few structural elements may not significantly improve the overall accuracy of the simulation. On the other hand, testing a large number of structural elements not only requires more extensive laboratory resources (e.g. hydraulic actuators, laboratory space, instrumentation, etc.), but also poses further challenges in the control of the actuators, experimental measurements, and handling of the potential errors. Therefore, the capabilities of the hybrid simulation framework need to be extended to test a larger number of physical components simultaneously. Approaches like the efficient selection of the physical substructures or model-updating methods in which experimental data can be used to update numerical elements in each time step of simulation can also be employed to further increase the efficiency of the hybrid simulation (Kwon and Kammula, 2013; Elanwar and Elnashai, 2016).

Concentrically braced frames (CBFs) are a class of SFRS that achieve their lateral stiffness and strengths mainly from axially loaded structural elements called braces. CBFs are regarded as one of the most economical systems to resist seismic loads in areas of any seismicity. They are most suitable for low-rise buildings in areas of high seismicity. Small lateral deflections and rather simple design of these systems make them attractive over alternative systems like moment resisting frames (MRFs). Application of CBFs has increased dramatically during the last two decades. Considerable improvements were made in the design of CBFs after significant damage was observed during several earthquakes in the 1980’s and 1990’s. These improvements led to the modern designs of CBFs and emergence of two new SFRSs: special concentrically braced frames (SCBF) and buckling restrained braced frames (BRBF). These systems provide reliable nonlinear response and deformation capacity during an earthquake. Both SCBF and BRBF are among the most popular SFRS in North America with the rapidly increasing application. The intended energy dissipation mechanism for SCBF is tensile yielding and compressive buckling of the braces while for BRBF it is yielding of the buckling restrained braces (BRB) both in tension and compression (Bruneau et al., 2011).

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There are several applications of hybrid simulations reported in the literature for investigating the seismic performance of SCBFs and BRBFs. Fahnestock et al. (2007) performed hybrid simulations to investigate the seismic performance of BRBFs. The physical substructure in their tests was a 60%-scale four-storey frame equipped with eight BRBs installed in chevron configuration and the displacement commands were applied at each floor level on the frame by multiple actuators. Although it provided realistic boundary conditions for the braces, various factors such as the laboratory space and cost/complexity of constructing the frame pose limitations on the number of stories and the scale of the tested frame in this testing approach. Following the same approach, Tsai et al. (2008) performed hybrid simulations of a full-scale three-storey BRBF with concrete-filled steel tube columns, which was one of the largest tests of this type. Tsai et al. (2013) did a hybrid simulation on a full-scale three-storey single-bay SCBF specimen and compared the results with the numerical analysis predictions. More recently, Wu et al. (2017) used hybrid simulation to investigate the seismic performance of a new implementation of BRBs in reinforced concrete (RC) frames. For this purpose, they tested a full-scale two-storey RC frame with BRBs installed in diagonal configurations. More details and reviews of the literature on steel BRBFs and SCBFs are presented in Chapters 4 and 5 of this report. A state-of-the-art review of the seismic design of steel structures including BRBFs and SCBFs is presented by Uang and Bruneau (2018).

Component- and system-level tests on SCBFs and BRBFs reported in the literature indicate that if the undesired modes of failure such as gusset plate buckling and fracture or BRB global buckling are prevented in the capacity design of the braced frames, the hysteretic response of the braces tested as isolated components is similar to their response when installed in a frame. Therefore, multi-element hybrid simulations on braced frames with only the braces tested as physical specimens can be well representative of the realistic response of the braced frames if the undesired failure modes are accounted for and the response of the brace elastic components and connections are explicitly modelled in the numerical substructure. Such approach facilitates testing braced frames with more storeys and a larger number of braces than previously tested. The results can not only contribute to a more realistic understanding from the actual seismic performance of braced frames with the larger number of storeys but also provide the benchmark test data to assess the accuracy of the existing numerical modelling techniques under earthquake loads that can be used for realistic seismic performance assessment of braced frames.

1.2 SCOPE AND OBJECTIVES

The above discussions outline the need for the development of a robust hybrid simulation platform to perform multi-element hybrid simulations of multi-storey braced frames. The main objectives of the research project presented in this report were the following:

- Development of a user-friendly, reconfigurable, and robust hybrid simulation platform to perform multi-element hybrid simulations of multi-storey braced frames with rate-independent hysteretic energy dissipating braces
- Development of a reusable yielding specimen with adjustable strength and hysteretic properties that can simulate the cyclic hysteretic response of hysteretic energy dissipative braces like a large-scale BRB
Chapter 1: Introduction

- Application of the multi-element hybrid simulation platform in seismic performance assessment of multi-storey SCBFs and BRBFs
- Study the impact of brace numerical modelling inaccuracies on local and global seismic response parameters in SCBFs and BRBFs
- Seismic performance assessment of SCBFs and BRBFs using state-of-the-art numerical models and the multi-element hybrid simulation results

1.3 THESIS CONTENT

This report presents the development of the University of Toronto’s 10-element hybrid simulation platform (UT10 hereafter) which is capable of testing and integrating the response of up to 10 large-capacity physical brace specimens into a hybrid simulation. UT10 uses the recently developed UT-SIM framework (Huang and Kwon, 2018; Mortazavi et al., 2017a, 2017b; Mojiri et al., 2015) to integrate the models through network communication. An open-source network interface program for actuator controllers with 10 independent controlled degrees of freedom, called NICON-10, is developed which combined with UT-SIM provides a reconfigurable framework to communicate the displacement commands and displacement/force feedback between the numerical substructure and the actuator control system through network communications. A reusable and adjustable brace specimen, referred to as adjustable yielding brace (AYB), was designed and fabricated to be used with the UT10. The brace was designed such that it had adjustable stiffness and strength. AYB was designed to simulate the hysteretic response of yielding braces such as BRBs thus facilitating seismic performance evaluation of multi-storey structures with hysteretic energy dissipative braces through hybrid simulations. In addition, a buckling specimen was designed to simulate the axial force-displacement hysteresis of conventional buckling braces in SCBFs. Cyclic tests were performed on the AYB specimen and the buckling specimen to evaluate their hysteretic response and to calibrate numerical models. A five-storey BRBF and a five-storey SCBF were designed and tested with AYB specimens and buckling specimens representing the braces. The results were compared with fully numerical models of the BRBF and SCBF and the effect of brace modelling inaccuracies on the seismic response predictions of these systems was investigated.

This report includes 6 chapters. Chapter 2 presents the development and performance verification of the UT10. Design challenges and details of the UT10 components including NICON-10 are presented in detail in this chapter. The cyclic and hybrid simulations performed in UT10 are briefly described and the test results pertaining to the performance and accuracy assessment of UT10 for multi-element hybrid simulations are discussed.

Chapter 3 presents the design details of the AYB specimen. Details of the cyclic test on a single-component and two six-component specimens are presented in this chapter and the results are compared with full-scale BRB cyclic test results from the literature. Advantages, potentials and limitations of AYB are also discussed.

Chapter 4 is about the seismic response assessment of BRBFs. A comprehensive review of the literature on past experimental and numerical seismic performance assessments of BRBFs is presented. Details of hybrid simulations

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on a 5-storey BRBF in UT10 with AYB specimens representing the physical braces are presented and the results are compared with fully numerical predictions using various BRB modelling techniques. A comprehensive study on the seismic performance of the BRBF is also presented in this chapter.

Chapter 5 discusses the seismic response assessment of SCBFs. This chapter starts with a comprehensive review of the literature on past experimental and numerical seismic performance assessments of SCBFs. Development and response verification of the buckling specimen with a hysteretic response similar to conventional braces is presented. The chapter continues with details of hybrid simulations on a 5-storey SCBF in UT10 with the buckling specimens representing the conventional braces. A comprehensive study on the seismic performance of the SCBF is also presented in this chapter.

Chapter 6 presents the summary of the report and the concluding remarks. A description of some of the ongoing and future experimental projects at the University of Toronto that are planned to use UT10 is presented in this chapter. Suggestions on future studies and developments are also presented and discussed in this chapter.

Appendix A presents a sample operation checklist for running a 4-element hybrid simulation with UT10. The checklist also includes the protocols for resuming an interrupted test, handling unexpected problems during a test, and managing the emergency shutdowns of the hydraulic system. Appendices B, C, and D present the design drawings of the UT10 support frame, AYB specimen, and the buckling specimen, respectively.

Some of the material presented in this report were published in the following journal/conference papers:


CHAPTER 2: DEVELOPMENT OF A 10-ELEMENT HYBRID SIMULATION PLATFORM

2.1 INTRODUCTION

This chapter presents the development and performance verification of UT10. This platform enables testing and integrating the response of up to 10 large-capacity physical brace specimens into an experimental-numerical hybrid simulation. There are numerous hybrid simulations reported in the literature on various types of physical substructures including braced frames. However, based on the author’s knowledge, UT10 facility is the largest of its kind when considering the number and capacity of the physical substructures. UT10 was built on the shell element tester (SET) facility which has been used during the last three decades for testing large-scale reinforced concrete shell elements at the University of Toronto. The existing slack in the actuator to specimen pin connections in SET created a challenge to accurately apply cyclic loads that are frequently experienced in hybrid simulations under earthquake loads. Furthermore, the actuators were designed and installed to operate in a coupled configuration and were mainly controlled in a force-controlled manner for the RC tests while in a hybrid simulation the loading actuators are expected to apply the axial loads on the brace specimens in an uncoupled configuration and in a displacement-controlled manner. The existing configuration of SET, simultaneous control of multiple loading actuators, high level of accuracy necessary in the hybrid simulations, and the required design flexibility needed for testing specimens with different behaviour and topology posed significant challenges for design and development of UT10 components.

This chapter starts with a description of the UT10 components. Challenges and details of the design and development of UT10 components including details on the hydraulic and control system, design of the UT10 support frame, types and configuration of specimens, and information on the numerical modelling platforms and data communication are presented. A significant portion of this chapter is devoted to the development of the network interface for actuator controller, NICON-10, that was developed as part of UT10 development. Various features of NICON-10, including an algorithm for displacement error compensation, are introduced and discussed. The cyclic and hybrid simulations performed in UT10 are briefly described and the results are used to verify the performance of UT10 for multi-element hybrid simulations. More details on these tests are presented in the next chapters. The chapter is concluded with a summary of the results and conclusions from the investigations presented in this chapter including a review of the advantages and limitations of UT10.
2.2 UT10 COMPONENTS

2.2.1 Actuators and control system

Forty in-plane and twenty out-of-plane servo-controlled hydraulic actuators are available within UT10. This system, called the SET, was initially designed and configured to test reinforced concrete shell elements in the Structural Testing Facility at the University of Toronto (Krischner, 1986; Ruggiero, 2015). Figure 2.1 shows an illustration of the in-plane and out-of-plane actuators. The top 15 actuators (10 vertical in-plane and 5 horizontal out-of-plane) can potentially be used to apply displacement commands on specimens in hybrid simulations. For hybrid simulation on 10 brace specimens in UT10, the specimens are controlled by the top 10 vertical in-plane actuators (see Figures 2.1b and c) which impose numerically predicted displacements to the brace specimens independently. The rest of the in-plane and out-of-plane actuators are either not used or are used to provide support for the UT10 support frame and brace specimens.

Twelve pumps are available in the Structural Testing Facility that can provide a supply of up to 360 gallons per minute (GPM) oil. The nominal pressure provided by the pumps in the high pressure mode is 3,000 psi. This pressure is stabilized by 12 hydraulic accumulators during the tests. A hydraulic service manifold (HSM) is also used that provides independent pressure regulation for the actuators and isolates UT10 hydraulic system from the rest of the equipment in the laboratory. The 3,000 psi oil pressure can be further increased to 4,500 psi with a booster pump. With this boosted pressure, each of the in-plane and out-of-plane actuators are able to simultaneously apply 800 kN and 400 kN force, respectively.

The actuators are controlled by an MTS® Flex Test 200 controller. The controller provides 60 control channels that can simultaneously control the actuators independently in force- or displacement-controlled configuration through PID algorithms. The controller is interfaced on two computers dedicated to the control of the actuators through the AeroPro™ software. MTS Data Display software is used to display and monitor all the control parameters, actuator forces and displacements, servo voltages, oil pressure, etc. As will be discussed in the next sections, hybrid simulations are performed using analog voltage communication with the MTS controller. Therefore, in 2014, the controller was upgraded and equipped with input/output cards with analog to digital (ADC) and digital to analog (DAC) converters. Figure 2.2 shows the UT10 control station during a 3-element hybrid simulation.
Chapter 2: UT10 Development

Figure 2.1: Illustration of the actuators: (a) the platform 3D overview, (b) platform front view, and (c) platform side view

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2.2.2 Support frame

2.2.2.1 Design challenges

All of the actuators are pinned at both ends. This causes a mechanism with four hinges for the loaded brace specimens if in-plane and out-of-plane fixed supports are not provided. To prevent this mechanism, the specimens and the actuators should be supported laterally in both the in-plane and out-of-plane directions while allowed to move freely in the vertical loading (axial) direction. In addition, real pin connections between the actuators and the loading yokes in SET were originally designed to facilitate installation/removal of a concrete shell element to/from the 60 actuators in a reasonable amount of time. Because the pin connections allow free rotation of the actuators in all directions, the connections may result in actuator misalignment and hence axial load eccentricity that can potentially impose moment and have a significant effect on the axial response of the brace specimens. The above original configurations of the SET actuators posed significant challenges for the construction of the test setup. A design solution was therefore sought that could provide and facilitate the following:

- The connection between brace specimens and the vertical in-plane actuators for loading in the axial direction
- The connection between the horizontal in-plane and out-of-plane actuators to stabilize the system
- Lateral support to the specimens while allowing them to move freely in the vertical (axial) direction
- Replaceable low friction interfaces between the lateral supports and the specimens to reduce the friction forces
- Sufficient stiffness and strength in the in-plane and out-of-plane directions to minimize the effect of bending caused by the second order effects (axial loading eccentricities) on the specimens
- A robust design that can accommodate brace specimens of different sizes, configurations, and load capacities especially for large-scale testing
- Installation/removal of the system to/from SET to make it possible to alternate SET tests on a reinforced concrete shell element to UT10 tests on brace specimens and vice versa

### 2.2.2.2 Design details

Several design iterations were attempted resulting in various complete designs and the most robust design was chosen as the final solution. Figure 2.3 shows a 3D illustration of the final design of the support frame and its components with three different types of potential brace specimens, connected to the actuators. In this design, the axial deformations are applied to the brace specimen by the top vertical actuators independently. The brace specimens are all connected to the top vertical in-plane actuators and they are connected to a stationary support plate at the bottom (base plate). As shown in Figure 2.3, the specimens are laterally supported both in the in-plane and out-of-plane directions by a lateral support system through loading shafts which can also constitute a portion of the specimen. The lateral support system and the base plate are connected to two side plates at their east and west sides through post-tensioned bolted connections. Extra holes were placed in the side plates as indicated in Figure 2.3 to enable installation of the lateral support system and the base plate in various locations along the vertical direction. This feature facilitates the installation of brace specimens with various configuration and lateral support requirements. Detailed design drawings of the support frame are presented in Appendix B.

Figure 2.4 shows the details of the lateral support system. As indicated in Figure 2.4a, the lateral support system is comprised of two side beams and a middle beam that are connected by 12 link beams that cross over the side beams and the middle beam. All beams are built from solid steel rectangular or square flat profiles. The sections and connections in the lateral support system are designed to resist the lateral forces in the worst case loading scenario when different arrangements of the brace specimens are tested under the maximum possible axial load eccentricity in the system. Low friction PTFE sheets are used to reduce friction between the lateral support system and the loading shafts when the brace specimens are moving in the axial direction. The PTFE sheets are connected to the beams through 1/4 inch socket cap screws that are threaded into the beams thus facilitating replacement of the damaged sheets. Figure 2.4b shows design details of one of the link beams with PTFE sheets. As can be seen in this figure, removable steel spacer plates are used between the PTFE sheets and the link beams to reduce the gap between the PTFE face and the loading shaft. The lateral support system is specifically designed to accommodate different loading shaft or brace specimen cross section sizes with the maximum possible size of 508 mm x 254 mm. This is made possible through a pattern of extra holes on the beams which allows adjusting the free space between the PTFE sheet faces. The adjustment directions are shown in Figure 2.4a. For large capacity specimens that should be vertically centred between two vertical loading actuators and require wider loading shafts, the middle beam should be removed. The link beams are connected to the top and bottom of the side/middle beams by 12.7 mm (1/2 inch) diameter socket cap screws. The screws pass through oversized 15.9 mm (5/8 inch) diameter through holes in the link beams and are post-tensioned into 12.7 mm (1/2 inch) blind threaded holes that were drilled into the top and bottom surfaces of the side/middle beams. The oversized holes create +/-1.6 mm clearance with the bolts which enables slightly moving the link beams and accommodating the tolerances in the loading shaft cross section dimensions thus minimizing the gap between the loading shaft and PTFE faces.
The actuators can be connected to the specimens or the UT10 support frame through two types of loading yokes (type 1 and type 2). The loading yokes are shown in Figures 2.3 and 2.5. Figure 2.5 also shows details of the connection configurations for the yokes. As can be seen from Figure 2.5, type 1 loading yoke connects to a single actuator. This configuration can be used for uncoupled controlled degrees of freedom (DOFs) like testing of uniaxial brace specimens in the vertical direction as shown in Figure 2.3. In order to provide larger loading for higher capacity uniaxial brace specimens, type 2 loading yokes can be used in a similar way. As indicated in Figure 2.5 in this configuration two vertical actuators can be used in parallel to simultaneously apply axial displacements on a uniaxial specimen. The type 2 loading yoke can also accommodate the simultaneous connection of a single out-of-plane actuator in addition to two vertical actuators as indicated in Figure 2.5c. This connection configuration enables application of combinations of axial/shear/bending forces to the specimens that are tested under a combination of loads. Examples of such specimens are coupling beams in coupled shear wall systems, beam-column-brace connections, or link elements in eccentrically braced frames. As indicated in Figure 2.3, type 2 loading yokes are also installed on the side plates and the base plate to connect the UT10 support frame to the stationary actuators.

Figure 2.3: 3D illustration of the UT10 actuators and support frame with three different types of potential brace specimens (the hoists, access stairs, access platforms, and instrumentation of the specimens are not shown in this figure)
Figure 2.4: Design details of UT10 lateral support system: (a) the lateral support system and (b) the link beam

Figure 2.5: Loading yokes and their connection configurations: (a) type 1 with 800 kN uniaxial configuration (b) type 2 with 1600 kN uniaxial configuration, (c) type 2 with combined load configuration
The full process of installation of the brace specimens and loading yokes to the support frame can be done outside the testing platform. The brace specimen-loading yoke-support frame assembly can be moved into the test platform by the overhead cranes and hoists and then simply installed by inserting the pins into the yokes. This procedure keeps the actuators free for other tests during the installation and removal of the braces inside the support frame. UT10 support frame was fabricated and assembled by Walters Group Inc. in Hamilton, Canada. Figure 2.6 shows the images of the support frame assembly during fabrication. Figure 2.7 shows the UT10 testing platform with three AYB specimens as connected to the actuators.

![Figure 2.6: Images of UT10 support frame during fabrication: (a) loading shafts and type 1 yokes, (b) solid steel square flat profiles, (c) assembled support frame during transition to the Structural Testing Facility (image also shows the laboratory machine shop lead), and (d) the fabricated and assembled support frame (image also shows the author)]
2.2.3 Specimens

The current configuration of UT10 is customized for axial loading of up to 10 braces with rate-independent hysteretic properties. Examples of such braces are conventional steel braces, friction dampers, BRBs, self-centring braces, cast steel yielding brace system (YBS), etc. UT10 provides space and axial loading capacity for testing up to ten 800 kN or five 1,600 kN capacity braces. The maximum length of the braces can be 1,660 mm. Figure 2.3 shows the 3D illustration of the UT10 support frame with three potential brace specimens: a buckling specimen which is a scaled version of a conventional brace (see Chapter 5), AYB specimen which is a scaled version of a BRB (see Chapter 3 and 4), and a full-scale YBS specimen with a hysteretic response similar to a BRB but with larger post-yield stiffness (Gray et al., 2014).

2.2.4 Numerical modelling platform and data communication

In hybrid simulations of braced frames, some or all of the braces are tested physically while the rest of the structure is modelled in a numerical modelling platform. Any structural analysis software package can be used as the numerical platform with UT10, provided that proper data communication is established between the numerical platform and UT10. The latter can be achieved via UT-SIM. UT-SIM is a generalized numerical/experimental simulation framework that was recently developed by Huang and Kwon (2018) at the University of Toronto to facilitate distributed multi-platform simulations between various experimental/numerical platforms/modules through network communication. The data communication between the numerical modelling platform and UT10 is established by a newly developed element, called SubStructure, within UT-SIM framework. The SubStructure element contains the information on all of the nodes in the numerical model that are connected to the physical
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substructure. An estimation of the initial elastic stiffness of the entire physical substructure is provided as a user input for the SubStructure element. This initial stiffness is used by the numerical modelling platform to formulate the effective stiffness matrix in the equivalent static formulation of the equation of motion for the entire structure (Huang and Kwon, 2018). The SubStructure element provides a newly developed standardized data exchange format and a communication protocol for communication between the numerical model and UT10 via a network connection. The network communication enables geographically distributed hybrid simulations that are planned in the future. UT-SIM is compatible with various numerical modelling platforms including Open System for Earthquake Engineering Simulation Platform (OpenSees) (McKenna et al., 2000), ABAQUS (Simulia, 2013), and S-FRAME (S-FRAME Software Inc., 2013). Various example applications of UT-SIM framework with several numerical modelling platforms are presented in Mojiri et al. (2015), Mortazavi et al. (2017a, 2017b), and Huang and Kwon (2018). OpenSees was used as the numerical modelling platform for the first hybrid simulation applications in UT10. Figure 2.8 shows the communication and data flow between UT10 components illustrated for three independent control DOFs (brace specimens).

![Figure 2.8: Communication and data flow between UT10 components illustrated for three independent control DOFs](image)

2.2.5 Network interface for actuator controller

An interface program is necessary to receive the displacement commands from the SubStructure element via network connection, process them, and communicate them to the MTS® FlexTest 200 controller. This program should be able to provide the following features:

- Receive command displacements from a numerical modelling platform through the network
- Perform forward coordinate transformation to convert the displacement commands received from the numerical model in global coordinates to the axial displacement commands in local coordinates that should be applied on the brace specimens
- Create a ramp to the target displacement commands to ensure smooth movement of the actuators and send the final processed displacement commands to the actuator controller
- Adjust the displacement commands to compensate for the errors if necessary (displacement error compensation)
- Filter the noises from the measured signals
- Record the measured forces and displacements of the brace specimens, convert them back to the global coordinates (backward coordinate transformation), and send them to the numerical modelling platform through the network
- Establish force and displacement safety limits on the command and measured displacements and forces and stop or pause the network data communication if the limits are tripped

In response to the above requirements, an interface platform called Network Interface for Controllers (NICON) was developed at the University of Toronto (Kammula et al., 2014; Zhan and Kwon, 2015; Mojiri et al., 2015). NICON is comprised of a National Instruments (NI) hardware and an NI LabVIEW script (National Instruments, 2015) and is able to receive command displacements from the Internet or Ethernet network, perform forward/backward coordinate transformation between global coordinate of the numerical model and the local axial coordinate of the specimens, and send the measured displacements and forces from the controller to the network. In addition, NICON performs displacement/force safety limit checks, produces ramps between two consecutive displacement commands, filters the measured signals, logs all data communicated through the program, and performs error compensation based on specimens’ actual deformations directly measured from the specimen. An updated version of NICON that is compatible with UT-SIM and SubStructure element for network communication is used for preliminary hybrid simulations with a single specimen in UT10 as presented in this report.

A newer version of NICON is developed by the author specifically for UT10. This version, called NICON-10, extends all the features of the original version to ten independent controlled DOFs. Some of the algorithms such as displacement error compensation algorithm, coordinate transformation algorithm, signal offset calculation, and data logging are further improved and adjusted in NICON-10. New features including communication with an image acquisition software, off-line simulation, test resumption, etc. are also implemented in NICON-10. In addition to 10 actuator force and 10 actuator strokes, NICON-10 is able to read up to 20 channels of independent analog signals to directly measure the specimens’ deformation. There is also a feature that enables reading two independent external measurements for the axial deformation of each specimen (using the 20 channels discussed above) and then using the average value of the two measurements as the final external measurement of the specimen.
2.2.5.1 Components and data flow of NICON-10

Figures 2.8 and 2.9 show the schematic overview of the communication and data flow between UT10 components and within NICON-10 illustrated for three independent control DOFs (brace specimens). As indicated in Figures 2.8 and 2.9, NICON-10 receives global displacement commands at the SubStructure nodes ($D_{c1}$) through a Transmission Control Protocol/Internet Protocol (TCP/IP) Ethernet network connection which are then transformed to the local axial displacement commands and then properly scaled (if necessary) for each of the brace specimens ($d_{c1}$) through a forward coordinate transformation and scaling process. The final displacement command signals for each of the actuators ($d_{c2}$) are produced based on $d_{c1}$ values by a ramp generation and error compensation algorithm in NICON-10. The ramp generation algorithm produces a stream of $d_{c2}$ signals to facilitate smooth movement of the actuators between two consecutive target displacements. The $d_{c2}$ signals are converted to analog voltage by DAC in the NI hardware and are subsequently sent to the MTS controller. In addition to 10 actuator force ($f_m$) and 10 actuator strokes ($d_{m,AC}$), NICON-10 is able to read up to 20 channels of independent analog signals to directly measure the actual displacement of the specimens ($d_m$) using linear potentiometers (LP). As indicated in Figure 2.9, all analog measurements are read and converted to digital signals by ADCs in the NI hardware and are filtered using a Butterworth low pass filter in NICON-10. The filtered measured external displacements ($d_{mF}$) are then fed back to the error compensation algorithm. Once convergence is reached in the error compensation algorithm, the filtered local displacement and force measurements of all the physical braces ($d_{mF}$ and $f_{mF}$) are assembled back into the global format ($D_m$ and $F_m$) in NICON-10 through a backward coordinate transformation and scaling process. The transformed signals are then sent to the SubStructure element through a TCP/IP Ethernet network connection. As can be seen in Figure 2.9, for extra safety additional limit checks are performed in NICON-10 to ensure that the command and measured signals are within acceptable ranges.

![Figure 2.9: Communication and data flow in NICON-10](image_url)
2.2.5.2 **NI hardware and junction box**

The data communication hardware is placed in a Junction Box. Figure 2.10 shows the UT10 Junction Box. This box accommodates an NI USB-DAQ Chassis with three modules. The modules produce and read output and input analog voltages, respectively, via DAC and ADC as shown in Figure 2.9. The junction box also accommodates a DC power supply for the external measurement devices. In order to facilitate connection to the NI modules, RJ50 I/O terminals are used for the actuator input/output signals and 4-pin connectors are used for the external measurements.

![UT10 Junction Box](image_url)

**Figure 2.10:** UT10 junction box: (a) side view and (b) top view

2.2.5.3 **LabVIEW script**

The algorithms used in NICON-10 are implemented in a LabVIEW script which controls and processes the input/output signals from the NI hardware. The LabVIEW script has a graphical interface that can be used to monitor all of the variables and signals in real time during a hybrid simulation. Figures 2.11 and 2.12 show images of NICON-10 user interface for control and monitoring of the signals. As indicated in Figure 2.11b the history of signals can be monitored in real time with time graphs. The LabVIEW script uses several input configuration files which enable the user to prescribe the values of various categories of simulation parameters including the element connectivity in the physical substructure, signal calibration factors, displacement and force limits, ramp and hold...
duration, error compensation tolerance limit, etc. Some of these parameters can also be adjusted manually by the user while a hybrid simulation is in progress. The LabVIEW script also produces output log files that record the values of many variables including the measured forces and displacements. Some of the improved and new features of NICON-10 that were implemented in the LabVIEW script are presented in detail in the next sections.

Figure 2.11: NICON-10 user interface: (a) portion of the main control and monitoring interface and (b) Graphs for monitoring the history of the signals in real time.
2.2.5.3.1 The forward and backward coordinate transformation

As indicated in Figure 2.9, a forward coordinate transformation and scaling is necessary to transform global displacement commands \( D_c \) at the physical substructure nodes to local axial displacement commands for each of the brace specimens \( d_{c1} \). In addition, a backward coordinate transformation and scaling is necessary to transform local displacement and force measurements of all of the physical braces \( d_{mF} \) and \( f_{mF} \) back into the global format \( D_m \) and \( F_m \). These procedures are depicted in Figure 2.13 for a single brace. The forward transformation and scaling is depicted in Figure 2.13a and is achieved using the following equations:

\[
U_{1c} = \lambda_x X_{1c} + \lambda_y Y_{1c} \quad (2.1)
\]
\[
U_{2c} = \lambda_x X_{2c} + \lambda_y Y_{2c} \quad (2.2)
\]
\[
d_{c1} = \frac{U_{2c} - U_{1c}}{(SF)_d} \quad (2.3)
\]

In these equations \( \lambda_x \) and \( \lambda_y \) are respectively \( \cos \) and \( \sin \) of the brace element inclination angle \( \theta \). \( X_{1c}, Y_{1c}, X_{2c}, \) and \( Y_{2c} \) are the displacement commands in \( X - Y \) global coordinate system at the beginning and end of the brace element. \( U_{1c} \) and \( U_{2c} \) are the displacement commands at both ends in the local axial direction of the brace element and \( (SF)_d \) is the displacement scale factor. The backward transformation and scaling is depicted in Figure 2.13b and is achieved using the following equations for the displacement measurements:

\[
U_{1m} = U_{1c} \quad (2.4)
\]
\[
U_{2m} = U_{1c} + d_{mF} (SF)_d \quad (2.5)
\]
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\[ X_{1m} = \frac{U_{1m} - \lambda_y Y_{1c}}{\lambda_x} \] (2.6)

\[ Y_{1m} = Y_{1c} \] (2.7)

\[ X_{2m} = \frac{U_{2m} - \lambda_y Y_{2c}}{\lambda_x} \] (2.8)

\[ Y_{2m} = Y_{2c} \] (2.9)

where \( X_{1m}, Y_{1m}, X_{2m}, \) and \( Y_{2m} \) are the displacement measurements in \( X - Y \) global coordinate system at the beginning and end of the brace element. \( U_{1m} \) and \( U_{2m} \) are the displacement measurements at both ends in the local axial direction of the brace element. The following equations are used for the backward transformation and scaling of the force measurements:

\[ F_{1m} = -f_{mf}(SF)_f \] (2.10)

\[ F_{2m} = f_{mf}(SF)_f \] (2.11)

\[ F_{x1m} = \lambda_x F_{1m} \] (2.12)

\[ F_{y1m} = \lambda_y F_{1m} \] (2.13)

\[ F_{x2m} = \lambda_x F_{2m} \] (2.14)

\[ F_{y2m} = \lambda_y F_{2m} \] (2.15)

where \( F_{x1m}, F_{y1m}, F_{x2m}, \) and \( F_{y2m} \) are the force measurements in \( X - Y \) global coordinate system at the beginning and end of the brace element. \( F_{1m} \) and \( F_{2m} \) are the force measurements at both ends in the local axial direction of the brace element and \((SF)_f\) is the force scale factor.

Since only the uniaxial response (axial force and axial deformation) of the brace specimens are experimentally evaluated in UT10, the brace end rotations and moments are not taken into account in the simulation. Therefore, the command rotations received from the SubStructure element in NICON-10 are not applied on the brace specimens in UT10. Instead, the moment feedback is set to zero representing pin end connections in brace specimens and the command rotations are directly returned to the SubStructure element as the measured rotations.

As previously mentioned, the SubStructure element represents all of the physical specimens in one physical substructure. To be compatible with this feature, NICON-10 extends the capabilities of the forward and backward coordinate transformation feature of the original version of NICON: In NICON-10 the connectivity of the structural elements in the physical substructure can be defined based on which NICON-10 transforms the global displacement commands at the physical substructure nodes to the local axial displacement commands for each of the brace specimens using equations (2.1-2.3). Based on the same connectivity relationship, NICON-10 assembles the local displacement and force measurements of all the physical braces back into the global format in the physical substructure using equations (2.4-2.15) which is compatible with the SubStructure element.

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2.2.5.3.2 Original displacement error compensation

During load direction reversals in a hybrid simulation, the clearances in the actuators’ pin connections cause some slackness in the connections between the actuators and the loading yokes which results in backlash and a difference between the actuator strokes and actual deformation of the brace specimens. The elastic deformations of the testing system including the actuators reaction frame, UT10 support frame, and connections as well as any slip in the connections can also contribute to the displacement control errors (Chang et al., 2015).

In order to reduce the systematic displacement errors, an error compensation algorithm was implemented in NICON-10. This algorithm checks if the difference between the actual displacements of the brace specimens applied by the actuators and the displacement commands are within a specified tolerance limit and adjusts the commands to the actuators to compensate for the errors. A schematic overview of the ramp generation and error compensation algorithm for a single control DOF is shown in Figure 2.14. Figure 2.15 shows an image of NICON-10 user interface for network communication, displacement error compensation, and coordinate transformation. As indicated in Figure 2.14a, when a new network displacement command is sent to NICON-10 by the SubStructure element, the algorithm calculates the initial error value ($e_0$) equal to the difference between the network new local displacement command ($d_{c1}^{j+1}$) and the current filtered measured displacement of the specimen ($d_{mF}$):
$e_0 = d_{c1}^{j+1} - d_{mF}$ \hfill (2.16)

The algorithm then performs an initial correction by adding $e_0$ to the final converged displacement command for the previous network displacement command ($d_{c2,p}^{j,n}$) and assigns the values for the previous and target displacement commands ($d_{c2,t}^{j+1,i}$ and $d_{c2,t+1}^{j+1}$):

$$d_{c2,t+1}^{j+1} = d_{c2,p}^{j,n} + e_0$$ \hfill (2.17)

$$d_{c2,t}^{j+1} = d_{c2,p}^{j,n}$$ \hfill (2.18)

The algorithm then creates a ramp between $d_{c2,t}^{j+1,i}$ and $d_{c2,t+1}^{j+1}$ in a timed loop that continuously produces the displacement command signals ($d_{c2,k}^{j+1,i}$) that are sent to the MTS controller:

$$d_{c2,t}^{j+1,k} = d_{c2,t}^{j+1,i} + k\delta d_k$$ \hfill (2.19)

where $\delta d_k$ is the small increment of displacement at the $k^{th}$ ramp step which is calculated based on the type of the ramp (linear interpolation or haversine). In the above notations $k$ and $i$ are the step numbers for the ramp generation and error compensation algorithms, respectively. Upon full completion of a ramp (i.e. $k = n$) in the timed loop, the error value in the $i^{th}$ error compensation step ($e_i$) is calculated and a new target displacement is determined by adding $e_i$ to the old target displacement:

$$d_{c2,t+1}^{j+1,i} = d_{c2,t}^{j+1,i} + e_i$$ \hfill (2.20)

This process is equivalent to the integration control term in a PID controller. The error compensation is continued until the desired accuracy is achieved ($|e_i| < \varepsilon$) for each of the control DOFs after which local converged displacement and forces for all of the specimens are re-assembled, scaled, and transferred back to the global coordinates, and sent back to the numerical modelling platform through the network connection. The tolerance limit ($\varepsilon$) can be manually adjusted during the hybrid simulation. As indicated in Figure 2.14a, for extra safety, a displacement limit check is performed in the timed loop after each iteration on the ramp generation and/or error compensation algorithms. Similar algorithms are successfully implemented for displacement error compensation during hybrid simulations by others (Kammula et al., 2014; Chang et al., 2015; Wang et al., 2018).
Figure 2.14: Ramp generation and displacement error compensation algorithms: (a) the original algorithms and (b) the improved algorithms
2.2.5.3.3 Improved displacement error compensation

The original error compensation algorithm presented above and in Figure 2.14a was successfully verified during the 1-element cyclic and hybrid simulations as discussed in section 2.3. However, the 3-element test unveiled some issues that were not observed in the 1-element tests. As a result, the error compensation algorithm was improved as shown in Figure 2.14b. The improved error compensation algorithm was successfully verified in the 2- and 4-element hybrid simulations on an SCBF.

The test results for the 1- and 3-element tests with AYB specimens are discussed in Section 4.5.5.1 and comparison of the hysteretic response of the AYBs in these tests are presented in Figures 4.19, 4.20, and 4.22. As can be seen from these figures, the hysteretic responses of the AYBs, as communicated through the network to the OpenSees model, were less smooth in the 3-element tests compared to the 1-element test. In addition, the 3-element test results involved large force drops in some cases during the nonlinear response of the AYB specimens as can be seen in Figure 4.22a. The jagged responses of the AYBs were a local effect and were not expected to have a large impact on the hybrid simulation results and the global response of the BRBF in the 3-element tests. However, it was expected that this issue may intensify and have a larger impact in hybrid simulations with a larger number of specimens. Further investigations indicated that the jagged responses were caused by excessive stress relaxation and displacement overshooting in the specimens during the hybrid simulations as discussed below.

Stress relaxation:

Investigations of the test results indicated that in the 3-element test that the displacement error compensation was performed for all the specimens independently, the displacement of the specimens for which the error compensation

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algorithm was converged had to be maintained by the actuators until the convergence of other specimens was achieved. This caused extra stress relaxation and hence excessive force drop during the long hold times. In order to reduce the stress relaxation in the specimens, as indicated in Figure 2.14b, in the improved version of the error compensation algorithm the force feedback for each of the specimens for which the error compensation algorithm is converged is measured immediately after convergence is achieved for that specimen and before any stress relaxation occurs in it. This measured force is then kept in the computer memory by NICON-10 while NICON-10 waits for other specimens to reach their target displacements and converge. Once all the specimens reach their correct target displacement, the recorded force measurements in the computer memory are all released to the OpenSees model via network communication at once. It should be noted that the above procedure does not eliminate the stress relaxation that naturally occurs in the specimens. Instead, it removes its effect from the simulation by measuring the forces in the specimens before any relaxation occurs.

**Displacement overshoot:**

The 3-element test results showed that there were a few cases where the specimens moved slightly more than the actuators (overshooting) during a displacement step. In all these cases, although the actuators reached their target command displacement accurately through the PID control algorithms in the MTS controller, the corresponding specimens moved slightly more than the actuators. This was due to the interaction of the specimens at their stationary ends that were fixed to the UT10 base plate (see Figure 2.3). To correct for the extra displacements in the specimens, the original error compensation algorithm reduced the actuator displacement command, which resulted in elastic unloading and hence a sharp force drop when the specimen was in the nonlinear response range. To overcome the overshooting and sharp force drop in the specimens, in the improved error compensation algorithm the displacement error calculation and verification is performed not only at the end of the displacement ramps but also during the small increments of displacement ($\delta d_k$) within the ramp as indicated in Figure 2.14b. In this figure, $e_i^k$ is the displacement error that is calculated at the $k^{th}$ ramp step and $i^{th}$ error compensation step. To avoid overshooting in the specimens, the ramp is stopped and the current displacement command ($d_{i2,p}$ in Figure 2.14b) is maintained as soon as the desired accuracy is achieved for each specimen during a displacement ramp.

**Accelerated error compensation:**

Due to the large slack in the actuators’ pin connections, the error compensation needs to perform several iterations during the axial load direction reversals. Since several load direction reversals can occur during a ground motion, the original error compensation algorithm can significantly extend the test duration. This issue was observed in the 1-element tests. However, during the 3-element tests that the load direction reversals occurred independently for each specimen, the impact of this issue was significantly intensified and as a result the test duration was considerably increased for the 3-element hybrid simulations. In order to resolve this issue, the error compensation algorithm was revised in the improved version to accelerate the error correction process and reduce the hold times during the specimen load direction reversals. To achieve this, as indicated in Figure 2.14b, the displacement error that is used as the correction ($e_i^n$) is multiplied by a factor of $\lambda$:
The λ factor is equal to 1 by default but it increases to a predetermined value (defined by the user) when the brace response reaches near axial load direction reversal and thus accelerates the error compensation algorithm during the specimen load direction reversals. The loading direction and specimen force readings are used to determine if the response is near axial load direction reversal. For this purpose, the algorithm first calculates the direction of the network displacement command increment. If the sign of the network displacement command increment \( (d_{c1}^{j+1} - d_{c1}^{j}) \) is similar to the sign of the error \( (e_{i}^{n}) \) and if the force value is within a predefined range close to zero (e.g. [-15,15 kN]), the algorithm assumes that the response is near axial load direction reversal and increases the λ factor.

2.2.5.3.4 Off-line simulation

As part of the development of the UT10, it was necessary to perform hybrid simulation rehearsals without physical specimens. For this purpose, an off-line simulation feature was added to NICON-10. When NICON-10 is run in the off-line simulation mode, the actuator displacement commands are calculated but they are not applied on the brace specimens. Instead, the brace specimen force feedbacks are calculated based on an analytical hysteretic relationship defined in NICON-10. At its current stage of development off-line simulation is implemented for linear elastic hysteretic relationships but it can be further improved to include more complex nonlinear relationships in the future.

Other hybrid simulation parameters like white noise in the measured signal, the slack in the connections, and the actuator force and displacement offsets can be simulated in the off-line simulation mode. The off-line simulation mode was successfully verified and used in several off-line hybrid simulation rehearsals to verify the performance of the new developments in NICON-10. Particularly, it was used to study the performance of error compensation algorithm, the effect of measured signal noise on the accuracy of the simulation in the elastic range, optimum tolerance limit for the displacement error compensation algorithm, and network communications with UT-SIM and ACTIA. Figure 2.16 shows an image of the off-line simulation tab in the NICON-10 user interface. As can be seen in this figure, in order to run an off-line simulation in NICON, the elastic stiffness of the brace specimens, the noise amplitudes, and the simulated force and displacement offset values can be provided by the user.
2.2.5.3.5 Test resumption

A hybrid simulation may need to be fully stopped before completion in unexpected circumstances such as problems with hydraulic pressure supply system, control system malfunctioning, numerical model convergence problems, or specimen response issues. A test resumption feature is implemented in NICON-10 that enables resuming an interrupted hybrid simulation. In a test resumption, first the restoring forces that were recorded during the interrupted hybrid simulation are used in the integration algorithm to advance the time step to perform off-line hybrid simulations up to the termination point of the interrupted hybrid simulation. Next, to continue the simulation, the specimens’ forces and displacement status are manually restored in UT10 using NICON-10 to the points when the test was initially interrupted. NICON-10 then continues the simulation by making it online and reconnecting the hybrid simulation to the physical specimens. The test resumption feature was successfully used during one of the 3-element hybrid simulations on a BRBF. This test was interrupted and had to be stopped when all specimens had experienced nonlinear deformation. Figure 2.17 shows an image of the test resumption tab in the NICON-10 user interface.
2.2.5.3.6 Application for camera triggering and image acquisition (ACTIA)

An Application for camera triggering and image acquisition (ACTIA) has been developed at the University of Toronto for image acquisition during hybrid simulation (ACTIA, 2017). ACTIA can connect to and trigger up to 8 digital cameras and then automatically save the images in a local hard drive. This feature can be used in a hybrid simulation to track the physical status of the specimens and also to create time-lapse videos. The cameras can be triggered simultaneously in ACTIA either manually by the user or automatically via an input text file with time stamps or via network communication. Figure 2.18 shows an image of ACTIA user interface during a 4-element SCBF hybrid simulation.

NICON-10 is able to establish a TCP/IP network connection with ACTIA. When the error compensation algorithm converges for a network displacement command for all of the brace specimens during a hybrid simulation, NICON-10 sends a signal to ACTIA via the network to trigger all the digital cameras. The user can choose to skip some of the time steps if a lower temporal resolution is required for the captured images. In addition, if an iterative time integration scheme is used in the hybrid simulation, the user can choose to capture images only for the last iteration. Figure 2.19 shows an image of the ACTIA tab in the NICON-10 user interface. The performance of the above implementations was successfully verified for non-iterative time integration during the BRBF hybrid simulations and for iterative time integration during the SCBF hybrid simulations.
Other improvements and new features that were implemented in NICON-10 are the following:

- Extra analog communication channel with the MTS controller which enables automatic stop or pausing of the network data communications in case the MTS controller needs to stop or pause during a test (i.e. emergency shut down, tripping displacement and/or force limits, etc.)

- An extended data logging mode to enable logging extra information during the tests which can be used to verify the performance of the hybrid simulation and to debug potential issues in the system.
- Real-time comparison of the hybrid simulation results with fully numerical predictions which enables monitoring the performance and accuracy of the simulation results as the test progresses. This feature facilitates determining if the simulation is not converging or if a manual adjustment is necessary for the tolerance limit of the error compensation algorithm during a hybrid simulation.

- Improvements in signal offset calculation: to measure the real displacement/force changes during a hybrid simulation, the feedback displacement/force signals should be offset by subtracting the initial value of the signal (offset value) from the current value of the signal. However, the offset value can be affected by the signal noises if the signal value at a specific time is used to calculate the offset value. In order to reduce the noise effect, the original filtered signal is measured during a time window (e.g. 2000 ms) and the mean of the signal is used as the offset value.

- DC power supply voltage correction: a single DC power supply was used for the linear potentiometers. The calibration factors for the linear potentiometers are calculated based on a constant reference supply voltage. The supply voltage may be slightly affected as the internal electric resistance of the linear potentiometers changes with their stroke during a hybrid simulation. This change directly affects the accuracy of the calibration factors and hence the displacement measurements by the linear potentiometers. To avoid inaccurate displacement measurements, the supply voltage is continuously measured in NICON-10 during a hybrid simulation and the linear potentiometer feedback signals are adjusted to compensate for the changes in the supply voltage. This is done by calculating a correction factor equal to the ratio of the current supply voltage and the reference supply voltage and multiplying the correction factor to all feedback signals of the linear potentiometers. If the tolerance of the supply voltage is more than a threshold value (e.g. 10%), NICON-10 pauses the test and warns the user to fix a potential issue with the DC power supply. An alternative solution to the above problem is to use independent power supplies for each of the linear potentiometers such that their interactions are eliminated.

2.2.6 Actuator control modes during installation and testing

UT10 can be mainly used in four modes: installation mode, alignment mode, loading mode, and at rest mode. The Installation mode involves moving the UT10 support frame and specimens into the platform and inserting the pins to connect them to the actuators. Two 5-ton capacity hoists are used to support the weight and move the UT10 support frame and specimens during the installation process (see Figure 2.7). Since the UT10 support frame is supported in space on actuators, its orientation needs to be properly aligned to minimize eccentric loading on the specimens. Once the support frame is aligned with the actuators, three laser diodes are used to mark the aligned position of the frame for future re INSTALLATIONS of the system. The loading mode starts after the alignment is achieved. In this phase, the network communication is established, and the hybrid simulation starts. When the actuators are connected but the hydraulic pressure is off, UT10 is in the at rest mode. This mode can occur between any other modes. During this mode, UT10 is supported vertically on two steel pedestals that support the weight of the UT10 support frame and specimens when the hydraulic pressure is off. To explain the configuration of the actuators during
different UT10 modes, simplified 3D schematic drawings of the UT10 with the actuators and three brace specimens are shown in Figure 2.20. In this figure, only the connected actuators are shown.

The connection configuration of actuators does not change in each of the UT10 modes. As can be seen from Figure 2.20, all of the bottom in-plane and out-of-plane actuators are connected. On the top, none of the out-of-plane actuators need to be connected and the vertical in-plane actuators are all disconnected except the ones that impose displacement commands to the specimens. All of the in-plane and out-of-plane actuators on the west and east sides of the UT10 platform are connected except for the two (or three) middle rows.

In the installation mode, all of the actuators are set to be force-controlled. Since all of the actuators are simultaneously connected to the same hydraulic system, the unconnected actuators need to be fully retracted to avoid their interaction with the UT10 support frame during the test. For this purpose, they are put into slight tension (e.g. 15 kN). To establish a pin connection between the connected actuators and UT10 support frame, each actuator is slowly moved by commanding it into slight tension or compression until proper alignment between the actuator clevis and its corresponding loading yoke is achieved and then the pin is inserted manually.

In the alignment mode, the unconnected actuators are set to be force-controlled and are commanded slight tension to stay retracted. In order to control the rigid body movements of the UT10 support frame during alignment, three in-plane and three out-of-plane actuators are set to be displacement-controlled. This selection makes the system statically determinate thus preventing the development of significant forces during the rigid body movement of the UT10 support frame and the specimens. The displacement-controlled actuators are shown in Figure 2.20a. In this figure, DC and FC refer to the displacement- and force-controlled actuators, respectively. To establish full alignment of the UT10 support frame, the displacement-controlled actuators are moved directly by the user displacement commands from the MTS controller.

In the loading mode, a minimum of four horizontal in-plane, two vertical in-plane, and four out-of-plane actuators are set to be displacement-controlled with fixed displacements (rigid links). This configuration makes the system statically indeterminate but it provides extra supports during the test and limits the rigid body movements and elastic deformations of the UT10 support frame. The number of rigid links can be increased if higher magnitudes of lateral loads are expected such as in the case of testing more than five specimens. Although multiple fixed supports are provided in this configuration for the system, the whole frame can still experience slight movement during the loading mode due to the slackness in the actuator-loading yoke connections. To remove this movement, the horizontal in-plane rigid links are set to slight tension while the out-of-plane rigid links are set to slight tension and compression in a checkerboard configuration to assure that complete contact is achieved in the pinned connection during the test. Figure 2.20b shows the control configuration of the actuators during the loading mode. In this figure, letter R refers to the rigid links and the letters C and T refer to compression and tension, respectively. Therefore, RT and RC refer to rigid links that are in slight tension and compression, respectively. The top vertical in-plane actuators that are connected to the specimens are set to be displacement-controlled. They are controlled by the MTS controller but their displacement commands are fed to the controller by external analog signals from NICON-10 (see Figure 2.8). These actuators are labelled as DC-E in Figure 2.20b where E stands for the external signal. The bottom actuators that are directly under each of the specimens, act as the direct support for the axial
forces in their respective specimens. However, if they are set as rigid links, the slack in their connections can potentially cause local bending and yielding in the base plate under significant axial loads. To resolve this issue, the bottom vertical in-plane actuators under each specimen are set to be force-controlled and their force command is programmed to be the force feedback of their respective top actuator that is connected to the same specimen minus the sum of the weight of both top and bottom actuators. These actuators are labelled as FC-S in Figure 2.20b where the letter S indicates the slaved control nature of these actuators.

![Diagram of actuator control modes](image)

**Figure 2.20:** Actuator control modes: (a) during alignment and (b) during loading
It should be noted that while the top vertical in-plane actuators are uncoupled, the bottom ones are all connected to the same base plate and can potentially share their loads. Therefore, if there is any delay in the response of the slaved force-controlled actuators, the two bottom rigid links partially take some of the unbalanced axial load in the specimens and therefore, the system stays stable in the vertical direction. In multi-element tests like 10-element tests that all of the bottom vertical actuators need to be used in a slaved force-controlled fashion, there will be no actuator available to act as the vertical rigid link and support the unbalanced vertical load. Two adjustable vertical support pedestals can be used in such cases which can be directly connected to the bottom of the side plates in the UT10 support frame and directly transfer the unbalanced axial loads of the specimens to the actuator’s reaction frame.

2.3 UT10 PERFORMANCE VERIFICATION

The performance of UT10 was verified through several cyclic and hybrid simulations on multiple specimens with various types of hysteretic responses. The specimens considered were a specimen with linear elastic response, AYB specimens with a hysteretic response involving significant plastic deformation similar to a BRB, and buckling specimens with a complex hysteretic response involving buckling, strength drop, and negative stiffness similar to a conventional buckling brace. The tests included one cyclic test on the elastic specimen, one cyclic test on the AYB specimen, one 1-element and two 3-element hybrid simulations on BRBFs with AYB specimens, one cyclic test on a buckling specimen, and one 2-element and two 4-element hybrid simulations on SCBFs with the buckling specimens.

A checklist has been developed for users to safely operate UT10. This checklist contains more than 140 steps that should be completed during the UT10 installation mode, alignment mode, loading mode, and at rest mode in order to complete a hybrid simulation with UT10. The steps contain details on the procedures to run the MTS® FlexTest 200 controller, AeroPro™ software and MTS Data Display in conjunction with NICON-10 and also to establish network connections between NICON-10, the numerical substructure, and ACTIA. The checklist also includes the protocols for resuming an interrupted test, handling unexpected problems during a test, and managing the emergency shutdowns of the hydraulic system. These steps were developed, implemented, optimized, and successfully verified through multiple single- and multi-element hybrid simulations with UT10. A sample checklist for 4-element hybrid simulations is provided in Appendix A.

2.3.1 1-element elastic cyclic test

Preliminary verification tests were performed on a single elastic specimen in UT10. The specimen was built from a 64x64x4.8 mm square hollow section conforming to ASTM A500 standard. The free length and cross section area of the specimen were 883 mm and 971 mm², respectively. The specimen was groove welded at both ends to two end plates which were bolted to the loading shaft and the base plate inside the UT10 support frame. Figure 2.21 shows the elastic specimen as installed inside the UT10 support frame.
The loading protocol involved a full cycle of 0.4 mm and a full cycle of 1 mm axial displacement. The predetermined displacement commands were sent by a MATLAB (MathWorks, 2015) script via a network connection and the SubStructure element to NICON. No coordinate transformation and scaling were necessary for this test. Figure 2.22 shows the axial force-displacement response of the specimen. The response is shown based on both the actual deformation of the specimen as measured by the linear potentiometers and the displacement of the actuators. The elastic stiffness of the specimen was once determined experimentally based on the actual response of the specimen obtained from the cyclic tests in UT10 and once analytically from $K = EA/L$ with 200 GPa for the elastic modulus ($E$), 883 mm for the length ($L$), and 971 mm$^2$ for the cross section area ($A$) of the specimen. The results indicated that the experimentally measured axial stiffness was 213 kN/mm which was 3% smaller than the analytical axial stiffness of the specimen. The results confirmed the accuracy of UT10. The axial stiffness of the specimen measured based on the actuator displacement was 107 kN/mm that was 50% smaller than the actual axial stiffness of the specimen. This was due to the fact that the actuator displacement contained elastic deformation of the reaction frame and connections in addition to the axial deformation of the brace specimen which reduced the measured stiffness. Figure 2.23 shows the recorded axial displacements during the test. Figure 2.23a shows the values of the axial displacements at each step. In this figure, four types of displacement are plotted. The first type is the network displacement command ($D_{c1}$ or $d_{c1}$) or the target displacement, the second type is the actual displacement of the specimen measured by the external linear potentiometer ($d_m$), the third type is the displacement command sent to the MTS controller ($d_{c2}$), and the fourth type is the measured displacement of the actuator ($d_{m,AC}$). The values of $d_m$, $d_{c2}$, and $d_{m,AC}$ are the recorded values after the displacement error compensation was converged. Therefore, the intermediate values between two consecutive target displacements before reaching the convergence are not presented in Figure 2.23a for these types of displacements. It can be observed from Figure 2.23a that there was approximately 3 mm difference between the actuator movements and the specimen deformation measured with an external linear potentiometer. This difference was due to the elastic deformations of the actuator reaction frames and their connections and to a greater extent due to the slackness in the actuator’s pin connections.

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presented in figures 2.22 and 2.23a also indicate that the maximum displacement difference due to the slackness at the pin connections was approximately 2 mm which occurs at the instant of axial force direction reversals. The results of Figure 2.23a show a maximum of approximately 0.75 mm displacement difference resulting from the elastic deformations in the system at the peak loads.

Figure 2.22: Axial force-displacement response of the specimen during the 1-element elastic cyclic test

Figure 2.23: Axial displacements of the actuator and specimen during the 1-element elastic cyclic test: (a) displacement values and (b) displacement errors

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Figure 2.23b shows the displacement error values calculated as the difference between the network displacement command (target displacement) and the actual displacement of the specimen measured by the external linear potentiometer. The specified tolerance value of +/-0.05 mm is also shown in this figure. The results presented in Figure 2.23b indicate that the displacement error compensation algorithm successfully reduced the displacement errors in the test setup. However, in a few of the steps, the error was slightly larger than the specified tolerance value. Further investigation of the results revealed that this issue was caused by the slight movements of the specimen which occurred after the convergence of the error compensation algorithm was achieved and before the actual displacement of the specimen was recorded and sent back to the MATLAB script via network communication. This issue was later resolved in the improved version of the displacement error compensation algorithm as discussed in section 2.2.5.3.3.

Figure 2.24 shows the recorded values of displacement between two consecutive target displacements. As indicated in the bottom graph, at the time 2010 s the target displacement sent from the MATLAB script to NICON through network changed from -0.25 mm to -0.5 mm. This change occurred sharply in an infinitesimal time corresponding to the speed of LabVIEW script. However, since the data recording was performed in 0.5 sec time steps (2 Hz), this transition is shown to happen in 0.5 sec in the graph. As indicated in the top graph of Figure 2.24, the target displacement was imposed to the actuator during a 3-sec haversine ramp and a 1-sec hold. The ramp in Figure 2.24 starts with -0.82 mm which is 0.57 mm smaller than the previous target displacement (-0.25 mm). The difference is due to the accumulated displacement error that is carried in the NICON command on top of the target displacement. The cumulative error, in this case, was -0.57 mm. It can be seen from the bottom graph in Figure 2.24 that after three ramps and holds (three iterations of error compensation algorithm), the target displacement was achieved by the specimen with errors within the prescribed maximum tolerance limit of 0.05 mm.
2.3.2 Multi-element tests with AYB specimens

UT10 performance was further verified by testing AYB specimens under cyclic loads and under actual ground motions. The hysteretic response of AYB specimen is similar to a BRB which involves significant axial yielding, plastic deformation and isotropic/kinematic hardening. Details on the design and development of the AYB specimen are presented in Chapter 3. 1- and 3-element hybrid simulations were successfully performed on a BRBF with AYB specimens in UT10. The tests confirmed the performance of the UT10 in capturing the response of the specimens under both cyclic loads and ground motions. Details on the performance of the UT10 as observed in these tests are presented in Chapter 3 section 3.4.42 for the cyclic loads and in Chapter 4 for the ground motions. The error compensation algorithm was successfully verified during the 1-element hybrid simulations as discussed in Chapters 3 and 4. The error compensation algorithm was further improved after some issues were observed with the control system during the 3-element tests on the BRBF. The improvements were implemented to avoid overshooting and excessive stress relaxation that occurred during the 3-element hybrid simulations and also to accelerate the error compensation process during the load direction reversals. Details of these improvements were discussed in Section 2.2.5.3.3. Figure 2.25 shows the displacement error values calculated as the difference between the reduced scale network displacement commands and the actual displacement of the AYB specimens measured by the external linear potentiometers during one of the 3-element hybrid simulations on the BRBF. The specified tolerance value of +/-0.06 mm is also shown in this figure. The results presented in Figure 2.25 indicate that the displacement error compensation algorithm successfully reduced the displacement errors in the test setup to values below the specified tolerance limit.

![Displacement error values during one of the 3-element hybrid simulations (3E-BRBF-HS1) on the BRBF with three AYB specimens](image)

**Figure 2.25:** Displacement error values during one of the 3-element hybrid simulations (3E-BRBF-HS1) on the BRBF with three AYB specimens

2.3.3 Multi-element tests on buckling specimens

UT10 performance was also verified by testing buckling specimens under cyclic loads and actual ground motions. The hysteretic response of this specimen is similar to conventional braces in SCBFs where the seismic energy is mainly dissipated through yielding of the braces in tension and nonlinear buckling of the braces in compression. Therefore, similar to conventional braces, the hysteretic response of the buckling specimens not only involves
isotropic and kinematic hardening but also it involves a sharp drop of compressive force and negative stiffness during and after buckling of the specimens. The complex hysteretic response of these specimens made them an interesting choice for performance verification of the UT10. A 1-element cyclic test was performed on a buckling specimen. This test was followed by 2- and 4-element hybrid simulations on an SCBF with buckling specimens. The performance of the UT10 and the functionality of the error compensation improvements that were implemented following the 3-element test on the BRBF were also verified. Detailed discussions on the results of these tests are presented in Chapter 5. Figure 2.26 shows the displacement error values calculated as the difference between the reduced scale network displacement commands and the actual displacement of the buckling specimens measured by the external linear potentiometers during one of the 4-element hybrid simulations on the SCBF. The specified tolerance value of +/-0.06 mm is also shown in this figure. The results presented in Figure 2.26 confirm the accuracy and performance of the displacement error compensation algorithm. The results indicate that the algorithm successfully reduced the displacement errors for all of the buckling specimens to values below the specified tolerance limit. The error values in a few of the steps exceeded the specified tolerance value for some of the specimens which was potentially due to the sharp compressive force drop and overshooting of the buckling specimens during their compressive buckling as discussed in Section 5.7.6.

Figure 2.26: Displacement error values during one of the 4-element hybrid simulations (4E-SCBF-HS1) on the SCBF with four buckling specimens

2.3.4 Future 10-element tests

Future projects are planned at the University of Toronto to use UT10 in its full capacity and perform 10-element hybrid simulations on concentrically braced frames. The mechanism and challenges for the 10-element tests are expected to be similar to the 3- and 4-element tests. However, more interaction of the specimens at the base plate are expected to happen for the 10-element tests compared to the 3- and 4-element tests. Following the implementation of the control and error correction improvements in NICON-10 and their successful verification in the 4-element test, it is expected that such interactions will be similarly controlled by NICON-10 during 10-element hybrid simulations.
2.4 SUMMARY AND CONCLUSIONS

Design, development, and performance verification of a 10-element hybrid simulation platform known as UT10 is presented in this chapter. UT10 enables multi-element hybrid simulations on braced frames and provides a reconfigurable platform with an open source user-friendly interface. Components of UT10 and their design details and development challenges are presented in this chapter. UT10 uses the sixty large capacity servo-controlled actuators that are available in the SET platform in the Structural Testing Facility of the University of Toronto. Details of the hydraulic and control system and the configuration of the actuators are discussed in detail. UT10 provides space and loading capacity for testing up to ten 800 kN and five 1600 kN capacity brace specimens. As part of UT10 development, a steel support frame was designed and constructed which provides lateral support for the brace specimens and limits the secondary effects due to the potential eccentric loads from the actuators. Data communication between a numerical platform and UT10 is established via UT-SIM which is a generalized numerical/experimental simulation framework that was recently developed at the University of Toronto. Therefore, any structural analysis software package compatible with UT-SIM communication protocol can be used in the hybrid simulations with UT10. The displacement commands and displacement/force feedbacks are communicated between UT-SIM and the actuator control system via an interface program called NICON-10. Various features are implemented in NICON-10 that facilitate hybrid simulations for 10 independent control DOFs. One of the most important features of NICON-10 is the displacement error compensation algorithm. This algorithm reduces the displacement errors during hybrid simulations. Details of the features of NICON-10 are presented and discussed in detail in this chapter. Performance of UT10 was evaluated through several tests under cyclic loads and actual ground motions with three types of brace specimens. These tests confirmed that the UT10 control and communication system operates flawlessly and the error compensation algorithm reduces the errors to the desired tolerance limits. The error compensation algorithm was further improved to resolve some issues observed after the 3-element tests. These improvements were successfully verified in the next multi-element tests. The responses of the brace specimens were captured smoothly and accurately during all of the tests. The verification test results confirmed the excellent performance of UT10 for doing multi-element hybrid simulations on braced frames. In summary, UT10 offers the following advantages:

- In its current configuration, up to 10 uniaxial large-capacity specimens with various sizes and shapes can be tested in UT10.
- The unique setup of the UT10, particularly the configuration of the 60 actuators available in the SET, the controllers, and the open-source and reconfigurable nature of the UT-SIM framework and NICON-10 interface provides the basis to adjust the platform for testing of specimens with complex loading configurations.
- The network communication feature in NICON-10 makes it possible to use the UT10 in geographically distributed hybrid simulation projects in future thus enabling this unique equipment to be shared in domestic and international collaborative projects.
- The error compensation feature in NICON-10 relaxes the requirements on tight and stiff connections and stiff reaction supports which otherwise are required in order to eliminate inaccurate analysis results, analysis instability, and analysis convergence issues arising from the accumulation of experimental errors during the tests.

The following aspects can be considered as limitations of UT10:

- The load capacity and the available space in UT10 requires scaling of the specimens in some applications.

- Since the actuators were initially not designed for testing at high loading rates, low flow-capacity pipes and servo valves were installed, which limit the velocity of actuators’ stroke. These hydraulic limitations and the slack in the connections restrict the application of the UT10 to slow hybrid simulations. The yield and ultimate strength of steel are increased at higher strain and loading rates. However, such strain rate effects are minimal and thus can be neglected during earthquake response of structures with periods more than 0.1 s which is the case for most of the steel braced frames (Mahin and Shing, 1985). Therefore, the slow rate of loading of the steel brace specimens in UT10 does not majorly impact the accuracy of the hybrid simulations on steel braced frames.

- Further improvements are necessary for the displacement error compensation algorithm to enable loading of a single specimen with multiple actuators such as uniaxial brace specimens with capacities larger than 800 kN or specimens tested under a combination of axial/shear/bending forces.
CHAPTER 3: DEVELOPMENT OF AN ADJUSTABLE YIELDING BRACE (AYB)

3.1 INTRODUCTION

As the first application of UT10, the seismic performance of structures with hysteretic energy dissipative braces and more specifically BRBFs was investigated. Therefore, as part of this project, a reusable and adjustable brace specimen, referred to as an Adjustable Yielding Brace (AYB), was designed and fabricated to be used specifically with the UT10. The brace was designed such that it had adjustable stiffness and strength. AYB was designed to simulate the hysteretic response of yielding braces such as BRBs thus facilitating the seismic performance evaluation of multi-storey structures with hysteretic energy dissipative braces through hybrid simulations. The AYB design objectives were the following:

- Simulate the cyclic hysteretic response of hysteretic energy dissipative braces like a large-scale BRB
- Have adjustable stiffness and strength such that one design can be used to simulate the response of BRBs with a range of stiffness and strength (e.g. BRBs at different stories of a building)
- Have a robust design and a response that is not significantly affected by eccentric axial loads that are present in UT10.
- Provide an acceptable level of displacement ductility capacity to enable tests with large displacement demands
- Can be restored to its initial undamaged condition with affordable (cost and time) modifications after each test thus facilitating performing several tests with one specimen (reusable)
- Can be adapted to the size limitations of UT10
- Can be produced in large quantities (economical and simple design)

The main concept underlying the design of AYB is yielding metal rods in tension and compression inside pipes which act as the restraining mechanism that eliminate the global buckling of the rods. A similar concept was previously adopted and tested by Christopoulos et al. (2002). In their study, they used energy dissipating bars as a source of energy dissipation in post-tensioned energy dissipating connections that were developed for moment-resisting steel frames. The energy dissipating bars were 22 mm diameter Dywidag threaded bars confined in steel cylinders. The bars were connected at both ends to steel couplers. The cyclic tests on the energy dissipating bars by Christopoulos et al. (2002) revealed that they can provide a stable response and good energy dissipation capacity.
However, it was observed that slack developed in the connections during the tension-compression load reversals due to the permanent deformations in the threaded connections.

This chapter presents the design details and the cyclic test results of the AYB specimen. The chapter starts with design details of a single-component yielding specimen with a single rod that was used in proof-of-concept cyclic tests. The chapter continues with the design details and cyclic test results of a six-component specimen, hereafter referred to as the AYB specimen, with six rods, which was tested inside an MTS loading platform to verify the scalability of the response of the specimen from single-component to multi-components. In the next section, AYB design improvements that were adopted to facilitate cyclic testing of AYB in UT10 are presented and the cyclic test results are discussed in detail. Since the cyclic testing of AYB in UT10 was the first UT10 test involving large forces, the test data was also used to verify the performance of UT10. The chapter concludes with advantages, potentials and limitations of AYB.

Parts of this chapter are published in the following journal paper:

### 3.2 SINGLE-COMPONENT YIELDING SPECIMEN

#### 3.2.1 Design

Figure 3.1 shows the design details of the single-component yielding specimen. As indicated in this figure, the single-component yielding specimen is mainly composed of a single fully threaded M20 rod (core rod) that acts as the yielding core and a seamless standard steel pipe (confining pipe) that provides confinement and buckling resistance along the core. The core rod had material specifications conforming to DIN 975 with specified yield and ultimate tensile strengths equal to 235 MPa and 400 MPa, respectively. The inside and outside diameter of the confining pipe are 20.7 mm and 33.4 mm, respectively. This configuration leaves 0.35 mm of clearance between the core rod and the confining pipe to accommodate the expansion of the core rod due to Poisson effects when the rod is in compression similar to a BRB. The core rod is the only source of stiffness and strength in the specimen. The energy is dissipated through yielding of the core rod in both tension and compression similar to a BRB yielding core. As indicated in Figure 3.1a the yielding length of the core rod in the single-component yielding specimen is 565 mm. Two set screws are threaded at the mid-length of the confining pipe as indicated in Figure 3.1a. When completely fixed, the set-screws connect the core rod to the confining pipe through a friction contact at the mid-length of the core rod. The connection at the mid-length of the core rod not only limits the rigid body movements of the confining pipe but also reduces the effect of friction between the core rod and confining pipe when the specimen is in compression. The specimen is protected against global buckling by two stiffener plates that are welded at each side of the confining pipe.
Figure 3.1: Design details of the single-component yielding specimen: (a) 3D illustration, (b) fabricated and instrumented specimen installed inside the MTS loading platform, (c) fabricated specimen instrumented with the linear potentiometer, and (d) details of the gripping pipe, the support pipe, and the linear potentiometer connection as fabricated.

The axial load is applied to the single-component yielding specimen via the gripping pipes at both ends of the specimen as indicated in Figure 3.1a. The gripping pipes are each 114.3 mm long standard steel pipes with an outside diameter of 42.2 mm. The core rod passes freely inside the gripping pipes and is post-tensioned (PT) at both
ends of the gripping pipes. As discussed in section 3.3, the core rod connections are post-tensioned in the AYB specimen to eliminate the formation of slack during loading reversals in the cyclic tests. The gripping pipe configuration in the single-component yielding specimen was therefore used to model this connection detail and to verify the performance of the post-tensioned connections in eliminating the formation of the potential slacks.

After performing a series of preliminary cyclic tests on the single-component yielding specimen it was discovered that although the global buckling of the core rod was prevented by the confining pipe, the unconfined lengths of the core rod at each end of the confining pipe experienced bending and developed local buckling. This resulted in premature fracture of the core rod in the unconfined region and limited the axial force and deformation capacity of the specimen. As an attempt to improve the design of the specimen to avoid this undesired occurrence, both unconfined ends of the core rod were protected against local buckling and bending by support pipes. Figure 3.1 shows the design details of the support pipe. A support pipe consists of a seamless standard steel pipe with 34 mm inside diameter and 48.2 mm outside diameter. As indicated in Figure 3.1, a high strength steel nut and washer is welded to one end of the support pipe that provides the connection of the support pipe to a core rod. The support pipe post-tensions and moves with the core rod and it is allowed to have free axial movement along the confining pipe in a telescopic configuration. Only 0.3 mm clearance is allowed between the inside of the support pipes and the outside of the confining pipe to minimize the lateral movement of the unconfined ends of the core rods with respect to the confining pipes. The inside of the support pipes is lubricated with grease to reduce the friction between the support pipes and the confining pipe. In order to reduce the bearing stress between the confining pipe and the core rod, the inside edges of the confining pipe are rounded at both ends as indicated in Figure 3.1a. Detailed design drawings of the single-component yielding specimen are presented in Appendix B.

3.2.2 Cyclic test results

Cyclic tests were performed on the single-component specimen in an MTS loading platform. For this purpose, the specimen was instrumented with a linear potentiometer which was connected to the support pipes to measure the axial deformation of the core rod. Figure 3.1b and 3.1c show the single-component specimen as fabricated, instrumented, and installed in the MTS loading platform. Figure 3.1d shows the details of the gripping pipe, the support pipe, and the linear potentiometer connection. The single-component specimen was loaded to two cycles of displacements corresponding to 0.7%, 1.5%, and 2.2% of axial strain with loading rates of 0.02 mm/s, 0.04 mm/s, and 0.06 mm/s, respectively. The displacements are within the range experienced by the yielding core of full-scale BRBs during an earthquake (see Chapter 4).

The cyclic test was performed successfully and ended by steel rupturing in the middle of the core rod at the end of the last cycle. In the last compression cycle, the specimen was pushed 1 mm more than the previous cycle following an actuator control issue. Figure 3.2 shows the cyclic hysteretic response of the single-component yielding specimen. It can be observed from this figure that the single-component specimen has repeatable and stable hysteretic response both in tension and compression. The hysteretic response features isotropic and kinematic hardening and significant overstrength similar to a full-scale BRB. It is also capable of simulating the Bauschinger effect that is observed in real BRBs (see Chapter 4). Furthermore, it is noted that similar to a BRB, the compression
strengths are larger than the tensile strengths with an average ratio of 1.09 over all of the cycles. The results generally indicate that the element can successfully simulate the hysteretic response of a real BRB. The smooth transition during the tensile and compressive force reversals indicates the effectiveness of the post-tensioning approach in eliminating the potential slacks in the threaded connections.

![Cyclic hysteretic response of the single-component yielding specimen](image.png)

**Figure 3.2:** Cyclic hysteretic response of the single-component yielding specimen

Figure 3.3 shows the single-component yielding specimen after the cyclic test. Figure 3.3a shows the core rod outside the confining pipe. Figure 3.3b shows a closer view of the tensile rupture region in the core rod. As can be seen from these figures, the core rod ruptured in tension at the mid-length in the confined zone after experiencing significant necking. As can be seen from Figure 3.3b the rupture occurred between two threads in a region with reduced cross section area. The threads cause stress concentration in the regions between the threads during the axial loading. The reduced area between the threads results in an effective cross section area \( A_{\text{net}} \) for the threaded rod that is smaller than its gross cross section area. The effective cross section area of the core rod was calculated from \( K_{\text{el}} = EA_{\text{net}}/L \) where \( K_{\text{el}} \), \( E \), and \( L \) are, respectively, the experimental initial elastic stiffness of the single-component specimen, the elastic modulus, and the yielding length of the core rod. The initial elastic stiffness of the core rod measured from the hysteretic response of the specimen was 73.3 kN/mm. Assuming 200 GPa for the elastic modulus of the core rod and using 565 mm for the yielding length of the core rod, the effective area of a single rod was calculated as 207 mm\(^2\). Based on this the effective cross section area of the core rod was 66% of the gross cross section area.

No excessive deformation, damage, and fracture were observed in the unconfined segments of the core rod. These observations confirmed that the support pipe design successfully eliminated the failure of the core rods in the end regions. Figure 3.3c shows the deformed shape of the core rod end regions after the cyclic test. Formation of compressive higher mode buckling in this region can be clearly observed from this figure. It can be also observed that some of the threads in this region had deformed which is the result of the core rod bearing against the inner ends of confining pipe during the test.

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
The cyclic test results on the single-component yielding specimen revealed that the design concept is robust and reliable for a single-component. In order to investigate the robustness and scalability of the response for specimens with multi-components a six-component specimen (the AYB specimen), was designed for testing in the MTS loading platform. Figure 3.4 shows the design details for the AYB. As indicated in this figure, the AYB specimen is essentially composed of six single-component yielding specimens (threaded rod-pipe elements) that are welded to a spacer plate and loaded in parallel via two universal connectors that are gripped by the MTS loading platform. As indicated in this figure the yielding length of the core rods in the AYB specimen is 640 mm which is slightly larger than the single-component specimen. The core rods can be installed in symmetric combinations of 2, 4, and 6 rods. Therefore, the strength and stiffness of the AYB can be adjusted by the number of core rods installed in the AYB and the core rod material properties.
As indicated in Figure 3.4a the threaded core rods are directly connected to the universal connectors using PT nuts and support pipes. This connection detail allows replacement of the cores after each test without the need to repair other parts of the AYB. Although the threads are necessary only at each end of the core rods, the core rods are fully threaded over their entire length to avoid concentration of nonlinear deformations in the threaded regions resulting in premature fracture of the rod in the connections. The spacer plate connects the six confining pipes together such that they all act as one rigid piece. This piece provides significant resistance to the global buckling of the AYB, but it can freely move in the axial direction with respect to other components of the AYB. Similar to the single-component yielding specimen, set screws are used to limit the rigid body movement of spacer plate and the confining pipes with respect to the core rods.

Figure 3.4: Design details of the six-component adjustable yielding brace (AYB) specimen: (a) 3D illustration, (b) fabricated and instrumented specimen installed inside the MTS loading platform, (c) close view of the fabricated specimen
The core rod threads in the threaded connections deform permanently when large inelastic deformation is imposed. This causes slack in the connections when load reverses from compression to tension or vice versa. Post-tensioning the connections keeps the threaded connection always in tension and hence eliminates the slack during the load reversals. Post-tensioning is done through two nuts pressing against the universal connectors. As long as the axial tension in a core rod is below the post-tensioning force in these connections, the nuts stay in full contact to the bottom connector and no slack occurs in the connections. However, when the axial tension in a core rod exceeds the post-tensioning force, the inner nut separates from the connector and causes a gap between the inner nut and the plate. Upon a load reversal, this gap is closed by the post-tensioning force in the connection but at the expense of reducing the post-tensioning force. If the axial load exceeds the post-tensioning force again in the next cycles, it will be reduced further and will eventually vanish, leaving a permanent gap (slack) in the connections. Therefore, to eliminate the formation of slack in the connections during the test, the post-tensioning force should be more than the maximum expected axial tension in the core rods. Based on the single-component test results presented in the previous section, the PT connections proved to be effective in eliminating the slack and are thus also implemented in the AYB design.

The PT force was roughly adjusted based on the number of turns of nut in the PT connections. The number of turns of nut to develop a PT force depends on several parameters including the magnitude of the PT force, the core rod material properties, the free length of the core rod between the PT nuts in the PT connection, and the snug-tightness of the nuts before applying the PT force. A preliminary value was determined based on simple calculations using the stress-strain response and the free length of the core rods between two PT nuts in the PT connections. The final number of turns of nut necessary to remove the slack in the PT connections was then determined following an iterative procedure during the preliminary cyclic tests on the single- and six-component specimens.

It should be noted that the PT force cannot be larger than the tensile strength of the core rods. Therefore, if the axial forces in the core rods approach their tensile capacity, they may exceed the PT force and thus cause the formation of slack in the PT connections. However, as is discussed in Section 3.5, for the targeted range of response of the braced frames that is considered for the hybrid simulations in UT10, the AYB is not expected to be loaded to levels close to the tensile strength of the core rods.

### 3.3.2 Cyclic test results

In order to do the cyclic tests on the AYB specimen, the specimen was instrumented with two linear potentiometers which were connected to the universal connectors at each side of the side plate. The measurements of these two linear potentiometers were averaged and the average value was used as the final axial displacement of the specimen. The AYB threaded rods were chosen from the same batch of rods used for the single-component specimen. Figures 3.4b and 3.4c show the AYB as fabricated, instrumented, and installed in the MTS loading platform. The loading protocol for the cyclic test started with two displacement cycles corresponding to 0.15% axial strain in the core rods in the elastic range to investigate the elastic response of the specimen. The rest of the cycles corresponded to 0.8%, 1.6%, 2.4%, and 3.3% axial strains which respectively corresponded to 1%, 2%, 3%, and 4% storey drift ratio (SDR) in the BRBF considered for the hybrid simulations in Chapter 4. Each displacement cycle was repeated twice to
verify the stability of the response under the same displacement demand. The axial displacement demands were calculated assuming that the AYB represents part of the yielding core of a full-scale BRB in the BRBF. The deformations of the elastic parts of the full-scale BRB, which were assumed to constitute half of the length of the BRB, were neglected in these calculations.

The cyclic test was performed on the AYB specimen in an MTS loading platform. The test was stopped following the steel rupturing in the middle of one of the core rods at the end of the 3% SDR cycle. The loading rates for the elastic cycle, 1% SDR, 2% SDR, and 3% SDR cycles were 0.01 mm/s, 0.02 mm/s, 0.06 mm/s, and 0.1 mm/s, respectively. Figure 3.5a shows the hysteretic response of the AYB specimen. In the second 2% SDR cycle a hydraulic shut down occurred in the laboratory which caused temporary loss of oil pressure in the MTS actuator. As can be observed in Figure 3.5a, this issue caused a sudden jump in the AYB axial displacement. However, this extra displacement did not impact the response of the AYB specimen in the next cycles. As can be observed from Figure 3.5a, the hysteretic response of the AYB specimen was similar to the single-component specimen. Slight shifts were observed during the 1% and 2% SDR cycles near the zero axial force in the hysteresis curves as can be seen in Figure 3.5a which indicated that slight slack developed in the threaded connections when the core rod experienced large nonlinear deformations. This observation revealed that the PT force was not sufficient and therefore, in the next tests on the AYB, the connections were post-tensioned more to avoid the formation of slacks in the connections.

Figure 3.5: Hysteretic response of the AYB specimen: (a) axial force-displacement response and (b) axial stress-strain response

In order to better compare the hysteretic response of the single-component specimen and the AYB specimen, the stress-strain responses of the two specimens are calculated and presented in Figure 3.5b. The stress values are calculated by dividing the measured axial forces by the net cross section area of the specimens. In this calculation, the net cross section area of the single-component specimen is considered 207 mm² which is the value calculated in section 3.2.2. The net cross section area of the AYB specimen is assumed to be 6 × 207 = 1243. The strain values are calculated by dividing the axial displacements by the yielding length of the specimens. As can be observed from
Figure 3.5b, except for the differences that are caused by the different axial displacement demands in the loading protocols of the two specimens, the responses of the two specimens are identical. The initial elastic stiffnesses of the two specimens are similar which confirms the scalability of the hysteretic response of the single-component specimen to six components.

It can be also observed from Figure 3.5b that the core rods showed significant overstrength. The initial yield strength of the rods measured based on the 0.2% offset method was 445 MPa which was 90% larger than their minimum prescribed yield strength specified by the manufacturer. This amount of strength variation was mainly due to the nature of the steel material used for threaded rods which usually have much larger strength than their minimum prescribed strength.

### 3.4 CYCLIC TESTING OF AYB IN UT10

#### 3.4.1 Design improvements

In order to perform the cyclic tests inside UT10, some adjustments were implemented in the AYB design. Figure 3.6 shows the details of the AYB specimen with the design adjustments. As indicated in this figure, a loading shaft is used with the specimens that transfers the axial loads from the UT10 loading actuator to AYB. The loading shaft is connected to the actuator through a type 1 loading yoke (see Chapter 2) at one end and to AYB at the other end. The loading shaft is designed to remain elastic during loading. It is also in contact with the lateral support beams in UT10 frame to provide lateral support for AYB during loading. As can be seen from Figure 3.6a, special top and bottom connectors were designed that provide the connection of AYB to the loading shaft and the base plate in the UT10 support frame. As indicated in this figure, the top connector is allowed to freely slide in the axial direction on both sides of the spacer plate to accommodate the axial movement of AYB while providing significant bending stiffness in other directions. Following design iterations and preliminary tests, it was discovered that the eccentric axial loads on the specimens in UT10 can cause significant bending, out-of-plane movement, and hence non-uniform loading of the rods at the top connector. Therefore, as can be seen in Figure 3.6a, the top connector was equipped with several stiffeners to reduce the lateral movement under eccentric axial loads by increasing the flexural stiffness of the specimen. In addition, all of the support pipes were fully welded to the top connector. Following these changes, as indicated in Figure 3.6b the design of the top connector was further simplified and optimized for the newly fabricated AYB specimens that were used in the hybrid simulations on the BRBF as discussed in Chapter 4. In this design, six cylindrical shaped support holes are machined in a steel block. These support holes act similar to the support pipes. Threaded holes are drilled at the end of the support holes in the steel block which together with the PT nuts provide the PT threaded connection for the core rods in the top connector with the revised design as indicated in Figure 3.6b. The yielding length of the core rods in the AYB specimens with the revised top connectors increased from 650 mm to 700 mm. During the preliminary tests in UT10, significant bending and weld fracture was also observed in the welded connection between the nut and the pipe in the bottom support pipes. Following this observation, the design of the support pipes used at the bottom of the AYB specimen was revised to improve the flexural stiffness of their connection to the nuts.
Figure 3.6: Design details of the six-component adjustable yielding brace (AYB) specimen for tests inside UT10: (a) the AYB specimen and the loading shaft, (b) revised design of the top connector, (c) revised design of the support pipe, and (d) the support pipes with revised design and bottom connector as fabricated and installed in AYB.

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Figures 3.6c and 3.6d show the details of the support pipe with the revised design. As can be seen in these figures, in the support pipes with the revised design the welded nuts are replaced with a high strength steel cap that is groove welded to the end of the support pipe. A threaded hole is drilled in the steel cap to provide the connection to the core rod similar to a nut.

As indicated in Figure 3.6a two bolts were also used on each side of the spacer plate to push against the top connector and facilitate disassembling the specimen and installation and removal of the core rods after each test. For the cyclic tests in the UT10, the AYB threaded rods were fully replaced with new ones. The new threaded rods were from the same class of material but were supplied from a new batch. Detailed design drawings of the AYB specimen are presented in Appendix B.

3.4.2 External instrumentation

Figure 3.7 shows a prototype of the AYB specimen that was fabricated for cyclic testing installed in the UT10 support frame. The external instrumentation of the specimen that was used for the cyclic tests is indicated in Figure 3.8. As indicated in this figure, similar to the tests in the MTS loading platform, the axial deformations of the AYB core rods were measured by two linear potentiometers (LP1 and LP2) at two sides of the AYB specimen. The average of the measurements of LP1 and LP2 (LP_{avg}) was used as the final axial deformation of the specimen. In order to better understand the mechanics of the AYB specimen during cyclic loading, a three dimensional coordinate measurement system, Nikon K610, was used to measure the deformations of the specimen at various locations. In Nikon K610, three cameras were used to track and record the movements of 6 infrared light emitting diode (LED) targets that were installed at various locations on the AYB specimen. The LED instrumentation map is shown in Figure 3.8 where LEDs are shown by circles. The vertical relative movements of targets 2 and 5 were used as an independent measurement for LP2 measurements and to verify the functionality and accuracy of the linear potentiometer measurements with respect to the 3D measurement system. The rest of the targets were used to measure the in-plane and out-of-plane movements of the UT10 support frame and the AYB specimen.

Figure 3.7: AYB specimen installed inside the UT10 support frame for cyclic testing
3.4.3 Loading protocol

A hybrid simulation strategy was employed to run the cyclic tests inside UT10. For this purpose, the numerical model for the hybrid simulation was created in OpenSees. The AYB specimen was modelled by a physical substructure represented by a two dimensional SubStructure element. In this model, all degrees of freedom were fixed at both ends of the element except for the horizontal movement of one end which was controlled by a cyclic displacement loading protocol through a displacement-controlled integrator. The loading protocol was similar to the loading protocol adopted for the AYB tests inside the MTS loading platform discussed in section 3.3.

3.4.4 Cyclic test results

The test started with two elastic cycles. On the compressive side of the first 1% SDR cycle, slight lateral movements was observed in the UT10 support frame. Therefore, the test was stopped and the specimen was completely unloaded to resolve this issue. The test was then restarted from the second 1% SDR cycle. At the beginning of the second 3% SDR cycle, one of the core rods ruptured in tension which resulted in a 15% drop in the tensile strength of the specimen. The test was then continued until the second core rod ruptured at the beginning of the first 4% SDR cycle which resulted in 17% further reduction in the tensile strength of the specimen. The third core rod ruptured at the beginning of the second 4% SDR cycle after which the test was stopped. All the rods ruptured in their confined zones.

3.4.4.1 Specimen hysteresis

Figure 3.9a shows the cyclic hysteretic response of the AYB specimen as communicated through the network to the OpenSees model. The subplot on the top left of this figure shows a closer view of the response. The response of the specimen during each displacement step as recorded by NICON is also plotted in this graph. It can be seen from this graph that the response of the specimen during the test was slightly unsmooth which was the direct result of the
test method that involved hold times during the loading and slight stress relaxation. However, the response of the specimen as communicated through the network to OpenSees was smooth and did not contain the stress relaxations, which is more representative of the actual response of the specimen under earthquake loads. However, as discussed in the next sections, the contribution of the stress relaxation on the hysteretic curves was negligible.

![Hysteretic response of the AYB specimen tested in UT10](image)

**Figure 3.9:** Hysteretic response of the AYB specimen tested in UT10: (a) recorded hysteretic behaviour during cyclic tests and (b) comparison of the response before core rod rupturing with full-scale BRB test results from Black et al. (2002, 2004)

The load drops in the tension region of the hysteretic response of the AYB specimen in Figure 3.9a were due to the core rod ruptures. As can be seen from this figure, the compressive response and strength of the specimen remained unchanged after the rupture of the first core rod. This was mainly due to the effect of the confining pipe which enabled the ruptured rod to keep its compressive capacity. However, the compressive response and capacity of the specimen reduced after the rupture of the second rod. This reduction was expected to be caused by the stronger secondary effects along the specimen resulting from the unsymmetrical response of the specimen after the failure of the second core rod. It can be also observed from Figure 3.9a that post-tensioning of the core rod connections in AYB fully eliminated the slackness during the load reversals.

The maximum tensile and compressive forces in the AYB specimen tested in UT10 were, respectively, 695 kN and 793 kN which were, respectively, 13% and 14% larger than the maximum forces reached by the AYB specimen in the MTS loading platform. The core rods of the AYB specimen used in the cyclic tests in UT10 were supplied from a batch that was different from the initial batch used for the cyclic tests on AYB in the MTS loading platform. The variability of AYB strength was therefore associated with the variability in the strength of the steel material used for fabrication of the threaded rods.

In order to compare the hysteresis shape of the AYB specimen with the response of a full-scale BRB, the cyclic response of a full-scale BRB specimen is overlapped on the response of the AYB specimen before failure of the core rods in Figure 3.9b. The BRB response was obtained from full-scale cyclic tests on specimen 99-3 performed...
by Black et al. (2002, 2004). This specimen was tested cyclically up to several axial strain levels of the BRB yielding core including 0.7%, 1.4%, and 2.1% that were close to the axial strain levels of the core rods in the tested AYB specimen. The cyclic force-deformation response of the AYB specimen and specimen 99-3 presented in Figure 3.9b are normalized to their respective maximum tensile force and deformation in the last cycle. As can be seen in Figure 3.9b prior to the failure of the first core rod, the AYB specimen showed stable and repeatable hysteretic response both in tension and compression. The softening of the cyclic response on the compression side and reduction in compressive yield strength of BRB after being loaded and unloaded in tension or vice-versa caused by the Bauschinger effect can be also observed. It is observed that the AYB specimen can successfully simulate the strain hardening and the Bauschinger effects that are observed in the full-scale BRB. Similar to a BRB, the compression strengths of the AYB specimen were larger than the tensile strengths with a maximum ratio of 1.13 in the first 3% SDR cycle. This difference was due to the effect of friction between the yielding core (core rod for AYB) and the confinement when the core was in compression.

3.4.4.2 UT10 system performance

3.4.4.2.1 Control system and error compensation

The cyclic tests on the AYB specimen were the first test in UT10 that involved a large amount of forces close to the loading capacity of the actuators. Therefore, in order to investigate the performance of the UT10 system during the cyclic tests, the deformations and forces in the system were recorded and further analyzed. The results are presented in Figure 3.10. Figure 3.10a shows the recorded values between two consecutive target displacements. As indicated in the middle graph, at the time 134.7 s the target displacement sent from the integration module (OpenSees) to NICON through network changed from 3.06 mm to 3.25 mm. This change occurred sharply in an infinitesimal time corresponding to the speed of LabVIEW script. However, since the data recording was performed in 0.1 sec time steps (10 Hz), this transition is shown to happen in 0.1 sec in the graph. As indicated in the top graph of Figure 3.10a, the target displacement was imposed to the actuator during a 2-sec haversine ramp and a 1-sec hold to avoid any sharp movement of the actuator and to give the actuator enough time to reach the specified target displacement and for the system to stabilize. Since a haversine loading ramp was used, the loading rate was not constant during each loading step. The fastest loading rates during the ramp reached 0.12 mm/s. The ramp in Figure 3.10a starts with 6.89 mm which is 3.83 mm larger than the previous target displacement (3.06 mm). This is because the NICON command carries the accumulated displacement error (due to the actuator slacks, etc.) on top of the target displacement (see Chapter 2 section 2.2.5.3). The cumulative error, in this case, was 3.83 mm. Actuator feedback in the top graph shows that the actuator follows the NICON displacement command accurately but with a 0.5-sec time delay. Note that the actuators were initially not designed for testing at high loading rates, low flow-capacity pipes and servo valves were installed, which limited the velocity of actuators stroke. The axial displacement measurements from LP1, LP2, and the average of the linear potentiometer measurements (LP_{avg}) considered as the specimen axial displacement are overlapped with the target displacement in the middle graph in Figure 3.10. It can be seen that the target displacement was achieved by the specimen with errors within the prescribed maximum...
tolerance limit of 0.05 mm. It can be also seen from this graph that the measurements of LP1 and LP2 deviated from each other by approximately 0.3 mm during this stage of loading.

![Figure 3.10](image)

**Figure 3.10:** Deformations and forces of the UT10 system: (a) between two consecutive target displacements and (b) throughout the cyclic test

### 3.4.4.2 Stress relaxation

The bottom graph in Figure 3.10a shows the variation of the axial force during the displacement ramp. It can be seen that the axial force increased during the ramp stage but started to drop during the hold stage. It also slightly dropped at the beginning of the ramp stage when the displacement increase rate was very slow. This force drop was associated with the stress relaxation in the specimen under slower strain rates at the beginning of the ramp stage and constant strain during the hold stage. Stress relaxation is a phenomenon observed in the metals which results from redistribution of elastic and inelastic strains (Stouffer and Dame, 1996). The stress relaxation was observed to occur mostly during the nonlinear response of the specimen. Further analyses of the results revealed that the amount of the force drop due to stress relaxation was maximum of 1.3 % of the total axial force in the specimen and therefore, it had a negligible effect on the overall response of the specimen. Similar observations have been made by other researchers. For example, Mosqueda et al. (2004) observed a 1.5% force drop during a 4.8 s hold time in hybrid
simulations on a two-storey shear frame substructure with steel columns. The stress relaxation can be further reduced by reducing the hold time. The hold time should be long enough to let the system stabilize and the actuators to settle to a displacement. But it should not be excessively long so that the stress relaxation is minimized in the specimens. The force drop due to the stress relaxation quickly recovers in the next displacement step and thus does not affect the response of the specimen in the next steps.

### 3.4.4.2.3 Friction

In UT10, the specimen axial force feedback is measured by the load cells that are installed at the ends of the top vertical actuators. However, the actual axial forces in the specimens are different from the actual forces in the actuators measured by the load cells. This difference is associated with the friction forces developed between the PTFE sheets on the lateral support beams in the UT10 support frame and the loading shafts when the specimens are moving in the axial direction. The amount of friction force can be obtained by multiplying the normal forces on the PTFE sheets by the coefficient of friction between the PTFE sheets and a lubricated steel surface. An upper bound conservative estimate of the friction coefficient is 0.12 based on the information provided on the PTFE material data sheet. The sum of the forces developed in the top out-of-plane and in-plane rigid links during the tests gives an estimate of the normal forces on the PTFE sheets. The magnitude of friction forces was calculated based on the above procedure for the AYB cyclic tests and the results revealed that the friction forces did not exceed 0.7% of the total axial force in the specimen. Therefore the effect of friction forces on the response evaluation of the specimen was negligible and the readings of the top actuator load cells were well representative of the amount of axial force in the specimen.

### 3.4.4.2.4 Lateral movements

The graphs in Figure 3.10b show the history of UT10 displacements and AYB axial force for the cyclic test after the test was restarted from the second 1% SDR cycle. The two top graphs show the average of the linear potentiometer measurements (LP\textsubscript{avg}) and the specimen axial force. The sub-plots in these graphs provide a closer view of the response during the 200-206 s and 330-370 s time windows. The plateaus in the graphs correspond to the times at which the specimen experienced no axial deformation. This happened during the hold stage between two consecutive target displacement commands as indicated in the 200-206 s time window. It also happened during error compensation, for instance during force direction reversals as indicated in the 330-370 s time window or when the test was paused following an incident or for checking the specimen.

As indicated in Figure 3.10b, the vertical displacements between points 2 and 5 were measured by LP2 and also by the optical measurement system. The bottom graph in Figure 3.10b shows the difference between the measurements obtained from LP2 and the optical system (LED5-LED2 V). This graph shows that the maximum differences between the two measurement methods were, respectively, 0.12 mm and 0.45 mm prior to and after the first rod rupture ignoring the local spikes right after the rupture of the rods. The spikes and increases in the measurement difference after the rupturing of the rods were primarily due to the movements of the LEDs at the rupturing instant with respect to their connection points. The above small differences between measurements of two independent sensors confirm the consistency and accuracy of both measurement techniques during the test.

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
To check the out-of-plane bending and deformations of the AYB specimen during the test, the difference between measurements of LP1 and LP2 (LP1-LP2) and the difference between the out-of-plane movements of LEDs 5 and 2 were calculated. The results of the former are presented in the bottom graph in Figure 3.10b. The same analysis was performed for the in-plane bending and movements of the specimen using the difference between the horizontal movements of LEDs 5 and 2 and the vertical movements of LEDs 6 and 4. The results indicated that the in-plane and out-of-plane bending and deformations of the specimen essentially occurred during the compression cycles. The in-plane and out-of-plane deformations were below 3 mm and 5 mm, respectively, which indicated that the lateral deformations of the specimen were well controlled by the UT10 support frame. The maximum difference between the out-of-plane movements of LEDs 6 and 4 which is an indication of the torsion of the specimen was below 0.3 mm during all cycles which was negligible. These results confirm that the UT10 frame successfully reduced the secondary effects on the specimen resulting from actuator eccentric loading during the test.

The histories of vertical, horizontal, and out-of-plane movements of LED1 which show the rigid body movements of the UT10 support frame were also studied and the results indicated that the rigid body movements of the UT10 support frame were below 0.65 mm with the maximums occurring at the peaks of the specimen axial load. The results confirmed that the rigid body movements of the UT10 support frame were successfully controlled by the rigid links during the test.

3.5 ADVANTAGES, POTENTIALS, AND LIMITATIONS OF AYB

Based on the cyclic test results discussed in this chapter, the AYB specimen is capable of simulating the cyclic hysteretic response of a full-scale BRB. Using the modular design of the brace, the stiffness and strength of the brace can be adjusted such that one design can be used to simulate the response of BRBs with a range of stiffness and strength (e.g. BRBs at different floors of a building). One potential feature of the AYB specimen is that its hysteretic properties (e.g. post-yield stiffness, self-centring, etc.) can be also adjusted by choosing various combinations of material properties for the core rods (e.g. high and low strength steel, shape memory alloys, etc.), thus facilitating simulating the hysteretic response of various types of hysteretic braces. The AYB specimen can be restored to its initial undamaged condition with affordable (cost and time) modifications after each test by only replacing the core rods thus facilitating performing several tests with the same AYB specimens. The AYB hysteretic response is resilient and is not significantly affected by eccentric axial loads that may develop in UT10. The ductility capacity of the AYB specimen was reached during the 3% SDR cycles and thus was below the capacity of the full-scale BRBs as reported in the literature. However, AYB still provided an acceptable level of displacement ductility capacity which enables running hybrid simulations on braced frames under large earthquakes. The AYB core rods are machined and threaded along their entire length and their modular configuration, material ductility, confinement, and debonding mechanism, are not similar to what is currently used in practice in a full-scale BRB. Based on this, the low-cycle fatigue response of the AYB specimen may not be similar to the response of a full-scale BRB. Therefore, the failure mechanisms, limit states, and capacities of the AYB specimen are not representative of the full-scale BRBs. Based on this care should be taken when interpreting the hybrid simulation results with AYB.
specimen used to represent full-scale BRBs. In such cases, the hybrid simulation results are only valid prior to the failure of the AYB core rods.

3.6 SUMMARY AND CONCLUSIONS

Development of an adjustable yielding brace (AYB) specimen was presented and discussed in this chapter. The AYB specimen was aimed to simulate the hysteretic response of yielding energy dissipative braces with a variety of strength, stiffness, and hysteresis shape features. The design of the AYB specimen is based on yielding core rods both in tension and compression inside confining pipes which prevent the global buckling and instability of the core rods in compression. The AYB specimen features connection details and a modular design which facilitates conducting several tests on the AYB and controlling of its hysteretic response. The design of the AYB was incrementally improved and the robustness of the design and stability and quality of the of the AYB hysteretic response was verified during several cyclic tests on single- and six-component specimens in UT10 with the presence of loading eccentricities and also in an MTS loading platform with more controlled loading conditions. Between tests, the AYB was restored to its initial undamaged condition and reused quickly with the replacement of the core rods. AYB showed stable and repeatable cyclic hysteretic response both in tension and compression. It was also able to simulate some of the features of the BRB hysteretic curve like the friction effect, isotropic hardening, and the Bauschinger effect thus providing a hysteretic curve similar to a full-scale BRB. The stiffness and strength of the AYB can be adjusted by controlling the number of core rods used in the specimen. Moreover, the hysteretic shape of the AYB can be potentially configured via the application of core rods with different material properties. For instance, controlled self-centring or post-yield stiffness features can be added to the hysteretic response of AYB if proper combinations of shape memory alloy and steel rods are used in it.

Cyclic tests in UT10 confirmed that the combination of the UT10 support frame and the rigid links provided sufficient support for the UT10 frame and AYB specimen during the test with minimal lateral movements and friction. The rigid body movements of the UT10 frame were limited to 0.65 mm and the maximum lateral movement of the AYB was below 5 mm in these tests. The estimated friction forces during the cyclic test was also below 0.7% of the measured axial force in AYB.
CHAPTER 4: MULTI-ELEMENT HYBRID SIMULATION AND SEISMIC PERFORMANCE ASSESSMENT OF A STEEL BUCKLING-RESTRAINED BRACED FRAME

4.1 INTRODUCTION

Buckling-Restrained Braced Frames (BRBF) are among the most popular SFRS. These systems provide lateral stiffness and energy dissipation through the elastic response and yielding of the buckling restrained braces (BRBs) both in tension and compression during an earthquake. Simple design, reliable energy dissipation mechanism, and large ductility capacity of these systems have dramatically increased their application in the last two decades in low- and mid-rise buildings in North America. This chapter presents the details and results of hybrid simulations on a 5-storey BRBF. These hybrid simulations were the first application of UT10 for seismic performance assessment of braced frames. The results contribute to a more realistic understanding from the actual seismic performance of the BRBFs with a large number of storeys. They can be also used as benchmark test data to assess the accuracy of the existing numerical modelling techniques that can be used for realistic seismic performance assessment of the BRBFs. More specifically, the test results can facilitate the identification of potential performance issues that may exist in the available numerical modelling techniques.

One 1-element and two 3-element hybrid simulations were performed on the BRBF in UT10. In these tests, AYB specimens are used to physically represent the BRBs in the first three storeys of the BRBF. The hybrid simulation results are used to verify the functionality and performance of UT10 for multi-element hybrid simulations on braced frames with yielding braces. The results are also compared with the predictions of the fully numerical BRBF models with two different material models for the BRBs to identify the impact of BRB modelling inaccuracies on the seismic response and performance assessment of BRBFs.

This chapter starts with a comprehensive review of background in section 2 on the response of BRBs and BRBFs based on the past experimental and numerical investigations in the literature. The chapter then continues with details of the BRBF design and numerical modelling in sections 3 and 4. In section 5 details of the 1-element and 3-element hybrid simulations on the BRBF with AYB specimens are presented and the results are compared with the fully numerical response predictions. A comprehensive study on the seismic performance of the BRBF based on fully numerical NRHA is presented in section 6. This chapter is concluded in section 7 with a summary of the results and conclusions from the investigations presented in the chapter.

Parts of the material presented in this chapter are published in the following journal/conference papers:
Mojiri, S., Kwon, O., Christopoulos, C. “Development of 10-Element Hybrid Simulation Platform and an Adjustable Yielding Brace for Performance Evaluation of Multi-Storey Braced Frames Subjected to Earthquakes”, Earthquake Engineering & Structural Dynamics (submitted, January 2018, first review received March 6, 2018, second review received August 17, 2018, revised manuscript submitted on September 19, 2018).


Some of the tasks associated with the investigations presented in this chapter were done in collaboration with another member of the research group, Mr. Pedram Mortazavi, MASc, P.Eng., who at the time of writing this report was a Ph.D. candidate at the Department of Civil and Mineral Engineering of the University of Toronto. Pedram designed the BRBF connections and the BRB encasing and helped with calibration and implementation of the connection models in the BRBF OpenSees model. He also performed 3D continuum FE simulations on the BRBF beam-column-brace sub-assemblages to determine their axial compressive capacity. Pedram has also generously helped with and participated in the 3-element hybrid simulations on the BRBF in the Structural Testing Facility of the University of Toronto.

4.2 BACKGROUND

4.2.1 Buckling-restrained Braced Frames

BRBFs are generally defined as the steel frames that use BRBs as their bracing mechanism. Detailed information on the design and response of BRBs and BRBFs can be found in Bruneau et al. (2011) and ATC (2015). A state-of-the-art review of the seismic design of steel structures including the BRBFs is also presented in Uang and Bruneau (2018). The intended energy dissipation mechanism in a BRBF is tensile and compressive yielding of the BRB and the other elements are capacity designed to ensure the formation of this mechanism. For instance, local buckling should be avoided in the BRB gusset plate connections under the BRB maximum compressive forces and the BRB encasing should have sufficient flexural stiffness to ensure that the BRB does not experience global buckling under the expected deformation demands and axial forces. BRBs can be installed in various configurations in braced frames. However, X-bracing configuration is not feasible. In addition, since BRBs can be produced in large capacities with minimal added costs, the single diagonal configurations can be generally more efficient than other V- or inverted V- (chevron) configurations.

A BRB element is composed of a steel core and an encasing mechanism which restrains the compressive buckling of the steel core. The steel core is axially decoupled from and hence can axially move freely relative to the encasing. This movement is facilitated by a de-bonding material or simply an air gap between the steel core and the encasing. Therefore, in a BRB, the axial loads are resisted only by the steel core. Figure 4.1 shows different components of a BRB with a rectangular steel core. As indicated in this figure, the BRB steel core is composed of three segments. The middle segment is called the confined yielding segment or simply the yielding core. The energy
dissipation in a BRB occurs through axial tensile and compressive yielding and plastic deformations in this segment. The ends of the confined yielding segment are reinforced to prevent local buckling and ensure elastic response of the steel core at both ends. These non-yielding segments are partly confined (restrained) by the encasing and partly unconfined. The confined non-yielding segments provide smooth transitions between the yielding core and the unconfined non-yielding segments. The unconfined non-yielding segments provide the BRB connection to the frame at both ends which can be achieved mostly through a bolted connection. Fully pinned and welded connections are also possible. Various configurations are proposed and tested in literature for the core cross section shape including cruciform, rectangular, and multilayer configurations. Similarly, different configurations are proposed and tested in the literature for the restraining mechanism. These generally include HSS sections filled with concrete or mortar and all-steel restraining mechanisms.

Figure 4.1: BRB components

Figure 4.2: BRB hysteretic response
Although the first mode buckling of the BRB yielding core is restrained by the encasing mechanism, the small gap between the steel core and the encasing allows this segment to buckle in higher modes and thus interact with the encasing. In addition, due to the Poisson effect, the steel core expands under compression. The air gap is, therefore, necessary to accommodate this expansion. The air gap and the de-bonding material reduce the friction forces that can develop in the steel core and the encasing under compression. However, as a result of the restraining mechanism, unlike the conventional braces, the compressive strength of the BRB is slightly larger than its tensile strength. The difference between the tensile and compressive strength of the BRB becomes larger as the friction forces increase during the large axial compressive displacements of the BRB yielding core.

Figure 4.2 shows the axial hysteretic response of a BRB. As can be seen in this figure, the axial hysteretic response of a BRB provides large energy dissipation. The hysteretic response involves significant tensile and compressive yielding and plastic deformation resulting in considerable combined isotropic and kinematic hardening. As can be seen in Figure 4.2, the cyclic hysteretic response of a BRB shows gradual stiffness change and softening at the onset of yielding during load reversals. This phenomenon is called the Bauschinger effect and is a natural property of the steel material (Bruneau et al., 2011). The Bauschinger effect is attributed to the decrease of the yield strength of steel when the strain direction changes. In the case of the cyclic response of BRBs, it results in a reduction in compressive yield strength and softening of the steel after being loaded and unloaded past the yielding point in tension or vice versa. The hysteretic response of the BRB is slightly asymmetric because of the difference in the compressive and tensile strength of the BRB. Since the local buckling of the BRB core is eliminated or controlled, large stress concentrations do not occur in the BRB core. The peak strain in the BRB core is anticipated to be 1-3% for severe ground motions depending on the length of the yielding core. Therefore, if the core is properly de-bonded from the encasing, BRBs have a large cumulative ductility capacity and rather long low-cycle fatigue life which is sufficient to sustain multiple severe earthquakes (Fahnestock et al, 2003; Sabelli et al., 2003; Tremblay et al., 2006).

4.2.2 Seismic performance of BRBFs

The BRBF concept was first developed in Japan by the pioneering work of Wakabayashi et al. (1973) on the response of steel plates sandwiched by two precast concrete panels and further advanced and experimentally verified by Watanabe et al. (1988) for concrete filled tubular encasings. The application of BRBFs rapidly increased after the 1995 Kobe earthquake in Japan and after the 1994 Northridge earthquake in North America. Since then there have been extensive research studies on the component- and system-level performance of BRBs. A summary of some of the studies are presented and discussed below:

4.2.2.1 Pioneering cyclic tests by Watanabe et al. (1988)

The pioneering cyclic tests on BRBs with single plate cores and concrete filled tubular encasings was performed by Watanabe et al. (1988). They tested five full-scale BRB specimens in a diagonal configuration in a single-bay single-storey steel frame under cyclic loads. They observed stable axial hysteretic response and significant energy absorption in the BRB specimens. They also observed that maximum 5% of the compressive axial load was transmitted to the encasing and concluded that their de-bonding mechanism worked well. For the specimen in which
the buckling load capacity of the encasing was smaller than the yielding load of the core, they observed an abrupt decrease in the compressive strength of the BRB following the global buckling of the specimen.

4.2.2.2 Component-level and sub-assemblage experimental and analytical investigations at the University of California, Berkeley

A series of component-level cyclic tests were performed at the University of California, Berkeley during 1999 and 2000 years on 5 full-scale concrete-filled BRB specimens manufactured by the Nippon Steel Corporation, Japan. The specimens and their slip-critical bolted connections were representative of the BRBs used in the University of California Davis Plant and Environmental Science Building and the Kaiser Santa Clara Medical Centre which were both among the first projects in North America that utilized BRBs. The BRBs were tested under various loading protocols including standard cyclic, large deformation, low-cycle fatigue, and earthquake induced displacement histories. A summary of the test results is presented in Black et al. (2002, 2004). The results indicated that all BRB specimens indicated stable and reliable nonlinear hysteresis and large energy dissipation and nonlinear ductility capacity. The test results confirmed that the BRBs are capable of not only providing the required rigidity to control the structural drifts but also delivering significant reliable energy dissipation capability. They measured the cumulative displacement ductility demands of the BRBs and concluded that the plastic deformation capacity of the BRBs well exceeded the minimum specified requirements for the projects. The experimental results also indicated that the stiffness of the elastic segments of the BRB considerably affects the overall stiffness of the BRBs increasing the global stiffness of the brace by 17%. Black et al. (2002, 2004) also did an analytical stability analysis on the BRBs. They studied various instability modes including the global flexural buckling of the BRBs, local buckling of the BRB yielding core, and the plastic torsional buckling of the unrestrained segment of the BRB core. They derived theoretical equations based on the incremental theory of plasticity to predict the stability of the BRBs which they then verified against the experimental results. Their investigations revealed that plastic torsional buckling of the unrestrained segment of the BRB core is the most critical instability mode. Further, they used a phenomenological model to predict the hysteretic response of the BRBs based on a Bouc-Wen model. They calibrated this model with the experimental results and used it to conduct analytical studies on the nonlinear response of an SDOF system comprised of a nonlinear BRB inside a linear diagonally braced single-storey single-bay frame. Their investigations revealed that the brace ductility demand increases with the period of the linear unbraced frame.

The first sub-assemblage tests on BRB specimens in the United States were conducted at the University of California, Berkeley. Details of the test results and investigations are presented by Mahin et al. (2004) and Uriz and Mahin (2008). The tests were performed as part of a design process for a building to replace the Stanley Hall at the University of California, Berkeley. The main purpose of the tests was to verify the impact of the frame rotational and flexural demands on the performance of the BRBs and their gusset plate connections. For this purpose, three steel braced frames representing the first storey of a tall building were tested under quasi-static cyclic loads. The BRBs were fabricated by the Nippon Steel Corporation in Japan and were installed in inverted V and diagonal configuration in the frames. The beam-column connections of the steel frame were ductile moment resisting connections and thus the moment action contributed significantly to the lateral strength of the braced frame.
specimens. They observed stable and reliable hysteretic responses from the BRB specimens similar to the component-level response that was expected. They also observed significant unexpected shear action and yielding in the columns and local buckling and fracture in the gusset plates during the tests. The local buckling and fracture of the gusset plates did not detrimentally impact the global response of the frames possibly due to the presence of the moment resisting beam-column connections. However, fracture of the gusset plates was deemed to be the result of a fabrication fault and repeated use of the steel frame for testing and was not expected to occur in real practice. The local buckling of the gusset plate was successfully controlled by stiffeners in the subsequent tests. The test results confirmed the effectiveness of using stiffeners perpendicular to the edges of the gusset plates in eliminating the local buckling of the gusset plates. Uriz and Mahin (2008) also performed numerical analyses on the sub-assemblage specimens. In their numerical study, they used Menegotto-Pinto material model and truss elements available in OpenSees and did not consider any flexural actions on the brace elements. The good match between the numerical and experimental results revealed that the in-plane flexural and rotational demands did not adversely affect the hysteretic response of the BRBs and had a negligible contribution to the global behaviour of the braced frame. Uriz and Mahin (2008) also performed nonlinear response history analysis (NRHA) on 3- and 6-storey BRBF and SCBFs under ground motions representing various hazard levels. In this study, they assumed rigid elements for the non-yielding portions of the BRB cores and employed a calibrated low-cycle fatigue model for the BRB core. Their analysis revealed that the BRBF performance was superior compared to the SCBF. The storey drifts were smaller in BRBFs compared to the SCBFs and they observed no BRB fracture due to low-cycle fatigue in any of their numerical analyses. They concluded that based on the brace deformations, the amount of damage in BRBFs during the frequent earthquakes was less than the SCBFs. For both BRBF and SCBF they observed large storey drift ratios (near 4%) under the MCE level ground motions that should be considered in the capacity design of the gusset plates.

4.2.2.3 System-level numerical study by Sabelli et al. (2003)

To identify the system-level seismic response and performance of BRBFs, Sabelli et al. (2003) performed NRHA on 3- and 6-storey BRBFs with stacked chevron bracing configuration under a suite of 20 ground motions representing the seismic hazards in Los Angeles. The ground motions were scaled to match various hazard levels corresponding to frequent, DBE and MCE level. The BRBFs were designed based on a capacity design approach. They investigated the effect of certain design parameters including the floor beam stiffness and the magnitude of the response modification factor (R). In the numerical model, the BRBs were modelled with truss elements and the BRB yielding core was assumed to constitute 70% of the BRB total length. The hysteretic response of this part was modelled by an elastic-perfectly plastic nonlinear hysteretic model. The non-yielding parts of the BRB core were modelled by elastic elements. Rigid beam-column connections were assumed for all floor levels except the top floor level and a fixed condition was assumed for the column base. The numerical results indicated an average peak SDR value of 1.6% and 4.5% for the 6-storey BRBF under the DBE and MCE ground motions, respectively. The average value of the maximum residual SDR was in the range of 0.4-0.7% for the frequent and DBE ground motions but significantly increased to 2.2% under the MCE ground motions. The results revealed that the continuity of the columns and the rigidity of the beam-column connections can successfully transfer the drift demands across the
BRBF height. It was observed that while the response of the BRBF under the frequent earthquakes was characterized by a flexural type of response with maximum SDR values on the top storeys, accumulation of lateral drift in the lower stories was more evident under the DBE and MCE level ground motions. Based on their investigation, increasing the floor beam stiffness slightly reduced the average peak SDR values but considerably reduced the average of BRB maximum and cumulative ductility demands. The height of the BRBF and the R value did not have a significant impact on the average of maximum SDR and BRB ductility demand. However, the average of the cumulative ductility demand for the 3-storey BRBF was less than half of the 6-storey BRBF.

4.2.2.4 Experimental and analytical investigations at Ecole Polytechnic of Montreal, Canada

To investigate the benefits of adopting BRBs with short length yielding cores, Tremblay et al. (2004) performed NRHA on three-storey steel frames braced with BRBs with long and short yielding core lengths as well as conventional braces. The frames were designed following the 2005 version of National Building Codes of Canada (NBCC) (NRC, 2005). In their numerical study, they modelled the BRBs with bar elements with equivalent cross section area taking into account the larger area of the BRB core non-yielding segments and connections. They calibrated the BRB numerical models with the cyclic test results on full-scale concrete-filled BRBs. The numerical predictions indicated damage accumulation in the first storey for the frames with all types of braces, but the damage accumulation was more pronounced for the frames with the short core BRBs and the conventional braces. In general, they observed smaller lateral drifts for the frames with conventional braces compared to the BRBFs. They showed that adopting short core BRBs and the conventional braces resulted in more symmetrical hysteretic response and hence less vulnerability to pulse-type ground motions compared to the long core BRBs. In addition, adopting the short core BRBs resulted in smaller lateral deformations but larger strain demands in the BRB cores compared to the long core BRBs. In their study, the BRBFs showed similar seismic performance compared to the frames designed with conventional braces. However, the BRBFs resulted in smaller foundation and base shear forces. As one of the first investigations on the response of the BRBs in North America, Tremblay et al. (2006) tested six full-scale BRBs subassemblies inside a single-storey steel frame. In their tests they investigated the type of the restraining mechanism (concrete-filled vs. all-steel), length of the BRB restrained yielding core, and the loading rate (dynamic vs. quasi-static). They observed stable and repeatable hysteresis with increasing strength and significant ductility capacity for the concrete-filled BRB specimens throughout the loading protocol regardless of the yielding core length. The all-steel BRBs, however, demonstrated local buckling at the ends of the restrained yielding core which increased the friction and the compressive strength of these specimen which eventually lead to concentration of compressive and tensile strain, respectively, at the ends and middle of the core and an early low-cycle fatigue fracture for the all-steel braces. They proposed to use a tight bolt spacing, stiffer restraining plates, minimized clearance between the core and the restraining mechanism, and a de-bonding material to reduce the local buckling of the core ends and lower the direction to achieve a more uniform strain distribution and thus a longer low-cycle fatigue life for the all-steel BRB specimens. They also observed that the loading protocol with more numerous but smaller amplitude plastic excursions, which is more representative of an actual earthquake loading, imposes less low-cycle fatigue demand on the BRB cores compared to the quasi-static loading protocol with less numerous but larger plastic amplitude excursions. They also observed that compared to the quasi-static loading.
protocol, the dynamic loading protocol with higher strain rate demands increase the yield strength of the BRB specimens by 5%. Their experimental investigations on the effect of the in-plane flexural demands on the BRB encasing revealed that such demands do not impact the performance of the BRB specimen. They investigated the energy dissipation capacity of the BRB specimen and a conventional brace and concluded that the cumulative energy dissipated by the concrete-filled BRB specimens during the quasi-static loading protocol was 770% larger than the conventional brace which confirms the superior energy dissipation capacity of the BRB specimens. The cumulative energy dissipation of the BRB specimens was also 22% larger than an equivalent elastic-plastic system mainly due to the gradual increase in the tensile and compressive strength of the BRB specimens as a result of hardening and friction effects. Dehghani and Tremblay (2018) further investigated and developed the all-steel BRB concept by designing and testing 12 full-scale all-steel BRB specimens with an improved restraining mechanism. The particularly investigated various restraining mechanisms and interface materials. To limit the flexural demands on the BRBs during large storey drifts, they used knife plate connections between the gusset plate and BRB end zones and allowed the formation of the plastic hinge in the knife plates. They observed stable and repeatable hysteretic response, extensive ductility capacity, and satisfactory performance from the all-steel BRB specimens sufficient for multiple long duration severe earthquakes.

4.2.2.5 System-level numerical and large-scale experimental simulations at ATLSS Center, Lehigh University

Numerical and large-scale experimental simulations were performed on a 4-storey BRBF in ATLSS Center, Lehigh University to assess the seismic performance of BRBFs and verify the building code seismic provisions and response predictions. As part of this research project, Fahnestock et al. (2007a) performed NRHA on a prototype building with 4 storey BRBFs and with BRBs installed in a stacked chevron configuration. They evaluated the response of the BRBF under 16 ground motions, selected and scaled to match DBE and MCE seismic hazard levels, and verified the performance of the BRBF based on the performance criteria defined in terms of yielding of beams, columns, and connections, storey drifts, and the brace ductility demands. Their analyses results indicated that the BRBF fulfilled the performance criteria for both life safety and near collapse performance levels under respectively DBE and MCE ground motions. Limited yielding was observed in the beams, columns and the beam-column connections. The average value of the transient and residual drifts under the DBE ground motions were respectively 2% and 0.5% with significant scatter between the ground motions. The values of the same responses were respectively 3.3% and 1.2% under the MCE ground motions. The drifts were uniformly distributed in the BRBF height with a slight concentration in the second storey. The average of the maximum and cumulative brace ductility demands reached 18 and 179 under the MCE ground motions. Their analysis suggested that the building seismic provisions especially the response modification coefficient ($R$) and the deflection amplification factor ($C_d$) were underestimating the actual ductile response and nonlinear deformations of the BRBFs. They also suggested a rigorous equation to more accurately predict the brace maximum ductility demand.

Fahnestock et al. (2007b) also performed real-time pseudo-dynamic hybrid tests on a 0.6-scale model of the BRBF. The main purpose of their tests was to better understand the system-level performance of the BRBs when...
installed in frames. The BRBF test specimen was a 4-storey braced frame with 8 BRBs installed in a stacked chevron configuration. The specimen was loaded by 4 actuators at each floor level through loading beams. The beam-column-brace connection detail in the BRBF specimen consisted of a fully pinned brace connection and a shear tab connection between floor beams and stub beams. This detail was chosen to reduce the flexural demands on the BRBs and improve the performance of the brace connection which was seen to be a critical failure mode hindering the actual component-level capacity of BRBs in the past tests. The BRBF specimen was tested under four ground motions representing the seismic hazards associated with frequent earthquakes, DBE, MCE, and an after-shock equivalent to 80% of the DBE. They also performed quasi-static cyclic tests on the BRBF specimen to verify the ultimate capacity of the system. Their test results revealed that the BRBF could withstand the earthquake loads with no strength degradation and with minor yielding and damage to the beams, columns, and connections. The maximum achieved transient and residual SDR during the DBE ground motion were 2% and 1.3%, respectively. These drift values increased to 4.8% and 2.7% under the MCE ground motion. The BRBs showed significant overstrength and hardening, stable hysteretic response, extensive energy dissipation, and large ductility capacity. The maximum ductility demand in the BRBs reach 26 under the MCE ground motion and the BRB specimens could withstand a maximum cumulative ductility demand of 453 during all tests. They observed that almost all the seismic input energy was dissipated in the BRB specimens mainly in the first two storeys. They also observed excellent performance from the beam-column-brace connections which they mainly attributed to the stocky gusset plates, the fully pinned brace connections, and the BRB end collars which restrained the out-of-plane deflection of the BRB and thus reduced the secondary effects on the gusset plates.

4.2.2.6 Pseudo-dynamic hybrid and numerical simulations of full-scale BRBFs at NCREE, Taiwan

As a US-Japan-Taiwan cooperative research program in 2003, seismic performance of a full-scale 3-storey BRBF with concrete-filled tube (CFT) columns was evaluated by a series of quasi-static pseudo-dynamic hybrid tests. A summary of these investigations is presented in Tsai et al. (2008). The BRBF specimen was designed following a displacement-based approach and had three bays. The BRBs were installed only in the middle bay in a stacked chevron configuration through bolted and welded connections to the gusset plates. Three different types of BRBs were installed in the BRBF specimen: double-core all-steel BRBs, single-core concrete-filled BRBs, and double-core cement mortar-filled BRBs. The BRBF specimen was built with concrete footings and slabs to develop the composite action of the beams. Except for the exterior connections, the beam-column connections were non-moment resisting. The seismic response of the BRBF was evaluated by a series of tests using frequent, DBE, and MCE ground motions which were then followed by cyclic tests to evaluate the ultimate capacity of the BRBF. The tests were performed in two phases and the BRBs were fully replaced after each phase. Out-of-plane buckling of the gusset plates was observed during the first phase tests which resulted in out-of-plane bending of the BRBs. Therefore, stiffeners were added to the free edges of the gusset plates to improve their stability for the rest of the tests. In the first phase of testing, the maximum SDR reached 0.5%, 1.9%, and 2.3% during the frequent, DBE, and MCE ground motions, respectively. These responses reached 2.4% and 2.6% during the DBE and MCE ground motions in the second phase of the tests. The beam-column connections were observed to experience yielding with

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
no fracture after all the tests. The concrete slabs started to crack during the DBE and MCE ground motions. All BRBs responded well and only fractured during the 3.8% drift cycle after experiencing extensive nonlinear deformations. Overall, the test results indicated that the BRBF performed very well during multiple large earthquakes and stiffening the gusset plates successfully eliminated the out-of-plane deformations of the BRBs. The response of the BRBF specimen was also numerically evaluated. In the numerical models, the BRBs were modelled by truss elements with an isotropic and kinematic strain hardening material. The numerical models successfully predicted the seismic response of the BRBF specimen. However, large errors were observed for the numerically predicted lateral deformations. The errors reduced significantly when the composite action of the slabs was considered in the numerical models. Tsai and Hsiao (2008) further studied the performance of the BRBs and the gusset plate connections. Their investigations revealed that the 0.65 effective length factor proposed by Thornton (1984) was not sufficient to ensure the stability of the gusset plate connections. In the absence of the free-edge stiffeners in the gusset plates, they proposed adopting a value of 2 instead of 0.65 for the gusset plate effective length factor.

In a separate study, a series of pseudo-dynamic hybrid and cyclic tests were performed on full-scale three-storey BRBF with a single bay in 2010 by Lin et al. (2012). The BRBF specimen was tested under DBE level ground motions. The main purpose of these tests was to verify the performance of special types of BRB gusset plate welded connections which were aimed to improve the system-level performance of the BRBs. The test results were also used to verify the system-level performance of a proposed concept for BRBs with a light design for the encasing (thin BRB). The test results indicated that the maximum SDR reached 3.3% under the DBE ground motions. The BRB specimens experienced a maximum core strain of 4% and sustained a large amount of plastic deformation without strength degradation throughout the tests. It was also observed that the welded gusset plates performed rather well under repeated large storey drift demands. Based on the experimental results, Lin et al. (2008) proposed equations to predict the maximum force exerted by the BRB core to the encasing that can be used to determine the thickness of the thin BRB encasing tube in the design procedure.

Khoo et al. (2016) studied the performance of the BRBs and the gusset plates under simultaneous in-plane and out-of-plane seismic loads. For this purpose, pseudo-dynamic and cyclic bidirectional tests were performed on a full-scale 2-storey BRBF with diagonal braces. Particularly, the frame action effect on the performance of the gusset plates was studied. The experimental results indicated that the BRBF reached SDR values of 2.2% and 1.5% in the in-plane and out-of-plane directions, respectively and the BRBs and the gusset plate connections showed stable behaviour throughout the tests. Their experimental and FE simulation investigation revealed that the frame action on the gussets is significant and should be accounted for in addition to the BRB axial forces to ensure reliable performance. Pseudo-dynamic hybrid tests were also performed by Wu et al. (2017) on full-scale 2-storey reinforced concrete (RC) frames braced with BRBs and the results showed that the BRBs can provide extensive stable energy dissipation and thus effectively control the seismic demands for this type of SFRS.
4.2.3 Numerical modelling of BRBs

Since the global buckling of the BRB core is restrained, the nonlinear response of the BRB is mainly controlled by the uniaxial nonlinear response of the metal used in the BRB yielding core. For this reason, BRBs are usually modelled by truss elements with a unidirectional nonlinear material model. NEHRP guidelines (ATC, 2015) recommends using nonlinear truss or frame elements and suggests that an elastic-plastic model with strain hardening and no stiffness and strength degradation provides reasonable accuracy for BRBs with well-detailed configurations. ATC (2015) requires calibration of the numerical models including the hardening parameters to be based on BRB cyclic tests data. It also requires consideration of the stiffening effect of the elastic parts of the BRB including the gusset plates and the non-yielding segments of the BRB core. Such effects can be explicitly considered by separately modelling each part of the BRB with its corresponding stiffness and material properties. Another approach is to use the cross section of the BRB yielding core uniformly between the BRB working points and retain the actual stiffness of the BRB by appropriately increasing the elastic stiffness of the BRB core material.

Various models were proposed and adopted in the literature for the nonlinear material model of the BRB yielding core. The simplest model is an elastic-perfectly plastic model with no hardening and zero post-yield stiffness. Sabelli et al. (2003) used this model to investigate the seismic performance of 3- and 6-storey BRBFs as discussed in section 4.2.2.3. Kiggins and Uang (2006) used a similar model to study the effect of dual systems in reducing the residual drifts in BRBFs. Although this model is very simple to use it tends to result in overestimation of the lateral deformations of the BRBFs. Bi-linear model with sharp transitions between the elastic and inelastic zones and with post-yield stiffness is the most popular and widely available modelling approach for the hysteretic response of the BRBs and has been adopted both in the industry and the research. As an example, Asgharian and Shokrgozar (2009) used Steel01 bi-linear model in OpenSees with 2% post-yield stiffness to perform a parametric study on the BRBF response modification factors. The parametric studies performed by Black et al. (2004) on the response of the single-storey BRBFs indicated that the participation of the BRB in the nonlinear response of the single-storey braced frames can be satisfactorily captured by a bi-linear model. Bouc-Wen model was used in several research studies to model the nonlinear response of BRBs (Black et al., 2004; Karavasilis et al., 2012; Dehghani, 2016). This model features a smooth transition between elastic and inelastic regions and is able to model the kinematic hardening. The modified Bouc-Wen model developed by Karavasilis et al. (2012) can model the combined effect of the isotropic and kinematic hardening. Tremblay et al. (2004) investigated the effect of the BRBF modelling accuracy on the numerical response predictions. They compared the numerical response of the BRBFs with a bi-linear model and also with a more realistic model with Ramberg-Osgood formulation. They concluded that using a bi-linear model for the hysteretic response of the BRBs results in an underestimation of the BRB deformation and strain demands compared to when a more realistic model with isotropic hardening and smooth transition between the elastic and inelastic regions is employed. Rossi (2014) further investigated the effect of isotropic hardening on the accuracy of the response predictions for multi-storey BRBFs. He used the model developed by Zona and Dall’ Asta (2012) with the combined effect of isotropic and kinematic hardening to model the nonlinear response of the BRBs. He then compared the results with bi-linear models with only kinematic hardening. He concluded that the bi-linear models that do not consider the isotropic hardening are likely to produce inaccurate response predictions and that this

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
inaccuracy increases with the increase in the number of storeys in BRBFs. He also concluded that depending on the level of the BRBF ductility demands, the actual response of the BRBFs can be overestimated or underestimated if bi-linear models are adopted. Similar observations were made in a sensitivity analysis conducted by Zsarnóczay (2013). Giuffre-Menegotto-Pinto material model (Menegotto and Pinto, 1973; Filippou et al. 1983) was used by a number of researchers to model the response of BRBs (Uriz and Mahin, 2008; Tsai et al., 2008; Prinz and Richards, 2011). This model is implemented as Steel02 uniaxial material model in OpenSees and can model combined kinematic and isotropic strain hardening. In this model, the isotropic hardening can be defined separately for the tensile and compressive forces. This feature can be used to model the increase of the compressive forces due to the friction effect in the BRB.

### 4.2.4 Summary of the findings

Based on the above review of the literature, the following key conclusions can be made:

- **BRBFs are rather modern steel SFRSs with rapidly increasing application in North America which provide superior and more reliable ductility and energy dissipation capacity compared to other conventional bracing systems like SCBFs both in component- and system-level configurations.**

- **The literature indicates that the strain in the BRB yielding core is within 1-2% or 3-5% for shorter cores during severe earthquakes. Due to the low axial strain demands and the rather uniform stress distributions in the BRB yielding core, the BRBs can show stable and reliable performance without signs of degradation or low-cycle fatigue fracture even under several severe earthquakes.**

- **BRBs show significant overstrength mainly due to the isotropic and kinematic hardening of the steel core and the effect of the restraining mechanism. Such effects should be considered in the capacity design of the BRBFs.**

- **The BRBFs can experience a large amount of SDR over 2% during DBE earthquakes and thus the BRBF design can be controlled by drifts. The performance investigations on the BRBFs in the literature indicates that the SDR values in these systems can easily increase to values above 3% during the MCE ground motions imposing increased risks of instability and collapse mainly due to the lack of adequate post-yield stiffness. This is a major drawback of BRBFs. For the same reason, these systems are also prone to the concentration of damage and formation of soft-storey mechanisms particularly in the first and second storeys under severe earthquakes.**

- **The frame action of the beam, columns, and the connections in BRBFs and the flexural capacity of the gravity supporting system has shown to be effective in reducing the SDR values during severe earthquakes. Various systems, such as BRBF-MRF dual systems, are also proposed to improve the post-yield stiffness of the BRBFs.**

- **The in-plane flexural and rotational demands in BRBFs do not hinder the excellent performance of the BRBs provided that proper design considerations are employed for the BRB gusset connections to ensure their stability during large BRB compressive actions and the frame actions.**
- The BRB concept has matured over the past two decades and various configurations (concrete-filled, all-steel, short-core, multi-core, hybrid, self-centring, etc.) are proposed based on this concept which are all proved and tested successfully.

- Bi-linear models with kinematic hardening are widely used particularly in the industry to model the hysteretic response of the BRBs. However, research studies indicate that such modelling technique does not address important features of the BRB response including the isotropic hardening and the smooth transition between the elastic to plastic response and thus the bi-linear model can result in erroneous estimation of the actual nonlinear response of the BRBs.

There are large amounts of large-scale experimental and analytical investigations in the literature on the component- and system-level performance of BRBFs. Many of the system-level tests were performed in a hybrid manner with the full BRBF tested as a physical specimen. Although this specimen configuration provides realistic boundary conditions for the braces, various factors such as the laboratory space and cost/complexity of constructing the frame pose limitations on the number of stories and the scale of the tested frame in this approach. It is also hard to directly measure the axial forces in the braces and thus the indirect force measurements based on methods like strain measurements can be potentially erroneous. On the other hand, it is proved by the system-level test results that if BRB elastic segments and gusset plates are designed properly, the system-level and component-level hysteretic response of the BRBs are almost identical. This observation suggests that if the beams, columns, and connections are numerically modelled with sufficient accuracy, since the hysteretic response of the BRBs has a dominant impact on the system-level seismic response of the BRBFs, there is no need to test the full frame in a hybrid simulation and multi-element hybrid simulations with only the braces tested as physical specimens can be employed instead.

The current practice in the structural engineering industry involves the employment of simplified models to model the hysteretic behaviour of the energy dissipating elements in structures (Dutta et al. 2010; Deierlein, et al., 2010, 2015). However, questions have been raised on the accuracy of current modelling tools and the code prescriptions adopted by the industry for NRHA and its impact on performance and risk assessment of structures. Recently, Applied Technology Council started a new project (ATC-114) to address some of these concerns aiming at developing updated hysteretic modelling parameters and acceptance criteria for seismic analysis and performance assessment of new and existing buildings (Hamburger et al., 2016; ATC, 2017a, 2017b). Since the nonlinear hysteretic response of a BRB is almost symmetric, it is regularly modelled with simple bi-linear models. It is believed that such simple models can predict the system-level response of the BRBFs with sufficient accuracy. However, the actual hysteretic response of a BRB involves modelling details including the combined effect of isotropic and kinematic hardening that can be modelled by more detailed material models. On the other hand, the accuracy of the numerical models in simulating the hysteretic response of BRBs under ramped cyclic loads does not guarantee the sufficiency of these models to replicate the actual response of the BRBs under irregular cyclic loads during an earthquake. In this context, experimental test data on the realistic system-level response of multi-storey BRBFs under actual earthquake loads facilitate evaluating the implications of adopting simple BRB hysteretic models on the seismic performance predictions of multi-storey BRBFs. Such test data also aid in the identification
of the most appropriate material models that can accurately predict the response of BRBs under any loading protocol.

Based on the above discussions, performing multi-element hybrid simulations on BRBFs with the braces tested as physical specimens are highly beneficial. Multi-element hybrid simulations on BRBFs provides better understanding from the actual seismic performance of the BRBFs with a larger number of storeys based on realistic experimental test data. The results can be also used as the benchmark test data to verify the accuracy of various numerical modelling techniques used for seismic performance assessment of BRBFs. In this context, development of the AYB specimen (discussed in Chapter 3) that can be used in the hybrid simulations can further facilitate performing multiple multi-element hybrid simulations and thus promote parametric experimental studies on the system-level implications of hysteretic shape and response of the yielding braces.

4.3 DESIGN OF THE BRBF

4.3.1 Building layout

The BRBF considered was the SFRS of a steel frame office building located on soil type C in Los Angeles. The BRBF was designed following American building design provisions ASCE 7-10 (ASCE, 2010), ANSI/AISC 360-10 (AISC, 2010a), ANSI/AISC 341-10 (AISC, 2010b). Details of the building plan are shown in Figure 4.3a. As indicated in this figure, two BRBF frames were used as the SFRS in the south-north direction. The SFRSs in the perpendicular direction were three-bay moment resisting frames (MRFs).

![Figure 4.3a: Plan of the building](image)

**Figure 4.3a:** The 5-storey building considered for the design of the BRBF: (a) plan of the building and (b) elevation of the BRBFs

Figure 4.3b shows the elevation of one of the BRBFs. As indicated in this figure, two braced bays were considered for each BRBF. Single braces in a zig-zag configuration were used in each braced bay. The single bracing configuration in each bay helps in reducing the axial strain demands in the BRB yielding cores which in turn reduces
the hardening of the yielding core and thus overstrength of the BRBs (Bruneau et al., 2011). The zig-zag configuration was adopted to minimize the axial loads in the floor beams (ATC, 2015). On the other hand, having two braced bays in each BRBF reduces the maximum forces at the column baseplates and aids in a better distribution of the building base shear under earthquake loads resulting in a more efficient design for the foundation. In addition, the number of braced bays increases the redundancy of the building and allows the reduction of the design base shear according to ASCE 7-10 (ASCE, 2010). All columns in the building were continuous over their length and had splices at 1.2 m above the third floor as indicated in Figure 4.3b. Continuity of the columns helps redistribution of the inelastic demands along the building height thus limits the concentration of deformation and damage and reduces the chances of formation of the soft storey mechanisms. The gravity loads in the building were mainly supported by the gravity columns.

4.3.2 Design of the elements

In order to design the BRBF, the base shear of the building in the south-north direction was determined from a modal response spectrum analysis. The target response spectrum considered was the design level seismic response spectrum of Los Angeles with 10% probability of exceedance in 50 years. The design spectrum was assumed to be approximately 2/3 of the risk targeted Maximum Considered Earthquake (MCE_R) level response spectrum with 2% probability of exceedance in 50 years. The MCE_R level response spectrum for Los Angeles and soil type C was determined from ASCE7-10 (ASCE, 2010) and the design response spectrum was obtained by considering 2/3 of the MCE_R level response spectrum values. The resulting design response spectrum is shown in Figure 4.4.

![Figure 4.4: 5% damped uniform hazard design response spectrum for Los Angeles](image)

The base shear of the building was calculated based on the full dead loads of the building. Since the building plan was symmetric, the torsional effects and the effects of the out-of-plane structural members were neglected except for the accidental torsional effects in the building. The lateral stiffness and energy dissipation was mainly provided by the brace elements in the BRBFs. Therefore, the brace elements in the BRBFs were designed for the full base shear forces. The columns and beams in the BRBFs were then capacity designed for the maximum expected tensile and compressive axial force capacity of the brace elements. The beams and columns were designed with ASTM A992 steel material with a specified yield strength of 345 MPa. Several design iterations were performed to
achieve the most efficient section sizes. The first and second modal period of the building with the final design in the south-north direction were 1.2 s and 0.44 s, respectively. The total design base shear of the building in this direction was 1500 kN. The final section sizes of the columns (C) and beams (B) are presented in Table 4.1. The column numbers are based on the numbering used in Figure 4.3.

Table 4.1: The BRBF final section sizes for columns and beams

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>Columns</th>
<th>Beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C1, C2, C3, C4</td>
<td>Gravity</td>
</tr>
<tr>
<td>1</td>
<td>W250x101</td>
<td>W310x67</td>
</tr>
<tr>
<td>2</td>
<td>W250x101</td>
<td>W310x67</td>
</tr>
<tr>
<td>3</td>
<td>W250x101 (bottom)</td>
<td>W310x67 (bottom)</td>
</tr>
<tr>
<td></td>
<td>W200x59 (top)</td>
<td>W200x42 (top)</td>
</tr>
<tr>
<td>4</td>
<td>W200x59</td>
<td>W200x42</td>
</tr>
<tr>
<td>5</td>
<td>W200x59</td>
<td>W200x42</td>
</tr>
</tbody>
</table>

The BRB design information is presented in Table 4.2. The steel area and yield strength of the BRBs were chosen to exactly match with the effective area and strength of the AYB specimens with 4 or 6 steel core rods. The results of the cyclic test on the AYB specimen in UT10 presented in Chapter 3 were used for this purpose. The design strength ($P_r$) of the BRBs were calculated from $P_r = 0.9P_y$ according to ANSI/AISC 341-10 (AISC, 2010b). The yielding capacity ($P_y$), the maximum probable tensile strength ($T_{pr}$) and the maximum probable compressive strength ($C_{pr}$) of the BRBs at each storey were calculated based on the number of core rods in the corresponding AYB specimen times the value of $P_y$, $T_{pr}$, and $C_{pr}$ of a single core rod. In order to determine the value of $P_y$, $T_{pr}$, and $C_{pr}$ of a single core rod, the $P_y$, $T_{pr}$, and $C_{pr}$ of the AYB specimen were determined from the cyclic test results in UT10 and the resulting values were divided by six. The $P_y$ of the AYB specimen was obtained based on the 0.2% offset method applied on the hysteretic curve of the AYB specimen and the values of $T_{pr}$ and $C_{pr}$ of the AYB specimen were chosen as the maximum achieved tensile and compressive forces in the 3% SDR cycle. The values of $T_{pr}$ and $C_{pr}$ were used in the capacity design of the BRBF members. ANSI/AISC 341-10 (AISC, 2010b) requires calculation of $T_{pr}$ and $C_{pr}$ based on the maximum forces experienced in the cyclic qualification tests on the BRB specimens with a maximum axial displacement amplitude corresponding to twice the design SDR which is $2 \times 2\% = 4\%$ for the BRBF considered in this study. However, as discussed in Chapter 3, during the cyclic tests on the AYB, one of the core rods fractured before the 4% SDR cycle. Therefore, the maximum achieved tensile and compressive forces in the 3% SDR cycle were used to calculate $T_{pr}$ and $C_{pr}$. These values are expected to be slightly smaller than the corresponding values at the 4% SDR. However, since the hysteretic response of the AYB reached a plateau representing a saturated isotropic hardening condition at these response limits, it is expected that the differences are negligible. On the other hand, the response history analysis of the BRBF presented in section 4.6.2 indicates that the SDR response of the BRBF is within 3% under the MCE ground motions which confirms that the 3% SDR is a realistic maximum response level for the BRBF. The values of the factored required strength ($P_u$) and the axial force demand to capacity ratios ($P_u/P_r$) for each brace are also presented in Table 4.2. The values suggest...
a rather efficient design in the first four storeys and slightly overdesign in the last storey. Based on ANSI/AISC 341-10 (AISC, 2010b), the strain hardening adjustment factor (ω) is calculated from $\frac{T_{pr}}{P_y}$ and the compression strength adjustment factor (β) is calculated from $\frac{C_{pr}}{T_{pr}}$. The results are 1.17 for ω and 1.14 for β which are both within the acceptable range for BRBs.

### Table 4.2: Design information for the BRBs in the BRBF

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>Number of core rods</th>
<th>Total core area (mm$^2$)</th>
<th>$P_u$ (kN)</th>
<th>$P_r$ (kN)</th>
<th>$C_{pr}$ (kN)</th>
<th>$T_{pr}$ (kN)</th>
<th>$P_u/P_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>1243</td>
<td>467</td>
<td>533</td>
<td>793</td>
<td>695</td>
<td>0.88</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>1243</td>
<td>410</td>
<td>533</td>
<td>793</td>
<td>695</td>
<td>0.77</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>1243</td>
<td>342</td>
<td>533</td>
<td>793</td>
<td>695</td>
<td>0.64</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>828</td>
<td>277</td>
<td>356</td>
<td>529</td>
<td>463</td>
<td>0.78</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>828</td>
<td>161</td>
<td>356</td>
<td>529</td>
<td>463</td>
<td>0.45</td>
</tr>
</tbody>
</table>

The BRBF connections and BRB components including the BRB encasing and the non-yielding segments of the BRB core were fully designed to obtain their real dimensions that were required in BRB numerical models. The non-yielding segments of the BRB core were designed from cruciform sections (see Figure 4.1). The gusset plate connections were capacity designed based on a Whitmore effective section and Thornton length (Whitmore, 1952; Thornton, 1984). Figure 4.5 shows the beam-column-brace connection details used in the BRBF. As indicated in this figure, a stiffener plate was used in the middle of the gusset plate to facilitate the bolted connection between the BRB cruciform section and the gusset plate. The floor beam was continuous under the gusset plate and was designed to be shop welded to the column flange at the floor beam end intersecting with the braces. The other end of the floor beams were designed to be connected to the column flanges with shear tab angles.

![Figure 4.5: Details of the BRBF beam-column-brace connection](image)

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4.4 **NUMERICAL MODELLING OF THE BRBF**

A nonlinear numerical model of the BRBF was created in OpenSees which was used in the hybrid simulations and in NRHA. Since the building plan was symmetric, the torsional effects and the effects of the out-of-plane structural members could be neglected in the numerical model. Therefore, it was sufficient to model a single BRBF bay. Figure 4.6 shows an overview of the BRBF numerical model. As indicated in Figure 4.6 an elastic leaning column was used to consider the $P - \Delta$ effect of the gravity loads in the building. The leaning column was continuous over its entire length and was pinned at the base. The cross section properties of the leaning column were equivalent to the properties of the gravity columns. Tributary gravity loads were calculated based on the entire dead and live loads and were applied on the leaning columns and the floor beams at different floor levels. The diaphragm effect of the floor slabs was taken into account by constraining the horizontal movements of the nodes at each floor level, including the nodes of the leaning column, to a master node as shown in Figure 4.6. The seismic masses were calculated based on the entire dead loads and were lumped at the master nodes at each floor level. Lumped plasticity models with fibre sections were used to model the nonlinear flexural response of the beams and columns. The fibre sections enabled automatic consideration of the interaction of the axial and flexural forces in the plastic hinges. However, they were not able to account for the shear response or failure in the elements. In the BRBF model, the plastic hinges were assumed to be located at the ends of the beams and columns.

![Figure 4.6: Details of the BRBF numerical model](image)

As indicated in Figure 4.6, rigid offsets were considered in the model to consider the rigidity of the beam-column-brace connections. Giuffre-Menegotto-Pinto uniaxial stress-strain relationship (Steel02 uniaxial material in OpenSees) with kinematic and isotropic strain hardening and a post-yield stiffness ratio of 2% was used as the
nonlinear material for the plastic hinge fibre sections. Damping was defined by introducing 3% numerical viscous damping in the first two vibration modes of the BRBF.

4.4.1 BRB yielding core

The nonlinear hysteretic response of the BRB yielding core was modelled using two different material models. The first model was a bilinear model with sharp transitions between the elastic and inelastic zones and with post-yield stiffness and kinematic strain hardening. This model is widely used both in research and industry to model the hysteretic response of BRBs. The second model was Giuffre-Menegotto-Pinto material model which provides smooth transitions between the elastic and inelastic zones and is capable of modelling the combined effect of kinematic and isotropic strain hardening. This model is capable of simulating the gradual softening of the response that results from the Bauschinger effect as discussed previously. Moreover, in this model, the amount of isotropic hardening is defined as a function of nonlinear axial deformations and can be controlled separately for the tensile and compressive forces. In order to capture the added compressive strength in the BRBs due to the friction effect, more compression hardening than tension hardening can be used in this model. This approach is also capable of modelling the increase of friction with the amount of nonlinear deformations in BRBs. In order to verify and compare the response of the material models, a 2D model of the AYB specimen was created in OpenSees. Figure 4.7 shows the details of the model. As indicated in this figure, the AYB specimen was modelled with a force-based beam-column element (forceBeamColumn element in OpenSees) with fibre sections and distributed plasticity model. The length of the element was equal to the length of the AYB yielding cores (650 mm). Steel01 and Steel02 uniaxial stress-strain relationships were used as the nonlinear material models in OpenSees for the first and second model, respectively. A rectangular cross section with an area equal to the area of the AYB specimen (6 × 207.1 m²) was considered for the fibre sections. The fibre discretization consisted of 5 and 2 subdivisions through the width and thickness of the cross section, respectively. Since no flexural and shear actions were expected in the element, the distribution of stresses were expected to be uniform. Therefore, the rather coarse discretization employed for the cross sections was deemed to provide sufficient accuracy. As indicated in Figure 4.7, the beam-column element was fully restrained at both ends except for the horizontal (axial) movements of the right end which was controlled in a displacement-controlled manner. A Gauss-Lobatto (Bathe, 1996) numerical integration method with 2 integration points over the beam-column element was adopted. The loading protocol was similar to the loading protocol of the cyclic tests on the AYB specimen.

Figure 4.7: Details of the 2D OpenSees model for AYB
Both Steel01 and Steel02 material models were calibrated using the 2D model discussed above based on the AYB cyclic test results discussed in Chapter 3. The values of the calibration parameters for both models are presented in Table 4.3. In order to take into account the increase of axial forces due to isotropic hardening in the bilinear model (Steel01), the post-yield stiffness parameter was selected such that the maximum axial force predicted by the bilinear model matched with the AYB maximum tensile force in the last cycle. Figure 4.8 shows the cyclic hysteretic response of Steel01 and Steel02 models overlapped on the AYB cyclic response. As can be seen from this figure both models accurately predicted the elastic stiffness and the maximum tensile force of the AYB. It can be also observed that unlike Steel01, Steel02 material model was able to predict the difference between the compressive and tensile forces resulting from the friction effects in the AYB. This was achieved by defining larger compressive isotropic hardening than tensile isotropic hardening in this model as indicated in Table 4.3. However, it can be observed from Figure 4.8b that the transition curve in the AYB cyclic response was stiffer on the tension side compared to the compression side and Steel02 was unable to accurately predict the transition curvature simultaneously in the tension and compression sides of the hysteretic curve. Based on the results presented in Figure 4.8, Steel02 model provided a more accurate prediction of the cyclic response of the AYB specimen compared to Steel01.

![Cyclic hysteretic response of Steel01 and Steel02 calibrated models overlapped on the AYB cyclic response: (a) Steel01 model and (b) Steel02 model](image)

**Figure 4.8:** Cyclic hysteretic response of Steel01 and Steel02 calibrated models overlapped on the AYB cyclic response: (a) Steel01 model and (b) Steel02 model

**Table 4.3:** Calibration parameters for Steel01 and Steel02 material models in OpenSees

<table>
<thead>
<tr>
<th>Description</th>
<th>Elastic modulus</th>
<th>Yield strength</th>
<th>Post-yield stiffness ratio</th>
<th>Transition parameters</th>
<th>Compression isotropic hardening parameters</th>
<th>Tension isotropic hardening parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td>$E$ (GPa)</td>
<td>$F_y$ (MPa)</td>
<td>$b$</td>
<td>$R_0$</td>
<td>$C_R1$</td>
<td>$C_R2$</td>
</tr>
<tr>
<td>Steel01</td>
<td>200</td>
<td>477</td>
<td>0.019</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Steel02</td>
<td>200</td>
<td>477</td>
<td>0.008</td>
<td>20</td>
<td>0.925</td>
<td>0.18</td>
</tr>
</tbody>
</table>

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4.4.2 Connections

Two different approaches were adopted in this study to model the response of the beam-column and brace connections. In the first approach, the effect of the connections was either not considered or considered using simplified modelling tools. Excessive yielding and small post-yield stiffness of the BRBs significantly increase the contribution of the moment action of the frame and the flexural response of the connections in a BRBF under large earthquakes. Therefore, recognizing the effect of connection response in a realistic evaluation of the nonlinear response of braced frames especially under large earthquakes (ATC, 2017), another BRBF model was developed which contained more detailed connection models. Details of the simplified connection model and detailed connection model are presented in the following sections.

4.4.2.1 Simplified model

The beam-column connection, column baseplates, and brace end connections were assumed to be fully pinned in the simplified model and therefore, no flexural actions were considered to act at the beam ends and on the BRBs in this model.

The axial stiffness of the BRBs is governed by the axial stiffness of the BRB yielding core. However, the effect of the non-yielding segments of the core and the BRB connections should be also accounted for in the numerical model. In order to consider these effects in the simplified model, the axial stiffness of the BRB ($K_{BRB}$) was assumed to be 1.45 times larger than the axial stiffness of the BRB determined based on the cross section area and elastic modulus of the yielding core ($A_c$ and $E_c$) and the centre-to-centre length of the brace measured from its working points ($L_b$):

$$K_{BRB} = 1.45E_c \frac{A_c}{L_b}$$  

(4.1)

The value of 1.45 was proposed by Choi et al. (2008) based on a review of the past experiments on BRB components. The BRB was then modelled with three beam-column elements connected in series. Figure 4.9 shows the BRB simplified model. As indicated in this figure, the middle element in this model represented the BRB yielding core and the other two elements (elastic elements) represented the elastic parts (non-yielding segments of the BRB core, gusset plates, and beam/column rigid offsets) at each end of the yielding core. The length of the yielding core ($L_c$) and each of the elastic elements ($L_s$) were assumed to be half and quarter of the brace centre-to-centre length ($L_b$), respectively. The axial stiffness of the BRB ($K_{BRB}$) can be calculated using the following equation:

$$\frac{1}{K_{BRB}} = \frac{1}{K_c} + \frac{2}{K_s} = \frac{L_c}{E_cA_c} + \frac{2L_s}{E_sA_s}$$  

(4.2)

where $K_c$ and $K_s$ are the axial stiffness of the yielding core and each of the elastic elements, respectively. $E_s$ and $A_s$ in Equation (4.2) are the elastic modulus and effective cross section area of the elastic elements, respectively. The cross section area of each of the elastic members ($A_s$) were determined by substituting $K_{BRB}$ from Equation (4.1) in Equation (4.2). Based on this the $A_s/A_c$ ratio was calculated as 2.64 and the equivalent area of the elastic members...
(\(A_y\)) were determined for the BRBs in each storey using the cross section area of the yielding cores (\(A_c\)) presented in Table 4.2.

\[
\begin{array}{ccc}
L_0 & L_1, E_1, A_2 & L_2, E_2, A_3 \\
\text{Elastic element} & \text{Yielding core element} & \text{Elastic element}
\end{array}
\]

**Figure 4.9:** BRB simplified model

### 4.4.2.2 Detailed model

#### 4.4.2.2.1 Beam-column connections

Figure 4.10 shows the BRBF model with detailed connection models. As indicated in this figure, in the detailed model the beam-column connections were modelled with zero-length rotational springs with the Hysteretic material model available in OpenSees. The rotational stiffness and strength of the model were calibrated based on the approach proposed by Liu and Astaneh-Asl (2004) considering the effect of the floor concrete slabs. Figure 4.11 shows the moment-rotation envelope and cyclic response of the shear tab connection model. Low-cycle fatigue damage was not considered in the connection model. The rotation capacity of the connection was evaluated as 0.16 rad. The panel zone shear deformations were not considered in the BRBF model as they were expected to have a negligible impact on the performance prediction of the BRBFs. Since the floor beams were continuous under the brace gusset plates, recognizing the rigidity of the gusset plates, the beam-column-brace connection was assumed to be rigid in the detailed model.

**Figure 4.10:** BRBF numerical model with detailed connection model

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Figure 4.11: Envelope and cyclic moment-rotation response of the shear tab connection model for the BRBF

In order to consider the stiffening effect of the gusset plates on the beams and columns, rigid elements were used along the beam and column elements as indicated in Figure 4.10. The lengths of the rigid elements were determined based on the actual gusset plate design dimensions for each floor following the recommendations of Hsiao et al. (2012).

4.4.2.2 BRB elastic parts and connections

In the detailed model, each BRB was modelled with 10 elements as indicated in Figure 4.10: 4 elements representing the axial and flexural stiffness of the gusset plate-BRB connection zones, 4 elements representing the confined and unconfined segments of the BRB cruciform sections, one element representing the BRB yielding core, and one element representing the BRB encasing. To model the contribution of the flexural stiffness of the BRB encasing and the gusset plate-BRB connection zones on the lateral response of the BRBF, their flexural stiffness was explicitly considered in the detailed model. For this purpose, realistic estimates of the axial and flexural stiffness of the assembly were obtained from component-level 3D continuum FE simulations of the gusset plate-BRB connection assembly with realistic design dimensions. The assembly was then modelled with a single element at each end of the brace in OpenSees with a representative axial stiffness. This element was assigned a high flexural rigidity. Rotational springs with calibrated flexural stiffness were used at the end of the flexurally rigid element to represent the flexural response of the gusset plate-BRB end connection assembly.

As indicated in Figure 4.10, the unconstrained and constrained parts of the BRB cruciform section and the BRB yielding core were modelled with a series of five force-based beam-column elements (forceBeamColumn element in OpenSees) with fibre sections and distributed plasticity model in OpenSees. The material properties of these elements were assigned based on the calibration parameters obtained from AYB cyclic tests as discussed in section 4.4.1. The BRB encasing was modelled with a single element extending between the ends of the unconfined cruciform sections as illustrated in Figure 4.10. This element was assigned a very small axial stiffness representing the decoupling of the BRB encasing and BRB core in the axial direction. The flexural stiffness of the BRB encasing
element was assigned based on the mechanical properties and composite flexural action of the encasing steel tubular section and the concrete infill (see Figure 4.1).

4.4.2.2.3 Low-cycle fatigue model

The unconfined cruciform sections of the BRB can experience flexural yielding and may develop large stress concentration and low-cycle fatigue fracture that can limit the ductility capacity of the BRB. Low-cycle fatigue in structural engineering is classically described by a linear log-log relationship between the number of constant amplitude cycles to failure of the material and the strain amplitude experienced in each cycle. This relationship is called the Coffin-Manson relationship (Fisher et al., 1997). This relationship is characterized by two empirical parameters, $\epsilon_0$ and $m$. Uriz and Mahin (2008) developed an OpenSees fatigue model and they calibrated and verified their proposed model with the experimental data from cyclic testing of the conventional and buckling-restrained braces. Their model was based on a modified rainflow cycle counting method and Miner’s rule for damage accumulation in the cross section fibres. In the BRBF with the detailed model, the unconfined cruciform sections were equipped with the low-cycle fatigue model developed by Uriz and Mahin (2008) to evaluate the amount of damage in this region. Since no fatigue parameters were found in the literature for cruciform sections, the Coffin-Manson parameters for the low-cycle fatigue model used for the unconfined cruciform sections were conservatively chosen as the proposed values by Uriz and Mahin (2008) for cold form steel rectangular HSS sections. These parameters were $\epsilon_0 = 0.091$ and $m = -0.458$.

4.4.2.3 Continuum FE model

System-level tests on BRBFs reported in Section 4.2 show that the gusset plate-BRB connection assembly may experience excessive rotation demands and local buckling under large BRB compressive loads that can cause global buckling of the BRB. This limit state occurs especially following large lateral deformations of BRBFs under severe earthquakes and potentially limits the axial load and ductility capacity of BRBs. As a result of the concentration of rotation especially in the BRB unconfined segments, plastic hinges can be formed in these regions and the BRB can deform into either an S-shape or U-shape depending on the direction of the BRB end rotations (Tsai and Hsiao, 2008). Figure 4.12 shows the concentration of rotations at BRB ends and possible deformed shapes of the BRB.

![Figure 4.12: Formation of plastic hinges at BRB ends and global buckling of the BRB](image)

Although the gusset plates and BRB unconfined segments were capacity designed and reinforced to avoid concentration of rotation at BRB ends and global buckling of the BRB, 3D continuum finite element (FE) models...
of the BRBF beam-column-brace sub-assemblage were developed and nonlinear buckling simulations were performed to determine the axial force capacity of the system. Such analyses were necessary since the concentration of rotation at BRB ends, local buckling of the gusset plate-BRB connection, and global buckling of the BRB were not explicitly considered in the BRBF numerical models and hybrid simulations. For this purpose, the gusset plate-BRB connection assembly, half of the BRB, and half of the adjacent beams and columns were modelled. Figure 4.13 shows the 3D continuum FE model for the BRBF beam-column-brace sub-assemblage. In order to consider the impact of the adjacent storeys, half of the column and the full gusset plate-BRB connection assembly in the top storey were also modelled. As indicated in Figure 4.13, the mid-points of the beam, column, and the BRBs were connected by rigid truss elements. This modelling approach was employed to reduce the size of the model assuming formation of inflection points at the mid-length of the beams and columns during lateral deformation of the BRBF. Similar modelling techniques were successfully employed by Kaneko et al. (2008). Horizontal loads were applied at the mid-point of the floor beam and their magnitudes were simultaneously increased similar to a pushover analysis. Various loading directions and ratios were considered for the loads applied on the two adjacent storeys.

Four-node shell elements were used to model the beam, column, and the gusset plate-BRB connection assembly. The BRB core was modelled with a truss element and the BRB encasing was modelled with a general beam element with negligible axial stiffness and a flexural stiffness representing the actual cross sectional properties of the concrete filled HSS in the BRB. The BRB core and encasing were modelled with elastic steel material while nonlinear models were considered for the rest of the model. This was done to ensure that yielding of the BRB core or encasing does not limit the force capacity of the gusset plate-BRB connection assembly. Four different models were created representing all possible combinations of the beams, columns, and BRB connections in the BRBF and the 3D continuum FE model geometries were created with the exact design dimensions. Separate analyses were performed on each model with the boundary conditions promoting either S-shape or U-shape buckling.

In order to perform nonlinear buckling analyses, an initial imperfection corresponding to the first buckling shape of the system was introduced into the model geometry. The latter was determined from linear buckling analysis of the system by solving the eigenvalue problem for the model. The imperfections were normalized such that the maximum imperfection at the gusset plate-BRB connection point was a portion of the gusset plate Thornton length. A sensitivity analysis was performed to determine the impact of the imperfection magnitude on the axial compressive capacity of the system and the results revealed that the amount of imperfection did not impact the overall response considerably. The low-cycle fatigue damage was not considered in the 3D continuum FE model.
Figure 4.13: The 3D continuum FE model for the BRBF beam-column-brace sub-assemblage: (a) deformed shape and displacement field (mm) after S-shape buckling and (b) deformed shape and Von Mises stress field (MPa) after S-shape buckling (FE response fields provided by Pedram Mortazavi, PhD Candidate at the Department of Civil and Mineral Engineering, University of Toronto)

Figure 4.14: Axial force-deformation response of the BRBF beam-column-brace sub-assemblage

The deformed shape, FE displacement field, and FE Von Mises stress field after an S-shape buckling of the BRBF beam-column-brace sub-assemblage are shown in Figure 4.13. The concentration of displacement and stress can be clearly seen at the unconfined cruciform section in Figures 4.13a and 4.13b. It can be also observed from these figures that the buckling occurred in the out-of-plane direction and caused some torsions in the floor beam. Figure 4.14 shows the axial force-deformation of the BRB for both a U-shape and S-shape buckling. The maximum probable compressive strength ($C_{pr}$) of the BRBs is also shown in this figure. The results indicate a linear elastic response prior to the compressive buckling of the sub-assemblage and a sharp drop of compressive capacity.
immediately after buckling followed by a more gradual strength degradation. It can be also observed from Figure 4.14 that the S-shape buckling compressive capacity of the system was 1270 kN that was 8% larger than the U-shape buckling compressive capacity. The compressive capacity for the U-shape and S-shape buckling were, respectively, 50% and 60% larger than $C_{pr}$ which confirms that the maximum axial force in the BRB yielding core could not cause global buckling of the BRB and failure of the gusset plates and thus the capacity design of the BRB non-yielding elements and the gusset plate was effective. Therefore, the axial load capacity of the BRBs were not limited by these undesired failure modes and it was not necessary to consider them in the analysis of the BRBF.

### 4.5 HYBRID SIMULATIONS

Three hybrid simulations were performed on the BRBF designed in section 4.3. The first hybrid simulation was performed with one physical specimen representing the yielding core of the BRB in the BRBF first storey. This simulation is hereafter referred to as 1E-BRBF-HS or the 1-element BRBF test. The second and third hybrid simulations were performed with three physical specimens representing the yielding core of the BRBs in the first three stories of the BRBF. These simulations are hereafter referred to as 3E-BRBF-HS1 and 3E-BRBF-HS2 or the 3-element BRBF tests. The main purpose of the hybrid simulations was to verify, for the first time, the performance of UT10 when testing large-capacity yielding specimens under earthquake loads. The results of the tests were also used to assess the accuracy of the adopted numerical models for the BRB yielding cores in the BRBF numerical models.

Based on the type of connection and BRB material model as discussed in Section 4.4, four different BRBF models were considered in the numerical analysis:

- **BRBF-S01**: BRBF model with simplified connection model and Steel01 BRB material model
- **BRBF-D01**: BRBF model with detailed connection model and Steel01 BRB material model
- **BRBF-S02**: BRBF model with simplified connection model and Steel02 BRB material model
- **BRBF-D02**: BRBF model with detailed connection model and Steel02 BRB material model

Different BRBF numerical models were used as the numerical substructure in the hybrid simulations. The information on the type of BRBF model used in each of the hybrid simulations is presented in Table 4.4.

<table>
<thead>
<tr>
<th>Hybrid simulation</th>
<th>BRBF model</th>
<th>Displacement scale factor</th>
<th>Ground motion No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1E-BRBF-HS</td>
<td>BRBF-S02</td>
<td>6.09</td>
<td>11</td>
</tr>
<tr>
<td>3E-BRBF-HS1</td>
<td>BRBF-S02</td>
<td>6.28 5.66 5.66</td>
<td>11</td>
</tr>
<tr>
<td>3E-BRBF-HS2</td>
<td>BRBF-D02</td>
<td>6.22 5.66 5.66</td>
<td>4</td>
</tr>
</tbody>
</table>

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4.5.1 Specimens

Three AYB specimens were fabricated and used as the physical specimens in all of the three hybrid simulations. After each hybrid simulation, the AYB core rods were removed and replaced with new core rods. The AYB specimens with the new core rods were then used in the next hybrid simulation. Similar to the cyclic tests, the axial deformations of each of the AYB specimens were measured using two linear potentiometers as described in Chapter 3.

The AYB specimen was designed to simulate the response of a portion of the yielding core of a full-scale BRB and did not include the response of the BRB end zones and connections. These parts were separately modelled in the BRBF numerical substructure. For the hybrid simulations, the total effective cross section area of the core rods in the AYB was similar to the cross section area of the yielding core of a full-scale BRB. Therefore, assuming small frictions in the system, axial forces in AYB were well representative of the axial forces in the yielding core of a full-scale BRB. Due to the size restrictions of UT10, the maximum yielding length of the AYB core rods was limited to a maximum of 700 mm. Therefore, the axial deformations of the AYB core rods were multiplied by a displacement scale factor, equal to the ratio of the length of the BRB yielding core and the length of the AYB core rods, to be representative of the axial deformations of the yielding core of the BRB. This approach was taken based on the assumption that the state of the axial stress and strain is almost uniform along the BRB yielding core. The displacement scale factors used in the BRBF hybrid simulations are presented in Table 4.4.

4.5.2 SubStructure element

A SubStructure element was used in the hybrid simulations to establish the communications between the numerical model and the AYB specimens. The SubStructure control nodes were located at the beginning and end of the yielding core of the BRBs. For 1E-BRBF-HS, the SubStructure element represented the yielding core of the first storey BRB in the BRBF model and in 3E-BRBF-HS1 and 3E-BRBF-HS2 the SubStructure element represented the yielding core of the three BRBs in the first three stories of the BRBF model. Figure 4.15 shows the BRBF model, SubStructure element, and the control nodes for 3E-BRBF-HS2. The initial stiffness of the SubStructure was prescribed to be 10% larger than the stiffness that was calculated based on the cross sectional and material properties of the BRB yielding core. Since the actual stiffness of a physical substructure is not known a priori, using a larger initial stiffness is regularly practiced for hybrid simulations to avoid convergence issues in the integration module during hybrid simulations.
4.5.3 Ground motions

Sixteen ground motions were selected and scaled to match the uniform hazard design response spectrum of Los Angeles following the ASCE 7-10 (ASCE, 2010) provisions and NEHRP-2011 (NEHRP, 2011) recommendations for selection and scaling of the ground motions for low and mid-rise buildings. All of the ground motions were used for seismic performance assessment of the BRBF as described in section 4.6. However, only two of the ground motions were used in the hybrid simulations. The scaling of the ground motions was done such that the average response spectrum was larger than the design response spectrum in the 0.2T-2T period range.

Figure 4.16 shows the design response spectrum for Los Angeles and the average response spectrum of the selected scaled ground motions. To show the scatter in the selection, the response spectrum of each of the selected scaled ground motions is also shown in grey colour in Figure 4.16. The ground motions were obtained from the Pacific Earthquake Engineering Research Centre (PEER) ground motion database (PEER, 2015). The selection of the ground motions was based on the soil type and the disaggregation data for MCE seismic hazard level (2475 years return period) for Los Angeles. The disaggregation data (mean for all hazard sources) obtained from the United States Geological Survey (USGS) website associated with 1.0 s spectral acceleration for soil type C was 7.1 for the magnitude of the earthquake (M), 10.5 km for the fault distance (R), and 1.25 for the epsilon (ε). Epsilon is a measure of scatter in the intensity of the ground motions from the mean value predicted by an attenuation equation. Based on the NEHRP-2011 (NEHRP, 2011), since the disaggregation magnitude was more than 7 and the disaggregation fault distance was less than 15 km, the location of the building was categorized as near-fault and thus ground motions with velocity pulse motions needed to be considered in the selection. A group of such pulse type ground motions are identified and listed in table C-2 of NEHRP-2011. NEHRP-2011 also provides an equation to determine the proportion of the number of the pulse type ground motions to the total number of ground motions selected:
Propotion of pulse type motion = \frac{\exp(0.905 - 0.188R + 1.337\epsilon)}{(1 + \exp(0.905 - 0.188R + 1.337\epsilon))} \quad (4.3)

Using $R = 10.5 \ km$ and $\epsilon = 1.25$, the above proportion becomes 0.65 which means that at least 10 ground motions from the 16 ground motions should be pulse type. For this study, 11 pulse type ground motions were selected. Many of these ground motions had pulse periods close to the principal period of the building in the south-north direction. Detailed information on the selected ground motions is presented in Tables 4.5 and 4.6. The ground motions in Table 4.6 with * in front of their numbers are the pulse type motions with forward directivity (FD). As can be seen from Table 4.6, three of the pulse type motions are FD-pulse type. $V_{s30}$ in Tables 4.5 and 4.6 is the shear-wave velocity averaged in the top 30 meters of the site soil and as can be seen from the values presented in the tables they are all very close or within the 360 m/s-760 m/s range that is associated to soil type C.

![Figure 4.16: The 5% damped uniform hazard design response spectrum and the response spectrum of the selected scaled ground motions for the BRBF](image)

### Table 4.5: No-pulse type ground motions selected for the BRBF analysis

<table>
<thead>
<tr>
<th>Record No.</th>
<th>Scale factor</th>
<th>Earthquake</th>
<th>Station</th>
<th>Year</th>
<th>M</th>
<th>R (km)</th>
<th>$V_{s30}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.30</td>
<td>&quot;Iwate_Japan&quot;</td>
<td>&quot;AKT023&quot;</td>
<td>2008</td>
<td>6.9</td>
<td>17.0</td>
<td>556</td>
</tr>
<tr>
<td>2</td>
<td>1.92</td>
<td>&quot;Tottori_Japan&quot;</td>
<td>&quot;TTR007&quot;</td>
<td>2000</td>
<td>6.6</td>
<td>11.3</td>
<td>470</td>
</tr>
<tr>
<td>3</td>
<td>1.44</td>
<td>&quot;Manjil_Iran&quot;</td>
<td>&quot;Abbar&quot;</td>
<td>1990</td>
<td>7.4</td>
<td>12.6</td>
<td>724</td>
</tr>
<tr>
<td>4</td>
<td>2.56</td>
<td>&quot;Hector_Mine&quot;</td>
<td>&quot;Hector&quot;</td>
<td>1999</td>
<td>7.1</td>
<td>11.7</td>
<td>726</td>
</tr>
<tr>
<td>5</td>
<td>0.99</td>
<td>&quot;Chuetsu-oki_Japan&quot;</td>
<td>&quot;Joetsu Kakizakiku Kakizaki&quot;</td>
<td>2007</td>
<td>6.8</td>
<td>11.9</td>
<td>383</td>
</tr>
</tbody>
</table>

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Table 4.6: Pulse type ground motions selected for the BRBF analysis

<table>
<thead>
<tr>
<th>Record No.</th>
<th>Scale factor</th>
<th>Earthquake</th>
<th>Station</th>
<th>Year</th>
<th>M (km)</th>
<th>V₃₀ (m/s)</th>
<th>Pulse period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6*</td>
<td>1.72</td>
<td>&quot;Superstition Hills-02&quot;</td>
<td>&quot;Parachute Test Site&quot;</td>
<td>1987</td>
<td>6.5</td>
<td>1.0</td>
<td>349</td>
</tr>
<tr>
<td>7</td>
<td>1.80</td>
<td>&quot;Northridge-01&quot;</td>
<td>&quot;LA - Sepulveda VA Hospital&quot;</td>
<td>1994</td>
<td>6.7</td>
<td>8.4</td>
<td>380</td>
</tr>
<tr>
<td>8</td>
<td>0.78</td>
<td>&quot;Northridge-01&quot;</td>
<td>&quot;Rinaldi Receiving Sta&quot;</td>
<td>1994</td>
<td>6.7</td>
<td>6.5</td>
<td>282</td>
</tr>
<tr>
<td>9</td>
<td>1.34</td>
<td>&quot;Kobe_Japan&quot;</td>
<td>&quot;Takarazuka&quot;</td>
<td>1995</td>
<td>6.9</td>
<td>0.3</td>
<td>312</td>
</tr>
<tr>
<td>10</td>
<td>1.41</td>
<td>&quot;Duze_Turkey&quot;</td>
<td>&quot;Bolu&quot;</td>
<td>1999</td>
<td>7.1</td>
<td>12.0</td>
<td>294</td>
</tr>
<tr>
<td>11*</td>
<td>3.97</td>
<td>&quot;Chi-Chi_Taiwan-03&quot;</td>
<td>&quot;TCU076&quot;</td>
<td>1999</td>
<td>6.2</td>
<td>14.7</td>
<td>615</td>
</tr>
<tr>
<td>12*</td>
<td>2.00</td>
<td>&quot;Loma Prieta&quot;</td>
<td>&quot;Saratoga - Aloha Ave&quot;</td>
<td>1989</td>
<td>6.9</td>
<td>8.5</td>
<td>381</td>
</tr>
<tr>
<td>13</td>
<td>2.75</td>
<td>&quot;Nahanni_Canada&quot;</td>
<td>&quot;Site 2&quot;</td>
<td>1985</td>
<td>6.8</td>
<td>4.9</td>
<td>605</td>
</tr>
<tr>
<td>14</td>
<td>2.40</td>
<td>&quot;Morgan Hill&quot;</td>
<td>&quot;Halls Valley&quot;</td>
<td>1984</td>
<td>6.2</td>
<td>3.5</td>
<td>282</td>
</tr>
<tr>
<td>15</td>
<td>1.31</td>
<td>&quot;Erzican_Turkey&quot;</td>
<td>&quot;Erzincan&quot;</td>
<td>1992</td>
<td>6.7</td>
<td>4.4</td>
<td>352</td>
</tr>
<tr>
<td>16</td>
<td>1.92</td>
<td>&quot;Cape Mendocino&quot;</td>
<td>&quot;Petrolia&quot;</td>
<td>1992</td>
<td>7.0</td>
<td>8.2</td>
<td>422</td>
</tr>
</tbody>
</table>

The ground motions with * in front of their numbers are the ground motions with forward directivity (FD).

Figure 4.17: The response spectrum of the ground motions selected for the BRB hybrid simulations

The BRBF hybrid simulations are performed with record No. 4 and 11 in Tables 4.5 and 4.6 which are, respectively, representative of the no-pulse type and pulse type ground motions. These records are specifically chosen for the hybrid simulations to investigate the effect of the type of ground motion on the BRBF response predictions with the hybrid simulations. Figure 4.17 shows the response spectrum of the ground motions selected for the hybrid simulations. The ground motion numbers used for each of the BRBF hybrid simulations are also presented in Table 4.4.

4.5.4 Numerical integration scheme

A non-iterative Alpha-OS integration scheme (Combescure and Pegon, 1997) was used with a linear algorithm in the OpenSees model to calculate the nonlinear response of the BRBF during hybrid simulations. Alpha-OS is a time integration scheme that is commonly used for hybrid simulations involving nonlinear response of structures. This

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scheme uses the Alpha-modified Newmark scheme to numerically damp the undesired spurious oscillations resulting from control errors. It also uses the operator splitting (OS) method (Nakashima et al., 1990) which is implicit and explicit for the linear and nonlinear part of the response of the structures, respectively. The Alpha-OS scheme is a non-iterative method and thus can be used with a linear algorithm in OpenSees. The linear algorithm mentioned here is the terminology used in OpenSees, and it does not mean that the algorithm can be used only for linear systems. Rather, it means that the system of equation is solved in a single iteration without further iterations in each load increment (i.e. time step). To use the Alpha-OS time integration scheme in OpenSees, the linear algorithm needs to be selected in OpenSees. The method, however, can be used for both linear and nonlinear structural response (Huang and Kwon, 2018).

The time step used for the BRBF simulation was 0.01 s. Preliminary fully numerical simulations were performed on the BRBF with the selected ground motions and various time step sizes. The results indicated that a time step of 0.01 s provides sufficient accuracy for the solution. Furthermore, the time step size for the hybrid simulations was chosen based on the timing limitations for running a single simulation in the structural laboratory.

4.5.5 Hybrid simulation results

Figure 4.18 shows the UT10 testing platform with AYB specimens after a 3-element BRBF test. ACTIA with four cameras were used to capture photos from UT10 and the specimens during the hybrid simulations.

The tolerance limit for the error compensation algorithm was set to 0.04 mm at the beginning of the tests. This value was chosen to be consistent with the accuracy of the external measurement system. Based on the preliminary fully numerical simulations on the BRBF this tolerance limit was around 0.3% of the maximum axial deformation of the BRBs. The tolerance limit was further relaxed up to 0.06 mm during the tests as the specimens entered the nonlinear response range and following the acceptable performance of the UT10 during the tests. The displacement errors for each of the AYB specimens during 3E-BRBFS01 and 3E-BRBFS02 models are compared with 1E-BRBFS01 and 3E-BRBFS02 models are compared with 3E-BRBFS01 results and fully numerical results of BRBF-D01 and BRBF-D02 models are compared with 3E-BRBFS02 results.

The hysteretic response of the AYBs obtained from the 3-element BRBF tests indicated that there were variations between the yield strength and elastic modulus of the core rod steel material used in these hybrid simulations and the corresponding values obtained from the AYB cyclic tests that were initially used for calibration of the material models. This was specifically observed for the core rods supplied from different batches. Since the main intent of comparison of the hybrid simulation results with the fully numerical results was to identify the impact...
of the inherent inaccuracies in the adopted BRB models, the uncertainties in the fully numerical response associated with the variation of yield strength and elastic modulus of the core rod steel material had to be eliminated. Therefore, the calibration of the BRB material models was updated based on the material properties of the core rods used in each of the 3-element BRBF tests. Updated calibration parameters were then used in the fully numerical BRBF models for the BRBs corresponding to the physical specimens in floors 1-3. Original calibration parameters were used for the BRBs in floors 4-5.

Figure 4.18: UT10 testing platform with three AYB specimens after a 3-element BRBF test: (a) south view and (b) north view

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In discussing the simulation results in this section and also in section 4.6, the location of the columns and braces are sometimes referred to with characters including a number 1 to 5 indicating the storey number that the brace/column belongs to and a letter N or S which respectively refers to the north or south directions (see Figure 4.6 for north and south directions). For instance column 2S refers to the column in the third storey on the south side of the bay in the BRBF model and BRB2 refers to the BRB in the second storey of the BRBF.

4.5.5.1 Response of the braces

Figure 4.19 shows the axial force and deformation response of BRB1 yielding core for 1E-BRBF-HS and 3E-BRBF-HS1. Figure 4.19a shows the hysteretic curves while Figure 4.19b shows the axial force and deformation response history. A closer view of the response in the first 3 seconds of the ground motion is also shown in Figure 4.19b. The BRBF-S02 fully numerical results are also shown in these figures. The results show that the response of the AYB was captured well by UT10 during the ground motion. It can be seen from Figure 4.19 that the response of the AYB specimen was similar during 1E-BRBF-HS and 3E-BRBF-HS1 tests. The results also indicated that the yield strength and elastic modulus of the core rod steel material used in the AYBs for 3E-BRBF-HS1 were, respectively, 5% and 10% larger than the original calibration values. As previously discussed, this was due to the variation of the steel material properties between the core rods.

It can be seen from Figure 4.19b that the fully numerical and experimental response histories were almost identical in the elastic response range at the beginning of the ground motion. This observation confirms the performance of UT10 and the sufficient accuracy of the error compensation algorithm for hybrid simulations. On the other hand, it can be observed from Figure 4.19 that the fully numerical predictions start to deviate from the experiments after the first tensile yielding occurs. A major difference is observed in the response during the first yielding of the brace in compression where the fully numerical model is unable to properly capture the softening of the response. In fact, the fully numerical model predicts a stiff response during the transition between the elastic and inelastic range while the real experimental results show a softer transition. This is despite the fact that the BRB numerical model, including its transition between the elastic and inelastic range, was carefully calibrated and well matched with the cyclic experimental results as discussed in Section 4.4.1. The stiff transition observed in the fully numerical model is mainly due to the slight unloading that has occurred just before the transition starts. This behaviour stems from a typical error that is inherent in the Menegotto-Pinto formulation used in Steel02 material model which causes artificial and non-behavioural hardening and stress overshooting upon slight unloading during cyclic stress-strain behaviour. This error has been also noticed and studied by a number of other researchers in the literature (Filippou et al., 1983; Bosco et al., 2016, 2017; Zsarnóczay and Vigh, 2017; Kolozvari et al., 2015, 2018). As can be seen from Figure 4.19a, this issue dominated the rest of the hysteretic response and caused a constantly maintained deviation during the rest of the response.
The AYB core rods for 3E-BRBF-HS2 were supplied from a new batch. Therefore, to determine the material properties of the new core rods, cyclic tests were performed on the new core rods using a single-component yielding specimen. The results revealed that the yield strength and elastic modulus of the new rods were, respectively, 20% and 10% larger than the original calibration values. The large variation of the yield strength was due to the fact that the core rods were fabricated following the requirements established for fastener steel material that allow large variations in the yield strength of the material. The material model calibration parameters used in the BRBF-D01 and BRBF-D02 were updated based on the new cyclic test results.
Figure 4.20: Axial force-deformation hysteretic response of BRB yielding cores during 3E-BRBF-HS1 compared to the fully numerical results with updated material model calibration factors: (a) BRB1, (b) BRB2, and (c) BRB3

Figures 4.20-4.23 show the axial force and deformation response of BRB yielding cores for 3E-BRBF-H1 and 3E-BRBF-HS2. The fully numerical results are also shown in these figures. As previously discussed, updated material model calibration parameters were used in the fully numerical models. A hydraulic problem caused temporary oil pressure fluctuation during 3E-BRBF-HS2. As a result, the AYB specimen representing BRB3 yielding core was slightly affected. This issue occurred at time 8.83 s of the ground motion. Figure 4.23 shows the axial force and displacement response history of this specimen. A closer view of the responses is also indicated in the sub-plots. As can be seen from these figures, the oil pressure fluctuations caused a 500 kN force drop in the AYB which was quickly recovered in the next time steps. Further investigations after the test revealed that this temporary force drop in BRB3 caused a 0.1 g amplitude spike in the third and fourth floor acceleration response which was very minor. Other global response parameters of the BRBF were not affected by this issue. As can be observed from Figure 4.22a the response of the BRB1 specimen shows a jagged response on the tension side. The
force drops in the jagged response were around 15 kN. It can be observed from Figures 4.20 and 4.22 that similar jagged response but with smaller amplitudes was also observed for other AYB specimens. Further investigations after the BRBF tests revealed that the jagged response was caused by excessive stress relaxation and displacement overshooting of the AYB specimens during the 3-element tests. However, this phenomenon was very local, and it did not have any effect on the response quantitates of the BRBF. The stress relaxation and displacement overshooting are discussed in detail in Chapter 2 Section 2.2.5.3.3. As mentioned in this section, further improvements were implemented in NICON-10 after the BRBF tests to avoid excessive stress relaxation and displacement overshooting of the specimens during future multi-element tests. The new implementations successfully eliminated these phenomena for the SCBF hybrid simulations (see Chapter 5).

**Figure 4.21:** Axial force and deformation response history of BRB1 yielding core during 3E-BRBF-HS1 compared to the fully numerical results with updated material model calibration factors
Figure 4.22: Axial force-deformation hysteretic response of BRB yielding cores during 3E-BRBF-HS2 compared to the fully numerical results with updated material model calibration factors: (a) BRB1, (b) BRB2, and (c) BRB3

Comparison of the results in Figures 4.20-4.23 shows that the hysteretic responses of all three AYBs were well captured simultaneously with UT10. The difference between the pulse and no-pulse type ground motions can be clearly seen from the hysteretic response of the AYBs. As can be seen from Figure 4.20 the AYBs under the pulse type ground motion underwent a single large nonlinear cycle resulting permanent deformation of AYBs in one direction while Figure 4.22 shows that AYBs underwent multiple nonlinear cycles throughout the no-pulse type ground motion.

Comparison of the fully numerical predictions with the hybrid simulation results indicates that both numerical models were able to properly predict the response of the AYBs under both pulse and no-pulse type ground motions. The responses perfectly matched in the first 3 seconds of the ground motions where the AYBs responded in the elastic range. The deviations started as the braces started to yield. The differences can be attributed to the stiffer transitions provided by both material models compared to the real experimental results and they seem to be affecting...
the axial displacement responses more than the axial force responses. In fact, the axial forces were well predicted by both numerical models during the entire duration of the ground motions. The results indicate that in general the numerical model with Steel02 material model provided better predictions for the hysteretic response shape compared to Steel01 for both ground motions.

Figure 4.23: Axial force and deformation response history of BRB3 during 3E-BRBF-HS2 compared to the fully numerical results with updated material model calibration factors

The ratios of the maximum tensile forces ($T_{max}$) and the maximum compressive forces ($C_{max}$) of the BRBs predicted by the fully numerical models with respect to the values measured during the hybrid simulations are presented in Table 4.7. The values show that almost in all cases the numerical predictions for the axial forces were smaller than the values measured during the hybrid simulations. However, as the results in Table 4.7 show, the numerical predictions were very close to the experimental results with errors in the range of 1-9% with an average error of 4%.

The maximum tension and compression displacement ductilities of the BRBs (respectively $\mu_{T,\text{max}}$ and $\mu_{C,\text{max}}$) during the hybrid simulations and the ratio of the fully numerical predictions with respect to the hybrid simulation results are presented in Table 4.7. The tension and compression displacement ductilities of the BRBs were calculated, respectively, by dividing the maximum axial tensile and compressive displacements of the full brace (including the elastic parts and gusset plates) by the axial yield displacement of the full brace. The results indicate that the maximum tension and compression displacement ductilities of the BRBs were respectively 5.3 and 4.9 for 3E-BRBF-HS1 which increased to 6.3 and 8.9 in 3E-BRBF-HS2. It can be observed that the ductilities were larger...
in BRB1 and under no-pulse ground motion (3E-BRBF-HS2). Comparison of the hybrid simulation results with the fully numerical predictions indicate that the compression ductilities were generally underestimated by the numerical models while the tension ductilities were generally overestimated by the fully numerical models. The results presented in Table 4.7 show that the numerical prediction relative errors were in the range of 1-46% with an average of 7% which is almost twice the errors in the axial force predictions. The average of fully numerical displacement ductility errors for Steel02 and Steel01 material models were respectively 2% an 12% which confirms the higher accuracy provided by Steel02 compared to Steel01.

**Table 4.7:** BRBF hybrid simulation response quantities for the physical brace specimens compared to their numerical predictions

<table>
<thead>
<tr>
<th>Response quantity</th>
<th>Material Model</th>
<th>1E-BRBF-HS</th>
<th>3E-BRBF-HS1</th>
<th>3E-BRBF-HS2</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{C_{\text{max,n}}}{C_{\text{max}}^*} )</td>
<td>Steel02</td>
<td>0.98</td>
<td>1.02</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Steel01</td>
<td>-</td>
<td>0.95</td>
<td>0.93</td>
</tr>
<tr>
<td>( \frac{T_{\text{max,n}}}{T_{\text{max}}} )</td>
<td>Steel02</td>
<td>0.94</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>Steel01</td>
<td>-</td>
<td>0.99</td>
<td>0.92</td>
</tr>
<tr>
<td>( \mu_{\text{c,max}} )</td>
<td>-</td>
<td>2.97</td>
<td>2.9</td>
<td>4.9</td>
</tr>
<tr>
<td>( \mu_{T_{\text{max}}} )</td>
<td>-</td>
<td>5.23</td>
<td>5.3</td>
<td>2.2</td>
</tr>
<tr>
<td>( \frac{\mu_{\text{c,max}}}{\mu_{\text{T,max}}} )</td>
<td>Steel02</td>
<td>0.83</td>
<td>0.74</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>Steel01</td>
<td>-</td>
<td>0.60</td>
<td>1.22</td>
</tr>
<tr>
<td>( \frac{\mu_{T_{\text{max}}}}{\mu_{T_{\text{max}}}} )</td>
<td>Steel02</td>
<td>1.23</td>
<td>1.31</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>Steel01</td>
<td>-</td>
<td>1.24</td>
<td>0.66</td>
</tr>
<tr>
<td>( E_d (kJ) )</td>
<td>-</td>
<td>82</td>
<td>84</td>
<td>65</td>
</tr>
<tr>
<td>( \frac{E_{d,n}}{E_d} )</td>
<td>Steel02</td>
<td>1.04</td>
<td>1.10</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>Steel01</td>
<td>-</td>
<td>1.07</td>
<td>0.98</td>
</tr>
</tbody>
</table>

* The letter \( n \) in the subscripts refers to the numerical predictions with either Steel02 or Steel01 material models

### 4.5.5.2 Energy response

Investigation of the energy absorption and dissipation of the SFRSs is a powerful tool for their performance assessment as it is directly related to the global response and level of structural damage in the SFRSs. Large amounts of energy dissipation in the main structural components in an SFRS, like the braces in braced frames, are indications of significant damage and hence increased risks of failure. Moreover, the comparison of the energy components helps with the assessment of the accuracy of the performance and response prediction tools. Experimental and numerical energy response assessments of various SFRSs were performed by many researchers including Tremblay et al. (2003) for conventional braces, Lestuzzi and Bachmann (2007) for reinforced concrete shear walls, and Mojiri et al. (2014) for reinforced concrete block masonry shear walls. Detailed discussions on the seismic energy response of the SFRSs and the approaches for the numerical computation of the various energy components can be found in
Uang and Bertero (1990) and Christopoulos and Filiatrault (2006). The cumulative relative input energy at time \( t \) 
\( (E_{in}(t)) \) can be calculated for a BRBF by numerically integrating the work done by the floors inertial forces due to 
ground acceleration over the floors relative horizontal displacements from the beginning of the ground motion until 
time \( t \). The cumulative hysteretic energy absorbed by the BRBs at time \( t \) 
\( (E_{abs}(t)) \) can be calculated by numerically integrating the work done by the BRB axial forces over their respective axial displacements until time \( t \) which is 
esentially the cumulative area under the axial force-displacement hysteretic curves of the BRBs. The cumulative 
hysteretic energy absorbed by the BRBs at the end of the ground motions and after all the free vibrations stop is the 
cumulative hysteretic energy dissipation in the BRBs \( (E_d) \). The \( E_d \) values for each BRB calculated from the hybrid 
simulation results are presented in Table 4.7. The ratio of the numerically predicted \( E_d \) values with respect to the 
values calculated from hybrid simulation results are also presented in this table. The history of the total input energy 
and the total energy absorbed by the BRBs in the first, second, and third stories during the 3-element BRBF tests 
are shown in Figures 4.24 and 4.25. The fully numerical results with Steel01 and Steel02 material models are also 
shown in these figures.

The results presented in Figures 4.24 and 4.25 and Table 4.7 indicate that the input energy from ground motion 
No. 4 during 3E-BRBF-HS2 was 1675 kJ which was more than 5 times larger than the input energy from ground 
motion No. 11 during 3E-BRBF-HS1 (311 kJ). The energy dissipation in BRBs was also 4-7 times larger in 3E-
BRBF-HS2 compared to 3E-BRBF-HS1. This observation is consistent with the generally larger BRB displacement 
ductility demands during 3E-SCBF-HS2 compared to 3E-SCBF-HS1 in Table 4.7. The larger energy dissipation 
and ductility demands is an indication of the larger level of damage in the BRBs during the no-pulse type ground 
motion.

It is also observed that 90% of the seismic energy entered the BRBF in only 1.5 seconds during 4-5.5s from the 
beginning of the ground motion during 3E-BRBF-HS1 while this happened in 12 seconds during 2-14s from the 
beginning of the ground motion during 3E-BRBF-HS2. The above observations are consistent with the pulse-type 
nature of ground motion No. 11 used in 3E-BRBF-HS1. In pulse type ground motions, most of the seismic energy 
is concentrated in a single large pulse.
Figure 4.24: Energy response history of BRBF during 3E-BRBF-HS1 compared to the fully numerical results with updated material model calibration factors.
Figure 4.25: Energy response history of BRBF during 3E-BRBF-HS2 compared to the fully numerical results with updated material model calibration factors

Figure 4.26: Contribution of BRB energy dissipation in the BRBF (fully numerical vs. hybrid tests)
The energy dissipation contribution of the BRBs from the total input energy in the BRBF is shown in Figure 4.26. The results indicate that for all cases most of the input energy was dissipated in BRB1 and nearly 50% of the input energy was dissipated in BRBFs in the first two storeys. The results show that 0-2% of the input energy was dissipated in the BRBF5 which indicates that this brace stayed almost elastic and undamaged during the earthquakes. These results show that most of the damage was concentrated in the first storey of BRBF. It can be observed that the distribution of energy dissipation is similar for both 1E-BRBF-HS and 3E-BRBF-HS1 although a larger number of braces were physically tested in 3E-BRBF-HS1. This is due to the fact that the brace with largest contribution in energy dissipation (BRBF1) was physically tested in both tests and that the Steel02 material model used in both tests provided sufficient accuracy for energy response evaluation of BRB2 and BRB3 in 1E-BRBF-HS. On the other hand, it can be observed that BRB1 provides larger energy dissipation during 3E-BRBF-HS2 compared to 1E-BRBF-HS and 3E-BRBF-HS1. This observation suggests that the BRBF was more prone to form a concentration of damage and thus soft storey behaviour under the no-pulse type ground motion compared to the pulse type ground motion. The larger energy dissipation percentage can be attributed to the increased number of nonlinear cycles during the no-pulse type ground motion. It can be also partly attributed to the larger stiffness of the BRBs in the BRBF model used in 3E-BRBF-HS2 resulting in more absorption of energy into the first storey.

The response history and distribution of energy absorption and dissipation presented in Figures 4.24-4.26 confirm the accuracy of the numerical models in predicting the energy response of BRBF. The results indicate that the largest errors were 11-47% and occurred for the energy dissipation of BRBF3. The fully numerical results predict larger energy dissipation for BRBF1 and smaller energy dissipation for BRBF3. The average error of the fully numerical BRBF models with Steel02 material model was 1% while it was 15% for the models with Steel01 material model. This observation confirms that Steel02 model performs better than Steel01 in the prediction of energy response of the BRBF, especially for the no-pulse type ground motion.

4.5.5.3 Storey drifts and floor accelerations

BRBF storey drift response history during 3E-BRBF-HS1 and 3E-BRBF-HS2 are presented in Figures 4.27 and 4.28. The maximum residual and transient SDR and floor acceleration profiles along the BRBF height for 1E-BRBF-HS, 3E-BRBF-HS1, and 3E-BRBF-HS2 are shown in Figures 4.29-4.31. The drift and acceleration response predictions based on fully numerical models with Steel01 and Steel02 material models with updated calibration parameters for the material models are also shown in these figures. The response histories indicate a perfect match between the hybrid simulation results and the fully numerical predictions in the first 2-3 s of the ground motion where the response is mainly dominated by the elastic response of the BRBs. The fully numerical predictions start to slightly deviate from the realistic hybrid simulation results after the first nonlinear cycle which is the direct result of the inaccuracies in the response of material models in the BRBF first three storeys. The results presented in Figures 4.27 and 4.28 indicate that the deviations between the fully numerical model predictions and the hybrid simulation results are larger for the first three floors. The Steel01 model shows a larger difference compared to Steel02 for the top two floors. In fact, the BRBF model with Steel02 material model provides response predictions with excellent accuracy throughout the entire ground motion for the top two floors while the BRBF model with Steel01 material model is unable to provide comparable accuracy.
The SDR profiles in Figures 4.29 and 4.30 show that the maximum transient and residual SDR reached, respectively, 2.1% and 1.2% during 1E-BRBF-HS and 3E-BRBF-HS1. The SDR values reached 3.4% and 2.1% during 3E-BRBF-HS2 for transient and residual SDR, respectively. The results also indicate the concentration of floor drifts in the first floor for all cases which is consistent with the observations from the energy responses of the BRBF and the observations in the literature. Similarly, the fully numerical and hybrid simulation results predict larger residual drifts in the first storey of the BRBF which is an indication of larger damages and nonlinear deformations in this region. The larger residual drifts for 3E-BRBF-HS2 compared to 1E-BRBF-HS and 3E-BRBF-HS1 is an indication of the larger level of damage in the BRBF under ground motion No. 4 which is again consistent with the larger energy absorption and dissipation observed from the energy response of the BRBF during this ground motion.

The relative error of the floor drifts and accelerations from the 1-element BRBF test and the fully numerical analyses are calculated for floors 1-3 with respect to the 3-element BRBF tests and are presented in Tables 4.8 and 4.9. The results of Table 4.8 indicate that the residual SDR values of the 1E-BRBF-HS test were 22% and 14% smaller than 3E-BRBF-HS1 at the second and third floors, respectively. These differences are mainly attributed to the larger number of physical braces used in the 3-element BRBF test which mainly affected the residual drifts at the floor levels corresponding to the extra physical braces. Comparison of the fully numerical SDR predictions with the SDR values of the 3E-BRBF-HS1 in Figure 4.29 and Table 4.8 shows that the fully numerical models overestimated the maximum SDR values at floors 1-3. Based on the error values presented in Table 4.8 it can be concluded that the type of material model mostly affected the residual SDR values. Steel02 material model resulted in more accurate residual SDR predictions compared to Steel01 material model (12% vs. 50% average relative error over the first three storeys). On the other hand, Steel01 provided slightly more accurate predictions for the maximum transient SDR compared to Steel02 (18% vs. 23% average relative error over the first three storeys). Comparison of the fully numerical SDR predictions with the SDR values of the 3E-BRBF-HS2 in Figure 4.30 and Table 4.9 shows that the fully numerical models underestimated the maximum SDR values at the first three floors. Steel01 material model resulted in more accurate residual SDR predictions compared to Steel02 material model (24% vs. 72% average relative error over the first three floors). On the other hand, Steel02 provided slightly more accurate predictions for the maximum transient SDR compared to Steel01 (12% vs. 18% average relative error at the first three storeys).

It can be observed from Figures 4.29 and 4.30 that the drift profiles from BRBF models with Steel01 material model show less concentration of drift in the bottom storeys and more uniform drift profile compared to the hybrid simulations and the BRBF models with Steel02 material model. This is due to the fact that larger post-yield stiffness was used in the calibration of the Steel01 material model to consider isotropic hardening in BRBs. The unrealistically larger post-yield stiffness used for this model helps in transferring the forces across BRBF height resulting in less concentration of demand in the bottom storeys. However, the realistic hybrid simulation results indicate that this behaviour could be unrealistic for BRBFs and the BRBF model with Steel01 material model is unable to properly predict realistic damage distribution and formation of soft storey mechanism in BRBFs. The
results generally suggest larger scatter between fully numerical predictions and hybrid simulation results for residual SDR values compared to the maximum transient SDR values.

The BRBF floor acceleration profiles shown in Figure 4.31 indicates a rather uniform distribution of acceleration over the floors. No acceleration amplification is observed in any of the floors. Instead, the ground acceleration is reduced over the building height particularly for 3E-BRBF-HS2 which is the direct consequence of large nonlinear deformations of BRBs in storeys 1-4. This observation also suggests a rather uniform and efficient distribution of stiffness along BRBF height. Comparison of the fully numerical and hybrid simulation results indicates that the acceleration response of the 1-element and 3-element BRBF tests and also the fully numerical models with Steel02 material model were very close with a maximum relative error of 9%. The fully numerical model with Steel01 material model shows larger differences especially for 3E-BRBF-HS2 with a maximum relative error of 51% and an average relative error of 35% over floors 1-3.
Figure 4.27: BRBF storey drift response history during 3E-BRBF-HS1 compared to the fully numerical results.
Figure 4.28: BRBF storey drift response history during 3E-BRBF-HS2 compared to the fully numerical results

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Figure 4.29: BRBF maximum SDR profiles during 1E-BRBF-HS and 3E-BRBF-HS1 compared to the fully numerical results (a) residual, (b) transient

Figure 4.30: BRBF maximum SDR profiles during 3E-BRBF-HS2 compared to the fully numerical results (a) residual, (b) transient
Figure 4.31: BRBF maximum absolute acceleration profiles during BRBF tests compared to the fully numerical results (a) 1E-BRBF-HS and 3E-BRBF-HS1, (b) 3E-BRBF-HS2

Table 4.8: Relative error of BRBF floor drift and acceleration results with respect to 3E-BRBF-HS1 results

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Error in residual SDR (%)</th>
<th>Error in max. transient SDR (%)</th>
<th>Error in max. floor acceleration (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1E-BRBF-HS</td>
<td>BRBF-S01</td>
<td>BRBF-S02</td>
</tr>
<tr>
<td>1</td>
<td>-8</td>
<td>39</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>-22</td>
<td>40</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>-14</td>
<td>70</td>
<td>10</td>
</tr>
<tr>
<td>Average</td>
<td>-15</td>
<td>50</td>
<td>12</td>
</tr>
</tbody>
</table>

Table 4.9: Relative error of BRBF floor drift and acceleration results with respect to 3E-BRBF-HS2 results

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Error in residual SDR (%)</th>
<th>Error in max. transient SDR (%)</th>
<th>Error in max. floor acceleration (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BRBF-D01</td>
<td>BRBF-D02</td>
<td>BRBF-D01</td>
</tr>
<tr>
<td>1</td>
<td>-36</td>
<td>-71</td>
<td>-21</td>
</tr>
<tr>
<td>2</td>
<td>-27</td>
<td>-71</td>
<td>-13</td>
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<td>3</td>
<td>-9</td>
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<td>-21</td>
</tr>
<tr>
<td>Average</td>
<td>-24</td>
<td>-72</td>
<td>-18</td>
</tr>
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</table>
4.6 SEISMIC PERFORMANCE ASSESSMENT OF THE BRBF

4.6.1 Pushover analysis

Pushover analysis was performed on the BRBF models to verify the impact of the type of material model, the connection models, and the $P - \Delta$ effects on the global and ultimate response of the BRBF. For this purpose, the BRBF model was laterally loaded incrementally in a displacement-controlled manner up to a top floor drift ratio of 2.7%. The equation proposed by ASCE7-10 (ASCE, 2010) was used for the lateral load vertical distribution in the pushover analysis. According to this equation, the lateral load factor can be obtained from the following equation:

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^b W_i h_i^k} \quad (4.4)$$

where $C_{vx}$ is the vertical distribution factor at $x^{th}$ floor, $W_x$ and $h_x$ are effective seismic weight and the total height of the $x^{th}$ floor, and $k$ is an exponent related to the principal period of the structure and is equal to 1.2 for the BRBF considered in this study.

Figure 4.32 shows the base shear vs. top floor drift response during the pushover analysis for the BRBF models considered in this study. As can be seen from Figure 4.32, the lateral responses of the BRBF models start with a linear elastic behaviour up to a base shear of approximately 500 kN. The curves then start to gradually soften which is an indication of the consecutive yielding of the BRBs in different storeys. The curves then reach a maximum point after which the base shear starts to degrade due to the $P - \Delta$ effects of the gravity loads. The points corresponding to the maximum base shear of the curves are shown with circles in Figure 4.32 and their corresponding base shear value ($V_{b,m}$) and top floor displacements ($\Delta_{roof}$) are presented in Table 4.10. A closer view of the curves at the yielding zone is also provided in Figure 4.32. It can be observed from Figure 4.32 that the material models did not affect the stiffness of the BRBF pushover curves prior to yielding. However, it can be observed from this figure and the values presented in Table 4.10 that the BRBF models with Steel01 material model provided larger post-yield stiffness and thus a delayed degradation of the lateral force capacity. It can be also observed that the BRBF models with detailed connection models provided 18% larger initial stiffness compared to the BRBF models with the simplified models. This is to some extent due to the added lateral stiffness provided by the rotational stiffness of the beam-column and beam-column-brace connections that are considered in the BRBF detailed models. It is also to a larger extent attributed to the larger axial stiffness of the BRBs in the detailed model. As discussed in Section 4.4.2.2, all parts of the BRBs including the gusset connection and the cruciform sections were fully designed and modelled in the BRBF detailed model. The resulting BRB models had larger overall axial stiffness than what was initially assumed for the full BRB in the simplified model.
Figure 4.32: Base shear vs. top floor drift response of the BRBF during the pushover analysis

Figure 4.33: BRBF SDR profile during the pushover analysis: (a) at the maximum top floor lateral displacement and (b) at the maximum base shear

Table 4.10: Base shear and drift values at the peaks of the BRBF pushover curves

<table>
<thead>
<tr>
<th>BRBF model</th>
<th>$V_{b,m}$ (kN)</th>
<th>$\Delta_{p,m}$ (mm)</th>
<th>$SDR_{p,m}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRBF-S02</td>
<td>548</td>
<td>143</td>
<td>1.4</td>
</tr>
<tr>
<td>BRBF-D02</td>
<td>575</td>
<td>196</td>
<td>2.0</td>
</tr>
<tr>
<td>BRBF-S01</td>
<td>537</td>
<td>247</td>
<td>2.6</td>
</tr>
<tr>
<td>BRBF-D01</td>
<td>610</td>
<td>392</td>
<td>3.9</td>
</tr>
</tbody>
</table>
Figure 4.33 shows the BRB SDR profile during the pushover analysis. Figure 4.33a shows the SDR profile at the maximum top floor lateral displacement and Figure 4.33b shows the SDR profile at the maximum base shear. The values of the maximum SDR when the BRBF reaches the maximum base shear ($SD_{R,v,m}$) are also presented in Table 4.10. The SDR profiles presented in Figure 4.33 indicate concentration of storey drifts in the lower storeys especially in the first storey of the BRBF. The results show a larger concentration of storey drifts for the BRBF models with Steel02 material model compared to the models with Steel01 material model. As previously discussed, this is due to the larger post-yield stiffness of the Steel01 material model that helps in a more uniform distribution of the storey drifts along the BRBF height. Figure 4.33b and the SDR values presented in Table 4.10 indicate that the storey drifts at the maximum base shear are larger for the detailed models compared to the simplified models which is primary due to the larger base shear capacity of these models resulting from inclusion of the actual flexural stiffness of the connections and the actual axial stiffness of the BRBs in the BRBF detailed models.

### 4.6.2 Response history analysis

NRHA was performed on the BRBF under a suit of 16 spectrum compatible ground motions. The results are presented in the following sections. The results were used to conduct a seismic performance assessment of the BRBF following ASCE 41-13 (ASCE, 2014) guidelines for seismic evaluation and retrofit of the existing buildings. ASCE 41-13 recommends considering the rotational stiffness and behaviour of the beam-column connections as well as the BRB gusset plate connections in the evaluation of the BRBF seismic performance. Accordingly, and following the observations from the pushover analysis on the potential impact of the behaviour of the connections on the ultimate response of the BRBF, the BRBF model with detailed connection models were used in the NRHA. In order to verify the impact of the type of material model adopted for the BRB yielding core, both Steel01 and Steel02 material models with the original calibration (see Section 4.4.1) were used for the BRBF performance assessment. Therefore, BRBF-D01 and BRBF-D02 models were separately used in the NRHA. These models are, respectively, referred to as Model 1 (or 1) and Model 2 (or 2) in the following discussions.

NRHA was performed on the BRBF under the 16 ground motions listed in Tables 4.5 and 4.6. Details on the selection of the ground motions and their characteristics were discussed in Section 4.5.3. The ground motions were selected and scaled to match the design response spectrum of Los Angeles with a seismic hazard level corresponding to 10% probability of exceedance in 50 years (475 years return period). These ground motions are referred to as the Design Basis Earthquakes (DBE) in the following sections. In order to assess the performance of the BRBF at higher hazard levels, the NRHA was also performed under the same ground motions scaled up by 50%. This level of seismic hazard corresponds to 2% probability of exceedance in 50 years (2475 years return period) and the resulting ground motions are referred to as the Maximum Considered Earthquakes (MCE) in the following sections.

### 4.6.2.1 Performance criteria

The performance of the BRBF was evaluated by studying the maximum response parameters of the BRBF at two levels: global level and local level. For the global level, the storey drifts and the absolute accelerations of the floors were analyzed. For the local level, the axial displacement ductility (maximum and cumulative) and the energy...
dissipation in the BRBs were evaluated. Nonlinear deformations in the beams and columns and the response of the beam-column connections were also investigated. The performance levels considered were immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The maximum acceptable values for the maximum transient and residual SDR (respectively $SDR_{T,m}$ and $SDR_{R,m}$), the brace axial displacement ductility ($\mu_{max}$), and the plastic rotation of the beams and columns for the performance levels considered in this study are presented in Table 4.11. The drift limits were roughly defined based on the amount of expected damage to the structural and non-structural elements and the ability of the main structural components to reliably carry the gravity loads thus maintaining the stability of the system. The damage levels corresponding to each of the performance levels are described in ASCE 41-13. In the evaluation of the damage levels corresponding to the drift values, it was assumed that the BRBs start to develop axial yielding at a SDR value of approximately 0.7%. This is consistent with the various experimental and numerical observations in this study and also in the literature. The 0.8% residual drift limit for the life safety performance level was chosen considering the ductility capacity of the BRBFs. This limit is consistent with the ranges defined in the literature for the safe evacuation of the building after an earthquake (McCormick et al., 2008; Erochko et al., 2011). The $SDR_{T,m}$ limit for the LS performance level in Table 4.11 were selected based on the ASCE 7-10 provisions. The brace ductility limits and the column plastic rotation limits were however selected based on the ASCE 41-13 provisions. Since the BRBF beams and columns did not fulfil the criteria for the compact sections defined in ASCE 41-10, the limits for the plastic rotation of the beams and columns were selected based on non-compact sections as per ASCE 41-13 provisions. It should be noted that while the drift values and limits can be used to illustrate the overall structural response at various performance levels, the actual performance of the BRBF should be evaluated based on the component and element deformation levels (ASCE 41-13). Therefore, the axial ductility of the BRBs and the plastic rotations at the beams and columns ends were given more weight in the evaluation of the seismic performance of the BRBF in this study.

**Table 4.11:** Acceptance criteria for the performance levels considered for the BRBF

<table>
<thead>
<tr>
<th>Response type</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storey lateral deflection</td>
<td>$SDR_{T,m}(%)$</td>
<td>0.7</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>$SDR_{R,m}(%)$</td>
<td>Negligible</td>
<td>0.8</td>
</tr>
<tr>
<td>Brace axial deformation</td>
<td>$\mu_{max}$</td>
<td>4</td>
<td>11</td>
</tr>
<tr>
<td>Column flexural rotation</td>
<td>$\theta_p/\theta_y$ ($P/P_{CL} &lt; 0.2$)</td>
<td>0.25</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>$\theta_p/\theta_y$ ($0.2 \leq P/P_{CL} \leq 0.5$)</td>
<td>0.25</td>
<td>1.2</td>
</tr>
<tr>
<td>Beam flexural rotation</td>
<td>$\theta_p/\theta_y$</td>
<td>0.25</td>
<td>3</td>
</tr>
</tbody>
</table>
4.6.2.2 Global response parameters

4.6.2.2.1 Storey drift ratio (SDR)

The $SDR_{T,m}$ and $SDR_{R,m}$ values are often used as global performance indicators in performance assessment of the buildings. They are directly related with and can show an overview of the amount of damage to the structural and non-structural components in the SFRS and the ability of the SFRS to carry the gravity loads after an earthquake. $SDR_{T,m}$ and $SDR_{R,m}$ values for the BRBF under each of the ground motions at DBE and MCE levels are presented in Tables 4.12 and 4.13. The results are shown for both models 1 and 2. Figures 4.34 and 4.35 show distribution of the maximum SDR values for all ground motions. The BRBF maximum SDR profiles for models 1 and 2 under DBE and MCE ground motions are shown in Figures 4.36-4.39. The average of the maximum values over all ground motions and the average plus one standard deviation (SD) for each storey is also shown in these figures. Finally, the contribution of each storey to the maximum lateral deflection of the top floor level is shown in Figures 4.40 and 4.41.

In order to achieve a reliable assessment, one should verify if the BRBF numerical models provide sufficient accuracy for the range of the response they are considered for and determine if the numerical response predictions are valid and meaningful for such response range. In addition, it should be verified if the main structural components can keep their integrity and the gravity support system is able to reliably transfer the gravity loads to the foundations. The unacceptable results should then be identified and excluded from the BRBF performance assessment. The review of the literature (e.g. Hsiao et al., 2013) indicated that a 5% drift limit can be approximately considered as an acceptable range for the validity of the braced frame numerical models with detailed connection models. The drift results presented in Tables 4.12 and 4.13 indicate that the maximum SDR for five of the MCE ground motions slightly exceeded the 5% limit but they were all below 5.5%. Therefore, considering the level of the modelling details adopted in the BRBF models, especially for the beam-column and brace connections, the numerical results for all ground motions were assumed to be acceptable in this study.

The results presented in Tables 4.12 and 4.13 indicate that the $SDR_{T,m}$ values for both models 1 and 2 were within the range of 1.4%-3.8% for the DBE ground motions and 2.1%-5.5% for the MCE ground motions. The $SDR_{R,m}$ values for both models were within 0.1%-1.8% range for the DBE ground motions and within 0.2%-3.4% range for the MCE ground motions. The results also show that the average of the maximum drift response of the BRBF was rather similar for the pulse and no-pulse type ground motions.

The results presented in Tables 4.12 and 4.13 and Figures 4.34 and 4.35 show that model 1 and model 2 predicted similar maximum transient drifts. The residual drifts, however, show slightly larger deviations between the two models with an average relative difference of 50% and 15% for the DBE and MCE ground motions, respectively. Figures 4.36-4.39 show that both models also predicted a similar distribution of the drifts along the BRBF height. Figures 4.40 and 4.41 indicate the contribution of each storey to the maximum lateral deflection of the top floor level for DBE and MCE ground motions. The results indicate that the first storey provided the largest contribution to the top floor lateral deflection amongst other storeys with an average contribution of 37% for both levels of ground motion. The results also show that, on average, more than 65% of the lateral deflection occurs in the first storey.
two storeys. Both model 1 and 2 predict a considerable concentration of drift in the first two storeys, especially in the first storey. This indicates large chances of soft storey mechanism in the building for both DBE and MCE ground motions.

Figures 4.36-4.39 show the maximum SDR for each storey. The average and average plus one standard deviation of the SDR values at each storey are also shown in these figures. The limits for the acceptance criteria for the three performance levels considered in this study are also indicated on these figures. The results indicate that the maximum residual and transient drifts occurred in the first storey. The maximum average transient SDR values for both models 1 and 2 were 2.8% and 4.4% under the DBE and MCE ground motions, respectively. The maximum average residual SDR values were 0.8% and 0.9%, respectively, for models 1 and 2 under the DBE ground motions. The maximum average residual SDR values were 1.5% and 1.7%, respectively, for models 1 and 2 under MCE ground motions. As can be observed from Figures 4.36-4.39, the average transient SDR in the first storey was slightly larger than the LS and CP drift limits under the DBE and MCE ground motions. The average residual SDR in the first storey was also slightly larger than the LS limit under the DBE ground motions but it satisfied the CP limit under the MCE ground motions. The average drifts under DBE and MCE ground motions respectively satisfied the LS and CP limits in the rest of the storeys. If a normal distribution is assumed for the SDR values, the average plus one SD values represent the 84-percentile which is representative of the SDR value which 84% of the data fall below it. Figures 4.36-4.39 indicate that for both models the average plus one SD values in the first two storeys did not satisfy the LS and CP performance limits, respectively, under DBE and MCE ground motions. The coefficient of variation (CV) was calculated for the SDR values. The CV range was 17-33% and 15-30% for the transient SDR values under the DBE and MCE ground motions, respectively. For the residual SDR values, the CV range was 52-133% and 44-134% under the DBE and MCE ground motions, respectively. The large CV values suggest that there was significant variability in the SDR predictions. The CV values for the residual SDR were 3-4 times larger than the CV values for the transient SDR. The large uncertainty observed for the residual drift demands is mainly due to the sensitivity of the residual drifts to several parameters including the hysteretic shape of the energy dissipating elements and the number/magnitude of the nonlinear cycles in the ground motions. The large scatter observed for the residual drifts is also consistent with the observations in the literature. The residual SDR values are believed to be extremely difficult to be reliably predicted.

A summary of the SDR response of low- and mid-rise BRBFs studied by other researchers under suites of DBE level ground motions for Los Angeles is presented in Table 4.14. Information on the type of brace model used in these studies and consideration of the contribution of the lateral stiffness of the gravity columns are also presented in this table. The results indicate that the drift values of the BRBF considered in this study are consistent with other studies but are larger than the results of most of the studies presented in Table 4.14.
### Table 4.12: Maximum values of the BRBF global response parameters under the DBE ground motions

<table>
<thead>
<tr>
<th>Ground motions</th>
<th>Rec 1</th>
<th>Rec 2</th>
<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
<th>Rec 11</th>
<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$SDR_{T,m}$(%)</td>
<td>1</td>
<td>2.5</td>
<td>1.4</td>
<td>3.2</td>
<td>2.5</td>
<td>3.5</td>
<td>3.3</td>
<td>2.8</td>
<td>2.7</td>
<td>2.6</td>
<td>2.2</td>
<td>3.1</td>
<td>2.3</td>
<td>2.8</td>
<td>2.8</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2.9</td>
<td>1.4</td>
<td>3.4</td>
<td>2.7</td>
<td>3.8</td>
<td>3.7</td>
<td>2.9</td>
<td>2.9</td>
<td>2.8</td>
<td>2.2</td>
<td>3.0</td>
<td>2.5</td>
<td>2.5</td>
<td>2.9</td>
<td>3.1</td>
</tr>
<tr>
<td>$SDR_{R,m}$(%)</td>
<td>1</td>
<td>0.8</td>
<td>0.9</td>
<td>0.8</td>
<td>0.4</td>
<td>1.7</td>
<td>0.7</td>
<td>0.3</td>
<td>1.1</td>
<td>0.6</td>
<td>0.7</td>
<td>1.8</td>
<td>0.7</td>
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<td>1.6</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.0</td>
<td>0.4</td>
<td>1.4</td>
<td>0.2</td>
<td>1.1</td>
<td>0.9</td>
<td>0.4</td>
<td>1.1</td>
<td>0.1</td>
<td>0.4</td>
<td>1.3</td>
<td>0.2</td>
<td>1.2</td>
<td>1.6</td>
<td>1.3</td>
</tr>
<tr>
<td>$FA_m(g)$</td>
<td>1</td>
<td>0.6</td>
<td>0.8</td>
<td>0.6</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
<td>1.5</td>
<td>0.5</td>
<td>0.8</td>
<td>0.8</td>
<td>0.6</td>
<td>0.8</td>
<td>0.6</td>
<td>0.6</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>0.7</td>
<td>0.6</td>
<td>0.7</td>
<td>0.5</td>
<td>0.5</td>
<td>1.2</td>
<td>0.5</td>
<td>0.7</td>
<td>0.7</td>
<td>0.5</td>
<td>0.6</td>
<td>0.5</td>
<td>0.6</td>
<td>0.5</td>
</tr>
</tbody>
</table>

### Table 4.13: Maximum values of the BRBF global response parameters under the MCE ground motions

<table>
<thead>
<tr>
<th>Ground motions</th>
<th>Rec 1</th>
<th>Rec 2</th>
<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
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<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$SDR_{T,m}$(%)</td>
<td>1</td>
<td>3.7</td>
<td>2.4</td>
<td>4.0</td>
<td>5.2</td>
<td>4.4</td>
<td>4.0</td>
<td>5.5</td>
<td>5.3</td>
<td>4.9</td>
<td>3.9</td>
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<td>5.0</td>
<td>4.7</td>
<td>4.5</td>
<td>5.3</td>
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<tr>
<td></td>
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<td>4.0</td>
<td>2.1</td>
<td>4.8</td>
<td>5.1</td>
<td>4.5</td>
<td>4.3</td>
<td>5.4</td>
<td>5.4</td>
<td>4.9</td>
<td>3.7</td>
<td>3.8</td>
<td>5.4</td>
<td>4.0</td>
<td>4.1</td>
<td>5.5</td>
</tr>
<tr>
<td>$SDR_{R,m}$(%)</td>
<td>1</td>
<td>1.4</td>
<td>1.4</td>
<td>1.1</td>
<td>0.8</td>
<td>0.2</td>
<td>1.0</td>
<td>0.2</td>
<td>3.4</td>
<td>2.2</td>
<td>0.6</td>
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<td>2.8</td>
<td>2.8</td>
<td>2.3</td>
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<tr>
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<td>1.8</td>
<td>1.0</td>
<td>0.8</td>
<td>0.8</td>
<td>3.0</td>
<td>2.3</td>
<td>1.0</td>
<td>1.6</td>
<td>2.0</td>
<td>2.4</td>
<td>2.5</td>
<td>3.2</td>
</tr>
<tr>
<td>$FA_m(g)$</td>
<td>1</td>
<td>0.7</td>
<td>0.9</td>
<td>0.7</td>
<td>1.0</td>
<td>0.6</td>
<td>0.9</td>
<td>1.7</td>
<td>0.6</td>
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<td>1.3</td>
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<tr>
<td></td>
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<td>0.8</td>
<td>0.6</td>
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<td>0.6</td>
<td>0.8</td>
<td>1.6</td>
<td>0.6</td>
<td>1.0</td>
<td>1.0</td>
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<td>0.8</td>
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</tbody>
</table>

### Table 4.14: Average SDR response of low- and mid-rise BRBFs under suites of ground motions with hazard levels corresponding to 10% probability of exceedance in 50 years (DBE) for Los Angeles

<table>
<thead>
<tr>
<th></th>
<th>Number of storeys</th>
<th>Gravity column lateral stiffness</th>
<th>Brace model</th>
<th>$SDR_{T,avg}$(%)</th>
<th>$SDR_{R,avg}$(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRBF in this study</td>
<td>5</td>
<td>Considered</td>
<td>Steel01/Steel02</td>
<td>2.8</td>
<td>0.9 (Steel01) 0.8 (Steel02)</td>
</tr>
<tr>
<td>Gray et al. (2014)</td>
<td>12</td>
<td>Considered</td>
<td>Steel02</td>
<td>3.0</td>
<td>2.1</td>
</tr>
<tr>
<td>Fahnestok et al. (2007a)</td>
<td>4</td>
<td>Considered</td>
<td>Isotropic-kinematic hardening</td>
<td>2</td>
<td>0.5</td>
</tr>
<tr>
<td>Sabelli et al. (2003)</td>
<td>6</td>
<td>Considered</td>
<td>Elastic-perfectly plastic</td>
<td>1.6</td>
<td>0.7</td>
</tr>
<tr>
<td>Uriz and Mahin (2008)</td>
<td>6</td>
<td>Considered</td>
<td>Steel02</td>
<td>1.4</td>
<td>--</td>
</tr>
</tbody>
</table>
Figure 4.34: Maximum SDR values obtained based on models 1 and 2 for the BRBF under the DBE ground motions
Figure 4.35: Maximum SDR values obtained based on models 1 and 2 for the BRBF under the MCE ground motions

Figure 4.36: Maximum transient SDR values for each storey of the BRBF under the DBE ground motions: (a) model 1 and (b) model 2

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Figure 4.37: Maximum residual SDR values for each storey of the BRBF under the DBE ground motions: (a) model 1 and (b) model 2

Figure 4.38: Maximum transient SDR values for each storey of the BRBF under the MCE ground motions: (a) model 1 and (b) model 2
Figure 4.39: Maximum residual SDR values for each storey of the BRBF under the MCE ground motions: (a) model 1 and (b) model 2

Figure 4.40: Contribution of the relative lateral deflection of the BRBF storeys to the top floor lateral deformation for models 1 and 2 when the top floor reaches the maximum lateral deflection under the DBE ground motions.
Chapter 4: BRBF Seismic Performance Assessment

4.6.2.2 Floor acceleration (FA)

The maximum absolute floor accelerations ($F_A$) in the BRBF based on models 1 and 2 under both DBE and MCE level ground motions are presented in Tables 4.12 and 4.13. Figures 4.42 and 4.43 show the profile of the maximum floor accelerations respectively for DBE and MCE ground motions. The average and average plus one standard deviation values for each floor level are also shown in these figures. The results presented in Tables 4.12 and 4.13 indicate that model 1 predicted slightly larger $F_A$ values for almost all the ground motions. The average relative difference was 15% for both DBE and MCE ground motions. This was due to the larger post-yield stiffness of the Steel01 material model in model 1 compared to the Steel02 material model in model 2. Both models, however, predicted similar distributions of floor acceleration along the building height. Figures 4.42 and 4.43 indicate that for both models 1 and 2, the $F_A$ average values of each floor level were within 0.42g-0.7g range for DBE ground motions and within 0.49g-0.94g range for the MCE ground motions. This indicates that although the MCE ground motion intensities were 50% larger than their corresponding DBE levels, the BRBF floor accelerations did not show a comparable increase in magnitude. This was due to the fact that almost all of the BRBs significantly yielded during the MCE ground motions and thus the BRBF did not provide further resistance to the ground acceleration under the MCE ground motions compared to the DBE ground motions. The CV values for maximum floor accelerations were within 4-40% for all the ground motions which indicated a much smaller scatter in the acceleration response compared to the residual drifts. The smaller scatter in the acceleration response was potentially due to the rather small variations of the ground motion spectral accelerations around the principal period of the BRBF as can be seen in Figure 4.16. The acceleration response and the principal period of the BRBF majorly depends on the elastic stiffness of the BRBF structural components that can be accurately modelled.

Figure 4.41: Contribution of the relative lateral deflection of the BRBF storeys to the top floor lateral deformation for models 1 and 2 when the top floor reaches the maximum lateral deflection under the MCE ground motions.
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4.6.2.3 Local response parameters

4.6.2.3.1 Response of the BRBs

Since significant nonlinear deformation and ductility is expected in the BRBs, ASCE 41-13 categorizes the axial actions in BRBs as deformation-controlled with ductile behaviour. The nonlinear response of the BRBs was evaluated with three parameters: the maximum axial ductility ($\mu_m$) defined as the ratio of the maximum BRB axial displacement to the BRB axial yield displacement, the maximum cumulative ductility ($\mu_{cu}$) defined as the ratio of the sum of the BRB axial plastic deformations during a ground motion to the BRB axial yield displacement, and the ratio of the total amount of BRB hysteretic energy dissipation to the total seismic energy that enters the BRBF.
during a ground motions ($E_d/E_{in}$). The maximum values for the above parameters under the DBE and MCE ground motions are presented in Tables 4.15 and 4.16. The values are presented only for the BRBs in the first four storeys (BRB1-BRB4) that experienced considerable yielding. The results indicate that in general model 2 predicted larger ductility demands especially for the cumulative ductility compared to model 1. The average values (average over the ground motions) of the maximum ductility demands predicted by model 2 for the BRBs in the first four storeys were up to 5% larger than model 1 predictions for both DBE and MCE ground motions. For the cumulative ductility demands, this difference was within 12-65%. The average of the model 2 predictions for the energy dissipation ratios was like the model 1 predictions for BRB1, up to 21% larger for BRB2 and BRB3, and up to 16% smaller for BRB4. The larger ductility demands in model 2 predictions compared to model 1 predictions can be attributed to the smaller post-yield stiffness and the gradual transitions between the elastic and inelastic response that was present in Steel02 material model in model 2. The BRB1 and BRB2 ductility demands presented in Tables 4.15 and 4.16 also show that the average values of the maximum ductility demands were generally smaller for the no-pulse type ground motions compared to the pulse type ground motions. However, the cumulative ductility demands for the no-pulse type ground motions were up to 37% larger than the pulse type ground motions. This indicates that larger damage occurred in the BRB1 and BRB2 yielding cores during the no-pulse type ground motions which was mainly due to the larger number of nonlinear cycles in these ground motions.

The maximum tension and compression ductility demands in the BRBF (maximum over all of the BRBs) are presented in Figures 4.44 and 4.45 for the DBE and MCE ground motions. The results indicate that the maximum tension and compression ductility demands were similar. The differences of the average values (over all of the ground motions) were within 1-7%. This indicates that although the BRBs were installed in a zig-zag configuration along the BRBF height, due to the symmetric response of the BRBs in compression and tension, they were equally loaded in tension and compression along the building height.

Figures 4.46 and 4.47 show the BRB maximum displacement ductility demands at different storeys for models 1 and 2 under the DBE and MCE ground motions. The average and average plus one SD of the maximum ductility demands for each BRB over all of the ground motions are also shown in these figures. The results indicate that both models predicted similar distribution for the BRB ductility demands along the building height. It can be also observed that the average and average plus one SD of the maximum ductility demands satisfied the LS and CP limits, respectively, for the DBE and MCE ground motions. The CV values were within 18-34%. The results also show that the maximum ductility demands occurred in the first storey BRBs with an average value of 7 which confirms the concentration of damage in the first storey which is consistent with previous observations from the BRBF drift responses. The ductility demands for BRB5 in Figures 4.46 and 4.47 reveal that this brace experienced minor yielding during the DBE and MCE ground motions.

The values of $E_d/E_{in}$ for each storey under the DBE and MCE ground motions are presented in Tables 4.15 and 4.16. The contribution of the energy dissipation in the BRBs in each storey to the total energy dissipated in the BRBF is shown in Figures 4.48 and 4.49. The results are presented for both model 1 and model 2. The results indicate that more than 50% of the seismic energy was dissipated in BRB1 and BRB2 during DBE and MCE ground motions with BRB1 providing the maximum contribution (approximately 35%). The results also indicate that other
energy dissipation mechanisms in the BRBF, including nonlinear deformation of beams and columns and majorly the viscous damping in the structure, only dissipated around 26% of the seismic energy on average in the BRBF.

The results presented in Tables 4.15 and 4.16 indicate that the maximum cumulative ductility demands on the BRBs were 91 and 144, respectively, for the DBE and MCE ground motions. Experimental results in the literature show that the maximum cumulative ductility capacity of full-scale BRBs can be as high as 600-1700 (Merritt et al., 2003). On the other hand, the BRB qualification tests required by the ANSI/AISC 341-10 (AISC, 2010b) imposes a cumulative ductility demand of 200 on the BRBs. Both the minimum qualification test requirements and the actual cumulative ductility capacity of the BRBs are considerably larger than the BRB maximum ductility demands during the DBE and MCE ground motions. Therefore, although low-cycle fatigue behaviour of the BRB yielding cores was not considered in the BRBF numerical models, low-cycle fatigue failure of the BRB yielding core was not expected to occur in any of the ground motions. The axial strain of the BRBs’ yielding core was measured during the NRHA. The results indicate that BRB1 yielding core experienced the maximum axial strains in the BRBF. The axial strains were similar for models 1 and 2. The average values were 2.2% and 3.6% for DBE and MCE ground motions, respectively. These values are within the ranges observed in other investigations reported in the literature (e.g. Tremblay et al., 2004; Lin et al., 2012). ASCE 41-13 requires the maximum BRB axial strain to be less than 2.5%. Therefore, the BRBs satisfied the ASCE 41-13 axial strain limit under the DBE ground motions. However, they failed to satisfy this limit under the MCE ground motions. As discussed above, the experimentally verified ductility capacity of the BRBs suggests significantly larger ductility capacities for BRBs compared to the minimum requirements by the codes. Therefore, the large axial strains during the MCE ground motions were not expected to limit the performance of the BRBF. As discussed in Section 4.4.2.2, the unconfined cruciform sections of the BRBs were equipped with a low-cycle fatigue model to evaluate the amount of damage in this region. The percentage of low-cycle fatigue damage in the middle and at the outermost cross section fibres were measured during the NRHA. The results indicated that the cruciform section experienced some yielding, especially during the MCE ground motions mainly due to the rotation demands at the ends of the BRB encasing. The results however indicate that the damage index did not exceed 5% and 14% during the DBE and MCE ground motions, respectively. Therefore, the BRB’s unconfined cruciform sections did not experience low-cycle fatigue fracture as well.

Based on ASCE 41-13 requirements, all the actions on the gusset plates should be considered as force controlled. The only exception is when the brace connections are explicitly modelled and their ductile response is verified by the experiments. The axial and flexural capacities of the BRB gusset plate connections were investigated through detailed 3D continuum FE models as discussed in Section 4.4.2.3. The results indicated that the axial load and ductility capacities of the BRBs used in the BRBF were not limited by global buckling of the braces and failure of the gusset plates. The flexural rotation of the BRB gusset plates measured during the NRHA indicate that the maximum rotation was reached during the MCE ground motions and was 0.0065 rad producing a 49 kN-m moment action which was within the elastic flexural response of the BRB gusset plate connections. Therefore, the moment contributions of the BRB gusset plates were negligible, and all gusset plates stayed elastic during both DBE and MCE ground motions. Based on the above discussion, it can be assumed that the gusset plates performed as expected without any considerable damage and fracture during the DBE and MCE ground motions.
Table 4.15: Maximum BRBF brace response values under the DBE ground motions

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Table 4.16: Maximum BRBF brace response values under the MCE ground motions

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Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Figure 4.44: Maximum axial displacement ductility of the braces in the BRBF based on models 1 and 2 under the DBE ground motions (positive is tension and negative is compression)

Figure 4.45: Maximum axial displacement ductility of the braces in the BRBF based on models 1 and 2 under the MCE ground motions (positive is tension and negative is compression)
Figure 4.46: Maximum axial displacement ductility of the BRBs in the BRBF under the DBE ground motions: (a) model 1 and (b) model 2

Figure 4.47: Maximum axial displacement ductility of the BRBs in the BRBF under the MCE ground motions: (a) model 1 and (b) model 2
4.6.2.3.2 Response of the beams, columns, and beam-column connections

The beams and columns in the BRBF were designed without consideration of seismic flexural demands at their ends. However, the storey drifts and the rigid beam-column-brace connections can create flexural demands in the BRBF beams and columns during the ground motions. Therefore, the nonlinear axial-flexural demands in the beams and columns were investigated during the NRHA. In order to model the diaphragm effect of the floor slabs, the horizontal movements of the nodes at each floor level were constrained in the BRBF model. Therefore, no axial forces were developed in the floor beams in the BRBF models. However, small magnitudes of axial force can...
develop in the BRBF floor beams during the ground motions due to the seismic shear flow transmitted from floor diaphragms to the floor beams. In order to consider these forces, the axial forces used in the capacity design of the beams were conservatively assumed to be present simultaneously with the peak flexural demands on the beams. In order to investigate the nonlinear deformations and yielding of the beams and columns during the ground motions, the beams and columns were assumed to have yielded if the strain in any of the outermost cross section fibres at the ends of the beams and columns reached the yield strain. Tables 4.17 and 4.18 show the number of beams and column ends that experience yielding during the ground motions. The results are presented for both models 1 and 2 and for the DBE and MCE level ground motions. The results indicate that both models predicted similar numbers of yielding in beams and columns. It can be also observed that the occurrence of yielding in columns significantly increased during the MCE ground motions compared to the DBE ground motions. This increase was larger than the increase in the occurrence of yielding in the floor beams and was mainly attributed to the larger frame actions and storey drifts during the MCE ground motions.

Table 4.17: Number of BRBF floor beam ends that yield under the DBE and MCE ground motions

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<td>1</td>
<td>2</td>
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</tr>
</tbody>
</table>

Table 4.18: Number of BRBF column ends that yield under the DBE and MCE ground motions

<table>
<thead>
<tr>
<th>Ground motions</th>
<th>Rec 1</th>
<th>Rec 2</th>
<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
<th>Rec 11</th>
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<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
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</thead>
<tbody>
<tr>
<td>DBE</td>
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<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<tr>
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<td>3</td>
<td>3</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

Based on ASCE 41-13, the axial compressive actions in the BRBF columns are categorized as force controlled with a brittle response and thus no nonlinear deformation is allowed for the axial compressive actions in the columns. The flexural actions in the columns, however, can be categorized as deformation-controlled with a ductile response provided that compressive axial loads in the columns are not larger than 50% of the column compressive load capacity ($P_{CL}$).

To evaluate the extent of flexural nonlinear deformations in the beams and columns, the plastic rotation ratio ($\theta_p/\theta_y$) was calculated at the beam and column ends in the BRBF. For this purpose, the yield rotation ($\theta_y$) for beams and columns were calculated using the equation below as proposed by ASCE 41-13:

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
\[ \theta_y = \frac{Z_b F_{ye} l_b}{6E I_b} \]  
(4.5)

\[ \theta_y = \frac{Z_c F_{ye} l_c}{6E I_c} \left( 1 - \frac{P}{P_{ye}} \right) \]  
(4.6)

where \(Z_b, l_b, \) and \(I_b\) are respectively the beam plastic section modulus, length, and moment of inertia. \(Z_c, l_c, \) and \(I_c\) are respectively the column plastic section modulus, length, and moment of inertia. \(F_{ye}\) and \(E\) are respectively the expected yield strength and elastic modulus of the steel material in the columns. Finally, \(P\) and \(P_{ye}\) in Equation (4.6) are the axial force demand and the expected axial yield strength of the columns. The plastic rotation \(\theta_p\) at each end section of the beams and columns was calculated assuming a uniform distribution of plastic curvature \(\phi_p\) over a plastic hinge length \(L_p\) at each end of the beams and columns using the following equation:

\[ \theta_p = L_p \phi_p \]  
(4.7)

where \(L_p\) is assumed to be equal to 90% of the beam or column cross section depth \(d_c\). \(\phi_p\) in Equation (4.7) was measured as the difference between the total curvature \(\phi\) and the yield curvature \(\phi_{ye}\) of the beam or column end section. The yield curvature in turn was calculated based on the yield strain \(\epsilon_{ye}\) of the steel material as \(\phi_{ye} = 2 \epsilon_{ye} / d_c\) assuming a symmetric strain distribution in the cross section. This assumption is accurate enough for beams and columns with small axial force demand to capacity ratio which was the case for all of the beams and most of the columns in the BRBF.

The columns under combined axial tension and bending stress are considered deformation-controlled based on ASCE 41-13. The flexural actions in the beams are also allowed to be considered deformation-controlled based on ASCE 41-13. The maximum of axial compressive demand to capacity \(\left( \frac{P}{P_{CL}} \right)\), the plastic rotation ratio \(\left( \frac{\theta_p}{\theta_y} \right)\), and the combined compressive axial-flexural strength ratio \(\left( \frac{P}{P_{CL}} + \frac{M}{M_{CL}} \right)\) for each of the column end sections were calculated and the average of the results over all of the ground motions were compared to the performance level limits defined in ASCE 41-13. Similar analyses were also performed for the plastic rotation ratio \(\left( \frac{\theta_p}{\theta_y} \right)\) of the beams. The results again indicated that both models 1 and 2 provided similar predictions for the nonlinear axial-flexural deformations in the BRBF beams and columns. Table 4.19 shows the average response of the beams and columns with considerable average flexural-axial nonlinear deformation and force demand. The rest of the BRBF beams and columns experienced minor nonlinear axial-flexural demands which well satisfied both the deformation and strength limits for the LS and CP performance levels under the DBE and MCE ground motions. The results presented in Table 4.19 indicate that the major nonlinear demands on the beams and columns occurred during the MCE ground motions. The maximum plastic deformations of beams and columns occurred in the first floor on the south side (Beam 1S and Column 1S-T). This was mainly due to the rigidity of the beam-column-brace connections. The large nonlinear demands on the top end of the third storey column on the south side of the BRBF (Column 3S-T) was due to the change in the column and BRB capacity, respectively at the BRBF third and fourth storeys. The plastic rotation ratio for Beam 1S satisfies both the LS and CP limits stated in Table 4.11. As can be
seen from Table 4.19 the $P/P_{CL}$ ratio for Column 1S-T and Column 3S-T was slightly greater than 0.5 which marginally did not qualify them to develop nonlinear flexural deformations. On the other hand, the maximum P-M ratios for these columns were larger than 1.0 implying that their combined axial-flexural strength was not sufficient if the columns were considered as force controlled members. However, the average of the maximum $\theta_p/\theta_y$ ratios for these columns were within 0.2-0.4 as indicated in Table 4.19 which reveals that although these columns experienced the largest nonlinear deformations in the BRBF, their nonlinear flexural deformations were still very small and far less than the limits specified for both LS and CP performance levels presented in Table 4.11. Based on the above discussion Column 1S and Column 3S can be considered a marginally sufficient for both LS and CP performance levels.

The maximum shear force demand to capacity ratio in the BRBF beams and columns was 0.3 for both DBE and MCE ground motions. Therefore, no brittle shear failure was expected to occur in the beams and columns. The maximum rotations of the beam-column shear tab connections were also evaluated. The maximum and the average rotations at the beam-column connections over all of the ground motions are presented in Table 4.20. The results show that the average and maximum rotations under the MCE ground motions were around 50% larger than the rotations under the DBE ground motions. The maximum connection rotation was 0.078 rad which occurred at the first floor during the MCE ground motions. This rotation was smaller than the 0.16 rad rotation capacity of the shear tab connections. However, this level of rotation could potentially cause significant damage to the floor concrete slab resulting in some strength degradation in the shear tab connections. It should be noted that the low-cycle fatigue damage was not considered in the connection model. However, it was not expected to cause significant strength degradation in the beam-column connections at the rotation levels experienced during the DBE and MCE ground motions. Figure 4.50 shows model 2 moment-rotation hysteresis for the first floor north beam-column connection under ground motion No. 4 at the DBE and MCE levels. Degradation of the flexural capacity of the connection due to the damages to the floor concrete slab can be observed from this figure.

**Table 4.19:** Average of maximum response of the beams and columns with considerable flexural-axial nonlinear deformation

<table>
<thead>
<tr>
<th>Location</th>
<th>Beam 1S</th>
<th>Column 1S-T</th>
<th>Column 3S-T</th>
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<tbody>
<tr>
<td>Hazard level</td>
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<tr>
<td>MCE</td>
<td>MCE</td>
<td>MCE</td>
<td></td>
</tr>
<tr>
<td>$(P/P_{CL})_{max}$</td>
<td></td>
<td></td>
<td></td>
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<td>-</td>
<td>0.54</td>
<td>0.52</td>
</tr>
<tr>
<td>2</td>
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<td>0.54</td>
<td>0.51</td>
</tr>
<tr>
<td>$(\theta_p/\theta_y)_{max}$</td>
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<td></td>
<td></td>
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<td>2</td>
<td>1.2</td>
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</tr>
<tr>
<td>Max. P-M ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>-</td>
<td>1.06</td>
<td>1.09</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>1.05</td>
<td>1.09</td>
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</tbody>
</table>
Table 4.20: The maximum and the average rotations at the beam-column connections over all of the ground motions

<table>
<thead>
<tr>
<th>Hazard level</th>
<th>Model</th>
<th>Average rotation</th>
<th>Max rotation</th>
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<tr>
<td>MCE</td>
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</tr>
<tr>
<td></td>
<td>2</td>
<td>0.058</td>
<td>0.076</td>
</tr>
</tbody>
</table>

Figure 4.50: Moment-rotation hysteresis for the BRBF first floor shear tab beam-column connection under ground motion No. 4 at the DBE and MCE levels (Model 2)

4.6.2.4 Discussion of the BRBF performance

The BRBF NRHA results were consistent with previous experimental and numerical investigations of the seismic performance of BRBFs reported in the literature that were discussed in Section 4.2. The following is the summary and discussion of the seismic performance of the BRBF based on the numerical analyses presented in this study:

- The BRBF experienced large transient and residual SDR during the DBE and MCE ground motions. The average of peak transient and residual drifts did not satisfy the LS limits under the DBE ground motions. The maximum transient SDR rapidly increased during the MCE ground motions to values above 4%. The average of the peak transient SDR under the MCE ground motions was beyond the 4% limit defined for the CP performance level. The large drifts observed for the BRBF was despite the fact the effect of gravity columns and the stiffness of the connections were explicitly considered in the BRBF model. The large drifts were primarily due to the small post-yield stiffness of the BRBs. The BRBF drift values under DBE ground motions suggest that the building will experience significant damage mainly in the non-structural elements like partitions during a design basis earthquake. The damages may be large enough to endanger the life safety and limit safe evacuation of the occupants. On the other hand, the BRBF may be in a near collapse condition after MCE level earthquakes. Significant damage is expected to occur in the gravity...
load supporting system and the foundations at this stage. It is mostly expected that the building cannot be reused and will need to be demolished after both design basis and maximum considered earthquakes.

- The BRBF floor accelerations were rather large during both DBE and MCE ground motions. However, no considerable acceleration amplification was observed to occur along the BRBF height. The rather large floor accelerations can potentially limit the comfort of the building occupants or cause damage to sensitive structural content during design basis and maximum considered earthquakes.

- Significant axial tension and compression yielding and energy dissipation occurred in the BRBs without any signs of global buckling or failure in the BRB elastic parts or the gusset connections. The contribution of flexural stiffness of the brace connections on the lateral response of the BRBF was negligible and the BRBF response was primarily governed by the axial hysteretic behaviour of the BRBs. The BRB ductility demands were far less than their capacity and they could dissipate the seismic energy without low-cycle fatigue failure. The ductility demands were less than the LS limits under both DBE and MCE ground motions.

- The BRB beams and columns experienced flexural yielding particularly in the first floor level due to the effect of storey drifts and the rigidity of the beam-column-brace connections. Such effects were not considered in the capacity design of the beams and columns. Formation of plastic hinges in the beams and columns significantly increased under the MCE ground motions. However, the axial-flexural demands on the beams and columns did not limit the performance of the BRBF as they were within the LS and CP acceptable limits for both DBE and MCE ground motions. Large rotation demands were observed in the shear tab beam-column connections particularly under the MCE ground motions causing damage to the floor concrete slabs. However, the maximum rotation demands were less than half of the rotation capacity of the shear tab connections and they could withstand BRBF large drifts without any problem.

- There was significant variability in the deformation response of the BRBF especially for the residual drifts with coefficients of variations in the range of 44-134%. In some cases, like the maximum residual SDR under the MCE ground motions, the average response satisfied the acceptance criteria, but the 84-percentile of the response did not fall within the acceptable range. Therefore, the response variability should be evaluated and considered when assessing the BRBF seismic performance based on nonlinear response history analysis results.

- Distribution of storey drifts, BRB energy dissipation, and BRB ductility demands along the BRBF height indicated a significant concentration of damage in the lower storeys of the BRBF, especially in the first storey. This type of behaviour prevents efficient contribution of the strength and ductility capacity of the top storey structural elements in the seismic response of the BRBF and thus is undesirable and should be avoided. It is expected that the BRBF forms a soft-storey mechanism especially as a result of the concentration of damage in the first storey during the MCE ground motions.

- The BRBF models with Steel01 and Steel02 material models generally predicted similar response and performance for the BRBF. The largest deviations were observed for the BRB cumulative ductility demands, floor accelerations, and the residual drifts. The BRBF model with Steel02 material model
predicted 12-65% larger cumulative ductility demands for the BRBs. This model also predicted a more intensified concentration of damage in the bottom storeys. On the other hand, the floor accelerations predicted by the BRBF model with Steel01 material model were on average 15% larger. The above differences were primarily attributed to the smaller post-yield stiffness and the gradual transitions between the elastic and inelastic response that was present in Steel02 material model compared to the Steel01 material model.

4.7 **SUMMARY AND CONCLUSIONS**

In this chapter, the hybrid simulations performed in UT10 on a 5-storey BRBF were presented. The hybrid simulations were the first application of UT10 for seismic performance assessment of braced frames. Three AYB specimens were used to physically represent the BRBs during the hybrid simulations. The results of the hybrid simulations were compared to the results of the BRBF fully numerical models with two different material models for the BRB yielding cores. Both material models were calibrated using the cyclic test results on AYB. The results of this study confirmed the accuracy and performance of UT10 for performing multi-element hybrid simulations on frames with yielding braces. Moreover, the results further elaborated the impact of BRB modelling inaccuracies on the seismic response and performance assessment of BRBFs and showed how hybrid simulation test results can aid in identifying limitations in existing models and enhancing our understanding of the seismic response of SFRSs.

The following is the summary and the conclusions that can be made from the investigations presented in this chapter:

- The review of the past literature on the seismic performance of the BRBFs indicate that that the BRB specimens provide large energy dissipation capacity reliably. Past test results indicate that the response of this system is highly dominated by the response of the BRBs. As a result, due to rather small post-yield stiffness of the BRBs, the BRBF system can show excessive drifts and concentration of damage during large earthquakes. Therefore, the BRBF numerical models should properly account for the realistic hysteretic response of BRBs to ensure realistic seismic performance assessment of BRBFs.

- Previous hybrid simulations on BRBFs were performed on braced frames with a maximum of 4 storeys due to the size of the physical substructure and laboratory resource limitations. Review of the literature reveals that the undesired failure modes including the concentration of the rotations at BRB ends and global buckling of the BRBs can be controlled and their effects on the system-level hysteretic response of the BRBs can be minimized through a capacity design procedure. Therefore, if the undesired BRB failure modes are carefully accounted for and the response of the BRB elastic components are explicitly modelled in the numerical substructure, multi-element hybrid simulations in UT10 on BRBFs with isolated AYB specimens representing a portion of the BRB yielding cores can be well representative of the realistic response of the BRBFs. Such multi-element hybrid simulations facilitate testing BRBFs with a larger number of storeys and physical brace specimens than previously tested and extends the available realistic test data on the seismic performance of the multi-storey BRBFs.
- A 1-element and two 3-element hybrid simulations were performed in UT10 on a 5-storey BRBF under pulse and no-pulse type ground motions. In these tests, AYB specimens were used to physically represent a portion of the BRB yielding core in the first three storeys. Two BRBF models with different levels of beam-column-brace connection modelling details were used as the numerical substructure. Detailed nonlinear buckling analyses were performed on 3D continuum FE models of the BRBF beam-column-brace sub-assemblages. The results indicated that the maximum axial force that can develop in the BRB yielding core did not cause global buckling of the BRB and failure of the gusset plates. Therefore, the axial load capacities of the BRBs were not limited by these undesired failure modes.

- The hybrid simulation results confirmed UT10 accuracy and performance for multi-element hybrid simulations on braced frames with yielding braces. The hybrid simulations indicated that the maximum residual SDR, maximum transient SDR, and maximum displacement ductilities of the BRBs were respectively 1.2%, 2.1%, and 5.3 during the pulse type ground motion. The results increased to 2.1%, 3.4%, and 8.9 during to the no-pulse type ground motion. The BRBF energy input and dissipation was more than 5 times larger during the no-pulse type ground motion compared to the pulse type ground motion. The results revealed a large drift response for the BRBF under both ground motions. Particularly, the results indicated significantly higher levels of damage in the BRBF during the no-pulse type ground motion compared to the pulse type ground motion.

- The hybrid simulation results in terms of storey drifts, floor accelerations, energy dissipation, and the braces axial displacement ductility demands were compared to the fully numerical model predictions for both ground motions. Two different material models including a bilinear model and a Giuffre-Menegotto-Pinto model were calibrated based on the AYB cyclic test results and used in the fully numerical models. The results indicated that both material models can predict the BRB maximum forces and energy dissipation with reasonable accuracy. Similar to the hybrid simulations, both models predicted concentrations of drift and damage in the bottom storeys. However, the bilinear model failed to predict the intensity of the drift concentration in the lower storeys. This happened due to the unrealistically large post-yield stiffness that was used in the calibration of this model which enabled a more uniform distribution of demands across the BRBF height. The stress overshooting in Giuffre-Menegotto-Pinto model was observed to produce unrealistic hardening after slight unloadings during the ground motions which was observed to affect the system-level response of the BRBF. In general, the inaccuracies in the material model was observed to affect the deformation response of the BRBF especially the residual drifts. Giuffre-Menegotto-Pinto model was observed to provide better accuracy than the bilinear model for estimation of the BRB displacement ductility for both ground motions and also for the residual drifts during the pulse-type ground motion. The accuracy of the bilinear model was observed to improve under the no-pulse type ground motion.

- Based on the discussions and observations in this chapter, the differences between the fully numerical and hybrid simulation results were not the result of poor calibration but also due to inherent limitations in the
numerical models which require further improvements to capture the actual hysteretic behaviour of the BRB during the cyclic loads in a ground motion.

- The seismic response and performance of the BRBF were evaluated under a suite of 16 spectrum compatible ground motions at two different hazard levels. The material models for the BRB yielding cores were calibrated to the AYB cyclic test results. The results indicated that the BRBF experienced large storey drifts and failed to satisfy the drift limits for the life safety and collapse prevention performance levels even under the design bases earthquakes. However, the local response of the beams, columns and BRBs were within the acceptable range. Most of the seismic energy was dissipated in the BRBs especially in the first storey of the BRBF. The BRBF showed concentrations of damage and potential for formation of soft storey mechanism in the bottom storeys. The excessive lateral deformation of the BRBF was primarily a result of low post-yield stiffness of the BRBs. Large drifts were observed despite the fact that the stiffening effects of the beam-column-brace connections and the lateral stiffness of the gravity support system were explicitly considered in the numerical analyses. The seismic performance of the building can be potentially improved if the BRBs are used in combination with backup structural systems that are able to provide lateral stiffness for the SFRS after yielding of the BRBs. The latter can be for instance achieved by using the BRBs in a dual system such as a BRBF-MRF.

- The NRHA results revealed that the BRBF models with the bilinear and the Giuffre-Menegotto-Pinto material models generally provided comparable seismic response and performance predictions for the BRBF. This was primarily attributed to the careful calibration of both models to existing experimental results. Following comparison of the results for both material models, if seismic performance assessment of the structure requires accurate prediction of floor accelerations, residual drifts, realistic predictions of damage concentration, and realistic estimates of BRB ductility demands, it is recommended to use models like Giuffre-Menegotto-Pinto material model that can provide realistic post-yield stiffness and gradual transitions between the elastic and inelastic portions of the BRB hysteretic response. Examples of such cases are when the building contents are sensitive to the floor accelerations or when the design of the BRBs requires a realistic assessment of the cumulative ductility demands after multiple consecutive earthquakes.

- It should be noted that the impacts of some of the limitations and inaccuracies of the Giuffre-Menegotto-Pinto material model on the global response of the structure, that were identified by the cyclic tests and the realistic hybrid simulations with two ground motions, were not studied under the 16 ground motions. In addition to the stress overshooting issue that was identified through the hybrid simulation results, the Giuffre-Menegotto-Pinto material model also failed to capture some features of the BRB cyclic response, including the unsymmetrical softening in the tension and compression. It is not clear how such inaccuracies may impact the response predictions for the BRBF under a wide range of ground motions. On the other hand, as discussed before, in some cases the response predictions provided by the bilinear model were closer to the realistic hybrid simulation predictions. Based on the above, a more complete conclusion on the accuracy of the adopted material models and their impact on the seismic performance assessment of the BRBF is possible only when more accurate responses of the BRBF are available under a spectrum of
the ground motions. With the potentials provided by the UT10 and AYB, the latter can be achieved by performing hybrid simulations on the BRBF under all of the 6 ground motions. Alternatively, more accurate material models without the limitations of the Giuffre-Menegotto-Pinto and bilinear models can be identified, using the hybrid simulation results. These models can then be used to produce realistic response predictions of the BRBF under a wider spectrum of ground motions.
CHAPTER 5: MULTI-ELEMENT HYBRID SIMULATION AND SEISMIC PERFORMANCE ASSESSMENT OF A SPECIAL CONCENTRICALLY BRACED FRAME

5.1 INTRODUCTION

In this chapter, the results of investigations on three topics are presented. The first topic is the development and response verification of a buckling specimen with a hysteretic response similar to conventional braces. The buckling specimen was aimed to be used in UT10 hybrid simulations to facilitate experimental seismic performance assessment of SCBFs. The scaling strategy and design details of the buckling specimen are discussed and the results of the cyclic tests on the buckling specimen in UT10 are presented. The accuracy of the scaling strategy is then evaluated by comparing the cyclic response of the buckling specimen with the cyclic response of the 3D continuum FE models of a full-scale design brace.

The second topic is the application and response assessment of UT10 for performing hybrid simulations on SCBFs. For this purpose, 2-element and 4-element hybrid simulations were performed on the SCBF in UT10 with the buckling specimens representing the physical braces. These hybrid tests were the most complex tests done in UT10 based on both the number of physical specimens tested and the nonlinearity and complexity of the hysteretic response of the specimens. The improvements to UT10 after the BRBF tests and the challenges faced during the SCBF hybrid simulations are discussed. The SCBF hybrid simulation results are compared with the fully numerical results and the accuracy of the brace numerical model in seismic performance assessment of the SCBFS is evaluated.

The last topic that is discussed in this chapter is the seismic performance assessment of an SCBF as one of the most popular CBFs for low- and mid-rise buildings. The same building considered for the BRBF was redesigned with an SCBF. The seismic performance of the SCBF is evaluated through nonlinear response history analysis (NRHA) on an advanced nonlinear numerical model of the SCBF in OpenSees with the state-of-the-art brace models. The analysis was performed under 16 ground motions selected and scaled to match the seismic hazard levels corresponding to design basis and maximum considered earthquakes.

This chapter starts with a review of the background on the seismic performance of the SCBF in Section 2. The chapter continues with the design and the cyclic tests on the buckling specimen in Sections 3 and 4. Details of the SCBF design is presented in Section 5. In Section 6, the hysteretic response of a full-scale brace is modelled and the results are compared with the cyclic tests on the buckling specimen. In section 7 the details of the 2-element and 4-element hybrid simulations on the SCBF with buckling specimens are presented. A comprehensive study on the seismic performance of the SCBF based on fully numerical NRHA is presented in section 8. The chapter is
concluded in section 9 with a summary of the results and conclusions from the investigations presented in this chapter.

Some of the tasks associated with the investigations presented in this chapter were done in collaboration with another member of the research group, Mr. Pedram Mortazavi, MASc, P.Eng., who at the time of writing this report was a Ph.D. candidate at the Department of Civil and Mineral Engineering of the University of Toronto. These parts are the preparation of the brace 3D continuum FE model and design of the SCBF connections. He also helped with the implementation of the numerical models for the braces and connections in the SCBF OpenSees model. Pedram has also generously helped with and participated in the UT10 tests in the Structural Testing Facility of the University of Toronto.

5.2 BACKGROUND

5.2.1 Special Concentrically Braced Frames (SCBF)

SCBFs are one of the most popular CBFs. They have special design details that allow them to maximize their nonlinear deformation and storey drift capacity during earthquakes. The intended energy dissipation mechanism for SCBFs is tensile yielding and compressive buckling of the braces. There is also some energy dissipation from the frame action of the beams and columns. The frame action contribution increases during large earthquakes and after complete fracture of the braces.

The braces in an SCBF are allowed to buckle under compressive loads and dissipate energy through the formation of plastic hinges at the mid-span and potentially at both ends of the brace as indicated in Figure 5.1a. Formation of plastic hinges during the buckling of the braces causes local buckling and significant nonlinear strain concentration which results in fracture due to low-cycle fatigue in the plastic hinge zones. Selection of compact sections with low width-to-thickness ratios as recommended by the design codes can control the amount of local buckling and can thus delay the low-cycle fatigue fracture of the braces in SCBFs. Figure 5.1b shows the cyclic hysteretic response of a single brace in an SCBF. As can be seen in this figure, the cyclic hysteresis of the braces is asymmetrical and involves yielding, hardening, a sharp drop of compressive forces after buckling, negative stiffness, and strength degradation. In order to achieve a symmetrical lateral response in a conventional CBF with buckling braces, opposing pairs of braces should be used. In this case, the combined response of the pairs of braces results in a symmetrical hysteretic response with pinching and progressive strength reduction. The brace gusset plates should be designed to reliably accommodate large rotations at the brace ends during the brace buckling. For this purpose, the gusset plate is allowed to form a plastic hinge close to the brace end. Figure 5.2 shows the gusset plate plastic hinge zone according to two different models. \( t_p \) in this figure is the gusset plate thickness.
In an SCBF, the beams, columns, and connections are capacity protected to limit the undesired failure modes and also to allow the braces to develop considerable nonlinear deformation and reach their strengths. The brace end connections in an SCBF are particularly designed to allow large deformations at the brace ends during the brace buckling and to undergo significant nonlinear deformation without any fracture following the formation of plastic hinges. The intended and favourable fracture location in the SCBFs is usually at the mid-span of the braces. The system incurs large deformations and concentration of damage following fracture of the braces. However, it still provides reserved strength from the frame action of the beam and columns and their connections which prevent the complete collapse of the system after the fracture of the braces. The most recent guidelines for the design and analysis of the SCBFs are presented by Sabelli et al. (2013) and ATC (2017b).

5.2.2 Seismic performance of CBFs with conventional braces

The response of the conventional braces and their connections, and the seismic performance of CBFs with conventional braces including steel SCBFs have been the subject of many research projects and experimental programs. Summaries of some these studies are presented and discussed below.
5.2.2.1 Component tests on struts by Black et al. (1980)

Black et al. (1980) did a series of cyclic axial loading tests on twenty-four steel struts with various steel profiles and different sizes representative of what used at the time in real practice in steel buildings. They investigated the effect of cross section shape, boundary conditions, loading type, and slenderness of the struts on their hysteretic responses. They concluded that the slenderness ratio (effective length divided by the radius of gyration of the cross section) of the struts not only controls their elastic response, but also it determines their inelastic hysteretic response. Their investigations revealed that the slenderness ratio of the struts is the single most effective parameter that determines the hysteretic response of the struts. Their tests also showed that the hysteretic performance of the braces is partially affected by the cross section shape of the braces mainly through the development of stress concentrations and local buckling in the brace and also the occurrence of lateral-torsional buckling for some torsional sensitive cross section shapes.

5.2.2.2 Component tests and buckling analysis of HSS and built-up braces by Goel (1992)

A series of experiments were conducted under the direction of Subhash C. Goel during 1980’s at the University of Michigan, USA, to investigate the performance of the bracing members and their connections designed following the code provisions at the time. They performed cyclic tests on compact square and rectangular HSS braces and observed severe damage and early fractures due to local buckling of the compression flanges in these braces. They proposed an empirical equation to predict the fracture life of the braces. In addition, they investigated the effect of concrete infilling on the hysteretic response of the braces and concluded that the concrete infilling can delay the formation of local buckling and can improve the hysteretic response of the braces. Based on experimental and analytical studies on double-angle and double-channel bracing members, they recommended spacing, width-thickness ratios, and configurations for the built-up braces to improve their seismic behaviour. They also verified the performance of single gusset plate connections with a free length equal to twice the plate thickness ($2t_p$) from the end of the brace (see Figure 5.2a) and confirmed that the gusset plate is able to reliably accommodate the rotation demands from the global out-of-plane buckling of the brace through the formation of the plastic hinges. A more detailed review of this research program is presented in Goel (1992).

5.2.2.3 Pseudo-dynamic and shaking table tests on six-storey steel buildings

A US-Japan cooperative research program on earthquake engineering was established in 1980’s which was aimed to improve seismic safety in practice (Hanson and Watabe, 1989). As part of this program a full-scale six-storey steel building was tested using a six degree of freedom pseudo-dynamic testing system in the large-size structure laboratory of the Building Research Institute (BRI) in Japan (Foutch et al., 1987; Yamanouchi et al., 1989). The test building satisfied the 1979 Uniform Building Code of the US and the 1980 Seismic Design Code of Japan. The loading was applied in the direction of the braced bay. The braced bay used a dual SFRS comprised of a moment resisting frame with rectangular ASTM A500 Gr. B HSS braces in stacked chevron configuration. The connection details were not representative of what used in practice i.e. no gusset plates were used in the design and the braces were directly welded to the beams. The effective damping values used for the dynamic analysis were 2% for the...
lowest three modes and 90% for the highest three modes. The test results indicated buckling of some of the braces in the in-plane and some in the out-of-plane directions. The fracture in the braces started at rather small storey drift ratios: 0.7% for the third storey braces and slightly more than 1% for the second storey braces. The local tears prior to fracture of the braces initiated at the corners of the HSS braces due to significant nonlinear strains and also the cold form nature of the steel material. This resulted in stiffness reduction and accumulation of storey deformation and damage in the second and third storeys. The maximum SDR in the second and third storeys reached 1.9% while it did not exceed 1% for the other storeys. It was observed that the vertical flexural deformation of the floor beams due to the difference in the tensile and compressive axial forces of the braces, limited the tensile yielding of the braces. The results also indicated significant shear action in the shear tab beam-column connections. It was concluded that the response of the dual system was governed by the braces in both elastic and inelastic ranges and that the contribution of the moment resisting frame was stable and predictable until the fracture of the braces. This contribution increased from 20% prior to buckling of the braces to 80% after severe buckling of the braces. As part of the same research program, a reduced-scale model of the same building with a scaling factor of 0.3 was tested under 20 ground motions using the shaking table at the University of California at Berkeley (Bertero et al., 1989). The largest ground motion represented the collapse limit state for the building. During this ground motion, the braces in the bottom five stories buckled, one brace in the fifth storey ruptured at the mid-span, and one brace on the fourth storey ruptured at the bottom end of the brace. The SDR reached 1.9% after the braces ruptured which exceeded the 1.5% prescribed by UBC. Severe local buckling in the plastic hinge locations causing brace rupturing was observed. It was therefore recommended to use width-thickness ratios below 18 and limit the slenderness ratio of the braces to avoid early local buckling and rupture in the braces. It was also recommended to have a more thorough investigation of the realistic values for the response modification factors (R) recommended by the codes at the time considering the realistic overstrength capacity of the system.

5.2.2.4 Experimental and analytical investigations at Ecole Polytechnic of Montreal, Canada

Tremblay et al. (2003) performed cyclic quasi-static tests on 14 single bracing and 10 X-bracing specimens at the Structural Engineering Laboratory of Ecole Polytechnic of Montreal, Canada to validate models to predict brace fracture. They also investigated the effective brace slenderness for the X-bracing configuration for design. The brace specimens were made from cold formed rectangular HHS and they were installed in a single-storey single-bay steel frame. The gusset plates were designed to have a free length of twice the gusset plate thickness (2t_p) from the end of the braces to allow rotation of the brace ends and formation of the plastic hinge in the gusset plate (see Figure 5.2a). They observed out-of-plane buckling and formation of plastic hinges at the brace ends in both single- and X-bracing configurations and at the mid-length of the single braces. Rupture of the braces was observed shortly after the formation of local buckling in the plastic hinges. Their test results confirmed that regardless of the brace end conditions and section size, the hysteretic shape of the braces with similar slenderness ratio is nearly identical. It was also shown that the braces with smaller slenderness ratio have more energy dissipation capacity than the braces with larger slenderness ratio. Their investigations revealed that in the X-bracing configuration, the compressive strength of the braces can be calculated with sufficient accuracy assuming a full support by the tension support at the mid-length of the compression brace. Finally, based on the test data, they developed an empirical model to
predict the fracture of the rectangular hollow sections for both single- and X-bracing configurations. Tremblay and Robert (2001) performed NRHA on chevron steel braced frames under 6 ground motions to investigate the stability of these systems. They specifically considered the effect of number of stories and the strength of the floor beams on the performance and stability of the braced frames. They evaluated the performance of the braced frames based on parameters like the storey drift, the axial tension and compression ductility of the braces, formation of plastic hinges and the level of axial load in beams and columns. They observed the concentration of damage and formation of storey mechanism mainly in the first storeys of the frames. The mean plus standard deviation of the maximum storey drifts for most of the frames that did not reach instability was below 2%. Their analysis revealed that if the floor beams in the chevron braced frames are designed to develop the tensile strength of the braces, the system shows better performance and dynamic stability behaviour. Based on these analyses, they prescribed some recommendations for the height limitations and the number of stories for the chevron braced frames with different design scenarios for the floor beams.

5.2.2.5 Component- and system- level analytical and experimental investigations by Uriz and Mahin (2008)

Uriz and Mahin (2008) performed a series of component- and system- level analytical and experimental investigations on conventional braces and braced frames. The primary objective of their investigations was to better understand the behaviour of the braced frames designed following the modern design provisions. They proposed a fibre-based multi-element buckling model with distributed plasticity to numerically simulate the hysteretic response of the conventional braces. They implemented their model in OpenSees. Furthermore, they proposed a phenomenological low-cycle fatigue damage model based on rainflow cycle counting method and Miner’s rule for damage accumulation in the cross section fibres. This model was particularly developed for use with their proposed OpenSees model for braces and was calibrated and verified with the experimental data from cyclic testing of the conventional braces. They conducted cyclic tests on a nearly full-scale two-storey single-bay chevron-braced frame. The braces were made from ASTM A500 Gr. B rectangular HSS and the gusset plates were designed to allow inelastic end rotation and out-of-plane buckling of the braces. The top floor drift ratio was 0.45% when the first storey braces started to buckle. They observed the concentration of frame lateral deformation and damage in the first storey braces after their buckling. The first brace in the first storey started to fracture at an SDR of 1.2% after which the first storey drift ratio increased to around 2.8%. Both braces in the first storey fully ruptured at an SDR of about 2.5% in the subsequent cycle. No sign of buckling was observed in the second storey braces which was due to the concentration of the damage in the first storey. Although the floor beams were capacity protected for the unbalanced forces from the braces in tension and compression, the uncontrolled flexibility of the beams limited the tensile yielding of the braces in the first storey thus limiting the energy dissipation capacity of the system. They observed significant frame action after full fracture of the first storey braces causing yielding in the beam-column connections and fractures in the first storey columns. The results of the system-level tests performed by Uriz and Mahin (2008) confirmed that the system-level response of the braces was generally similar to their component-level response. They also performed numerical simulations of the test specimen and concluded that their proposed fibre-based numerical model for the conventional brace can predict the hysteretic response and fatigue life of the
conventional braces with adequate accuracy. They used their proposed model to perform a comprehensive performance assessment of three- and six-storey special concentrically chevron-braced-frames under ground motions representative of various hazard levels. The average SDR for the three- and six-storey SCBFs under 10% in 50 years hazard level (design-level) were, respectively, 1.6% and 1.1%. Based on their analysis, all the design-level records caused damage concentration in the first storey, none of such records caused brace fracture for the three-storey models, and only 2 records completely fractured a brace in the six-storey models. They concluded that while the performance of the SCBFs is acceptable for low and moderate hazard levels, they have a much wider response and their performance can dramatically drop under more severe hazard levels due to the brace fractures and the resulting sudden loss of strength and damage concentration in the SCBFs under such hazard levels.

5.2.2.6 Experimental and analytical investigations at the University of Washington, USA

An extensive experimental and analytical research project was defined at the University of Washington, USA, to investigate the seismic response and performance of SCBFs. As part of this project a series of cyclic tests were conducted on 38 full-scale one- two-, and three-storey single-bay diagonally and X-braced steel SCBFs (Lehman et al., 2008; Roeder et al., 2011). Various gusset plate configurations, including rectangular and tapered, were used for the gusset plates. Their tests showed that the seismic performance of SCBFs is strongly affected by the performance of the gusset plate connection and extensive nonlinear deformation should be expected in this area. They observed better performance for the gusset plates designed based on an elliptical model to accommodate the nonlinear deformations in the gusset plate compared to the linear model as indicated in Figure 5.2. Further, their tests showed larger inelastic deformation capacity for the wide flange braces compared to HSS braces which in turn increased the deformation demands on the gusset plate connections. As part of the same project, Hsiao et al. (2012) developed a simplified model to more accurately predict the response of the SCBFs especially the nonlinear deformations in the gusset plate connections. They used a rotational spring to model the flexural response of the gusset plates. They verified the accuracy of their model based on the experimental test results and also compared the results to the other connection modelling techniques including the fully fixed and free connection models. The results were used to develop a so-called Balanced Design Procedure (BDP) for the SCBF gusset plate connections which was aimed to improve the ductile yielding, drift capacity, and seismic response of SCBFs. In their proposed design approach, the strength of the yielding mechanisms and failure modes are balanced while the undesired failure modes are suppressed. The BDP is achieved by certain balance factors which are calibrated based on the experimental results. They observed a 46% increase in the drift capacity of the SCBFs designed with rectangular gusset plates following their proposed BDP design concept.

5.2.2.7 Cyclic component-level and pseudo-dynamic system-level tests by Tsai et al. (2013)

As an attempt to investigate the seismic performance of multi-storey SCBFs with an improved design, Tsai et al. (2013) conducted a series of component- and system-level tests and analysis. They specifically investigated the performance of the SCBFs with braces buckling in the in-plane direction with a special knife plate detail for the brace connection. They considered a three-storey office building located in Los Angeles with SCBFs in the perimeter of the building. The braces were installed in a two-storey X-bracing configuration and were designed
from steel wide flange sections with knife plate connection detail. They first conducted a series of component-level cyclic tests on five full-scale braces with various types of knife plate connection details to investigate the component-level performance of each detail. Further, they did cyclic analysis on 2D nonlinear FE models of the SCBF in ABAQUS and OpenSees. The ABAQUS model was created using 2D shell elements while fibre-based elements were used for beams, columns and braces in OpenSees. They observed a good match between the shell and fibre-based models. They also did NRHA on the SCBF model in OpenSees under 20 ground motions representing the design basis seismic hazard level in Los Angeles, USA. Further, they did a pseudo-dynamic hybrid test on a full-scale three-storey single-bay SCBF specimen and compared the results with the numerical analysis predictions. They observed that the OpenSees numerical model can accurately predict the peak experimental responses. However, it could not track the storey residual deformations accurately. The experimental results showed the concentration of damage and storey drift in the first and second storeys. None of the braces fractured during the test and the third storey braces remained elastic. The knife-plate connections showed large inelastic rotation capacity and satisfactory performance throughout the test and the SCBF could reach large SDR values of 4.9% and 4% in the first and second storeys, respectively.

5.2.2.8 E-Defence shaking table test on a single-storey SCBF

There are very few dynamic tests reported in the literature on large-scale SCBFs. One of such tests is the shaking table tests conducted by Okazaki et al. (2012) on a 75% scale single-storey single-bay steel SCBF. The test was conducted on the E-Defence shaking table platform in Japan. Their SCBF was designed following modern design provisions for SCBFs with moment resisting beam-column connections. In their frame specimen, the braces were installed in a chevron configuration and were designed from square HSS. The beam was designed for the unbalanced force induced by the braces in the mid-span of the beam. The gusset plates were designed according to the BDP concept proposed by Lehman et al. (2008) using an elliptical model. The frame was tested under the JR Takatori motion recorded from the 1995 Kobe earthquake which included a large velocity pulse. The ground motion was introduced to the specimen seven times with increasing amplitude levels. The braces buckling and fractures occurred within a few cycles between -1.5% and 1% SDRs. These SDR values were considered low by the authors due to the rather large width-thickness ratio of the HSS sections which was slightly above the code proposed limit for the SCBFs. The drifts increased to a maximum of 2.8% after the braces fractured. The moment action of the frame after the fracture of the braces was significant and was reported to be 80% of the SCBF lateral strength before the fracture of the braces. Although no sign of plastic hinge formation was observed in the floor beam, the braces did not reach their tensile yielding due to the downward deflection of the beam. The braces fractured near the mid-length and the performance of the gusset plates was excellent. They also performed a numerical simulation of the tested SCBF with a fibre-based multi-element model with consideration of the low-cycle fatigue and fracture for the braces and a fibre-based element for the yielding part of the gusset plates. Although the numerical results showed acceptable accuracy for low to moderate amplitude levels of the ground motion, it showed considerable deviations from the experimental results in the largest amplitude ground motion. The authors mainly attributed the deviations to the difficulty and sensitivity of their model to predict fracture in the braces.
5.2.2.9 Collapse analysis and loss estimation of steel buildings with SCBFs by Hwang and Lignos (2017)

Hwang and Lignos (2017) performed a comprehensive collapse risk analysis and loss estimation for steel buildings with SCBFs. In their study, they performed incremental dynamic analysis on advanced 2D numerical models of the steel SCBFs in low- and mid-rise archetype buildings. The SCBFs were designed based on modern design provisions with round and rectangular HSS braces in a two-storey X-bracing configuration. In their numerical models, they used the fibre-based multi-element model for the braces with the rotational spring model for the gusset plates proposed by Hsiao et al. (2012). To predict fracture in the braces, they adopted the low-cycle fatigue modelling recommendations proposed by Karamanci and Lignos (2014). In their models, they explicitly considered the flexural contribution of the gravity columns in the buildings using an equivalent gravity frame. Their analyses revealed that there are some losses in the SCBFs due to the flexural buckling of the braces under frequent and design level earthquakes. However, the losses in the buildings with SCBFs are majorly due to the damage to the acceleration sensitive structural and non-structural elements. Furthermore, their numerical investigations revealed the positive effect of the composite slab action and the gravity framing in controlling storey drift and damage concentrations in SCBFs. They concluded that modelling such effects is necessary to achieve a realistic evaluation of the collapse and demolition losses in steel buildings with SCBFs under large earthquakes.

5.2.3 Numerical modelling of the conventional braces

Various approaches have been developed and adopted in literature and industry to model the axial hysteretic response of the conventional braces. These approaches can be categorized into three main groups: phenomenological model, physics-based model, and continuum FE model (Hsiao, 2012). The phenomenological modelling is based on simplified axial force-axial deformation hysteretic rules that are defined using certain empirical parameters (Nilforoushan, 1973; Jane and Goel, 1978). The physics-based model consists of two elastic elements with a zero-length plastic hinge model. Based on the actual response of the conventional braces, this model assumes concentration of nonlinear deformations in the plastic hinges which are located at the critical locations in the brace. The hysteretic response of the plastic hinges is based on inelastic axial force-rotation relationships that can be calibrated using the material properties and geometry of the braces (Ikeda and Mahin, 1986). The continuum FE model works based on FE principles by numerically solving the solid mechanics equations at certain integration points inside subdomains defined through the meshing of the brace volume and by finding the brace deformations at a grid of nodes. Important aspects including local buckling, stress concentration, low-cycle fatigue, and multi-axial state of stress and strain can be explicitly considered in the continuum FE model and thus the continuum FE model is the most complete and complex approach to model the complex response of the conventional braces (Yoo et al., 2008; Lumpkin et al., 2012). However, it is computationally expensive and its application is mainly limited to component-level brace response investigations in research studies. Although the phenomenological and physics-based models do not offer the accuracy of the continuum FE model, they are much more computationally efficient than the continuum FE model and thus have a wider application.
A fibre-based multi-element model has been implemented, verified, and adopted by researchers (Gunnarsson, 2004; Uriz et al., 2008; Tremblay, 2008; Hsiao et al., 2012; Okazaki et al., 2012; Tsai et al., 2013; and Hwang and not be explicitly captured in this model and thus the fatigue life and fracture of the brace may not be accurately predicted especially for braces with non-compact sections. This uncertainty should be recognized when analyzing the response and performance of the SCBFs using this model (Uriz and Mahin, 2008).

5.2.4 Summary of the findings

Based on the above review of the literature, the following key conclusions can be made:

- There is an extensive amount of experimental investigations in the literature to study the component-level response of the conventional braces and their connections. However, there are limited experimental results on the system-level response of the SCBFs. Most of the existing experimental investigations on the system-level response of the SCBFs with modern designs are limited to 1-3 storey buildings.

- The hysteretic response and low-cycle fatigue life of the conventional braces and their connections control the seismic performance of the conventional CBFs. Therefore, it is important to have an accurate hysteretic and low-cycle fatigue model for the braces and their connections in order to achieve a reliable performance assessment of SCBFs based on numerical simulations. The accuracy of the numerical models should be verified via static and dynamic component- and system-level experiments which further emphasizes the importance of performing experimental-numerical hybrid simulations on SCBFs.

- The hysteretic response of conventional braces is complex and includes tensile yielding, nonlinear compressive buckling, hardening, negative stiffness, and tensile/compressive strength degradation. The elastic and inelastic hysteretic response of the conventional braces is mainly controlled by their effective slenderness. Regardless of the brace end conditions and section size/shape, the hysteretic shape of the braces with similar slenderness ratio is nearly identical.

- Braces with smaller slenderness ratios have more energy dissipation capacity but shorter low-cycle fatigue life than the braces with larger slenderness ratio. Low-cycle fatigue life of the conventional braces is controlled by the local buckling and width-to-thickness ratio of the cross section. Experiments show that cold form HSS sections have a shorter low-cycle fatigue life than other open sections like W sections.

- Seismic performance of SCBFs is strongly affected by the gusset plate connection and extensive nonlinear deformation should be expected in this area. Therefore, it is crucial to design the gusset plates to allow large nonlinear rotations at the brace ends without any fracture in the gusset plate and its welds. Designs based on the linear or elliptical models provide acceptable performance for the gusset plates.

- The braces in SCBFs can buckle in early stages of the response causing damage to structural and non-structural elements in the building.

- In an SCBF the fracture due to low-cycle fatigue initiates at the mid-span of the braces normally in the lower storeys. Brace fractures cause drift and damage concentration and formation of soft storey mechanism.
- The moment contribution of the beam, columns and their connections is increased after the brace fractures which delays the full collapse of the SCBFs.

- The downward deflection of the floor beams in steel chevron braced frames prevents the development of the full tensile strength of the braces thus limiting the energy dissipation capacity of the system even if the floor beams are designed for the unbalanced force from the connecting braces.

- The composite slab action and the gravity framing play an important role in limiting the drifts in SCBFs during large earthquakes.

- Based on the existing experimental and numerical analysis, the performance of SCBFs is acceptable and predictable during low and moderate earthquakes. However, their response is uncertain under larger earthquakes due to the uncertainty in the fracture of the braces. SCBF performance can potentially drop significantly under severe hazard levels due to the brace fractures and the resulting sudden loss of strength and damage concentration in the SCBFs.

- There are various types of strategies in the literature to model the hysteretic response of the conventional braces including rheological, physics-based, fibre-based, and continuum FE models. While the most accurate predictions can be obtained from the continuum FE models, the fibre-based multi-element model seems to model the brace response accurately with rather low computational cost and thus is the recommended model in the latest nonlinear modelling provisions (ATC, 2017).

Significant improvements have been made toward the ductile and more reliable design of the CBFs in the last two decades and various types of modelling approaches were proposed in the literature to numerically simulate the seismic response of these systems. However, very few tests were performed on large-scale multi-storey SCBFs designed according to modern design provisions. In the context of performance-based design and considering the vast popularity of these systems and the rapid increase in their applications, more experimental investigations on multi-storey SCBFs needs to be performed to better understand the actual system-level response, capacity, and performance of these systems in multi-storey buildings and to verify the accuracy of the existing numerical modelling techniques. In addition, it is noted by researchers that compared to other systems like MRFs, the nonlinear dynamic response of short-period braced frames is more sensitive to the hysteretic shape of the yielding elements (Uriz and Mahin, 2008). Based on the above discussions and taking into account the paramount impact of the complex hysteretic response of the conventional braces on the seismic performance of the SCBFs, performing multi-element hybrid simulations on SCBFs with the braces tested as physical specimens is a big step towards achieving realistic understanding from the actual performance of the SCBFs and the accuracy of the existing numerical modelling techniques.
5.3 DESIGN OF THE BUCKLING SPECIMEN

5.3.1 Scaling strategy

The space limitations in UT10 does not allow testing of full-scale conventional brace specimens. Therefore, the brace specimen should be properly scaled in length in order to fit the size limitations of the UT10. A full scaling strategy based on regular similitude laws cannot be used in this case for the following reasons:

- Based on the preliminary design calculations, a minimum length scale factor of 6 was necessary. If the specimen is fully scaled, assuming that material strength is retained during the scaling procedure, the brace axial strength should be scaled by a minimum scale factor of 36 and therefore the axial load strength of the buckling specimen will be very small (55 kN for a brace with 2000 kN axial load strength). Considering the large loading capacity of the UT10 (800 kN-1600 kN), a regular full scaling strategy will result in inefficient use of the loading capacity of the UT10.

- The critical compressive stress and hence the slenderness ratio of the design brace should be retained during the scaling procedure. To achieve this, a full scaling procedure requires that the boundary conditions of the ends of the design brace be fully simulated in UT10. This can be achieved if the gusset plates are also tested with the specimens or if the rotational and lateral deformations of the specimen are controlled by other actuators during the test. Since for the SCBF hybrid simulations the gusset plates were not tested with the brace specimens and the specimens were only axially controlled by a single actuator, the boundary conditions of the buckling specimens could not be fully simulated in UT10 and therefore, a regular full scaling strategy was not feasible.

- Finally, in order to facilitate performing multiple hybrid simulations with multiple elements, the buckling specimen should be simple in design and shape, economically affordable to be fabricated in large quantities, and easily replaceable after the tests which can not be achieved by adopting a regular full scaling strategy.

A special scaling strategy was used to address the above issues. The proposed scaling strategy is based on the assumption that the effective slenderness ratio is the most important parameter on the hysteretic shape and response of the conventional braces. Another assumption is that the local buckling in the brace is minimized or eliminated by choosing a compact cross section and thus the shape of the brace cross section has a negligible impact on the hysteretic response of the brace. As discussed in section 5.2 these assumptions are both supported by several large-scale experiments in the literature (Black et al., 1980; Tremblay et al., 2003). Therefore, in the scaling strategy proposed and adopted in this study, the effective slenderness ratio is retained but it is permitted to change the shape of the cross section. On the other hand, since the effect of the boundary conditions is implicitly considered in the effective slenderness ratio, the boundary conditions of the buckling specimen do not necessarily need to be similar to the design brace. The proposed scaling strategy is illustrated in Figure 5.3. In this strategy, the design brace is first fully scaled down following the regular similitude equations (Harris and Sabnis, 1999) such that the axial strength of the scaled design brace falls within the loading capacity of the UT10 actuators. The scale factor for this scaling level is referred to as $\lambda_1$ in Figure 5.3. In the next step, the scaled design brace is further scaled down with
a scale factor of $\lambda_2$ as indicated in Figure 5.3. This scaling level is only applied on the length of the brace and the cross section area is kept unchanged. As mentioned above, the value of the slenderness ratio should be retained during the scaling. Therefore, the radius of gyration of the buckling specimen cross section and hence its shape should be adjusted to retain the value of the slenderness ratio with the actual effective length of the buckling specimen reflecting the actual boundary conditions of the buckling specimen in UT10. The material properties like the elastic modulus and the yield strength are assumed to be similar in the design brace, scaled design brace, and the buckling specimen. Following the above scaling strategy, to represent the hysteretic response of the design braces, the axial forces and displacements of the buckling specimen should be multiplied by $\lambda_1^2$ and $\lambda_1\lambda_2$, respectively. The accuracy of the above scaling strategy was verified via detailed nonlinear 3D continuum FE analysis presented in the section 5.6.

**Figure 5.3:** Scaling strategy for the buckling specimen

### 5.3.2 Specimen design

Figure 5.4 shows the design details of the buckling specimen. As indicated in this figure, a rectangular cross section (steel plate profile) was selected for the buckling specimen. This decision was made to simplify the design and fabrication of large quantities of the buckling specimen. As indicated in this figure, the buckling specimen connects to a loading shaft and the UT10 base plate, respectively, through the top and bottom connectors. A bearing type bolted connection detail is used to connect the buckling specimen to the top and bottom connectors with 2 sets of 4-7/8 inch high strength bolts at each end of the buckling specimen. Shim plates with 0.25-inch thickness are welded at each end of the buckling specimen with peripheral fillet welds to provide extra strength and prevent failure of the specimen in the connection region. Based on the configuration shown in Figure 5.4 the buckling specimen will be axially loaded via the loading shaft inside UT10 and it will buckle around its weak axis. Based on this design configuration a free length ($L_1$) of 600 mm was chosen for the buckling specimen. The thickness ($t$) of the buckling specimen was selected based on an assumed effective length factor ($K$) for the buckling specimen and an optimized
range for the target slenderness ratio for the design brace in the SCBF. The boundary conditions of the buckling specimen with bolted connections is similar to a fully clamped connection with $K = 0.5$. However, since the connections are bearing, and slight clearance exists between the connectors and the shim plates, the ends of the buckling specimen can have slight restricted rotation and sway and therefore, a larger value of $K = 0.7$ was assumed. This assumption was later verified based on cyclic tests on the buckling specimen as discussed in section 5.4. The optimized range of the slenderness ratio for the design brace was chosen based on the values prescribed by the design standards and the literature for optimum design of SCBFs. The maximum slenderness prescribed by the ANSI/AISC 360-10 (AISC, 2010a) for axially loaded elements is 200. Since the brace profile in an SCBF is selected based on its compressive load capacity, a maximum slenderness of 200 assures that braces with inefficiently large profiles are not selected for the design. On the other hand, a too small slenderness ratio results in design braces with compressive load capacity closer to the tensile load capacity. Although this results in the selection of smaller brace profiles, it causes large nonlinear compressive strains over a large part of the brace section during the buckling of the brace which increases the chances of local buckling and thereby reduces the brace low-cycle fatigue life during an earthquake. This phenomenon was observed and studied by several researchers (Tang and Goel, 1989; Goel and Lee, 1992; Tremblay, 2000; Tremblay, 2008). Following the above considerations and based on the information in the literature on the optimum range of the brace slenderness in SCBFs (Sabelli et al., 2013) a design slenderness ratio in the range of 40-100 was chosen for the design braces in the SCBF and hence for the buckling specimen. Using a value of $K = 0.7$, the thickness of the buckling specimen was chosen to be $t = 15.9 \, mm \ (5/8 \, inch)$. The slenderness ratio of the buckling specimen was then calculated as $K L / t = 0.7 \times 600 \times \sqrt{12} / 15.9 = 92$ which is within the target range for the design brace.

The width ($b$) of the specimen was selected such that the maximum strength of the buckling specimen was below the loading capacity of the UT10 actuators. For this purpose, the actual tensile strength of the buckling specimen material was determined by monotonic tests on tensile coupons. The stress-strain curve for one of the tensile coupon tests is shown in Figure 5.5. As can be seen from this figure, the gradual yielding of the steel material is representative of the response of the cold-rolled sheets used in the fabrication of the buckling specimen. The average ultimate tensile strength ($F_u$) based on the tensile coupon tests was $506 \, MPa$. The average yield strength ($F_y$) calculated based on the 0.2% offset method was $438 \, MPa$, and the average elastic modulus was evaluated as $215.7 \, GPa$. Based on these results the width of the buckling specimen was chosen as $b = 85 \, mm$ based on which the maximum tensile capacity of the buckling specimen without consideration of hardening became $684 \, kN$. Detailed design drawings of the buckling specimen are presented in Appendix C.
Figure 5.4: Design details of the buckling specimen

Figure 5.5: Stress-strain curve of one of the tensile coupon tests on the buckling specimen steel material

5.4 CYCLIC TEST ON THE BUCKLING SPECIMEN

A cyclic test was performed in UT10 on the buckling specimen with the design dimensions $L = 600 \, \text{mm}$, $b = 85 \, \text{mm}$, and $t = 15.9 \, \text{mm}$. Figure 5.6 shows the buckling specimen fabricated for the cyclic test. The cyclic test results were used to assess the cyclic hysteretic response and characteristics of the buckling specimen. The cyclic test results were also used to calibrate the design brace OpenSees model and to verify the accuracy of the scaling factors.
strategy as discussed in sections 5.6. In order to perform the cyclic tests in UT10, similar to the cyclic test on AYB, a hybrid simulation strategy was employed to apply displacements on the buckling specimen. In this strategy, the buckling specimen was modelled by a physical substructure represented by a two dimensional SubStructure element in OpenSees. In this model, all degrees of freedom were restrained at both ends of the element except for the axial movements of one end which was controlled by a cyclic displacement loading protocol through a displacement-controlled integrator.

![Figure 5.6: The buckling specimen fabricated for the cyclic test](image)

### 5.4.1 Loading protocol

The displacement loading protocol is shown in Figure 5.7. As indicated in this figure, the loading started with a small elastic cycle starting from negative (compression) displacements to characterize the elastic mechanical properties and the buckling load of the specimen. The loading was then continued with two cycles of displacements corresponding to 0.5%, 1%, 2%, 3%, and 4% SDR in the first storey of the SCBF. The axial displacements were calculated assuming that the brace free length constituted 70% of the distance between the brace working points. In this calculation, the vertical deflection of the floor beam was neglected.

![Figure 5.7: Displacement loading protocol for the cyclic test on the buckling specimen](image)

### 5.4.2 Instrumentation

Figure 5.8 shows the buckling specimen instrumentation for the cyclic tests. The axial deformation of the buckling specimen was measured by two linear potentiometers ($L_{P1}$ and $L_{P2}$) that were connected at the two sides of the specimen as indicated in Figure 5.8a. The average of the measurements of the two linear potentiometers ($L_{P_{avg}}$) were used as the final axial deformation of the buckling specimen. In order to capture the buckled shape of the...
specimen, measure the lateral deformation of the buckling specimen during buckling, and understand the actual boundary conditions of the buckling specimen, the buckling specimen was instrumented with a three dimensional coordinate measurement system that tracked the 3D movements of 28 infrared light emitting diodes (LEDs) connected along the buckling specimen with a Nikon K610 camera. Figure 5.8a shows the LED instrumentation map. As indicated in this figure, two LEDs (LEDs 1 and 2) were used to capture the movements of the ends of the buckling specimen. The rest of the LEDs (LEDs 3-28) were used to measure the 3D movements of 13 almost equally-spaced points along the buckling specimen length. Two LEDs were used for each point to provide back up measurements since it was expected that one of the LEDs could be covered by the linear potentiometer rod (LP rod) during the specimen buckling and thus would have interrupted readings during the specimen buckling. As can be seen from Figure 5.8a, if the specimen buckles in the positive horizontal direction, LEDs 16-28 can be covered by the LP rod and hence can not be tracked by the camera. In such case, the readings of LEDs 3-15 can be used to measure the deformations.

![LED instrumentation map](image1)

**Figure 5.8:** Buckling specimen instrumentation: (a) instrumentation map and (b) instrumentation as installed on the specimen (front view)

### 5.4.3 Test results

#### 5.4.3.1 Specimen response

Figure 5.9 shows the deformed shape of the buckling specimen under compressive loads during the cyclic test. As indicated in this figure, the buckling specimen buckled around its weak axis. The cyclic hysteretic response of the buckling specimen is shown in Figure 5.10. The cyclic hysteretic response shows slight yielding and hardening on the tension side, nonlinear buckling on the compression side, and an overall pinched hysteretic response which is
very similar to the hysteretic response of the buckling braces in SCBFs. Both tensile and compressive strength degradation can be observed in the hysteretic response of the buckling specimen. The initial critical compressive load and stress of the buckling specimen were 346 kN and 258 MPa, respectively. As can be seen in Figure 5.10, the critical load reduced in the subsequent compressive cycles which was the direct result of the accumulation of imperfections in the buckling specimen during repeated buckling in the compression cycles. Due to limited tensile yielding, these imperfections were not fully straightened under the tensile loads in the subsequent tension cycles. The compressive strength reduction was also due to the elongation of the specimen after yielding in tension which increased the effective length of the brace and thus reduced its critical compressive load. The tensile strength of the buckling specimen slightly increased in the first tension cycle for each displacement level due to the isotropic hardening of the steel material. However, the permanent elongation of the buckling specimen in the first tension cycles caused a reduction of the tensile strength in the subsequent tension cycle under the same displacement level.

Figure 5.9: Deformed shape of the buckling specimen under compressive loads during the cyclic test
Figure 5.10: Cyclic hysteretic response of the buckling specimen

5.4.3.2 Specimen lateral deflection profile and effective slenderness ratio

The 3D deformations of the buckling specimen were measured with the three dimensional coordinate measurement system during the cyclic test. The lateral deflection profiles of the buckling specimen at different phases of the compressive loading is shown in Figure 5.11. The black circles in this figure are LEDs 1-15. The missing circles are the LEDs that were detached during the loading and hence were not properly tracked by the camera. The profiles for the 3% and 4% SDR loading cycles are not shown in Figure 5.11 since many of the LEDs were detached due to significant deformation of the buckling specimen and hence the available measurements were not sufficient to build a complete lateral deflection profile at these loading cycles.

Figure 5.11: Lateral deflection profile of the buckling specimen
Chapter 5: SCBF Seismic Performance Assessment

The kinks in the lateral deflection profile of the buckling specimen at different compressive loading stages in Figure 5.11 clearly indicates the formation of plastic hinges at both ends and particularly at the centre of the specimen after buckling. It can be also seen that the top of the specimen was not fully fixed and moved 2-3 mm laterally during the compressive loading. The shape of the lateral deflection profiles measured by the LEDs is conforming to the actual deflection of the specimen shown in Figure 5.9.

The actual effective length factor of the buckling specimen can be calculated using the specimen’s measured critical stress and the ANSI/AISC 360-10 (AISC, 2010a) equations for flexural buckling of members with no slender elements. Based on these equations, assuming $K = 0.7$:

$$\frac{KL}{r} = \frac{0.7 \times 600 \times \sqrt{12}}{15.9} = 92 \leq 4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{215700}{438}} = 104.5 \quad (5.1)$$

therefore,

$$F_{cr} = [0.658 F_y/F_s] \quad (5.2)$$

where $F_{cr}$ and $F_y$ are the critical stress and yield strength, respectively. $F_e$ is the elastic buckling stress defined as:

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \quad (5.3)$$

Having $F_{cr} = 258 \, MPa$ as measured from the cyclic tests, $F_y = 438 \, MPa$ and $E = 215.7 \, GPa$ as measured from the tensile coupon tests, $L = 600 \, mm$, and $r = 15.9/\sqrt{12} = 4.6$, using equations (5.2) and (5.3) the effective length factor of the buckling specimen is back calculated as $K = 0.596 \approx 0.6$. The actual slenderness of the buckling specimen based on the calculated effective length factor is equal to 79 which again satisfies the condition in equation (5.1). The actual slenderness of the buckling specimen is also within the optimum range of 40-100 for the SCBF as discussed in section 5.3.2. The 0.6 value for $K$ is also consistent with the boundary conditions of the buckling specimen which was slightly more relaxed than a fully clamped boundary condition with $K = 0.5$ as discussed in section 5.3.2.

5.4.4 Advantages and limitations of the buckling specimen

Design of the buckling specimen is very simple and compact. The cyclic test results and the results of the 3D continuum FE simulations presented in section 5.6 indicated that the buckling specimen can accurately simulate the actual complex hysteretic behaviour of a full-scale conventional brace. It should be noted that since plastic hinges form at the ends of the buckling specimen in addition to its mid-length, the hysteretic response of the buckling specimen is considered to also include the effect of the plastic hinges that form in the gusset plates at both ends of the design braces in the SCBF. The buckling specimen can be dimensioned to represent a spectrum of effective slenderness ratio, cross section area, and length for design braces. The bearing type bolted connections at the ends of the buckling specimen enables installation and removal of the buckling specimen with minimum time and effort. These features facilitate performing many multi-element hybrid simulations with physical brace specimens on the
SCBFs which can be used for conducting experimental parametric and sensitivity analysis of the response of the SCBFs.

The cross section shape of the buckling specimen is different from the full-scale design braces. Therefore, the stress distributions and concentrations in the buckling specimen can be different from the design braces particularly during the compressive buckling and post-buckling phases of the response. Consequently, the local buckling and the low-cycle fatigue life of the buckling specimen is not expected to be representative of the design braces. Similarly, although the effect of formation of plastic hinges in the gusset plates is included in the hysteretic response of the buckling specimen, the gusset plate fracture and low-cycle fatigue life cannot be captured from the response of the buckling specimen. These limitations, however, will have a negligible impact on the response of the SCBF if the local buckling of the design braces is eliminated or controlled by selecting compact brace sections and if the SCBF is not tested under very extreme hazard levels causing very large nonlinear deformation demands on the design braces and the gusset plates. Furthermore, if the gusset plates are designed following modern design guidelines to provide reliable strength and ductility capacity for the braces, they are not expected to fracture prior to the braces.

5.5 DESIGN OF THE SCBF

5.5.1 Building layout

The south-north SFRS of the 5-storey building previously studies in Chapter 4 was redesigned with SCBFs following ASCE 7-10 (ASCE, 2010), ANSI/AISC 360-10 (AISC, 2010a), ANSI/AISC 341-10 (AISC, 2010b). In the new design, the design braces were in stacked chevron configuration. Figure 5.12 shows the building plan and the elevation of the SCBFs. Similar to the design of the BRBFs in Chapter 4, two bays of braced frame at each side of the building were considered to provide a better distribution of the building base shear under earthquake loads resulting in a more efficient design for the foundation. Furthermore, this selection increased the redundancy of the building and allowed for the reduction of the design base shear according to ASCE 7-10 (ASCE, 2010). Similar to the BRBF design, the columns were continuous over their entire length with splices at 1.2 m above the third floor as indicated in Figure 5.12b. This continuity helps in redistribution of the inelastic demands along the building height thus reducing chances of soft storey mechanism.
5.5.2 Design of the elements

In order to design the SCBF, the base shear of the building in the south-north direction was determined from a modal response spectrum analysis. Similar to the BRBF design, the target response spectrum considered was the design level seismic response spectrum of Los Angeles with 10% probability of exceedance in 50 years. The design response spectrum is shown in Figure 5.13. The procedure to determine the seismic design forces based on modal response spectrum analysis was similar to the procedure explained in Section 4.3.2 for the BRBF. The first and second modal period of the building with the final design in the south-north direction were 0.55 s and 0.21 s, respectively. The total design base shear of the building in this direction was 3145 kN. The final section sizes of the columns (C) and beams (B) are presented in Table 5.1. The column numbers are based on the numbering used in Figure 5.12.

**Figure 5.13:** 5% damped uniform hazard design response spectrum for Los Angeles with the first and second modal period of the SCBF
Compact round hollow sections with ASTM A500 Gr. B steel material were used as design braces in the SCBF. In order to satisfy the scaling requirements for the hybrid simulations as discussed in section 5.3.1, the round hollow sections in the first two storeys were chosen such that they had nearly the same effective slenderness ratio as the buckling specimen. The value of the effective length factor of the design braces was assumed to be 0.9 in this study for the design of the SCBF to account for the flexibility of the gusset plate connections. This assumption was verified via detailed 3D continuum FE analysis of the first storey design brace and gusset plates as discussed in section 5.6. The design information for the final selection of the design braces is presented in Table 5.2. As can be seen from this table, the slenderness ratio of the design braces in the first and second storeys were, respectively, 77 and 75 which were very close to the slenderness ratio of the buckling specimen (79). It should be noted that in the calculation of the slenderness ratio of the design braces, the free length of the brace included the plastic hinges at the ends of the braces that were expected to form during the buckling of the design braces. For this purpose, the plastic hinge length was assumed to be twice the thickness of the gusset plate following the linear model for the plastic hinge zone (see Figure 5.2a). To satisfy the scaling requirements, it was assumed that the design braces had similar material properties as the buckling specimen. Therefore, the yield strength and elastic modulus of the steel material used in the calculation of the design compressive strength ($P_y$), expected tensile strength ($T_{pr}$), expected compressive strength ($C_{pr}$), and expected post-buckling strength ($C_{pr}'$) of the design braces were $F_y = 438$ MPa and $E = 215.7$ GPa which were obtained from tensile coupon tests on the buckling specimen material. The value of 438 MPa for the yield strength of the design brace was very close to the expected yield strength of ASTM A500 Gr. B ($R_yF_y = 1.4 \times 315 = 441$ MPa) as prescribed by ANSI/AISC 341-10 (ref). $P_u$ in Table 5.2 is the factored required compressive strength for one brace.

### Table 5.1: The SCBF final columns and beams section sizes

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>Columns</th>
<th>Beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C1, C2, C3, C4</td>
<td>Gravity B1-2, B3-4</td>
</tr>
<tr>
<td>1</td>
<td>W310x158</td>
<td>W310x67</td>
</tr>
<tr>
<td>2</td>
<td>W310x158</td>
<td>W310x67</td>
</tr>
<tr>
<td>3</td>
<td>W310x158 (bottom)</td>
<td>W310x67 (bottom)</td>
</tr>
<tr>
<td>4</td>
<td>W200x86</td>
<td>W200x42</td>
</tr>
<tr>
<td>5</td>
<td>W200x86</td>
<td>W200x42</td>
</tr>
</tbody>
</table>

### Table 5.2: Design information for the design braces of the SCBF

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>Section size</th>
<th>$KL/r$</th>
<th>$P_u(kN)$</th>
<th>$P_{pr}(kN)$</th>
<th>$C_{pr}(kN)$</th>
<th>$T_{pr}(kN)$</th>
<th>$C_{pr}'(kN)$</th>
<th>$P_u/P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HSS 141x9.5</td>
<td>77</td>
<td>686</td>
<td>851</td>
<td>1078</td>
<td>1568</td>
<td>323</td>
<td>0.81</td>
</tr>
<tr>
<td>2</td>
<td>HSS 141x9.5</td>
<td>75</td>
<td>633</td>
<td>868</td>
<td>1099</td>
<td>1568</td>
<td>330</td>
<td>0.73</td>
</tr>
<tr>
<td>3</td>
<td>HSS 127x9.5</td>
<td>84</td>
<td>547</td>
<td>682</td>
<td>864</td>
<td>1397</td>
<td>259</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>HSS 127x6.4</td>
<td>83</td>
<td>428</td>
<td>477</td>
<td>604</td>
<td>955</td>
<td>181</td>
<td>0.9</td>
</tr>
<tr>
<td>5</td>
<td>HSS 102x6.4</td>
<td>105</td>
<td>217</td>
<td>264</td>
<td>335</td>
<td>753</td>
<td>100</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
The SCBF connections were fully designed to obtain the realistic connection details and dimensions required for the numerical modelling of the SCBF and also to obtain the realistic free length of the braces for design purposes. Figure 5.14 shows details of the beam-column-brace connection adopted in the design of the SCBF. As can be seen from this figure, stub beam connections with shear tab plates were used for the beam-column connections. This configuration is allowed in ANSI/AISC 341-10 (AISC, 2010b) as a beam-column connection for braced frames and provides significant rotation capacity outside the gusset plates. In this configuration, the shear tab is welded to the stub beam at one end and bolted to the floor beam at the other end as indicated in Figure 5.14. The gusset plates were designed to provide rotation capacity for the out-of-plane buckling of the braces following a linear model for the gusset plate plastic hinge zone (see Figure 5.2a). In order to improve the performance of the gusset plates in the brace connections, the gusset plates were designed with free-edge stiffener plates as shown in Figure 5.14. In order to connect the round hollow braces to the gusset plates, the hollow brace sections were slotted and welded to the gusset plates. This configuration facilitates the out-of-plane flexural buckling of the braces.

![Figure 5.14: Details of the SCBF beam-column-brace connection](image)

5.6 NUMERICAL MODELLING OF THE DESIGN BRACES

5.6.1 Continuum FE model

In order to verify the accuracy of the scaling strategy, a 3D continuum FE model of the first storey design brace and gusset plate connections was constructed and the cyclic axial force-deformation hysteretic response of the 3D continuum FE model was compared to the scaled-up responses of the buckling specimen.
5.6.1.1 Model configuration

The 3D continuum FE model geometry was created with the exact design dimensions of the design brace and the gusset plate. The 3D continuum FE model geometry and mesh is shown in Figure 5.15a. The stress-strain results from the monotonic tensile coupon tests on the buckling specimen steel material were directly used to create the material model for the 3D continuum FE analysis. However, since the cyclic response of the material was not available, hardening of the steel material was not modelled in the 3D continuum FE model. The low-cycle fatigue damage was also not considered in the 3D continuum FE model. In order to model the brace imperfection, an intentional camber with a maximum amplitude of 0.1% of the total length of the brace (end to end length of the brace excluding the gusset plate) was created in the model geometry. This amount of imperfection is within the range of 0.05-0.1% for conventional braces as recommended by Uriz et al. (2008) which is also used by others in the literature (Tremblay, 2008; Okazaki et al., 2012; Tsai et al., 2013). All degrees of freedom were fixed at the gusset plate end at one end of the brace. The movements of the nodes at the gusset plate end at the other end of brace were all constrained to the movements of a single master node. The 3D continuum FE model was loaded in a displacement-controlled manner by moving the master node in the brace axial direction. The movement of the master node was according to the displacement loading protocol used for the buckling specimen with the values of displacements scaled up using a proper displacement scale factor. The force and displacement scale factors were determined based on the dimensions of the first storey design brace and gusset plates and the dimensions of the buckling specimen following the scaling strategy discussed in section 5.3.1. The force and displacement scale factors for the first storey design brace were 2.7 and 6.7, respectively.

5.6.1.2 Results

Figure 5.15b shows the deformed shape and the Von Mises stress field after the brace buckling in the first compression cycle. Formation of plastic hinges can be clearly observed at the mid-span of the brace and within the gusset plate in this Figure which confirms that the brace and the gusset plates behaved as expected. No concentration of stress is observed in the brace cross section which suggests no potential for formation of local buckling for the range of displacements considered in this study. In addition, the compactness of the brace section eliminated the chances of formation of local buckling. In general braces with circular cross sections develop less stress concentration during brace global buckling compared to other shapes like rectangular hollow sections which reduces the chances of local buckling and increases the low-cycle fatigue life of round hollow sections compared to other shapes (Black et al., 1980; Fell et al, 2009). Figure 5.16 shows the cyclic response of the design brace 3D continuum FE model compared to the experimental response of the buckling specimen. The response of the buckling specimen is scaled up using the displacement and force scale factors. The SDR values shown in this figure were recalculated based on the actual free length of the design braces and the actual displacement scale factors obtained following the full design of the SCBF. Therefore, they are slightly larger than the SDR values considered for the cyclic testing of the buckling specimen. Comparison of the responses indicates an excellent match between the 3D continuum FE model results and the experimental results thus confirming the accuracy of the scaling strategy adopted for the buckling specimen. It should be noted that in this study the low-cycle fatigue was not modelled in the 3D continuum
FE model and thus the model did not capture damage and fracture due to low-cycle fatigue of the material. The effect of residual stresses in the brace cross section was also not considered in the model. The residual stresses are expected to slightly reduce the compressive strength of the design brace.

Figure 5.15: The 3D continuum FE model for the first storey design brace and gusset plate connections: (a) model geometry and mesh (front view) and (b) deformed shape and Von Mises stress field after brace buckling (top side view) (Figure prepared by Pedram Mortazavi, PhD Candidate at the Department of Civil and Mineral Engineering, University of Toronto)

The values of $T_{pr} C_{pr}$, and $C_{pr}'$ obtained from the cyclic test on the buckling specimen and the predictions based on the AISC 341-10 equations, the 3D continuum FE model, and the calibrated OpenSees brace model (discussed in section 5.6.2) for the first storey braces are presented in Table 5.3. The buckling specimen results are scaled up using the force and displacement scale factors. In Table 5.3, $C_{pr}$ is the compressive force at the first buckling of the brace. The values of $C_{pr}'$ for the cyclic tests and the numerical models were obtained by averaging the buckling load over all cycles until the end of the 3% drift cycle excluding the first buckling load ($C_{pr}$). Finally the values of $T_{pr}$ were considered as the maximum tensile force over all cycles until the end of the 3% drift cycle. The values of $T_{pr}$, $C_{pr}$, and $C_{pr}'$ obtained from the 3D continuum FE model results presented in Table 5.3 are very close to the cyclic test results. The maximum error is 1% for $C_{pr}$ and $T_{pr}$ and 6% for $C_{pr}'$. The results presented in Table 5.3 also indicate that the experimental values for $T_{pr}$ and $C_{pr}'$ are respectively 8% and 89% larger than the AISC 341-10 predictions while the experimental values for $C_{pr}$ is 14% smaller than the AISC 341-10 predictions. The experimental results reveal that while the accuracy of the code predictions are fairly good for the expected tensile and compressive strengths of the brace, the code predictions are rather poor for the expected post-buckling strength of the brace.
Figure 5.16: Cyclic response of the design brace 3D continuum FE model compared to the scaled-up response of the buckling specimen: (a) all cycles, (b) the first two cycles, (c) 1.1% drift cycles, (d) 2.2% drift cycles

The design brace effective length factor ($K$) was evaluated based on the 3D continuum FE model predictions for $C_{pr}$ and the ANSI/AISC 360-10 (AISC, 2010a) equations for flexural buckling of members with no slender elements. The procedure was similar to the procedure explained for the buckling specimen in section 5.4.3.2. The resulting value for the effective length factor was $K = 0.92$ which was very close to 0.9 value assumed in section 5.5 for the design of the SCBF.
<table>
<thead>
<tr>
<th>Response type</th>
<th>AISC 341-10</th>
<th>Cyclic test (scaled up)</th>
<th>3D Continuum FE model</th>
<th>2D OpenSees model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{pr} (kN)$</td>
<td>1568</td>
<td>1692</td>
<td>1703</td>
<td>1763</td>
</tr>
<tr>
<td>$C_{pr} (kN)$</td>
<td>1078</td>
<td>924</td>
<td>936</td>
<td>1232</td>
</tr>
<tr>
<td>$C'_{pr} (kN)$</td>
<td>323</td>
<td>611</td>
<td>648</td>
<td>740</td>
</tr>
</tbody>
</table>

### 5.6.2 Fibre-based multi-element model

A Fibre-based multi-element model was adopted to model the braces that were not tested physically in the hybrid simulations presented in see section 5.7 and also in the numerical performance assessment of the SCBF presented in section 5.8. Therefore, a 2D fibre-based multi-element model of the SCBF first storey design brace and gusset plates was created in OpenSees. The model was then calibrated based on the buckling specimen cyclic experimental results and the accuracy of the model in simulating the response of the buckling specimen and thus the design brace was evaluated.

#### 5.6.2.1 Model details

The design brace 2D model was created in OpenSees. Figure 5.17 shows the details of the model. As indicated in this figure, the model was fully restrained at both ends except for the horizontal (axial) movements of the right end which was controlled in a displacement-controlled manner. The loading protocol was similar to the loading protocol of the cyclic tests on the buckling specimen and the 3D continuum FE model.

##### 5.6.2.1.1 Gusset plates

The gusset plates were modelled by rigid elements. The lengths of the rigid elements were determined following recommendations of Hsiao et al. (2012). The nonlinear flexural response of each of the gusset plates in the plastic hinge region was modelled by a nonlinear force-based beam-column element (forceBeamColumn element in OpenSees) with fibre cross sections and with distributed plasticity model and material properties similar to the buckling brace. As can be seen in Figure 5.17, the nonlinear element connected the end of the brace to the end of the gusset plate rigid elements. This element simulated the out-of-plane flexural behaviour of the gusset plate in the plastic hinge region. Therefore, following a linear model for the plastic hinge zone in the gusset plate, the length of the element was equal to two times the gusset plate thickness, the thickness of the element was equal to the thickness of the gusset plate, and the width of the element was equal to the Whitmore width defined based on a 45 degree projection angle from the end of the braces. The element cross section was oriented such that the element bent around its weak flexural axis in the SCBF 2D model. This modelling approach was previously adopted and experimentally verified by Uriz and Mahin (2008). Hsia et al. (2012) proposed using a rotational spring with a rheological backbone curve instead of the force-based element. However, Terzic (2013) showed that both approaches provide a similar level of accuracy.
### 5.6.2.1.2 Design brace

The design brace was modelled in OpenSees with ten force-based beam-column elements (forceBeamColumn element in OpenSees) with fibre sections and distributed plasticity model with nodes defined over a parabolic initial camber. Figure 5.17 shows the details of the design brace model. Figure 5.17a shows the brace 2D model and Figure 5.17b indicates the fibre discretization adopted for the brace cross section. A single corotational truss element with infinitesimal cross section area was also placed between the gusset plate rigid element ends parallel to the multi-element brace model. This element improved the convergence of the numerical integration for the brace model. Similar to the 3D continuum FE model, a maximum imperfection equal to 0.1% of the total length of the brace (end to end length of the brace excluding the gusset plate) was considered to build the initial camber. The region of the brace welded to the gusset plate was modelled as bare brace without consideration of the gusset plate stiffening in this region. Giuffre-Menegotto-Pinto (Steel02 material model in OpenSees) uniaxial stress-strain relationship with kinematic and isotropic strain hardening was used as the nonlinear material for the fibre sections. The material model parameters were selected within the calibration process discussed in the next section. The effect of the residual stresses was not considered in the brace model. In the brace model, Gauss-Lobatto (Bathe, 1996) numerical integration method with 5 integration points over each element was used. In this integration method, two of the integration points were located at the ends of each element thus enabling extracting analysis results at the ends as well as at the mid-span of the braces. The fibre discretization for the round hollow sections of the design brace consisted of 60 and 10 subdivisions, respectively, in the circumference and radial directions as indicated in Figure 5.17b. A fine discretization was adopted following the recommendations of Uriz and Mahin (2008) in order to more accurately capture the distribution of the inelastic deformations in the brace which in turn resulted in a better estimation of the brace low-cycle fatigue life and fracture.

![Figure 5.17: Details of the design brace 2D OpenSees model: (a) brace model and (b) brace cross section discretization](image-url)
5.6.2.1.3 Low-cycle fatigue model

Low-cycle fatigue in structural engineering is classically described by a linear log-log relationship between the number of constant amplitude cycles to failure of the material and the strain amplitude experienced in each cycle. This relationship is called the Coffin-Manson relationship (Fisher et al., 1997). This relationship is characterized by two empirical parameters, \( \varepsilon_0 \) and \( m \). Uriz and Mahin (2008) developed an OpenSees fatigue model and they calibrated and verified their proposed model with experimental data from cyclic testing of the conventional braces. Their model is based on a modified rainflow cycle counting method and Miner’s rule for damage accumulation in the cross section fibres. The Coffin-Manson parameters for the low-cycle fatigue model used in this study were conservatively chosen as the values proposed by Uriz and Mahin (2008) for cold form steel rectangular HSS sections for braces that are not reinforced at their connection net area. These parameters are \( \varepsilon_0 = 0.091 \) and \( m = -0.458 \).

5.6.2.2 Model calibration and cyclic response

The yield strength and elastic modulus of the Steel02 material model in OpenSees were, respectively, \( F_y = 438 \text{ MPa} \) and \( E = 215.7 \text{ GPa} \) which were the values used in the design of the braces in the SCBF and obtained from the tensile coupon tests on the buckling specimen steel material. The strain hardening ratio used was 1% which was consistent with the ATC (2017b) recommendations for nonlinear modelling of the conventional braces. The rest of the material parameters including the parameters for isotropic hardening and the Bauschinger effect were calibrated based on the results of the cyclic tests on the buckling specimen.

The analysis was performed once with the low-cycle fatigue model and once without it assuming infinite ductility capacity for the brace material. Figures 5.18 shows the cyclic hysteretic response of the buckling specimen and the OpenSees design brace model without consideration of the low-cycle fatigue. The response of the buckling specimen in this figure is scaled up with a displacement scale factor of 6.7 and a force scale factor of 2.7. The buckling specimen was assumed to replicate the combined response of a design brace and the plastic hinge area within the gusset plates at both ends of the design brace. As such, the axial displacements for the OpenSees model in Figures 5.18 and 5.19 were measured from the ends of the gusset plate rigid elements in the model to include the axial deformations of the plastic hinge area. The values of \( T_{pr}, C_{pr}, \) and \( C'_{pr} \) obtained from the cyclic repose of the OpenSees model are presented in Table 5.3.

It can be observed from Figure 5.18 that the OpenSees model closely tracked the experimental response for all displacement cycles. The model captured the inelastic buckling, post-buckling response, tensile yielding, and strength degradation of the brace with satisfactory accuracy. This not only confirmed the performance of the OpenSees brace model to predict the complex cyclic response of the conventional braces, but also it confirmed the accuracy of the scaling strategy adopted for the buckling specimen. Some deviations were also observed between the experimental response and the OpenSees model results, particularly in the initial and residual compressive strengths. Based on Figure 5.18 and the values presented in Table 5.3, the OpenSees model provided accurate predictions for the value of \( T_{pr} \) which was only 4% larger than the buckling specimen cyclic test. However, the OpenSees predictions for \( C_{pr} \) and \( C'_{pr} \) were 33% and 21% larger than the cyclic test results. On the other hand, as can be seen in Figure 5.18, the OpenSees model predicted a sharp drop of axial force after brace buckling while the
experimental results showed a more gradual change. Similar observations was made by Uriz et al. (2008). The differences between the OpenSees model response and the cyclic test and 3D continuum FE model responses can be attributed to the effect of multi-axial stress and strain which was not considered in the OpenSees model. Similar to the 3D continuum FE model, the larger $C_{pr}$ and $C_{pr}'$ values in the OpenSees model can also be attributed to the effect of the residual stresses which was not considered in both the OpenSees and 3D continuum FE models but existed in the buckling specimen and the design brace. It should be noted that the $C_{pr}$ value is very sensitive to the initial imperfections and state of initial stresses in the brace.

![Graphs showing cyclic response comparison](image)

**Figure 5.18:** Cyclic response of the infinitely ductile (no fatigue) design brace OpenSees model compared to the scaled-up response of the buckling specimen. (a) all cycles, (b) the first two cycles, (c) 1.1% drift cycles, (d) 2.2% drift cycles.

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
In addition, as can be seen from Figure 5.18(a), the model predicted steadily increasing and larger tensile strength for the brace in the last two cycles (3.3% and 4.5% drifts) compared to the experiment. This was potentially due to the fact that compared to the buckling specimen, the OpenSees brace model recovered the imperfections from the previous compressive cycle and thus developed tensile strength of the brace with a faster rate. It can also be due to the accumulation of low-cycle fatigue damage in the buckling specimen which was not captured by the model.

Figure 5.19 shows the cyclic hysteretic response of the buckling specimen and the OpenSees model with consideration of the low-cycle fatigue. As can be seen from Figure 5.19, unlike the buckling specimen, the OpenSees model predicted initiation of damage and fracture during the cyclic test. This observation is consistent with the fact that the brace fatigue model was calibrated for cold form rectangular HSS which has lower fatigue life compared to the buckling specimen that has a solid rectangular cross section. However, as previously discussed, since the cross section shapes of the design brace and the buckling specimen were different, the fatigue life of the buckling specimen was not representative of the fatigue life of the design brace. The OpenSees model predicted fracture during the second 2.2% drift cycle at the mid-span of the brace model. This is consistent with previous cyclic test results on HSS braces reported in the literature which show brace fracture for SDR ranges of 1.5-3% (Lee, 1987; Tremblay et al., 2003; Celik et al., 2004; Uriz and Mahin, 2008; Lehman et al., 2008). Multiple full displacement cycles in the cyclic test impose a large nonlinear demand on the brace which does not necessarily occur during a ground motion. Therefore, the 2.2% drift limit for the brace fracture is considered as a conservative estimate for the response of the brace under ground motions.

Figure 5.19: Cyclic response of the design brace OpenSees model with low-cycle fatigue compared to the scaled-up response of the buckling specimen. (a) all cycles, (b) 2.2% drift cycles

Lateral deflection profiles of the design brace OpenSees model and the buckling specimen at the instant of buckling and in the first compressive cycles of the 0.5%, 1%, and 2% drift cycles are shown in Figure 5.20. The circles on the buckling specimen profiles are the location of the LEDs and the circles on the model profiles are the element nodes. The deflections of the scaled brace are scaled up with the displacement scale factor. It can be
observed from Figure 5.20 that the lateral deflection profiles of the design brace model and the buckling specimen were very similar which confirmed the accuracy of the scaling strategy. The differences in the boundary conditions of the design brace model and the buckling specimen can be clearly seen in the deflection profiles. The ends of the buckling specimen were more rotationally restrained compared to the design brace model. The design brace model maximum lateral deflection at the mid-span was 25% smaller than the buckling specimen at the instant of buckling. This difference reduced to 14%, 9%, and 7%, respectively, in the 0.5%, 1%, and 2% drift cycles. Deviations in the deflection profiles were mainly due to the differences in the boundary conditions of the buckling specimen and the design brace model. The plastic hinges were formed in the design brace model and the buckling specimen at higher response levels and the lateral deflections were dominated by the plastic rotations at the plastic hinges. This reduced the effect of the boundary conditions and thus the lateral deflection deviations between the design brace and the buckling specimen decreased at the higher response levels.

Figure 5.20: The lateral deflection of the design brace OpenSees model (M) and the buckling specimen (S) (scaled up)

5.7 HYBRID SIMULATIONS

Three hybrid simulations were performed on the SCBF. The first hybrid simulation was performed with two physical specimens representing the two braces in the first storey of the SCBF. This simulation is hereafter referred to as 2E-SCBF-HS or the 2-element SCBF test. The second and third hybrid simulations were performed with four physical specimens representing the four braces in the first two stories of the SCBF. These simulations are hereafter referred to as 4E-SCBF-HS1 and 4E-SCBF-HS2 or more generally the 4-element SCBF tests. The main purpose of the hybrid simulations was to verify the performance of UT10 when testing multiple specimens with complex buckling response and to assess the accuracy of the adopted numerical models for the conventional braces in the SCBF.
5.7.1 Specimens

Ten buckling specimens were fabricated and used as the physical specimens in the three hybrid simulations. Similar to the cyclic tests, the axial deformations of each of the buckling specimens were measured using two linear potentiometers as described in section 5.4. The displacement demands received in NICON-10 from the numerical model and the displacement and force feedback of the buckling specimens that were sent back from NICON-10 to the numerical model were properly scaled using the displacement and force scale factors. The displacement scale factors for the first and second storey design braces were 6.7 and 6.5 respectively. The slight difference between the two scale factors was due to the longer length of the first storey design braces compared to the second storey design braces. The force scale factor was 2.7 for both the first and second storey design braces. The force scale factor was similar for the first and second storeys since the cross section area of the design braces were similar in both storeys.

5.7.2 Numerical model

OpenSees was used as the integration module for the hybrid simulation. Therefore, a numerical model of the SCBF was created in OpenSees. Since the building plan was symmetric, the torsional effects and the effects of the out-of-plane structural members was neglected and hence it was sufficient to model a single SCBF bay. Figure 5.21 shows an overview of the SCBF numerical model adopted for the 4E-SCBF-HS1 and 4E-SCBF-HS2. A similar model was also used for the 2E-SCBF-HS but only with the braces in the first storey tested physically. The type of elements used to model the leaning column, columns, and floor beams and the modelling details on how the gravity and lateral loads were applied to the SCBF model were similar to the BRBF model presented in section 4.4. The SCBF floor beams were modelled with two elements extending between the rigid offset elements as indicated in Figure 5.21. Therefore, the floor beams were allowed to form a plastic hinge at their intersection with the braces gusset plates at the mid-span of the beams.

Figure 5.21: SCBF model, SubStructure element, and the control nodes for 4E-SCBF-HS1 and 4E-SCBF-HS2
5.7.2.1 Connections

Buckling and fracture of the conventional braces significantly increases the contribution of the moment action of the frame and the flexural response of the connections in an SCBF (Foutch et al., 1987; Yamanouchi et al., 1989; Roeder et al., 2011; Hsiao et al., 2012; Sabelli, 2013). On the other hand, the compressive strength of the braces is highly affected by their boundary conditions induced by their end connections. A pinned connection model results in an underestimation of the compressive strength of the conventional braces and overestimation of the lateral deformations of the SCBF. On the other hand, a fully fixed model results in an overestimation of the compressive strength of the braces and underestimation of the SCBF lateral drifts (Hsiao et al., 2012; ATC, 2017). Based on the above, the beam-column and brace connection modelling is important for a realistic evaluation of the nonlinear response of the SCBFs especially under large earthquakes (ATC, 2017).

In this study, the shear tab connection between the stub beam and the floor beam was modelled with a zero-length rotational spring with the Hysteretic material model in OpenSees, similar to the BRBF model. The rotational spring was connected between the ends of the floor beam element and the rigid element representing the rigid part of the gusset plate-stub beam section as indicated in Figure 5.21. The rotational stiffness and strength of the model were calibrated based on the approach proposed by Liu and Astaneh-Asl (2004) considering the effect of the floor concrete slabs. Figure 5.22 shows the moment-rotation envelope and cyclic response of the shear tab connection model. Low-cycle fatigue damage was not considered in this connection model. The rotational capacity of the connection was evaluated as 0.16 rad. The panel zone shear deformations were not considered in the SCBF model. However, they were expected to have minor impact on the performance prediction of the SCBFs. The connection of the columns to the ground were modelled as fully pinned for the hybrid tests and the fully numerical analyses presented in the hybrid simulation section (Section 5.7).

![Figure 5.22: Envelope and cyclic moment-rotation response of the shear tab connection model](image-url)

The brace gusset plate connections were modelled following the same approach explained in section 5.6.2 for the 2D OpenSees model and based on the gusset plate design dimensions for each storey. Details of the gusset plate models are illustrated in Figure 5.21.

5.7.2.2 Braces

The braces that were not tested physically were modelled following the same approach that was adopted in section 5.6.2 for the 2D OpenSees model. Therefore, the fibre-based multi-element model was adopted to model the braces.
in the top four stories in the 2E-SCBF-HS and the braces in the top three stories in the 4E-SCBF-HS1 and 4E-SCBF-HS2. The material model parameters and imperfection ratio used for the braces in all storeys were similar to the calibrated values obtained in section 5.6.2. The cross section dimensions of the brace models were chosen based on the actual brace design dimensions for each storey. Details of the brace models are illustrated in Figure 5.21.

5.7.2.3 SubStructure element

A SubStructure element was used in the hybrid simulations to establish the communication between the numerical model and the buckling specimens. Similar to the cyclic tests, each of the buckling specimens physically represented a single brace and the gusset plate plastic hinge areas at both ends of the brace. Therefore, the SubStructure control nodes were located at the ends of the gusset plate rigid elements in the SCBF model. For 2E-SCBF-HS, the SubStructure element represented the two braces (and their end gusset plate plastic hinges) in the first storey of the SCBF model and in 4E-SCBF-HS1 and 4E-SCBF-HS2 the SubStructure element represented the four braces (and their end gusset plate plastic hinges) in the first two stories of the SCBF model. Figure 5.21 shows the SubStructure element and the control nodes for the 4-element SCBF tests. Similar to the BRBF tests, the initial stiffness of the SubStructure was prescribed to be 10% larger than the analytical stiffness that was calculated based on the cross section and material properties of the design braces.

5.7.3 Numerical integration scheme

Preliminary studies on the SCBF response indicated that although Alpha-OS method was stable and accurate for the BRBF hybrid simulations, it was unstable and could not be used for the hybrid simulations on the SCBF which involved a larger amount of nonlinearities compared to the BRBF. The Alpha-OS technique is unconditionally stable for the linear (elastic) and softening-type nonlinear problems. It is also stable for a range of nonlinear problems with time-step sizes that can be much larger compared to other explicit techniques. The Alpha-OS method can lose its accuracy and stability for highly nonlinear problems (Nakashima et al., 1990). In such cases, iterative implicit techniques can be used instead to handle the nonlinearity of the structure (Shing and Manivannan, 1990; Shing et al., 1991; Mosqueda and Ahmadizadeh, 2011). Therefore, based on the above discussion, a Newton-Raphson iterative algorithm with an implicit average acceleration Newmark integration scheme (Newmark, 1959) with a 0.01 s time step was used for the hybrid simulations on the SCBF.

One point of concern when using iterative schemes in hybrid simulations is the increased number of sub-steps (iterations) to reach the convergence during each time step. Since all of the displacement steps should be applied to the specimens by the actuators, the test duration will potentially increase significantly when an iterative integration scheme is used in the hybrid simulation. To avoid this, a preliminary investigation was conducted to determine the optimum number of iterations. For this purpose, a nonlinear response history analysis was performed on a fully numerical model of the SCBF (all braces modelled numerically in OpenSees) with maximum 10 iterations representing the most accurate response (more iterations would improve the accuracy only marginally). The results were then compared to a fully numerical simulation with 3 iterations. It was observed that the results of the analysis with 3 iterations were almost identical to the analysis with 10 iterations. However, in a hybrid simulation with 2 or 4 physical specimens, a lower number of iterations may be still feasible since much of the nonlinearity of the model
is handled by experimental testing of the braces. Therefore, it was decided to use 2 iterations in the SCBF hybrid simulations. The hybrid simulations with 2 iterations were successfully performed without any instability and with excellent accuracy as discussed in section 5.7.5. Post processing of the SCBF hybrid simulation results later revealed that if a third iteration had been allowed, the resulting displacement increment would have been smaller than the error compensation tolerance limit and therefore a third iteration was unnecessary.

### 5.7.4 Ground motion selection

Sixteen ground motions were selected and scaled to match the uniform hazard design response spectrum of Los Angeles following the ASCE 7-10 (ASCE, 2010) provisions and NEHRP-2011 (NEHRP, 2011) recommendations for selection and scaling of the ground motions for low and mid-rise buildings. All of the ground motions were used for the fully numerical seismic performance assessment of the SCBF described in section 5.8. However, only two of the ground motions were used in the hybrid simulations. The procedures for scaling and selection of the ground motions for the SCBF were similar to the procedures explained in Section 4.5.3 for the BRBF and thus they are not repeated here. Figure 5.23 shows the design response spectrum for Los Angeles and the average response spectrum of the selected scaled ground motions. To show the scatter in the selection, the response spectrum of each of the selected scaled ground motions is also shown in grey colour in Figure 5.23. The ground motions were obtained from the Pacific Earthquake Engineering Research Centre (PEER) ground motion database (PEER, 2015). Similar to the BRBF, 11 pulse-type ground motions were selected for the SCBF analyses. These ground motions had pulse periods close to the principal period of the building in the south-north direction. Detailed information on the selected ground motions is presented in Tables 5.4 and 5.5. The ground motions in Table 5.5 with * in front of their numbers are the pulse type ground motions with forward directivity (FD). As can be seen from Table 5.5, three of the pulse type motions were FD-pulse type. $V_{30}$ in Tables 5.4 and 5.5 is the shear-wave velocity averaged in the top 30 meters of the site soil and as can be seen they are all close to or within the 360 m/s-760 m/s range that is associated to soil type C.

![Figure 5.23: The 5% damped uniform hazard design response spectrum and the response spectrum of the selected scaled ground motions for the SCBF](image-url)
Table 5.4: No-pulse type ground motions selected for the SCBF analysis

<table>
<thead>
<tr>
<th>Record No.</th>
<th>Scale factor</th>
<th>Earthquake</th>
<th>Station</th>
<th>Year</th>
<th>M</th>
<th>R (km)</th>
<th>$V_{so}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>&quot;Niigata_Japan&quot;</td>
<td>&quot;NIGH11&quot;</td>
<td>2004</td>
<td>6.6</td>
<td>8.9</td>
<td>375</td>
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<tr>
<td>2</td>
<td>1.14</td>
<td>&quot;Chuetsu-oki_Japan&quot;</td>
<td>&quot;Joetsu Kakizakiku Kakizaki&quot;</td>
<td>2007</td>
<td>6.8</td>
<td>11.9</td>
<td>383</td>
</tr>
<tr>
<td>3</td>
<td>2.68</td>
<td>&quot;Iwate_Japan&quot;</td>
<td>&quot;AKT023&quot;</td>
<td>2008</td>
<td>6.9</td>
<td>17.0</td>
<td>556</td>
</tr>
<tr>
<td>4</td>
<td>1.70</td>
<td>&quot;Manjil_Iran&quot;</td>
<td>&quot;Abbar&quot;</td>
<td>1990</td>
<td>7.4</td>
<td>12.6</td>
<td>724</td>
</tr>
<tr>
<td>5</td>
<td>2.25</td>
<td>&quot;Hector Mine&quot;</td>
<td>&quot;Hector&quot;</td>
<td>1999</td>
<td>7.1</td>
<td>11.7</td>
<td>726</td>
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</table>

Table 5.5: Pulse type ground motions selected for the SCBF analysis

<table>
<thead>
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<th>Record No.</th>
<th>Scale factor</th>
<th>Earthquake</th>
<th>Station</th>
<th>Year</th>
<th>M</th>
<th>R (km)</th>
<th>$V_{so}$ (m/s)</th>
<th>Pulse period (s)</th>
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<td>6*</td>
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<td>&quot;Coyote Lake Dam - Southwest Abutment&quot;</td>
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<td>0.5</td>
<td>561</td>
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<td>7</td>
<td>1.63</td>
<td>&quot;Morgan Hill&quot;</td>
<td>&quot;Halls Valley&quot;</td>
<td>1984</td>
<td>6.2</td>
<td>3.5</td>
<td>282</td>
<td>0.6</td>
</tr>
<tr>
<td>8</td>
<td>2.70</td>
<td>&quot;Nahanni.Canada&quot;</td>
<td>&quot;Site 2&quot;</td>
<td>1985</td>
<td>6.8</td>
<td>4.9</td>
<td>605</td>
<td>0.6</td>
</tr>
<tr>
<td>9</td>
<td>1.23</td>
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<td>&quot;LA - Sepulveda VA Hospital&quot;</td>
<td>1994</td>
<td>6.7</td>
<td>8.4</td>
<td>380</td>
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<td>10</td>
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<td>&quot;Takarazaka&quot;</td>
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<td>&quot;Bolu&quot;</td>
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<td>&quot;TCU076&quot;</td>
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<td>&quot;Chi-Chi_Taiwan-06&quot;</td>
<td>&quot;TCU078&quot;</td>
<td>1999</td>
<td>6.3</td>
<td>11.5</td>
<td>443</td>
<td>0.8</td>
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<td>14*</td>
<td>1.61</td>
<td>&quot;N. Palm Springs&quot;</td>
<td>&quot;North Palm Springs&quot;</td>
<td>1986</td>
<td>6.1</td>
<td>4.0</td>
<td>345</td>
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<td>15</td>
<td>2.92</td>
<td>&quot;Loma Prieta&quot;</td>
<td>&quot;Gilroy - Gavilan Coll.&quot;</td>
<td>1989</td>
<td>6.9</td>
<td>10.0</td>
<td>730</td>
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</tr>
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<td>16</td>
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<td>&quot;Northridge-01&quot;</td>
<td>&quot;Rinaldi Receiving Sta&quot;</td>
<td>1994</td>
<td>6.7</td>
<td>6.5</td>
<td>282</td>
<td>1.1</td>
</tr>
</tbody>
</table>

The ground motions with * in front of their numbers are the ground motions with forward directivity (FD).

Figure 5.24: The response spectrum of the ground motions selected for the hybrid simulations
The hybrid simulations were performed with record No. 5 and 13 in Tables 5.4 and 5.5 which were, respectively, representative of the no-pulse type and pulse type ground motions. These records were specifically chosen for the hybrid simulations to investigate the effect of type of ground motion on the SCBF response predictions with the hybrid simulations. Figure 5.24 shows the response spectrum of the ground motions selected for the hybrid simulations.

5.7.5 Hybrid simulation results

2E-SCBF-HS and 4E-SCBF-HS1 were performed under ground motion No. 13. 4E-SCBF-HS2 was performed under ground motion No. 5. ACTIA with two cameras was used to capture photos from UT10 and the specimens during the hybrid simulations. Figures 5.25 and 5.26 show the UT10 testing platform with the buckling specimens during the 2- and 4-element hybrid simulations, respectively. Similar to the BRBF tests, the tolerance limit for the error compensation algorithm was set to 0.04 mm at the beginning of the test which was 0.6% of the maximum axial deformation of the buckling brace specimens. The tolerance limit was further relaxed up to 0.06 mm during the tests as the specimens entered the nonlinear response range and following the acceptable performance of the UT10 during the tests. The displacement errors for each of the buckling specimens during 4E-SCBF-HS1 are shown in Figure 2.26.
The hybrid simulation results were compared to the fully numerical results. The fully numerical results were obtained by conducting fully numerical analyses on the OpenSees numerical model of the SCBF with all braces modelled numerically. In order to achieve the best possible accuracy for the fully numerical analyses, smaller time steps were used for the numerical integration of the fully numerical model.

In discussing the simulation results in this section and also in section 5.8, the location of the columns and braces are sometimes referred to with two characters including a number 1 to 5 indicating the storey number that the
brace/column belongs to and a letter N or S which respectively refers to the north or south directions (see Figure 5.21 for north and south directions). For instance column 2N refers to the column in the second storey on the north side of the bay in the SCBF model and brace 1S refers to the brace in the first storey on the south side of the bay in the SCBF model.

![Figure 5.27](image_url)

**Figure 5.27:** Axial force-deformation hysteretic response of the SCBF braces for record No. 13 (fully numerical vs. hybrid tests): (a) 1N, (b) 1S, (c) 2N, and (d) 2S

### 5.7.5.1 Response of the braces

Figure 5.27 shows the axial force-deformation hysteretic response of the SCBF braces in the first two stories for 2E-SCBF-HS and 4E-SCBF-HS1. The axial force and displacement response histories of brace 1N are also shown in Figure 5.28. Figures 5.29 and 5.30 show similar graphs for 4E-SCBF-HS2. The results show that the response of the buckling specimens was properly captured by UT10 during the ground motions. It can also be observed that the fully numerical and hybrid simulation response predictions closely matched with each other for both ground motions. Similar to the cyclic loading, the numerical model predicted a larger compressive strength especially
during the first buckling of the braces. However, these differences seemed not to be affecting the rest of the hysteretic response of the braces. The buckling specimen full-scale $C_{pr}$ values obtained based on the cyclic and hybrid tests are listed in Table 5.6. As can be observed from the values in Table 5.6, the $C_{pr}$ values were within 800-1229 kN range with a mean value of 1040 kN and a CV of 13%. The average experimental value for $C_{pr}$ was only 4% smaller than the 1078 kN value predicted based on the AISC 341-10 equations. The variations observed for $C_{pr}$ were due to the variations in the steel material properties and most importantly the uncontrollable axial loading eccentricities in the UT10 loading platform. The axial eccentricity was largely a result of small misalignments and imperfections in the specimens and also the lateral movements of the loading actuators and the loading shafts. The compressive strength of the conventional braces during the first buckling is very sensitive to the axial eccentricities. However, such eccentricities are also expected to exist in the steel braced frames construction in real practice.

![Figure 5.28: Axial force and deformation response history of brace 1N for record No. 13 (fully numerical vs. hybrid tests)](image)
Figure 5.29: Axial force-deformation hysteretic response of the SCBF braces for record No. 5 (fully numerical vs. hybrid tests): (a) 1N, (b) 1S, (c) 2N, and (d) 2S
Figure 5.30: Axial force and deformation response history of brace 1N for record No. 5 (fully numerical vs. hybrid tests)

Table 5.6: Full-scale compressive strength of the buckling specimens tested in UT10

<table>
<thead>
<tr>
<th>Test</th>
<th>Cyclic</th>
<th>2E-SCBF-HS</th>
<th>4E-SCBF-HS1</th>
<th>4E-SCBF-HS2</th>
<th>Average</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brace</td>
<td>1N*</td>
<td>1S</td>
<td>1N</td>
<td>1S</td>
<td>2N</td>
<td>2S</td>
</tr>
<tr>
<td>$C_p_r (kN)$</td>
<td>924</td>
<td>1229</td>
<td>1215</td>
<td>1048</td>
<td>800</td>
<td>1073</td>
</tr>
</tbody>
</table>

* The number refers to the storey number and the letter refers to the direction (N for north and S for south)

Table 5.7: SCBF hybrid simulation response quantities for the physical brace specimens compared to their numerical predictions

<table>
<thead>
<tr>
<th>Test</th>
<th>2E-SCBF-HS</th>
<th>4E-SCBF-HS1</th>
<th>4E-SCBF-HS2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brace</td>
<td>1N*</td>
<td>1S</td>
<td>1N</td>
</tr>
<tr>
<td>$C_{max,n}$/$C_{max}$</td>
<td>1.04</td>
<td>1.08</td>
<td>1.22</td>
</tr>
<tr>
<td>$T_{max,n}/T_{max}$</td>
<td>0.99</td>
<td>0.99</td>
<td>0.95</td>
</tr>
<tr>
<td>$\mu_{c,max}$</td>
<td>4.7</td>
<td>10.7</td>
<td>4.9</td>
</tr>
<tr>
<td>$\mu_{T,max}$</td>
<td>6.1</td>
<td>1.9</td>
<td>6.0</td>
</tr>
<tr>
<td>$\mu_{c,max,n}/\mu_{c,max}$</td>
<td>1.10</td>
<td>0.91</td>
<td>1.04</td>
</tr>
<tr>
<td>$\mu_{T,max,n}/\mu_{T,max}$</td>
<td>0.96</td>
<td>1.25</td>
<td>0.97</td>
</tr>
<tr>
<td>$E_d (kJ)$</td>
<td>173</td>
<td>137</td>
<td>178</td>
</tr>
<tr>
<td>$E_{d,n}/E_d$</td>
<td>1.18</td>
<td>1.30</td>
<td>1.15</td>
</tr>
</tbody>
</table>

* The letter $n$ in the subscripts refers to the numerical predictions

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The ratios of the maximum tensile forces \( T_{\text{max}} \) and the maximum compressive forces \( C_{\text{max}} \) of the braces predicted by the fully numerical models with respect to the values measured during the hybrid simulations are presented in Table 5.7. The values show that the maximum compressive forces predicted by the fully numerical models were 4-64% more than the hybrid simulation measurements. These variations were mainly due to the variations in the compressive strength of the physical braces. The maximum tensile forces however were much closer to the numerical predictions with errors in the range of 1-12%.

The maximum tension and compression displacement ductilities of the SCBF (respectively \( \mu_{T,\text{max}} \) and \( \mu_{C,\text{max}} \)) during the hybrid simulations and the ratio of the fully numerical predictions with respect to the hybrid simulation results are presented in Table 5.7. The tension and compression displacement ductilities of the physical braces were calculated by deviding the maximum axial tensile and compressive displacements of the physical braces during the hybrid simulations by their axial tensile yield and compressive buckling displacements, respectively. The axial compressive buckling displacements of the physical braces were 6.0 mm for the first storey braces and 5.9 mm for the second storey braces. These values were calculated using the cyclic test results on the buckling specimen by reading the axial compressive displacement of the buckling specimen at the first compressive buckling and then multiplying it by the first and second storey displacement scale factors. The numerical predictions for the tension and compression displacement ductilities of the braces were calculated by deviding the maximum axial tensile and compressive displacements of the braces during the ground motion obtained from the fully numerical analyses by their axial tensile yield and axial compressive buckling displacements, respectively. In order to determine the numerical predictions for the compressive buckling displacement of the braces, numerical monotonic compressive simulations were performed on the calibrated numerical models of the design braces in OpenSees and the brace axial displacements corresponding to the maximum compressive force prior to buckling were recorded. The numerical predictions of the braces axial compressive buckling displacement were 6.5 mm for the first storey braces and 6.4 mm for the second storey braces. The small differences between the numerical and experimental predictions of the axial compressive buckling displacements were mainly due to the accuracy of the OpenSees brace model.

The first storey brace displacement ductilities measured during 2E-SCBF-HS and 4E-SCBF-HS1 closely matched. It can be observed from Figures 5.27 and 5.29 and from the displacement ductility values that the fully numerical models and the hybrid simulations predicted extensive tensile yielding and compressive buckling in the first storey braces for both ground motions. Comparison of the fully numerical predictions with the hybrid simulation results indicate that the ductilities were both underestimated and overestimated by the numerical models. The errors were within the range of 1-26% with an average of 3% for the tension ductilities and 6% for the compression ductilities.

### 5.7.5.2 Energy response

Similar to the BRBF, the energy response of the SCBF was investigated to assess the performance and structural damage in the SCBF. Detailed information on energy calculation methods is provided in Section 4.5.5.2. The \( E_d \) values for each brace calculated from the hybrid simulation results are presented in Table 5.7. The ratio of the numerically predicted \( E_d \) values with respect to the values calculated from hybrid simulation results are also
presented in this table. The history of the total input energy and the total energy absorbed by the braces in the first and second stories for ground motions 13 and 5 are shown in Figures 5.31 and 5.32, respectively. As can be seen from these results, for each of the ground motions 5 and 13 the hybrid simulation input energy histories were nearly identical in value and distribution in time compared to their respective fully numerical predictions. However, compared to the hybrid simulation results, the fully numerical predictions for energy dissipations were respectively 14-32% more for the first storey braces and 7-13% less for the second storey braces except for brace 2N in 4E-SCBF-HS1. These observations confirm the acceptable accuracy of the fully numerical model in predicting the energy response of the SCBF. The results also indicate that the accuracy of the numerical model generally decreased and the numerical model energy predictions became less conservative when the braces experienced larger nonlinear deformations like in the first storey braces. On the other hand, comparison of the fully numerical energy prediction errors for ground motion 5 and 13 indicate that for these ground motions the type of ground motion (pulse or no-pulse) was not significantly affecting the accuracy of the fully numerical energy response predictions. It is also observed that for ground motion 13, 70% of the seismic energy entered the SCBF in only three seconds during 5-8s from the beginning of the ground motion while for ground motion 5 this happened in 8 seconds during 4-12s from the beginning of the ground motion. This observation is consistent with the pulse-type nature of ground motion 13 and concentration of most of the seismic energy in a single large pulse which is the typical characteristic of pulse-type ground motions. The total input energy and energy dissipation in the braces during ground motion 5 was nearly twice the ground motion 13 which indicates larger damage in the brace elements. This is consistent with the generally larger displacement ductility demands for the braces during 4E-SCBF-HS2 compared to 4E-SCBF-HS1 in Table 5.7.

Figure 5.33 shows the contribution of energy dissipation from the braces in different storeys of the SCBF based on the fully numerical and hybrid simulation results. The results show that while the fully numerical predictions were larger than the hybrid simulation results for the first storey braces, they were fairly accurate in predicting the energy dissipation distribution in the braces along the SCBF height. The 2-element and 4-element SCBF tests also showed very similar results. It can be observed from Figure 5.33 that nearly half (38-39% based on the hybrid simulations and 48% based on the fully numerical analysis) of the seismic energy that entered the SCBF during the ground motions was dissipated in the first storey braces. This was almost twice the contribution of the second storey braces. The total contribution of the top three storey braces was 5-10% which indicates small or no nonlinear deformation in the braces in these stories. The rest of the energy was dissipated mainly through the inherent viscous damping in the system or by the nonlinear deformations in other structural elements. These results again confirm that most of the damage was concentrated in the SCBF first storey during both ground motions.
Figure 5.31: Energy response history of the SCBF for record No. 13 (fully numerical vs. hybrid tests)
5.7.5.3 Storey drifts and floor accelerations

The history of the SCBF storey drifts for the fully numerical models and the hybrid simulations are presented in Figures 5.34 and 5.35. The profiles of the maximum residual and transient SDR and floor accelerations along the building height are also presented in Figures 5.36-5.38. As can be seen from Figures 5.34 and 5.35 the fully numerical-Rec.
numerical and the hybrid simulation SDR values closely matched for both ground motions. The closer views of the lateral relative response of the first floor in the first 2.4 seconds in Figure 5.34 shows that the numerical predictions perfectly matched with the hybrid simulation results where the response was mainly dominated by the elastic response of the braces. The hybrid simulation responses started to slightly deviate from the fully numerical predictions at the end of the ground motions which was the direct result of the differences between the numerical models and physically captured hysteretic response of the braces in the nonlinear range of response in the SCBF first two storeys. There were also some larger deviations observed for 2E-SCBF-HS for the SDR values in the third and fourth storey. The deviations started from 5.55 s of the ground motion No. 13 and caused smaller predictions for the third storey SDR values and larger predictions for the SDR in the fourth storey compared to 4E-SCBF-HS1. Further investigations revealed that these deviations were mainly caused by the response of the 3S brace. This brace fully buckled during the 4E-SCBF-HS and the fully numerical analysis, while it remained almost in the elastic range of response during the 2E-SCBF-HS. Although these differences caused 53% error in the maximum SDR in the fourth storey, they did not majorly impact other SCBF response parameters like the maximum overall SDR in all storeys and the floor accelerations in the SCBF. Since the brace buckling load can be highly sensitive to many variables and it occurs quite fast, and since in all the simulations this brace was modelled numerically, it is impossible to identify which response for the 3S brace is more realistic. However, for conservative design, the buckled response should be considered, although, as mentioned above, it did not considerably impact the maximum response parameters of the SCBF.

The maximum SDR profiles in Figures 5.36 and 5.37 show that the fully numerical and the hybrid simulation results predicted a 2.2% maximum transient SDR in the SCBF for both ground motions. The concentration of the lateral deformations in the first storey suggests a soft storey type of response for the SCBF. This is consistent with the energy dissipation distributions shown in Figure 5.33 and the observations in the literature. Similarly, except for the 2E-SCBF-HS results for the fourth storey, the fully numerical and hybrid simulation results predicted larger residual drifts in the first storey of the SCBF which was also an indication of larger damages and nonlinear deformations in this region. The maximum residual SDRs in the SCBF for both ground motions were within 0.25-0.65% range. Figures 5.36 and 5.37 also indicate that compared to the maximum transient SDR values, the residual SDR values displayed larger scatter between the fully numerical predictions and the hybrid simulation results.

The profile of the maximum floor accelerations along the SCBF height is shown in Figure 5.38. The results show that the fully numerical and the hybrid simulations predicted similar distribution of acceleration along the SCBF height. The maximum difference between the fully numerical predictions and the hybrid simulation results was 45% in the first floor for ground motion No. 13 and 22% in the third floor for ground motion No. 5. All of the results indicated a rather uniform distribution of maximum acceleration in floors 1-4 and an acceleration amplification for the top floor levels of the SCBF. The maximum floor acceleration of the top floor was 1.2g and 1.1g based on respectively 4E-SCBF-HS1 and 4E-SCBF-HS2.
Figure 5.34: SCBF storey drift response history for record No. 13 (fully numerical vs. hybrid simulations)
**Figure 5.35:** SCBF storey drift response history for record No. 5: fully numerical vs. hybrid test

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Figure 5.36: SCBF maximum SDR profiles for record No. 13 (fully numerical vs. hybrid tests) (a) residual and (b) transient

Figure 5.37: SCBF maximum SDR profiles for record No. 5 (fully numerical vs. hybrid tests) (a) residual and (b) transient
5.7.5.4 Storey shear hysteretic response

The storey shear vs. SDR hysteresis of the first and second stories for ground motions 13 and 5 are respectively shown in Figures 5.39 and 5.40. The results are shown for the fully numerical and the hybrid simulations. In these figures, only the contribution of the storey braces are considered in the calculation of the storey shears and therefore, these graphs indicate the combined response of the north and south braces at each storey. It can be observed from Figures 5.39 and 5.40 that the fully numerical results matched well with the hybrid simulations for both ground motions. The hysteretic responses show a significant nonlinear response in the braces and energy dissipation in the first and second storeys for both ground motions. The hysteretic responses were pinched which was the direct result of the buckling of the braces. Such pinched hysteretic response is a characteristic response of SCBFs.

Figure 5.39: Storey shear (braces contribution) vs. SDR hysteresis of SCBF braces for record No. 13 (fully numerical vs. hybrid tests): (a) first storey and (b) second storey
5.7.6 Control challenges and solutions

2E-SCBF-HS was stopped at the beginning of the hybrid simulation shortly after the first buckling of the buckling specimen representing the 1N brace. Further investigations revealed that when the compressive force suddenly dropped during buckling of this specimen, the slaved force-controlled actuator underneath the specimen was not able to follow the force command fast enough and therefore a large PID control error occurred for this actuator which exceeded the limit for the control error in the controller and stopped the simulation. On the other hand, it was observed that the buckling specimen moved slightly more than the loading and slaved actuators at the specimen top and bottom. This was mainly due to the snap-back of the elastic deformations in the UT10 actuator reaction frame which occurred when the compressive force suddenly dropped after the first buckling of the specimen. This issue caused a sudden force change in the hysteretic response of the brace. This can be seen in the 2E-SCBF-HS curve in Figure 5.27a. To further illustrate this event, the force and deformation response of the 1N brace at the time of this event is shown in the sub-plots in Figure 5.28. As can be seen from these sub-plots the integration module received an unrealistic 1300 kN sudden change in the brace force feedback at time 2.44 s of the ground motion. However, further investigations revealed that the axial command and feedback displacements were not affected. This issue which slightly affected the response of the 1N brace did not occur for other specimens and had a negligible impact on the global response of the SCBF. One approach to avoid this issue is to slightly increase the control error limits for the loading and slaved actuators and manually expand the ramp and hold durations in NICON-10 shortly before the expected first buckling of the buckling specimens. The limits and durations can be manually adjusted back to their original values after the first buckling of the buckling specimens. This strategy was adopted for 4E-SCBF-HS1 and 4E-SCBF-HS2 and successfully resolved the control issue during the first buckling of the buckling specimens for these simulations.
5.8 SEISMIC PERFORMANCE ASSESSMENT OF THE SCBF

The hybrid simulation results presented in section 5.7 confirmed the accuracy and suitability of the brace models for seismic performance assessment of the SCBFs. Therefore, the same model was adopted to conduct a full performance assessment of the SCBF. The performance assessment was done following the ASCE 41-13 (ASCE, 2014) guidelines for seismic evaluation and retrofit of the existing buildings.

The numerical model used for performance assessment of the SCBF was similar to the model discussed in section 5.7 for the fully numerical model with all of the braces numerically modelled and calibrated. The seismic performance of the SCBFs is affected by the flexural response of the column baseplate connections. Such effects can sometimes have serious impacts on the design of the steel frames particularly for medium and high-rise SCBFs. Due to the uncertainty of response and lack of knowledge on the flexural hysteresis of the column baseplates, especially in braced frames, they are usually modelled as either fixed or pinned in practice. The pin assumption is believed to result in conservative estimation of the lateral deformations of the structure. This can potentially result in overly conservative designs for drift-controlled structures. On the other hand, some studies have shown that the pinned baseplate assumption still results in good prediction of forces in the columns (Zareian and Kavinde 2013). Based on the above and in order to investigate the sensitivity of the SCBF performance predictions to the column baseplate connection model, the rotation of the SCBF column baseplates was once modelled as fully fixed (hereafter referred to as model F) and once modelled as fully pinned (hereafter referred to as model P) in the numerical analysis that is presented in this study.

NRHA was performed on the SCBF under the 16 ground motions listed in Tables 5.4 and 5.5. These ground motions were selected and scaled to match the design response spectrum of Los Angeles with a seismic hazard level corresponding to 10% probability of exceedance in 50 years (475 years return period). These ground motions are referred to as the Design Basis Earthquakes (DBE). In order to assess the performance of the SCBF at higher hazard levels, the SCBF model was also subjected to the same ground motions scaled up further by 50%. This level of seismic hazard corresponds to 2% probability of exceedance in 50 years (2475 years return period) and the resulting ground motions are referred to as the Maximum Considered Earthquakes (MCE).

5.8.1 Performance criteria

Similar to the BRBF performance assessment, the seismic performance of the SCBF was evaluated by studying the maximum response parameters of the SCBF at two levels: global level and local level. For the global level, the storey drifts and the absolute accelerations of the floors were analyzed. For the local level, the axial displacement ductility, damage, and energy dissipation in the braces were evaluated. Development of nonlinear deformations in the beams and columns and the response of the beam-column connections were also investigated. The performance levels considered in this study were immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The maximum acceptable values for $SDR_{T,m}$, $SDR_{R,m}$, $\mu_{T,max}$, $\mu_{C,max}$, and the plastic rotation of the columns for the performance levels considered in this study are presented in Table 5.8. The limits for the drifts were roughly defined based on the amount of expected damage to the structural and non-structural elements and the ability of the main
structural components to reliably carry the gravity loads thus maintain the stability of the system. The damage levels corresponding to each of the performance levels are described in ASCE 41-13. In the evaluation of the damage levels corresponding to the drift values, it was assumed that the braces start to develop flexural buckling at an SDR value of approximately 0.5%. This is consistent with the various experimental and numerical observations in this study and also in the literature (Hwang and Lignos, 2017). The $SDR_{T,m}$ limit for the LS performance level in Table 5.8 was chosen according to the ASCE 7-10 provisions. The brace ductility limits and the column plastic rotation limits were however chosen based on the ASCE 41-13 provisions. Since the SCBF columns did not fulfil the criteria for the compact sections defined in ASCE 41-13, the limits for the plastic rotation of the columns were chosen based on non-compact sections as per ASCE 41-13 provisions. Similar to the BRBF performance assessment, the braces axial ductility and the columns plastic rotations were given more weight in the evaluation of the seismic performance of the SCBF in this study.

| Table 5.8: Acceptance criteria for the performance levels considered for the SCBF |
|---------------------------------|--------|-----|-----|
| Response type                      | IO    | LS  | CP  |
| Storey lateral deflection          | $SDR_{T,m}$(%)  | 0.5  | 2.5  | 4   |
|                                  | $SDR_{R,m}$(%)  | Negligible | 0.5  | 2   |
| Brace axial deformation           | $\mu_{T,max}$  | 1.5  | 9    | 12  |
|                                  | $\mu_{c,max}$  | 1.5  | 7.7  | 9.4 |
| Column flexural rotation          | $\theta_{P}/\theta_{y}$ ($P/P_{CL} < 0.2$)  | 0.25  | 3 | 4 |
|                                  | $\theta_{P}/\theta_{y}$ ($0.2 \leq P/P_{CL} \leq 0.5$)  | 0.25  | 1.2 | 1.2 |

5.8.2 Global response parameters

5.8.2.1 Storey drift ratio (SDR)

$SDR_{T,m}$ and $SDR_{R,m}$ values for the SCBF under each of the ground motions at DBE and MCE levels are presented in Tables 5.9 and 5.10. The results are shown for both models P and F. Figures 5.41 and 5.42 show the distribution of the maximum SDR values for all of the ground motions. The SCBF maximum SDR profiles for models P and F under DBE and MCE ground motions are shown in Figures 5.43-5.46. The average of the maximum values over all ground motions and the average plus one SD for each storey is also shown in these figures. Finally, the contribution of each storey to the maximum lateral deflection of the top floor is shown in Figures 5.47 and 5.48.

As discussed in Section 4.6.2.2.1, the ground motions with unacceptable results should be identified and excluded from the SCBF performance assessment. The drift results presented in Tables 5.9 and 5.10 indicate that the maximum SDR for four of the MCE ground motions slightly exceeded the 5% limit specified in Section 4.6.2.2.1 but they were all below 5.5%. Therefore, considering the level of the modelling details adopted in the SCBF models, especially for the beam-column and brace connections, the numerical results for all ground motions were assumed to be acceptable in this study.

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Table 5.9: Maximum values of the SCBF global response parameters under the DBE ground motions

<table>
<thead>
<tr>
<th>Ground motions</th>
<th>Rec 1</th>
<th>Rec 2</th>
<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
<th>Rec 11</th>
<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
</tr>
</thead>
<tbody>
<tr>
<td>$SDR_{T,m}$(%)</td>
<td>P</td>
<td>1.8</td>
<td>3.2</td>
<td>2.0</td>
<td>2.2</td>
<td>2.3</td>
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<td>2.1</td>
<td>1.8</td>
<td>1.9</td>
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<td>2.2</td>
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<tr>
<td></td>
<td>F</td>
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<td>1.2</td>
<td>1.1</td>
<td>1.5</td>
<td>1.8</td>
<td>1.5</td>
<td>1.3</td>
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<tr>
<td>$SDR_{R,m}$(%)</td>
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<td>0.3</td>
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<td>0.6</td>
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Table 5.10: Maximum values of the SCBF global response parameters under the MCE ground motions

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<th>Ground motions</th>
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<th>Rec 4</th>
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<th>Rec 6</th>
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<th>Rec 10</th>
<th>Rec 11</th>
<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
</tr>
</thead>
<tbody>
<tr>
<td>$SDR_{T,m}$(%)</td>
<td>P</td>
<td>3.5</td>
<td>5.5</td>
<td>3.8</td>
<td>4.7</td>
<td>5.5</td>
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<td>4.6</td>
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<tr>
<td></td>
<td>F</td>
<td>3.1</td>
<td>4.5</td>
<td>1.9</td>
<td>2.9</td>
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<td>2.5</td>
<td>2.6</td>
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<td>4.1</td>
<td>2.1</td>
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<tr>
<td>$SDR_{R,m}$(%)</td>
<td>P</td>
<td>1.5</td>
<td>3.6</td>
<td>0.4</td>
<td>0.3</td>
<td>3.8</td>
<td>0.7</td>
<td>1.1</td>
<td>0.6</td>
<td>0.5</td>
<td>0.3</td>
<td>0.9</td>
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<td>3.4</td>
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</tr>
<tr>
<td></td>
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<td>1.3</td>
<td>1.5</td>
<td>0.5</td>
<td>0.8</td>
<td>1.2</td>
<td>0.4</td>
<td>0.7</td>
<td>0.4</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>0.5</td>
<td>0.3</td>
<td>2.2</td>
<td>0.4</td>
</tr>
<tr>
<td>$FA_{m}(g)$</td>
<td>P</td>
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<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.2</td>
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<td>1.2</td>
<td>1.3</td>
<td>1.1</td>
<td>1.3</td>
<td>1.4</td>
</tr>
<tr>
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<td>1.3</td>
<td>1.3</td>
<td>1.5</td>
<td>1.2</td>
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<td>1.2</td>
<td>1.4</td>
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<td>1.4</td>
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The results presented in Tables 5.9 and 5.10 and Figures 5.41 and 5.42 show that the SCBF maximum SDR predicted by model F was considerably smaller than model P for most of the ground motions. The values show that the average difference was 26% and 22%, respectively, for $SDR_{T,m}$ and $SDR_{R,m}$ under the DBE ground motions and 27% and 26%, respectively, for $SDR_{T,m}$ and $SDR_{R,m}$ under the MCE ground motions. The maximum drift profiles in Figures 5.43-5.46 further show that the distribution of the maximum SDR was also considerably affected by the modelling assumption for the column baseplates. While model P predicted the concentration of lateral deformations in the first storey, model F predicted a more uniform distribution of the storey drifts along the SCBF height and even larger SDR in the second and third stories compared to the first storey. The smaller SDR values for model F particularly in the first storey was due to the more restrained boundary condition of the columns and braces in this storey compared to model P.

Based on the values presented in Tables 5.9 and 5.10, the $SDR_{T,m}$ values for both P and F models were within the range of 1.1%-3.2% for the DBE ground motions and 1.8%-5.5% for the MCE ground motions. The $SDR_{R,m}$ values for both P and F models were within the range of 0.0%-1.3% for the DBE ground motions and 0.1%-3.8% for the MCE ground motions. While the largest $SDR_{T,m}$ and $SDR_{R,m}$ values occurred under the no-pulse type ground motions, the SDR response of the SCBF did not generally suggest a significant difference between the pulse and no-pulse type ground motions.

Figures 5.43-5.46 show the maximum SDR for each storey for different ground motions. The average and average plus one SD of the SDR values at each storey are also shown in these figures. The limits for the acceptance standards were

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criteria for the three performance levels considered in this study are also indicated in these figures. The results show that for both model P and model F the average of the maximum SDR values satisfied the LS limits under the DBE ground motions and CP limits under the MCE ground motions. According to model P results, the maximum of the average transient SDR was 2.1% and occurred in the first storey under the DBE ground motions. However, model F predicted a maximum average SDR value of 1.4% occurring in the second storey. As indicated in Figure 5.44, for the MCE ground motions, model P predicted a maximum average transient SDR value of 4% occurring in the first storey while model F prediction was 2.8% and occurring in the second storey. The model P maximum average residual SDR under the DBE and MCE ground motions were respectively 0.4% and 1.4% which occurred in the first storey of the SCBF. These values were near twice the model F predictions for both ground motion levels. If a normal distribution is assumed for the SDR values, the average plus one SD values represent the 84-percentile which is representative of the SDR value which 84% of the data fall below it. Figures 5.43-5.46 indicate that based on the model P predictions for the maximum SDR values, the average plus one SD values did not satisfy the performance limits. The CV range was 16-38% and 21-51% for the transient SDR values under the DBE and MCE ground motions, respectively. For the residual SDR values, the CV range was 54-103% and 55-96% under the DBE and MCE ground motions, respectively. The large CV values suggest that there was significant variability in the SDR predictions. The variabilities increased with the increase in the ground motion hazard level which was mainly due to the larger nonlinear deformations in the SCBF which caused larger uncertainties due to the modelling inaccuracies in the response predictions. The CV values for the residual SDR were almost twice larger than the transient SDR. The large scatter in the residual SDR values was consistent with the observations in the literature and was mainly due to the sensitivity of the residual drifts to several parameters including the number/magnitude of the nonlinear cycles in the ground motions and the hysteretic shape of the energy dissipating elements. The residual SDR values are believed to be extremely difficult to be reliably predicted.

Figures 5.47 and 5.48 indicate the contribution of each storey to the maximum lateral deflection of the top floor level for DBE and MCE level ground motions. The results indicate that for model P and for both levels of ground motion, the first storey constituted the maximum contribution amongst other storeys with an average contribution of 43% for both levels of ground motion. For model F however, this average contribution reduced to 18% and 21% under the DBE and MCE ground motions, respectively. The average contribution of the third storey almost doubled in model F compared to model P while the average contribution of the second storey remained unchanged. These observations are consistent with the observations on the maximum SDR profiles in Figures 5.43-5.46.

A summary of the SDR responses of 6-storey SCBFs studied by other researchers under suites of DBE level ground motions for Los Angeles is presented in Table 5.11. Information on the type of brace model used in these studies and consideration of the contribution of the lateral stiffness of the gravity columns are also presented in this table. The results indicate that the drift values of the SCBF considered in this study are consistent with other studies for both transient and residual SDR.
Table 5.11: Average SDR response of SCBFs under suites of ground motions with hazard levels corresponding to 10% probability of exceedance in 50 years (DBE) for Los Angeles

<table>
<thead>
<tr>
<th></th>
<th>Number of storeys</th>
<th>Gravity column lateral stiffness</th>
<th>Brace model</th>
<th>$SDR_{T,avg}$ (%)</th>
<th>$SDR_{R,avg}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCBF in this study</td>
<td>5</td>
<td>Considered</td>
<td>Fibre-based multi-element</td>
<td>2.1 (P)</td>
<td>0.4 (P)</td>
</tr>
<tr>
<td>Sabelli (2001)</td>
<td>6</td>
<td>Considered</td>
<td>Phenomenological</td>
<td>1.4 (F)</td>
<td>0.2 (F)</td>
</tr>
<tr>
<td>McCormick et al. (2004)</td>
<td>6</td>
<td>Not considered</td>
<td>Phenomenological</td>
<td>1.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Uriz and Mahin 6 (2008)</td>
<td>6</td>
<td>Considered</td>
<td>Fibre-based multi-element</td>
<td>0.4</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 5.41: Maximum SDR values obtained based on models P and F for the SCBF under the DBE ground motions
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Figure 5.42: Maximum SDR values obtained based on models P and F for the SCBF under the MCE ground motions

Figure 5.43: Maximum transient SDR values for each storey of the SCBF under the DBE ground motions: (a) model P and (b) model F

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Figure 5.44: Maximum transient SDR values for each storey of the SCBF under the MCE ground motions: (a) model P and (b) model F

Figure 5.45: Maximum residual SDR values for each storey of the SCBF under the DBE ground motions: (a) model P and (b) model F
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Figure 5.46: Maximum residual SDR values for each storey of the SCBF under the MCE ground motions: (a) model P and (b) model F

Figure 5.47: Contribution of the relative lateral deformation of the SCBF storeys to the top floor lateral deformation for models P and F when the top floor reaches the maximum lateral deformation under the DBE ground motions
Figure 5.48: Contribution of the relative lateral deformation of the SCBF storeys to the top floor lateral deformation for models P and F when the top floor reaches the maximum lateral deformation under the MCE ground motions

5.8.2.2 Floor acceleration (FA)

The maximum absolute floor accelerations ($FA_m$) in the SCBF based on models P and F under both DBE and MCE level ground motions are presented in Tables 5.9 and 5.10. Figures 5.49 and 5.50 show the profile of the maximum floor accelerations respectively for DBE and MCE ground motions. The average and average plus one SD values for each floor level are also shown in these figures. The $FA_m$ values in Tables 5.9 and 5.10 indicate that there was no considerable difference between the model P and model F predictions. Furthermore, there was no considerable difference between the $FA_m$ values under the MCE ground motions compared to the DBE ground motions. This can also be observed from the profiles in Figures 5.49 and 5.50. This observation is mainly due to the fact that during the MCE ground motions, most of the braces were buckled and were in their nonlinear response range with negative post-buckling or limited positive post-yield stiffness. Therefore, the SCBF did not provide larger stiffness and more resistance to the acceleration under the MCE ground motions compared to the DBE ground motions. It can be observed from Figures 5.49 and 5.50 that the average acceleration for each floor level was in the range of 0.85g-1.27g for both models P and F and under both the DBE and MCE ground motions. Acceleration amplification was observed for the top floor. The average acceleration amplification was around 60% and 12% with respect to the ground motion maximum acceleration, respectively, under the DBE and MCE ground motions. The larger top floor acceleration amplification under the DBE ground motions was due to the elastic response of the braces in the fifth storey. These braces however buckled and did not provide further resistance under the MCE ground motions. The CV values for the maximum floor accelerations were within 6-29% under the DBE and MCE ground motions. These values suggest a generally smaller scatter for the acceleration response of the SCBF compared to the SDR responses. The smaller scatter in the acceleration response can be due to the rather small variations of the ground motion spectral accelerations around the principal period of the SCBF as can be seen in Figure 5.23. In addition, the

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principal period and maximum acceleration response of the SCBF majorly depends on the elastic stiffness of the structural components which can be accurately modelled. The rather large floor accelerations and the acceleration amplification observed for the SCBF is the characteristic response of the braced frames with large lateral stiffness. These levels of floor acceleration are expected to damage structural contents under both the DBE and MCE ground motions.

Figure 5.49: Maximum acceleration for each floor of the SCBF under the DBE ground motions: (a) model P and (b) model F

Figure 5.50: Maximum acceleration for each floor of the SCBF under the MCE ground motions: (a) model P and (b) model F
5.8.3 Local response parameters

5.8.3.1 Response of the braces

Based on ASCE 41-13, the axial actions in the SCBF braces are categorized as deformation controlled with a ductile behaviour and thus a significant nonlinear response is allowed to develop axially in the braces. Therefore, the performance of the braces was verified mainly by measuring their axial plastic deformation in the form of axial tension and compression displacement ductility ($\mu_{T,m}$ and $\mu_{C,m}$) during the ground motions. In addition to this, the percentage of low-cycle fatigue damage ($\lambda_m$) in the braces outermost cross section fibres and the amount of hysteretic energy dissipation ($E_d$) in the braces were assessed. The axial displacement ductility and the energy values were calculated following the approach explained in section 5.7.5. Tables 5.12 and 5.13 show the maximum responses of the braces based on models P and F under the DBE and MCE ground motions, respectively. The results in these tables are shown for the storeys that demonstrate significant nonlinear response i.e. storeys 1-3 under the DBE ground motions and storeys 1-4 under the MCE ground motions. The ductility and damage values in Tables 5.12 and 5.13 are the maximum responses of both braces in each storey. The energy dissipation ratios ($E_d/E_i$) are the ratio of the total energy dissipated by the braces at each storey divided by the total seismic energy that enters the SCBF during the ground motions ($E_i$).

The results presented in Tables 5.12 and 5.13 indicate that model P predicted larger ductility demand, damage, and energy dissipation compared to model F in the first storey braces for all ground motions. It can be also seen that model P response predictions were smaller than model F for the third storey braces. Both models predicted almost similar response values for the second storey braces. This is consistent with the previous observations on the SDR profiles and lateral deflection contribution of the storeys.

Based on the results presented in Tables 5.12 and 5.13, larger ductility demands caused more damage and energy dissipation in the braces. The maximum ductility values in the SCBF (maximum over all of the storeys) are also presented in Figures 5.51 and 5.52 for the DBE and MCE ground motions. Based on these figures, the magnitudes of the brace maximum compression ductility in the SCBF were larger than the magnitudes of the maximum tension ductility by 53%-111% under the DBE ground motions and 24%-85% under the MCE ground motions. This was mainly due to the fact that the vertical deformation of the floor beams limited the tensile yielding and elongation of the braces and thus in most of the ground motions, the tensile elongation of the braces was smaller than their compressive deformation. In addition, the tensile yield displacement of the braces was larger than their compressive buckling displacement which resulted in larger magnitudes of the compressive displacement ductilities compared to the tensile displacement ductilities. Figures 5.51 and 5.52 also show that the maximum ductility of the SCBF predicted by model P was larger than model F prediction. This was mainly due to the more uniform distribution of the deformation demands along the SCBF stories as a result of fixing the base of the columns in model F.

Figures 5.53-5.56 show the maximum displacement ductility of the SCBF north and south braces at different storeys for models P and F under the DBE and MCE ground motions. The average and average plus one SD of the maximum ductility of each of the braces over all of the ground motions are also shown in these figures. The compression and tension axial displacement ductility limits for the performance levels considered in this study were:

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presented in Table 5.8 are also shown in these figures. The results again confirm that the magnitudes of the compression ductility were larger than the magnitudes of the tension ductility. Furthermore, it can be seen from these figures that the north braces have similar average ductility responses compared to the south braces. Based on both model P and F predictions, the average values of the braces ductility satisfied the LS performance level requirement under the DBE ground motions. However, the average plus one SD values for the compression ductility did not even satisfy the CP performance level based on model P predictions. Figure 5.55 shows that based on model P predictions, the average of the compression ductility of the braces in the first storey fell outside the CP performance level limit under the MCE ground motions. However, based on model F predictions, all of the braces satisfied the CP performance level requirements. The ductility values for the fifth storey braces indicate that they remain essentially elastic with no buckling or undergo minor nonlinear deformations under both DBE and MCE ground motions. The ductility values showed a considerable amount of scattering both for the tension and compression ductility. The average CVs for the brace maximum displacement ductility under DBE and MCE ground motions were 44% and 56% respectively. This shows that the variations became larger as the brace nonlinear deformations increased at higher seismic hazard levels.

The values of \( E_d / E_{in} \) for each storey under the DBE and MCE ground motions are presented in Tables 5.12 and 5.13. Figures 5.57 and 5.58 compare the contribution of the energy dissipated by the braces in each storey to the total energy dissipated by the SCBF until end of each of the DBE and MCE ground motions. The results are presented for both model P and model F. The average of the model P results for all ground motions indicate that 45% and 39% of the total seismic energy was dissipated by the braces in the first storey of the SCBF under the DBE and MCE ground motions, respectively. This contribution reduced to around 15% for model F results. The contribution of the third storey however increased from 3-7% based on model P results to 20% based on model F results. These results are consistent with the previous observations on the storey lateral deflections and axial displacement ductility of the braces.

Based on the values presented in Table 5.12, the maximum value for the percentage of low-cycle fatigue damage \( \lambda_m \) was 57% under the DBE ground motions which implies that no fracture initiated in any of the braces under the DBE ground motions. The values of \( \lambda_m \) in Table 5.13 indicate that model F predicted initiation of fracture in the second storey braces under ground motion No. 2 and 5 and in the fourth storey braces under ground motion No. 9. Further investigation of the damages in the fibres located at the middle of the cross section in these braces was done to verify if they were fully fractured. The results reveal that based on model F predictions full fracture occurred only in the brace 2N under ground motion No. 2 for the MCE ground motions. The results in Table 5.13 also indicate that based on model P predictions, there was significant damage in the first storey braces under ground motion No. 2, 3, 4, 5, 9, and 13. Further investigation of the damages revealed that the north brace under ground motions No. 3 and No. 4 and No. 13, the south brace under ground motion No. 5, and both braces under ground motion No. 9 in the first storey of the SCBF were fully fractured during the ground motions. In all of these cases the, initiation of fracture occurred at the end of the ground motions after many large nonlinear cycles and when the braces were being straightened while reloading from compression to tension. Therefore, full fractures of the braces did not cause any instability in the system and nor did it affect the maximum response parameters of the SCBF. In all of the cases

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with brace full fracture, the SCBF kept its integrity until the end of the earthquake and no collapse occurred. Concentration of damages in the first storey braces based on model P predictions was consistent with the previous observations from the SDR, axial displacement ductility of the braces, and the energy dissipation contribution of the first storey braces. Comparison of the $\lambda_m$ values for the no-pulse type ground motions (No. 1-5) with the $\lambda_m$ values for the pulse-type ground motions (No. 6-16) reveals that the no-pulse type ground motions on average caused more low-cycle fatigue damage in the braces. However, the average axial displacement ductility demands during the pulse type and no-pulse type ground motions were almost similar. This was due to the fact that although the maximum compressive and tensile deformation of the braces were not significantly different under the pulse and no-pulse type ground motions, the braces underwent more number of large amplitude nonlinear cycles during the no-pulse type ground motions causing more low-cycle fatigue damage and fracture in the braces under no-pulse type ground motions compared to the pulse type ground motions. This observation emphasizes the importance of consideration of the type of the ground motions in the seismic performance assessment of the braces in the SCBF.

Table 5.12: Maximum SCBF brace response values under the DBE ground motions

<table>
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<tr>
<th>Ground motions</th>
<th>Rec 1</th>
<th>Rec 2</th>
<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
<th>Rec 11</th>
<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
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<th>Rec 16</th>
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### Table 5.13: Maximum SCBF brace response values under the MCE ground motions

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<thead>
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<th>Ground motions</th>
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<th>Rec 3</th>
<th>Rec 4</th>
<th>Rec 5</th>
<th>Rec 6</th>
<th>Rec 7</th>
<th>Rec 8</th>
<th>Rec 9</th>
<th>Rec 10</th>
<th>Rec 11</th>
<th>Rec 12</th>
<th>Rec 13</th>
<th>Rec 14</th>
<th>Rec 15</th>
<th>Rec 16</th>
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</thead>
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<tr>
<td>$\mu_{cm}$ (P)</td>
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<td>22.8</td>
<td>15.1</td>
<td>18.5</td>
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<td>13.1</td>
<td>16.0</td>
<td>15.4</td>
<td>13.6</td>
<td>12.9</td>
<td>14.0</td>
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<td>14.4</td>
<td>21.5</td>
<td>12.5</td>
<td>21.4</td>
</tr>
<tr>
<td>$\mu_{T,m}$ (P)</td>
<td>8.3</td>
<td>14.7</td>
<td>5.9</td>
<td>8.7</td>
<td>12.9</td>
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<td>6.9</td>
<td>10.8</td>
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<tr>
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<td>103</td>
<td>313</td>
<td>278</td>
<td>154</td>
<td>14</td>
<td>25</td>
<td>45</td>
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<td>13</td>
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<td>$E_d/E_{in}$ (P)</td>
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<td>0.38</td>
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<td>0.40</td>
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<td>0.42</td>
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<td>0.14</td>
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<td>0.15</td>
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<td>11.9</td>
<td>15.4</td>
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<td>10.0</td>
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<td>0.24</td>
<td>0.20</td>
<td>0.20</td>
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<td>0.03</td>
<td>0.16</td>
<td>0.06</td>
<td>0.05</td>
<td>0.07</td>
<td>0.10</td>
<td>0.05</td>
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</tr>
<tr>
<td>$\mu_{cm}$ (F)</td>
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<td>5.9</td>
<td>7.3</td>
<td>3.4</td>
<td>6.0</td>
<td>2.9</td>
<td>2.1</td>
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<td>4.7</td>
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<tr>
<td>$\mu_{T,m}$ (F)</td>
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<td>6.3</td>
<td>5.9</td>
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<td>3.4</td>
<td>6.0</td>
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<td>7.3</td>
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<td>4.7</td>
</tr>
<tr>
<td>$\lambda_n$ (%)</td>
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<td>4</td>
<td>8</td>
<td>4</td>
<td>2</td>
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<td>4</td>
<td>1</td>
</tr>
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<td>0.01</td>
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<td>0.03</td>
<td>0.02</td>
<td>0.06</td>
<td>0.10</td>
<td>0.06</td>
<td>0.03</td>
<td>0.02</td>
<td>0.07</td>
<td>0.02</td>
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</tr>
<tr>
<td>$E_d/E_{in}$ (P)</td>
<td>0.09</td>
<td>0.04</td>
<td>0.12</td>
<td>0.13</td>
<td>0.03</td>
<td>0.07</td>
<td>0.03</td>
<td>0.01</td>
<td>0.20</td>
<td>0.13</td>
<td>0.05</td>
<td>0.05</td>
<td>0.07</td>
<td>0.06</td>
<td>0.13</td>
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</table>
Figure 5.51: Maximum axial displacement ductility of the braces in the SCBF based on models P and F under the DBE ground motions (positive is tension and negative is compression)

Figure 5.52: Maximum axial displacement ductility of the braces in the SCBF based on models P and F under the MCE ground motions (positive is tension and negative is compression)
Figure 5.53: Maximum axial displacement ductility of the braces in the SCBF based on model P under the DBE ground motions (positive is tension and negative is compression): (a) north braces and (b) south braces.
Figure 5.54: Maximum axial displacement ductility of the braces in the SCBF based on model F under the DBE ground motions (positive is tension and negative is compression): (a) north braces and (b) south braces.
Figure 5.55: Maximum axial displacement ductility of the braces in the SCBF based on model P under the MCE ground motions (positive is tension and negative is compression): (a) north braces and (b) south braces.
Figure 5.56: Maximum axial displacement ductility of the braces in the SCBF based on model F under the MCE ground motions (positive is tension and negative is compression): (a) north braces and (b) south braces.

Figure 5.57: Contribution of the braces energy dissipation in the SCBF based on models P and F under the DBE ground motions.
In order to evaluate the response and performance of the brace gusset plates, ASCE 41-13 requires that all the actions on the gusset plates be considered as force controlled unless the brace connections are explicitly modelled and their ductile response is verified by the experiments. The gusset plate connections in the SCBF in this study were specifically designed to allow the formation of plastic hinges in the connections and accommodate the large nonlinear demands from the conventional braces. Based on the review of the literature presented in section 5.2, the configuration adopted in this study for the gusset plates has been verified by several experiments in the literature and it has been confirmed that it is able to reliably accommodate extensive nonlinear deformations. Therefore, it was assumed that the gusset plates performed as expected without any considerable damage and fracture during the DBE and MCE ground motions and thus the behaviour of the gusset plates in the SCBF was not specifically evaluated in this study.

### 5.8.3.2 Response of the beams, columns, and beam-column connections

Nonlinear deformations and yielding of the beams and columns during the ground motions were studied. For this purpose the beams and columns were assumed to have yielded if the strain in any of the outermost cross section fibres at the ends of the beams and columns reached the yield strain. The results indicated that the floor beams did not experience any yielding and nonlinear deformation during the ground motions. However, these results are considered to be overly conservative since the horizontal movements of the nodes at each floor level were constrained in the SCBF model in order to enforce the diaphragm effect of the floor slabs. As a result of this modelling assumption, no axial forces were developed in the floor beams in the SCBF models. In reality, significant axial forces are expected to develop in the beams due to the brace horizontal force components. However, both the flexural and axial actions due to the brace vertical and horizontal force components at the mid-span of the floor beams were considered in the capacity design of the beams and therefore the performance and capacity of the beams were expected to be sufficient during the ground motions.
Unlike the beams, the responses of the columns were studied more thoroughly as they were expected to experience flexural actions due to the storey drifts which were not considered in the capacity design of the columns. The number of column ends that yielded under the DBE and MCE ground motions are presented in Table 5.14. The results indicate that the occurrence of yielding in the columns increased significantly during the MCE ground motions compared to the DBE ground motions. This was primarily due to the increased frame action of the columns and larger storey drifts during the MCE ground motions. The results also indicate that compared to model P, model F predicted more instances of column end yielding. Figures 5.59 and 5.60 show the column yielding locations and the values of plastic rotation ratio ($\theta_p/\theta_y$) in the SCBF for models P and F under the MCE ground motions. The yielding locations are shown by solid circles in these figures. In these figures, the values of the plastic rotation ratio are only shown for the locations with significant yielding. It should be noted that the yielding did not necessarily occur simultaneously at the locations shown in these figures. It can be observed from Figures 5.59 and 5.60 that ground motions No. 2, 4, 14, and 16 imposed the largest flexural demands on the columns at the MCE level especially at base of the first storey columns for model F. Based on the results for the DBE ground motions and as can be seen from Figures 5.59 and 5.60 model P predicted yielding at the top ends of the first storey columns for most of the DBE and MCE ground motions. Model F however predicted significant yielding mainly at the base of the first storey columns. The results also indicate that for some of the ground motions there was column yielding at the top ends of the second storey columns and at the column splice location and top ends of the third storey columns. This was potentially due to the higher modes effects in the SCBF. The extent of column yielding reached the fourth storey columns for some of the MCE ground motions but no yielding occurred in the fifth storey columns under the DBE and MCE ground motions.

To evaluate the extent of flexural nonlinear deformations in the columns, the plastic rotation ratio ($\theta_p/\theta_y$) was calculated at the column ends in the SCBF. For this purpose, a similar approach as explained in section 4.6.2.3.2 for the BRBF beams and columns was also adopted for the SCBF. Based on ASCE 41-13, the axial compressive actions in the SCBF columns are categorized as force controlled with a brittle response and thus no nonlinear deformation is allowed for the axial compressive actions in the columns. The flexural actions in the columns however, can be categorized as displacement controlled with a ductile response if the compressive axial load in the column is not more than 50% of the column compressive load capacity ($P_{CL}$). The columns under combined axial tension and bending stress are considered deformation controlled based on ASCE 41-13. The maximum of axial compressive demand to capacity ($P/P_{CL}$), the plastic rotation ratio ($\theta_p/\theta_y$), and the combined compressive axial-

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flexural strength ratio \( \frac{P}{P_{CL}} + \frac{M}{M_{CL}} \) or the P-M ratio) for each of the column end sections were calculated and the average of the results over all of the ground motions were compared to the performance level limits as defined in ASCE 41-13. Table 5.15 shows the average response of the columns with significant average flexural-axial nonlinear deformation and force demand. The rest of the SCBF columns experienced negligible nonlinear deformation and/or axial-flexural force demand which well satisfied both the deformation and strength limits for the LS and CP performance levels under both DBE and MCE ground motions. The results presented in Table 5.15 indicate that based on model P predictions except for the top and bottom ends of column 1S (1S-B and 1S-T), all other columns had \( \frac{P}{P_{CL}} \leq 0.5 \) which qualified them to be considered as displacement controlled for the flexural actions. The average of the maximum plastic rotation ratio for these columns were 0-0.1 which was well below the 1.2 limit for the LS and CP performance levels. The flexural actions at 1S-B and 1S-T locations should be considered as force controlled and therefore, their average maximum P-M ratio should be less than 1.0. This was well satisfied at the bottom of the first storey column under the DBE and MCE ground motions (Max. P-M ratio = 0.61-0.62) and almost satisfied for the top of the column under the MCE ground motion (Max. P-M ratio = 1.03). Therefore, based on the model P predictions, all of the SCBF columns satisfied both the LS and CP performance levels under the DBE and MCE ground motions. The results presented in Table 5.15 indicate that based on the model F predictions except for 1S-B and 1S-T locations, all other columns had \( \frac{P}{P_{CL}} \leq 0.5 \) which qualified them to be considered as displacement controlled for the flexural actions. Among these columns, the average of the maximum plastic rotation ratio at 1N-B was 2.3 under the MCE ground motions which was larger than the 1.25 limit. On the other hand the maximum P-M ratio for this column was 1.17 which is above 1.0. Therefore, the flexural actions at 1N-B under the MCE ground motions did not satisfy the LS and CP limit state criteria. The flexural actions at 1S-B and 1S-T locations should be considered as force controlled and therefore, their average maximum P-M ratio should be less than 1.0. This was well satisfied at the top of the first storey column under the MCE ground motions (Max. P-M ratio = 0.79) but it was not satisfied for the bottom of the column under the DBE and MCE ground motions (Max. P-M ratio = 1.08-1.19). Therefore, based on model F predictions, the flexural actions at 1S-B under the DBE and MCE ground motions did not satisfy the LS and CP limit state criteria.

The shear response of the columns is considered force controlled. The maximum shear force demand to capacity ratios in the SCBF columns were 0.3 and 0.5, respectively, under the DBE and MCE ground motions. Therefore, the shear capacities of the columns were well above the maximum shear demands and no brittle shear failure was expected in the columns.
Figure 5.59: Column yielding locations and plastic rotation ratio ($\theta_p/\theta_y$) values in the SCBF for models P and F under MCE ground motions No. 1-8 (values of plastic rotation ratio for locations with negligible yielding are not shown)

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Figure 5.60: Column yielding locations and plastic rotation ratio ($\theta_p/\theta_y$) values in the SCBF for models P and F under MCE ground motions No. 9-16 (values of plastic rotation ratio for locations with negligible yielding are not shown)

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The maximum rotations of the beam-column shear tab connections were evaluated. The maximum and the average rotations at the beam-column connections over all of the ground motions are presented in Table 5.16. The results show that the average and maximum rotations under the MCE ground motions were almost twice the rotations under the DBE ground motions. The maximum connection rotation was 0.062 rad. At this rotation, significant damage occurs in the floor concrete slabs and the connection potentially experiences strength degradation. However, the maximum rotation was still much smaller than the 0.16 rad rotation capacity of the shear tab connections. It should be noted that the damage due to the low-cycle fatigue was not considered in the connection models but it was not expected to cause significant strength degradation in the beam-column connections at these rotation levels. Figure 5.61 shows the model P moment-rotation hysteresis of the first floor north beam-column connection under ground motion No. 2 at the DBE and MCE levels. The hysteretic curve shows that the connection underwent significant nonlinear deformations, particularly during the MCE ground motion. The hysteretic curve also indicates that the nonlinear deformations were mainly caused by the negative moments in the connection.

### Table 5.15: Average of maximum response of the columns with considerable flexural-axial nonlinear deformation (see Figures 5.59 and 5.60 for the location of the columns)

<table>
<thead>
<tr>
<th>Location</th>
<th>1N-B</th>
<th>1N-T</th>
<th>1S-B</th>
<th>1S-T</th>
<th>2S-T</th>
<th>3N-S</th>
<th>3N-T</th>
<th>3S-S</th>
<th>3S-T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hazard level</td>
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<td>MCE</td>
<td>MCE</td>
<td>DBE</td>
<td>MCE</td>
<td>MCE</td>
<td>MCE</td>
<td>MCE</td>
<td>MCE</td>
</tr>
<tr>
<td>$\left( \frac{P}{P_{CL}} \right)_m$</td>
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<td>0.50</td>
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<td>0.58</td>
<td>0.58</td>
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<td>0.49</td>
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<td>0.57</td>
<td>0.57</td>
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<td>0.41</td>
<td>0.41</td>
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<tr>
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<td>0.0</td>
<td>0.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.2</td>
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<td>0.54</td>
<td>0.95</td>
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<td>1.19</td>
<td>0.79</td>
<td>0.76</td>
<td>0.73</td>
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### Table 5.16: The maximum and the average rotations at the beam-column connections over all of the ground motions

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<th>Max rotation</th>
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<tr>
<td>MCE</td>
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<td>F</td>
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<td>0.057</td>
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5.8.4 Discussion of the SCBF performance

The following is the summary and discussion of the seismic performance of the SCBF based on the numerical analyses presented in this study:

- The SCBF experienced rather a large transient and residual SDR. The average SDR values, however, were within the acceptable range for the LS and CP performance levels, respectively, under the DBE and MCE ground motions.

- The magnitudes of the transient SDR during both the DBE and MCE ground motions suggest that the SCBF experienced significant damage in the non-structural elements like the partitions. The large residual SDR during the MCE ground motions suggests that although the SCBF did not collapse during these ground motions, it potentially experienced significant structural damage requiring a considerable amount of repair. In some ground motions, the residual SDR reached values above 2% which suggest that the SCBF might not be economical to be repaired and most probably should be demolished after those earthquakes.

- The SCBF floors experienced rather large accelerations during both DBE and MCE ground motions with an acceleration amplification on the top floor level. Large floor accelerations are a characteristic of the seismic response of the braced frames and are due to their rather large lateral stiffness. The large floor accelerations can have potential implications on the comfort of the building occupants or the safety of the sensitive structural content.

- All of the braces in the first four storeys of the SCBF started to buckle during the DBE ground motions. The braces in the first three storeys though experienced significant buckling and a large amount of nonlinear deformation. This can potentially cause damage to the partitions and structural contents due to the out-of-plane deformations of the braces after their buckling even during the DBE level earthquakes. In addition, this also indicates that the brace gusset plates will experience large deformations and should be designed to accommodate significant nonlinear response even for not very rare earthquakes.

Figure 5.61: Model P moment-rotation hysteresis for the first floor north beam-column connection under ground motion No. 2 at the DBE and MCE levels.
- The braces experienced large displacement ductility demands, especially in the first three storeys. None of the braces experienced initiation of fracture during the DBE ground motions. A few of the braces fully fractured at the end of the MCE ground motions. The full fracture of these braces mainly occurred in the model P first storey. However, the full brace fractures did not cause any instability or collapse in the SCBF in all cases. The braces experienced nonlinear deformation both in tension and compression although the vertical deflections of the floor beams limited the tensile deformations of the braces. The compressive displacement ductility demands were larger than the tension ductility demands. All the braces satisfied the LS and CP acceptance criteria for the displacement ductility of the braces, respectively, under the DBE and MCE ground motions. The only exceptions were model P first storey braces which did not satisfy the CP acceptance criteria under the MCE ground motions.

- Some of the columns started to yield and experience nonlinear deformations at their ends. The yielding mainly occurred during the MCE ground motions and in the first storey columns. Model P predicted yielding at the top ends of the first storey columns. However, the amount of yielding in these columns were all within the acceptable range for the LS and CP performance levels under both DBE and MCE ground motions. Model F predicted the largest nonlinear demands at the base of the first storey columns. Based on model F predictions, the nonlinear deformations and axial-flexural capacity at the first storey columns were beyond the acceptable range for both the LS and CP performance levels under the DBE and MCE ground motions. This was despite the fact that the columns were capacity designed to stay elastic for the expected axial forces in the braces. The formation of plastic hinges and yielding in the column ends were due to the flexural actions caused by the storey drifts which were neglected in the capacity design of the columns. These effects were intensified following the fracture of the braces. The results, however, suggest that they can cause significant plastic rotation demands in the columns depending on the flexural response of the baseplates or the amount of SDR. Such demands can potentially cause the collapse of the SCBF if they are not carefully evaluated and accounted for in the capacity design of the columns. Model F results suggest that the SCBF column section sizes should be slightly increased to safely accommodate the nonlinear flexural demands at the column baseplates. This is necessary to avoid potential collapse of the building due to excessive yielding and failure of the first storey columns during the DBE and MCE ground motions. The column yielding observations in this study are consistent with observations in the literature including the numerical studies by Tremblay and Robert (2001) and the experimental shaking table tests by Okazaki et al. (2012).

- Significant variability was observed in the deformation response of the SCBF especially for the residual drifts with coefficients of variations in the range of 54-103%. In some cases, like the model P maximum transient/residual SDR and the brace compression ductility, the average response satisfied the acceptance criteria but the 84-percentile of the response did not fall within the acceptable range. The response variability should be evaluated and considered when evaluating the SCBF seismic performance based on nonlinear response history analysis results.

- The assumption adopted for modelling the SCBF column baseplates had a considerable impact on the global and local response parameters of the SCBF. Assuming a fully fixed model (model F) compared to a fully
pinned model (model P) resulted in smaller SDR at the first storey and smaller axial demands on the first storey braces at the expense of imposing significant flexural demands at the column baseplates. It also affected the soft storey response and damage concentration pattern in the SCBF. While a fully pinned assumption predicted a soft storey mechanism and concentration of damage and the peak responses in the first storey, a fully fixed assumption resulted in a more uniform distribution of damage and maximum response quantities in the building height resulting in less damage to the first storey and more damage to the third storey compared to the fully pinned assumption.

The SCBF SDR values and performance observed in the numerical study presented in this study were consistent with other system-level numerical and experimental investigations reported in the literature as discussed in section 5.2 like the shaking table tests by Okazaki et al. (2012), the pseudo-dynamic tests by Tsai et al. (2013), the numerical investigations by Tremblay and Robert (2001) and Huang and Lignos (2017), etc. A comprehensive summary of the SDR values observed in some of the previous investigations are presented in Tables 2.1, 2.2, and 2.3 in the PhD dissertation report by Fell (2008).

Based on the analysis performed in this study on the SCBF for the earthquake records and hazard levels considered, the SCBF will undergo extensive damage under rather rare earthquakes but the life safety and collapse prevention is assured which implies that the SCBF remains safe for the occupants under the design basis earthquakes to evacuate the building but cannot be safely used again without significant repair. Therefore, if the building is not economically and functionally important, the design could be acceptable. In this case, the structure requires extensive repair or may not be reused and may have to be demolished after strong and rare earthquakes. For such earthquakes, the facilities in the structure may be completely damaged.

5.9 SUMMARY AND CONCLUSIONS

This chapter presented hybrid simulations and seismic performance assessment of a five-storey SCBF with braces in stacked chevron configuration. Development, design, and cyclic response assessment of a buckling specimen that simulates the axial force-displacement hysteresis of conventional braces was presented in detail. The buckling specimen was used in multi-element hybrid simulations in UT10 on the 5-storey SCBF. The hybrid simulations verified the performance of UT10 for multi-element hybrid simulations. The results also confirmed the satisfactory accuracy of the adopted 2D brace numerical model which was then used in a fully numerical seismic response and performance assessment of the SCBF in OpenSees. The following is the summary and the conclusions that can be made from the investigations presented in this chapter:

- The review of the past literature on the seismic performance of SCBFs revealed that despite the increasing popularity of the SCBFs and the recent developments in ductile design and numerical modelling of these systems, the experimental results on the system-level response of multi-storey SCBFs with the most recent ductile designs seem to be insufficient. More experimental investigations are necessary to achieve a realistic understanding from the seismic performance of the SCBFs and also to verify the capabilities of the existing
numerical models to realistically predict the seismic performance of multi-storey SCBFs. Multi-element hybrid simulations on multi-storey SCBFs greatly facilitate this process.

- A scaling strategy was proposed and adopted based on maintaining the effective slenderness ratio of the conventional braces based on which a buckling specimen with simple geometry was designed to simulate the hysteretic response of full-scale conventional braces. The cyclic test results on the buckling specimen in UT10 showed that the buckling specimen is capable of producing an axial force-displacement hysteretic response that is identical to the response of full-scale design braces.

- The hysteretic response of one of the SCBF design braces in the first storey with the end gusset plates was modelled once with a 3D continuum FE model and once with a 2D fibre-based multi-element model. The numerical responses were compared to the cyclic test results on the buckling specimen scaled up to represent the full-scale design brace response. The results showed that the 3D continuum FE model and the buckling specimen hysteresis were nearly identical confirming the accuracy of the scaling strategy, the performance of the UT10 in capturing the cyclic complex buckling response of the buckling specimen, and the ability of the buckling specimen to simulate the hysteretic response of the full-scale conventional braces. The calibrated 2D model was less accurate than the 3D continuum FE model for the compressive strength and post-buckling response transitions mainly due to the multi-axial stress effect that was not captured in the 2D model. The errors for the 2D model were in the range of 21-33% for the initial and residual compressive strengths of the brace. The overall hysteresis of the 2D model showed an excellent agreement with the test results.

- Multi-element (4-element and 2-element) hybrid simulations were performed in UT10 on the SCBF with buckling specimens physically representing the braces in the first two storeys and the results in terms of storey drifts, floor accelerations, energy dissipation, and the braces axial displacement ductility demands were compared to the fully numerical model predictions with the 2D fibre-based multi-element brace model. The comparison of the results indicated that the numerical model was capable of predicting the component- and system-level response of the conventional braces during an earthquake with sufficient accuracy. Both the fully numerical and the hybrid simulations predicted similar distributions of drift and energy dissipation in the SCBF. They both showed significant nonlinear deformation demand and energy dissipation in the first storey braces and predicted the formation of soft-storey in the first storey of the SCBF. The fully numerical and hybrid simulations both predicted a maximum SDR of 2.2%, large floor accelerations, and acceleration amplification on the top floor level for both ground motions considered. The fully numerical tension and compression ductility demand predictions were on average 20% and 8% smaller than the hybrid simulation results, respectively. The 4-element hybrid simulations did not show significant improvement in the accuracy of the response predictions compared to the 2-element SCBF tests and the fully numerical model. This observation indicates that the brace numerical model accuracy in the second storey braces was less affecting the global response of the SCBF for the ground motions considered. This was mainly because of the high accuracy of the brace numerical models and also the accumulation of the major nonlinear deformations in the first storey braces of the SCBF which were physically tested in both hybrid tests.
- The results of cyclic tests on the buckling specimen and the multi-element hybrid simulations on the SCBF proved that the UT10 performed well in capturing the complex hysteretic response of the buckling specimen. Detailed analysis of the hybrid simulation results confirmed that the new implementations in NICON-10 after the 3-element tests on the BRBF successfully eliminated the extra stress relaxation in the specimens and the overshootings of the actuators which were observed in the 3-element tests on the BRBF. The results confirmed the performance of UT10 for running hybrid simulations with 4 specimens.

- The seismic response and performance of the SCBF were numerically investigated using the same numerical model that was verified by the hybrid simulations. The fully numerical analyses were performed using two SCBF OpenSees models with two different boundary conditions for the column baseplates under 16 ground motions at two seismic hazard levels corresponding to the DBE and MCE hazard levels for Los Angeles. The results indicated that the damage distribution prediction and the failure mechanism can be largely affected by the modelling assumption adopted for the column baseplates. The results suggest that in order to gain a realistic understanding from the actual seismic performance of the SCBF, damage concentration, and potential failure mechanisms, one needs to employ a realistic model for the flexural response of the column baseplates or to conduct a sensitivity analysis to capture all possible seismic responses of the SCBF. The performance of the SCBF was assessed at the local and global levels. The results indicated significant buckling of the braces in the first three storeys and thus potential extensive damage even during the DBE ground motions. The results also indicated that the SCBF experienced rather a large transient and residual SDR and high levels of floor acceleration under both DBE and MCE ground motions. Considerable nonlinear deformations were observed in the columns especially during the MCE ground motions which can potentially cause the collapse of the SCBF if they are not carefully evaluated and accounted for in the capacity design of the columns.

- While significant scatter was observed for the deformation response parameters of the SCBF, particularly for the storey drifts, for the ground motions considered, in most of the cases the SCBF satisfied the performance criteria for life safety under the DBE ground motions and collapse prevention criteria under the MCE ground motions. In either case, the SCBF requires extensive repair or may not be reused and may have to be demolished after strong and rare earthquakes. For such earthquakes, the facilities in the structure may be completely damaged.
CHAPTER 6: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY AND CONCLUSIONS

As the need for a realistic understanding of the behaviour and accurate prediction of the seismic performance of SFRS is increasingly mandated by advanced seismic design and assessment standards, hybrid simulation as a method that efficiently integrates realistic experimental data into the structural response predictions is becoming highly popular for seismic performance assessment of SFRS. In this context, development of the UT10 is a step forward in facilitating hybrid simulations on multi-storey braced frames by providing a customizable, reconfigurable, and user-friendly platform for performing hybrid simulations with up to 10 physical brace specimens. General summary and conclusions from this study are presented in the following. More detailed and specific conclusions about each aspect of the study are presented in the summary and conclusions section of the corresponding chapter.

6.1.1 Development of UT10 and hybrid simulations

- Performance of UT10 was evaluated through several tests under cyclic loads and actual ground motions with three types of brace specimens. These tests confirmed that the UT10 control and communication system operates flawlessly and the error compensation algorithm reduces the errors to the desired tolerance limits. 1-, 2-, 3-, and 4-element hybrid simulations were successfully performed on a 5-storey BRBF and a 5-storey SCBF through the network communication established between the numerical modelling platform and the actuator control system via UT-SIM and NICON-10. Following the 3-element BRBF test, further issues and challenges for testing multiple specimens in UT10 were identified and further improvements and developments were implemented in the error compensation algorithm which were successfully tested in the 2-element and 4-element SCBF tests.

- The AYB specimen was designed, fabricated, and successfully tested in UT10 under both cyclic and earthquake loads. For both types of loads, AYB showed stable and repeatable hysteretic response both in tension and compression. It was also able to reproduce some of the features of the BRB hysteretic response like the friction effect, isotropic hardening, and the Bauschinger effect thus providing a hysteretic curve similar to a full-scale BRB. The core rods of the AYB specimens were replaced after each test and the AYB specimens were reused in multiple hybrid simulations.
A buckling specimen was designed and fabricated to simulate the hysteretic response of conventional braces following a scaling strategy. The scaling strategy was based on maintaining the effective slenderness ratio of the conventional braces and was successfully verified via 3D continuum FE simulations. The cyclic and hybrid simulation test results on the buckling specimen in UT10 showed that the buckling specimen is capable of producing an axial force-displacement hysteretic response that is identical to the response of full-scale design braces under both cyclic and earthquake loads.

Multi-element hybrid simulations were performed on a 5-storey BRBF and a 5-storey SCBF. The results in terms of storey drifts, floor accelerations, energy dissipation, and the braces axial displacement ductility demands were compared to the fully numerical model predictions. The investigations revealed how the global and local response of the multi-storey braced frames can potentially be affected by small modelling details in the hysteretic behaviour of the bracing elements. The impact of increasing the number of physical specimens on realistic response prediction of the braced frames was also observed. The fully numerical models used brace models that were well calibrated to the results of cyclic tests on the physical specimens with ramped cyclic loading protocols. Therefore, the differences between the fully numerical and hybrid simulation results, especially for the BRBF, were not a result of poor calibration but were rather due to inherent limitations in the numerical models. The latter could be only observed during irregular cyclic earthquake loads. This observation elaborates the fact that accurate calibration of the numerical models based on ramped cyclic tests results does not necessarily guarantee their accuracy during actual earthquake loads and more thorough loading protocols representative of real earthquakes are necessary to ensure sufficient accuracy is achieved by the numerical models. The above observations show how hybrid simulation test results can aid in identifying limitations in existing models, enhancing our understanding of the seismic response of SFRSs, and identifying the most appropriate models that can accurately predict the response of the SFRSs under any loading protocol.

6.1.2 Potentials and limitations of hybrid simulations using UT10

- Development of UT10, the AYB specimen, and the buckling specimen greatly facilitate hybrid simulations on multi-storey braced frames. Using UT10 with the AYB and buckling specimens potentially enables performing multiple hybrid simulations on braced frames under a suite of ground motions which in turn helps in more realistic assessment and calibration of existing modelling techniques for braces. In addition, UT10 can be used to develop a realistic understanding from the actual performance and response of newly developed innovative bracing systems under earthquake loads.

- The network communication capability of UT10 makes it possible to use UT10 in geographically distributed hybrid simulation projects in future thus enabling this unique equipment to be shared in domestic and international collaborative projects.

- It is of prime importance to consider the range of response of the braced frames that are tested with UT10. While UT10 can provide a realistic prediction of the response of braced frames under design level earthquake loads, the hybrid simulation results can potentially lose accuracy when the near collapse
response of the braced frames is considered. In near collapse range of response, parameters like the fatigue failure of the beam-column-brace connections, the fatigue failure in the braces and their connections, the effect of floor slabs, and the contribution and failure of the gravity columns can potentially dominate the response and performance of braced frames. In such circumstances, in order to obtain a realistic response prediction, rigorous numerical models should be adopted in the numerical substructure to accurately capture such effects. Significant advances are made in the simulation of fatigue failure using techniques based on continuum FE method (Fell, 2008; Martyr and Christopoulos, 2018). If properly calibrated, such techniques can predict the fatigue failure of structural components with a high level of accuracy and thus can facilitate realistic seismic performance assessment of structures. In this context, UT-SIM framework can be potentially used to integrate numerical models of various complexity made in different numerical modelling platforms with the experimental test results through a distributed multi-platform hybrid simulation (Huang and Kwon, 2018). In addition, if fatigue failure of the design braces is expected under the earthquake loads, the fatigue response of the physical specimens should be representative of the actual response of the design braces to properly capture the limit states corresponding to the potential low-cycle fatigue failure in the braces.

- The current configuration of UT10 allows testing brace specimen with a maximum length of 1,660 mm and with a maximum strength of 800 kN which limits the size and capacity of the specimens that can be tested in UT10. The latter requires scaling of the specimens in some applications. The maximum available length can, however, be potentially increased to 3,160 mm if the bottom actuators are fully removed from the UT10 setup and the specimens are directly connected to the reaction frame at their bottom (see Figure 2.1). The maximum capacity of the specimens can also be increased to 1,600 kN if two actuators are used simultaneously to load a single specimen. However, to achieve this, further developments in the displacement error compensation algorithm should be made to enable coordination of error compensation between the two actuators loading a single uniaxial specimen.

### 6.1.3 BRBF and SCBF seismic performance

- The seismic response and performance of the BRBF were evaluated under a suite of 16 spectrum compatible ground motions at two different hazard levels. The material models for the BRB yielding cores were calibrated to the AYB cyclic test results. The results indicated that the BRBF experienced large storey drifts and failed to satisfy the drift limits for the life safety and collapse prevention performance levels even under the design bases earthquakes. However, the local response of the beams, columns and BRBs were within the acceptable range. Most of the seismic energy was dissipated in the BRBs especially in the first storey of the BRBF. The BRBF showed the concentration of damage and potential for formation of soft storey mechanism in the bottom storeys. The excessive lateral deformation of the BRBF was primarily a result of low post-yield stiffness of the BRBs. Large drifts were observed despite the fact that the stiffening effects of the beam-column-brace connections and the lateral stiffness of the gravity support system were explicitly considered in the numerical analyses. The seismic performance of the building can
be potentially improved if the BRBs are used in combination with backup structural systems that are able to provide lateral stiffness for the SFRS after yielding of the BRBs. The latter can be for instance achieved by using the BRBs in a dual system such as a BRBF-MRF.

- The seismic performance analysis of the SCBF under a suite of 16 spectrum compatible ground motions including pulse type and no-pulse type earthquakes, revealed that in most of the cases the SCBF satisfied the performance criteria for life safety under the DBE ground motions and collapse prevention criteria under the MCE ground motions. Formation of soft-storey mechanism, significant ductility demands and buckling of the braces, large storey drifts, and yielding of the columns were observed under both DBE and MCE ground motions which imply the need for extensive repair or complete demolish of the SCBF after strong and rare earthquakes. The results were sensitive to the baseplate modelling assumptions which indicates the necessity to employ realistic models for the baseplate or to conduct a sensitivity analysis to capture all possible seismic responses of the SCBF.

### 6.1.4 Numerical modelling limitations

The following modelling limitations should be considered when analyzing the BRBF and SCBF performance based on the numerical results presented in this study:

- The numerical models adopted for the BRBF and SCBF in this study was two dimensional. This implies that the torsional effects, bidirectional seismic loading, the effects of the out-of-plane forces and deformations on the structural elements, and the contribution of the MRF in the perpendicular direction (see Figures 4.3 and 5.12) were neglected. Since the plan of the building was symmetric and the braced frames and MRF did not share any columns, these effects were assumed not to have a major impact on the in-plane response of the BRBF or SCBF. However, a more accurate and complete performance assessment requires consideration of the above in the numerical model.

- The effect of the gravity columns was considered by continuous elastic leaning columns in the BRBF and SCBF models in this study. While this is a common approach to take into account the $P - \Delta$ effect of the gravity loads in the model, it may not model the actual lateral strength contribution of the gravity columns. Several studies have shown that the gravity columns can potentially reduce the storey drifts and increase the reserved capacity of the SFRSs at their near collapse response phase (Gupta and Krawinkler, 1999; Flores et al., 2014; Elkady and Lignos, 2015). In addition, since the gravity columns are majorly responsible to carry the gravity loads, their actual response and performance should be evaluated when assessing the near collapse response and performance of the entire building. It is therefore recommended to use realistic connection models between the gravity columns and the floor beams and to use nonlinear elements to model the gravity columns in order to properly capture the potential nonlinear response and damage in the gravity columns when exposed to large lateral deformations during the MCE ground motions. The continuous elastic model used for the gravity columns in the BRBF and SCBF model in this study can to some extent capture the lateral contribution of the gravity columns. However, it is recommended to use more rigorous techniques like the modelling techniques proposed by Gupta and Krawinkler (1999) and adopted by Hwang.
and Lignos (2017) to more realistically capture the beneficial influence of gravity framing on limiting the storey drifts especially during MCE ground motions.

### 6.2 FUTURE WORK AND RECOMMENDATIONS

The work completed as part of the research project presented in this report enabled, for the first time, multi-element hybrid simulations with a large quantity of large capacity uniaxial physical specimens at the Structural Testing Facility of the University of Toronto. Several projects are planned at the University of Toronto that will be using UT10 or a modified version of it. These projects are the following:

- A project involving 10-element hybrid simulations on BRBFs with 10 AYB specimens representing the BRBs
- A project involving 4-element hybrid simulations on a 5-storey steel frame equipped with full-scale cast steel yielding brace systems (YBS) (Gray et al, 2014)
- A project involving 4-element hybrid simulations of eccentrically braced steel frames with full-scale replaceable cast steel link elements (Tan and Christopoulos, 2016). This project requires further developments of UT10 to enable application of combinations of axial/shear/bending forces to the specimens.

The study presented in this report paved the way for future developments, improvements, and investigations. Below are some recommendations and aspects of this study that can be implemented and further investigated in the future:

- As discussed in Chapter 2, the target displacement commands were applied to the actuators in a ramp and hold fashion. The hold times were necessary to let the system stabilize and the actuators to settle to a displacement. However, as discussed in Chapter 3, Section 3.4.4.2.2, the hold time caused stress relaxation. The stress relaxation was observed to be maximum of 1.3 % of the total axial force in the specimen and its effect was negligible. However, it increased the simulation time to some extent. To avoid these issues, the loading procedure in NICON-10 can be further improved in future to employ more advanced continuous algorithms (continuous hybrid simulation) (Magonette, 2001; Mosqueda et al., 2008) to eliminate the hold times during hybrid simulations.
- As discussed in Chapter 2 an error compensation algorithm is implemented in NICON-10 to correct for the displacement errors and avoid both the displacement undershooting and overshooting during multi-element hybrid simulations. A potentially more efficient solution to avoid both undershooting and overshooting of the specimens is to use the displacements directly measured from the specimens instead of actuator displacements as the feedback for the PID control loops in the MTS controller. However, this approach can be unsafe as there will be no direct control over the actuator movements during a test. A better solution can be achieved by implementing PID control loops in NICON-10 which use the displacements directly measured from the specimens as the feedback to correct the displacement commands that are sent to the MTS controller. These PID control loops in NICON-10 will act on top of
the PID control loops in the MTS controller which directly control the movements of the actuators. It is planned to verify this solution in future projects.

- As mentioned before, in order to enable the 1,600 kN loading capacity of UT10 for a single uniaxial specimen, further developments are necessary for NICON-10 to coordinate the displacement error compensation between two actuators that load a single uniaxial specimen. These improvements can be implemented in future versions of NICON-10.

- The AYB core rods were supplied from steel threaded rods that were fabricated following the requirements established for fastener steel material. These requirements allow large variations and overstrength in the yield strength of the material and a rather low ductility capacity for the steel material. In order to ensure limited strength variability and large ductility capacity for the AYBs for future hybrid simulations, it is recommended for future tests to supply the rods from steel materials with more controlled strength properties and larger ductility capacity. Furthermore, it is recommended to cut the core rods from bars that are selected from the same batch fabricated by the same supplier. Cutting the bars from a single bar would be the best approach to ensure that the variability of the mechanical properties is minimized among the core rods.

- Further improvements can be implemented in the design of AYB to use plain rods instead of threaded rods to avoid residual stresses resulting from threading procedure in the rods which can potentially contribute to lower ductility capacity and early failure of the threaded rods. This together with adopting more controlled steel material properties can result in more ductility capacity for the AYBs with plain core rods. However, a threaded connection may not be any more feasible in the AYBs with plain core rods. Instead, connections based on gripping of the core rods and friction between the grips and the core rods can be adopted.
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Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto


Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto


APPENDIX A: UT10 OPERATION CHECKLIST

Prepared by Saeid Mojiri and Pedram Mortazavi
Ph.D. candidates at the Department of Civil and Mineral Engineering of the University of Toronto

This appendix contains a sample checklist for running a 4-element hybrid simulation with UT10. The checklist also includes the protocols for resuming an interrupted test, handling unexpected problems during a test, and managing the emergency shutdowns of the hydraulic system. The steps were developed, implemented, optimized, and successfully verified through several single- and multi-element hybrid simulations with UT10. Details of the steps may need to be changed based on the type of the simulation and specimens. The user is advised to use these steps with his own discretion and directly consult with the laboratory technicians for running any tests. The steps were developed based on the authors’ experience with UT10 and in close collaboration with previous users of the SET and the laboratory technicians. The authors particularly benefited from generous advice from Dr. Giorgio T. Proestos, Dr. David M. Ruggiero, and Professor Evan C. Bentz for the development of this checklist.

TEST

1. Make sure no object (e.g. concrete pieces, etc.) exists between the actuator clevises and cylinders and the actuators can be fully retracted if necessary.
2. Check alignment of the actuator string potentiometers.
3. Make sure the connections of the external instrumentation on the specimens are not loose.
4. Check the racks and make sure they allow for free movement of the actuators in the desired direction.
5. Disconnect the hoist chains from the frame, and lift the free hoists.
6. Check alignment of the frame.
7. Turn on FlexTest controller.
8. Turn on UT10 control platform.
9. Turn on the power supply controller, turn the voltage to zero. Afterwards, turn on the power supply.
10. Turn on the server and client computers.

11. Run System Loader on the server computer. Wait until a full network connection is established between the two computers.

12. Run MTS AeroPro software on the client computer.


16. Make sure the RJ50 input and output cables are connected to the UT10 control platform and make sure they are not loose.

17. In order to facilitate the execution of steps during different phases of the test, make the following actuator groups:

   A. **Nonconnected Actuators:** This group will contain all the actuators that are not connected during the test. In Four-Element tests, this group will contain actuators 16, 17, 19, 21, 23, 25, 26, 27, 28, 29, 30, 32, 33, 34, 37, 38, 39, 42, 43, 44, 47, 48, 49, 52, 53, 54, 57, 58, and 59.

   B. **Horizontal Connected Actuators:** This group will contain all the horizontal connected actuators, both in-plane and out-of-plane ones as indicated in Figure 1. In the Four-Element test, this group will contain actuators 11, 12, 13, 14, 15, 31, 35, 36, 40, 41, 45, 46, 50, 51, 55, 56, and 60.

   C. **Vertical Bottom Actuators:** All the bottom actuators will be connected to the test frame in all tests. Specifically, this group will contain actuators 1, 2, 3, 4, 5, 6, 7, 8, 9, and 10.

   D. **Vertical Top Connected Actuators:** This group will contain the actuators above the frame that are connected to the active or inactive specimens.

      As an example, in a One-Element Test where the specimen is being loaded through actuator 18, and two loading shafts are connected to actuators 22 and 24 (whether there are inactive specimens or no specimens at all), this group will contain actuators 18, 22, and 24.

   E. **Vertical Top Actuators:** This group will contain all the top actuators. Specifically, this group will contain actuators 16, 17, 18, 19, 20, 21, 22, 23, 24, and 25.

   F. **Loading Actuators:** This group will contain the actuators above the frame that are connected to the active specimens during the test. As illustrated in Figure 1, in the Four-Element test, this group will contain actuators 18, 20, 22, and 24. Make sure the stop at level is checked for the actuators in this group.

   G. **Horizontal Rigid Links:** This group will contain the displacement-controlled horizontal actuators, acting as rigid links to ensure the stability of the frame during the test. As illustrated in Figure 1, in the Four-Element test, this group will contain actuators 12, 14, 36, 40, 45, 51, 55, and 60.
H. **Vertical Rigid Links:** This group contains the vertical displacement-controlled actuators that act as rigid links. These actuators are used for moving the frame during the lifting phase and act as supports during the testing phase. As illustrated in Figure 1, in the Four-Element Test, this group will contain actuators 2 and 4.

I. **Connected Nonslaved Force-Controlled Actuators:** In the Four-Element test, as illustrated in Figure 1, this group will contain actuators 1, 6, 8, 10, 11, 13, 15, 31, 35, 41, 46, 50, and 56.

J. **Slaved Force-Controlled Actuators:** This group will contain all the vertical force-controlled actuators whose force commands slave to their corresponding Loading Actuators. As illustrated in Figure 1, in the Four-Element test, this group will contain actuators 3, 5, 7, and 9. Make sure stop at level is checked for the actuators in this group.

K. **Connected Actuators:** This group will contain all the connected actuators. These actuators are shown by black colour numbers in Figure 1.

L. **Rigid Link Actuators:** This group will contain all the actuators that act as rigid links, as shown in Figure 1.

M. **Top Connected Inactive Actuators:** This group will contain top actuators with loading shafts that are connected to inactive specimens or have no specimens during the test. This group must be double-checked for each test.

In a One-Element Test where the specimen is being loaded through actuator 18, and two loading shafts are connected to actuators 22 and 24 (whether there are inactive specimens or no specimens at all), this group will contain actuators 22 and 24.

18. The test will have five different phases: **Verification, lifting, locking, loading, unloading.**

**Verification:**

19. In this phase, the configuration of the actuators is as shown in Figure 1 where R indicates the rigid links. The rigid links are displacement-controlled [2], the top loading actuators are displacement-controlled by external signals [SBC 2 slaved to 3], and the rest of the actuators are force controlled [1]. The slaved force-controlled actuators are shown by circles.

20. Check the control modes for all actuators. Note that for a previously defined and used test file, this check is not necessary.

21. Check the set points for all actuators.

- The force setpoint for **Nonconnected Actuators** should be 15 kN to fully retract them.

- The force setpoint for all **Connected Nonslaved Force Controlled Actuators** should be 0 kN, for horizontal ones, and -3 kN for bottom vertical actuators.

- The force setpoint for all **Top Connected Inactive Actuators** should be +3.0 kN.

- The force setpoint for **Slaved Force Controlled Actuators** should be -6 kN.
- The displacement setpoint for all *Rigid Link Actuators* should be their actual displacement readings.

- The displacement setpoint for all *Loading Actuators* should be zero.

22. Make sure the “stop at level” boxes are checked for *Loading Actuators* and *Slaved Force Controlled Actuators*.

23. Make sure displacement limits for *Nonconnected Actuators* are off (both [2] and [3]), and the force limits are on. The force limits should be +/- 20 kN for the inner limits and +/- 30 kN for the outer limits.

24. Set the load limits for *Connected Actuators* to +/- 50 kN, for their inner limits, and to +/- 60 kN, for their outer limits.

25. Set the load limits for *Top Connected Inactive Actuators* to +/- 15 kN, for their inner limits, and to +/- 20 kN, for their outer limits.

26. Set the load limits for *Loading Actuators* and the *Slaved Force Controlled Actuators* to +/- 60 kN, for their inner limits, and to +/- 70 kN, for their outer limits.

27. Set the load limits for *Vertical Rigid Links* to +/- 70 kN, for their inner limits, and to +/- 80 kN, for their outer limits.

28. Set the displacement limits [2] of the actuators as specified in Table X, plus their current reading:

<table>
<thead>
<tr>
<th>Actuators</th>
<th>Inner Limits</th>
<th>Outer Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Connected Actuators</td>
<td>+/- 5.0 mm</td>
<td>+/- 7.0 mm</td>
</tr>
<tr>
<td>Vertical Top Connected Actuators</td>
<td>+15.0, -5.0 mm</td>
<td>+20.0, -8.0 mm</td>
</tr>
<tr>
<td>Vertical Bottom Actuators</td>
<td>-15.0, 5.0 mm</td>
<td>-20.0, +8.0 mm</td>
</tr>
</tbody>
</table>

29. Set the displacement limits [3] for *Vertical Top Actuators* to +/- 250 mm.

30. Make sure both the displacement and force limits are on for all *Connected Actuators*.

31. Enable the *Equilibrium3*, *Equilibrium5*, *Equilibrium7*, and *Equilibrium9* watch signals.

32. Set the limits for equilibrium watch signal to +/- 25 kN.

33. Make sure the detectors for inner and outer limits and also errors are enabled and reset.

34. Make sure the integrator is off.

35. Hit the *Stop* button, check it is working, and then remove the interlock first on the stop button and then in the test file.

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36. Run NICON-10 LabView script.

37. Check that NICON-10 is reading correct forces and displacement of *Loading Actuators*.

38. Make sure external instrumentation on the specimens are working properly and are read correctly by UT10 control platform. For this purpose, lightly tap on the external measurement connection rods and monitor the changes in NICON-10.

39. Turn on *EX Voltage Correction* in NICON-10. Adjust the voltage to read 10 volts.

40. Verify the voltage scale factor for the external power source in NICON-10. It should be very close to one (e.g. 1.0003).

41. In NICON-10, turn *Control* on.

42. Enable offsets in NICON-10. Then, offset actuator forces, displacements, and external measurements. Afterwards, execute the offset to make the desired command.

43. In NICON-10, verify the displacement and force signal noises. The noises should be around +/-0.02 mm and +/-0.2 kN for displacement and force measurements respectively.

44. Enable the Simulation Mode in Aeropro.

45. In the Execution tab, in Aeropro, set the Next End Level to 1, and click on Forward, without Data Acquisition.

46. Verify if AeroPro is reading the command from NICON-10 (shown as IN 3). Change command values in NICON-10 and check if AeroPro can see the changes. Note that small errors (i.e. +/- 0.02 mm) are acceptable.

47. Make sure that displacement command is equal to the measurement in NICON-10 for *Loading Actuators*. Offset the readings in NICON.

48. Verify that in AeroPro the setpoint for displacement command mode [2] is zero and the total external command [2] is equal to the current displacement measurement. (Total command = setpoint + input, and Feedback = Total Command).

49. Turn on the AeroPro button in NICON-10. The light should be green to indicate that communication for the digital signal is working.

50. Stop the Aeropro test file.

51. Ensure that the Aeropro button is automatically turned off in NICON-10. This verifies that NICON-10 switches from automatic to manual mode when an error occurs in Aeropro.

52. Uncheck the simulation mode in Aeropro.

**LIFTING:**

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53. Ensure that previous test data are retrieved. Reset the data acquisition recordings and sample counter from the Recorders tab.

54. Enable the external displacement limits [3] for Loading Actuators. The limits are similar to the internal displacement limits [2].

55. In the Execution tab in Aeropro set the load time (loading ramp duration) to 40 sec.

56. Once more, ensure that the set points are equal to displacement readings for rigid links.

57. For loading actuators, ensure that the total command is equal to the feedback.

58. Review the events log.

59. Set the Next End Level, in the Execution Tab, to 1.

60. Make sure the oil return valves are open and then open the oil supply valve.

61. Ensure that the booster pump is off.

62. Set the oil pressure to low.

63. Immediately press the forward button once to command the set points. Make sure data acquisition is enabled. Wait until the Pause button becomes stable green in AeroPro. Note that sometimes it might be required to press this button two times, as the program may not respond the first time.

64. Check for any oil leakage.

65. Check the loads, displacements and movements of all the actuators. Make sure no extra forces are added to any of the actuators especially for the Loading Actuators. If necessary, relax the actuators.

66. Turn on AeroPro communication button in NICON-10.

67. When all the movements stop, set the displacement limits for the Nonconnected Actuators as +/- 5.0 mm, and +/- 7.0 mm, for inner and outer limits plus current reading, respectively. This does not to be changed for test files that have previously been used.

68. Enable the displacement limits for Nonconnected Actuators.

69. Check and ensure that the limits for all actuators are enabled.

70. Set the load time (loading ramp duration) to 11 sec.

71. In NICON-10 re-offset all the signals and command current position of the Loading Actuators again.

72. In NICON-10, increase the maximum allowable slackness, in the network tab, to 25 mm.

73. In NICON-10 change ramp time to 12000 ms, and hold time to 1000ms.

74. In Aeropro, in Control Mode tab increase the displacement setpoint for the Vertical Rigid Links (C2 and C4) in small steps until the full weight of the frame in carried by them. Relax the load developed in the
Appendix A: UT10 Operation Checklist

Loading Actuators by reducing the displacement command in NICON-10 simultaneously. Displacement steps of 1.0 mm can be used.

- When the frame is completely lifted, the sum of the forces in the Vertical Rigid Links should be around \(-68\) kN. This is roughly the weight of the frame and half of the weight of all of the connected actuators.
- Make sure no significant extra load is developed in other rigid links.
- Note that you should add offset (existing + added) in the user commands in NICON-10.
- Make sure the loading actuator is in the slack at the end of lifting so that when high pressure is turned on no extra load is developed in the specimen.
- The compressive load associated with slack is \(-5.5\) kN and \(-3.0\) kN for the AYB and buckling brace specimens, respectively.

75. Check the alignment of the frame with level/lasers. If necessary, make an adjustment with the rigid links.

76. In NICON-10, change the ramp time back to 800 ms. Note that hold time is 400 ms.

77. Set the displacement limits for the Vertical Rigid Links (C2 and C4) to +/- 5.0 mm for inner and +/- 6.0 mm for outer limits, plus their current reading.

78. Set the displacement limits [2] and [3] for Loading Actuators to a small number (i.e. +/- 5.0 mm for inner and +/- 6.0 mm for outer limits), plus their current reading.

79. Set the slack limit in NICON-10 back to 7.0 mm, for AYB, and 25 mm, for BPS.

80. Offset the external displacement and forces for the loading actuators in NICON-10.

81. Enable the integrator.

82. Switch to high pressure. Note that in the high pressure state, the Aeropro should read roughly 2750 psi.

83. Watch the development of extra forces in the rigid links

LOCKING:

84. Watch the displacement limits for the Horizontal Rigid Links to make sure they allow for slight movement of these actuators in the locking process.

85. Change the displacement setpoints for the Horizontal Rigid Links such that slight tension and compression is developed in them as indicated in Figure 1:

   a. Develop slight tension in C40 and C55.
   b. Develop slight tension in C36 and C51.
   c. Develop slight compression in C14 and slight tension in C12.

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d. Develop slight compression in C45 and slight tension in C60.

86. Relax the specimen to make sure no extra force is developed in the specimen.

87. Make sure that the load balance and pattern is maintained based on figure 1 at the end of the locking process.

LOADING:

88. Set the displacement limits of the Loading Actuators, both for command [3] and feedback [2] to allow for +/-4 mm inner and +/-5 mm outer movement. Set the force limits to +/-150 kN inner and +/-170 kN outer.

89. Set the force limits for the Slaved Force Controlled Actuators, to +/-150 kN inner and +/-170 kN outer.

90. Turn on the booster pump by going to the Digital Outputs tab in AeroPro and clicking on the toggle icon for 01.

91. Make sure that the load balance and pattern is maintained based on figure 1 after the booster pump is turned on.

92. Watch of the development of extra forces especially in the specimen.

93. Read the pressure before and after the booster pump and verify if they make sense. The pressure before and after booster pump should read 2650.0 psi and 4320.0 psi, respectively.

94. Offset displacement and force measurements in NICON-10.

95. If needed, play with the Loading Actuator displacement setpoints in AeroPro to make sure the command and feedback are equal for them as read in NICON-10 (e.g.: A18: 0.06, A22: -0.03, A24: -0.04)

96. Change the force limits of Rigid Links to +/-300 kN and +/-350 kN for the inner and outer loops, respectively.

97. Turn on the Start Server button in NICON-10.

98. Run OpenSees model file.

99. Turn on Start Communication button in NICON-10.

100. In NICON-10, go to the ACTIA tab and turn on the ACTIA button.

101. Turn on the Start Server.ACT button in the ACTIA tab.

102. Reset the F-D graphs in NICON-10.

103. Write the time in AeroPro and NICON-10.

104. For a few first steps, execute the test manually using the Execute Target CMD button in NICON-10.

105. After a few steps, set the mode to Auto in NICON-10.

106. Set the error limit to 0.04 mm.

107. Change the ramp time to 800 ms and the hold time to 400 ms.
108. After a few steps increase the displacement and load limits of the Loading Actuators and force limits of Slaved Force Controlled Actuators.

109. Monitor the response of the specimens and the development of forces. Loosen the force and displacement limits for the Loading Actuators and force limits for Slaved Force Controlled Actuators and Rigid Links as necessary during the test.

110. After completion of the test, tighten the force and displacement limits as required. The limits must particularly be adjusted for Loading Actuators, Slaved Force Controlled Actuators, and the Rigid Links.

**UNLOADING:**

111. When the test finishes, write the AeroPro and NICON-10 time.

112. Turn off the Network Communication button in NICON-10.

113. Quit OpenSees.

114. Reduce the load limits on the Loading Actuators and the Slaved Force Controlled Actuators, according to their current force reading.

115. In the user input tab, in NICON-10, check that the displacement commands match the current readings of the Loading Actuators.

116. Set the load limits for Rigid Links to +/- 60 kN, for their inner limits, and to +/- 70 kN, for their outer limits.

117. Change NICON-10 analysis mode from automatic to manual.

118. Unload the specimens by relaxing the Loading Actuator in NICON-10. Make sure they stay in the slack at the end.

119. Set the ramp time in NICON-10 back to 12000 ms, and hold time to 1000ms.

120. When the specimens are fully relaxed, relax the Horizontal Rigid Links (unlock them).

121. Change the inner and outer displacement limits of the Vertical Top Connected Actuators to -15.0, +5 mm and -20.0, +8.0 mm plus their current reading.

122. Set the load limits for Loading Actuators and Slaved Force Controlled Actuators to +/- 50 kN, for their inner limits, and to +/- 60 kN, for their outer limits.

123. Set the load limits for Vertical Rigid Links to +/- 70 kN, for their inner limits, and to +/- 80 kN, for their outer limits.

124. Reduce the inner and outer force limits of the Horizontal Rigid Links back to +/-50 kN and +/-60 kN, respectively.

125. Set the displacement [2] limits for vertical rigid links to +15, -5, +20, and -8 mm.

126. Turn off booster pump by going to the Digital Output tab and clicking on the toggle checkbox.
127. Change the setpoint of the loading actuator back to zero.

128. Switch to low pressure.

129. Turn off the integrator.

130. In the Execution tab in Aeropro, set the loading time to 11 seconds.

131. In NICON-10, change the maximum allowable slackness to 7 mm, for AYBs and 25 mm for the BPS.

132. Offset the forces and displacements in NICON-10.

133. Start lowering the frame using the same procedure in steps 62-64.

134. When the loads in Vertical Rigid Links (C2 and C4) drop to values above -20 connect the chains to the frame and keep them loose.

135. Stop AeroPro and Data Acquisition System.

136. Turn off the pressure and then close the supply valve.

137. Stop NICON-10.

138. Retrieve the DAQ data in AeroPro.

139. Unload the UT10-3EA-test test file.

140. Close all software.

141. Shut down all the computers.

142. Turn off FlexTest.

**ERROR HANDLING PROCEDURES**

**Procedure to Resume the Test when Aeropro Stops**

If Aeropro stops for any reason, during the simulation, NICON-10 is programmed to switch from the Auto mode to the manual mode. This will pause the simulation, and an error message will be generated in NICON-10 listing the possible reasons that may have stopped Aeropro. The following procedure can be followed for resuming the test:

1. Identify the reason that has caused Aeropro to stop. If the test has stopped because a limit has been exceeded, after carefully studying the reasons, modify the limit to allow the test to continue.

2. Go to the Detector tab in Aeropro, and reset and enable to limits.

3. Go to the Execution tab in Aeropro, and set the Next End Level to 1.0.

4. In the Execution Tab, set the loading time to 40.0 seconds.

5. Click on Forward.

6. Carefully watch the development of forces and displacements.
7. Ensure that the new limits are set properly to allow the execution of the test.
8. If the error handling has occurred during the lifting of the unlifting phase, change the loading time to 15.0 seconds, after continuing to lift, or lower the frame.

Procedure for NICON-10 Complete Restart with the AeroPro on High Pressure

In some applications, it may be required to completely stop or restart NICON-10 while AeroPro is still on high pressure. Stopping NICON-10, while AeroPro is still on high pressure, may be applicable in cases where the test needs to be stopped before the completion of the simulation, due to lack of time, or required modifications to the test setup and instrumentations.

Restarting NICON-10, while AeroPro is still on high pressure, may be applicable in cases where two tests are scheduled in a single day, and the second test is to be carried out before unlocking or lowering the frame. In both cases, the following procedure is recommended for error handling:

1. Change NICON-10 from the automatic mode to the manual mode.
3. Reduce the limits on the specimens as required and possible.
4. Stop NICON-10.
5. Retrieve the OpenSees Data.
6. Change the OpenSees script as desired, if a new test will be executed. Or Open a new OpenSees file
7. Stop the test in AeroPro to keep the current status of the actuators and the specimens.
8. Keep the pressure as high, so that the system will not see sudden force or displacement changes.
10. Turn on Control.
12. Check feedback in Aeropro, to ensure that it matches the commands.
13. Check if the command in NICON-10 is equal to the feedbacks in Aeropro.
14. In Aeropro, In the Execution tab, set the Next End Level to 1.0.
15. In the same tab, set the loading time to 40.0 seconds.
18. If the error handling procedure was carried out to stop the test, go to step 110, and follow the procedure to lower the frame.
19. If the error handling procedure was carried out to run a new test, go to step 87 to start loading the system, for the new simulation.
Appendix A: UT10 Operation Checklist

**Procedure to Stop the Test when NICON-10 Stops Working**

In rare instances and depending on the test computational demand, NICON-10 may stop sending commands to Aeropro and completely freeze. This happened during the 3-element hybrid simulation on three AYBs on Oct. 17th of 2017. In such a case, the test needs to be stopped, without imposing a sudden displacement demands on the specimens. Next, if desired, the test can be resumed using the *Procedure for Test Resumption*.

In such applications, the following procedure is recommended:

1. Reduce the load limits for *Loading Actuators*, *Slaved Force Controlled Actuators*, and *Rigid Link Actuators*. The upper and lower bounds should be determined based on the load in the specimens when the test was stopped.
3. Unlock the blue frame by changing the set points of the Rigid Links. This would be the opposite of the procedure in *Locking* (steps 83-86).
4. Stop the test in Aeropro.
5. Turn off the booster pump.
6. Switch to low pressure.
7. Turn integrator off.
8. Restart the NICON-10 computer, and re-open NICON-10 when the system loads.
9. Go to the *Procedure for NICON-10 Restart* and continue till step 16.
10. After step 16, if you desire to continue the test, go to the *Test Resumption Procedure*.

**Procedure for Test Resumption Using the UT-10 Hybrid Simulator**

The following steps are recommended for test resumption, with the assumption that the test is resumed on a different day. This means that the specimens have reached zero force and only have residuals. In addition, the steps are based on the assumption that the system experiences a complete restart. For resuming the test on the same day, without system shutdown, some of the steps may be skipped.

1. Run the Checklist for UT10 Multi-Element Tests from Step 1 to Step 76.
3. For the *Loading Actuators*, set the displacement ([2] and [3]) limits to their current reading +/- 1.0 mm, for the inner limits, and +/- 2 mm, for the outer limits.
4. Set the force limits for the *Loading Actuators* and the *Slaved Force Controlled Actuators* to +/- 70 kN, for their inner limits, and +/- 80 kN, for the outer limits.
5. Continue the steps in the Checklist for UT10 Multi-Element Tests from Step 78 to Step 98. Skip steps 87, 88, and 95.
6. After Step 98, use the following steps:
7. In NICON-10, go to the Test Resumption tab and turn off the *All Specimen Feedback* button.

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8. In NICON-10, go to the Network tab and turn on Start Server.
9. Run OpenSees.
10. Turn on Start Communication in NICON-10.
11. NICON-10 will start running the simulation from the log file with data from the previously stopped test. Wait until the portion of the simulation from the log file is finished.
12. When the log file finishes, OpenSees will wait until the next step command is executed in NICON-10.
13. In NICON-10, turn on the specimen feedback 1–4.
14. In NICON-10, in the Test Resumption tab, enter the correct specified offset based on the OpenSees log file at zero loads for all braces. Note that displacements are scaled. For instance, based on the August 15th test results, after unloading the specimens the displacement offsets were -13.142 mm for specimen 1, 9.961 mm for specimen 2, and -6.118 mm for specimen 3.
15. In NICON-10, enable the Test Resumption.
16. Offset displacements and forces for the specimens two times.
17. Reset Auto Scaled Tared Measured force and displacements F-D graphs.
18. In Aeropro, adjust the force and displacement limits for the Loading Actuators and the force limits for the Slaved Force Controlled Actuators according to the target force and displacement for the specimens.
19. In Aeropro, increase the force limits for the Rigid Links to +/- 300 kN, for the inner limits, and +/- 350 kN, for the outer limits.
20. In the user input tab, start changing the displacement such that the displacement in specimen reaches the displacement, at which the previous simulation was stopped at. Note that the user displacement shows the actual displacement of the instruments.
21. Check if you have reached the desired points. Check force readings as well.
22. After the specimens reach the desired displacements, the rest of the simulation can be carried out by executing the commands in NICON-10.
23. Turn on resume the test button in the Test Resumption tab.
24. Re-adjust the limits for the Loading Actuators and the Slaved Force Controlled Actuators accordingly.
25. Turn on automatic mode.
26. The test will resume.
27. In order to finish the test, follow Steps 102 to 138 in the Checklist for UT10 Multi-Element Tests.
Figure A.1: Actuator configuration during the loading phase of the test (shown for four specimens)
APPENDIX B: DESIGN DRAWINGS OF THE UT10 SUPPORT FRAME
Bill of Material

<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>PART NAME</th>
<th>Drawing No.</th>
<th>Material</th>
<th>Weight (kg)</th>
<th>QTY.</th>
<th>Total Weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Base Plate</td>
<td>P1</td>
<td>350W</td>
<td>289.9</td>
<td>1</td>
<td>289.9</td>
</tr>
<tr>
<td>2</td>
<td>Side Plate</td>
<td>P2</td>
<td>350W</td>
<td>662.37</td>
<td>2</td>
<td>1324.7</td>
</tr>
<tr>
<td>3</td>
<td>Middle-Beam</td>
<td>P3</td>
<td>C1018</td>
<td>143.15</td>
<td>1</td>
<td>143.2</td>
</tr>
<tr>
<td>4</td>
<td>Side-Beam</td>
<td>P4</td>
<td>C1018/350W</td>
<td>232.51</td>
<td>2</td>
<td>465.0</td>
</tr>
<tr>
<td>5</td>
<td>Lateral Support 1</td>
<td>P5</td>
<td>C1018</td>
<td>36.05</td>
<td>4</td>
<td>144.2</td>
</tr>
<tr>
<td>6</td>
<td>Lateral Support 2</td>
<td>P5</td>
<td>C1018</td>
<td>35.99</td>
<td>8</td>
<td>287.9</td>
</tr>
<tr>
<td>7</td>
<td>Spacer 1</td>
<td>P6</td>
<td>350W</td>
<td>0.15</td>
<td>40</td>
<td>6.0</td>
</tr>
<tr>
<td>8</td>
<td>Spacer 2</td>
<td>P7</td>
<td>350W</td>
<td>13.47</td>
<td>6</td>
<td>80.8</td>
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<tr>
<td>9</td>
<td>Washer 1</td>
<td>P8</td>
<td>350W</td>
<td>0.09</td>
<td>24</td>
<td>2.2</td>
</tr>
<tr>
<td>10</td>
<td>Washer 2</td>
<td>P9</td>
<td>350W</td>
<td>0.05</td>
<td>12</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Notes:
- All dimensions are in millimeters [inches].
- Maximum tolerance is ±0.127/mm [0.005"] unless otherwise indicated on the drawings.
- All materials are steel with grades based on CSA G40.21 unless otherwise stated.
- The specified thread depth in the tapped holes includes 3 unusable threads.
- An overall view of the final assembly of the frame is provided in this sheet and in sheet No. 3, to give further insight into the design, location and future installation of the required sections. Therefore, assembling the frame is not required. Only the sections and their corresponding holes and welds are required as per the drawings provided in the following sheets.
- The instructions for painting the required sections is provided in sheets No. 13 and 14.

All dimensions are in millimeters [inches] and maximum tolerance is ±0.127/mm [0.005"] unless otherwise indicated on the drawings.
Appendix B: UT10 Design Drawings
Note:
- Use flat sections with low tolerance on outer dimension size.
- 4 quantity of Lateral Support 1 and 8 quantity of Lateral Support 2 are required.

All dimensions are in millimeters [inches] and maximum tolerance is ±0.127 mm [±0.005"] unless otherwise indicated on the drawings.
### Appendix B: UT10 Design Drawings

<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>PART NUMBER</th>
<th>MATERIAL</th>
<th>DESCRIPTION</th>
<th>QTY.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Middle Beam</td>
<td>C1018 Steel</td>
<td>Low friction bearing support for the specimen</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>Hex Socket Bolt</td>
<td>Normal Strength</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table:**
- **Material:** C1018 Steel
- **Description:** Low friction bearing support for the specimen
- **Quantity:** 100

**Diagram:**
- The diagram shows a 10-element hybrid simulator frame with labeled parts 1, 2, and 3.

---

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Appendix B: UT10 Design Drawings

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APPENDIX C: DESIGN DRAWINGS OF THE AYB SPECIMEN
Support pipe
OD = 1.9
Wall thickness: 0.281"

SECTION A-A
SCALE 1:1

M20-10 hex nut

4 mm

76.2 [3.00]

0.482 [1.90]

All dimensions are in millimeters (inches).
Appendix C: AYB Design Drawings

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto
Appendix C: AYB Design Drawings

Saeid Mojiri, Department of Civil & Mineral Engineering, University of Toronto

All dimensions are in millimeters [inches].
All materials are steel with grades based on CSA G40.21 unless otherwise stated.
APPENDIX D: DESIGN DRAWINGS OF THE BUCKLING SPECIMEN
All dimensions are in millimeters (inches).
All materials are steel with grades based on CSA G40.21 unless otherwise stated.