SEISMIC ASSESSMENT AND DYNAMIC TESTING OF REINFORCED CONCRETE FRAMES RETROFITTED WITH A SHAPE-MEMORY ALLOY BRACE SYSTEM

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SEISMIC ASSESSMENT AND DYNAMIC TESTING OF REINFORCED CONCRETE FRAMES RETROFITTED WITH A SHAPE-MEMORY ALLOY BRACE SYSTEM

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LIST OF SYMBOLS AND ABBREVIATIONS

Abbreviations

ACI  American Concrete Institute
AISC  American Institute of Steel Construction
ASCE  American Society Of Civil Engineers
AWS  American Welding Society
BRB  Buckling-Restrained Braces
CSUS  Central and Southeastern United States
ELF  Equivalent Lateral Force
FEMA  Federal Emergency Management Agency
FRP  Fiber-Reinforced Polymer
HSS  Hollow Structural Sections
HSS  Hollow Steel Section
IM  Intensity Measure
LIS  Linear Inertial Shaker
LSP  Linear Static Procedure
LVDT  Linear Variable Differential Transformer
NEES  Network for Earthquake Engineering Simulation
NLTHA  Non-Linear Time History Analyses
NMSZ  New Madrid Seismic Zone
NTHA  Nonlinear Time History Analysis
PBEE  Performance-Based Earthquake Engineering
PGA  Peak Ground Acceleration
PSDM  Probabilistic Seismic Demand Model
RC  Reinforced Concrete
RCGF  Reinforced Concrete Gravity Frames
SC-BRB  Self-Centering Buckling-Restrained Brace
SMA  Shape Memory Alloys
SRA  Seismic Risk Assessment
UCLA  University of California, Los Angeles
WUS  Western United States

Symbols

\( M_j \) equivalent joint rotational moment
\( v_j \) joint shear stress
\( A_j \) joint area
\( L_b \) beam total length
\( b_j \) joint effective width
\( L_c \) column total length
\( j \) internal moment arm factor (assumed to be 0.875)
\( d_b \) beam effective depth
\( \alpha \) constant equal to 2 for the top floor joints and 1 for the others
\( \theta_j \) joint rotation
\( \gamma_j \) joint shear strain
\( S_d \) median value of the demand as a function of \( IM \)
\( \beta_{d,IM} \) Dispersion of the demand as a function of \( IM \)
$S_c$  Median value of the capacity limit states

$\beta_c$  Dispersion of the capacity limit states

$\beta_m$  The modeling uncertainty (assumed to be 0.2, Celik & Ellingwood 2010)

$\Phi[\cdot]$  standard normal cumulative distribution function.

$V$  Base shear

$\beta$  reliability coefficient

$\mu$  Mean

$\sigma$  standard deviation

$r$  radius of gyration

$I$  moment of inertia

$A$  Area

$\phi F_{cr}$  critical resistance capacity

$R_n$  nominal capacity, connection elements (bolts or welds)

$e_b$  Gusset plate eccentricity to beam

$e_c$  Gusset plate eccentricity to column

$d_b$  Depth of steel beam section

$d_c$  Depth of steel column section

$K_1$  Effective length factor in plane of bending

$V_c$  Column to gusset plate interface force - vertical

$P_u$  Ultimate applied force

$H_c$  Column to gusset plate interface force - horizontal

$H_c$  Beam to gusset plate interface force - horizontal

$V_b$  Beam to gusset plate interface force - vertical

$M_c$  Beam to gusset plate interface moment

$f_{vb}$  Interface stress due to $H_b$ – beam to gusset weld
\[ f_{ab} \] Interface stress due to \( V_b \) – beam to gusset weld

\[ f_{b-avg} \] Average interface stress – beam to gusset weld

\[ f_{des-b} \] Design interface stress – beam to gusset weld

\( D_{weld} \) Weld thickness, in

\( t_{min} \) Minimum weld thickness

\[ f_{vc} \] Interface stress due to \( H_c \) – column to gusset weld

\[ f_{bc} \] Interface stress due to \( V_c \) – column to gusset weld

\[ f_{c-avg} \] Average interface stress – column to gusset weld

\[ f_{des-c} \] Design interface stress – column to gusset weld
SUMMARY

Region-level seismic risk assessments have estimated the economic and life-safety impact of a large magnitude earthquake in the US New Madrid Seismic Zone at over $300 billion dollars and close to 100,000 casualties, respectively. Seismic rehabilitation of structures has been a research priority for the last 3 decades as practitioners and academics recognize the need to address safety of older structures. Non-ductile reinforced concrete (RC) frame structures have been a focus of this research due to their prevalence in the Central and Eastern US. Retrofits have been developed and implemented to address local (component-level) and global (system-level) deficiencies in these structures, based on past performance during earthquakes as well as testing at both the component and system levels. However, prior research and existing solutions have several limitations. Regarding testing, most prior research has focused on single components – which fail to capture the interaction of multiple components in a full structural system; or reduced scale systems – which do not appropriately replicate critical resistance mechanisms such as the bond between concrete and reinforcing bars. Existing retrofit solutions have been generally effective in increasing the life-safety for occupants of deficient existing structures but have shown two main limitations. The first limitation is that these retrofits address the life-safety issue by preventing structural collapse, but the levels of damage they withstand following seismic activities typically render the full structure irreparable, which does not improve the economic impact. The second limitation is that their construction is invasive, which may disrupt building occupants and operations. Accordingly, the study presented herein attempts to address these limitations.
by proposing a retrofit solution that is reusable, self-centering, limits overall structural
damage, and has a minimally invasive construction and installation procedure. The
retrofit was tested dynamically in a full-scale system to overcome the limitations of prior,
reduced-scale experimental testing research.

A bracing system with shape memory alloy (SMA) components was designed as a
retrofit to dissipate energy and limit residual drifts in an RC frame. The brace was tested
independently and as the main component of a retrofit scheme in a full-scale dynamic test
of a prototype reinforced concrete moment frame representative of low-rise office
buildings in the Central and Eastern US from the 1950s-1970s. A plane-frame two-story,
two-bay specimen –part of a test-bed of four full-scale, nominally identical structures
designed as part of a companion study– was retrofitted with the SMA bracing system and
tested using a linear inertial shaker to assess the seismic performance. The SMA brace
was also tested independently, in quasi-static cycles, to quantify the full hysteretic
behavior. Using results from the dynamic and quasi-static tests, plus the results from the
same two-story two-bay frame in the as-built condition (no retrofits, results from a
companion study), numerical models were calibrated to assess the fragility of the two-
story two-bay frame in both as-built and retrofitted condition and to compare the seismic
performance.

Experimental and analytical results showed that incorporating the SMA bracing
device effectively reduced peak and residual inter-story drifts compared to a non-
retrofitted frame. While the as-built structure developed a global sway mechanism during
testing at story drifts above 1.5%, the retrofitted structure showed no visible damage at
similar drift magnitudes. The retrofitted structure also withstood multiple tests at larger
input accelerations than those of the as-built frame. Numerically, it was shown that the SMA brace retrofit significantly reduces the probability of exceeding all damage states at given spectral accelerations. Most notably, the probability of exceeding the complete damage state was reduced from 50% (as-built) to less than 2% (SMA brace retrofit) at a 0.78g spectral acceleration.

Along with these results, the design and assembly steps for experimental testing suggest that this SMA retrofit can be beneficial in practical applications where disruptions to building occupants are a concern. The retrofit design procedure, its behavior and response to all test loads, a qualitative evaluation of the design method, seismic risk performance assessment, and implications on future research are discussed.
CHAPTER 1. Introduction

1.1. Background and Motivation

Direct economic losses incurred in seismic events have ranged from $7 billion USD in the 1989 Loma Prieta earthquake, to $200 billion USD in the 1995 Kobe earthquake (Bertero & Bertero, 2002), and over $300 billion USD in the 2011 Tohoku earthquake (Daniell, Khazai, Wenzel, & Vervaeck, 2011). In a recent impact assessment report (Elnashai, Cleveland, Jefferson, & Harrald, 2009), researchers from the Mid-America Earthquake Center estimated that an Mw 7.7 earthquake in the New Madrid Seismic Zone (NMSZ) could result in approximately $300 billion in economic losses and nearly 86,000 human injuries and fatalities. About 715,000 buildings (primarily in Tennessee, Arkansas, and Missouri), including 130 hospitals, could end up damaged from this seismic event. These losses are compounded once social impacts and indirect economic losses are considered. These issues form the basis for the topics addressed in this thesis.

Recent experiences and seismic provision updates (American Society of Civil Engineers [ASCE], 2016), primarily within the Central and Eastern USA, amplify the need to increase knowledge of the performance of older structures during seismic events in these areas. With regards to urban areas, the adoption of seismic retrofitting is recognized as an integral factor in alleviating the damage caused by seismic events (Pampanin, 2006). Pampanin, along with other researchers (e.g. Bertero & Bertero, 2002; Reitherman, 2012), argues that before building codes that consider seismic loading were introduced in the late 1960s, the general philosophy for structural design did not offer the
necessary robustness to counteract potential seismic demands. Evidence for these arguments is demonstrated by damage to buildings observed after several major earthquakes over the last 25 years, including the 1994 Northridge Earthquake, 1995 Kobe Earthquake, 1999 Kocaeli Earthquake (damage examples shown in Figure 1.1), 2001 Bhuj Earthquake, 2008 Wenchuan Earthquake, and 2010 Haiti Earthquake, 2011 Tohoku Earthquake and 2011 Christchurch Earthquake. Reinforced concrete structures in both developing and developed countries have experienced significant damage during earthquakes. The poorest seismic performance is observed in buildings with moment-resisting frames rather than buildings where shear walls provide the largest portion of lateral strength and stiffness. The most typical structural deficiencies include concentration of story drifts at the ground floor (soft story mechanism), beam-column joint failure and premature shear failure due to poor detailing. Other failure types include poor lap splice and anchorage detailing, as shown in Figure 1.2.

Figure 1.1 – Damage to reinforced concrete buildings during the 1999 Kocaeli Earthquake: (a) Soft story mechanism, (b) Failure due to beam-column joint damage and (c) Shear failure in a column. (Photos as shown in Wright, 2015, originally from Sezen et al., 2000)
Figure 1.2 – 1999 Kocaeli Earthquake: (a) column failure due to insufficient lap splice length (Photo originally from K. J. Elwood) and (b) pull-out failure of beam longitudinal reinforcement (Cosgun, Turk, Mangir, Cosgun, & Kiymaz, 2020)

It should be mentioned that in cases where the seismic demand was much higher than the corresponding capacity for a significant number of structural members, the overall building stability was jeopardized, resulting in partial or total collapse (pancake-type collapse) of the structures, with dramatic consequences for their occupants (see Figure 1.3).
Figure 1.3 – Collapses of reinforced concrete buildings due to strong ground motions: collapse of frame due to damage in columns and beam-column joints after 2008 Wenchuan Earthquake (left – Zhao, Taucer, & Rossetto, 2009), collapse of 2nd and 7th story of an 8-story building after 1995 Kobe Earthquake (top right – Mitchell, DeVall, Kobayashi, Tinawi, & Tso, 1996, and pancake-type collapse of nonductile building after 1999 Kocaeli Earthquake. (bottom right – Sezen et al., 2000).

Given these design weaknesses, recent years have seen an increased interest in addressing the impact of seismic activity on the built environment through assessing the feasibility of varied retrofits for buildings and infrastructure. Special research has been undertaken, resulting in guidelines for improving the structural rehabilitation, such as ASCE-SEI 41 (American Society of Civil Engineers - ASCE Committee, 2007) and Federal Emergency Management Agency (FEMA) 547 (FEMA, 2006).
In general, modern strategies for retrofitting structural systems have improved upon conventional retrofit/rehabilitation techniques, with main objectives being the increase of ductility, stiffness and strength compared to the original system. Conventional retrofitting methods may consist of addition of new walls, column section enlargement through concrete jackets or wing walls in continuity of existing columns, or addition of external steel bracing, as shown in Figure 1.4 and Figure 1.5. Addition of vertical members enhances significantly ductility, stiffness and strength. New concrete walls (Figure 1.4a) mainly increase the stiffness and the strength of the structure, while jacketing techniques (Figure 1.4b and Figure 1.5) aim primarily to improve the member and structure ductility. The use of steel bracing systems (Figure 1.4c) increases mainly the ductility and the stiffness of the building. The introduction of dissipation systems aims to reduce the seismic action on the structure. However, due to their relatively high cost they are implemented less often than the other more conventional retrofitting methods mentioned.
Figure 1.4 – Examples of conventional retrofitting for reinforced concrete buildings: a) rebar cage for new shear wall, b) rebar cage for column enlargement, c) external steel braced frame (Photos from Shaik, 2019 for [a, b], and “Seismic retrofit,” n.d. for [c])

Figure 1.5 – Examples of conventional retrofitting methods for reinforced concrete vertical members: a) walls strengthening using plate bonding, b) column strengthening using a combination of steel plates and angles, and c) column strengthening using steel jacket along the entire column height (Heiza, Nabil, Meleka, & Tayel, 2014)
Building on this narrative, recent research (e.g. Reitherman, 2012) recognizes that seismic retrofit and rehabilitation techniques have benefitted from increased access to several advanced materials, such as fiber-reinforced polymers and higher strength steels.

This evolution has led to researchers investigating retrofit schemes that can not only achieve an improved overall performance in terms of strength, robustness, and reduced deformations, but can also address the myriad of factors that must be considered during the decision process of seismic retrofitting. Building owners and users are typically mostly concerned about construction cost, short-term disruption of occupants, long-term building functionality, and aesthetics. In addition, engineers must also consider several issues when designing a retrofit scheme, such as constructability and building accessibility, quality assurance, vulnerability during construction, conflicts with non-structural building components, building code requirements, and cost. The assortment of factors which potentially affect the end product of seismic rehabilitation makes it essential to address all relevant issues during the retrofit decision-making process in order to obtain the most satisfying results (Hall & Wiggins, 2000). The significance of each factor depends on the type of building. Furthermore, regional differences in seismic hazard result in different levels of seismic risk, even for comparatively similar buildings.

Considering that direct economic losses that arise as a result of major seismic events have increased exponentially since the 1989 earthquake in Loma Prieta, North California (Bertero & Bertero, 2002), the reality of addressing seismic retrofitting is not only an engineering issue, but also a financial, economic, and social one (Pampanin, 2006). Further indirect economic losses can also be considered with regards to the impact at the commercial and social level. The outcome of recent assessments, such as the Mid-
America Earthquake Center report (Elnashai et al., 2009), highlight the importance of seismic retrofitting as an intervention process that can mitigate these losses.

Moehle (2000) and Reitherman (2012) note that retrofit design guidelines need to be updated regularly. The 1994 Northridge earthquake is cited as a case in point where latter insights helped identify the limitations of welded steel frames employed in buildings and subject to seismic activity, which was previously deemed as an adequate design. Other examples include the potential structural performance limitations of reinforced concrete gravity frames (see Figure 1.6). Reinforced Concrete Gravity Frames (RCGF – Figure 1.6), exhibiting non-ductile behavior, have been widely used for building construction in areas of low, moderate, and high seismicity (Beres, White, & Gergely, 1992; Bracci, Reinhorn, & Mander, 1995; Celik & Ellingwood, 2009; French, 2004; Jeon, DesRoches, Brilakis, & Lowes, 2012; Ramamoorthy, Gardoni, & Bracci, 2006). Detailing deficiencies in frame elements of non-ductile RCGF buildings make these structures very susceptible to earthquake damage. A study by Beres et al. (1992) identified the following details as potentially critical to safety during an earthquake:

1. Longitudinal reinforcement ratio in columns not exceeding 2%.
2. Lap splices of column reinforcement at the maximum moment regions.
3. Wide spacing of column ties that provides little concrete confinement.
4. Little to no transverse reinforcement within the beam-column joint region.
5. Bottom beam reinforcement with a short embedment length into the column.
6. Construction joints near the beam-column joints.
7. Columns having bending moment capacities close to those of the beams.
Given this variety of potentially critical deficiencies and the prevalence of this type of building in earthquake-prone regions, much research has been dedicated over the past decades to developing retrofits which increase the seismic performance of these structures.

Since the 1940s (FEMA, 2006), retrofit schemes and techniques have been used in the Western United States (WUS) to address ductility, strength, and stiffness deficiencies in RCGF buildings. Some conventional schemes (American Concrete Institute [ACI], 2013; FEMA, 2006) commonly used in the practice include addition of new elements (e.g. braces, shear walls), strengthening/stiffening of existing elements (e.g. column wrapping or jacketing), as well as employment of external ties or post-tensioning (Priestley & Lew, 1994). These retrofits may effectively increase the peak strength and energy dissipation capacity, as well as favorably alter the ductility characteristics of structural members (e.g. ElGawady, Endeshaw, McLean, & Sack, 2010; Engindeniz,
Kahn, & Abdul-Hamid, 2005). It is worth noting that many of these retrofit techniques were tested as either individual members or parts of small subassemblages, rather than as part of a full structural system. This is a limitation in the field of research that the present study addresses.

Moehle (2000) identifies the 1971 San Fernando earthquake as a catalyst for advancing several structural programs that were aimed at identifying and mitigating the risk of seismic activity. This catalyst was further bolstered by seismic events in California in 1989 and 1994, as well as Japan in 1995. Due in part to these events, the seismic rehabilitation of buildings was encouraged in California and across parts of the USA. Several researchers, such as Padgett, Dennemann, and Ghosh (2010), Ghosh and Padgett (2011), and Speicher, Hodgson, DesRoches, and Leon (2009) also indicate that research-based knowledge of seismic activity and retrofit design has increased in recent years due to the impact of these and other similar events and their effects on older buildings and infrastructure throughout the world. Moehle (2015) argues that the Western US still has many buildings within regions of high seismicity that fail to adhere to modern seismic code requirements and, as such, are vulnerable to either damage or collapse when experiencing seismic activity.

Previous research in this field has included evaluation of retrofits on a significant number of building components that are typically found within both the CSUS and the WUS (Wright, 2015). However, there is still a need for undertaking a comparative assessment of the feasibility of retrofit techniques based on a risk perspective. Since much of the previous analytical work was based on scaled experimental research, there is also a need for validation against experimental data of full-scale building system tests as
opposed to tests that concentrate solely on building components or structures that are scaled down. The present study addresses these issues based on modern Seismic Risk Assessment (SRA), which involves the study of hazard and context (i.e. risk), and consequence. In the framework for SRA, fragility curves are an essential input. Fragility curves indicate the probability of a building system, or component, being damaged beyond a specified state, conditioned on a given level of ground motion intensity. Data from fragility curves are used for loss estimates and cost-benefit analysis. Given the incurred cost of rehabilitating a building, or the potential future cost of not rehabilitating – expressed in additional losses over those of a retrofitted building if a seismic event occurs – these analyses are necessary for effective investment in seismic risk mitigation. Further details about this framework are given in Chapter 3.

1.2. Thesis Objectives

The primary objective of the research proposed herein is to develop and test innovative SMA retrofits for Reinforced Concrete Gravity Frame (RCGF) buildings and compare their performance to other (innovative and conventional) retrofits. The research aims to provide practitioners with a new method of retrofitting RCGFs that may provide potential advantages over conventional retrofits for buildings under specific seismic risk conditions. To meet this objective, the following tasks are undertaken, as follows:

(1) Identification and evaluation of vulnerable RCGF building components and feasible retrofit measures for buildings in areas of low-to-moderate seismicity, such as the CSUS.
(2) Development of RCGF numerical models with explicit consideration of vulnerable detailing deficiencies. Initial modeling and analysis efforts focus on investigating retrofit schemes for a full-scale frame to be tested experimentally, as described in (3). Results from this initial analysis are used to aid in retrofit selection for experimental tests.

(3) Design and full-scale testing of retrofits selected in (2). The testing program includes individual quasi-static and varied-strain rate testing of retrofit components as well as dynamic testing of the installed (fully-built) retrofit in a two-story, two-bay RCGF.

(4) Re-calibration of numerical models using results from companion study’s full-scale experimental tests. Subsequently, analysis of as-built and retrofitted two-, four-, and six-story RCGF buildings assuming both low-to-moderate and high seismic hazard levels.

(5) Formulation of probabilistic seismic demand models using results from numerical analysis in order to develop component and system fragility curves for RCGF buildings.

By meeting these objectives, this study results in the following original contributions to the field (further details in Chapter 8):

- Development, testing, and formulation of a detailed design procedure for a new class of retrofit devices
- First-of-its-kind full-scale broadband shaking experimental test of a nonductile RC structure retrofitted with a SMA bracing system.
• Documentation and public dissemination of full test data set for future researchers

1.3. Organization of Thesis

This dissertation is organized into the following chapters and appendices:

Chapter 2 presents a summary on existing literature about retrofit schemes and techniques for non-ductile RC frames. Conventional as well as more recent innovative retrofits are presented for both the local member level and global system retrofitting. The use of shape memory alloys in seismic applications is also summarized.

In Chapter 3, the preliminary modelling and fragility analysis of an as-built non-ductile RC frame is described. This frame is representative of the experimental frame tested in Wright’s (2015) study, a companion to this thesis. Three retrofit options are also modelled and analyzed, and their performance is compared to that of the as-built frame.

Chapter 4 presents the design of the SMA retrofit device for full-scale experimental testing. The initial SMA element testing, to establish design criteria, and the design procedure for all steel elements are detailed.

Chapter 5 describes the full testing plan and results. The chapter covers brace fabrication and assembly as well as test preparation and results for full-scale retrofitted frame testing and individual brace component testing.

Chapter 6 presents the refined modelling and fragility analysis of an as-built frame and a frame retrofitted with the SMA brace device tested experimentally. The
models used in these analyses were calibrated against the experimental results shown and discussed in Chapter 5.

Chapter 7 presents a proposed methodology for the practical design of an SMA brace device for retrofitting non-ductile RC frames.

Chapter 8 presents the major conclusions drawn from the research project, identifies topics that require further research, and suggests recommendations for future research in the use and experimental testing of SMA brace devices for seismic applications.
1.4. References


American Society of Civil Engineers (ASCE) Committee. (2007). Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06). American Society of Civil Engineers. Reston, VA: ASCE.


CHAPTER 2. Literature Review

2.1. Introduction

As stated in Chapter 1, this research aims to develop a retrofit to improve seismic performance of RCGFs. In achieving this objective, numerical models are used to 1) understand and explicitly evaluate building prototypes with vulnerable detailing deficiencies and 2) design, build, and test retrofits to address these deficiencies and improve the RCGF seismic performance. Tasks needed to achieve the focus of this study include:

1) Assessing the structural vulnerabilities and deficiencies numerically via Opensees models (McKenna, Scott, & Fenves, 2009).

2) Using analysis results to inform the selection of retrofit schemes and design the retrofit.

3) Designing a test procedure to capture the important properties of the main retrofit components and a full-scale retrofitted test frame, and

4) Post-processing and interpreting the experimental test data.

5) Re-calibrating the numerical models using results from the full-scale experimental tests.

To address these research needs, existing literature encompassing the relevant topics listed above was reviewed. Since item (2) led to the selection of diagonal braces using SMA components, prior use of SMAs in earthquake engineering was also reviewed. Non-ductile RCGF vulnerabilities were reviewed from two perspectives,
global structural deficiencies and local component deficiencies. Prior experimental tests and retrofits that address both types of deficiencies were studied.

As indicated in the literature, RCGFs exhibiting non-ductile behavior, are widely used for building construction within regions of low, moderate, and high seismicity (Beres et al., 1992; Bracci et al., 1995; Celik & Ellingwood, 2009; French, 2004; Jeon et al., 2012; Ramamoorthy et al., 2006). Historically, retrofit schemes and techniques have been used in high seismicity areas, such as the Western US (WUS) to improve ductility, strength, and stiffness deficiencies in RCGF buildings. In RCGFs, the critical vulnerable components are usually columns and beam-column joints, where common deficiencies, further discussed in this chapter, include low reinforcement ratios, inadequate spacing of transverse reinforcement and the use of short compression lap splices. Local column retrofit techniques typically involve the use of external confinement in order to increase strength or enhance the bond stress of reinforcement. Retrofit techniques that have been found effective for lap splice enhancement include steel jackets (Aboutaha, Engelhardt, Jirsa, & Kreger, 1996) and Fiber-Reinforced Polymer (FRP) wrapping, as shown in Figure 2.1 (ElGawady et al., 2010; Elsouri & Harajli, 2011; Ghosh & Sheikh, 2007; Harries, Ricles, Pissiki, & Sause, 2006). Literature regarding these techniques is presented in Section 2.2.1.
Literature is also reviewed in respect of modeling perspectives. Existing numerical studies have found various modeling approaches that provide suitable analytical options for multiple analysis scenarios (e.g. time-histories of ground motion suites or detailed FEA subassembly models). Modeling approaches are still the subject of continued academic and practical discussion, so the review shown here is meant to be a summary, rather than an exhaustive examination of the topic.

This chapter then progresses towards the use of SMAs for seismic applications in Section 2.3. This approach has been pioneered by Graesser and Cozzarelli (1991), who evaluated the use of SMA as seismic dampers. Their research proposed an SMA constitutive model verified with experimental results and suggested that hysteretic modeling and strain-rate characterization of SMAs is key for future design applications in seismic rehabilitation of structures. Section 2.3 also reviews the works of Ocel et al. (2004), who showed that SMA tendons possess the capacity for strain recovery following exposure to large strain demands akin to those caused by large seismic events; and

Figure 2.1 – Column retrofits: a) composite wrapping, b) steel jacketing (Photos from Heiza et al., 2014)

(a) (b)
DesRoches and Delemont (2002), who also reinforce that retrofit benefits can be found through the re-centering and damping properties of SMAs. Further, the section on SMAs for seismic applications reviews research into the feasibility of SMAs specifically for practical building seismic performance scenarios. One example is the work of McCormick, DesRoches, Fugazza, and Auricchio, F. (2007a), who developed analytical models to compare the performance of concentrically braced frames using SMA bracing.

2.2. Background on Retrofit Schemes and Techniques for Non-ductile Frames

Detailing deficiencies in frame elements of non-ductile RCGF buildings make these structures very susceptible to earthquake damage. Critical issues with regards to earthquake performance (Beres et al., 1992) include: longitudinal reinforcement ratio in columns not exceeding 2%; lap splices of column reinforcement at the maximum moment regions, wide spacing of column ties that provides little concrete confinement; lack of transverse reinforcement within the beam-column joint region, bottom beam reinforcement with a short embedment length into the column; construction joints near beam-column joints, and columns having bending moment capacities close to those of the beams.

Retrofits for RCGFs can enhance structural performance through global capacity or demand modifications, of local structural component enhancement. However, seismic performance is only one of several factors that must be considered during the decision process of seismic retrofitting. Building owners and users are typically also concerned about construction cost, short-term disruption of occupants, long-term building functionality, and aesthetics. In addition, engineers must also consider several issues
when designing a retrofit scheme, such as constructability and building accessibility, quality assurance, vulnerability during construction, conflicts with non-structural building components, building code requirements, and cost. The assortment of factors which potentially affect the end product of seismic rehabilitation makes it essential to address all relevant issues during the retrofit decision-making process in order to obtain the most satisfying results (Hall & Wiggins, 2000). The significance of each factor depends on the type of building. Furthermore, regional differences in seismic hazard result in different levels of seismic risk, even for comparatively similar buildings.

Specific to the US space, seismic retrofit research has seen a combination of individual investigations and coordinated research programs. The National Science Foundation (NSF) was involved in the creation and funding of some early seismic rehabilitation research during the 1980s. While these research programs laid the foundation for later work, the early efforts of the NSF programs stopped short of addressing issues regarding construction and performance when aiming to develop a research-based set of design guidelines (Beres, Pessiki, White, & Gergely, 1996). Later, in the early 1990s, the NSF created a five-year research program that aimed to address these issues. The objective of this reformed program was to realize and advance information regarding the vulnerability of existing structures against varying types of seismicity, as well as develop systems for improving economic construction techniques that could enhance, repair, or strengthen vulnerable structures (Beres et al., 1996). Jirsa (1996) showed that this program directly led to further research being undertaken by the National Center for Earthquake Engineering Research. This body of work forms the basis for the current study, which builds on these prior research findings by assessing the
feasibility of innovative retrofit techniques to enhance the seismic performance of older, non-ductile RC buildings, primarily addressing known ductility, strength, and stiffness deficiencies within these structures.

The advancement of Performance-Based Earthquake Engineering (PBEE) has led to increases in retrofit performance levels. Issues of structural safety, integrity, and functionality have benefited from increased research, which has translated into primary retrofitting goals going beyond just protecting human life and now increasingly focusing on increased economic viability (Speicher et al., 2009). Seeking performance such that buildings might need repair but not replacement has led to many innovative devices being developed with a holistic approach towards the construction and performance process, including constructability, economy, and expected structural response. The advancement has been possible due to a concerted effort involving academic research, engineering practice, and policy that aims towards increased safety and performance of buildings and structures in withstanding the impact of seismic events.

In the WUS, seismic retrofit techniques have been incorporated for several decades. But awareness of seismic hazard in the Central and Southeastern United States (CSUS) has only increased recently. This increasing awareness has triggered interest in the study of RCGF vulnerability in the CSUS and the evaluation of retrofits on numerous building components typical to the CSUS, as well as the WUS. However, there is a strong need for a comparative study of the feasibility of various retrofit schemes from a risk assessment perspective, validated against experimental data of full-scale building system tests rather than tests of building components. The present study aims to address that need.
Previous research (e.g. Filiatrault & Cherry, 1988; Reitherman, 2012) demonstrated benefits from conventional retrofit schemes such as addition of cross-bracing or new structural walls. Other well-studied techniques include the reduction of seismic demand through supplementary damping as well as the use of base isolation systems (De Luca, Mele, Molina, Verzeletti, & Pinto, 2001; Kam & Pampanin, 2008; Matsagar & Jangid, 2008).

Filiatrault and Cherry (1988) showed that increasing local structural element capacity results in increased global strength capacity. They argued that selective local component upgrades could be a cost effective retrofit solution. The caveat is that the designer must ensure that local component strengthening does not lead to mechanisms or brittle failure of existing elements that are not enhanced. Other techniques that have been used to enhance weak or non-ductile structures include sliding connections to accommodate seismic movements and the employment of friction dampers to dissipate energy.

In terms of analytical advancements, since the early 2000s (Liu, Burns, & Wen, 2003) it has been recognized that improvements in computational power and quantity of available experimental test data have allowed the numerical modeling of structures to become increasingly more sophisticated. These improvements, in combination with the change from deterministic design procedures to probability-based criteria, have led to the increased use of reliability methods as tools to assess the socioeconomic consequences of engineering decisions.
In the following sections, examples of potential retrofits are shown. The examples are discussed in terms of their physical application, as well as the techniques used to analyze them. While the variety of potentially critical RC frame deficiencies is large, when addressing them through retrofitting, they can be summarized in terms of local-member deficiencies vs global structural performance. As such, the discussion that follows is separated into these two broad topics.

2.2.1. Local Member Retrofits

Retrofits at the structural member level generally involve an increase in the strength and ductility of the member. The most vulnerable components in RCGFs are columns and beam-column joints. Common deficiencies in columns of RCGFs include low reinforcement ratios, inadequate spacing of transverse reinforcement and short compression lap splices. Local column retrofit techniques typically involve external confinement in order to increase strength or enhance the bond stress of reinforcement. Retrofit techniques that have been found effective for lap splice enhancement include steel jackets (Aboutaha et al., 1996) and FRP wrapping (ElGawady et al., 2010; Elsouri & Harajli, 2011; Ghosh & Sheikh, 2007; Harries et al., 2006). Chang (2002) recognized that the use of steel jackets can lead to increased corrosion, thereby leading to structural weakness. Speicher et al. (2009) showed that splice failures tend to result in vertical cracks developing within columns that are adjacent to failing bars, with the result being that the strength of the structure rapidly degrades. Speicher et al. (2009) also argued that short compression lap splices are unable to advance yield stresses in any reinforcement due to rapid deterioration of splice bonds occurring shortly after yielding. As such, short
compression lap splices are not suitable for use within areas of high seismicity. Findings in this dissertation’s companion study (Wright, 2015) also support this argument.

In terms of numerical modeling, different approaches have been used when considering column retrofits in the structural analysis. These include explicit modeling of the wrapping and jacketing properties (Liel & Deierlein, 2008), modifications to material model confinement factors (Hueste & Bai, 2007), or to the moment-curvature properties of the finite elements (Ramamoorthy et al., 2006).

Common retrofit techniques include the strengthening and stiffening techniques of existing elements, including column wrapping and jacketing (Harries et al., 2006). Retrofit techniques of this type are used as a means of increasing peak strength capacities as well as enhancing the energy dissipation ability of structural members (e.g. ElGawady et al., 2010). Aboutaha et al. (1996) and Seible, Priestley, Hegemier, and Innamorato (1997) and have shown the benefits of providing confinement (via jacketing, for example) to increase bond stress between spliced bars, helping delay strength degradation during cyclic column rotation.

Retrofits intended to address ductility issues, which can be used in order to help improve overall lap splice performance, have been the focus of study by a number of researchers, including Coffman, Marsh, and Brown (1993), Choi, Chung, Choi, and DesRoches (2012), Harries et al. (2006), Kahn (1980), Nesheli and Meguro (2005), Saatcioglu and Yalcin (2003), Sause, Harries, Walkup, Pessiki, and Ricles (2004), and Shin and Andrawes (2011) tested brittle rectangular column configurations retrofitted using FRP jackets, with and without the presence of lap splices. These studies found that,
for columns with deficient reinforcement but without short lap splices, displacement ductility could be significantly increased through FRP jacketing, with ductility of eight, seven, and six percent being realized with jackets that used six, four, and two plies, respectively. For columns with short lap splices, although ductility was still limited by the spliced bars, the flexural capacity of the column could be improved up to the nominal value through jacketing. Similar column confinement studies for rectangular and circular columns have been undertaken by Saatcioglu and Yalcin (2003); Nesheli and Meguro (2005); Shin and Andrawes (2011), and Choi et al. (2012).

Saatcioglu and Yalcin (2003), and Nesheli and Meguro (2005) combined jacketing and wrapping methods with pre-tensioning to increase column shear strength. Aboutaha et al. (1996) also used a variety of steel jacket configurations to increase the strength of shear critical columns. Priestley, Verma, and Xiao (1994) studied similar applications of steel jackets. All the above listed studies indicate that, when compared against reference specimens without retrofit, columns exhibited increases in strength and ductility. Galal, Arafa, and Ghobarah (2005) used glass FRP and carbon FRP wrapping to enhance a shear critical RC column. The column retrofitted with carbon FRP showed a significant increase in capacity. Similar studies by Seible et al. (1997), Ye, Yue, Zhao, and Li (2002), and Haroun and Elsanadedy, (2005) show comparable results. In terms of analysis and numerical modelling of these retrofit schemes, different approaches found in the literature include explicit modeling of the wrapping and jacketing properties (Liel & Deierlein, 2008), and modifications to the material model confinement factor (Hueste & Bai, 2007) or to the moment-curvature properties of the finite elements (Ramamoorthy et
Many of these approaches are based on the results of the aforementioned experimental studies.

Concerning conventional retrofit techniques studied experimentally and developed in practice for beam-column joint retrofits, Engindeniz et al. (2005) presented a thorough review of strengthening and retrofitting for these critical RC frame components. The reader is referred to the study for further details.

Other researchers (e.g. Elnashai & Pinho, 1998; Kam & Pampanin, 2008) have also presented alternative retrofit techniques. Kam and Pampanin showed counterintuitive selective weakening techniques, rather than strengthening, for joint retrofits. These, they argue, may be more cost-effective retrofits than column or bracing systems in pre-1970s RCGFs. However, it is not clear if they only considered upfront costs. In addition, these selective weakening techniques have not been as widely adopted as strengthening methods in the practice, possibly due to additional lack of research. In contrast, Elnashai and Pinho tested selective upgrading techniques, resulting in selective increases to strength-only, stiffness-only, or ductility-only properties through the addition of external steel elements to RC walls. Each selective improvement could be achieved without significantly altering other structural properties.

2.2.1.1. Modeling Perspectives

In terms of RC frame component modeling, different approaches have been used when considering structural analysis of retrofits. Effective modeling techniques remain a topic of debate in the research community, especially for non-ductile joints where behavior is governed by shear and bond slip. Considering beam-column joints, joint
macro-models have been developed for numerical analysis of RC frame buildings. Several models applicable to non-ductile joints have been proposed by Celik and Ellingwood (2009), Lowes and Altoontash (2003), Mitra and Lowes (2007), Phan, Saiidi, Anderson, and Ghasemi (2007), Sharma, Eligehausen, and Reddy (2011), Shin and LaFave (2004), Tajiri, Shiohara, and Kusuhara (2006), and Youssef and Ghobarah (2001) among others, indicating a wealth of literature in the area. A model based on the studies by Jeon et al. (2012) is proposed for modeling as-built frames in the present study. Despite the large number of proposed models in the literature, few investigations exist regarding the modeling implications of joint retrofits in non-ductile RC frames and the existing ones mostly involve detailed continuum finite element models (Deaton, 2013; Sharma, Genesio, Reddy, & Eligehausen, 2009), which are not practical for the proposed study. However, results in the present study suggest that joint retrofits (on their own) may not be as effective for reducing global seismic vulnerability as column or bracing system retrofits (see Section 3.2). Thus, simple modifications to the joint model parameters may be sufficient for the purposes of fragility studies.

Youssef and Ghobarah (2001) developed global macro models for RC beam-column joints and structural walls. The models represent flexural deformations in plastic hinge regions as well as shear and bond slip deformations given their premise that deformations within beam-column joints act as a contributing factor to drift of RC frames. In addition, since failure can occur through cumulative concrete crushing from applied beam and column moments, bond slip, and/or joint shear failure, their models are capable of capturing (idealized) potential failure mechanisms due to crushing of concrete, bond slip or shear; with allowance for the simultaneous progress in each mode. They
argued that the inclusion of these non-ductile structural characteristics in analytical models is important to correctly predict the seismic response of RC frames and determine the failure modes.

2.2.2. Global Retrofits

Common retrofit techniques to address global strength, stiffness, and ductility deficiencies commonly involve the addition of new elements such as shear walls, braces, braced frames, and supplemental damping. Shear walls are typically introduced as infill walls to increase the frame’s lateral stiffness. This effectively reduces the displacement demand on the structure and also increases the global lateral strength. Experimental infill wall studies in the literature include reinforced concrete, precast concrete, and steel plate walls (Altin, Ersoy, & Tamkut, 1992; Canbay, Ersoy, & Ozcebe, 2003; De Matteis, Formisano, & Mazzolani, 2009; Higashi, Endo, Ohkubo, & Shimizu, 1980; Kahn & Hanson, 1979; Kara & Altin, 2006; Sonuvar, Ozcebe, & Ersoy, 2004). Analytical models of infill shear walls available in the literature include equivalent strut models (e.g. Erberik & Elnashai, 2004; Madan, Reinhorn, Mander, & Valles, 1997) and continuum finite element models with smeared and discrete cracking (Stavridis & Shing, 2010). Hueste and Bai (2007), and Rossetto and Elnashai (2005) performed fragility analysis with masonry infill walls and concrete shear walls, respectively.

Bracing systems and added frames have also been used to stiffen non-ductile RC frames (Bush, Wyllie, & Jirsa, 1991; Higashi et al., 1980; Masri & Goel, 1996). In the present study, the preliminary assessment of the proposed bracing system retrofit assumed a perfect brace-to-joint connection. But previous research (Bush et al., 1991) has
shown that strengthening of existing components may be required to accommodate the modified structural behavior which may induce large shears into the beams, columns, and joints. This issue was later addressed in the retrofit connection detailed design (further details in Chapter 5 and Chapter 6).

Supplemental damping retrofit techniques aim to improve the energy dissipation characteristics of a structure (Lu, Zhou, & Yan, 2008; Molina, Sorace, Terenzi, Magonette, & Viaccoz, 2004; Naeim & Rhaman, 2000; Pekcan, Mander, & Chen, 1995; Shen, Soong, Chang, & Lai, 1995). By increasing the effective damping, displacements, and in some cases accelerations and base shear, may be decreased. Guneyisi and Altay (2008), and Taflanidis and Beck (2009) performed fragility analysis of RCGFs considering supplemental damping and found that seismic performance was significantly improved. Preliminary analysis in the present study indicated that a proposed SMA bracing system could also improve seismic performance by significantly reducing peak drifts as well as residual drifts.

2.3. Use of Shape Memory Alloy Devices for Seismic Applications

In addition to conventional retrofits, innovative devices based on Shape Memory Alloys have also been developed and studied for seismic retrofit purposes. These metallic alloys show attractive properties for seismic applications, such as the super-elastic and shape memory effects. These properties are described in Section 2.3.1. Examples of previous research that has shown the benefits of using SMA for seismic applications are given in Section 2.3.2.
2.3.1. Material Properties and Behavior

2.3.1.1. SMA Microstructure

The microstructure of SMA is composed of two basic ordered atomic phases; Austenite and Martensite which are responsible for the shape memory characteristics of the material. Austenite is stable at high temperatures and low stresses, possesses symmetric structure, and has a B2 body-centered atomic structure, as shown in Figure 2.2. On the other hand, Martensite is stable at low temperatures and high stresses, possesses a B19’ rhombic geometry, and can be found with either twin variants or a single favored variant (Wayman & Duerig, 1990). Martensite has lower stiffness and strength than Austenite. More detailed information about the microstructure and crystallography of SMAs is outside the scope of the present research but can be found in the literature.

Figure 2.2 – 2D microstructure representation of the two atomic phases of SMA (figure from Speicher, 2010).

2.3.1.2. Fundamental mechanical features of SMA

Shape memory alloys have two fundamental mechanical characteristics – shape memory and superelasticity, which are illustrated in Figure 2.3 and Figure 2.4. These features depend on the following SMA characteristic temperatures: $M_s$ and $M_f$ (the
temperatures at which Martensite formation begins and ends, respectively) as well as $A_s$ and $A_f$ (the temperatures at which Austenite formation begins and ends, respectively).

The shape memory effect starts occurring when the SMA is in the martensitic phase in a twinned orientation, i.e. at low temperature and high stress (bottom left side of Figure 2.3). When stress is applied to the material, reorientation of the twinned structure towards a detwinned single variant occurs to accommodate the resulting strains. During unloading (stress removal), the detwinned structure remains deformed, as shown in the bottom right side of Figure 2.3. If the metal is then heated above $A_f$ (top left side of Figure 2.3) and then cooled back below $M_f$ (bottom left side of Figure 2.3) its original shape is recovered. This occurs because the heating and cooling sequence causes the reorientation of the Martensite crystal structure into the low temperature twinned orientation.

The super-elastic behavior is exhibited when the SMA is in the austenitic phase, i.e. at high temperature and low stress (top left side of Figure 2.3). When the SMA is loaded beyond a specific stress level, the additional strain is accommodated through transformation of the Austenite into detwinned Martensite, creating a loading plateau. Full transformation corresponds to the end of the loading plateau. Further loading results in formation of slip planes (as is typically observed in metals) which in turn causes permanent deformations as the unloading stiffness is lower than the loading stiffness. However, when the load is released, it is observed that an unloading plateau is reached, where the detwinned Martensite is transformed back into Austenite. This allows the material to fully recover its shape, i.e. without residual strains.
Figure 2.3 – 2D microstructure representation of shape memory effect and super-elasticity (figure from Speicher, 2010).

Figure 2.4 – Stress-strain relationship of SMA showing (a) super-elastic effect and (b) shape memory effect (figure from Speicher, 2010).
2.3.1.3. **Specific features of NiTi SMA**

NiTi SMA is nowadays considered to be the alloy of choice for most applications in civil engineering as it presents several advantages over Fe-Mn-Si, Cu-Zn-Al, and Cu-Al-Ni SMAs, the most important ones being its enhanced fatigue and corrosion resistance, stable hysteresis, and large strain recoverability. Hence, the present research focuses on NiTi SMA with emphasis on wires and rods. Frick et al. (2005) and Tyber et al. (2007) provide an in-depth review of the basic material microstructural characterization of NiTi SMAs, the impact of precipitates presence on the mechanical properties, as well as an explanation of the role of subphases (R-phase) during the martensitic transformation. Several works concluded that the super-elastic effect is more pronounced for wires than for rods (Dolce & Marnetto, 1999; MANSIDE, 1998). More recent studies (McCormick, Tyber, DesRoches, Gall, & Maier, 2007b; Tyber et al., 2007) demonstrated that the choice of appropriate chemical composition, heat treatment and deformation processing can confer sufficient super-elastic properties to both rods and wires.

The mechanical behavior of NiTi SMA is largely influenced by the amount of thermal processing or annealing. Annealing leads to precipitation of Ni$_3$Ti$_4$ within the material microstructure, which results in slippage reduction and consequently in smaller residual deformations (Tyber et al., 2007). In addition, annealing leads to enhanced arrangement of dislocations that are present if no thermal processing is applied. The optimum aging temperature is around 400 °C (Miyazaki, 1990) while a protocol for thermal processing of super-elastic NiTi SMA is described by McCormick et al. (2007b).
The applied loading rate significantly influences the NiTi SMA behavior for the frequency range often encountered during strong ground motions (0.2 Hz to 4.0 Hz). An increase of the load rate, or equally of the strain rate, results in higher loading plateau and lower amount of hysteretic damping (DesRoches, McCormick, & Delemont, 2004; Dolce & Cardone, 2001; Tobushi, Shimeno, Hachisuka, & Tanaka, 1998).

The work of Wu, Yang, Pu, and Shi (1996) also investigated the load rate effect by conducting cyclic strain rate tests in a liquid environment. It was found that the amount of increase in loading plateau stress and the amount of decrease in unloading plateau stress become more pronounced if the specimen is not able to dissipate heat. This suggests that larger diameter rods will generally show increased strain rate dependence compared to smaller rods.

2.3.2. Seismic Retrofit Applications of SMAs

Research on NiTi Shape Memory Alloys (SMA) for use in structural retrofits has resulted in the development of devices with high damping values (5-10% equivalent viscous damping) and re-centering capabilities (Speicher et al., 2009). Analytical studies of multi-story buildings with SMA bracing systems subjected to large ground motions have shown that, when compared to steel bracing systems, SMA bracing systems may reduce peak interstory drifts by an average of 75% and residual drifts by over 90% (McCormick et al., 2007a).

Pampanin (2006) summarized some advantages of SMA use in seismic applications. SMAs as a means of post-tensioning, in series with a typical brace or as a dissipater and re-centering system in a base isolation intervention, were shown as recent
and useful examples. In particular, the bracing system based on re-centering and dissipating wires of Dolce, Cardone, and Marnetto (2000), and the post-tensioning system of Castellano, Indirli, and Martelli (2001), used to rehabilitate a historical building in Italy, were cited as useful examples. Such approaches can be incorporated as stand-alone solutions, or in combination with other retrofit techniques, where additional performance enhancements can also be realized through advanced energy dissipation. In the first large-scale test of SMA connections for moment resisting frames, Ocel et al (2004) investigated the use of martensitic NiTi tendons as the primary resisting elements of an exterior moment connection (further details below).

Graesser and Cozzarelli (1991) were pioneers in suggesting the use of SMA within seismic applications. They developed a SMA material model that characterized the stress-strain relationship of experimentally observed Nitinol cyclic response. The possibility of achieving large hysteresis without plastic deformations, they argued, is particularly relevant to earthquake engineering passive damping schemes. Analytical results indicated that a simple 1D hysteretic model could match the experimentally observed hysteretic behavior, up to 4.5% strain levels.

Early experimental studies include Krumme, Hayes, and Sweeney (1995), who tested the performance capabilities of a sliding SMA device while using opposing pairs of NiTi tension elements to resist sliding. The results of the experimental study were later applied to analytical studies on non-ductile concrete frame buildings retrofitted with the sliding SMA device. The results indicated that inter-story drifts and column rotational demands could be reduced when compared against non-retrofitted structures. Additional studies by Wilde, Gardoni, and Fujino (2000), expanded on Graesser and Cozzarelli’s
(1991) earlier research by incorporating post-transformation martensite hardening at large strains to study base isolation in bridges. This was an important modification since the model could accommodate the large strains structural elements could be subjected to during severe earthquakes. Wilde et al.’s (2000) research found that bridge deck displacements can be reduced with the use of SMA dampers.

Similarly, DesRoches and Fenves (2000) studied the use of SMA restrainers to prevent bridge collapse due to unseating. The research resulted in a design procedure for hinge restrainers that accounted for bridge frame inelasticity and frame period ratio. Bridge performance was then parametrically studied and SMA restrainers were compared to unrestrained multi-frame bridges. It was shown that a design with SMA restrainers limits relative displacements to designer-specified values in a wide variety of bridges and reduces displacements when compared against similar unrestrained superstructures.

DesRoches and Delemont (2002) built on the study from DesRoches and Fenves (2000) by further studying the application of SMA restrainers to prevent excessive movement and unseating of intermediate hinges and abutments within bridges. Their research included experimental tests of SMA restrainer bars in order to refine the analytical SMA material model. It also included an analytical study of bridge abutments, previously not considered in DesRoches and Fenves (2000). The study indicated that bars could be subjected to cyclical strains up to 8% with minimal residual deformation. Analytical results showed, once again, that SMA restrainers reduce relative hinge displacements, including in the bridge abutments, especially when compared to conventional steel cable restrainers.
Dolce et al. (2000) designed and tested reduced- and full-scale SMA devices, including braces and sliding isolation devices. Using a bracing scheme that allowed multiple SMA wire configurations, they showed that the damping and re-centering brace properties could be altered by varying the arrangement and/or quantity of wires. After testing multiple cycles on each brace (300 cycles on average for each brace without SMA wire substitutions) they demonstrated that SMA wires have stable and repeatable cyclic behavior.

Practical use of SMA, for seismic retrofitting, was reported by both Castellano et al. (2001) and Indirli, Castellano, Clemente, and Martelli (2001), where conventional and innovative techniques and materials were used to complete the retrofit a bell tower in Italy in 1999. Pre-stressing steel bars were installed in each inside corner of the tower and SMA wires were installed at the third-floor level in each bar. The wires were posttensioned to prevent tensile stresses in the tower’s masonry during a seismic event. The tower experienced similar magnitude earthquakes pre- and post-retrofit (1996 $M_L = 4.8$ and 2000 $M_L = 4.5$). While the tower was seriously damaged in 1996, it experienced no damage in the 2000 earthquake. However, due to the variety of retrofits used, it is unclear to what degree the SMA devices contributed to the improved performance.

A retrofit focus also formed part of research undertaken by Tirelli and Mascelloni (2000), who performed shake-table tests on unreinforced masonry walls retrofitted with SMA cross-bracing tendons. The tests resulted in the collapse of the non-retrofitted walls but only minor damage on the retrofitted wall. Similarly, Ohi (2001) used SMA elements to brace existing steel structures. Braces re-centered to their original geometry after being
subjected to as much as 5% strain. Results also showed that braces provided hysteretic damping at strains beyond 1%.

Ocel et al. (2004) used SMA tendons to build a partially restrained steel moment connection. The connections were tested under cyclic load up to 4% drift (~4-6% SMA strain demand). SMA tendons acted as a fuse, sustaining large deformations while allowing the steel elements to remain elastic. The SMA tendons were then heated to induce the shape memory effect and the connections were retested. During the retest, hysteresis loops showed no loss of strength or stiffness compared to the initial test. These results imply that such connections may be reused after a seismic event if the SMA component can be appropriately heat treated above transformational temperatures. These findings have since been the subject of additional research where SMAs have been deployed within ‘fuse-type’ devices intended to increase the overall performance of structural systems and reusability of specific components. As an example, McCormick et al. (2007a) studied analytical models to compare concentrically braced frames with either conventional steel braces or SMA braces. Using an extensive set of ground motions, it was determined that SMA braces may offer several improvements over conventional braces, including reduced column drift ratios as well as reduced maximum and residual story drifts for both mild and severe ground motions.

Other researchers have studied the characteristics of NiTi SMA to pinpoint and/or address issues that could affect mechanical performance. Eggeler, Hornbogen, Yawny, Heckmann, and Wagner (2004) noted that NiTi SMAs are liable to structural and functional fatigue. They showed this to be a concern when long-term cyclic loading is to be considered, which could lead to fracture. They also showed that functional fatigue
may lead to superelasticity degradation during the cyclic deformation of NiTi SMAs. Kang and Song (2015) also studied how superelasticity and the effect of shape memory in NiTi SMAs can assist in creating effective wear resistance but found that the shape memory effect can be degraded leading to a loss of functional capability.

McCormick et al. (2007a) undertook analytical studies of SMA bracing systems in multi-story buildings subjected to large seismic motions. Results indicated that SMA braces could reduce peak inter-story drifts by roughly 75% and residual drifts by over 90% when compared to the performance of steel bracing systems. Similar research by Andrawes and DesRoches (2007) indicated that the SMA superelastic effect may lead to retrofit schemes with mechanical characteristics that are considered ideal for passive control of building structures. Gao, Jeon, Hodgson, and DesRoches (2016) argue that SMAs offer an increased potential when used within structures and buildings in line with PBEE outcomes and requirements due to the inherent benefit of superelasticity. In the study of Gao et al., the lateral force resisting system used a cross-braced structure with an SMA ring. They showed that this system can be applied to retrofit projects to mitigate the cyclic loading damage via help of the re-centering process once energy dissipation has occurred. The conclusions of Gao et al. agree with the earlier assessments of Andrawes and DesRoches (2007), who showed how the unique superelastic traits of SMAs may enable structural systems to regain their original geometries following seismic events that led to large deformations. Andrawes and DesRoches cited previous research indicating that the hysteretic properties of SMAs are dependent on characteristics such as the chemical composition of the components used, the manufacturing process and the strain rate during the loading process. However, they showed that differences in the hysteretic
properties found within the material only lead to small differences in the effect on structural response resulting from seismic activity. To enable the study of SMA for seismic applications, Andrawes and DesRoches proposed that the hysteretic properties of SMAs can be assumed as being defined by a set of independent parameters. They demonstrated this use through case studies to calculate the impact of variability in assessing the effectiveness of SMAs as core restrainers of bridges and bracings for buildings. The actual impact and effect of changing hysteresis was shown as more pronounced in SMA bracing, compared to SMA restrainers. The study concludes that the benefits of superelastic SMAs are, in general, stable regardless of their hysteretic properties.

Soul and Yawny (2015) also found that functional fatigue, indicated by SMA residual deformation when cyclic loading was applied, could be an issue when assessing dynamic loading. However, they indicated that the use of SMA still offers attractive qualities when considering the level of protection provided for structural systems against dynamic loads. The successful implementation of SMAs, they concluded, required the conditioning of functional fatigue. They indicated that pre-straining NiTi SMA wires can improve the level and extent of tension/compression cycles. Pre-straining wires helps to absorb deleterious residual deformation without affecting the self-centering capabilities upon unloading. This property influenced the design philosophy of the SMA device in the present study.

Youssef, Alam, and Nehdi (2008) researched the possibility of using super-elastic SMAs as reinforcement within beam-column joints of reinforced concrete structures. The research included testing of two large scale beam-column joint specimens. One specimen
was reinforced with only regular steel rebar and the other included NiTi longitudinal rebar. The testing resulted in the SMA reinforced joint showing small residual displacement compared to the conventionally reinforced joint. In addition, the SMA reinforced joint showed a plastic hinge following the cyclic loading that was located at a more favorable distance away from the column face compared to the conventionally reinforced joint. Youssef et al. (2008) concluded that the subassembly results from the study could be used to formulate numerical models in order to simulate the performance of SMA-reinforced concrete beam-column joints as part of RC multi-story frames.

Zhu and Zhang (2008) developed a self-centering friction damped brace where SMA wires were used to re-center the brace. Energy dissipation was achieved by sliding friction between adjacent steel members. The braces were compared to buckling-restrained braces (BRB) in 3- and 6-story frames. Results showed that the friction damped brace eliminated residual deformations and reduced inter-story drifts when compared to the BRB.

Miller, Fahnestock, and Eatherton (2011) developed and tested a self-centering buckling-restrained brace (SC-BRB). This brace combines the favorable characteristics of BRBs with the self-centering property of SMAs. SMA rods are attached to the BRB portion of the brace using concentric tubes and free-floating end plates to induce elongation in the rods when the brace is both in tension and compression. Results from large-scale testing showed that the SC-BRB provides stable hysteretic response with appreciable energy dissipation, self-centering capability, and substantial deformation capacity.
Further comparative tests in SMA use for bridge restrainers were done by Johnson, Padgett, Maragakis, DesRoches, and Saiidi (2008). Here, large scale experimental studies were used to compare SMA restrainers against traditional steel restrainers. The results showed that forces in SMA and steel restrainers were comparable. However, SMA restrainers had minimal residual strain after repeated loading and could undergo many cycles with little strength or stiffness degradation. Johnson et al. (2008) study comprised shake table tests on a test specimen used to simulate in-span hinges, as part of a multiple-frame concrete box girder bridge. Padgett et al. (2010) also investigated, through a similar test setup, the use of restrainers at the deck-abutment interface of comparable multi-frame multi-span bridges. The Padgett et al.’s study confirmed the same favorable SMA restraining characteristics when used as unseating prevention devices.

In the area of building retrofit, further research by Speicher (2009) developed and tested three SMA-based systems intended to improve frame response during seismic events. Speicher’s study comprised: 1) a tension/compression brace, 2) an interior beam-column connection and, 3) an articulated quadrilateral bracing system. All devices allowed the structural frame system to recover most of (85%) or all the deformation and showed equivalent viscous damping in a range of 4%-13%. The results further reinforce the evidence that SMA-based systems have an advantage in terms of reducing residual drifts.

Research by Yang, DesRoches, and Leon (2010) led to the design of a hybrid device that combined energy dissipation through steel hysteresis and re-centering capabilities from SMA wires. A design methodology was provided for the device. Results
from analytical studies showed that if the force distribution between the SMA wires and the energy dissipation system is adequately proportioned (following their proposed guidelines), the device exhibits substantial re-centering capacity while maximizing energy dissipation. For a 3-story model, peak drifts were comparable with those of buckling-restrained brace systems, but residual drifts were reduced significantly.

The cited studies above serve as evidence to the various favorable properties of SMAs for seismic applications. However, the material cost must also be considered when selecting the appropriate retrofit methods. Presently, the relatively high cost of NiTi SMAs is the major disadvantage when compared to structural steel. A comparison of 2019 prices indicates that NiTi SMA costs $15-45/kg (Alibaba, 2019), while the cost of structural steel is approximately $0.60/kg (Focus-Economics, 2019).

However, the cost of NiTi SMA has been trending down over the last 3 decades. NiTi SMA costs were $45.7/kg in 1991 (Kurtz, 1991), so the cost is now cheaper, relative to inflation. Around the same time period (early 1990s) the cost of structural steel was $0.34/kg (EconStats, n.d.). So, the NiTi SMA-to-steel price ratio has decreased from 134 in 1991 to 25-75 in 2019. If this SMA cost reduction trend continues, it will render the material more economically feasible for structural engineering applications. Lastly, if life-cycle costs are considered when selecting materials for retrofitting, SMAs become more attractive because of their reusability, even after large seismic events.
2.4. References


CHAPTER 3. Preliminary Analysis of Test Frame for Selection and Design of Retrofits

The as-built and retrofitted full-scale RC test specimens documented in this study were devised as part of a companion study by Wright (2015). Wright undertook research to design a test-bed of four nominally identical 2-story, 2-bay reinforced concrete gravity frames (RCGFs) that were evaluated under dynamic loads. The study comprised a prototype RC moment frame representative of the interior column line of low-rise office building archetypes typically built in the central and southeastern US (CSUS) in the 1950s-1970s. Wright notes that this structure type was employed in the study because of the commonality in local construction methods plus the ability to create multiple plane frame specimens in a single test-bed. The subsequent experimental design allowed for assessment of full-scale specimens independent of one another, using the same testing equipment and methods.

A summary of the key points of Wright’s design and their relation to the present study are included here. For the full detailed design, the reader is referred to Wright (2015) (Chapter 3 and Appendix A). For consistency with the 1950-1970s design practice, Wright’s structural design did not include seismic loads. The gravity load design considered self-weight loads and loads imposed by testing equipment. To account for additional dead and live loads that would be present in a finished commercial building but not the lab specimen, five hundred pounds per linear foot were added across the beams via steel rails resting on the concrete slabs. The total combined load was indicative of the full design dead load plus an added thirty percent of the intended design live load.
Structural reinforcement details also remained consistent with typical 1950-1970’s design. Detailing includes multiple deficiencies due to the lack of lateral load considerations. For the columns, the design included wide column shear tie spacing to capture column susceptibility to shear failure. The column lap splice at the foundation level was purposely designed as a short compression splice, which could lead to brittle splice failure under tensile demands (due to lateral loads or vertical accelerations). Beam reinforcement deficiencies included short embedment lengths of positive column rebar as well as inadequate shear reinforcement spacing near the beam-column joint. The beam-column joint itself included no transverse shear reinforcement. The lack of transverse ties leads to potential brittle shear failures and to increased frame flexibility, which could negatively contribute to increased story drifts.

Wright conducted modal and pushover analyses as part of the frame structural and experimental design. In the present study, in addition to a modal analysis (for model validation), nonlinear time-history analyses were used. The objectives of the analysis were two-fold: (1) to provide a basis for vulnerability models that could be later updated and refined with experimental results, and (2) to aid in retrofit selection (and later design) by identifying vulnerable components and possible retrofit techniques. In this chapter, the modelling and analysis are presented for two cases: the as-built experimental frame (Section 3.1), and retrofitted frames (Section 3.2).

3.1. As-built Frame Modelling and Analysis

The full-scale experimental test setup that will be built as part of this project (hereby referred to as ‘test frame’) consists of four identical 2-story, 2-bay RC frames
designed to resemble typical RC frame construction in the central and southeastern US (CSUS) prior to the 1970’s. The ACI 318-63 Building Code (ACI Committee, 1963) and the CRSI Design Handbook, Volume II, 1963 ACI Code (Reese, 1965) were used throughout the design process. In this study, all modeling parameters are selected to match the test frame design. Frame elevation is shown in Figure 3.1. Geometry and reinforcement layouts of beams and columns are shown in Figure 3.2.

![Figure 3.1 – Test frame – elevation view](image-url)
In accordance to typical practice at the time (Bracci, Reinhorn, & Mander, 1992), no consideration was given to lateral loads during the design. While structures designed for gravity loads still possess inherent lateral strength that may be capable of resisting moderate earthquakes (Hoffman, Kunnath, Mander, & Reinhorn, 1992; Kunnath, Hoffmann, Reinhorn, & Mander, 1995), potential deficiencies in detailing of members may result in poor performance during seismic activity (ACI Committee, 2002; Beres et al., 1992; Bracci et al., 1995). For example, Beres et al. (1992) identified the following details as potentially critical to safety during an earthquake: (1) longitudinal reinforcement ratio in columns not exceeding 2%, (2) lap splices of column reinforcement at the maximum moment region, (3) wide spacing of column ties that provides little concrete confinement, (4) little to no transverse reinforcement within the joint region, (5) bottom beam reinforcement with a short embedment length into the

Figure 3.2 – (a) Column, beam, and joint reinforcement details, (b) beam cross sections
column, (6) construction joints near the beam-column joints, and (7) columns having bending moment capacities close to those of the beams. All deficiencies except (1) and (6) were considered in this study.

For this study, nonlinear finite element analysis was performed in OpenSees (McKenna et al., 2009). A general schematic of the numerical model is shown in Figure 3.3. Finite element modeling allows for explicit consideration of column and beam moment capacities. To capture the effects of other deficiencies, component and material models validated by previous researchers were incorporated into the structural frame. Figure 3.4 shows backbone curve schematics for these component and material models, as well as fiber discretization for column and beam finite element fiber sections. A general overview of these models is shown next. For detailed descriptions, the reader is referred to the corresponding studies.

Figure 3.3 – Overview of the numerical model of the structure in OpenSees.
Figure 3.4 – Element discretization around interior column. Material constitutive curves (not to scale) shown for joint rotational spring, lap-splice reinforcement, and zero-length section bottom reinforcement

The Figure 3.5 shows a close-up view of the beams and columns discretization at the cross-sectional level. For both the beam section and the regular column section only 2 fibers were used for the concrete cover (unconfined concrete), while for the confine core 24 fibers and 16 fibers were used respectively. For the column base where lap splices are arranged a denser discretization compared to the regular column was selected (20 fibers for the confined core and 4 fibers for the unconfined cover) because the moment gradient changes more rapidly than close to column mid-height.
Table 3.1 – OpenSees Concrete02 material parameters used in the preliminary model

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<td>350 ksi</td>
</tr>
</tbody>
</table>
where the material parameter notation is as follows [OpenSees Wiki (2012)].

- \( f_{pc} \) concrete compressive strength at 28 days (compression is negative)*
- \( \varepsilon_{pc0} \) concrete strain at maximum strength*
- \( f_{pc} \) concrete crushing strength *
- \( \varepsilon_{pU} \) concrete strain at crushing strength*
- \( \lambda \) ratio between unloading slope at \( \varepsilon_{pU} \) and initial slope
- \( f_t \) tensile strength
- \( E_{ts} \) tension softening stiffness (absolute value) (slope of the linear tension softening branch)

Table 3.2 – OpenSees steel rebar material model parameters (Giuffré-Menegotto-Pinto Model – Steel 02) used in the preliminary model

<table>
<thead>
<tr>
<th></th>
<th>Columns</th>
<th>Slab beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st story</td>
<td>2nd story</td>
</tr>
<tr>
<td>( f_y )</td>
<td>75.0 ksi</td>
<td>75.0 ksi</td>
</tr>
<tr>
<td>( E )</td>
<td>29,000 ksi</td>
<td>29,000 ksi</td>
</tr>
<tr>
<td>Strain hardening ratio, ( b )</td>
<td>0.01</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Lap splices in regions of maximum moment (e.g. just above the foundation – Figure 3.2) have been shown to contribute to strength and stiffness degradation in columns (Aboutaha et al., 1996; Melek & Wallace, 2004). Barkhordary and Tariverdilo (2011) developed an analytical model to capture this degrading behavior. The model was validated with experimental tests from Melek and Wallace (2004) and Aboutaha et al. (1996). These tests had column geometry and reinforcement layouts comparable to those of the test frame columns (e.g. 24” lap splice length, #8 rebars). Therefore, the model of Barkhordary & Tariverdilo is used in this study. The spliced reinforcement backbone relationship is shown in Figure 3.4 (bottom right). Corresponding parameters are shown in Table 3.3. In this model, the effects of lapped bars are captured at the material level by modifying the tensile stress-strain behavior of the elements used to model the steel. The
compression strain stress is modeled as typical steel rebar (i.e. no modification). Given the short length of the splice being modelled, it was assumed that the bar would slip before yield occurred. The max bar stress is calculated by assuming a crack surface around each spliced bar with the height equal to the lap splice length and a perimeter, p. The maximum developable force of the bars is calculated by multiplying the area of this failure surface by the tensile strength of concrete. Assuming that the slip resistance is provided by a truss mechanism of 45 degrees between the bars and the surrounding concrete, the tensile stress in concrete will be equal to the bond stress of the spliced bar. By this assumption, it is possible to develop a relationship between the maximum bar stress and the tensile strength of the concrete, as follows:

\[ f_{s,max}A_b = f_t p l_s \]  

(3.1)

where \( A_b \) is the bar cross sectional area, \( f_t \) is the tensile strength of concrete (0.33 \( f'_c \) MPa), where \( f'_c \) denotes stress corresponding to the concrete compressive strength), \( l_s \) is the lap splice length, and \( p \) is the perimeter of the failure surface, which is defined as follows for rectangular columns:

\[ p = \frac{s}{2} + 2(d_b + c) \leq 2\sqrt{2}(d_b + c) \]  

(3.2)

where \( s \) is the average distance between spliced bars, \( c \) is the concrete cover and \( d_b \) is the diameter of the longitudinal bars. The upper limit in this equation typically applies to widely spaced bars. Increasingly larger crack openings initiate softening in the bar stress-strain behavior. However, at large slips there is a zone of residual stress, which corresponds to frictional stress developed in the failure surface. Using the shear-friction
concept, it is assumed that the transverse reinforcement crossing the crack plane provides the necessary friction to transfer bond stresses. Considering a friction factor $\mu$ on the failure surface, the frictional stress on the crack plane can develop the following residual force in the longitudinal bars:

$$\mu n_l n_t A_h f_{yh} = n A_b f_r$$  \hspace{1cm} (3.3)

where $n_l$ denotes the number of transverse reinforcement legs perpendicular to the crack plane, $n_t$ is the number of transverse reinforcements in the lap splice length, $f_{yh}$ is the yield strength of transverse reinforcement (with a maximum of $0.015E_s$) where $E_s$ is the bar’s modulus of elasticity. In this equation, $n$ is the number of spliced longitudinal bars developed by frictional stress in the crack plane. The friction factor, $\mu$, was assumed equal to 0.4 (Raous & Karray, 2009) in this study.

The deformation due to bar slip localizes in the length $l_{ss}$. The slipped bar strain corresponding to peak stress for the case of slip before yielding is calculated as:

$$\varepsilon_s = \varepsilon_{st} + \varepsilon_{ss} = \frac{f_{s,max}}{E_s} + \frac{\delta_{s,max}}{l_{ss}}$$  \hspace{1cm} (3.4)

where $\delta_{s,max}$ is the slip displacement at max stress. Barkhordary and Tariverdilo (2011) assumed this slip was equal to 1mm. In the present study, the same value was assumed for the preliminary model. The slip was then calibrated to match empirical results from the companion study test frame (Wright, 2015) in the updated model.

Table 3.3 – Lap splice reinforcement parameters – refer to Figure 3.4

<table>
<thead>
<tr>
<th>$F_{s,max}$</th>
<th>$\varepsilon_s$</th>
<th>$f_r/f_y$</th>
<th>$\varepsilon_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.71</td>
<td>0.0047</td>
<td>0.39</td>
<td>0.035</td>
</tr>
</tbody>
</table>
To assess the performance of the beam-column joint region, joint shear behavior and poor anchorage of beam reinforcement are considered. The model proposed by Celik and Ellingwood (2009) is used to establish a joint moment-rotation (M-θ) relationship. The M-θ backbone is derived from the shear stress-strain relationship of the joint using the joint geometry and equilibrium as described in Equation 3.5 and Equation 3.6:

\[
M_j = \frac{v_j A_j}{\left(1 - \frac{b_j}{L_b}\right) / j d_b - \alpha / L_c}
\]  

\[
\theta_j = \gamma_j
\]

where \(M_j\) = equivalent joint rotational moment; \(v_j\) = joint shear stress; \(A_j\) = joint area; \(L_b\) = beam total length; \(b_j\) = joint effective width; \(L_c\) = column total length; \(j\) = internal moment arm factor (assumed to be 0.875); \(d_b\) = beam effective depth; \(\alpha\) = constant equal to 2 for the top floor joints and 1 for the others; \(\theta_j\) = joint rotation; and \(\gamma_j\) = joint shear strain. Since the joint rotation is the angle change between the two adjacent edges of the panel zone, the joint rotation equals the joint shear strain. Figure 3.3 (top right) depicts the joint rotational spring M-θ relationship. Joint rotation (θ) values are shown in Table 3.4.

Table 3.4 – Joint rotational spring parameter, θ (rad)*

<table>
<thead>
<tr>
<th></th>
<th>Cracking</th>
<th>Yielding</th>
<th>Ultimate</th>
<th>Residual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0007</td>
<td>0.006</td>
<td>0.02</td>
<td>0.065</td>
</tr>
</tbody>
</table>

*θ values chosen from empirical data (Celik & Ellingwood, 2009)
Celik and Ellingwood calibrated their model against a series of experimental sub-assembly tests with no transverse reinforcement in the joint. The tests included joints with well anchored beam reinforcement (Walker, 2001) as well as beam reinforcement with short embedment lengths (Pantelides, Hansen, Nadauld, & Reaveley, 2002). For beams with poorly anchored bottom reinforcement, the joint M-θ envelope was reduced to account for the decreased beam negative moment capacity. However, additional rotation due to reinforcement slip was ignored. The parameters for the joint rotation spring used in the preliminary model are shown in Table 3.5.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>External beam-column joint</th>
<th>Internal beam-column joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_{Pf1}$, $e_{Pf2}$, $e_{Pf3}$, $e_{Pf4}$</td>
<td>[975 3,187 3,564 975] in-kips</td>
<td>[1,071 3,302 3,701 1,071] in-kips</td>
</tr>
<tr>
<td>$e_{Pd1}$, $e_{Pd2}$, $e_{Pd3}$, $e_{Pd4}$</td>
<td>[0.0007 0.006 0.02 0.065] rad</td>
<td>[0.0007 0.006 0.02 0.065] rad</td>
</tr>
<tr>
<td>$e_{Nf1}$, $e_{Nf2}$, $e_{Nf3}$, $e_{Nf4}$</td>
<td>negative of positive parameters</td>
<td>negative of positive parameters</td>
</tr>
<tr>
<td>$e_{Nd1}$, $e_{Nd2}$, $e_{Nd3}$, $e_{Nd4}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$r_{DispP}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$f_{ForceP}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$u_{ForceP}$</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>$r_{DispN}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$f_{ForceN}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$u_{ForceN}$</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>$g_{K1}$, $g_{K2}$, $g_{K3}$, $g_{K4}$, $g_{KLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$g_{D1}$, $g_{D2}$, $g_{D3}$, $g_{D4}$, $g_{DLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$g_{F1}$, $g_{F2}$, $g_{F3}$, $g_{F4}$, $g_{FLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$gE$</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$dmgType$</td>
<td>energy</td>
<td>energy</td>
</tr>
</tbody>
</table>

Unlike the Celik & Ellingwood model, no strength reduction factor is assigned to the joint rotational spring in the present study. Instead, the behavior of beam
reinforcement with short embedment length is considered using a bond slip displacement model formulated by Berry & Eberhard (2007). In this model, the OpenSees zeroLengthSection element is assigned to beam ends at the joint face. The zeroLengthSection allows for explicit modeling of the section geometry and reinforcement, but the materials are given a stress-displacement (σ-Δ), rather than stress-strain, relationship. The concrete σ-Δ envelope follows the formulation of Berry and Eberhard, while the bottom steel reinforcement σ-Δ (values shown in Table 3.6) was formulated using bond values proposed by Mitra and Lowes (2007). Figure 3.4 (top left) shows the stress-displacement curve for the beam bottom steel reinforcement.

It is assumed that accounting for joint shear behavior and reinforcement slip separately (as described above) allows for more flexibility when modeling the test frame in a retrofitted state. To ensure that appropriate behavior of the joint region was captured, analytical and experimental results of interior and exterior joint subassemblies were compared. The joint model described above (rotational spring plus zeroLengthSection σ-Δ element) was compared to results from Celik and Ellingwood (2008), and the corresponding experimental tests (Pantelides et al., 2002; Walker, 2001). Analytical results from this study’s joint model are well correlated with aforementioned analytical and experimental results.

The model of Mander, Priestley, and Park (1988) was used to account for differences in ductility and compressive strength of confined and unconfined concrete. Only the sections with closed stirrups (cross-sections 1 in Figure 3.2) were assumed to have confined cores. Joint moment-rotation relationships were calculated using moment-curvature analysis for every connection and the effective slab width was defined
according to the recommendations of ACI 318-05 Building Code (ACI Committee, 2005).

**Table 3.6 – Zero-length section reinforcement (beam bottom rebar) parameters**

<table>
<thead>
<tr>
<th>Stress</th>
<th>$f_{1+}/f_y$</th>
<th>$f_{2+}/f_y$</th>
<th>$f_{3+}/f_y$</th>
<th>$f_{1-}/f_y$</th>
<th>$f_{2-}/f_y$</th>
<th>$f_{3-}/f_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.64</td>
<td>0.64</td>
<td>0.15</td>
<td>-0.78</td>
<td>-1.25</td>
<td>-0.15</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Displacement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta_{1+}$</td>
</tr>
<tr>
<td>0.0049</td>
</tr>
</tbody>
</table>

In addition to component models described in the previous section, distributed plasticity elements were used to model columns and beams. For dynamic analysis, mass was lumped at every beam-column connection. Mass and stiffness proportional Rayleigh damping was considered in the first two modes. The critical damping ratio was assumed to follow a lognormal distribution with a mean value of 0.04 (Newmark & Hall, 1982) and a 25% coefficient of variation for a corresponding normal distribution (Healy, Wu, & Murga, 1980). Mean concrete compressive strength and steel yield strength were increased by 25% from their nominal values to account for conservatism in nominal to in-situ strength and apparent strength increase under seismic loading rates (Aslani & Miranda, 2005).

The test frame was designed assuming two-way slab action. A concrete unit weight of 145 pcf was assumed for self-weight calculations. A triangularly distributed load was assigned to the beams during the design process. OpenSees only allows
modeling of uniformly distributed loads. Thus, triangular gravity loads were discretized into uniform loads for each individual beam element as shown in Figure 3.6. Note that in the refined model used to validate the model against test results (Section 6.1), a different loading configuration was considered to reflect the true experimental specimen load distribution.

**Figure 3.6 – Gravity load discretization for beam elements. The arrangement shown here corresponds to beams on the frame’s left bay. The frame is symmetric about the interior column.**

3.1.1. As-Built Test Frame Fragility

This section covers the seismic vulnerability assessment of the as-built test frame. It includes descriptions of the ground motion suite and probabilistic parameters used in the seismic demand analysis, and the development of fragility curves.
3.1.1.1. **Ground Motion Suite**

A suite of 240 synthetic ground motions developed by Fernandez and Rix (2006) are used to perform Nonlinear Time History Analysis (NTHA). These probabilistic ground motions were generated for eight cities within the upper Mississippi Embayment. The suite contains ground motions consistent with hazard levels of 10%, 5%, and 2% probability of exceedance in 50 years and is considered representative of the seismic hazard in the CSUS.

3.1.1.2. **Probabilistic Seismic Demand Model (PSDM) and Fragility Curves**

The 240 Rix-Fernandez ground motions are randomly paired with 240 test frame models to create 240 frame – ground motion pairs which are statistically significant and nominally identical. Uncertainty in modeling parameters was considered using a Latin Hypercube sampling technique (McKay, Beckman, & Conover, 1979). The concrete compressive strength ($f_c$), steel yield strength ($f_y$), and damping ratio ($\zeta$) were treated as random variables with the associated probability distributions shown in Table 3.7. Considering all uncertainties, eigenvalue analysis revealed a range of fundamental periods from 0.42 s to 0.64 s for the full suite of 240 frame-ground motion pairs.

**Table 3.7 – Modeling uncertainties**

<table>
<thead>
<tr>
<th>RVs</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c$ (ksi)</td>
<td>5.0</td>
<td>0.18</td>
<td>Normal</td>
</tr>
<tr>
<td>$f_y$ (ksi)</td>
<td>75.0</td>
<td>0.11</td>
<td>Lognormal</td>
</tr>
<tr>
<td>$\zeta$</td>
<td>0.05</td>
<td>0.25</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

*Damping ratio parameters were chosen for agreement with those suggested by Healy et al. (1980), Newmark and Hall (1982), and Nielson and DesRoches (2007).*
A full NLTHA is performed for each frame – ground motion pair and the maximum structural demand (e.g. inter-story drift) is recorded. A seismic intensity measure (IM) is chosen. Then, assuming that the median seismic demand can be predicted from a power law model (Cornell, Jalayer, Hamburger, & Foutch, 2002), a linear regression of the demand-intensity measure pairs is performed in the log-transformed space. The regression is used to formulate the so called PSDM in terms of a lognormal distribution. Several researchers have studied the effects of using different IMs. A summary may be found in Padgett et al. (2008).

In this study, the peak ground acceleration (PGA) and spectral acceleration at the fundamental period of the frame ($S_{a-T1}$), at 1 s ($S_{a-1.0s}$), and at 0.2 s ($S_{a-0.2s}$) are chosen as IMs for the as-built frame. Figure 3.7 shows the PSDMs for maximum inter-story drift ($\theta_{max}$) as a function of the aforementioned IMs. Comparing the IMs shows that Sa-1 is the most efficient (lowest dispersion, $\beta_{d/IM}$). Thus, this demand model will be used to develop fragility curves for the as-built frame, as well as the frame with components in a retrofitted state. For a detailed description on the choice of optimal IM, refer to Padgett et al. (2008).
The capacity limit state for maximum inter-story drift (%) is also assumed to be lognormally distributed. The median values obtained from HAZUS-MH (FEMA, 2003) are 0.5, 0.8, 2.0, and 5.0 for the slight, moderate, extensive, and complete damage states, respectively. The prescriptive capacity dispersions for the lower and higher limit states are 0.25, and 0.47, respectively.

Fragility curves consider the probability that the seismic demand (D) placed on the structure exceeds the capacity (C) conditioned on the chosen IM. Having defined the
PSDMs and limit states as described in the previous sections, the fragility is evaluated as in Equation 3.7:

\[ P[D > C|IM] = \Phi \left[ \frac{\ln(S_d/S_c)}{\sqrt{\beta_d^2 + \beta_c^2 + \beta_m^2}} \right] \]  

(3.7)

where \( S_d \) and \( \beta_{d|IM} \) = median value and dispersion of the demand as a function of \( IM \), respectively; \( S_c \) and \( \beta_c \) = median value and dispersion of the capacity limit states, respectively; \( \beta_m \) is the modeling uncertainty (assumed to be 0.2, Celik & Ellingwood, 2010) and \( \Phi[\cdot] \) = standard normal cumulative distribution function. Figure 3.8 illustrates the fragility curves for the as-built test frame using \( S_{a-1.0s} \) as the IM.

![Figure 3.8 – Fragility curves for the preliminary as-built test frame](image)

Figure 3.8 – Fragility curves for the preliminary as-built test frame
3.2. Retrofitted Frame Modelling and Analysis

The initial retrofit evaluation compared multiple options to address the multiple deficiencies. The analytical modelling is discussed next, followed by the preliminary results, where fragility curves were developed and used to compare the seismic performance of potential retrofits. To aid in selection of experimental retrofits, several cases of a retrofitted building are analyzed and compared to the as-built test frame. Three fragility estimates of the test frame in a retrofitted state are presented in the following sections. Section 3.2.1 presents the PSDM and fragility curves for the test frame considering a retrofitted column lap splice region. Fragility of the frame with a retrofitted beam-column joint region is shown in Section 3.2.2. Lastly, frame fragility considering an SMA bracing system is shown in Section 3.2.3.

3.2.1. Fragility of the frame with retrofitted lap splice

Previous research has shown that retrofitting columns with deficient lap splices via column jacketing along the splice length may effectively prevent lap splice failures (ElGawady et al., 2010) and increase column performance to that of a column with splice development lengths that account for possible tensile demands. Thus, a retro-fitted column was modeled by assigning a reinforcing steel material with strain hardening (i.e. Steel02 in OpenSees) to the lap splice region, rather than the constitutive relation shown in Figure 3.4. Figure 3.9 and Figure 3.10 show the PSDM and fragility curves, respectively, for the test frame with retrofitted lap splice regions.
Figure 3.9 – PSDM for the test frame with retrofitted a lap splice region

\[ \theta_{\text{max}} = 3.3275 S_{a-1.0s}^{0.94881} \]
\[ \beta_{d|M} = 0.43382 \]
\[ R^2 = 0.79627 \]

Figure 3.10 – Fragility for the test frame with a retrofitted lap splice
3.2.2. *Fragility of the Frame with Retrofitted Beam Column Joints*

For the purpose of an initial fragility estimate, it is assumed that a retrofitted beam-column joint region may prevent additional beam rotation due to reinforcement slip. Such a retrofit may also allow the beams to develop a larger fraction of their flexural capacity (Bracci et al., 1995; FEMA, 2006). For modeling purposes, this behavior is achieved by removing the `zeroLengthSection` element (Figure 3.4) from the analytical model. The PSDM conditioned on $S_{a-1.0s}$ is shown in Figure 3.11. Fragility curves were calculated for the same four limits states described in Section 3.1.1.1. The fragility curves are shown in Figure 3.12.

![PSDM for the test frame with retrofitted joints](image)

**Figure 3.11 – PSDM for the test frame with retrofitted joints**
3.2.3. Fragility of the Frame with an SMA Bracing Retrofit

For the initial fragility estimate of a frame retrofitted with a SMA brace, the brace was assumed to have a 4 ft SMA stroke length. The brace element had a fixed connection to the beam column centerlines, and the non-SMA elements of the brace were elastic beam elements with the stiffness of typical structural steel (i.e. $E_s = 29,000$ ksi), with an area of $15\text{in}^2$. The PSDM conditioned on $S_{a-1.0s}$ is shown in Figure 3.13. Relative to the as-built frame and the two retrofitted frames discussed previously in this chapter, there is more dispersion in the distribution of drift values for this frame. Arguably, this could be due to the braces providing a higher increase in lateral stiffness than the other two retrofits. Thus, a 1.0 sec. spectral acceleration better matches the response period of the
other frames than this one. However, the same intensity measure was used for comparison purposes. Fragility curves are shown in Figure 3.14.

Figure 3.13 – PSDM for the test frame with a brace retrofit

Figure 3.14 – Fragility curves for the test frame with a brace retrofit
3.3. Comparison of Fragility Curves for As-built and Retrofitted Test Frames

Table 3.8 shows the median spectral acceleration values for all limit states in the three structures presented in this chapter. It can be seen that retrofitting the joint region without retrofitting the column worsens the seismic performance. The median ground motion that will exceed the complete damage state decreases from 1.137g for the as-built frame to 1.074g to the frame with retrofitted joints (a 5.5% decrease). Conversely, a retrofit in the lap splice region significantly improves the seismic performance across all limit states. The best improvement in seismic performance comes from adding a brace retrofit, with a significant improvement in the median fragility for all limit states, compared to the as-built frame.

Table 3.8 – Median spectral acceleration values (g) to exceed limit states

<table>
<thead>
<tr>
<th>Structure</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-built</td>
<td>0.124</td>
<td>0.195</td>
<td>0.471</td>
<td>1.137</td>
</tr>
<tr>
<td>Retrofitted splice</td>
<td>0.136</td>
<td>0.223</td>
<td>0.585</td>
<td>1.536</td>
</tr>
<tr>
<td>Retrofitted joints</td>
<td>0.123</td>
<td>0.191</td>
<td>0.453</td>
<td>1.074</td>
</tr>
<tr>
<td>Brace retrofit</td>
<td>0.170</td>
<td>0.326</td>
<td>1.153</td>
<td>4.082</td>
</tr>
</tbody>
</table>

3.4. Concluding Remarks

The seismic fragility assessment of a 2-story, 2-bay non-ductile RC building was carried out using non-linear time history analyses (NLTHA). This structure is representative of pre-1970’s construction in low-to-moderate seismic zones in the US.
Thus, several reinforcement detailing deficiencies were considered in the as-built frame analytical model. To aid in the selection of retrofits for future experimental testing and establish a basis for further modeling of retrofits, two of these deficiencies, namely short lap splices near the column foundation and inadequate anchorage of beam reinforcement, were also modeled in a retrofitted state. Preliminary fragility estimates were developed for the structure in three conditions: as-built, with retrofitted column lap splices, and with retrofitted beam-column joints.

Under the assumed inter-story drift-intensity measure relationship, the 5% spectral acceleration at 1 s provided the most efficient PSDM. This is indicative of global stiffness degradation since there is more dispersion in the PSDM conditioned on spectral acceleration at the fundamental period of the undamaged structure. That said, there is a large amount of dispersion in the upper range of ground motion intensity for both IMs. Several data points in this region lie outside the one-standard deviation value of the regression model and above a 10% inter-story drift level. Model updating using experimental data is required to improve the accuracy of these values.

Retrofitting the column lap splice region improved the seismic performance for all limit states. This result indicates that the as-built structure is strongly affected by an inadequate beam-to-column strength ratio (i.e. a ‘strong-beam weak-column’ design).

The probability of exceeding the complete damage state increased slightly for a structure with retrofitted beam-column joints. Further studies are needed to determine if this result is a product of the chosen PSDM, or if the increase in the beam-to-column strength ratio (due to the beam developing a larger proportion of its ultimate strength)
caused a corresponding increase in the drift demand. Due to no experimental frames being retrofitted solely in the beam-column-joint region in either the current study or Wright’s companion study, this point remains a suggestion for future research.

Adding a brace retrofit significantly improved structural performance for all limit states, compared to the as-built frame as well as the frames with retrofits that directly addressed column or beam column joint reinforcement deficiencies. Given this improvement in performance, it was agreed to continue the experimental portion of this research with a SMA brace retrofit.
3.5. References


ACI Committee. (2005). *Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05).* Detroit, MI: American Concrete Institute.


CHAPTER 4. Preliminary Design of SMA Brace Device

Following the comparative analysis described in Chapter 3, a SMA bracing device was chosen to retrofit one of the experimental test frames. In this chapter, the design and installation process of this device is described. Section 4.1 describes the initial testing of the SMA components. This testing was undertaken to understand the material behavior, and to sort out any manufacturing, delivery, and installation issues before design and assembly of the full-scale test specimen. The complete design procedure of the full-scale device is described in Section 4.2, including SMA and steel components, plus connections and anchoring to concrete.

4.1. Initial Element Testing (Wires & Rods)

Due to differences in ease and cost effectiveness of machining and fabrication of SMA components, a decision was made to design and test two separate brace retrofit solutions, each one incorporating a different SMA device. After discussions with the fabricator and materials scientist (D. Hodgson, Nitinol Technology, Inc.), it was concluded that enhanced seismic performance could be achieved using braces where the main damping and re-centering component (i.e. the SMA part) was either a series of pre-tensioned strands or a pre-tensioned rod. SMAs in these specific configurations have not been previously tested to strain limits that would be applicable to seismic displacement demands. Thus, to better understand the behavior of these elements before finalizing the retrofit design, a series of smaller specimens –fabricated using the same methods that would be used for the full-scale specimens– were tested up to increasing strain limits.
The test specimens included two circular SMA rods, each with a 2 ft. total length (13.5 in. gage length) and a 0.851 in. diameter, as well as two sets of bundled SMA wires (2 mm wire), also with a 2 ft. total length, and 10 total wires (double-sided) per bundle. Test results for the SMA wires and rods are presented in Section 4.1.1 and Section 4.1.2, respectively.

4.1.1. Testing and Results: SMA Wires

Preliminary test specimens and tensile test setup are shown in Figure 4.1 and Figure 4.2. Two preliminary sets of wire specimens were tested. The 1st specimen that was sent by the manufacturer, shown in Figure 4.1, had an end connection detail that failed in a brittle manner after just one cycle, so no results are reported. The manufacturer then sent a 2nd specimen, with a different end connection, as shown in Figure 4.2a. This connection was tested using a transfer setup similar to the one that was planned for the full-scale brace (Figure 4.2c). The preliminary test results showed that this connection performed adequately so a similar connection detail was designed for the full-scale specimen (details in Chapter 5).

The 2nd specimen consisted of 0.07874 in. (2 mm.) wire, wound in 2 10-wire layers, with a 2 ft gage length. For these tests, due to equipment limitations, no unloading test data was recorded. Only the loading phase of the tensile tests was measured. However, this was enough information for experimental test design purposes, since the driving factors behind the design were the stress and strain during the yield and fully transformed martensite phases, needed to determine area and stroke length.
Figure 4.1 – Wire end connection of initial test specimen. As seen on the right photo, this connection resulted in a brittle failure.

Figure 4.2 – (a) SMA wire end connection, (b) tensile test setup, and (c) wire test setup connection detail.
Tests were conducted from 2% strain to 12%, in 2% increments (the specimen failed at 11.4% strain). Results indicate expected NiTi SMA behavior, with a well-defined austenite-to-martensite transformation at approximately 2% strain followed by a stress plateau and then a typical stress-induced martensite stiffening after approximately 8% strain. Figure 4.3 shows the detailed test data and modulus calculations for the 6% strain test. Figure 4.4 shows the test data for all 6 tests. Results parameters of interest are summarized in Table 4.1.

Figure 4.3 – Results for 6% strain SMA wire test
Figure 4.4 – Results for all SMA wire tests

Table 4.1 – Test results for SMA wire specimen

<table>
<thead>
<tr>
<th>Specimen gage length, in:</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter per wire, in:</td>
<td>0.07874</td>
</tr>
<tr>
<td>Total area (10 wires, double sided), in²:</td>
<td>0.0974</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Test 4</th>
<th>Test 5</th>
<th>Test 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Elongation, %</td>
<td>2.0</td>
<td>4.0</td>
<td>6.0</td>
<td>8.0</td>
<td>10.0</td>
<td>11.4</td>
</tr>
<tr>
<td>Young’s Modulus, psi</td>
<td>5620000</td>
<td>5413300</td>
<td>5342000</td>
<td>4790669</td>
<td>5249000</td>
<td>4697500</td>
</tr>
<tr>
<td>Load @ 1% strain, lbf</td>
<td>5220</td>
<td>5093</td>
<td>5093</td>
<td>4801</td>
<td>4967</td>
<td>4626</td>
</tr>
<tr>
<td>Load at 0.2 offset, lbf</td>
<td>N/A</td>
<td>6817</td>
<td>6817</td>
<td>6915</td>
<td>6817</td>
<td>6622</td>
</tr>
<tr>
<td>Peak Load, lbf</td>
<td>6593</td>
<td>8547</td>
<td>8856</td>
<td>9309</td>
<td>12013</td>
<td>15230</td>
</tr>
<tr>
<td>Stress @ 1% strain, psi</td>
<td>53600</td>
<td>52300</td>
<td>52300</td>
<td>49300</td>
<td>51000</td>
<td>47500</td>
</tr>
<tr>
<td>Stress at 0.2% offset, psi</td>
<td>N/A</td>
<td>70000</td>
<td>70000</td>
<td>71000</td>
<td>70000</td>
<td>68000</td>
</tr>
<tr>
<td>Peak Stress, psi</td>
<td>67697</td>
<td>87760</td>
<td>90930</td>
<td>95584</td>
<td>123350</td>
<td>156380</td>
</tr>
</tbody>
</table>
It is important to note that the martensite phase peak stress was more than double the load at 2% offset. For the retrofit device, this signaled the importance of limiting the SMA strain to 8%, in order to avoid a significant overstrength demands on steel and anchoring components of the retrofit. The 0.2% offset stress roughly coincided with the stress at 2% strain.

The retrofit device design requires pre-tensioned SMA wires. In order to avoid wires going slack due to permanent ‘set’, the pretension was set at 2% strain. This allowed for a usable range of SMA strain of 6% (2%-8%).

4.1.2. Testing and Results: SMA Rods

The SMA rods were tested outside Georgia Tech labs due to equipment availability. Tests were performed by TEC Services (https://tecservices.com/) in Lawrenceville, GA. Test specimens consisted of two circular SMA rods, each with a 2 ft. total length (13.5 in. gage length) and a 0.851 in. diameter. Tests were conducted from 3.5% crosshead elongation (2% extensometer) to 6% crosshead elongation (6% extensometer) for specimen 1, and 2% crosshead elongation to 8% for specimen 2 (details in Table 4.2 and Table 4.3). In contrast to the SMA wire specimen, which failed at 11.4% strain, these results point to issues with the SMA threaded connection. Both specimens failed below a 7.5% elongation, specimen 1 due to fracture of the threaded rod portion (see Figure 4.5 for test results and Figure 4.6 for the specimen after failure), and specimen 2 due to shearing failure in the threaded nut (presumably due to the fine threads). Test result details are shown in Table 4.2 for specimen 1 and Table 4.3 for specimen 2.
Similar to the SMA wires, the rods show a well-defined ‘yield’ point within range of 49 to 55 ksi following a transition from initial linear behavior (Figure 4.5). However, the usable strain range after yield was smaller than that of the wires, thus, the basis of design for full-scale brace specimen was done to accommodate the usable range (i.e. the ‘plateau’ region before full martensite transformation) of the SMA wires (details in Section 4.2.2).

Figure 4.5 – Results for 6% strain SMA rod test (specimen failed before reaching 6%)
Table 4.2 – SMA rod Specimen 1 - Tensile Test Results

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Test 4</th>
<th>Test 5&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gage length, in</td>
<td>13.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter, in</td>
<td>0.815</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area, in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>0.569</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Elongation, Crosshead, %</td>
<td>3.5</td>
<td>5.0</td>
<td>4.0</td>
<td>4.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Young’s Modulus, psi</td>
<td>6882600</td>
<td>6752700</td>
<td>6496400</td>
<td>6606900</td>
<td>5980600</td>
</tr>
<tr>
<td>Load @ 1% strain, lbf</td>
<td>32101</td>
<td>31258</td>
<td>30147</td>
<td>29357</td>
<td>28058</td>
</tr>
<tr>
<td>Load at 0.2 offset, lbf</td>
<td>32263</td>
<td>31436</td>
<td>30344</td>
<td>29176</td>
<td>28294</td>
</tr>
<tr>
<td>Peak Load, lbf</td>
<td>33872</td>
<td>37725</td>
<td>34933</td>
<td>34816</td>
<td>40270</td>
</tr>
<tr>
<td>Stress @ 1% strain, psi</td>
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<td>54900</td>
<td>53000</td>
<td>51600</td>
<td>49300</td>
</tr>
<tr>
<td>Stress at 0.2% offset, psi</td>
<td>56900</td>
<td>55200</td>
<td>53300</td>
<td>51300</td>
<td>49700</td>
</tr>
<tr>
<td>Peak Stress, psi</td>
<td>60000</td>
<td>66000</td>
<td>61000</td>
<td>61000</td>
<td>71000</td>
</tr>
<tr>
<td>Max Elongation, Extensometer, %</td>
<td>2.0</td>
<td>6.0</td>
<td>Ext slip</td>
<td>Ext slip</td>
<td>6.0</td>
</tr>
</tbody>
</table>

<sup>a</sup>Test 5 – threaded portion failed at both ends (see Figure 4.6)

Table 4.3 – SMA rod Specimen 2 - Tensile Test Results

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Test 4&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Test 5&lt;sup&gt;b&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gage length, in</td>
<td>13.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter, in</td>
<td>0.851</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area, in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>0.569</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Elongation, Crosshead, %</td>
<td>2.0</td>
<td>4.0</td>
<td>6.0</td>
<td>8.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Young’s Modulus, psi</td>
<td>6895000</td>
<td>6814400</td>
<td>6664000</td>
<td>6266500</td>
<td>6388300</td>
</tr>
<tr>
<td>Load @ 1% strain, lbf</td>
<td>32836</td>
<td>32555</td>
<td>31967</td>
<td>29800</td>
<td>27575</td>
</tr>
<tr>
<td>Load at 0.2 offset, lbf</td>
<td>33233</td>
<td>32866</td>
<td>32128</td>
<td>30120</td>
<td>26990</td>
</tr>
<tr>
<td>Peak Load, lbf</td>
<td>34652</td>
<td>35269</td>
<td>40430</td>
<td>41310</td>
<td>38279</td>
</tr>
<tr>
<td>Stress @ 1% strain, psi</td>
<td>57700</td>
<td>57200</td>
<td>56200</td>
<td>52400</td>
<td>48500</td>
</tr>
<tr>
<td>Stress at 0.2% offset, psi</td>
<td>58400</td>
<td>57800</td>
<td>56500</td>
<td>52900</td>
<td>47400</td>
</tr>
<tr>
<td>Peak Stress, psi</td>
<td>61000</td>
<td>62000</td>
<td>71000</td>
<td>73000</td>
<td>67000</td>
</tr>
<tr>
<td>Max Elongation, Extensometer, %</td>
<td>2.0</td>
<td>2.0</td>
<td>5.5</td>
<td>7.5</td>
<td>N/A</td>
</tr>
</tbody>
</table>

<sup>a</sup>Test 4 – Coupling threads started shearing
<sup>b</sup>Test 5 – Coupling threads failed (shear)
4.2. Brace Design Procedure

The SMA stress-strain test results shown in the previous section were used in the retrofit design process to determine the total SMA area and stroke needed to meet inter-story drift criteria in the retrofitted frame. Along with these results, both the as-built model and a preliminary SMA braced model were subjected to ground motion sets to determine reliability-based target drifts. In addition, a linear static analysis procedure was carried out to ASCE 41-06 (2007) to determine peak shear demands in each story and subsequently each brace. The procedure is outlined below.

4.2.1. Global Design Methodology

The geometry and loading parameters detailed in the companion study (Wright, 2015) were used to inform the global design of the brace retrofit. The SMA area was designed according to analysis results from the equivalent lateral force (ELF) procedure.
in ASCE 7-10. The full details of the ELF are found in Appendix A. The basic assumptions for the retrofit design for the retrofit are shown below:

- The building is near Memphis, TN (to follow the assumed Central & Southeastern US research basis, exact location shown in Appendix A)
- Site class was assumed as D
- The structures fundamental period was taken as $T_n = 0.48$ secs (from the Opensees model)
- The building has a risk category II

Assuming a site latitude and longitude of 35.697 and -89.82, respectively, and the above parameters, yielded a required design force per brace of 29.32 kips. Again, for the full analysis procedure, refer to Appendix A.

The brace target stroke was determined from preliminary fragility analysis (details in Chapter 3 and Appendix A). Peak story drifts and peak SMA strains were recorded in each run. Peak demands were checked and compared to the design criteria previously stipulated to determine satisfactory performance. SMA stroke length was iteratively changed until criteria were satisfied and performance achieved the target reliability (Appendix A). It was determined that the brace needed to accommodate an SMA deformation between 1.3 and 4 inches. Given the results from SMA wire and rod testing, there was a desire to limit the max SMA strain to approximately 7.5%. Since an approximately 2.5% pre-strain would be applied (for stiffness purposes and to prevent the SMA element from going slack due to permanent set), an additional 4 in. max deformation required a 6.0 ft SMA gage length. A 6 ft SMA length would limit the max
strain to 8-8.5% (4in/72in = 5.6% strain, added to 2.5% pre-strain), ensuring the full martensite phase transformation would not be completed.

Given the preliminary results, a preliminary design procedure was developed for the full SMA retrofit design. The design, detailed in the remainder of this Chapter, considered the required anchoring to existing concrete elements, transfer of SMA forces to the steel elements, and constructability issues. A proposed refined design procedure, following the full-scale experimental testing, is detailed in Chapter 7.

4.2.2. Detailed Steel Design Checks

The full brace devices consists an SMA component (either wires of rods, similar to the specimens shown in Section 4.1) and seven distinct steel components to help transfer forces between the RC frame and the SMA element. Starting from the RC frame, the load path goes through the steel elements in the following order to reach the SMA element:

- Steel anchors
- Steel adapter piece
- Gusset plate and knife plates
- Steel tube end plates
- Steel tubes and doubler plates
- Steel-SMA attachment piece

Design details are discussed next. The steel-SMA attachment piece details are shown in Section 4.2.2.1. Section 4.2.2.2 shows the steel tubes and doubler plates. The
end plates are shown in Section 4.2.2.3. Finally, Section 4.2.2.4 covers the steel adapter pieces, gusset plates, and concrete anchors.

4.2.2.1. Steel-SMA Attachment Piece

The SMA component (wire or rod) needed a path to transfer forces to the steel tubes, and a pre-tensioning method. The thickness of the attachment piece side plates was designed to minimize elastic deformations, which could lead to adverse pre-stress losses. The attachment piece end plate was designed to provide adequate bearing area during pre-stress jacking. The side plates and end plates were welded together using fillet welds. Appendix B shows calculations for bearing area, elastic deformations, and welds. An image of the attachment piece, along with the still uninstalled SMA strands, is shown in Figure 4.7a. A schematic of the attachment piece inside the steel tubes is shown in Figure 4.7b.

Figure 4.7 – (a) Steel attachment piece (with yet uninstalled SMA strands), and (b) schematic of SMA strands and steel tube cross-sections.
4.2.2.2. Steel Tubes and Doubler Plates

The global brace design followed a capacity design philosophy where the steel tubes were intended to remain elastic under all expected SMA forces. The design intention was to have a fully reusable retrofit component where the inelastic deformation occurs only in the SMA component so that post-event deformations are recoverable. Rectangular hollow structural sections (HSS) were chosen for two reasons:

1) A tube-in-tube configuration, symmetrical in section with respect to the SMA components was preferred so that demands were resisted primarily through axial force (i.e. prevent eccentricity relative to the SMA force resultant).

2) Rectangular HSS are commonly found in the US market. Prior design iterations considered built-up sections, which would have been costlier than HSS sections.

The tubes required access openings in specific locations, as shown in Figure 4.7a and Figure 4.7c, for installation and pre-tensioning, as well as slots for the pin elements that transferred forces between the SMA and the steel. To limit localized deformations in the slots (Figure 4.8b) doubler plates were used.

The capacity design dictated that the tubes should accommodate SMA forces up to an 8% SMA strain. The braces were designed to hit a “hard stop” and prevent SMA strains beyond 8%, to prevent the SMA from reaching the full martensitic phase, where large increases in forces are possible with relatively small additional strain increases. The full martensite phase transformation is not desirable due to 1) initial testing showing that rupture could occur shortly after 10.5-11% strain, and 2) potential yielding of the steel elements or failure of the connections due to overstress.
The final retrofit design used 8x8x3/8” (small tube) and 9x9x3/8” (large tube) sections. The design checks for the tubes followed the steel design provisions of AISC 360-10 (AISC Committee, 2010) and considered tension member checks (Chapter D AISC 360-10 provisions), compression member checks (Chapter E AISC 360-10), slenderness checks for sections with access holes, and tolerance checks. Full AISC 360-10 calculations are shown in Appendix B.

4.2.2.3. Steel Tube End Plates

The tube elements were partitioned for:

1) Accommodating small tolerances – rather than try to accommodate the tight tolerances at each end of the brace (at the gusset plates), partitioning the brace into three separate elements along its length allowed for any deviations in tolerances to be accommodated by multiple bolt openings, rather than the one large bolt at the gusset plate.

2) Ease of installation. For lab testing purposes, ease of construction was an important consideration. Excluding the concrete drilling and the gusset plate site welds, the entire installation and (post-test) removal of the braces was done by a team of 3 students using a manual forklift, heavy duty towing straps, and a set of open-ended and socket wrenches. For practical applications where construction budget is a concern, this is obviously an advantage since retrofit works may be done by small teams using minimal equipment.

3) Limiting the main section’s length to ease the pre-tension procedure, as described in Section 4.2.2.1 (SMA Attachment Piece).
The end plates design was controlled by the need for a bearing area for pre-tensioning and enough area to accommodate tension bolts. Design checks per AISC 360-10 provisions included compressive checks, tensile weld checks, crippling of HSS sidewalls, and yield limit states. Full design calculations are shown in Appendix B – End plate connection. End plates are shown below in Figure 4.8c.

Figure 4.8 – Steel tubes, doubler plates, access opening, and end plate, during (a) and after (b) (c) SMA pre-tensioning
4.2.2.4. Steel Adapter Pieces, Gusset Plates, Concrete Anchors

The connection of the brace to the existing concrete structure consisted of several subcomponents or elements, each one requiring specific design considerations:

1) Gusset plates and bolts: For the braces to act as a true pin, the gusset plate had only one bolt in the connection. The large bolt diameter needed to resist all demands required a custom fabricated bolt. The effect of small tolerances, while not a controlling factor in the gusset plate design, did influence design decisions elsewhere, as described in Section 4.2.2.2. The gusset plate is shown in Figure 4.9a (without bolt) and Figure 4.9b (with bolt).

2) Steel ‘adapter’ elements: A robust force transfer between existing RC and the brace retrofit was required for testing purposes. There was additional conservatism in the design of the transfer pieces to prevent any form of brittle failure in the interface with the RC structure. However, for practical design purposes, smaller and shorter sections may be used as the steel adapter transfer. The adapter elements are shown in Figure 4.9b.

3) Concrete anchors were designed per ACI 318-11 (ACI, 2011) recommendations, using Simpson Strong Tie adhesive anchors. The full design procedure is shown in Appendix C, as output of Simpson Strong Tie’s Anchor Designer Software (https://www2.strongtie.com/software/anchordesigner.html).
Figure 4.9 – Steel Adapter Pieces, Gusset Plates, Concrete Anchors
4.3. Concluding Remarks

The design and installation process of an SMA retrofit device was described. Initial testing of SMA components was done to understand the material behavior and to sort out any manufacturing, delivery, and installation issues before design and assembly of the full-scale test specimen. The individual SMA tests showed that the SMA wires and rod broke at the 11.20% and 7.80% strains, respectively. These results indicate that both SMA wires and rods can be successfully applied into braces without breaking as long as the design appropriately limits the peak SMA strain demand to the permissible strain range (9.00% for SMA wires and 6.00% for SMA rods).

The design basis and checks of all steel components for the full retrofit device were also summarized (and respective appendices for full calculations were listed). In general, a conservative design approach was followed for the steel elements to ensure elastic response of steel elements. However, during pre-tensioning it was noticed that the pins that connect the steel attachment piece to the SMA wires was not designed with sufficient stiffness. Thus, the pins bent, which ends up reducing the initial stiffness of the SMA braces (further details in Chapter 5). Accordingly, a deflection check for the pins in the braces should be performed during a brace design (further details in Chapter 7).

This preliminary design experience, along with the results shown in Chapter 5, helped inform a detailed retrofit design procedure. This procedure is discussed in Chapter 7.
4.4. References


American Society of Civil Engineers (ASCE) Committee. (2007). *Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06)*. American Society of Civil Engineers. Reston, VA: ASCE.

CHAPTER 5. Dynamic Shaker Tests of a Shape Memory Alloy Braced Frame

In this chapter, test plan details and results are presented for dynamic RC frame shaker tests and individual brace tests. Fabrication and installation details are described in Section 5.1. Test plans and results are shown in Sections 5.2 and 5.3, respectively, for all dynamic shaker-induced forced vibration tests on the full-scale retrofitted RC frame. Individual testing of each brace (quasi-static and strain-rate) follows in Section 5.4. In addition to presenting the test program, results are shown and discussed for all tests.

The testing described herein is based on retrofitting an existing test structure that was designed as part of a companion study (Wright, 2015). All design, fabrication, installation and construction details that follow in this chapter are specific to the SMA brace retrofit. The reader is referred to Chapter 4 in the companion study for construction details of the RC frame.

5.1. Brace Fabrication & Installation

Congruent with typical practical construction applications, the brace fabrication was driven by design parameters as well as availability of, and budgeting for, materials and elements.

5.1.1. Materials

A variety of materials were used for the different elements that comprised the full retrofit. They are listed below in Table 5.1.
<table>
<thead>
<tr>
<th>Element</th>
<th>Material</th>
<th>Notes / Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMA wire</td>
<td>NiTi - ASTM F2063</td>
<td>Memry Alloy 55.8% weight Ni*</td>
</tr>
<tr>
<td>SMA rod</td>
<td>NiTi – ASTM F2063</td>
<td>Memry Alloy 55.8% weight Ni*</td>
</tr>
<tr>
<td>Steel tubes</td>
<td>ASTM A500 Grade B</td>
<td></td>
</tr>
<tr>
<td>Steel attachment piece</td>
<td>ASTM A572 Grade 50</td>
<td></td>
</tr>
<tr>
<td>SMA-steel transfer pin</td>
<td>ASTM A490</td>
<td></td>
</tr>
<tr>
<td>SMA rod nut</td>
<td>ASTM A563 Grade D</td>
<td>High Hardness</td>
</tr>
<tr>
<td>Brace end plate</td>
<td>ASTM A572 Grade 50</td>
<td></td>
</tr>
<tr>
<td>End Plate bolts</td>
<td>ASTM A490</td>
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</tr>
<tr>
<td>Gusset plate</td>
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</tr>
<tr>
<td>Gusset plate bolt / nut</td>
<td>ASTM A354 Grade BD</td>
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</tr>
<tr>
<td>Brace knife plates</td>
<td>ASTM A572 Grade 50</td>
<td></td>
</tr>
<tr>
<td>Steel adapters</td>
<td>ASTM A992</td>
<td></td>
</tr>
<tr>
<td>Anchor bolts</td>
<td>ASTM F1554 Grade 36</td>
<td>Used with Simpson Strong-Tie AT-XP high-strength acrylic adhesive**</td>
</tr>
<tr>
<td>Large shims plates</td>
<td>ASTM A572 Grade 50</td>
<td></td>
</tr>
<tr>
<td>Small shims</td>
<td>300 Series Stainless</td>
<td></td>
</tr>
</tbody>
</table>

*More info on Memry alloys: https://www.memry.com/

**Simpson Strong-Tie adhesive:
5.1.2. SMA Device Design Philosophy Background

SMAs have significantly higher costs than conventional construction materials such as steel (further details in Chapter 2). To increase cost-effectiveness when using SMAs for enhanced seismic performance, the amount of SMA material must be reduced relative to other materials in the lateral load resisting system. For metal alloys with similar uniaxial material constitutive curves in compression and tension, such as SMAs or structural steel, it is most efficient to load the material in tension since buckling reduces the net compressive capacity of a section relative to the tensile capacity. In conventional steel bracing, braces are configured such that there is always a brace element in tension, regardless of the direction of building sway. This loading configuration is commonly achieved by using, for example, X-bracing or Chevron bracing (Figure 5.1), where one brace works in tension when the building sways in one direction, and the opposite brace works in tension when sway is reversed. In these types of bracing systems, buckling propagates through the braces loaded in compression (as shown in Figure 5.2), leaving the brace in tension to complete the lateral load resisting path between stories. These configurations, of course, require pairs of braces so that there is always at least one brace resisting lateral loads through tension for each building sway direction.

Figure 5.1 – Schematic of braced frames, chevron (left) and X-bracing (right)
Buckling-restrained braces (BRBs) circumvent the need to configure pairs of braces since they have stable hysteresis in both tension and compression. A single brace can resist loads in either sway direction. BRBs don’t always result in materials saving, since the brace section could require as much, or more material than a pair of conventional braces. However, BRBs allow for much faster erection times which lead to cost savings during construction (reduced contractor erection labor). In addition, seismic damage in BRBs is concentrated in a relatively small area (the brace’s yielding core), which may result in reduced repair and replacement costs after a seismic event compared to conventional steel bracing.

In the present study, constructability and replacement advantages formed the design philosophy basis for the SMA brace device. The device developed in this project uses a configuration of steel tubes and load transfer pins that allows the SMA material to always remain in tension, regardless of the building sway direction, thereby reducing the amount of SMA material needed and avoiding the need to brace each direction of building sway individually. Similar to BRBs, one brace can resist load in both sway directions, allowing for faster erection times; and the damage is concentrated in only the
SMA portion of the brace. SMAs can be restored back to their original shape through heat (shape-memory effect), thus, replacements are avoided and repair costs are reduced.

5.1.3. Retrofit Device Description

The brace device allows both overall extension (tension) and contraction (compression) while subjecting the NiTi SMA component to only tension (see Figure 5.5). As shown in Figure 5.3, the SMA brace devices consisted of an SMA component (rods or wires) that connected to a steel load transfer component (herein named steel transfer piece). These components were housed inside two steel hollow structural sections (HSS). For each brace, two square HSS tubes were used. An 8x8x3/8 was placed inside a 9x9x3/8, as shown in Figure 5.15. Slots and holes were cut on different sides of the tubes for access, assembly, or load path purposes. On the top portion of the tubes, an 8” x 6.25” rectangular hole was cut so that a worker could access the tubes and place the SMA component inside. The SMA component (rod or wires) was attached to a steel transfer piece. The transfer pieces for the SMA wires are shown in Figure 5.3 (there are 4 transfer pieces in the photo).

The wires (gage length 6 ft) were shipped to the testing lab in a 7 ft long box and were already wound by the manufacturer around 1”-diameter wood dowels (Figure 5.3a). They were held in place with a one-by-four board, as shown in Figure 5.3c (top wires). The assembly process began with transferring the wires from the wood dowels to a 1”-diameter steel pin (Figure 5.3b, bottom pin) that connected the wires to the steel transfer pieces. For reference, in Figure 5.3c, the bottom wires have already been connected to the steel transfer piece (the holes cannot be seen because they were covered with black tape to prevent the pin from sliding out). These steel transfer pieces had a 2nd pair of holes to
accommodate a 2nd “pin” (Figure 5.3b, top threaded bolt) that would transfer the load from these pieces to the steel tubes. Thus, the full load path from the RC frame to the SMA component consisted of the following:

- Load was transferred from the RC frame to the steel “L” adapters via post-installed adhesive anchored threaded rods (Figure 5.7)
- A gusset plate was welded to the steel adapters. This plate connected to the braces through a single 2”-diameter bolt and transferred load from the “L” adapters to the brace tubes through the bolt, knife plates, and brace end plates (Figure 5.15)
- The braces transferred load to the SMA via the steel transfer pieces, as described in the previous paragraph.
Figure 5.3 – (a) SMA wire end connection close-up, (b), steel pins for steel transfer piece connections (c) and SMA wires connected to steel transfer pieces. Note that in (c) the bottom wires had already been connected via the 1”-diameter pin (the holes for the pin cannot be seen because they were covered with black tape to prevent the pin from sliding out. The top wires had not been connected yet. These wires are shown here still in the configuration the manufacturer used for shipment. They were wound around the wood dowel shown in (a) and held in place using a one-by-four board.
The braces with SMA rods (instead of wires) followed the same load path but instead of requiring a pin to connect to the steel transfer pieces, the steel transfer pieces for the rods had a 1” thick plate where the rod was directly connected using a threaded nut. A schematic of both the wire and rod to steel transfer connection is shown in Figure 5.4. The rod center portion had a diameter of 0.851’’ while the threaded ends had a diameter of 1’’. The ratio of central to end diameter equal to 0.851.

A schematic showing the mechanism that keeps the SMA component in tension during both overall extension and contraction of the brace is shown in Figure 5.5. The devices were designed and assembled such that the SMA component remained in tension while the HSS tubes were pulled apart (steel in tension) or pushed together (steel in compression) to allow for the use of very slender SMA sections.

![Schematic of the brace interior](image)

**Figure 5.4** – Schematic section cut of the brace interior (viewed from the top). The red element represents the SMA component, either wires (top) or rod (bottom). Wires are connected to the steel transfer piece through a circular pin. Rods are connected to a nut that bears on a 1” thick plate.
Figure 5.5 – Schematic of SMA wire extension for both brace extension and brace compression
To ensure a state of permanent tension in the SMA (i.e. prevent any slack due to residual deformation), a pre-strain was applied before the device was installed in the structure (or in this case, the test rig). The pre-straining procedure was performed using Dywidag bars (Dywidag, 2019) and a hydraulic hollow cylinder piston (Figure 5.15e). Slots were cut on the HSS tube sides to insert the high-strength pin shown in Figure 5.3b. These pins served as the load transfer between the SMA component and the steel tubes. The slots were cut to a 4” length to allow relative displacement of the tubes. A total of three tubes were tested, two with SMA wires that were used to retrofit the RC test frame, and one with a SMA rod. The only difference between the steel components of this brace and the ones with SMA wires are the steel attachment pieces and the locations of the slots and access holes, to accommodate the different lengths of the SMA rod vs wires.

The loading configuration is shown in Figure 5.25. An 85-kip hydraulic actuator mounted on a strong wall was used to load the braces. The opposite brace end was bolted to an A992 W14x145 steel section with an A500 HSS 12x4x1/2 diagonal strut. These were anchored to the strong floor to provide a very stiff (fixed) support condition. A steel pedestal provided a vertical reaction for the brace on the actuator side since only the longitudinal brace force was of interest.

5.1.4. Fabrication and Installation Sequence

The first and presumably most critical step in assembling a bracing retrofit for a non-ductile RC frame is determining the anchoring locations. Due to the lack of transverse beam-column joint reinforcement and inadequate column shear reinforcement, the braces were aligned such that the working point was concentric with the centerline of
the beam-column intersection, as shown in Figure 5.6. This arrangement helped to limit shear forces in the RC elements, which was intentional since shear response was shown to be brittle during the tests of Frame 1 (Wright, 2015).

Figure 5.6 – Frame schematic showing the brace working points

TEC Services (https://tecservices.com/) was hired to locate rebar in the existing RC frame. Once the rebar was located, holes were drilled at each location where an anchor would be installed using Simpson Strong Tie chemical adhesives, as shown in Figure 5.7.
Figure 5.7 – a) Holes for anchor placement, b) placing adhesive and retrofit bolt anchor, c) finished anchors in one RC frame column, and d) installed steel adapters

While preparatory RC frame work was ongoing, a local steel detailer handled most steel cutting, drilling, and welding work offsite and Nitinol Technology, Inc. was preparing all the SMA components. Sample images of the steel tubes during machine shop work are shown in Figure 5.8.
5.2. Dynamic Shaker Tests – Test Program

The retrofitted RC frame experimental testing program involved a sequence of tests with a hydraulic linear inertial shaker (LIS, shown in Figure 5.9a), capable of imparting 80 kips of lateral load (beyond-design level loads), or a smaller eccentric mass shaker (Figure 5.9b), used to find resonant frequencies. The full RC frame tests were split into 2 sets of tests, hereby referred to as Frame 4a and Frame 4b tests (the number ‘4’ is used due to this being the 4th frame tested in a series of nominally identical as-built test frames). The remainder of Section 5.2 describes the experimental test setup for Frames 4a and 4b. Test results are described in Section 5.3.

Figure 5.8 – Steel tubes with welded end and knife plates (left) and steel shapes before detailing (right)
Following this test sequence, the SMA brace devices used as the main retrofit components were tested individually. This series of tests is described separately in Section 5.4.

![Figure 5.9 – a) Linear inertial shaker (LIS), b) eccentric mass shaker](image)

**Figure 5.9** – a) Linear inertial shaker (LIS), b) eccentric mass shaker

### 5.2.1. Test Specimen

A schematic of the final configuration of the fully loaded and retrofitted RC test frame is shown in Figure 5.10a. Figure 5.10b shows an image of the 1st story with the full test setup.
Additional gravity loads in excess of the structural specimen self-weight were required to reach realistic service conditions in the beams and columns. The loads were applied to the structure using lengths of railroad rail that had been cut into four, eight and twelve-foot lengths weighing approximately 172, 345, 525 pounds respectively. To avoid accidental enhancement of the flexural rigidity of the beams and slabs through composite action, the rails were placed on the slab surfaces orthogonally to the in-plane direction. The rails were elevated from the slab surface by placing them on 2x4 pressure treated lumber that was glued with construction adhesive to the slabs. Raising the 12-ft. lengths of rail in this fashion was required to prevent them from dragging against the slab surfaces of the adjacent frames. Figure 5.10a shows the arrangement of the railroad rails. The linear inertial shaker – weighing roughly 28 kips – also constituted a large portion of the supplemental dead loads.

During the testing of Frame 1 (as-built frame) it was noted that due to the tributary area mismatch between the columns in the prototype frame and the test
specimen, additional loads were also required in the columns to achieve axial load levels representative of the service load conditions. However, the arrangement of the railroad rails to achieve those loads in the test frame was not possible due to the location and arrangement of the tension-only braces used for experimental collapse prevention. The support for these safety braces interfered with the location where additional dead loads were meant to be placed around the columns. In addition, there was interference with instrumentation near the beam-column joints. The LIS also created limitations on the ability to distribute the rails on the roof while ensuring safety of the sensitive testing equipment. Accordingly, the correct simulation of gravity loads in the as-built frame was not achieved. The axial loads in the exterior test columns were roughly 50% of those from the prototype, while the axial loads in the interior columns were closer to 75% of those from the prototype.

Since this brace design for Frame 4 was done using the prototype loads per Wright’s (2015) original design, additional rails were added relative to those in Frame 1 so that the full gravity load of the prototype design was achieved. However, due to the limitations noted above, the distribution of these loads was not fully symmetrical with respect to the center columns as intended in the original prototype. As discussed in detail in Section 5.3.1, these additional loads make it more difficult to directly compare Frame 4a or 4b to Frame 1 (the as-built test specimen), as the added load changes both the inertial characteristics of the frame as well as the stress levels in the columns prior to retrofitting. However, the decision was taken to use additional loads since the global performance of the brace was deemed as more important than a direct comparison of Frame 4 vs Frame 1 RC elements.
The lateral loads were applied to the structure using two mobile shakers, shown in Figure 5.9, from the George E. Brown Network for Earthquake Engineering Simulation’s (NEES) University of California, Los Angeles (UCLA) equipment site (nees@UCLA). The full details about these shakers can be found in the companion study (Wright, 2015).

5.2.2. Sensor Locations

The test structure was instrumented with 96 metal foil strain gages, 4 large stroke (50 inch) string potentiometers (string pots), 47 linear variable differential transformers (LVDTs) and 42 wired accelerometers. The instrumentation was distributed throughout the structure to gain insight into the global structural response as well as the behavior of local elements. Figure 5.11a shows the location of LVDTs and string pots. Accelerometers are shown in Figure 5.11b and strain gauges in Figure 5.11c.

5.2.2.1. Displacement and Strain Sensors

The drifts of the 2nd story and roof were monitored using Celesco PTIA-50 string potentiometers. Two additional string potentiometers were used to measure relative displacements of the brace tubes during each test. The string pot layout is shown in Figure 5.11a.

To monitor local damage around connections of frame members (i.e. column to foundation, beam-column joints), rotations at the end of each member relative to adjacent frame members were measured using several arrangements of LVDTs. The arrangements depended on the location of the element being measured, as shown in Figure 5.11c. For details about the specific LVDT instruments used, the reader is referred to Wright (2015).
5.2.2.2. **Accelerometers**

A total of 42 high-resolution accelerometers, representing a total of 60 acceleration channels (9 triaxial accelerometers) were used to measure the vibrational response of the test frame. The same configuration used in Frame 1 was used for all Frame 4 tests. Thus, the reader is referred to Wright (2015) for the technical details of the accelerometers. The layout is shown in Figure 5.11b.
See the sensor sheet for exact locations of accelerometers.
Figure 5.11 – a) String potentiometer and LVDT locations, b) accelerometer locations, and c) strain gauge locations

5.2.3. Loading Sequences

One of the primary objectives of this overall research project (i.e. the present thesis plus all companion studies) was to compare the strength and stiffness deterioration of retrofitted frames (Frames 2-4) against the as-built frame (Frame 1) under a similar set of input demands. Thus, the testing sequence for Frames 4a and 4b was very similar to
that of Frame 1. Since initial analysis for Frame 1 had suggested that a brittle collapse mechanism could form suddenly, input loading histories were gradually increased in magnitude in a series of tests to generate successively larger drift demands. This sequence generated multiple test results, which allowed:

1) Visual damage assessments after each test

2) Damage assessments through estimation of shifts in natural structural frequencies using the eccentric mass shaker

3) Multiple data sets to evaluate damage sequences (this would not be possible if only one test was performed which immediately led to brittle failure)

To facilitate comparison with Frame 1, a similar test sequence was followed. Frame 4a was subjected to LIS tests, using an El Centro displacement input, and EMS tests, using a sine wave input. The El Centro displacement input waveform, scaled to 1”, is shown in Figure 5.12. The EMS tests were either a sine sweep or a dwell, used to find resonant structural frequencies. During a sweep, the EMS would input a sine wave from 0 Hz to 7 Hz and back to 0 Hz over a 140 second span. The input frequency would change at 0.1 Hz/second (i.e. 70 seconds up to 7 Hz and another 70 seconds back to 0 Hz). Approximate structural mode frequencies would be estimated by calculating the single-sided amplitude spectrum of the in-plane roof acceleration. Afterwards, a sine dwell test would be performed, where the EMS input frequency was manually adjusted at frequencies near the 1\textsuperscript{st} and 2\textsuperscript{nd} mode estimates from the sine sweep. Roof acceleration would be monitored while the EMS input dwelled at particular frequencies during steady state response. The input frequencies with the maximum roof amplitude response were deemed as natural frequencies of the test frame.
Frame 1 was tested with a series of LIS El Centro inputs, followed by LIS sinewave inputs (either single or double sinewave pulses). The pulse inputs could impart a larger drift response on the frame, thus resulting in increased damage. Frame 4a was only tested with 7 El Centro LIS inputs, up to an El Centro input scaled to a 12” amplitude, plus 22 EMS since sweeps or dwells, but no LIS pulses. Due to difference in stiffness between the 1st and 2nd story (the 2nd story did not have a brace retrofit and was thus more flexible), there was concern that pulse inputs would result in a soft-story mechanism on the 2nd story. Such a mechanism would result in structural damage that would prevent proper testing of the brace retrofit. Thus, it was decided to use the sway prevention cables as stiffeners on the 2nd story, as shown in Figure 5.13, for additional testing. This series of tests (Frame 4b) included 9 El Centro LIS tests, 6 sinusoidal wave (pulse) LIS tests, and 30 EMS sine sweeps or dwells. The test sequences for both Frames 4a and 4b are shown in Table 5.2.
Figure 5.13 – Frame 4b schematic, showing 2nd story sway prevention cables used for stiffening
Table 5.2 – Loading schemes for Frame 4a and Frame 4b tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Shaker Vibration Type</th>
<th>1\textsuperscript{st} story SMA Brace (Frame 4a)</th>
<th>1\textsuperscript{st} story SMA Brace 2\textsuperscript{nd} story cable stiffened (Frame 4b)</th>
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<td></td>
<td>Peak shaker absolute acceleration</td>
<td>Peak shaker absolute acceleration</td>
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<td>El Centro scaled to 1”</td>
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<td>El Centro scaled to 10”</td>
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</tr>
<tr>
<td>7</td>
<td>El Centro scaled to 12”</td>
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</tr>
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<td>9</td>
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5.2.4. Design Specifications and Approaches

The following sections describe the specifications and design basis used for the prototype RC structure and steel elements of the brace retrofit.

5.2.4.1. Specifications for non-ductile RC frames

The prototype RC frame was representative of low-rise RC concrete office buildings in the Central and Eastern US build in the 1950s-1970s. Therefore, the structure was designed only for gravity loads, which is consistent with design practices of that time. Several detailing deficiencies were purposely included in the design, namely 1) short lap splices in columns, 2) insufficient beam bottom rebar anchor lengths in beam-column joints, and 3) inadequate shear reinforcement in beam-column joints. The provisions in Building Code Requirements for Reinforced Concrete (ACI Committee 318-63, 1963) and CRSI Design Handbook Volume II 1963 ACI Code Working Stress Design (Reese, 1965) were used in the design and proportioning of members. The reader is referred to Section 3.2 in the companion study (Wright, 2015) for the full RC frame design details.

5.2.4.2. Specifications for Steel Braces, Connections, and Adapters

The provisions in Specification for Structural Steel Buildings (ANSI/AISC-360-10) and Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-10) were used for the design and proportioning of all structural steel members that composed the brace retrofit. The design checks and corresponding provisions considered are detailed below. The full set of calculations is given in Appendix B.
5.2.4.2.1. L-shaped steel adapters

To connect the braces to the RC frame, steel W-sections were attached to the frame columns, beams, and foundation slab in an L-shaped configuration (Figure 5.14). As part of the research project, this steel configuration was named an ‘adapter’ because these steel shapes adapt the geometry of the RC frame to handle the forces and geometry of the brace. The adaptation consists of two main characteristics. First, these steel shapes collect the load demand going into the brace through a relatively large length of the beams and columns, helping reduce the likelihood of a brittle shear failure in these elements due to collector overstress. Second, the geometry created by these adapters placed the working point of the braces at the center of the beam column joint, which was desirable since the RC frame had no joint shear reinforcement.

The W-sections were connected to the RC components through post-installed anchors (Figure 5.7). The anchors were ASTM F1554 Grade 36 threaded bolts, installed using Simpson Strong-Tie AT-XP high-strength acrylic adhesive. The W-sections consisted of a W8x40 (the horizontal element that connected to the RC frame’s beams or foundation) and a W8x67 (the vertical element that connected to the RC frame’s columns). These shapes were chosen for two reasons: 1) W-shapes are widely available and thus relatively inexpensive compared to other potential steel shapes that could have functioned as adapters; and 2) these specific sizes can resist brace overstrength without nonlinear deformation. The design philosophy for this retrofit device dictated that the SMA would act as the main energy dissipation element. Residual deformation resulting from nonlinear SMA behavior is recoverable, but nonlinear deformation in the steel elements is not recoverable. Thus the steel elements were designed to remain elastic. The
length of the steel element was dictated by the allowable interface forces per bolt in the connection to the RC frame. These forces were determined using ACI 318-11 (calculations in Appendix C). The connection from the adapters to the brace was done through a gusset with a single bolt, to achieve a true pin connection. A schematic showing the location and lengths of the steel adapter pieces (for the right bay of the RC test frame) as well as the gusset plate and brace is shown in Figure 5.14. The bolt for this gusset plate connection can be seen in Figure 5.15a and Figure 5.15b.

Figure 5.14 – Location and size of steel adapter elements on the right bay of the RC test frame. For reference, a schematic of the brace is shown.
5.2.4.2.2. Steel Brace Tube Design

The steel tubes were supported in a true ‘pin’ boundary condition. Thus, they would be subjected mainly to tension and compression forces. Chapter D provisions in ANSI/AISC 341-10 were used to satisfy tensile capacity requirements. Provisions from Chapter E were considered for compression design. Section E6, which takes into account slenderness of built-up sections, was considered in the design. The reason was uncertainty about being able to procure a satisfactory hollow steel section (HSS). Thus, built up tubes were also considered. Provisions from Chapters J and K were considered in the design of all connections. The bracing tubes had several connections to accommodate 1) the transfer of forces from the SMA components to the tubes, tube-to-tube connections (for constructability purposes), and tube to end plate (as shown in Figure 5.15b).
Figure 5.15 – Steel components: a) steel adapters, gusset plate and bolt; b) close-up view of brace end section connected to gusset plate; c) top and bottom brace end sections, west bay; d) brace main section, showing access holes, doubler plate, and transfer pin; e) brace main section during pre-straining of SMA wires; and f) fully assembled and connected brace retrofit
5.2.4.2.3. Steel Tube Pin Connection – End, Knife, and Gusset Plates

To achieve a true pin support condition for the brace, the end connection relied on a single ‘pin’, which consisted of a large diameter bolt that connected two knife plates to the gusset plate attached to the steel adapters, as seen in Figure 5.15a. The design of the bolt followed the provisions of ASTM A354 Grade BD. Due to the required large diameter, the bolt was custom ordered at Atlanta Rod and Manufacturing (http://www.atlrod.com/), a specialty fastener manufacturer. Future research should consider a support condition that does not rely on a true pin connection to avoid the higher costs of custom-made bolts and to limit any potential risk of constructability issues.

All plate connections were fillet welded. Weld design followed the provisions of American Welding Society AWS D1.1 Structural Welding – Steel (AWS, 2000) and Specification for Structural Steel Buildings (AISC Committee, 2010). Sample weld details are shown in Figure 5.16. All steel components were shop drilled and welded (and later assembled in field), with the exception of the gusset plate welds. Due to availability of a welder in the testing lab, the gusset plate to adapter connection was field-welded. Because of the connection’s geometry, the welding had to follow after connection to the RC frame. This was not an issue thanks to the availability of a welder on-site. However, if site welding is to be avoided for practical construction scenarios, the gusset plate to adapter connection may be designed using bolts instead of welds.
The steel adapter pieces were attached to the RC frame using post-installed anchor rods. Ground penetrating radar was used to locate the rebar, and anchors were installed using adhesive anchors, following the provisions from ACI-318-10, Appendix D (Appendix D of ACI-318-10 (ACI Committee, 1963)). The post-installed anchors presented the most challenging constructability issue for this retrofit scheme. The relatively shallow depth of the RC sections limited the allowed embedment length, which required a larger number of anchors. Uniform anchor placement becomes more difficult with an increased number of anchors or increasing rebar density. Even with the relatively light reinforcement present in these RC elements, the difficulty in anchor placement dictated the drilling of holes in the steel adapters. The final drilled patterns were non-uniform. It is recognized that denser reinforcement cages in equally shallow concrete sections could render this retrofit scheme infeasible in its current configuration.
Additional research is needed to determine the feasibility of alternative anchoring methods.

5.2.4.3. Preliminary Analysis Results of Base-excited SMA Frames for Design of Shaker-excited SMA Frames

The design of the prototype Frame 1 was part of a companion research (Wright, 2015). As the period required for SMA order, fabrication, and preliminary testing for approval would reach approximately 6 months, the design of the SMA retrofit had to be based on analysis results assuming base-excitation rather than the actual linear shaker roof-excitation of Frame 1. Target accelerations for the structure were established based on criteria of geographical locations within the CSUS where 1970s construction of a building with the structural characteristics and deficiencies of the prototype structure (as discussed in Chapters 1-3) could be reasonably expected. Memphis, TN was chosen based on urban density (i.e. a higher likelihood of commercial buildings similar to the prototype structure) and a relatively high seismic hazard, compared to other CSUS locations, based on ASCE 7-10 hazard maps (ASCE, 2010). The test results for Frame 1 (Wright, 2015) indicated that the maximum expected base acceleration would be approximately 1.2g (Pulse 8’’), which confirmed the assumed base-excitation for the SMA retrofit.

The main drivers behind the configuration of the bracing system were the required SMA area and stroke length. To determine the area, a linear static procedure (LSP) analysis was done, using the parameters shown previously in Section 4.2.1, as taken from ASCE 41-06 (ASCE, 2007) provisions. For design purposes, it was assumed that the
brace would resist the entirety of the lateral load demand. Full LSP details are found in APPENDIX A.

After establishing the required SMA area based on LSP analysis, a preliminary model was built in OpenSees (McKenna et al., 2009). The model used the parameters proposed by Speicher et al. (2009) to characterize the SMA material behavior and elastic response was assumed in all steel retrofit elements. Using the required SMA area, a suite of 240 ground motions characteristic of the CSUS region (Fernandez & Rix, 2006) was used to perform fragility analysis. The goal of this analysis was two-fold:

1) to confirm that the maximum forces generated in the brace were similar to those determined using LSP, and  
2) to determine the stroke length needed to limit inter-story drifts to values that would satisfy the required system target reliability.

Full suite fragility analyses were run for SMA stroke lengths of 4, 5, and 6 ft. It was agreed with the research team that a target system reliability, $\beta = 2.5$, was congruent with current design practices. Under this assumption, a relative story displacement beyond 1.3 in would result in a ‘Complete’ damage state (FEMA, 2003).

Following this analysis, a SMA stroke length of 6 ft was chosen for the test specimen. This choice was based on 6 ft being short enough to limit max story displacements to less than 1.3 in (to meet the target reliability of the ‘Complete’ damage state) but long enough that the SMA strains would not lead to an austenite-martensite phase transformation, which could result in forces significantly larger than the design level forces, potentially leading to brittle brace connection failures.
5.2.4.4. SMA Brace Configuration in Experimental Frame

During design, due to constructability and schedule considerations, the addition of braces was only considered for the 1st story. The set of reasons for not implementing the brace retrofit in the 2nd story was the following:

1) The as-built prototype structure was designed with more ductile 2nd story columns in order to prevent brittle failure of that story and drive the forces from the roof mounted shaker all the way to the foundation. Thus, the 2nd story could withstand higher drifts in the as-built condition. In addition, if additional stiffness was needed, the auxiliary sway prevention cables could be used to stiffen the 2nd story. Therefore, it was not deemed critical to retrofit the 2nd story with the full bracing system just for the purposes of testing brace performance.

2) The assembly and construction of braces in the 2nd story would require additional construction equipment, time and/or labor. Availability of additional labor that was allowed to operate construction equipment was sparse and additional time was not feasible due to the testing schedule with NEES/UCLA.

3) In a real seismic excitation scenario, i.e. base excitation instead of roof excitation, a structure with a soft-story at ground level might primarily (or only) require retrofitting of said soft-story. The design of the experimental structure purposely included higher stiffness and ductility in the 2nd story.

5.2.5. Constructability Observations

After finalizing the assembly and testing process, several observations were made about the constructability of the bracing retrofit. The observations are summarized below,
starting the advantages that this configuration can offer over more conventional retrofit schemes and following with improvements that could make the installation/construction process even easier in the future.

The biggest benefit, from a constructability point, is that this brace configuration allows for an installation process that uses no heavy machinery in the field – assuming low-rise construction. If the retrofit is used for mid- or high-rise retrofitting, cranes would facilitate the job, but then cranes would likely be required in such scenarios regardless of the retrofit scheme. Other than site welding of the gusset plate, the entire assembly process was completed by a team of 3 people using hammer drills, a manual lift, and a variety of wrenches. The SMA pre-straining was done on site for convenience since the entire assembly was done on site, but this step can easily be performed off-site and the brace can be shipped already assembled. A different steel adapter and/or gusset plate configuration could prevent the need for site welding if lack of welders is a concern. Thus, for buildings where disruptions to operations could cause financial concerns (e.g. commercial office space) this retrofit could provide an attractive option compared to more intrusive options.

There are two main improvements that should be followed on future iterations of this device, both regarding the transfer pin used on the configuration that had SMA wires (Figure 5.3b and Figure 5.17). The transfer of the SMA strands from their shipment configuration to the steel attachment pieces was, by far, the most difficult part of the assembly. In addition, the stiffness of the pin was inadequate for the forces imparted by the SMA wires. Thus, it is suggested that future iterations of this device use a pin where the cross-sectional shape is a ‘stadium’ (i.e. a rectangle with circular ends, as shown in
Figure 5.17) instead of a circular cross-section. These pins should be manufactured before the SMA wires are machined. The pins must be sent to the machinist, who should wind the wires around the pin for shipment. Although it incurs additional shipping costs, this process will prevent potentially lengthy delays when transferring the wires to the steel attachment piece. To minimize pre-strain loss due to pin deformation, a 1” x 3” stadium shape is recommended for a 0.6 in² SMA wire cross-sectional area. Further details are given in Section 5.3.5.1.

Figure 5.17 – SMA wire transfer pin geometry (left, cylinder) and suggested shape for future designs (stadium, right)

5.3. Dynamic Shaker Tests – Test Results

The previous section covered details about dynamic testing for Frames 4a and 4b, which were subjected to 7 and 15 LIS tests, respectively. In this section, the structural
response of the test frame to the applied forced vibrations is discussed. A summary of the system level response of the RC frame and the brace retrofit is given in Section 5.3.1, for both the frame with an as-built 2nd story (Frame 4a, Section 5.3.1.1) and the frame with an enhanced stiffness 2nd story (Frame 4b, Section 5.3.1.2). Comparisons to the structural performance of Frame 1, the as-built RC test frame from Wright’s companion study, are also shown in the respective sections.

5.3.1. Results Summary and Damage Sequence

To facilitate comparison against Frame 1, as well as general discussion of results, in the following sections the same nomenclature and sign convention adopted by Wright (2015) will be used. The reader may refer to Wright’s study for full details, but the main nomenclature and sign convention key points are repeated below, for the reader’s convenience.

The nomenclature referenced for the beams and columns is outlined in Figure 5.18. The beam-column joints are referenced as a combination of the column line (E, C, or W) and level (2 or R) where they are located.

Figure 5.18 – Schematic of the nomenclature for (a) beam and column members, (b) column hinge zones, and (c) beam hinge zones
The sign conventions used when discussing instrumentation data are summarized in Figure 5.19. Positive drifts are defined as those corresponding to eastward displacements (i.e. X+ corresponds to the east direction). Section rotations are considered positive about the global Z-axis. Accordingly, under typical frame action, positive inter-story drifts generate negative rotations in all column ends and positive rotations in all beam ends. Tensile strains in rebar are taken as positive and compressive strain as negative.

![Figure 5.19 – Sign convention used for strain and displacement data](image)

5.3.1.1. **Frame 4a Performance and Comparison to As-Built Frame**

Frame 4a was subjected to a series of 7 LIS tests using El Centro input displacements with increasing peak amplitudes, ranging from 1” to 12”. After each test using the linear shaker on the roof, the periods of vibration were identified through the EMS installed on the second floor to evaluate variations in the lateral stiffness. The experimentally identified periods for Frame 4a are listed in Table 5.3. The initial periods
(frequencies) of Frame 4a for the first and second vibration modes were 0.36 s (2.80 Hz) and 0.23 s (4.34 Hz), respectively. At the end of the series of 7 LIS tests with increasing amplitudes, the periods increased slightly to 0.39 and 0.25 s, however, no visible structural damage was detected after any of the tests. In addition, the as-built frame withstood less absolute acceleration than Frames 4a and 4b (see Table 5.3) and still suffered more significant structural damage. For further details on the sustained drifts and structural damage of the as-built frame, the reader is referred to Wright (2015). Lastly, for the El Centro series of tests (the only tests that all 3 frames were subjected to) the stiffness degradation, as measured implicitly through period elongation, was more significant for Frame 1 than for Frames 4a and 4b.
Table 5.3 – Loading schemes for Frame 1, Frame 4a and Frame 4b tests

<table>
<thead>
<tr>
<th>Shaker Vibration Type</th>
<th>Frame 1</th>
<th>Frame 4a</th>
<th>Frame 4b</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak shaker abs. acc.</td>
<td>T₁</td>
<td>Peak shaker abs. acc.</td>
</tr>
<tr>
<td>El Centro scaled to 1”</td>
<td>0.0489</td>
<td>0.48</td>
<td>0.0859</td>
</tr>
<tr>
<td>El Centro scaled to 2”</td>
<td>0.0915</td>
<td>0.49</td>
<td>0.1563</td>
</tr>
<tr>
<td>El Centro scaled to 4”</td>
<td>0.1642</td>
<td>0.52</td>
<td>0.3120</td>
</tr>
<tr>
<td>El Centro scaled to 6”</td>
<td>0.2012</td>
<td>0.51</td>
<td>0.4069</td>
</tr>
<tr>
<td>El Centro scaled to 8”</td>
<td>0.2790</td>
<td>0.53</td>
<td>0.4943</td>
</tr>
<tr>
<td>El Centro scaled to 10”</td>
<td>0.2914</td>
<td>0.53</td>
<td>0.4962</td>
</tr>
<tr>
<td>El Centro scaled to 12”</td>
<td>-</td>
<td>-</td>
<td>0.5002</td>
</tr>
<tr>
<td>Pulse 4”</td>
<td>0.4331</td>
<td>0.54</td>
<td>-</td>
</tr>
<tr>
<td>Pulse 8”</td>
<td>1.2038</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Pulse 12”</td>
<td>1.7901</td>
<td>0.54</td>
<td>-</td>
</tr>
<tr>
<td>Double Pulse 16”</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Double Pulse 20”</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Double Pulse 26”</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Responses of the SMA braced frame and the as-built frame to shaker vibration with a variety of acceleration magnitudes ranging from 0.086 to 0.50g were investigated by analyzing the experimental data of floor displacements. The time histories of inter-story drift ratios of the as-built frame and first story SMA braced frame under shaker acceleration of 0.30g are shown in Figure 5.20. The first-story peak inter-story drift ratio for frame 4a was 0.05%, significantly less than the 0.33% of the as-built frame. The relatively small inter-story drift ratio in the first story of frame 4a, compared to the as-built frame, indicates that the SMA braces effectively suppressed the first story vibration. In addition, for other vibration magnitudes, as shown in the envelopes of the first story inter-story drift ratios in Figure 5.21, the SMA braces successfully limited the drift ratio to under 0.20%. For frame 4a, however, since there were no SMA braces in the second story, the second-story inter-story drift ratio increased as the vibration magnitude increased, leading to an obvious drift concentration in the second story (Figure 5.14). Since the LIS was installed on the roof of the two-story frame, the reduced stiffness of the 2nd story dissipated the input amplitudes, thus preventing inertial response from transferring below to the first story through the weak 2nd story columns. After the larger amplitude El Centro tests, it was evident that larger input amplitude pulses could potentially create a soft story mechanism in the 2nd story. Thus, to continue investigating the behavior of the 1st story SMA brace retrofit, it was necessary to increase the 2nd story stiffness.
5.3.1.2. Frame 4b Performance and Comparison to As-Built Frame

To transfer more excitation energy from the roof to the first story, the second story was enhanced by using the sway prevention steel cables that were originally intended only to prevent excessive inter-story. For this new configuration, hereby deemed
Frame 4b, the periods for the first and second modes were 0.42 and 0.17 s, respectively (Table 5.3). After running a 2nd series of LIS El Centro excitations up to 12” peak displacement (in the same fashion as frame 4a), the periods remained effectively unchanged. However, following the LIS single pulse vibration sequences, the estimated first mode period, found through EMS sine dwells, increased significantly from 0.77 s to 1.67 s and then to 1.43 s following the single pulse tests with peak amplitudes of 4.0, 8.0, and 12.0 in. (101.6, 203.2, and 304.8 mm), respectively. Even after this pronounced change in the system’s first period, there was no visible cracking in the columns or the beam-to-column connections. During the double pulse sequence, the first period remained essentially unchanged, at approximately 1.67 s. Note that the periods experimentally identified through the use of the eccentric shaker on the second floor include the mass of the linear shaker on the roof since the linear shaker stopped and was rigidly connected to the roof floor during the modal period identification.

Frame 4b response to LIS forced vibrations resulted in shaker input acceleration magnitudes ranging from 0.88 to 3.95g. The time histories of inter-story drift ratios for Frame 4b compared to the as-built frame under shaker acceleration of 0.29g are shown in Figure 5.22. In these tests, the first-story peak inter-story drift ratio for Frame 4b frame was 1.94%, which was smaller than the 2.6% drift for the as-built frame. The 2nd story drift was also significantly smaller, at 0.4%, vs 2.06% for the as-built frame. This reduction indicates that the 2nd story could have also benefited from using a brace retrofit. For other vibration magnitudes, Figure 5.23 shows envelopes of the inter-story drift ratios. The SMA braces successfully limited the ratio to under 1.94% in the 1st story for all shaker accelerations, while the as-built frame had drift ratios of up to 2.6%, under
smaller shaker accelerations. It is worth noting that in the companion study, Wright (2015) concluded that if not for the sway prevention cables in the 1st story, the as-built frame would have collapsed under the last 3 LIS tests, all of which resulted in 1st drifts larger than 2%.

Figure 5.22 – Inter-story drift ratios of the as-built and SMA & steel-cable retrofitted RC frames under shaker accelerations of 3.50g and 3.95g, respectively

Figure 5.23 – Envelopes of peak inter-story drift ratios of the as-built and retrofitted RC frames for shaker acceleration ranging from 0.00 to 0.60g

In order to quantify the uniformity of inter-story drifts over the structure height, the drift concentration factors (DCFs – MacRae, Kimura, & Roeder, 2004) for the
retrofitted frames were calculated. The DCF is the ratio of the peak inter-story drift ratio for any story to the roof drift ratio. For a 2-story building is defined by the following equation:

\[
\text{DCF} = \max\left\{ \frac{\Delta_1}{h_1}, \frac{\Delta_2}{h_2} \right\} \div \frac{\Delta_{\text{roof}}}{H}
\]

where \( \Delta_i \) is the displacement of the \( i^{\text{th}} \) story, \( h_i \) is the \( i^{\text{th}} \) story height, \( \Delta_{\text{roof}} \) is the roof story displacement and \( H \) is the height of the structure. If the DCF value is equal to 1.00, the structure will develop a uniform story drift distribution. For a 2-story structure, a DCF close to or equal to 2, signifies a soft-story.

Figure 5.24 compares the DCF for Frame 1 and Frame 4b for two input excitation cases:

- El Centro ground motion scaled to 8’’
- Sine Pulse scaled to 12’’

The pulse-12” excitation was chosen as the representative stronger excitation because, for Frame 1, excitations stronger than this one led to unrealistically high drifts for Frame 1 that would have caused the collapse of the structure if not for the sway prevention safety cables (see Wright, 2015). The results show that the DCF is higher for the retrofitted structure compared to the as-built structure. This is in accordance with the results observed in Figure 5.23, where the 1st story drift envelopes for Frame 1 and Frame 4b have similar magnitude but the 2nd story envelopes are smaller for Frame 4b. However, it must be noted that the DCF is associated to an overall lower roof drift (\( \psi_{\text{roof}} = \Delta_{\text{roof}} / H \)) for the retrofitted frame (0.88%) than the as-built frame (1.01%).

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Figure 5.24 – Comparison of Drift concentration factors for El Centro scaled to 8” (left) and sine pulse excitation at 12” (right) for Frame 1 (top) and Frame 4b (bottom). (Note: sidesway scaled 30x)

A test plan was devised for each individual brace to investigate the response and performance repeatability during multiple cycles at increasing strain levels, as well as different load rates. Test preparation details are shown next, followed by a summary and discussion of the results.

5.3.2. Test Specimens and Loading Scheme

The SMA brace devices consisted of an SMA component (rods or wires) housed inside two steel hollow structural sections (HSS). For each brace, two square HSS tubes were used. An 8x8x3/8 was placed inside a 9x9x3/8, as shown in Figure 5.15d. Slots and
holes were cut on different sides of the tubes for access, assembly, or load path purposes. The devices were designed and assembled such that the SMA component remained in tension while the HSS tubes were pulled apart (steel in tension) or pushed together (steel in compression). This allowed for the use of very slender SMA sections.

To achieve a state of permanent tension in the SMA (i.e. prevent any slack due to residual deformation), a pre-strain was applied before the device was installed in the structure (or in this case, the test rig). The pre-straining procedure was performed using Dywidag bars (Dywidag, 2019) and a hydraulic hollow cylinder piston (Figure 5.15e).

Slots were cut on the HSS tube sides to insert a high-strength pin. These pins served as the load transfer between the SMA component and the steel tubes. The slots were cut to a 4” length to allow relative displacement of the tubes. A total three tubes were tested, two with SMA strands that were used to retrofit the RC test frame, and one with a SMA rod. The only difference between the steel components of this brace and the ones with SMA wires are the steel attachment pieces (as shown in the schematic in Figure 5.4) and the locations of the slots and access holes, to accommodate the different lengths of the SMA rod vs wires.

The test configuration is shown in Figure 5.25. An 85-kip hydraulic actuator mounted on a strong wall was used to load the braces. The opposite brace end was bolted to an A992 W14x145 steel section with an A500 HSS 12x4x1/2 diagonal strut. These were anchored to the strong floor to provide a very stiff (fixed) support condition. Since only the longitudinal brace force was of interest (plus the actuator is double-hinged), a steel pedestal provided a vertical reaction for the brace on the actuator side.
5.3.3. Sensor Locations and Details

A total of seven sensors were used during the individual brace tests. Force and displacement readings were obtained from the load cell and LVDT in the 85-kip actuator used to load the specimens. Additionally, a string potentiometer was used to measure relative displacement between the tubes and four strain gauges were placed on the outside tube at mid-span. A layout of the instrumentation is shown in Figure 5.26.

Data from all sensors was collected using a National Instruments DAQ-mx connected to a Dell laptop computer. Sensors recorded data at 1kHz and wrote to output approximately every 0.5 seconds.
5.3.4. Loading Sequences

A variety of loading sequences were used to identify possible differences in performance due to loading rate. The main sequence shown in Figure 5.27 was used as the 1st test for all three braces. It consisted of 3 full cycles to seven deformation targets of 0.5, 1, 2, 3, 4, 5, and 5.42% over the initial pre-strain; 5.42% corresponding to a 3.9” displacement, or just prior to reaching the “hard stop” (steel-on-steel contact at brace end plates). This test was performed in a quasi-static manner, at a 2 in/min loading rate. In addition, one of the SMA wire braces was tested with the same protocol at 10, 20, and 40 in/min. This same brace was then tested using variable load rate protocols.

![Figure 5.27 – Cyclic load displacement for individual brace tests](image-url)
5.3.5. Test Results

Results for all brace tests are presented in the following sections. The results and number of tests for the braces with SMA wires differed from those of the SMA rods. Thus, they are shown in different sections.

5.3.5.1. Brace #1 (SMA wires)

The 1st SMA wire brace was only tested quasi-statically (2 in/min) using the main loading sequence (Figure 5.27). Results are shown in Figure 5.28. Overall, the brace was able to deliver good hysteretic damping capacity with almost full re-centering after each cycle. The stiffness degraded quickly once the loading started. However, the initial SMA phase transformation did not occur until the 1% strain level was exceeded. The initial change in stiffness occurred due to bending in the pins supporting the SMA wires and connecting the attachment piece to the HSS side walls (Figure 5.29). The pre-strain loss due to the deformation of the 1” diameter circular pin was calculated based on the stress-strain diagram of Figure 5.30 (processed from the preliminary SMA tests described in Chapter 4). For the calculation of the elastic pin deformation it was assumed that the 60 kip peak load acts on one third of the supported pin length \(c = 1.417 \text{ in}\) and the following equation was applied:

\[
\Delta e_t = \frac{w\cdot c}{96\cdot E\cdot I} \left( 2\cdot L^3 - L\cdot c^2 + 0.25\cdot c^3 \right) = 0.06409 \text{ in}
\]

The plastic deformation of the pin was measured in the lab and found equal to 3/16”. Since two pins were used, the total deformation is calculated as 0.503 inches. The initially applied pre-stain deformation was 2 inches and the length of each unloaded wire
was 72 inches, which leads to an initial pre-strain deformation of 2.77%. Therefore, the remaining pre-strain is \([2 \text{ in} - 0.503 \text{ in}] / 72 \text{ in} = 2.08\%\), which corresponds to a pre-strain loss of 0.69%.

To fix the pin deformation issue, a stadium shape may be used. A 1” x 3” stadium shape pin (see Figure 5.17), only increases the cross-sectional area 3.5 times (0.785 in\(^2\) and 2.785 in\(^2\) for the 1”-dia circular pin and stadium shape, respectively) but the moment of inertia increases 30-fold (0.049 in\(^4\) to 1.5 in\(^4\) for the 1”-dia circular and stadium shapes, respectively). To reach a moment of inertia of 1.5 in\(^4\) with a circular shape, the radius of the pin should be increased up to 1.175 in. Therefore, a circular shape would require 56% more material than a stadium shape (cross-sectional area of 4.34 in\(^2\) and 2.785 in\(^2\), respectively). The stadium shape deformation is assuming a peak load of 60 kips acting on the full supported pin length (4.25 in) can be calculated as follows:

\[
\Delta = \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot 60 \text{ kip} \cdot 4.25 \text{ in} \cdot 4.25^4 \text{ in}^4}{384 \cdot 29'000 \text{ksi} \cdot 1.5 \text{ in}^4} = 0.00138 \text{ in}
\]

The corresponding stress on the stadium shape would result in no plastic deformation, thus, the stadium shape would only deform 0.00138 in under the same pre-strain load, compared to the approximately \(\frac{1}{4}\)’’ maximum deformation that was observed by using the circular pin.
Figure 5.28 – SMA wire brace hysteresis

Figure 5.29 – Bending of the high-strength pin connecting SMA wires to the steel attachment piece
Figure 5.30 – Pre-strain loss due to bending of the pin that connects the SMA wires to the steel attachment piece

As seen in Figure 5.28, the SMA did not reach its forward transformation stress during the 0.5% and 1% strain cycles and the wire strands unloaded elastically. During the subsequent cycles, a well-defined change in stiffness occurred just after the 1% strain level. On the brace shortening test portion (positive quadrant in Figure 5.28), there is minimal strength or stiffness degradation on repeated cycles. However, the brace elongation part does exhibit strength degradation. Also noticeable on this negative quadrant is a kink in the curve around -3% strain where the load dropped approximately 8 kips during every cycle, and an instant 10 kip drop in load once the cycle’s peak is reached and the actuator reverses. It was noticed that this occurred due to misalignment of one of the floating steel attachment pieces. This misalignment caused the piece to rub against the sides of the inner HSS tube, which caused friction. This friction led to the
extra 10 kips of force observed during brace elongation. The misalignment also led to the piece “catching” the edge of one of the access holes, which occurred around -3% strain. In general, the brace behavior was controlled by the SMA strand behavior and the residual deformation was minimal, which satisfied the design objectives.

5.3.5.2. Brace #2 (SMA rods)

The 2nd brace (with an SMA rod) was also tested only quasi-statically (2 in/min) using the main loading sequence (Figure 5.27). The results are shown in Figure 5.31. A series of subsequent tests were planned, but the rod fractured, as seen in Figure 5.32, during brace shortening in the 3rd cycle up to 5% strain. In contrast to Brace #1, degradation of cyclic loading and unloading strength was observed (both during brace elongation, unloading degradation during brace shortening). Strain accumulation led to approximately 0.2% residual deformation and also to a loss of pre-strain evidenced by ‘slip’ at 0 and 0.2% strain.

Figure 5.31 – SMA rod brace hysteresis

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5.3.5.3. **Brace #3 (SMA wires)**

The 3\textsuperscript{rd} brace (with SMA wires) was tested quasi-statically (2 in/min) and with increasingly faster load rates (10, 20, and 40 in/min) using the main loading sequence. It was also tested with patterns of variable load rates to investigate the response to in-cycle strain rate changes, which would occur during ground motion response. During the initial quasi-static test, the first cycle showed a very flat plateau region. After this cycle, successive forward transformations were well-rounded without any flat plateau, as seen in Figure 5.33a.

During the additional tests, the effects of wire training and loading rates can be observed. With an increase in the number of cycles (i.e. further testing), the enclosed areas of the hysteretic loops shrink, due to a decrease in the forward transformation strength and an increase in the reverse transformation strength. These effects lead to a significant decrease in energy dissipation capacity, as shown in Figure 5.33b. During
faster loading rates, both the forward and backward transformation stresses are increased. This could be due to a strain rate effect, but also due to transformation heating. Given the higher speed, the wires may not be able to dissipate the heat fully, which could cause the stresses to be higher for both upper and lower plateaus.

There is an unexpected sloped at the upper end of the 40 in/min cycles (Figure 5.33a) which is due to instrument limitations. The loading rate surpassed the instrument recording rate so there are missing data points.
5.4. Testing Summary and Conclusions

In this chapter, test plan details and results were presented for dynamic RC frame shaker testing and individual brace tests. Retrofit fabrication and installation details were shown, with enough detail to reproduce all steel components of the test specimens (note that a detailed design procedure and example is given in Chapter 7). For the SMA components, in addition to the details provided in Chapter 7, this chapter provided information about the materials scientist, fabricators and machinists that collaborated on the project, in case the reader wants to follow up for information on building a similar SMA component.
Full-scale shaker tests indicated that that the SMA braces successfully suppressed the first story inter-story drift, as compared to the as-built frame. The first story returned to its original position following all shaker tests. There was no visible damage observed in the brace-to-beam/column connections, demonstrating that the steel beam/column adapters, designed according to the AISC seismic provisions for steel brace end connections and ACI 318-11 for anchoring-to-concrete, successfully transferred the brace axial forces to the reinforced concrete frame. The designed embedment lengths for seismic anchors prevented the need for drilling anchor holes completely through the columns (i.e. from one side’s surface all the way through to the other side).

Individual testing (quasi-static and variable strain-rates) of each brace was also discussed. Results showed that the number of cycles significantly affected the behavior of the SMA braces. A larger the number of cycles results in higher re-centering capacity but less energy dissipation capacity. The braces with SMA showed more cyclic repeatability, with both braces showing substantial re-centering and energy dissipation capacity after multiple tests, each with up to 21 cycles from 0.5% strain up to 5% strain (plus a 2% pre-strain). The test specimen with an SMA rod, however, showed cyclic strength degradation after multiple cycles, and the rod fractured just below a 5% strain (plus a 2% pre-strain) during the 1st test. This fracture is similar to the one seen in the preliminary specimen described in Section 4.1.2, and thus, SMA wires are recommended over rods for this retrofit device.

Constructability observations were also made, following the lessons learned during manufacturing, assembly, construction, and testing of these retrofit devices. The main beneficial observation was that the installation process required little to no heavy
construction equipment and a small crew. Installation of the main section of the brace device on site, the most complex task, required only 3 people. Most other assembly or construction tasks were done by only person. Improvements were recommended for two aspects of the design and machining/assembly process.
5.5. References


American Society of Civil Engineers (ASCE) Committee. (2007). *Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06)*. American Society of Civil Engineers. Reston, VA: ASCE.


CHAPTER 6. Refined Analysis and Seismic Assessment

Updated analytical models that were calibrated and validated against experimental results are shown in this chapter. Models were calibrated for the individual brace retrofit component, and the global Frames 1 (as-built), 4a (retrofitted with SMA braces in the 1st story) and 4b (Frame 4a with an enhanced 2nd story for experimental testing purposes). Details regarding model calibration and comparisons against relevant experimental results are shown in Section 6.1.

A seismic assessment using the updated models is shown in Section 6.2. Fragility analysis was done on an analytical model based on the calibrated Frame 1 and another based on the calibrated Frame 4a. The models represent one bay of the prototype structure, as-built, or retrofitted with the SMA brace device. For Frame 4, it is assumed that both stories are retrofitted using the full SMA brace scheme and, for practical purposes, 2 of the 5 bays in the prototype structure would be retrofitted. Thus, a leaning column is added to account for the destabilizing vertical load (from the non-retrofitted bays) that each retrofitted frame must also support. The models were also compared to fragility analysis results of the same prototype structure retrofitted under two additional schemes: 1) with a buckling-restrained brace or 2) with the same SMA brace configuration but only half of the SMA wire area. These analyses were done to compare the performance of the SMA brace retrofit against other retrofit methods and to check if cost savings could be achieved by reducing the amount of SMA material but still achieving safe performance. Details of these comparisons are shown in Section 6.3.
6.1. Updated Models – Comparison with Experimental Results

A discussion of the modelling procedure and results follows in this section. The updated models for the as-built frame (Frame 1), and retrofitted frames without (Frame 4a) and with (Frame 4b) an enhanced 2nd story are shown in Sections 6.1.1, 6.1.2, and 6.1.3, respectively.

6.1.1. As-built Frame (Frame 1)

The companion study to this project (Wright, 2015) identified the main drivers of the brittle seismic response in the as-built experimental frame (Frame 1) as:

1) The short compression lap splice in the 1st story
2) The short embedment length of bottom reinforcement bars in beams, and
3) The lack of adequate transverse reinforcement in columns.

Preliminary analytical models, as shown in Chapter 3, already accounted for the short compression lap splice and the short beam rebar embedment length explicitly, through derivation of analytical parameters found in the literature. The lack of adequate transverse column reinforcement was considered implicitly by adjusting the concrete material model to account for a lack of confinement.

The benchmark model for validation was the model described in Chapter 3. To compare that model with the experimental results, the gravity loads and corresponding masses were updated to match the loads used in the experimental as-built frame (Frame 1). Details about Frame 1 test loads can be found in Section 4.4 in the study of Wright.
Concrete strengths were also updated to match the 28-day cylinder tests listed in Wright (2015), Section 4.2.

The input acceleration for model validation was the absolute shaker acceleration as measured during testing. The analytical model was subjected to the full LIS testing sequence of Frame 1 and the following parameters were manually calibrated to match the testing response:

- Damping ratio
- Lap splice model material backbone curve (see Section 3.1)
- Rebar slip model (see Section 3.1)
- Steel material stiffness – changing the stiffness value was an implicit way to change the section stiffness. Rebar location scans revealed that rebar placement was irregular and, in some instances, rebars were found to be inches off their intended location, in section. This variation in section geometries leads to differences in section stiffness. Trying to match these changes through adjustment of the rebar location in the model’s section is rather impractical. Thus, a different section stiffness (compared to the nominal geometry) was implicitly calibrated through adjustment of the material stiffness.
- Beam-column joint rotational spring

The model calibration consisted of the steps listed below. Table 6.1 lists the applied changes to the OpenSees parameters to refine the Frame 1 model. For material models, the material type is specified. Parameter definitions can be found in the OpenSees Wiki (2012).
1. In the refined numerical model, the longitudinal steel rebars in the columns are arranged at 2.375 in. measured from the column surface to the rebar center (i.e., clear concrete cover 1.5 in. + #3 transverse tie 3/8 in. + half of the longitudinal #8 steel rebar 0.5 in.).

2. Although the material nonlinear behaviors of steel rebars and concrete were already considered in both preliminary and refined numerical models, the flexural rigidity of columns in the refined model was reduced by 30%.

3. The yield strength of steel rebars decreases to 65 ksi from 75 ksi of the preliminary model, and the compressive strength of concrete also decreases from -5 ksi to -3 ksi. Moreover, the compressive strength of confined concrete computed using the Mander model is lower than that using the Kent and Park model.

4. The beam-to-column joints in the refined model were modified to consider the nonlinear pull out responses of steel rebars that were placed at the beam bottom and extended into the joints by 6 in. A zerolengthsection element was added between the beam element and joint panel element. In the beam-to-column section of the zerolengthsection element, the material of the pulled-out steel rebars adopts the BarSlip model in OpenSees. The employed values of parameters for the BarSlip model are listed in Table 6.1.

5. The lap-spliced failure in the longitudinal steel rebars in the columns and the pull out failure of the steel rebars in the beams are major damage observed in the experimental as-built frame. As a result, the failure modes dominated the
frame response. The stiffness of the rotational spring for the joint panel is constant in the refined numerical model.

6. Both tension and compression strengths of the lap-spliced steel rebars in the first floor were adjusted to be lower values. In the real frame, the lap-spliced length of steel rebars in the second floor is longer than that in the first floor. Therefore, the lap-spliced mechanism in the second floor was considered in the refined model. The pinch behavior and damage were taken into account in the lap-spliced model.

7. The real SMA braces exhibited more full loops than the hysteresis predicted by the preliminary numerical model. The calibrated values of parameters for the SMA model in the refined frame model are listed in Table 6.1. The forward yield strength increases to 63.3 ksi from 50.0 ksi, and the backward yield strength decreases to 14.0 ksi from 30 ksi. However, the effect of pretension in real SMA wires was not as efficient as the preliminary model expected. Thus, the stiffness parameters of the SMA wires were adjusted to be lower values.

8. The mass values in the second and roof floors for the refined model were adjusted to reflect the real experimental mass.

9. The Rayleigh damping ratio for the refined model is 2.0%, slightly lower than 2.5% used for the preliminary model. The 2.0% value was determined through a series of comparisons of floor displacements between simulation and experimental results.
<table>
<thead>
<tr>
<th>Components</th>
<th>Preliminary Model</th>
<th>Refined Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lap-spliced steel rebar model</td>
<td><strong>OpenSees Hysteretic Material Parameters</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( f_{1p}, f_{2p}, f_{3p} )</td>
<td>54.75, 7.8, 7.8 (ksi)</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{1p}, \varepsilon_{2p}, \varepsilon_{3p} )</td>
<td>0.0047, 0.0113, 0.035</td>
</tr>
<tr>
<td></td>
<td>( f_{1n}, f_{2n}, f_{3n} )</td>
<td>-75.0, -93.75, -93.75 (ksi)</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{1n}, \varepsilon_{2n}, \varepsilon_{3n} )</td>
<td>-0.00259, -0.10600, -0.15900</td>
</tr>
<tr>
<td>pinchX, pinchY, damage1, damage2, ( \beta )</td>
<td>1.0, 1.0, 0.00, 0.00, 0.00</td>
<td>0.8, 0.3, 0.00, 0.02, 0.00</td>
</tr>
</tbody>
</table>

### 2nd floor

<table>
<thead>
<tr>
<th>Components</th>
<th>Preliminary Model</th>
<th>Refined Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{1p}, f_{2p}, f_{3p} )</td>
<td>( X^* )</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{1p}, \varepsilon_{2p}, \varepsilon_{3p} )</td>
<td>( X^* )</td>
</tr>
<tr>
<td></td>
<td>( f_{1n}, f_{2n}, f_{3n} )</td>
<td>( X^* )</td>
</tr>
<tr>
<td>pinchX, pinchY, damage1, damage2, ( \beta )</td>
<td>( X^* )</td>
<td></td>
</tr>
</tbody>
</table>

### Concrete model

<table>
<thead>
<tr>
<th></th>
<th>Kent &amp; Park model (OpenSees Concrete02)</th>
<th>Chang &amp; Mander model (OpenSees Concrete07)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f' )</td>
<td>-5.0 ksi</td>
<td>-3.0 ksi</td>
</tr>
<tr>
<td>( f_y )</td>
<td>75.0 ksi</td>
<td>65.0 ksi</td>
</tr>
<tr>
<td>Transverse reinforcement</td>
<td>2#3 @ 7&quot; (slab beam)</td>
<td>2#3 @ 7&quot; (slab beam)</td>
</tr>
<tr>
<td></td>
<td>2#3 @ 12&quot; (1st-story column)</td>
<td>2#3 @ 12&quot; (1st-story column)</td>
</tr>
<tr>
<td></td>
<td>2#3 @ 6&quot; (2nd-story column)</td>
<td>2#3 @ 6&quot; (2nd-story column)</td>
</tr>
</tbody>
</table>

### Steel rebar model

<table>
<thead>
<tr>
<th></th>
<th>Giuffré-Menegotto-Pinto Model (OpenSees Steel02)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st- and 2nd-story columns</td>
<td></td>
</tr>
<tr>
<td>( f_y, E )</td>
<td>75.0 ksi, 29000.0 ksi</td>
</tr>
<tr>
<td>Components</td>
<td>Preliminary Model</td>
</tr>
<tr>
<td>------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>Strain hardening ratio, $b$</td>
<td>0.01</td>
</tr>
<tr>
<td>Slab beams</td>
<td></td>
</tr>
<tr>
<td>$f_y, \ E$</td>
<td>75.0 ksi, 29000.0 ksi</td>
</tr>
<tr>
<td>Strain hardening ratio, $b$</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Rebar pullout model in beam-to-column sections

<table>
<thead>
<tr>
<th>$f_c', f_y, E_o, f_w, E_{sh}$</th>
<th>$X^*$</th>
<th>Opensees BarSlip Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_b, l_d, d, h, n_p$</td>
<td>$X^*$</td>
<td>1.13&quot;, 6&quot;, 0.75&quot;, 0.75&quot;, 1</td>
</tr>
<tr>
<td>$bs\ Flag, \ Type, \ Damage$</td>
<td>$X^*$</td>
<td>Weak, beamBot, Damage1</td>
</tr>
</tbody>
</table>

Rotational spring for beam-column joints

<table>
<thead>
<tr>
<th>$X^{**}$</th>
<th>348,000 in-kips/rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>(linear elastic rotational spring)</td>
<td></td>
</tr>
</tbody>
</table>

Floor weight

<table>
<thead>
<tr>
<th>2nd-floor: $m_1, m_2, m_3$</th>
<th>25.92, 45.39, 25.92 kips</th>
<th>28.7, 47.0, 25.0 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof: $m_1, m_2, m_3$</td>
<td>25.15, 44.63, 25.15 kips</td>
<td>11.4, 31.6, 15.6 kips</td>
</tr>
</tbody>
</table>

Rayleigh damping ratio

| 2.5% | 2.0% |

$X^*$: Not considered

$X^{**}$: Preliminary model had 2 types of rotational springs for beam-column joints (one for internal and one for external joints). The parameters are shown in Table 6.2.
Table 6.2 – Parameters for beam-column joint rotational spring, preliminary model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>External beam-column joint</th>
<th>Internal beam-column joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_{Pf1}$, $e_{Pf2}$, $e_{Pf3}$, $e_{Pf4}$</td>
<td>[975 3,187 3,564 975] in-kips</td>
<td>[1,071 3,302 3,701 1,071] in-kips</td>
</tr>
<tr>
<td>$e_{Pd1}$, $e_{Pd2}$, $e_{Pd3}$, $e_{Pd4}$</td>
<td>[0.0007 0.006 0.02 0.065] rad</td>
<td>[0.0007 0.006 0.02 0.065] rad</td>
</tr>
<tr>
<td>$e_{Nf1}$, $e_{Nf2}$, $e_{Nf3}$, $e_{Nf4}$</td>
<td>negative of positive parameters</td>
<td>negative of positive parameters</td>
</tr>
<tr>
<td>$e_{Nd1}$, $e_{Nd2}$, $e_{Nd3}$, $e_{Nd4}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$r_{DispP}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$f_{ForceP}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$u_{ForceP}$</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>$r_{DispN}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$f_{ForceN}$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$u_{ForceN}$</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>$g_{K1}$, $g_{K2}$, $g_{K3}$, $g_{K4}$, $g_{KLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$g_{D1}$, $g_{D2}$, $g_{D3}$, $g_{D4}$, $g_{DLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$g_{F1}$, $g_{F2}$, $g_{F3}$, $g_{F4}$, $g_{FLim}$</td>
<td>0 (all)</td>
<td>0 (all)</td>
</tr>
<tr>
<td>$g_{E}$</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$dmgType$</td>
<td>energy</td>
<td>energy</td>
</tr>
</tbody>
</table>

Figure 6.1 compares the response of the calibrated analytical model with the experimental response of the tested frame for both the 1st story displacement and the roof displacement. The comparison is presented for tests 21, 28, 39, and 45; which correspond to the following inputs:

- Test 21: El Centro waveform, 8” peak displacement
- Test 28: Double Pulse 4”
• Test 39: 6-pulse sequence at 1.8 Hz
• Test 45: 4” sine pulse at 1.8 Hz
6.1.2. Retrofitted Frame with 1st Story SMA Bracing (Frame 4a)

Figure 6.1 – Comparison of Frame 1 experimental and simulation results for a) Test 21: El Centro 8”, b) Test 28: Double Pulse 4”, c) Test 39: 6-pulse sequence at 1.8 Hz, and d) 4” sine pulse at 1.8 Hz

The validation of Frame 4a had two separate steps. First, the SMA material model was calibrated using the results from the individual brace component testing. Table 6.3 shows the changes in the SMA material model parameters compared to the preliminary model. Figure 6.2 shows a comparison of the calibrated model with the experimental results for the test described in Section 5.4.4.3, which was subjected to the main loading sequence described in Section 5.4.3.
Table 6.3 – Comparison of OpenSees SMA Material Model Parameters, Preliminary and Refined Models

<table>
<thead>
<tr>
<th>SMA</th>
<th>OpenSees Custom Material</th>
<th>Preliminary Model</th>
<th>Refined Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E, E', E''$</td>
<td></td>
<td>500,000; 75, 800 (ksi)</td>
<td>11,880; 1,003; 1,100 (ksi)</td>
</tr>
<tr>
<td>$\sigma_{\text{ams}}, \sigma_{\text{amf}}$</td>
<td></td>
<td>50.0, 30.0 (ksi)</td>
<td>63.3, 14.0 (ksi)</td>
</tr>
<tr>
<td>$\varepsilon_{\text{in}}, \varepsilon_{\text{pl}}, \varepsilon_{\text{sh}}, \varepsilon_{\text{res}}$</td>
<td></td>
<td>0.0, 0.1050, 0.075, 0.00</td>
<td>0.00288, 0.5, 0.1, 0.01</td>
</tr>
<tr>
<td>$\nu, \eta, \beta$</td>
<td></td>
<td>0.1, 1.0, 1.0</td>
<td>1.0, 1.0, 1.0</td>
</tr>
</tbody>
</table>

Figure 6.2 – Adopted OpenSees (McKenna et al., 2009) SMA brace model, calibrated to reflect the SMA brace test results described in Section 5.4.4.3

In addition to checking the quasi-static component testing results, the brace hysteresis during dynamic full frame testing was also verified. However, due to sync and sensor noise issues (Figure 6.3), this data was not used for calibration.
Figure 6.3 – Brace hysteresis during dynamic testing for Single Pulse scaled to 12". (a) west brace force vs relative displacement, (b) east brace force vs relative displacement, (c) west brace force vs 1st story displacement, and (d) east brace force vs 1st story displacement. Force was calculated using strain gauge data. Relative brace displacements and story displacements are calculated from string potentiometer data. Note: For this test, the brace relative deformation contributed 59% of the total 1st story peak displacement (2.08 in). The brace peak relative displacement was 1.02 in (brace longitudinal direction). Projected horizontally, this would be equivalent to 1.22 in of story displacement.

Following the SMA material model calibration, the validated Frame 1 model, as discussed in Section 6.1.1, was used as the basis for the Frame 4a model. The steel adapters were modelled explicitly, as shown in Figure 6.4, using their true section and nominal material properties. The adapters were connected to the RC beam elements using rigid links. The brace model consisted of the following components:
1) Rigid link to represent gusset plate length

2) A gap element to capture the tolerance of the gusset plate hole relative to the ‘pin’ diameter. It was visually observed during the experimental testing and confirmed by the results that the tolerance between these components led to relative frame movement without engaging the braces. Therefore, it was important to capture this behavior by the model.

3) Elastic beam column elements to model the brace steel tubes. These elements always remained elastic (per the design intent) so an elastic material model was used.

4) A corotational truss element with the previously calibrated SMA material model.

Schematic drawings of the refined Frame 4a model, showing node locations for all elements, are shown in Figure 6.4.
Figure 6.4 – Schematic of Frame 4a model, showing locations of nodes, rotational springs, rigid links, beam column elements, and truss elements for SMA material.
To compare the analytical model to the experimental results, the gravity loads and corresponding masses were modified to match those of the experimental test (refer to Section 5.2.3). Concrete strengths were also updated to match the 28-day cylinder tests listed in Wright, Section 4.2 (2015), since Frame 4 concrete pours were from different batches than the pours for Frame 1.

The input acceleration for model validation was, as it was for Frame 1, the absolute shaker acceleration as measured during testing. The analytical model was subjected to the full LIS testing sequence of Frame 4a. Figure 6.5 shows a comparison of analytical vs experimental time-histories for Frame 4a under the El Centro 6” loading sequence.
6.1.3. Retrofitted Frame with 1st Story SMA Bracing (Frame 4a) and Enhanced 2nd story for Experimental Purposes

For Frame 4b, the validation was done using the Frame 4a analytical model, with cross-bracing tension only elements added to the 2nd story. These bracing elements were given an area that matched that of the physical cables, 0.442 in$^2$. The Elastic-Perfectly Plastic material was used in OpenSees with zero compressive capacity, and a linear elastic stiffness in tension. Since the only change in the stiffness between Frames 4a and 4b came from these cables, the OpenSees elastic material stiffness was manually calibrated to match the experimental story displacement response of Frame 4b tests.
Truss elements were connected to the rigid links of each beam column joint in the 2\textsuperscript{nd} story and roof to create the X-bracing pattern.

This analytical model was subjected to the full LIS testing sequence of Frame 4b (Table 5.2). Figure 6.6 shows a comparison of analytical vs experimental time-histories for Frame 4b under the 12” pulse loading sequence. It can be observed that the analytical model captures very accurately the experimental behavior of the frame.

![Figure 6.6](image)

(a)  
(b)

**Figure 6.6 – Comparison of Frame 4b experimental and simulation results for 12” pulse test, for (a) the 2nd floor and (b) the roof**

6.2. Seismic Assessment using Refined Models

This section covers the seismic vulnerability assessment of the refined as-built test frame model and the frame retrofitted with SMA braces. The assessment follows the same procedure described in Section 3.1.1. Results are discussed below for the prototype as-built structure (Section 6.2.1) and a retrofitted ‘prototype’ structure based on the Frame 4a retrofit model.
6.2.1. As-Built Frame (Frame 1) Fragility Assessment

As was performed in the preliminary fragility assessment, which is described in Chapter 3, the capacity limit state for maximum inter-story drift (%) was assumed to be lognormally distributed. The median values obtained from HAZUS-MH (FEMA, 2003) are 0.5, 0.8, 2.0, and 5.0 for the slight, moderate, extensive, and complete damage states, respectively. The prescriptive capacity dispersions for the lower and higher limit states are 0.25, and 0.47, respectively. Figure 6.7 and Figure 6.8 show the PSDM and fragility curves, respectively, for the refined as-built frame model.

![PSDM curve](image)

Figure 6.7 – PSDM for the model based on refined Frame 1 model, conditioned on the spectral acceleration at 1.0s
6.2.2. Retrofitted Frame (based on Frame 4a) Fragility Assessment

Figure 6.9 and Figure 6.10 show the PSDM and fragility curves, respectively, for the refined retrofitted frame model. For Frame 4a, the spectral acceleration at 1s yielded a seismic demand model with significantly larger dispersion (see Figure 6.11), presumably because of the increased stiffness provided by the brace. Thus, the PSDM conditioned on a spectral acceleration at the structure’s fundamental period (Figure 6.9) is used for comparison against the as-built structure. For the as-built frame, the PSDM conditioned on a spectral acceleration at the fundamental period is also appropriate for comparison purposes. This PSDM also showed relatively low dispersion and a good linear fit – $R^2 = 0.67$ vs the $R^2 = 0.7$ of the PSDM in Figure 6.7. For the limit states of ‘Slight’,

![Figure 6.8 - Fragility curves for the model based on refined Frame 1 model, conditioned on the spectral acceleration at 1.0s](image-url)
‘Moderate’, and ‘Extensive’ damage, the median fragility values were comparably similar for both Frame-1 PSDMs, as seen in Table 6.4.

Figure 6.9 – PSDM for the model based on refined Frame 4 model, conditioned on a spectral acceleration at the 1st fundamental period of the structure

Figure 6.10 – Fragility curves for the model based on refined Frame 4 model, conditioned on a spectral acceleration at the 1st fundamental period of the structure
Figure 6.11 – PSDM for the model based on a refined Frame 4 model, conditioned on a spectral acceleration at 1s. Note that the PSDM fit, as measured by the $R^2$, is worse than that shown in Figure 6.8, which is conditioned on spectral response at the structure’s fundamental period

6.2.3. Comparison of Fragility Curves for As-built and Retrofitted Test Frames

Table 6.4 shows the median spectral acceleration values for all limit states in the two structures presented in the previous section. The SMA bracing system significantly improves the seismic performance for all limit states. The median ground motion that will exceed the extensive damage state increases from 0.47g for the as-built frame to 1.76g for the retrofitted frame, when comparing fragility curves conditioned on the same IM (in this case, $S_a$ @ the structure’s fundamental period, $T_1$).
Table 6.4 – Median spectral acceleration values (g) for limit state exceedance

<table>
<thead>
<tr>
<th>Structure</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-built (IM = $S_a$ @ 1s)</td>
<td>0.253</td>
<td>0.362</td>
<td>0.67</td>
<td>1.276</td>
</tr>
<tr>
<td>As-built (IM = $S_a$ @ $T_1$)</td>
<td>0.222</td>
<td>0.288</td>
<td>0.47</td>
<td>0.775</td>
</tr>
<tr>
<td>Brace Retrofit (IM = $S_a$ @ $T_1$)</td>
<td>0.564</td>
<td>1.156</td>
<td>1.760</td>
<td>2.988</td>
</tr>
</tbody>
</table>

6.3. Comparison of Refined Retrofitted Models - SMA Brace Device (using Half the SMA area) vs Buckling Restrained Brace Retrofits

The retrofitted test frame model was assessed again using two different retrofit schemes. The 1st scheme adopted the same brace configuration as the ‘prototype’ model based on Frame 4a but only 50% of the SMA area used in the original retrofit was assigned to each brace. Hereinafter, this model is referred to as the half-SMA model. The modeling for this half-SMA was very straightforward. The exact same model was used, but the area for the element representing the SMA component was halved.

For the 2nd retrofit scheme, the frame was also braced, but with buckling-restrained braces. The BRB model is described next in Section 6.3.1. For comparison purposes, two BRB retrofit models were assessed. One model had BRBs with a 0.6 in² core steel area and the 2nd model had BRBs with a 0.3 in² core steel area, to match the full- and half-SMA models. The reason for the half-area comparisons is that under the test conditions, the SMA brace was overdesigned. The brace successfully limited interstory drifts and thus structural damage. However, the brace design can accommodate up to 5.5% SMA strain beyond the 2.78% pre-strain, for a total usable strain of 8.28%. During simulation
of the model based on Frame 4b geometry and test loading, as shown in Section 6.2, the peak SMA strains were less than 0.2%. These strains match the strain levels seen during testing. Thus, a model with half the SMA area was evaluated to examine performance with a more economical brace. Results of both SMA models are evaluated against a BRB retrofit simulation to establish a comparison with an established brace retrofit method. The results of these vulnerability assessments are discussed in Section 6.3.2. The performances of both retrofit schemes are compared to each other and against the original ‘prototype’ retrofit based on Frame 4a.

6.3.1. Buckling Restrained Brace Model

The model for the BRB was based on research by Zsarnoczar (2013). The method developed by Zsarnoczar was adopted because it allows a single material model to be used with a single beam finite element to capture both kinematic and isotropic hardening, as well as the asymmetric tension vs. compression response that is typical of BRBs. This implementation allows for simulation of accurate BRB response while remaining computationally efficient for response history analyses of ground motion suites. Zsarnoczar’s material model was incorporated into OpenSees as Steel4 (Steel4 Material, 2015). Zsarnoczar validated the model using cyclic component experimental results (in a similar manner to the calibration of the SMA model in the present study), and the simulation results were in good agreement with the experimental behavior, as seen in Figure 6.12 below (from the plot in Table B3.9 of Zsarnoczar, 2013).
Figure 6.12 – Comparison of experimental and numerical cyclic response of a buckling restrained brace. Original figure is a plot in Table 3.9B from Zsarnoczar (2013).

The BRB assumed for comparison purposes in this study followed the calibration details of specimen PC250, as presented in Zsarnoczar (2013). The reader is referred to Zsarnoczar’s work for full details, but a summary follows here for convenience.

Test specimen PC250 was tested at UC San Diego, had an 18’-6” total brace length, and an A36 steel core with a 14’-11.5” yielding zone length. This configuration resulted in hysteretic behavior (Figure 6.12) that cycled between a yield and ultimate stress range of 42-63 ksi in tension and 42-87 ksi in compression (at 2.5% strain). This range covers this study’s SMA brace (wires) stress range of 44-55 ksi (tension) and 44-61 ksi (compression), as seen in the quasi-static test results. Therefore, the *Steel4* parameters
that Zsarnoczar used to match the PC250 test results were adopted in this study to simulate a BRB retrofit for Frame 1. These Steel4 parameters are summarized in Table 6.5. Definitions for each parameter can be found in the OpenSees Wiki (Steel4 Material (2015)). For full modelling details and parameter derivation, the reader is referred to Zsarnoczar (2013). In the present study’s model, the BRB steel core yielding section was assigned an area of 0.6 in².

Table 6.5 – Steel4 parameters used for the BRB OpenSees model (from Zsarnoczar, 2013)

<table>
<thead>
<tr>
<th>Steel Core Properties</th>
<th>Steel4 Material Model Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material – A36</td>
<td>Tension</td>
</tr>
<tr>
<td>Characteristic</td>
<td>Actual</td>
</tr>
<tr>
<td>fᵧ</td>
<td>36 ksi</td>
</tr>
<tr>
<td>fᵤ</td>
<td>58 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Kinematic Hardening Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone</td>
<td>Thickness</td>
</tr>
<tr>
<td>Yielding</td>
<td>0.8 in</td>
</tr>
<tr>
<td>Transition</td>
<td>0.8 in</td>
</tr>
<tr>
<td>Elastic</td>
<td>0.8 in</td>
</tr>
<tr>
<td>Area in yielding zone</td>
<td>0.775* in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Backbone</th>
<th>Isotropic Hardening</th>
</tr>
</thead>
<tbody>
<tr>
<td>ω₂%</td>
<td>1.42</td>
</tr>
<tr>
<td>Conversion Factors</td>
<td>bᵢso,u</td>
</tr>
<tr>
<td>fₛM</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note: 0.775 in² is the area of specimen PC250, as reported in Zsarnoczar (2013). In the present study, the core yielding area assumed equal to 0.6 in².
6.3.2. Performance Comparison of BRB and Half-SMA Area Retrofitted Frames

Table 6.6 shows the median spectral acceleration values for all limit states in the two structures described in the previous section (BRB and SMA with half-area), and compared to the prototype model based on Frame 4a (Section 6.2). Predictably, reducing the SMA area by 50% (0.3 in$^2$ instead of 0.6 in$^2$) results in decreased seismic performance. However, the retrofitted structure still performs significantly better than the as-built, with a median spectral acceleration of 0.997g to exceed the extensive damage, compared to 0.47g for the as-built structure. The SMA brace provides performance that is similar to what could be achieved with a BRB, in terms of median probability of exceeding specific limit states. However, the SMA brace results in reduced residual drifts compared to both the as-built frame and the frame with BRB (Table 6.7).

Table 6.6 – Median spectral acceleration values (g) to exceed limit states

<table>
<thead>
<tr>
<th>Structure</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-built (IM = $S_a @ T_1$)</td>
<td>0.222</td>
<td>0.288</td>
<td>0.47</td>
<td>0.775</td>
</tr>
<tr>
<td>Full-SMA area (IM = $S_a @ T_1$)</td>
<td>0.564</td>
<td>1.156</td>
<td>1.760</td>
<td>2.988</td>
</tr>
<tr>
<td>Half-SMA area (IM = $S_a @ T_1$)</td>
<td>0.372</td>
<td>0.693</td>
<td>0.997</td>
<td>1.578</td>
</tr>
<tr>
<td>Full-area BRB (IM = $S_a @ T_1$)</td>
<td>0.642</td>
<td>1.162</td>
<td>1.646</td>
<td>2.551</td>
</tr>
<tr>
<td>Half-area BRB (IM = $S_a @ T_1$)</td>
<td>0.297</td>
<td>0.522</td>
<td>0.726</td>
<td>1.099</td>
</tr>
</tbody>
</table>
Table 6.7 – Largest peak and residual drifts. Note that the largest value for each model resulted from different ground motion records in the suite. Thus, the ground motion PGA is specified for each drift value.

<table>
<thead>
<tr>
<th>Structure</th>
<th>1st Story</th>
<th>GM PGA (g)</th>
<th>2nd Story</th>
<th>GM PGA (g)</th>
<th>1st Story</th>
<th>GM PGA (g)</th>
<th>2nd Story</th>
<th>GM PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-built</td>
<td>5.9</td>
<td>0.51</td>
<td>0.61</td>
<td>0.34</td>
<td>2.46</td>
<td>0.30</td>
<td>0.03</td>
<td>0.63</td>
</tr>
<tr>
<td>Full-SMA</td>
<td>0.41</td>
<td>0.71</td>
<td>0.38</td>
<td>0.71</td>
<td>0.005</td>
<td>0.13</td>
<td>0.0018</td>
<td>0.13</td>
</tr>
<tr>
<td>Full-BRB</td>
<td>0.7</td>
<td>0.67</td>
<td>0.22</td>
<td>0.72</td>
<td>0.057</td>
<td>0.74</td>
<td>0.007</td>
<td>0.60</td>
</tr>
<tr>
<td>Half-SMA</td>
<td>0.53</td>
<td>0.65</td>
<td>0.51</td>
<td>0.70</td>
<td>0.017</td>
<td>0.64</td>
<td>0.021</td>
<td>0.26</td>
</tr>
<tr>
<td>Half-BRB</td>
<td>1.18</td>
<td>0.67</td>
<td>0.27</td>
<td>0.74</td>
<td>0.25</td>
<td>0.67</td>
<td>0.012</td>
<td>0.67</td>
</tr>
</tbody>
</table>

Selected 1st story time-history displacements are also shown (Figure 6.13) to visually appreciate the reduction in story displacement achieved through retrofitting. The results shown are for the Rix-Fernandez suite (2006) 2475-yr return period Cape Girardeau, MO, record #10 [0.57g PGA, Figure 6.13 (a)] and 2475-yr return period Paducah, KY, record #04 [0.65g PGA, for Figure 6.13 (b)-(d)]. The time-histories seen in Figure 6.13 (b), (c), and (d) show that the SMA brace can provide structural response comparable to a BRB retrofit, with reduced peak story displacements. It should be noted that the as-built displacement shown in Figure 6.13 (a) corresponds to a peak drift of 2%. As indicated in Wright (2015), 2% drifts would have likely caused the collapse of this structure if the collapse prevention safety cables had not been in place. So, while numerically the OpenSees model registered peak drifts above 2% (e.g. ground motion 239 as shown in Table 6.7), physically the time-history shown in Figure 6.13 (a) would have likely resulted in collapse at or before the 75 sec mark.
Figure 6.13 – Comparison of 1st story displacements for select Rix-Fernandez suite (2006) motion time-histories: (a) as-built vs. SMA braced frames 0.6 in² (100% area) and 0.3 in² (50% area) (b) SMA braced both 0.6 in² and 0.3 in² (100% and 50% area), (c) SMA braced vs BRB, 0.6 in² (100% area) for the core resisting component, and (d) SMA braced vs BRB, 0.3 in² (50% area) for the core resisting component. The results shown are for the 2475-yr return period Cape Girardeau, MO, record #10, 0.57g PGA [plot (a)] and 2475-yr return period Paducah, KY, record #04, 0.65g PGA [for plots (b)-(d)].
6.4. Concluding Remarks

Models previously developed for preliminary fragility assessments, as described in Chapter 3, were calibrated using results from the experimental testing program discussed in Chapter 5. In this chapter, updated models were presented for an as-built 2-story, 2-bay frame (herein referred to as ‘Frame 1’) representative of a single bay in the prototype structure (Wright, 2015) and the same frame retrofitted with the SMA bracing system that was tested experimentally (herein referred to as ‘Frame 4’).

Using the updated models, seismic fragility assessments were carried out using NLTHA and fragility estimates for both models were compared to discuss the enhanced performance of the retrofitted frame. For the as-built frame, the PSDM considering the 5% spectral acceleration at 1 s as an intensity measure provided the best PSDM fit out of the 4 IMs that were considered. However, for the retrofitted model, it was the spectral acceleration at the structure’s 1st fundamental period that resulted in the best PSDM out of the four examined IMs. The retrofitted structure showed a significant improvement in seismic performance over the as-built structure for all limit states. Of particular importance for practical purposes, the median probability of exceeding the extensive damage state increased from a spectral acceleration of 0.67g (indicative of design level shaking in a relatively hazardous CSUS location such as Memphis, TN) in the as-built structure to 1.76g in the retrofitted structure. This performance increase indicates that enhanced safety can be achieved when SMA bracing is adopted for retrofitting.

The calibrated models were also compared to a buckling-restrained braced frame, using the original brace (0.6 in² SMA area) as tested and described in Chapter 5, as well
as a brace with half the SMA area (0.3 in\(^2\)). The results indicate that the SMA brace developed in this study could provide better performance improvements than a BRB, with the added benefit of reduced maximum and residual drifts. Even with a reduced amount of SMA material, significant reductions in interstory drifts and, correspondingly, probabilities of exceeding performance limit states are achieved by the SMA brace retrofit, when compared to a non-retrofitted frame model.

The results of this assessment seem promising for regions of moderate seismicity such as the CSUS. Several areas around the World face similar conditions as the CSUS, namely high prevalence of RC buildings with deficient reinforcement detailing in locations of similar seismicity. The SMA bracing scheme presented in this study could be a valuable retrofit alternative for buildings in these scenarios.

Future research for this retrofit scheme could focus on two particular topics independently. The first one would be assessing if the same scheme could work in buildings of similar size and geometry but in areas of higher seismicity, e.g. the US west coast. The second area of research could study using the SMA bracing scheme in buildings facing a similar seismic hazard as the CSUS, but with different geometries, e.g. more floors, different bay width to floor height ratios, different structural configurations such as frames with infill walls or dual frame-wall systems.
6.5. References


CHAPTER 7. Refined Retrofit Design Procedure – Design

Recommendations for SMA Brace Device

Based on the preliminary design and numerical and experimental results, a more robust design procedure for structures retrofitted with SMA braces is proposed. The first step is to generate a set of ground motion records which considers the seismic hazard of the structure site to be retrofitted. Following is the establishment of numerical models for the as-built and SMA retrofitted frames of the structure. For the SMA braced model, the forward transformation yield stress for Ni-Ti superelastic wire SMA is assumed to be 60 ksi (414 MPa); the total SMA area is arbitrarily assumed to an initial reasonable number for later use in the iterative design process. Subsequently, both the as-built and SMA braced frame models are subjected to the selected ground motion set. The peak story drifts and peak SMA strains are recorded in each run, and then the maximum values among the peak demands from all the runs are selected to check if the design criteria (e.g., inter-story drift or SMA strain) are satisfied. If any of the design criteria are not satisfied, another iteration of SMA design area begins. Once all the design criteria are satisfied, the design for the SMA is final. During all simulations, the peak maximum forces experienced in the braces are also recorded for designing adjacent elements, such as the brace steel tubes, attachments, adapter, etc. A flowchart of the retrofit design for existing structures retrofitted with SMA braces is shown in Figure 7.1.
7.1. Retrofit Design Example

In this section, the design flowchart shown in Figure 7.1 is expanded upon, for the convenience of readers who wish to go through the full design of the retrofit device described in this thesis. The steps shown next have been covered elsewhere in this document, but they are summarized here as a step-by-step guide. Relevant sections (in this document) and references (outside of this document) are given where applicable.
7.1.1. Ground Motion Generation

As described in Chapter 3 and Chapter 6, the ground motion suite developed by Fernandez & Rix (2006) was used in this project since the suite was developed for the Upper Mississippi embayment region, which is the region of interest for the structure assessment and retrofit design that comprise this study. For site specific analyses where a ground motion suite is not available, ASCE 7-16 (ASCE, 2016) provides general guidelines for generating ground motions.

In ASCE 7-16, the seismic hazard in and around the Upper Mississippi embayment was updated. Thus, an example comparison of the Fernandez-Rix suite mean spectrum to the design spectrum per ASCE 7-16, for a site with latitude 35.697 and longitude -89.82 is shown in Figure 7.2.

![Figure 7.2 – Comparison of Fernandez-Rix mean spectrum to ASCE design spectrum](image)

Figure 7.2 – Comparison of Fernandez-Rix mean spectrum to ASCE design spectrum
The full details for ground motion selection or generation are outside the scope of this design example (and this study) so the reader is referred to (ASCE, 2016; Baker & Lee, 2018; Haselton et al., 2012; Fernandez, 2007) for more information.

7.1.2. Numerical Models

In this study, Opensees (McKenna, 2009) was used to develop the numerical models of the as-built and retrofitted frames, as described in Section 3.1 and Section 6.1.2, respectively. For nonlinear analysis, elements contributing to structural response must be modeled with the required parameters that dictate ductile or brittle response. For example, for non-linear elements, these parameters could include the material backbone curves, loading/unloading stiffness, pinching, and cyclic degradation. ASCE 7-16 and ASCE 41-13 offer general guidelines for modeling and acceptance criteria of these elements, based on whether they are “critical”, “ordinary”, or “non-critical” and governed by force-controlled or deformation-controlled actions. For low-rise, non-ductile RC frame structures, the modeling methods shown in Chapter 3 of this study may serve as the basis for modeling. For different structure types, the reader is referred to the following (non-exhaustive) list of references:

- For general new design of tall buildings: PEER TBI Guidelines for Performance Based Seismic Design of Tall Buildings (PEER, 2017)
7.2. Seismic Analysis

The basis of analysis for this study was a non-linear time history using a 240-ground motion suite and a 2D model. Some of the decisions taken for analysis in this study differ significantly from generally accepted analysis requirements of more practical (i.e. realistic) structures. The reasons behind these discrepancies stem from the fact that the analysis was used for design and comparison of structural elements in a controlled lab setup. Mainly, the experimental test frame was constrained against out-of-plane motion, the foundation was over-designed to limit any soil-structure or foundation flexibility effects, and the structure was meant to represent a general region of seismic hazard (the Central US, New Madrid Fault Zone) rather than a specific site. Thus, given these discrepancies, a 2D model was chosen rather than a 3D model (since the test structure would be purposely constrained out-of-plane), a fully rigid foundation was modeled, and a 240-motion suite was chosen for fragility analysis.

However, in this section, examples for more practical design scenarios are given. For general guidance, the reader is referred to ASCE 41-13, where general analysis requirements are given for four different analysis procedures, namely the linear static procedure (LSP), linear dynamic procedure (LDP), the nonlinear static procedure (NSP), and the nonlinear dynamic procedure (NDP). Due to the brittle nature of the RC structure investigated and the lack of existing studies (other than this one) on the recommended retrofit, only nonlinear dynamic procedures are recommended at this time. ASCE 41-13 provides a thorough checklist, with guidelines, for the necessary considerations that lead to an acceptable nonlinear dynamic analysis. The following aspects should be considered
by the designer, at a minimum, but it is noted that this is not an exhaustive list, and guidelines or standards other than ASCE 41-13 may also offer general guidance:

- **Gravity load combinations during seismic excitation**
  - Section 3.1 in this study
  - For general assessment and retrofit: Section 7.2.2 in ASCE 41-13

- **Structural element rigidity and strength assumptions, as well as non-structural element rigidity and/or mass contributions**
  - Section 3.1 and Section 6.1.2 in this study
  - For other structural elements/systems: e.g. ASCE 41-13 Chapter 9 (steel systems) or Chapter 10 (concrete systems)

- **Damping assumptions**
  - In this study, damping was assumed based on existing literature (as detailed in Section 3.1) for preliminary analysis and later calibrated against experimental results (as detailed in Section 6.1.1).
  - For general assessment, ASCE 41-13 Section 7.2.3.6 offers guidance

- **Foundation modeling assumptions**
  - In this study, the analysis model used a fixed foundation as this accurately represented the test structure
  - For general assessment, ASCE 41-13 Section 7.2.3.5 offers general guidance

- **Out-of-plane, overturning, and vertical effects**
These were ignored in the present study since out-of-plane response was explicitly prevented and vertical or overturning effects were assumed negligible due to the structural configuration.

- **Multi-directional and concurrent seismic effects**
  - Ignored for this study due to the experimental setup. The excitation was aligned with the test frame’s longitudinal direction.
  - ASCE 41-13 Section 7.2.5 provides guidance for multi-directional and concurrent seismic effects.

- **Number of acceptable ground motions in the analysis suite**
  - The full 240 ground motion Fernandez-Rix suite (Fernandez, 2007) was used in this study, since the structural configuration was meant to represent a structural archetype representative of a wide region (the New Madrid Seismic Zone) and the comparative assessment was based on fragility results using cloud analysis.
  - However, for site-specific design of a single structure, ASCE 41-13 prescribes a required number of ground motions in one suite, based on the analysis type and hazard condition, and the required minimum does not exceed 10. Refer to ASCE 41-13 Section 7.2.5 for further details.

- **Consideration of P-Δ effects**
  - In the preliminary assessment (Chapter 3), P-Δ were not considered, as only the test structure itself, a plain, 2 story, 2-bay frame was considered. For the updated assessment (Chapter 6), global P-Δ effects were considered through a leaning column model.
• For general design and analysis, ASCE 41-13 Section 7.2.6 offers guidance.

• Soil-structure interaction

  o Soil-structure interaction was ignored in the present study due to the foundation configuration.
  
  o For general design and analysis, ASCE 41-13 Section 7.2.7 offers guidance

7.2.1. Design and Acceptance Criteria

For this study, the acceptance criteria were based, at a global level, on inter-story drift, and at a local level on SMA strains. The design criteria in this study was based on experimental testing. For a practical design, the retrofit criteria should be based on the same criteria outlined here, as shown in Figure 7.1. The global inter-story drift criteria should be based on the guidelines that govern the design, based on structure type and applicable jurisdictions. Within the USA, ASCE 7-16 and/or ASCE 41-16 are recommended.
7.3. References

American Society of Civil Engineers (ASCE) Committee. (2013). Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-13). American Society of Civil Engineers. Reston, VA: ASCE.


CHAPTER 8. Summary, Conclusions, Research Impact, and Recommendations for Future Work

8.1. Summary

Direct economic losses incurred in seismic events have ranged from $7 billion USD in the 1989 Loma Prieta earthquake, to over $300 billion USD in the 2011 Tohoku earthquake (Daniell, Khazai, Wenzel, & Vervaeck, 2011). Impact assessment reports (Elnashai et al., 2009) from the Mid-America Earthquake Center estimated that an Mw 7.7 earthquake in the New Madrid Seismic Zone (NMSZ) could result in approximately $300 billion in economic losses and nearly 86,000 human injuries and fatalities. About 715,000 buildings (primarily in Tennessee, Arkansas, and Missouri), including 130 hospitals, could end up damaged from this seismic event. These losses are compounded once social impacts and indirect economic losses are considered. In addition, recent experiences and seismic provision updates (ASCE, 2016), primarily within the Central and Eastern USA, have amplified the need to increase knowledge of the performance of older structures during seismic events in these areas.

These issues motivated the current study and its companion studies (Wright, 2015; Shin, 2017), to experimentally and analytically investigate the seismic performance of a series of prototype frames in as-built and multiple retrofitted conditions. Provisions of ACI 318-63 (ACI Committee, 1963) were used to design the interior column line of a prototype reinforced concrete gravity frame for the experimental evaluation of the four side-by-side planar frame specimens. The first of four test specimens was tested as part of
Wright’s companion study in the as-built configuration under seismic loads using a roof mounted inertial shaker to define the frame’s system-level performance.

The other three frames were tested after incorporating near surface mounted bars and FRP wrapping (Frame 2 – Wright, 2015), an FRP column jacketing retrofit (Frame 3 – Shin, 2017), and a SMA bracing system retrofit (Frame 4, this dissertation). In addition, this study also conducted a seismic assessment using fragility analysis to compare the performance of the SMA retrofitted frame (Frame 4) against the as-built frame (Frame 1), in a design loading scenario of the 5-bay 2-story prototype (Wright, 2015) structure considering a seismic hazard representative of the New Madrid Seismic Zone.

Based on preliminary design assumptions, a seismic fragility assessment of a 2-story, 2-bay non-ductile RC building was carried out using NLTHA to select the retrofit scheme for experimental testing. Several reinforcement detailing deficiencies representative of pre-1970’s construction in low-to-moderate seismic zones in the US were considered in the as-built frame analytical model. A frame model retrofitted with a preliminary version of the SMA brace retrofit was compared against a model with retrofits addressing the short lap splice deficiency near the column foundation and another model with retrofitted beam-column joint reinforcement. Results showed the SMA bracing scheme provided a superior increase in performance compared to the other models considered. Thus, this scheme was chosen for refined design and experimental testing.

After establishing global design criteria and parameters following the preliminary fragility assessments and the design from the companion study (Wright, 2015), material
and component tests were done on the SMA parts that would comprise the main resistance and energy dissipation element of the brace retrofit scheme. Component tests were done on 2 ft. long SMA wire specimens and circular SMA rods (13.5 in. gage length). Test results showed a well-defined austenite-to-martensite transformation at approximately 2% strain followed by a stress plateau and then a typical stress-induced martensite stiffening after approximately 8% strain. The ‘yield’ stress (approximately 70 ksi, refer to Chapter 4) and length of the post-transformation plateau were used in a refined analysis to establish the desired SMA area and stroke length for full-scale system experimental testing. Provisions from ASCE7-10 (ASCE, 2010), AISC 360-10 (AISC, 2010), AISC 341-10 (AISC, 2010), and ACI 318-11 (ACI, 2011) were used to establish global design parameters, detailed steel design, seismic design compliance, and concrete anchoring design, respectively.

Full-scale experimental testing followed the detailed retrofit design, assembly, and installation of the SMA bracing system. Using a mobile shaker system, a series of full-scale dynamic tests were conducted on a two-story, two-bay, non-ductile RC test frame retrofitted with the SMA bracing scheme in the first story. The full-scale dynamic shaker loading provided more realistic behavior of the test frame than previous shake table tests found in the literature, which used reduced-scale specimens (e.g. Bazant & Kim, 1984; Bazant, Sener, & Prat, 1988; Litle & Paparoni, 1966). The dynamic responses for the retrofitted test frame were compared to those of the identical as-built RC test frame (Frame 1), which was previously tested. This comparison served to quantify the effectiveness of the SMA bracing system. The installation of the retrofit system was shown to be effective in reducing the inter-story drift ratio and mitigating the soft-story
mechanism found in the 1st story of the as-built test frame. The main components of the retrofit scheme, the SMA braces, were also tested individually to quantify the full cyclic response and refine the material models in numerical analysis.

Based on the measured dynamic responses, the preliminary numerical models used in the retrofit selection stage were updated. To validate the calibrated models, the simulated responses from the numerical frame models with calibrated lap splice and bond-slip beam rebar (in the as-built configurations) and updated SMA material model (for the retrofitted configuration) were compared to the experimental responses in terms of displacement time history and peak inter-story drift ratios. The numerical simulation showed reasonable agreement with the experimental results, with a maximum variation of approximately 15% peak inter-story drift for El Centro input excitations. The validated models were later used in a final fragility analysis to statistically assess the improvement in global performance of a 2-bay, 2-story RC structure retrofitted with the SMA bracing scheme. Global performance, based on peak inter-story drift ratios, improved for all limit states.

Finally, a design procedure is given for a detailed retrofit design that utilizes the tested SMA brace device. This procedure is recommended for use in practical retrofitting schemes of structures that meet the design assumptions used in the present study.

8.2. Conclusions

1) When testing structural response using a shaker mounted on the roof or any higher story within a structure, careful consideration must be given to the vertical stiffness distribution of the structure relative to the location of the shaker. During
the experiment design portion of this study, it was initially concluded that the higher stiffness and ductility of the 2nd story—increased reinforcement and confinement in the beams, columns, and joints relative to the 1st story due to the roof-mounted shaker—would be sufficient to forego the addition of bracing the 2nd story with a full set of SMA braces. However, as shown in Section 5.3.1.1, this was not the case because the 2nd story had to be enhanced in order to safely transfer the shaker induced inertial force into the 1st story and allow the frame specimen to be tested beyond an input acceleration of 0.6g.

2) The SMA brace device effectively reduced inter-story drifts and visible cracking/damage compared to the as-built frame. In Wright's (2015) companion study, it is shown that the as-built frame would have likely collapsed at around a 2% story drift had the sway prevention cables not been in place. Moreover, the fragility assessment showed that the median ground motions that lead to exceedance of all limit states are significantly increased for the retrofitted structure compared to the as-built one (for “extensive damage” 0.47g and 1.76g, respectively). Therefore, it is concluded that the SMA brace device can effectively mitigate life-threatening damage following a seismic event in the low-rise nonductile RC frames typical to the CSUS.

3) If compared to the as-built response and the response of a BRB-retrofitted structure, SMA bracing also reduces residual drifts. The largest residual drift of any SMA brace simulation was only 0.017%. For the BRB braced frames, the largest residual drift was 0.25%. This difference is significant since researchers (McCormick, Aburano, Ikenaga, & Nakashima, 2008) have concluded that
residual drifts greater than 0.5% may represent a complete loss of a building structure from an economic perspective. In addition, previous research (Ocel et al., 2004; Speicher, DesRoches, Leon, 2011) has demonstrated that post-test heat treatment of SMA elements with residual deformations can restore the SMA to its original shape. The re-centering capability together with the SMA element reusability significantly reduces the life-cycle costs of SMA retrofitting schemes.

4) SMA wires have more attractive characteristics compared to SMA rods, with respect to both fabrication and treatment procedures as well as structural behavior. Component testing of individual prestressed SMA braces showed that the SMA wires can provide hysteretic damping and a high degree of re-centering after multiple cycles of beyond-yield stress loading at varied strain rates. The wires performed better than SMA threaded rods, which failed suddenly in one of the component tests after brittle rupture immediately adjacent to the threads. In addition, the SMA manufacturer indicated that heat treatment and cold working methods are easier for SMA wires compared to SMA rods.

8.3. Research Impact

This study presented the development and testing of an innovative retrofit device for use in nonductile reinforced concrete frames. The research resulted in the following contributions to the field of knowledge in structural engineering:

- Development, testing, and formulation of a detailed design procedure for a new class of retrofit device that takes advantage of the super-elastic, and shape-memory properties of shape-memory alloys, which may result in energy
dissipation and re-centering of structural systems. As evidenced by test results and analytical assessments, this retrofit scheme is characterized by properties that offer valuable performance improvements for nonductile structures in areas of moderate seismicity, with energy dissipation being an improvement over performance of as-built nonductile structures and re-centering and advantage compared to conventional retrofit methods (such as steel bracing and BRBs). Preparatory assembly and installation for full-scale testing showed that construction for this method can be performed using light construction equipment and a small construction crew, which may lead to retrofitting schedules that are less disruptive to continued building operations than conventional retrofit schemes.

- First-of-its-kind full-scale broadband shaking experimental test of a nonductile RC structure retrofitted with a bracing system. Testing done for this dissertation and its companion studies showed that scaled testing may not adequately capture the degradation or failure mechanisms present in nonductile RC frames. Although it is obviously more expensive, it is recommended that further experimental of nonductile RC systems or components uses full-scale specimens.

- The full data set collected during this study has been documented and made publicly available for use by future researchers. In particular, acceleration and displacement data may be of interest to researchers trying to validate numerical models at a system level and establish SMA material models for use within larger component- or system-level analyses.
8.4. Recommendations for Future Work

This research presented first of its kind full-scale tests for an innovative brace retrofit system using shape memory alloys and system level tests of a low-rise reinforced concrete gravity frame, retrofitted with the SMA bracing system, using forced vibration linear shakers. As with the companion studies (Wright, 2015; Shin, 2017), much was learned, but limitations in the existing body of knowledge remain. To extend the state-of-the-art in this area, research into the following topics is recommended:

1) For the as-built condition, performance was evaluated using the finite element numerical model that represented the full-scale as-built test frame specimen. However, performance for the as-built frame can be varied depending on material properties of concrete and steel, aspect ratios (height-to-depth or length-to-depth) of column and beam elements, longitudinal reinforcement ratios, transverse reinforcement ratio, and types of transverse reinforcement. These parameters can affect failure modes of RC building structures under seismic loading scenarios. To generalize performance criteria for existing non-ductile RC frames, a variety of input parameters associated with structural detailing should be added.

2) This study focused on low-rise structures in areas of moderate seismicity. Further research could study and compare the effects of varied seismic hazard levels and structural heights on the performance of structures retrofitted with the SMA bracing system.

3) Some improvements for areas of concern regarding the newly develop SMA retrofit were mentioned throughout this study. Of particular importance were:
a. The connection of the steel adapters to the RC frame. In RC frames with denser rebar layouts, post-anchoring directly to columns and/or the underside of beams may prove impractical. Different anchoring methods should be studied to improve the constructability of the proposed SMA retrofit.

b. To achieve a true ‘pin’ support condition for the brace, single large diameter bolts were custom manufactured for this research. Custom manufacturing of bolts is more expensive and time consuming than procuring standard bolt sizes regularly used in typical steel construction. Different gusset plate support configurations may be studied to determine the feasibility of using more practical assembly/construction methods.

c. The pin support used to transfer SMA forces to the steel tubes suffered from deficient stiffness. This was an error in preliminary design assumptions that can be easily fixed and studied using a stiffer element. It is expected that an increase in global bracing system stiffness, coming from the increased transfer pin stiffness, will improve the system-level structural performance of a building retrofitted with this bracing system.

d. Using SMA wires is highly recommended over SMA rods. Thread cutting an SMA rod is very difficult so threads much thinner than those in conventional rods/bolts were used, which required custom machining of a nut and a very long thread length in order to have enough threaded area to withstand the shear forces of the nut connection. Even with the long thread length, two of the short (2 ft.) specimens used in the preliminary testing
and one of the 6ft. full scale specimens failed in a brittle manner due to either thread shear failure or fracture initiated by cracking at the start of the threads.

e. The SMA wire ends should be rolled around a rod with a diameter slightly larger than the ‘pin’ element that will be used to transfer forces to the steel tubes. During assembly, the hardest step was transferring the SMA wires from their shipment container to the steel transfer pieces. Significant care was required to ensure no wire damage occurred during this transfer. Assuming that the same extremely careful handling of the SMA materials will be met in a typical construction site might be risky, and costly. Alternatively, the entire SMA and steel tube component should be factory assembled and shipped as a unit to the construction site.

4) For the retrofitted condition, the present work only focused on an SMA bracing system. The companion studies of Wright (2015), and Shin (2017) looked at different configurations of FRP-based column and beam-column joint retrofitting systems. Seismic resistance of existing non-ductile RC frames can be strengthened by various retrofit systems, such as infilled wall systems, additional FRP wrapping methods, or buckling-restrained bracing systems. To develop a comprehensive retrofit approach considering various retrofit systems, a similar fragility assessment procedure as followed in this dissertation needs to be performed for various retrofit systems, and results must be compared. Ideally, this could be done for a suite of combined building configurations (as outlined in point 1 above) and retrofit schemes. Clearly this would be a very extensive
analytical research project, which could potentially produce more than one academic dissertation.

5) A general recommendation for overall structural assessments using statistical methods (e.g. fragility analysis) is to aggregate results and parameters for existing research of seismically vulnerable buildings into databases that could be used to calibrate numerical models of future studies. Such databases would facilitate comparison studies across institutions since the models would be validated using the same base conditions.
8.5. References


APPENDIX A. Calculations for Preliminary Analysis of the Brace

Retrofit

The design of the retrofit device followed a 2-stage process. In the 1st stage, an equivalent lateral force procedure was used to design the SMA area needed in each brace. Then, using a non-linear time-history analysis and with an SMA material model developed by Speicher (2009), the SMA component area and required stroke length were confirmed based on a target reliability via iteration with multiple analytical stroke lengths.

The initial SMA area was designed according to analysis results from an equivalent lateral force (ELF) procedure in ASCE 7-10 (2010). Though the ASCE 7-10 ELF procedure is meant for new design, not structural retrofits, it was deemed appropriate as an initial SMA area estimate, since the performance would be corroborated later through nonlinear analysis. The detailed ELF calculations are shown next.

A.1. Linear Static Procedure

The equivalent lateral force procedure as outlined in ASCE 7-10 was used to determine the peak forces each brace would need to resist given the parameters of the brace test frame. At the time of the preliminary design, the full details of the NEES linear inertial shaker were not known, so assumptions were made to determine the lateral forces that could be encountered during testing. To follow the ELF, the fundamental period was determined as $T = 0.48\, \text{s}$, through eigenvalue analysis using the OpenSees (McKenna et
al., 2009) model described in Chapter 3. The following parameters were assumed for the ELF procedure:

- Site latitude: 35.697
- Site longitude: -89.820

Which yielded the following spectral accelerations parameters, taken from (https://seismicmaps.org/):

- Short-period spectral acceleration, $S_S$: 1.907g
- Long period spectral acceleration, $S_1$: 0.681g
- Assuming a site class D (typically assumed when site soil information is not available or unreliable), the following site class parameters are obtained:
  - Short-period site coefficient (at 0.2 s-period), $F_a$: 1.0 (ASCE 7-10 Section 11.4.3)
  - Long-period site coefficient (at 1.0 s-period), $F_v$: 1.5 (ASCE 7-10 Section 11.4.3)
  - Site class adjusted short-period spectral acceleration, $S_{MS} = F_a S_S = 1.907g$
  - Site class adjusted short-period spectral acceleration, $S_{M1} = F_v S_1 = 1.022g$
  - Short-period design spectral acceleration, $S_{DS} = 2/3 S_{MS} = 1.271g$
  - Long-period design spectral acceleration, $S_{D1} = 2/3 S_{M1} = 0.681g$
  - $T_S = S_{D1}/S_{DS} = 0.536s$, and $T_0 = 0.2 T_S = 0.107s$

For $T = 0.48$ s, as determined by eigenvalue analysis, $T_0 < T < T_S$, and $S_a = S_{DS} = 1.271g$. Given the nature of the retrofit, a composite (steel and concrete) special concentrically braced frame was assumed, per Table 12.2-1 in ASCE 7-10, this results in a Response Modification Coefficient, $R = 5$. The prototype building is assumed to be a
regular occupancy commercial building, thus the importance factor, $I_e = 1$, per ASCE 7-10 Section 11.5.1.

The seismic base shear is given by $V = C_s W$, as defined in ASCE 7-10 Section 12.8.1, where:

$$C_s = \frac{S_{DS}}{(R/ I_e)}$$

Which yields $C_s = 1.271 / 5 = 0.254$, and $V = 0.254 \times W = 48.86$ kip, where the effective seismic weight, $W = 192.16$ kip was determined from all dead loads plus 25% of the live loads, for a corresponding tributary area of half a bay width (18’ bays, as shown in Figure A.1) per the prototype building design (Wright, 2015). It was conservatively assumed that the braces would resist the full lateral load. Both bays of the test frame would be retrofitted so each brace needed to resist:

$$V_{per-brace} = 0.5 \times (48.86) = 24.43$$ kip laterally,

which equaled:

$$F_{brace} = 24.43 \times (21.6/18) = \textbf{29.32 kip}$$

where 21.6 ft (the brace length) is the hypothenuse of a triangle with a 12 ft height (the story height) and an 18 ft base (the bay width).
Figure A.1 – Schematic showing dimensions of the prototype structure and highlighting the column of interest (the basis of the experimental test frame)

A.2. Reliability-based Stroke Length Calculations

For the initial estimate of allowable story drift, results from analysis using the full Rix suite of ground motions (Fernandez & Rix, 2006) and a preliminary version of the model described in Chapter 3 of this dissertation were investigated. The probabilistic seismic demand model based on these results is shown in Figure A.2. A target system reliability, $\beta = 2.5$ was chosen, which targeted brace deformations of approximately 1.3 in. from a suite of analyses. Details about these calculations follow next.
If the demand (D) and capacity (C) distributions are lognormal, as assumed in this case (see Section 3.1.1.2), and the probability of failure (Z) is defined as lognormal, Z = D/C then Melchers (1999), shows that the probability of failure, p_f, can be simplified to:

\[
p_f = \Phi(-\beta) = \Phi \left\{ -\frac{\ln(\mu_C/\mu_D)}{\left(\frac{1}{\sigma_C^2} + \frac{1}{\sigma_D^2}\right)^{1/2}} \right\} \tag{A.1}
\]

Where:

- \( \Phi(\cdot) \) = standard normal cumulative distribution function
- \( \beta \) = reliability coefficient
- \( \sigma = \frac{\sigma}{\mu} \)

Figure A.2 – Probabilistic seismic demand model for the preliminary test frame model
\( \mu, \sigma = \) mean and standard deviation, respectively, of the normally distributed natural logarithm of a lognormal random variable, e.g. \( \ln(X) \), where is \( X \) is lognormal

This equation A.2 is analogous to the fragility formulation used in this thesis, as described in Section 3.1.1.2, repeated below for convenience:

\[
P[D > C|IM] = \Phi \left[ \frac{\ln \left( \frac{S_d}{S_c} \right)}{\sqrt{\beta_{d,IM}^2 + \beta_c^2 + \beta_m^2}} \right]
\]  

(A.2)

If we ignore the epistemic uncertainty term, \( \beta_m \), and substitute the terms \( S_d \) and \( \beta_{d,IM} \) = median value and dispersion of the demand as a function of an intensity measure, respectively; as well as \( S_c \) and \( \beta_c \) = median value and dispersion of the capacity limit states, for the respective \( \mu_D, V_D, \mu_C, \) and \( V_C \) in equation A.1, after taking the terms inside the standard normal function, and rearranging, we get the following expression (note that the dispersion terms in equation A.3 below were kept as \( V_d \) and \( V_C \) to avoid confusion with the reliability coefficient, \( \beta \)):

\[
S_d \approx \exp \left[ -\beta \sqrt{V_C^2 + V_{d,IM}^2} + \ln(S_c) \right]
\]  

(A.3)

Thus, for a rough initial estimate of an appropriate SMA stroke length for retrofitting, given a desired target reliability, one can back-calculate the demand parameter from the as-built analysis by choosing a target reliability coefficient; and
median and dispersion values for a particular capacity limit state, C (e.g. the ‘Complete’
damage limit states, from HAZUS, [FEMA, 2003]). Per Melchers (1999), the choice of \( \beta \)
is not trivial but can be based semi-intuitively on values from existing structures or codes.
Typically, a range of 2.5-3.5 is selected for structures under most loading conditions,
with earthquake loading falling towards the lower end, which reflects a tradeoff between
the high consequence of failures with the high initial costs of construction to resist
typically high earthquake loads (Melchers, 1999). Thus, assuming \( \beta = 2.5 \), taking \( S_C = 5, \)
\( V_C = 0.47 \) (i.e. the median and dispersion for the ‘Complete’ damage limit state) and \( V_{d,IM} \)
= 0.36775 (per Figure A.2), yields \( S_d = 1.124\% \). Based on the geometry of a single bay in
the prototype structure, this target drift demand translates to a brace deformation, \( \delta \) for a
story displacement, \( \Delta \), as follows:

\[- \Delta = 12 \text{ ft} \times \left( \frac{1.124}{100} \right) = 1.62 \text{ in}, \text{ where 12 ft is the story height, and} \]
\[- \delta = \left[ (18 \text{ ft} + \Delta)^2 + (12 \text{ ft})^2 \right]^{1/2} - \left[ (18 \text{ ft})^2 + (12 \text{ ft})^2 \right]^{1/2} = 1.35 \text{ in}, \text{ where 18 ft is the}
\text{ bay width.} \]

Per Figure A.3, the usable strain in the SMA wires is 9\%, to avoid subjecting
other elements to the significant overstrength that can occur beyond 9\% SMA strain. Pre-
strain should be set to the yield point -in this case, 2.8\% strain- to avoid any potential
slack due to residual strain. In this case, assuming a 4 ft gage length, 1.35 in + 1.34 in
(which is a 2.8\% strain), equals 2.69 in, or 5.61\% strain. So, nonlinear time-history
analysis was run using the same preliminary model of the prototype test frame, described
above, with a brace component in each 1st story bay. This component was modeled using
two elastic beam elements at the ends (with the stiffness of steel) and a 4-ft truss element
using an SMA material model and the area determined through the ELF procedure described in Section A.1.

![Stress vs Strain Graph]

**Figure A.3 – Stress vs strain relationship of the SMA wires in quasi-static loading**

However, nonlinear time-history analysis using a 4 ft SMA gauge length showed that SMA strains surpassed the 9% strain target multiple times. So, the analyses were re-run with 5 ft and 6 ft SMA lengths to find a gauge length that met the target seismic demand (inter-story drifts below 1.124%) and the SMA strain limits (under 9% SMA strain). A 6ft gauge length satisfied these criteria.
A.3. References

American Society of Civil Engineers (ASCE) Committee. (2007). *Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06)*. American Society of Civil Engineers. Reston, VA: ASCE.


APPENDIX B. Calculations for the Design of Brace Steel Components

The design of all steel components is shown in this chapter. The provisions in *Specification for Structural Steel Buildings* (ANSI/AISC-360-10) and *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10) were used for the design and proportioning of all structural steel members that composed the brace retrofit. The design checks calculations are shown below. For the design basis, refer to Chapter 5.

B.1. Steel Brace Tube Design

The steel tubes were subjected mainly to tension and compression forces. ANSI/AISC 341-10 Chapter D provisions were used to satisfy tensile capacity requirements and Chapter E provisions were considered for compression design. The calculations from these two chapters are shown next. It is recommended that the reader follows this Appendix alongside a copy ASIC 360-10, which, as of May 2019, can be found here:


The assumed material parameters were:

- Material modulus: 29,000 ksi
- Material density: 490 pcf
- Steel grade: A500 Grade B (hollow structural sections)
  - Yield strength, $F_y = 46$ ksi; Ultimate strength, $F_u = 58$ ksi
The tube geometry:

\[ L = 26 \text{ ft} = 312 \text{ in}; r = 3.10 \text{ in}, \text{thus}\ L/r < 300 \]

\[ \phi_t P_n = F_y A_g = 361 \text{ kips} \]

\[ \phi_t = 0.9 \]

\[ \phi_t = 0.75 \]

\[ A_e = A_n U \]

\[ \text{This provision is not applicable because tensile rupture is controlled by provision D5. Details below.} \]
D4. Built up-members – not applicable

D5. Pin-connected members

- Tensile rupture, \( P_n = \phi \cdot F_u \cdot A_e = 44.9 \text{ kips (controls)} \), with effective net area, \( A_e = 2tb_e \), where:

\[
b_e = 2t + 0.63 \text{ in} = 1.38 \text{ in}, \text{ “but not more than the actual distance from the edge of the hole to the edge of the part, in the direction normal to the applied force”}, \text{ which in this case is } h/2 - D_{pin}/2, \text{ where } D_{pin} = 0.875 \text{ in and } h = 8 \text{ in}
\]

- Shear rupture, \( P_n = 0.6F_u \cdot A_{sf} = 106 \text{ kips} \), with shear area \( A_{sf} = 2t(a + d/2) = 4.08 \text{ in}^2 \) where:

  - \( a = \) shortest distance from edge of pin hole to edge of member, measured parallel to the direction of the force = 5.0 in (this was a very conservative initial estimate since dimensions were not final)
  - Diameter of pin, \( d = 0.875 \text{ in} \)
  - Thickness of plate, \( t = 3/8 \text{ in} \)

D6. Eyebars – not applicable

B.1.2. Design provisions, compression

E1. General provisions:

- Compressive strength resistance factor, \( \phi_c = 0.9 \)

- Per Table User Note E1.1, the applicable Chapter E section for HSS without slender elements is E3. Slenderness is determined through Section B4.1, as follows:
Flanges of rectangular HSS and boxes of uniform thickness:

- Width-to-thickness ratio, \( b/t < 1.12 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} \)
- \( b/t = 19.9 < 1.12 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} = 1.12 \left( \frac{29000}{46} \right)^{\frac{1}{2}} = 28.12 \)

Webs of rectangular HSS and boxes

- Width-to-thickness ratio, \( h/t < 2.42 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} \)
- \( h/t = 19.9 < 2.42 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} = 60.76 \)

E2. Effective length:

- Length factor, \( K = 1 \) (pin-pin support condition)
- Laterally unbraced length, \( L = 21.6 \) ft = \( (12^2 + 18^2)^{\frac{1}{2}} \)
- Radius of gyration, \( r = 3.10 \) in
- \( KL/r = 83.6 < 200 \) (limit for members in compression)

E3. Flexural buckling of members without slender elements (HSS 8x8x3/8 is not slender, per calculation shown in E1):

- \( P_n = F_{cr} A_g \) where:
  - \( F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y \) if \( KL/r < 4.71 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} \)
  - \( F_{cr} = 0.877 F_e \) otherwise
- Where \( F_e \) is the elastic buckling stress:
  - \( F_e = \frac{\pi^2 E}{\left( \frac{KL}{r} \right)^2} \)
  - \( F_e = 40.95 \) ksi
  - \( KL/r = 83.6 < 188 = 4.71 \left( \frac{E}{F_y} \right)^{\frac{1}{2}} \), thus
  - \( F_{cr} = \left[ 0.658 \frac{46}{40.95} \right] 46 = 28.75 \) ksi
- \( P_n = 28.75 \) ksi \( (10.4 \text{ in}^2) = 299 \) kips
B.1.3. Design provisions, welding (for doubler plate)

- J2. Welds, Item 2. Fillet welds
  
- Weld minimum thickness: 3/16 in for plates of thickness over ¼ in up to ½ in
  
- Weld max thickness: for edges of material ¼ in of more in thickness, not greater than the thickness of the material minus 1/16 in. In this case:
  
  \[
  \text{3/8 in - 1/16 in = 0.31 in}
  \]

- So, assume a ¼ in weld:

- Effective throat, \( t_e = 0.707 \times (0.25 \text{ in}) = 0.18 \text{ in} \)

- Filler metal strength, \( F_{EXX} = 70 \text{ ksi} \)

- Nominal weld strength, \( R_n \), is the minimum of:
  
  \[
  \begin{align*}
  R_{n1} &= \phi t_e (0.6 F_{EXX}) = 0.75 \times (0.18 \text{ in}) \times (0.6 \times 70 \text{ ksi}) = 5.57 \text{ k/in} \\
  R_{n2} &= \phi t_w (0.6 F_u) = 0.75 \times (3/8 \text{ in}) \times (0.6 \times 58 \text{ ksi}) = 9.79 \text{ k/in} \\
  R_{n3} &= t_w (0.6 F_y) = (3/8 \text{ in}) \times (0.6 \times 46 \text{ ksi}) = 10.35 \text{ k/in}
  \end{align*}
  \]

- Required weld length = 30 kip / \( R_n = 5.4 \text{ in} \)

B.2. Steel Attachment Piece

B.2.1. Attachment Piece Dimensions

The attachment piece is a pin connected element, always in tension, so the dimensions were based on AISC Section D5. Pin-connected members (in tension), as follows:

- D5. Pin-connected members:
  
  \[
  \begin{align*}
  \text{Steel plates, } F_y \text{ and } F_u: & \text{ 50 ksi and 65 ksi, respectively}
  \end{align*}
  \]
Plate thickness, $t = 1$ in (thickness chosen to minimize elastic deformations)

Tensile rupture, $P_n = \phi_t F_u A_e = 256.4$ kip, with effective net area, $A_e = 2tb_e$, where $b_e = 2t + 0.63$ in $= 2.63$ in, “but not more than the actual distance from the edge of the hole to the edge of the part, in the direction normal to the applied force”, which in this case is $h/2 - D_{pin}/2$, where $D_{pin} = 1.0$ in and $h$ $= 6.5$ in

Shear rupture, $P_n = 0.6F_u A_{sf} = 312$ kips, with shear area $A_{sf} = 2t(a + d/2)$ $= 8.0$ in$^2$ where:

- $a$ = shortest distance from edge of pin hole to edge of member, measured parallel to the direction of the force $= 3.5$ in
- Diameter of pin, $d = 1.0$ in
- Thickness of plate, $t = 1$ in
B.2.2. Attachment Piece Welding

- Weld minimum fitness = 5/16 in, for plates over ¾ in thickness
- Effective throat, \( t_e = 0.707 \times \frac{5}{16} \text{ in} = 0.22 \text{ in} \)
- Steel plates, \( F_y \) and \( F_u \): 50 ksi and 65 ksi, respectively
- Attachment piece plate width, \( t = 1 \text{ in} \)
- Nominal weld strength, \( R_n \), is the minimum of:
  - \( R_{n1} = \phi \ t_e \times \left( 0.6 \times F_{EYX} \right) = 0.75 \times (0.22 \text{ in}) \times (0.6 \times 70 \text{ ksi}) = 6.93 \text{ k/in} \)
  - \( R_{n2} = \phi \ t \times \left( 0.6 \times F_u \right) = 0.75 \times (1 \text{ in}) \times (0.6 \times 65 \text{ ksi}) = 29.25 \text{ k/in} \)
\[ R_{n3} = t (0.6 F_y) = (1 \text{ in})(0.6 \text{ 50 ksi}) = 30.0 \text{ k/in} \]

- Required weld length = 30 kip / \( R_n = 4.32 \text{ in} \)

### B.3. End Plate Connection

The end plates were meant to provide a continuous connection between 1) brace tube elements, for ease of constructability, and 2) brace tube and knife plate, in order to connect to the gusset plate, which is the main connection to transfer forces from the RC frame to the tubes. Design of HSS member connections is governed by AISC 360-10 Chapter K provisions. For this design, the design example K.10 from the AISC Design Examples Manual (AISC, 2017) was followed. As with prior sections in this appendix, it is recommended that the reader follows along with a copy ASIC 360-10 as well as a copy of the Design Examples Manual, which may be found here: https://www.aisc.org/globalassets/aisc/manual/v15.0-design-examples/aisc-design-examples-v15.0.pdf

- End plate material properties: ASTM A36, \( F_y \) and \( F_u \) = 38 ksi and 58 ksi, respectively.

- Preliminary size of 4-bolt group: \( r_{ut} = P_u/n = 60 \text{ kip}/4 = 15 \text{ kip} \)
  - \( 3/4 \)-in diameter Group A bolts, \( \phi r_n = 29.8 \text{ kips} \) per AISC Design Manual Table 7-2

- End-plate thickness considering prying action:
  - \( a' = (a + d_b/2) \leq (1.25b + d_b/2) = 1.25 + (0.75/2) = 1.625 \text{ in} < 1.25 (1.25) \)
  - \( + 0.75/2 = 1.938 \text{ in} \), OK (Steel Design Manual Eq. 9-23)
b' = b - db/2 = 1.25 - 0.75/2 = 0.875 in (Steel Design Manual Eq. 9-18)

ρ = b' / a' = 0.451 (Steel Design Manual Eq. 9-22)

Tributary length per bolt:

p = full plate width/# of bolts per side = 14 in / 1 = 14in

hole diameter, d' = 13/16 in

δ = 1 - d'/p = 1 - (13/16 in)/ 14 in = 0.942 (Steel Design Manual Eq. 9-20)

β = 1/ ρ (ϕrn / rut - 1) = 1/0.538 (29.8/20 - 1) = 0.96 (Manual Eq. 9-21)

α' = 1 / δ [β / (1 - β)] = 1 / 0.942 [0.96 / (1-0.96)] = 25.48 > 1, thus α' = 1

Use t = 0.5 in so further prying action checks are not necessary (also, for experimental purposes, it was agreed that it was best to avoid prying action)

• Required weld sizes:

F_{EXX} = 70ksi

φ R_n = φ F_{nw} A_{we}, where φ = 0.75, and

- F_{nw} = 0.6 F_{EXX} [1 + 0.5 \sin^{1.5} (90)] = 63 ksi

- A_{we} = 0.707 \left(\frac{D}{16}\right) l_{we}, where D is weld size in 16ths of an inch, and l_{we} is the weld length.

A 3/16 in weld with at least an 10 in length satisfies the required strength.

B.4. Gusset Plate Connection
The gusset plate design followed the provisions of AISC 360-10, and was guided by the AISC Design Examples Manual. For the gusset plate design, the connection geometry and forces are needed. For this particular connection, the following structural elements were used:

- Beam: W8x40
- Column: W8x67
- Brace: HSS 9x9x3/8

For an initial estimate, a gusset plate thickness, $t = 5/8''$ was chosen. An initial estimate of the compressive capacity can be obtained from AISC 360-10 Table 4-22, as follows:

- The radius of gyration, $r = (I/A)^{\frac{1}{2}}$, where $I =$ moment of inertia, and $A =$ area. For a rectangular section, this simplifies to $r = t / (12)^{\frac{1}{2}} = 0.18$in (for the assumed $t$)
- For an assumed ‘fixed-fixed’ support condition, the effective length factor, $K = 0.65$
- The gusset plate length, $L = 10.4375$ in
- Thus, $KL/r = 37.6$. For this value, from AISC 360-10 Table 4-22, $\phi F_{cr} = 40.5$ ksi, and $R_n = \phi F_{cr} t = 3.17 = 80 \textbf{kip} < 60 \text{ kip}$ (peak demand from brace)

Check the connection interface forces:

- Beam centerline eccentricity, $e_b = d_b/2 = 4.125$ in
- Column centerline eccentricity, $e_c = d_c/2 = 4.5$ in
- Interface force angle, $\theta = 57^\circ$
• $K_I = e_c \tan \theta - e_c = 1.852$ in

• $\alpha = \alpha_b = 10-19/32”$

• $\beta_b = 5-11/16”$

• $\beta = (\alpha - K_I) / \tan \theta = 5.677$ in

• $r_c = [(\alpha + e_c)^2 + (\beta + e_b)^2]^{\frac{1}{2}}$

• Interface forces (use to design welds):
  - $V_c = P_u (\beta / r_c) = 25$ kip
  - $H_c = P_u (e_c / r_c) = 20$ kip
  - $H_b = P_u (\alpha / r_c) = 47$ kip
  - $V_b = P_u (e_b / r_c) = 18.3$ kip
  - $M_c = H_c (\beta - \beta_b) = 0.21$ kip-in (negligible)

• Check beam local limit states
  - Gusset-to-beam welds:
    - $f_{vb} = H_b / 11” = 4.281$ kip/in
    - $f_{ab} = V_b / 11” = 1.667$ kip/in
    - $f_{b-avg} = (f_{vb}^2 + f_{ab}^2)^{\frac{1}{2}} = 4.59$ kip/in
    - $f_{des-b} = 1.25 f_{b-avg} = 5.74$ kip/in
    - $D_{weld} = f_{des-b} / (2 * 1.392$ kip/in$) = 2.06$, i.e. a $3/16$” fillet weld satisfies strength requirements, however, the minimum thickness for a $5/8”$ plate is $1/4”$
  - Check plate thickness satisfies welding limit states
    - $t_{min} = 6.19 D_{weld} / F_u = 0.196” < 5/8”$, so OK
  - For beam local web yielding: $d_b < \alpha$, thus:
- $R_n = (k_b + l_g) F_y t_w = 283 \text{ kip} > Pu, \text{ so OK}$

  o Check web crippling, for $\alpha > d_b/2$, from AISC 360-10 Table 9-4:
    - $R_n = 2 [(\phi R_3 + l_g \phi R_4)] = 356 \text{ kip}, \text{ so OK}$

- Check column local limit states
  
  o Gusset-to-column welds:
    - $f_{vc} = H_c / 9" = 2.23 \text{ kip/in}$
    - $f_{ac} = V_c / 9" = 2.804 \text{ kip/in}$
    - $f_{bc} = M_c / S_w = 0.016 \text{ kip/in}$
    - $f_{c-avg} = \sqrt{f_{vc}^2 + (f_{ac} + f_{bc})^2} = 3.57 \text{ kip/in}$
    - $f_{des-c} = 1.25 f_{c-avg} = 4.457 \text{ kip/in}$
    - $D_{weld} = f_{des-c} / (2 * 1.392 \text{ kip/in}) = 1.6$, i.e. a 2/16” fillet weld satisfies strength requirements, however, the minimum thickness for a 5/8” plate is ¼”

  o Check plate thickness satisfies welding limit states
    - $t_{min} = 6.19 D_{weld} / F_u = 0.152" < 5/8”, \text{ so OK}$

  o For column local web yielding: $\beta < d$, thus:
    - $R_n = (2.5 k_c + l_g) F_y t_w = 351 \text{ kip} > Pu, \text{ so OK}$

- Check web crippling, for $\beta > d_b/2$, from AISC 360-10 Table 9-4:
  
  o $R_n = 2 [(\phi R_3 + l_g \phi R_4)] = 730 \text{ kip}, \text{ so OK}$
APPENDIX C. Calculations for Concrete Post-Installed Anchors

Simpson Strong Tie adhesive anchors (AT-XP) were used to connect the retrofit device to the existing reinforced concrete frame. For design, the Simpson Anchor Designer software was used, with design parameters based on ACI 318-11 [reference: ACI Committee 318. 2011. Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11). Detroit, MI: American Concrete Institute]. The frame column geometry and peak shear and axial forces, as obtained from nonlinear analysis at the interface connection elements between steel adapters and concrete frame, were used as input for the Anchor Designer. The Anchor Designer report output is shown below.
1. Project Information
   - Customer company:
   - Customer contact name:
   - Customer e-mail:
   - Comment:

2. Input Data & Anchor Parameters
   - General
     - Design method: ACI 318-11
     - Units: Imperial units
   - Anchor Information:
     - Anchor type: Bonded anchor
     - Material: F1564 Grade 35
     - Diameter (inch): 0.500
     - Effective Embedment depth, h_e (inch): 5.750
     - Code report: IPM-O UES ER-263
     - Anchor category: Yes
     - Number of anchors: 1
     - c_e (in): 15.93
     - S_n (in): 3.00
   - Location:
   - Fastening description:
   - Load and Geometry
     - Load factor source: ACI 318 Section 9.2
     - Load combination: not set
     - Seismic design: Yes
     - Anchors subjected to sustained tension: No
     - Ductility section for shear: D.3.3.4.2 not applicable
     - Ductility section for shear: D.3.3.5.2 not applicable
     - Dv factor: not set
     - Apply entire shear load at front row: No
     - Anchors only resisting wind and/or seismic loads: Yes
     - Base Material
       - Concrete: Normal-weight
       - Concrete thickness, h (in): 12.00
       - State: Cracked
       - Compressive strength, f_c (psi): 3500
       - W_c: 1.2
       - Reinforcement condition: B tension, A shear
       - Supplemental reinforcement: Not applicable
       - Do not evaluate concrete breakout in tension: No
       - Do not evaluate concrete breakout in shear: No
       - Hole condition: Dry concrete
       - Inspection: Continuous
       - Temperature range: 1
       - Ignore load requirement: Not applicable
       - Build-up grout pad: No
     - Base Plate
       - Length x Width x Thickness (inch): 6.00 x 12.00 x 0.50

<Figure 1>
Recommended Anchor
Anchor Name: AT-XP0 - AT-XP w/ 1/2"@ F1554 Gr: 36
Code Report Listing: IAPMO UES ER-203

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5996 W. Las Positas Boulevard Pleasanton, CA 94566 Phone: 925.550.8000 Fax: 925.847.3871 www.strongtie.com
### 3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_a$ (lb)</th>
<th>Shear load $x$, $V_{ax}$ (lb)</th>
<th>Shear load $y$, $V_{ay}$ (lb)</th>
<th>Shear load combined, $V(V_{ax})^2 + (V_{ay})^2$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>2</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>3</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>4</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>5</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>6</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>7</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
<tr>
<td>8</td>
<td>339.3</td>
<td>0.0</td>
<td>1607.1</td>
<td>1607.1</td>
</tr>
</tbody>
</table>

**Sum:**
- $2714.0$ on $x$, $E_0$ (inch): $0.00$
- $12857.0$ on $y$, $E_y$ (inch): $0.00$

Maximum concrete compression stress ($f_{cc}$) $0$,

Resultant compression force ($V_c$): $0$

Eccentricity of resultant tension forces in $x$-axis, $e_{ax}$ (inch): $0.00$

Eccentricity of resultant tension forces in $y$-axis, $e_{ay}$ (inch): $0.00$

Eccentricity of resultant shear forces in $x$-axis, $e_{ax}$ (inch): $0.00$

Eccentricity of resultant shear forces in $y$-axis, $e_{ay}$ (inch): $0.00$

### 4. Steel Strength of Anchor in Tension (Sec. D.5.1)

$$N_a = 0.75f_{y}d_{a}A_{sa}(\text{Eq. D-6})$$

### 5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$$N_{sl} = 0.75f_{c}d_{i}A_{sa}g_{sa}(\text{Eq. D-6})$$

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

**Simpson Strong-Tie Company Inc.**
9995 W. Las Positas Boulevard, Pleasanton, CA 94566
Phone: 925.955.9000 Fax: 925.947.5881 www.strongtie.com
8. Steel Strength of Anchor in Shear (Sec. D.6.1)

<table>
<thead>
<tr>
<th>$V_{u}$ (lb)</th>
<th>$\phi_{steel}$</th>
<th>$\phi$</th>
<th>$V_{steel}$</th>
<th>$V_{u}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4940</td>
<td>1.0</td>
<td>0.65</td>
<td>0.65</td>
<td>2720</td>
</tr>
</tbody>
</table>

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)
Shear parallel to edge in y-direction:

$V_{c} = \min(f_{c}V_{u}, f_{e}V_{e}, V_{c, u}, V_{c, e}, 1.200 \times 1.200 \times 1.200 \times 1.200 \times 1.200)$ (Eq. D-33 & Eq. D-34)

<table>
<thead>
<tr>
<th>$A_{c}$ (in$^2$)</th>
<th>$A_{bc}$ (in$^2$)</th>
<th>$V_{c, u}$</th>
<th>$V_{c, e}$</th>
<th>$V_{c, u}$</th>
<th>$V_{c, e}$</th>
<th>$V_{c, u}$</th>
<th>$V_{c, e}$</th>
<th>$V_{u}$ (lb)</th>
<th>$\phi$</th>
<th>$\mu V_{c, u}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>128.0</td>
<td>72.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.200</td>
<td>1.200</td>
<td>1.200</td>
<td>1.200</td>
<td>3551</td>
<td>6.75</td>
<td>11185</td>
</tr>
</tbody>
</table>

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{c, p} = \phi \min(A_{bc}V_{c, p}, A_{bc}V_{c, p}) = \phi \min(A_{bc}V_{c, p}, A_{bc}V_{c, p})$ (Eq. D-41)

<table>
<thead>
<tr>
<th>$A_{bc}$ (in$^2$)</th>
<th>$A_{bc}$ (in$^2$)</th>
<th>$V_{bc, u}$</th>
<th>$V_{bc, e}$</th>
<th>$V_{bc, u}$</th>
<th>$V_{bc, e}$</th>
<th>$V_{bc, u}$</th>
<th>$V_{bc, e}$</th>
<th>$N_{b}$ (lb)</th>
<th>$N_{b}$ (lb)</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.02</td>
<td>144.56</td>
<td>0.900</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>7946</td>
<td>12476</td>
<td></td>
</tr>
</tbody>
</table>

$\phi V_{c, p}$ (lb)

17245

11. Interaction of Tensile and Shear Forces (Sec. D.7)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Factor Load, $N_{a}$ (lb)</th>
<th>Design Strength, $\phi N_{a}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>339</td>
<td>6176</td>
<td>0.05</td>
<td>Pass</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>2714</td>
<td>6065</td>
<td>0.45</td>
<td>Pass (Governs)</td>
</tr>
<tr>
<td>Adhesive</td>
<td>2714</td>
<td>6082</td>
<td>0.45</td>
<td>Pass</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear</th>
<th>Factor Load, $V_{s}$ (lb)</th>
<th>Design Strength, $\phi V_{s}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>1607</td>
<td>2729</td>
<td>0.59</td>
<td>Pass</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>6439</td>
<td>11195</td>
<td>0.57</td>
<td>Pass</td>
</tr>
<tr>
<td>Pryout</td>
<td>12857</td>
<td>17245</td>
<td>0.75</td>
<td>Pass (Governs)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Interaction check</th>
<th>$N_{a}/\phi N_{a}$</th>
<th>$V_{bc}/\phi V_{bc}$</th>
<th>Combined Ratio</th>
<th>Permissible</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sec. D.7.3</td>
<td>0.45</td>
<td>0.75</td>
<td>119.7 %</td>
<td>1.2</td>
<td>Pass</td>
</tr>
</tbody>
</table>

AT-XP w/ 1/2"Ø F1554 Gr. 36 with hcf = 5.750 inch meets the selected design criteria.

12. Warnings

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5995 W. Las Positas Boulevard Pleasanton, CA 94566 Phone: 925.565.9000 Fax: 925.847.3971 www.strongtie.com
- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.