DEVELOPMENT OF SEISMIC INFILL WALL ISOLATOR SUBFRAME (SIWIS) SYSTEM

A Thesis in

Architectural Engineering

by

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ABSTRACT

The Seismic Infill Wall Isolator Subframe (SIWIS) system is developed in this study for use in building frames with masonry infill walls in order to prevent damage to columns or infill walls and to minimize life-safety hazards during potentially damaging earthquakes. Because of the conventional tight-fit construction within structure frames, infill walls participate in resisting wind and seismically induced loads. Although beneficial during wind loads and minor earthquakes, the lateral load resistance of infill walls during strong events can damage the wall or the frame because the infill wall is usually treated as a non-load bearing wall, and, thus, is not designed to carry in-plane loads. Complete isolation of infill walls by separation gaps as an alternative construction suffers from lack of convenient and satisfactory solutions for fire and acoustic protection of the gap and the out-of-plane stability of the wall. The SIWIS system, which consists of two vertical and one horizontal sandwiched light-gauge steel studs with SIWIS elements in the vertical members, is designed to allow infill wall-frame interaction under wind loading and minor-to-moderate earthquakes for reduced building drift, but to disengage them under severe damaging events. The SIWIS system acts as a “sacrificial” component or a “structural fuse” to protect the infill wall and frame from failure.

An experimental testing program planned and carried out tested the concept of the SIWIS system. The experimental program mainly included a series of tests on three different designs for fuse element (centerpiece) including concrete disk, steel disk, and lumber disk followed by a series of in-plane static pushover tests on a scaled two-bay three-story steel frame in three forms of bare frame, rigid frame, and pinned frame equipped with an SIWIS device. Generalized nonlinear finite element modeling schemes were developed for infilled steel frames with and without SIWIS system. Validation of modeling schemes was accomplished by comparing the experimental observations with the numerical predictions for: (1) a previously tested and studied single-bay single-story infilled steel frame selected from the literature; and (2) the tested two-bay three-story steel frame. The analytical and experimental results show that the concept of the proposed system works as a “seismic isolation” system for infill walls by utilizing, initially, the beneficial stiffness and strength effects of the infill wall up to a predefined point, but, ultimately isolating the infill wall from the frame for their safety. Practical design approaches were proposed and applied to three examples including: low rise, mid-rise, and high-rise buildings in high seismic and wind zones to demonstrate the performance of the proposed system.
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Chapter 1

INTRODUCTION

1.1 Introduction

In steel and concrete moment frame construction, infilling some of the bays with walls made of masonry units is a common practice in many countries. Masonry infill walls are usually specified by architects as interior or exterior partitions in such a way that they do not contribute to the vertical gravity load-bearing capacity of the structure. However, because of their conventional tight fit construction within structure frames and their large in-plane stiffness, infill walls participate in resisting wind and seismically induced loads. Although beneficial during wind loads and minor earthquakes, the in-plane load resistance of infill walls during strong events can damage the wall or the frame because the infill wall is usually treated as a non-structural element and thus not designed to carry in-plane loads. Soft stories and short columns are two most commonly observed structural damages caused by infill walls in earthquakes.

In this dissertation, a Seismic Infill Wall Isolator Subframe (SIWIS) system is introduced in detail and several alternatives are developed for use in building frames with masonry infill walls in order to prevent damage to columns or infill walls and minimize life-safety hazards during potentially damaging earthquakes. The solution proposed is a logical approach that can be classified as a seismic isolation solution. The optimum solution would be to try to use (to a certain degree) the beneficial effects of strength and stiffness of the infill wall to reduce the story drift during low to moderate seismic events. However, during strong shakings, the sacrificial subframe can be designed to always isolate the infill wall from the frame in order to prevent failure of the wall (including cracking) as well as damage to the frame. The SIWIS system, which consists of two vertical and one horizontal sandwiched light-gauge steel plates with “fuse” elements in the vertical members, is designed to allow infill wall-frame interaction under wind loading and minor to moderate earthquakes for reduced building drift but to disengage them under damaging events. The SIWIS system acts as a “sacrificial” element just like a “fuse” to save the infill wall and the infilled frame from failure.
1.2 Problem Statement

Masonry infill walls, which are usually specified by architects, are considered to be non-structural elements in structural analysis and design procedures. However, depending on their construction details, they can significantly influence the seismic behavior of the structure such as strength, stiffness, and ductility during seismic events. In other words, under seismic in-plane loads, the infill walls can interact with the confining frames in resisting in-plane forces (e.g., Drysdale et al. 1999, Paulay and Priestley 1992). In seismic areas, ignoring this interaction is not always the safest, since this action can dramatically change the stiffness and the dynamic characteristics of the structure, and hence, its response to seismic loads.

For this reason, two methods of construction have been proposed and considered in the literature for infill walls (e.g., Tomazevic 1999, Paulay and Priestley 1992, and Dowrick 1987). The first method is to integrate the infill wall with the structural frame and basically turn it into a shear wall (Tight fit infill wall in Figure 1.1). The second approach is to isolate the infill wall from the structural frame by leaving gaps between them (Isolated infill wall in Figure 1.1).

Figure 1.1: Tight fit and isolated masonry infill walls confined with structural frame

In the case of tightly fitted construction, normally the infill walls are not structurally connected to the frame, but instead there is bearing interaction. Traditionally, such infill walls, which are often planned in the architectural layout of a building, have not been designed as structural in-plane load carrying elements. The interaction between infill walls and confining frames can have a significant effect on the global seismic response of the entire structure as well as the response of the individual members. The presence of infill walls increases the lateral
stiffness of the structure, decreases its fundamental period, and as a result leads to larger shear forces, primarily during the elastic response phase. In the case of tight fit construction, depending on the details of their construction (partial vs. complete infill), infill walls interaction with the confining frames could lead to the possibility of premature column failure as a result of short column effect or to increased levels of unaccounted ductility demand in columns. Furthermore, an irregular and even sometimes regular arrangement of infill walls can modify the in-plane stiffness distribution in plan and elevation, which can result in increased torsional effects and soft story mechanism. As masonry infill walls are usually less deformable than the structural frame, with increased seismically induced story drift, they first experience damage in the form of cracks and separation from the structural frame. The development of cracks in infill walls will substantially increase the seismic energy dissipation capacity of the building. But on the other hand, this behavior will reduce the lateral stiffness and strength of the infill walls resulting in redistribution of in-plane loads from the infill walls to the structural system. The exact interaction mechanisms and influences of infill walls are very complicated and further research is underway. According to Fardis et al. (1999), the current knowledge about the seismic response of masonry infilled structures is inadequate to develop reliable and detailed rules to account for the presence of infill walls in the seismic assessment of existing structures or in the design of new ones.

An alternative solution is to separate the infill walls from the structure by leaving gaps between them. The structural movements in such cases should be estimated to determine suitable gap sizes. It is very important to ensure that the gaps will not be incidentally filled with mortar or other stiff materials during construction procedure. According to Dowrick (1987), two main performance problems with this approach need to be solved. The first difficulty is that providing effective and sufficient out-of-plane stability for the infill walls can require inconvenient details. The second problem arises when an attempt is made to fulfill the acoustic and fire insulation requirements of the separation gap. In this dissertation, an alternative method is introduced that can be classified as a seismic isolation solution with the use of a sacrificial subframe and fuse element.

1.3 Research Objectives

The main objective of this dissertation is to develop and test the concept of the proposed SIWIS system. This dissertation aims to lay the groundwork and direction for further development of SIWIS system with the ultimate goal of its being manufactured and implemented
in the building industry. The importance of the research is highlighted by reviewing the poor seismic performances of infill walls in past earthquakes.

It is intended to develop reliable and generalized nonlinear finite element modeling schemes for the masonry infilled steel frames with and without SIWIS system. The modeling schemes are initially validated through a simulation procedure of a previously tested and studied single-bay single-story infilled steel frame selected from the literature. After preliminary validation of general modeling schemes, the use of an SIWIS system in a typical two-bay three-story frame is analytically studied. A series of parametric analyses are conducted on the two cases including single-bay single-story steel frame and two-bay three-story steel frame with the objective of better understanding the behavior of this system and the influences of different parameters and conditions. Besides analytical verification, it is intended to experimentally validate and test the concept of the proposed SIWIS system as well. To this end, an extensive experimental program has been planned and carried out. The test program included five tests on three different designs for SIWIS element followed by series of static tests on a scaled two-bay three-story steel frame with different configurations. It is intended to confirm the validation of the developed finite element models by simulating all conducted tests on the two-bay three-story steel frame. Finally, this study aims to develop inclusive and practical design approaches for the new system and its components in a multi-bay multi-story building. The performance and advantages of the proposed system are demonstrated in three actual examples including a low-rise, a mid-rise, and a high-rise building.

1.4 Organization of the Dissertation

The research program includes development of the concept, numerical modeling, experimental testing, and finally proposing design approaches for the SIWIS system as an alternative for construction of infill walls. In Chapter 2, different solutions considered in the past for infill walls are discussed and their problems are indicated. The main seismic problems of infill walls are briefly reviewed and their past earthquake performances are examined as evidence. A literature review on in-plane strength of masonry walls including experimental studies, analytical methods, and empirical equations is presented. For the purpose of this dissertation, the use of empirical equations for prediction of the in-plane strength of masonry walls is comprehensively reviewed. The finite element modeling of masonry infilled steel frames developed by other researchers are reviewed and briefly explained.
Chapter 3 develops the details of an innovative SIWIS system to solve the mentioned problems and to improve the seismic performances of infilled frames. For the centerpiece of the proposed system, several designs are suggested including; compression disk, tension element, friction device, and jagging and releasing element. The advantages and disadvantages of different options are discussed and compared.

In Chapter 4, the finite element method is used to predict and simulate the response of infilled steel frames with and without SIWIS system. First, two dimensional nonlinear finite element models are developed for a single-bay single-story steel frame in the form of bare frame and infilled frame using the ANSYS6.1 (2002) program. Preliminary validation of these models is achieved using existing experimental results of the selected case study. With the use of developed models and by simulating the behavior of fuse element, a two dimensional nonlinear finite element modeling scheme is developed for infilled steel frame with SIWIS system. Besides the single-bay single-story steel frame, a two-bay three-story steel frame is also modeled. The two models are subjected to series of in-plane loading under pushover and cyclic loading patterns.

Chapter 5 presents the experimental program of the study. In this chapter, a scaled two-bay three-story steel frame model is subjected to static pushover in-plane loading in a series of tests with different bracing conditions including: bare frame, fully and half braced frame with SIWIS elements, and pinned frame with SIWIS elements. The results of tests on single brick walls are used to determine the required capacities for the SIWIS elements. A rod pulling test is planned and conducted to provide the information needed to design the diagonal bracings of the test frame. In this chapter, the performances of disks for SIWIS element using three different disk material including; concrete disk, steel disk, and lumber disk is comprehensively studied through five series of tests. The experimental observations are also used for final validation of the developed finite element modeling schemes.

In Chapter 6, the main design considerations for the masonry infill wall, SIWIS element, and structural frame are addressed. Practical design approaches for a multi-bay multi-story building equipped with SIWIS system are developed. Then, the developed design approaches are implemented into three typical buildings including a low-rise (4-story), a mid-rise (8-story), and a high-rise (20-story) buildings in high seismic and wind zones to illustrate the design procedure and to demonstrate the performances of SIWIS system in these three buildings.

Chapter 7 summarizes the main outcome of the experimental and numerical investigations carried out in this research. Important conclusions are highlighted and core needs for future studies are identified.
Chapter 2

LITERATURE REVIEW

2.1 Practice of Masonry Infill Walls

Application of masonry infill walls in steel and concrete structures is a broad and common practice in many countries. As mentioned in Section 1.2, two construction approaches including infill wall integration (tight-fit construction) and infill wall isolation have been proposed and considered in the literature for infill walls (e.g., Tomazevic 1999 and Dowrick 1987) (Figure 1.1).

Dowrick (1987) indicated that limited data exist on the detailing of masonry construction in the case of tight-fit construction, and he listed a few recommendations as general guidelines including: (1) there should be no gap between the infill wall and the frame to prevent possible pounding damages; (2) through structural connections to the frame at the top of the wall panel, out-of-plane stability should be provided; and (3) shear failure of the masonry infill wall should be eliminated by either using stronger masonry wall or reinforcing it. According to Dowrick (1987), due to lack of an accepted method for calculating necessary reinforcement, the infill wall panel may be considered as a normal masonry shear wall for this purpose. In this form of construction, because the infill walls are usually erected after the upper beam has been constructed, the placing of full-height vertical reinforcements can be troublesome. Alternatively, external reinforcing metal sheets or Fiber Reinforced Polymer (FRP) layers bonded to the sides of the wall can be used. As shown in Figure 2.1, sometimes, in a controlled local area (usually end strips), lightweight infill walls may be allowed to fail to prevent complete failure and minimize earthquake repairs (Dowrick 1987).

The use of cover plates or flexible sealants can provide average insulation against sound and fire. According to Dowrick (1987), finding a proper material that is able to maintain and reserve its sound and fire insulation as well as its flexible properties for a long time is very difficult. Dowrick (1987) mentioned a few materials that may seem suitable for use in the separation gap and indicated their deficiencies. Mono-Lasto-Meric, an acrylic based sealant, is a
long lasting soft material, but is not appropriate for gap sizes over 20 mm. Foamed polyurethane is another example of permanently flexible material, which can also provide moderate sound insulation, but may have low fire resistance. Declon-156 a polyester/polyurethane foam, which bubbles up in case of fire, can be a good choice for fire-insulation. Figures 2.2 to 2.5 show some details used for isolated masonry infill walls.

Figure 2.1: Details of light infill walls to allow damage to limited end strips in earthquake (Dowrick 1987)

Figure 2.2: Details of light infill walls for small seismic movements (i.e., suitable for stiff structural frame or minor earthquakes) (Dowrick 1987)
Figure 2.3: Top connection of infill walls for out-of-plane stability (Dowrick 1987)

Figure 2.4: Isolated infill wall with separation gap (Dowrick 1987)

Figure 2.5: Plastering detail for protecting separation gap between infill wall and frame (Dowrick 1987)
In an army technical manual published by United States Army Corps of Engineers (TM 5-809-10: Seismic Design for Buildings 1992), non-structural masonry walls are required to be isolated where they are unable to resist in-plane lateral forces, but are required to be designed for out-of-plane forces. Proposed typical details for isolating masonry walls from the structure are shown in Figure 2.6.

Naeim (2001) recommends isolating heavy masonry walls, which are acceleration and deformation sensitive elements, from the frame with a sufficiently sized gap. The top connections of these walls to the frame or the roof structure should provide effective out-of-plane stability, but accommodate in-plane movements. Figures 2.7 and 2.8 show some details for top connection for heavy and light isolated masonry wall partitions, respectively.

Figure 2.6: Typical details for isolating masonry walls from structure (TM 5-809-10 1992)
Figure 2.7: Top connection details for isolated heavy masonry walls (Naeim 2001)

Figure 2.8: Connection details for isolated light masonry walls (Naeim 2001)
As proposed by Mazzolani and Piluso (1996) and shown in Figure 2.9, other similar top connections between masonry infill wall and structural frame using a mechanism able to provide suitable in-plane flexibility and out-of-plane stability can be considered.

![Top connection systems for isolated infill walls](image)

Figure 2.9: Top connection systems for isolated infill walls (Mazzolani and Piluso 1996)

According to Dowrick (1987), neither of the two approaches discussed (tight fit and isolated infill walls) has been very successful and satisfactory so far. Despite the fact that the importance of the infill wall’s effects on the performance of entire structure is well received by many researchers and practitioners, most building codes still do not have strict and clear provisions for seismic design of infill walls. Modern seismic codes disregard or very briefly refer to the effects of infill walls and there is extended room for their improvements (e.g., Negro and Verzeletti 1996, Fardis and Calvi 1995).

The requirements and recommendations of an American and a European building code for infill walls are reviewed next.

MSJC (2002) specification requires that masonry walls should not be connected to structural frames unless their connections and masonry walls are designed and detailed to resist interacting forces and to accommodate the design story drift (typically ranging from 1% to 2% of the story height). There are no isolation details in the specification and it only requires considering sufficient clearance between the frame and masonry walls and providing flexible or slip type connections to allow adequate in-plane movements. The separation joint should provide enough out-of-plane stability as well. If masonry infill walls are not intended to behave as shear walls, then the conventional practice for infill walls is to use an unreinforced masonry infill wall
for low seismic regions (Seismic Design Category (SDC) A and B). For moderate to high seismic regions (e.g., SDC C and D), according to MSJC (2002), infill walls should be isolated from the structure and at the same time horizontal or vertical reinforcement should be used for infill walls in SDC C, but both should be used for walls in SDC D.

European code EC8 (1998) recommends the seismic analysis of the masonry infilled frames of ductility class “M” or “H” (three ductility classes of concrete structures are defined by EC8 including L, M, and H, based on degree of ability of the structure to display ductile behavior) to be performed on bare structural system (i.e., ignoring the stiffening effects of the infill walls, but including their weight), while demanding additional criteria for the influence of the infill walls, which primarily causes remarkable increases in the effects of seismic action. These criteria include; (1) decrease the fundamental period of the bare structure, which results in an increase in seismic forces, (2) double the accidental eccentricity even for uniformly distributed infill walls in the plan, and (3) increase the seismic action effects by a factor varying from 1.1 to 1.7 in the case of infill walls’ irregularities in elevation. According to Penelis and Kappos (1997), the restrictive regulations of EC8 lead the designer to choose separation of the masonry infill wall and structural frame rather than integrating them.

2.2 Past Earthquake Performances of Masonry Infill Walls

A review of performances of masonry infill walls in past earthquakes confirms that when they are tightly fitted within the structural frames, which is often the case, they are very vulnerable, and damage to these walls results in significant economic loss and human casualties even if the structural frame is only lightly damaged. To illustrate this, examples of most typical damages to infilled frame systems are presented in Appendix A in a series of photos mostly from recent earthquakes.

Cracking of infill walls, which is the most common form of non-structural damage in minor to moderate earthquakes, can demand expensive repair cost as well as temporary shutdown of normal operation of the building. Stronger earthquakes can lead to total collapse of the infill walls with the hazard of falling debris into the streets and neighboring buildings. In the Nicaragua 1972 earthquake, a large number of the total 5000 casualties were associated with falling masonry walls and roofs (NISEE 1997).

In strong shakings, the two most common and severe damages to the structure caused by infill walls are soft story and short column mechanisms. In most residential and commercial
buildings, the first story (ground) needs to have open bays due to the need for parking spaces or stores. However, the upper stories are usually infilled with stiff masonry walls. This results in dramatic change in in-plane stiffness of the structure in elevation and can cause the formation of a soft story. Sometimes, the soft story can occur even in the case of continuity of infill walls in elevation by initial failure of the lower story’s infill walls. The soft story mechanism was the most common cause of building failure in the Turkey 1999 earthquake.

In the case of partially infilled frames, which are due to the presence of windows, shear failure of the effectively shortened columns can occur before columns reach their full flexural capacity. If not appropriately considered in the design phase, this can cause the failure of columns followed by collapse of whole building.

Asymmetric layout of infill walls in the plan can cause failure of the building by significantly increasing the torsional effects. A high portion of failures are associated with corner buildings that had open frames on the street sides while the two off street side frames were infilled.

2.3 In-Plane Behavior and Strength of Masonry Walls

2.3.1 Background

The seismic in-plane behavior and strength of masonry walls have been subjected to a considerable number of research programs including experimental and analytical studies over the last few decades.

Single-story and multi-story masonry walls, full-scale and reduced-size specimens made of different materials and reinforcement types and patterns, single-wythe and multi-wythe construction, walls with and without openings, and fully and partially grouted walls have been tested at different boundary conditions. Different testing procedures, static and dynamic, cyclic and monotonic, have been used in order to simulate the effects of seismic loads. Experimental studies of masonry walls have been conducted by numerous researchers in different countries, notably in Australia [Klopp and Griffith (1993), Zhuge et al. (1994, 1996)], Chile [Hidalgo and Luders (1987), Luders and Hidalgo (1986, 1988)], China [Xia et al. (1986)], Greece [Tassios et al. (1984)], Italy [Bernardini et al. (1997), Cantu and Macchi (1979), Giuffre’ et al. (1984), Macchi (1982)], Japan [Hirashi (1985), Imai and Miyamoto (1988), Kaminosono et al. (1988), Matsumura (1985, 1987, 1988), Okamoto et al. (1987), Wakabayashi


Several researchers have proposed different empirical equations to approximate the in-plane strength of masonry walls, notably, Blondet et al. (1989), Brunner and Shing (1996), Fattal (1993), Guiqiu et al. (1997), Matsumura (1985, 1987, 1988, 1990), Magenes and Calvi (1997), Shing et al. (1990, 1993), Tomazevic (1999), Tomazevic and Lutman (1988). The results of either experimental tests or analytical studies or both have been used to calibrate and validate the proposed equations. The use of empirical equations is popular in building design codes, since they significantly simplify the efforts towards estimation of strength of masonry walls. For instance, UBC (1997), Eurocode 6 (1996), NEHRP (2000), and MSJC (2002) have adopted such equations for nominal strength of masonry walls.
2.3.2 In-Plane Behavior of Unreinforced Masonry Walls (URM)

According to the experimental and earthquake observations, there are three types of failure modes for unreinforced masonry walls when subjected to in-plane loads. The mechanism depends on variety of parameters such as the geometry of the wall panel, particularly, aspect ratio (height to length), material properties of masonry units and mortar, boundary conditions, and construction method. These three failure modes as schematically shown in Figure 2.10 include sliding shear failure, diagonal shear failure, and flexural failure.

Sliding shear failure of masonry walls usually takes place in the case of low vertical load (axial compression) and poor quality of mortar material. In this case, the upper part of the wall panel slides on a horizontal crack formed along the bed joint as shown in Figure 2.10(a).

Diagonal shear failure is the most observed failure mode in typical masonry walls. This failure mainly occurs in the walls with a relatively low aspect ratio, which is the most common practice in masonry wall construction. This mechanism occurs where the developed principal tensile stress in the wall under a combination of axial compression and an in-plane load exceeds the tensile strength of masonry materials. The diagonal crack path can be extended through masonry units and mortar joints including bed joints (horizontal joints) and head joints (vertical joints) as illustrated in Figure 2.10(b).

In the case of improved tensile strength of masonry material and increased horizontal to compression load ratio, masonry wall can fail in flexural mode. The flexural failure can be in the form of rocking in the wall tension zone (opening of the bed joints) or crushing of compression zone at the corners of the wall (toe) as shown in Figure 2.10(c). For unreinforced masonry walls,
all of these failure modes are characterized by brittle behavior with rapid decrease in wall resistance and very limited post failure deformation.

2.3.3 In-Plane Strength of Masonry Walls Using Empirical Equations

The use of empirical equations for estimating nominal strength of masonry walls is the primary interest of this dissertation. Therefore, the emphasis of this section of literature review is on empirical equations. Some of the empirical equations proposed by different researchers for predicting the in-plane strength of masonry walls are summarized in this section.

2.3.3.1 Sliding Shear Failure

It has been generally accepted that the sliding shear resistance of an unreinforced masonry wall, \( V_{\text{slide}} \), can be defined according to friction theory by the following equation:

\[
V_{\text{slide}} = V_o + \mu N
\]  

In Equation (2.1), \( V_o = \tau_o A \) is the initial shear resistance under zero compressive load; \( \tau_o \) is the cohesion; \( A \) is the horizontal cross sectional area of the wall; \( \mu \) is the coefficient of friction of masonry relative to mortar joint; \( N = \sigma_c A \) is the compression axial load, and \( \sigma_c \) is the average axial compression stress. The values of \( \tau_o \) and \( \mu \) should be determined by test. In the case of pre-cracked mortar joints due to tension loads, the cohesion term \( (V_o) \) needs to be neglected.

According to Drysdale et al. (1999), the cohesion and the coefficient of friction typically range from 35 to 100 psi and 0.6 to 1.0, respectively, depending on material properties and surface roughness. However, for design purposes, masonry codes currently specify lower values for these parameters. For example, MSJC (2002) code has specified an allowable cohesion ranging from 15 to 60 psi and a single allowable coefficient of friction 0.45. Eurocode 6 (1996) has specified a cohesion ranging from 14.5 psi to 43.5 psi depending on the masonry unit and mortar types and a single value of 0.4 for the coefficient of friction.
2.3.3.2 Diagonal Shear Failure

Several empirical equations have been proposed by different researches to predict the shear resistance of masonry walls. Some of the prominent expressions are summarized next.

2.3.3.2.1 Fattal (1993)

As part of the National Institute of Standards and Technology (NIST) masonry research program, Fattal (1993) developed an empirical expression for predicting the shear strength of fully-grouted and partially-grouted masonry shear walls. The expression is a modification of one developed as part of the Japanese component of a joint U.S.-Japan Technical Coordinating Committee on Masonry Research (JTCCMAR) program (Matsumura, 1985; 1987; 1988). Fattal (1993) demonstrated improvements in accuracy with these modifications. According to Fattal (1993), the nominal shear strength of masonry walls is given by the following expressions:

\[
V_{\text{shear}} = (v_m + v_s + v_a)A
\]  

\[
v_m = k_v k_u \left[ \frac{0.5}{r + 0.8} \right] + 0.18 \sqrt{f'_{m} f_{yv}(\rho_v)^{0.7}}
\]

\[
v_s = 0.011 k_v \gamma f_{yv} \rho_v^{0.31}
\]

\[
v_a = 0.012 k_v f'_{m} + 0.2 \sigma_c
\]

In these equations, \(v_m\), \(v_s\), and \(v_a\), respectively, are the contributions of masonry, horizontal reinforcement, and axial compression stress; \(f'_{m}\) is the compressive strength of masonry; \(\rho_v\) and \(\rho_h\) are the ratios of vertical and horizontal reinforcement, respectively; and \(f_{yv}\) and \(f_{yh}\), are the nominal yield stresses of vertical and horizontal reinforcements, respectively. The dimensionless factors \(k_v\), \(k_u\), and \(\gamma\) are given in Table 2.1. The factor \(\delta\) is equal to 1.0 for walls with points of inflection at mid-height (doubly fixed end), and 0.6 for cantilever walls. Parameter \(r\) is the aspect ratio of wall defined as below:

\[
r = \frac{H}{L}
\]
Table 2.1: Factors for Fattal (1993) expression

<table>
<thead>
<tr>
<th>Factor</th>
<th>Fully-Grouted Walls (Clay or Concrete Masonry)</th>
<th>Partially-Grouted Clay Walls</th>
<th>Partially-Grouted Concrete Masonry Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_o$</td>
<td>$1.00$</td>
<td>$0.80$</td>
<td>$0.80$</td>
</tr>
<tr>
<td>$k_u$</td>
<td>$1.00$</td>
<td>$0.80$</td>
<td>$0.64$</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>$1.00$</td>
<td>$1.00$</td>
<td>$0.60$</td>
</tr>
</tbody>
</table>

For unreinforced masonry walls, Fattal (1993) expression results in the following equations:

\begin{align*}
V_{\text{shear}} &= (0.012 f'_{m} + 0.2 \sigma_o) A \quad \text{Fully-Grouted Walls} \\
V_{\text{shear}} &= (0.0096 f'_{m} + 0.2 \sigma_o) A \quad \text{Partially-Grouted Walls}
\end{align*}

(2.7a) (2.7b)

2.3.3.2 Shing et al. (1990)

As part of the U.S. component of the JTCCMAR program, Shing et al. (1990) proposed an expression for in-plane shear strength of fully-grouted masonry walls. The proposed expression takes into account the contribution of masonry, horizontal reinforcement, and axial compression stress. However, the contribution of the axial compression stress is combined with the shear strength component of the masonry. According to Shing et al. (1990), the total in-plane shear strength of masonry walls is given by the following equation:

\begin{align*}
V_{\text{shear}} &= V_m + V_s \\
V_m &= c_1 (\rho_f, f_{ys} + \sigma_v) + c_2 A_s f'_{m} \\
V_s &= \left[ \left( \frac{L - 2d'}{s} \right) - 1 \right] A_{sh} f_{sh}
\end{align*}

(2.8) (2.9) (2.10)

In above equations, $V_m$ and $V_s$ are the shear strength contributions of the masonry and horizontal reinforcement, respectively; $L$ is the length of the wall; $d'$ is the centroidal distance of the vertical tension reinforcement to the nearest jamb (edge of the wall); $A_{sh}$ is the area of single horizontal reinforcement, and $s$ is the vertical spacing of horizontal reinforcement. The constants $c_1$ and $c_2$ are equal to $0.0018/\sqrt{\text{psi}}$ and $2.0 \sqrt{\text{psi}}$, respectively. The term $\left[ (L - 2d')/s - 1 \right]$ reflects the number of effective horizontal reinforcing bars. The cross-sectional area of the wall ($A_n$) in Equation (2.9) is based on the net horizontal section. However, the ratio of vertical reinforcement
\( \rho \) and the axial compression stress \( \sigma_c \) must be defined in a consistent manner, because the products of these two parameters and the net area must be equal to the total area of vertical steel \((A_v)\) and the net compression force \(N\), respectively. The steel strength \((V_s)\) is based on a 45° truss idealization of the masonry wall, in which horizontal reinforcing bars with area \(A_{sh1}\) are placed at a uniform spacing \(s\) along the height of the wall. For bond beam specimens, \(s\) is taken as one-half the height of the panel, and \(A_{sh1} = A_b\) is taken as the total area of steel in the bond beam. For unreinforced masonry walls, the Shing et al. (1990) expression results as follows:

\[
V_{\text{shear}} = [0.0018\sigma_c + 2]A_n\sqrt{f_m^*}
\]  

(2.11)

2.3.3.2.3 Shing et al. (1993)

Shing et al. (1993) proposed an improved version of the earlier expression by considering the effects of aspect ratio and boundary conditions of a wall. Furthermore, Shing et al. (1993) showed by a finite element analysis results that it is unlikely that all of the horizontal reinforcing bars reach the yield limit simultaneously when the maximum shear develops and thus, the resistance of horizontal reinforcement is overestimated. Therefore, two major modifications are proposed. First, the residual strength \(V_m\) is divided into two components, \(V_c\) and \(V_i\). \(V_c\) corresponds to the shear resistance developed in the compression toe, which depends on the compressive stress at the toe and the compressive strength of masonry. \(V_i\) represents the aggregate-interlink force developed along a diagonal crack, which is a function of the quantity of the vertical steel and the axial compressive stress \(\sigma_c\) for a given aggregate mix. Secondly, since the horizontal steel may not all reach the yield strength at the maximum shear, a reduction factor is applied. These result in Equation (2.12). The strength provided by masonry at the toe is given as Equation (2.13).

\[
V_{\text{shear}} = V_c + V_i + V_s
\]  

(2.12)

\[
V_c = C_1 f_m^* \sqrt{1 - \frac{C_2 \sigma_c}{f_m^*}} A_n
\]  

(2.13)

In these equations, coefficient \(C_1\) reflects the percentage of the total wall area effective in resisting shear at the compression toe. Coefficient \(C_2\) is used to estimate the level of compressive
stress at the compression toe. As the aspect ratio decreases, a larger portion of the wall area is expected to resist shear, indicating an increase in coefficient \( C_1 \). However, the masonry compressive stress in the toe, \( C_2 \sigma_c \) should decrease, resulting in a reduction of coefficient \( C_2 \).

The shear resistance provided by aggregate interlock is expressed as follows:

\[
V_i = C_1 \left[ C_4 \rho_y f_{yy} + \sigma_c \right] A_n
\]  
(2.14)

In Equation (2.14), coefficient \( C_3 \) reflects the friction along the crack and depends on the roughness of the crack surface. The remaining terms correspond to the approximate vertical load acting on the diagonal crack. Coefficient \( C_4 \) accounts for the fact that not all of the vertical steel will reach its tensile yield stress when the shear capacity is reached. The resistance provided by the horizontal steel is given as follows:

\[
V_s = C_5 \left[ \frac{l - 2d'}{s} - 1 \right] A_{sh} f_{sh}
\]  
(2.15)

In Equation (2.15), coefficient \( C_5 \) represents the reduction factor for contribution of the horizontal reinforcement. Coefficients \( C_i \)'s can be calibrated with experimental data or a finite element model. Shing et al. (1993) proposed the following values for these coefficients by performing regression analysis on the results of a parametric finite element study: \( C_1 = 0.04; C_2 = 4.5; C_3 = 0.25; C_4 = 0.667, \) and \( C_5 = 0.75 \). Substituting these coefficients in above equations for unreinforced masonry walls, the Shing et al. (1993) expression results in the following equation:

\[
V_{shear} = 0.04 f_n' \sqrt{1 - \frac{4.5 \sigma_c}{f_n'}} A_n + 0.25 \sigma_c A_n
\]  
(2.16)

2.3.3.2.4 Magenes and Calvi (1997)

Based on experimental and numerical simulations, Magenes and Calvi (1997) proposed the in-plane resistance of unreinforced masonry walls to be the lowest strength associated with the two failure modes, one based on friction theory and one based on principal tensile theory:

The first failure mode (brick shear failure):
\[ V_{\text{shear}1} = \frac{f_{ct}}{2.3(1 + \alpha_v)} \sqrt{1 + \frac{\sigma_c}{f_{ct}}} \]  

(2.17)

where, \( f_{ct} \) is the tensile strength of the brick unit.

The second failure mode (joint shear failure):

\[ V_{\text{shear}2} = \tau_u A \]  

(2.18)

\[ \tau_u = \min \left\{ \begin{array}{l} \tau_{cs} = \frac{1.5 \tau_u + \mu \sigma_c}{1 + 3 \tau_u \alpha_v / \sigma_c} \quad \text{(cracked)} \\ \tau_{uc} = \frac{\tau_u + \mu \sigma_c}{1 + \alpha_v} \quad \text{(uncracked)} \end{array} \right. \]  

(2.19)

where, \( \alpha_v \) is the shear ratio as defined as follows:

\[ \alpha_v = \frac{M}{VH} = \frac{\psi^* H}{L} = \psi^* t \]  

(2.20)

In Equation (2.20), \( M \) is the maximum bending moment corresponding to the in-plane shear load \( V \). The parameter \( \psi^* \) is to take a value of 1.0 when the wall is fixed on one end and free to rotate on the other end (cantilever wall) and a value of 0.5 when the wall is fixed at both ends. Mann and Muller (1982) proposed a correction factor for cohesion and coefficient of friction to take into account for different brick size as the following equations, in which, \( \Delta_x \) and \( \Delta_y \) are the length and height of the masonry brick unit, respectively.

\[ \overline{\tau} = \kappa \tau \text{ and } \overline{\mu} = \kappa \mu \]  

(2.21)

\[ \kappa = \frac{1}{1 + \mu 2 \Delta_y / \Delta_x} \]  

(2.22)

2.3.3.2.5 Guiqiu et al. (1997)

Based on a statistical analysis on the experimental results of 60 unreinforced masonry wall specimens, Guiqiu et al. (1997) proposed Equation (2.23) for shear strength of unreinforced
masonry walls that takes into account the strength of masonry material, geometry of wall, and axial compression stress. In Equation (2.23), \( \psi \) is a coefficient representing the effects of the aspect ratio of the masonry wall given by Equation (2.24).

\[
V_{\text{shear}} = f'_m \left[ 0.02 + 0.88 \left( \frac{\sigma_c}{f'_m} \right) - 0.9 \left( \frac{\sigma_c}{f'_m} \right)^2 \right] \psi A
\]  
(2.23)

\[
\psi = 0.96 - 0.68 \log(r)
\]  
(2.24)

2.3.3.2.6 Tomazevic and Lutman (1988)

Tomazevic and Lutman (1988) proposed the following expression for in-plane resistance of unreinforced masonry walls based on the principal tensile stress theory:

\[
V_{\text{shear}} = A \left( \frac{f_t}{b} \right) \sqrt{1 + \frac{\sigma_c}{f_t} + \Phi_h A_{sh} f_{sh}}
\]  
(2.25)

In Equation (2.25), \( f_t \) is the tensile strength of masonry; \( A_{sh} \) is the total area of the horizontal reinforcement, and \( \Phi_h \) is the horizontal reinforcement capacity reduction factor. Tomazevic and Lutman (1988) proposed the value of \( \Phi_h = 0.4 \) based on the experimental results. Coefficient \( b \) defines the shear stress distribution factor based on aspect ratio \( r \) as:

\[
b = \begin{cases} 
1.5 & \text{for } r \geq 1.5 \\
1 & \text{for } 1.5 > r > 1.0 \\
1 & \text{for } r \leq 1.0 
\end{cases}
\]  
(2.26)

2.3.3.2.7 Tomazevic (1999)

Tomazevic (1999) improved the previous expression by taking into account the resistance contributions of horizontal and vertical reinforcement as follows:

\[
V_v = V_c + C_{sh} V_{sh} + V_{sv}
\]  
(2.27)
\[ V_m = \frac{A_t}{b} \sqrt{1 + \frac{\sigma_s}{f_t}} \]  

(2.28)

\[ V_{sh} = 0.9d \frac{A_h}{s} f_{yh} \]  

(2.29)

\[ V_{sv} = \sum 0.806d_r^2 \sqrt{f_m f_{sy}} \]  

(2.30)

In above equations, \( d \) is the effective depth of the wall; \( d_{rv} \) is the diameter of vertical reinforcing bar; \( f_m \) is the compressive strength of embedding mortar or grout, and \( C_{rh} \) is the reduction factor for resistance of horizontal reinforcement. Tomazevic (1999) reviewed several series of experimental tests and concluded that the reduction factor can vary between 0 and 0.5 depending to the characteristics of the masonry units and the mortar as well as the reinforcement details. In the case of lack of experimental results, Tomazevic (1999) proposed a value of 0.3 for the reduction factor. The third term for the contribution of vertical reinforcing bars is concluded based on the dowel mechanism of vertical reinforcement at shear failure of masonry walls.

### 2.3.3.2.8 UBC (1997)

The 1997 edition of the Uniform Building Code includes a formula for the nominal shear strength of masonry walls that recognizes the contributions provided by masonry \( (V_m) \) and horizontal reinforcement \( (V_h) \) as follows:

\[ V_{sharc} = V_m + V_h \]  

(2.31)

\[ V_m = C_d A_e \sqrt{f_m} \]  

(2.32)

\[ V_h = A_e \rho_h f_{yh} \]  

(2.33)

In these equations, \( A_e \) is the effective horizontal cross sectional area of the wall and coefficient \( C_d \) is taken as follows depending on the shear ratio of the wall:

\[ C_d = \begin{cases} 2.4 & \text{for } \alpha_v < 1/4 \\ 2.8 - 1.6\alpha_v & \text{for } 1/4 < \alpha_v < 1 \\ 1.2 & \text{for } 1 < \alpha_v \end{cases} \]  

(2.34)
2.3.3.2.9 NEHRP (2000) & MSJC (2002)

According to 2000 Edition of the NEHRP Provisions, the nominal shear strength of masonry walls is given by the following expressions:

\[ V_{\text{shear}} = V_m + V_s \]  \hspace{1cm} (2.35)

\[ V_m = c_1 (4 - 1.75\alpha_v) A_n f_m + 0.25 A_n \sigma_v \]  \hspace{1cm} (2.36)

\[ V_s = 0.5d \frac{A_{sl}}{s} f_{sh} \]  \hspace{1cm} (2.37)

In Equation (2.36), constant \( c_1 \) is equal to 1.0 \( \sqrt{\text{psi}} \), and the shear ratio \( \alpha_v \) needs not to exceed 1.0. There is a maximum \( V_{\text{max}} = 6A_n \sqrt{f_m} \) for \( \alpha_v < 0.25 \) and \( V_{\text{max}} = 4A_n \sqrt{f_m} \) for \( \alpha_v < 1.0 \). The expression for the nominal strength of shear reinforcement in NEHRP (2000) is identical to that in the UBC (1997), but only 50% of this strength is assumed to contribute to the shear resistance at maximum capacity. Unlike UBC (1997), NEHRP (2000) has included the contribution of the axial compression stress in shear resistance of masonry walls. The Masonry Standards Joint Committee (MSJC 2002) has adapted the same empirical equations as NEHRP (2000) for nominal shear strength of masonry walls.

2.3.3.2.10 Eurocode 6 (1996)

The theory of friction has been accepted for evaluation of shear resistance of masonry walls by Eurocode 6 (1996). According to Eurocode 6 (1996), the in-plane resistance of masonry walls is evaluated as follows:

\[ V_{\text{slide}} = (\tau_o + 0.4\sigma_v) A + 0.9 \rho_s f_{sh} \]  \hspace{1cm} (2.38)

In Equation (2.38), \( \tau_o \) is the cohesion ranging from 14.5 psi to 43.5 psi depending on the masonry unit and mortar types. Eurocode 6 (1996) has specified a single value of 0.4 for the coefficient of friction.
2.3.3.3 Flexural Failure

Unlike shear resistance, relatively accurate methods are available to calculate the in-plane flexural resistance of masonry walls. Two methods are presented here assuming a symmetrical vertical reinforcement arrangement along the horizontal cross section of masonry walls.

According to Tomazevic and Lutman (1988) the flexural capacity of masonry walls can be expressed by the following expression:

\[
M_n = \frac{\sigma_c t L^2}{2} \left(1 - \frac{\sigma_v}{\kappa f_v}\right) + (L - 2d')A_f f_{yv} \tag{2.39}
\]

In Equation (2.39), \(\kappa\) is a coefficient that takes into account the vertical stress distribution at the compressed toe. A common assumption is an equivalent rectangular stress block with \(\kappa = 0.85\). Having the flexural capacity of the section, the nominal in-plane resistance associated with the flexural resistance of the wall is calculated based on the boundary conditions as follows:

\[
V_{\text{flexural}} = \frac{M_n}{\psi' H} \tag{2.40}
\]

The second approach is based on NEHRP (2000) code as subsequently explained. Using the equivalent rectangular stress block and assuming all of the vertical reinforcement concentrated in the exterior cells, only one-half of the vertical steel is effective in tension. Static equilibrium for vertical forces on the wall results in the following equation:

\[
0.85 f'_{w, t} a t = 0.5 A_{\rho_v} f_{yv} + A_n \sigma_c \tag{2.41}
\]

In above equation, the first term is the resultant compression force in the masonry in the depth of effective compression block \((a)\); the second term is the net force of vertical reinforcement in tension, and the last term is the net axial compression force on the wall. By solving Equation (2.41), Equation (2.42) is the result for the depth of effective compression block, in which, \(L_g\) is the distance from the jamb to the edge of the nearest ungrouted cell, and \(t_f\) is nominal (gross) thickness of the web.
Knowing the depth of the effective rectangular compression block \((a)\), the internal resisting moment is calculated about the neutral axis of the wall section. Approximating the centroidal distance of the vertical tension steel to the nearest jamb as \(0.5L_g\); Equation (2.43) is the result for the nominal moment capacity of the wall. The nominal shear resistance associated with the flexural capacity can be similarly obtained by Equation (2.40).

\[
M_a = \left( \frac{tL}{2} \right) \rho_s f_{sv} \left( L - \frac{L_g + a}{2} \right) + \sigma_c \left( L - a \right)
\]  

(2.43)

2.4 Finite Element Modeling of Masonry Infill Walls

Several finite element modeling approaches have been proposed by different researchers to simulate the in-plane behavior of masonry infill walls. A review of some of these methods that have been used in this study for modeling of infilled steel frames is presented in this section.

2.4.1 Micro Model by Seah (1998)

Seah (1998) introduced a very sophisticated micro model in which the masonry panel was modeled as a series of elastic blocks with orthotropic properties connected by a system of springs. The blocks were assumed to be linearly elastic up to the point of crushing while the springs were introduced to handle failure due to tensile and shear stresses in the mortar joints when their capacities are exceeded. This model can account for the complex behavior of masonry infill due to cracking and contact and separation at frame/panel interfaces. The computer program "INFRAME32" was developed using an object orientated programming (OOP) approach in C++ to implement the analytical technique for the micro model. It was compiled to be used on a very fast micro-computer. A schematic of the infilled steel frame model is shown in Figure 2.11.
2.4.2 Macro Model by Seah (1998)

A macro model, in which the panel as a whole was replaced by a non-linear diagonal spring, was also proposed by Seah (1998) which was more practical for design purposes. Seah (1998) developed a general three-dimensional frame program called EPIFRAME for the macro model for the analysis of building frames with infills. The force-deformation response for the brace may be obtained from testing or it may be generated using the analytical technique of the micro model. Figure 2.12 shows the macro model with a brace introduced in the frame to achieve the same effect. The force-deformation response of the brace can be established with reference to Figure 2.12 as follows:

\[
 C_d = \frac{H}{\cos \theta} \tag{2.44}
\]

\[
 \Delta_d = \Delta_h \cos \theta - \Delta_v \sin \theta \quad \tag{2.45}
\]

In these equations, \( \Delta_d \) and \( C_d \) are, respectively, axial force and deformation of diagonal brace; \( \Delta_h \) and \( \Delta_v \) are, respectively, the horizontal and vertical deformations of top corner of the
frame, and $H$ is the applied horizontal load. It should be pointed out that these equations give the relationship of the equivalent diagonal force and deformation for the infilled frame system and, therefore, includes the rigidity of the surrounding frame. In the macro model, hinges were introduced to eliminate the lateral resistance of the frame to assure that the lateral resistance of the system is provided solely by the diagonal brace shown in Figure 2.13.

As indicated, experiments or detailed finite element analyses are required to generate the force-deformation curve for the equivalent diagonal brace for any infilled frame. This is neither practical nor cost effective. Therefore, Seah (1998) suggested conservative and simplified force-deformation curves for a wide range of infilled steel frames. A typical simplified curve suggested by Seah (1998) is shown in Figure 2.14. The force and deformation values of four points of this curve (i.e., points a, b, c, and d) depend on the panel dimensions and the material properties of the steel frame members and the infilled masonry wall. By performing a large number of analyses using the micro model, Seah (1998) developed a series of tables including the force and deformation values for these four points for a wide range of infilled steel frames.
2.4.3 Macro Model by El-Dakhakhni et al. (2003)

El-Dakhakhni et al. (2003) introduced a simple method for estimating the stiffness and the in-plane capacity of concrete masonry-infilled steel frames (CMISFs). In this method, each masonry panel was replaced by three struts with force-deformation characteristics based on the orthotropic behavior of the masonry infill wall. Assuming orthotropic behavior instead of real anisotropic behavior for the masonry infill wall gives a better approximation than simply using isotropic behavior. For an isotropic material, the stress-strain curve is independent of the orientation. An anisotropic material has characteristic orientation and the stress-strain curve is dependent to the orientation. An anisotropic material with one or more planes of symmetry is known as an orthotropic material. The proposed model is schematically shown in Figure 2.15.
El-Dakhakhni et al. (2003) considered an elastic behavior for steel frame members, while the beam-column connections were modeled by a trilinear rotational spring element. A typical joint behavior is shown in Figure 2.16. The ultimate moment capacity of the trilinear rotational spring, $M_{pj}$, was defined as the minimum of the column’s, beam’s, or the connection’s ultimate capacity. The rotational stiffness of the spring can be calibrated so that the lateral stiffness of the frame model matches that of the actual bare frame, which can be obtained experimentally or using elastic analysis. El-Dakhakhni et al. (2003) suggested that the contact lengths between the masonry wall panel and the frame shown in Figure 2.15 to be approximated by the following equations:

$$
\alpha_c h = \sqrt{\frac{2(M_{pc} + 0.2M_{pc})}{f_{m-0}^{'}\alpha}} \leq 0.4h \quad (2.46)
$$

$$
\alpha_b l = \sqrt{\frac{2(M_{pb} + 0.2M_{pb})}{f_{n-90}^{'}\alpha}} \leq 0.4l \quad (2.47)
$$

In above equations, $\alpha_c$ is the ratio of the column contact length to the height of the column; $\alpha_b$ is the ratio of the beam contact length to the span of the beam; $h$ and $l$ are column height and beam length, respectively; $M_{pc}$ and $M_{pb}$ are column and beam plastic moment capacities, respectively; $f_{m-0}^{'}$ and $f_{n-90}^{'}$ are the compressive strength of the masonry wall parallel and normal to the bed joint, respectively, and $t$ is the thickness of the wall. In Figure 2.15, the area of the equivalent diagonal region of the masonry wall panel, $A$, was defined by the following equation:

$$
A = \frac{(1-\alpha_c)\alpha_c h t}{\cos(\theta)} \quad (2.48)
$$

For the diagonal struts representing masonry infill wall, as shown in Figure 2.17, instead of using parabolic stress-strain relation it was suggested to approximate it into a trilinear relation, which is simpler and more practical for analysis. Knowing the stress-strain relation, the area, and the length of each of the three struts, one can obtain the force-deformation relation for each strut.
Figure 2.15: Infilled steel frame macro model using three-diagonal strut
(El-Dakhakhni et al. 2003)

Figure 2.16: Typical beam-column connection model behavior (El-Dakhakhni et al. 2003)

Figure 2.17: Simplified trilinear relations: (left) stress-strain relation of masonry, (right) force-deformation relation for struts (El-Dakhakhni et al. 2003)
2.5 Summary and Concluding Remarks

Masonry infill walls are being widely used in steel and concrete structures in many countries. Two common practices of masonry infill walls include tight-fit and isolated constructions. Neither of these two approaches has been very successful and satisfactory so far. In the case of completely isolated infill walls, it is very difficult to provide an effective out-of-plane stability for the wall. Sufficient acoustic and fire protection of the separation gaps is another concern for isolated infill walls. In the case of tight fit construction, masonry infill walls have been long known to considerably affect the response of the infilled frame structures. The interaction between infill wall and structural frame is not typically accounted for in analysis and design. Despite this fact, most building codes still do not have strict and clear provisions for seismic design and construction of infill walls. They neglect or very briefly refer to the effects of infill walls. The consequence is that the infill walls continue to be vulnerable to damage in earthquakes. Damage to infill walls can result in significant economic loss and human casualties even if the structural frame is only lightly damaged. Cracking and collapse of infill walls, soft story, short column, and premature shear failure of column are most commonly observed damages.

Over the last few decades, a significant number of analytical and experimental research programs have been carried out to study the seismic in-plane behavior and strength of masonry walls. These studies and earthquake observations have shown that there are mainly three types of failure modes for masonry walls when subjected to in-plane loads including: sliding shear failure, diagonal shear failure, and flexural failure. The mechanism depends on variety of parameters such as the geometry and material properties of masonry wall. For a typical masonry wall in a typical building, diagonal shear failure is the most observed failure mode.

Experimental tests, finite element modeling, and empirical equations can be used to predict the diagonal shear strength of masonry walls. For design purposes, the use of empirical equations is popular and convenient, since they significantly simplify the efforts towards estimation of strength of masonry walls. Most building codes have implemented such simplified procedures such as UBC (1997), Eurocode 6 (1996), NEHRP (2000), and MSJC (2002). However, for detailed investigation of behavior of masonry infill walls, either experimental tests or numerical simulations should be considered. Several researchers have proposed different micro and macro finite element modeling methods for infilled frames. The use of micro models is very complicated and costly and the use of macro models is more popular. Seah (1998) proposed a
very sophisticated micro model in which the masonry panel was modeled as a series of elastic blocks connected by a system of springs. This model can closely predict the failure mode and the load-deflection history for an infilled steel frame. A macro model, in which the panel as a whole was replaced by a non-linear diagonal spring, was also proposed by Seah (1998). El-Dakhakhni et al. (2003) introduced another macro method for masonry infilled steel frames, in which each masonry panel was replaced by three struts. These macro models, which are more practical for design purposes, do not have the accuracy and power of micro models. However, they are convenient tools to estimate the stiffness and strength of masonry infill walls within acceptable degree of accuracy.
Chapter 3

SEISMIC INFILL WALL ISOLATOR SUBFRAME (SIWIS) SYSTEM

3.1 Introduction to SIWIS System

The solution proposed in this dissertation consists of using subframes to be attached to the structural frame. The infill wall then is to be constructed within the subframe. The concept of SIWIS system is illustrated in an example in Figure 3.1, which shows a subframe system including two vertical members and one horizontal member placed between an infill wall and the structural frame. Figure 3.2 shows a section through a column (steel or reinforced concrete), where it can be seen that vertical member of the subframe consists of two sandwiched light-gage steel plates with a “fuse” element between them. Within the subframe member at the top of the wall between the beam and the wall, which will not have fuse elements, and in the open spaces of the subframe vertical elements, there can be a flexible filler material such as polyester/polyurethane foam. This flexible filler material should provide the needed sound insulation and fire-resistance. Through specific support to the side and top subframe members, the out-of-plane stability of the infill wall will be provided. The fuse element is intended to make a rigid like connection between the infill wall and the frame to engage the infill wall in in-plane load resistance up to a certain load followed by its failure once this load limit exceeds. The whole procedure will result in disengagement of the infill wall from interaction with the frame. The SIWIS element can also include a spring component of desirable stiffness to control in-plane movement after the failure of the fuse element (Figure 3.2b). Furthermore, a dashpot mechanism can be added to the element to enhance seismic performances of a structure by increasing its energy dissipation capacity (Figure 3.2c). In this dissertation, brittle and flexible elements are considered in finite element modeling and only the brittle element is used in the experimental program.
The SIWIS system is designed such that under small to moderate levels of structure frame in-plane interface force, the subframe will act as a rigid link and transfers the force to the infill wall for the benefit of its stiffness in reducing drift. At larger levels of in-plane force (the designed threshold of the subframe), the fuse elements of the subframe will break and allow the structural frame to displace without transferring force to the infill wall. Different grades (e.g., low, medium, high) for the subframe vertical members can be designed and specified according to the type and predicted strength and stiffness properties of the infill wall. This way, if the infill
wall is known to have poor quality or if the stiffness of the infill wall is simply not to be relied upon, mild grade of the subframe will then be suitable.

The SIWIS system can be used with many types of masonry infill walls including walls with or without openings, partial or full infills, and with masonry units ranging from high strength concrete masonry blocks and clay brick units to lower strength masonry such as thin wall hollow clay tile units and autoclaved aerated concrete (AAC) blocks. Figure 3.3 shows an example of an application of the SIWIS system for a partially infilled frame.

Figure 3.3: An example of SIWIS system used in a partially infilled frame

3.2 SIWIS Element

The SIWIS element is the centerpiece in the proposed seismic isolation system. The fuse-like element inside the infill wall isolator subframe is used to establish “rigid-free” isolation between frame and infill wall in a strong earthquake event. The fuse element needs to break at a certain compression load level to allow disengagement of the masonry wall from frame. The capacity of the fuse element is determined based on the cracking load of the infill wall by considering enough safety margin. The element has no tension resistance.

The design of the SIWIS element should take into account the possibility of replacing the broken part (or whole element) after its failure through specific small access windows on the
subframe. It can be designed in such a way to allow the whole element to be replaced as a compartment. Alternatively, only the failing piece of the element can be replaced. Another important parameter is the effective age of a SIWIS element. The age of a building can reach many decades, thus the SIWIS element and its material should keep their strength and stiffness properties for decades. It may also be desirable to replace all SIWIS elements in a building periodically such as every 20-30 years depending on the material used in the SIWIS element design. This can be practically done through specific access windows on the subframe. From an economical point of view, the SIWIS element has two types of costs. First, the construction cost including material, construction, and assemblage, and second, the repair cost including checking after any seismic activity and replacing the SIWIS element after its failure.

Several designs for the fuse component of the SIWIS element are proposed in this dissertation. From several proposed designs, three were comprehensively studied through experimental tests and one was chosen for the main tests on the two-bay three-story frame in Chapter 5. However, all designs are considered in finite element modeling in Chapter 4. These proposed designs are introduced next.

### 3.2.1 Compression Disk Design

Figure 3.4 shows a conceptual schematic of compression disk design for a fuse element. Basically three separate parts, including a rod for exerting force, a disk intended to fail under a certain compressive load, and an open cylindrical piece as the support, are used. In this design, the disk component acts as a fuse and should be designed to fail at a certain compressive load. The disk component is the part to be replaced after its failure.
Three different designs are proposed for a compression disk failing component including concrete disk, steel and epoxy disk, and lumber disk as explained next. These three designs are experimentally tested and reported in Chapter 5.

3.2.1.1 SIWIS Element Using Concrete Disk

Concrete has a high compression strength that makes it suitable for the fuse element. Concrete can maintain its structural properties for very long periods of time such as the age of the building. For the purpose of application of a fuse element, concrete is more suitable for greater thickness. For the relatively small dimensions of the disk, which can be about 4 in. to 8 in. in diameter and preferably less than 2 in. in thickness, concrete with regular gravel sizes may not result in homogenous and uniform disks. Thus, smaller gravel sizes and a specific concrete mix need to be used. Photos of an SIWIS element with a concrete disk that has been used in an experimental test are shown in Figure 3.5. There is a notch on top of the concrete disk to accommodate the positioning of the rod as well as to lead the punching of the rod through the concrete disk. The top plate seen in Figure 3.5 is used in a test setup for distributing compression load over the larger area on the masonry wall. The rod penetrates the concrete disk and the whole system works as a rigid-free fuse element.

Figure 3.5: Photos of SIWIS element and concrete disk
3.2.1.2 SIWIS Element Using Steel Disk and Epoxy

An alternative design for the disk component of the fuse element is to replace the concrete disk with a steel disk assembly in the form of two steel disks attached by adhesive epoxy. The use of epoxy can give better control over strength of the fuse element. The desirable strength of the fuse element can be achieved by using a suitable epoxy and appropriate surface area for epoxy joint. Nowadays, there are many types of epoxies with varying strengths and stiffness properties. However, the use of steel disks can make the SIWIS element heavier and more expensive in comparison to concrete disks. Two different cases are considered for the steel disk assembly including; tension case and shear case. These two cases are schematically shown in Figure 3.6 along with photo of SIWIS element.

![Steel Disks with Epoxy](image)

Figure 3.6: Conceptual schematic and photo of SIWIS element using steel disk and epoxy

3.2.1.3 SIWIS Element Using Lumber Disk

Wood is another useful material for the fuse and similar to concrete and steel can maintain its structural properties for very long periods of time if kept safe from environmental effects. There are many lumber types with different structural and mechanical properties.
Figure 3.7 shows photos of an SIWIS element and a lumber disk that have been used in an experimental test. Similar to the concrete design, the rod pushes and causes failure of the lumber disk, thus making the whole system work as a fuse element.

Two concerns about lumber is the long-term performance and fire protection. Physicochemical processes, which can occur by atmospheric moisture and contaminants, may change the physical and chemical properties of the lumber and ultimately, change the strength of the disk. Therefore, it is necessary to provide effective environmental protection for the element. The lumber disk is located in the closed compartment inside the vertical member of the SIWIS subframe. Furthermore, the subframe itself is a closed member and meant to provide sufficient acoustic and fire protection for the separation gap. Thus, an effectively atmospheric protection is provided for the lumber disk for long periods of time. For the same mentioned reasons, lumber disk, which is a combustible element, is effectively protected for fire.

![Figure 3.7: Photos of SIWIS element using lumber disk](image)

### 3.2.2 Friction Element Design

An alternative design for the SIWIS element is the use of the concept of coulomb friction theory similar to friction damper (dissipater) devices. A conceptual design for a compression only
friction element is shown in Figure 3.8. Basically, three steel plates and two brass shims are sandwiched together by a bolt. A Belleville locking washer is used under the nut to ensure effective maintenance of normal tension load. A direct tension indicator (DTI) washer is used under the bolt head to accurately and satisfactorily control the amplitude of the applied normal tension load. Alternatively, torsional friction disks can be used as shown in Figure 3.9.

Figure 3.8: A conceptual design for SIWIS element using frictional plates

Figure 3.9: Conceptual designs for SIWIS element using torsional friction disks
A major advantage of the friction device over the compression desk is that there is no failing element. The two steel plates or disks slide relative to each other and they only need to be returned to their initial positions after slide (re-set). Thus, there is no need to replace any component (sacrificial element) and this is cost effective. Although no failure occurs in this element, it still acts like a “fuse” by means of initiation of slide once the applied compressive load exceeds the friction resistance. The capacity of the element can be controlled by the magnitude of the applied tension load on the bolts and number of bolts.

One concern about friction devices is physicochemical processes, which can occur by atmospheric moisture and contaminants in the interfacial region of the friction surfaces. This process may alter the physical and chemical properties of the surfaces and ultimately, change the frictional force. Thus, it is necessary to provide effective environmental protection for the friction surfaces. Corrosion of metal is another issue, which needs to be addressed (use of corrosion-resistant metals). It is important to ensure that the friction material and surfaces are selected appropriately. Two central criteria that need to be satisfied include; (1) reliable, constant, and permanent coefficient of friction and (2) elimination of tearing and bearing fractures. It is essential to maintain a normal tension load on the bolt during the life of the element. The change in this load (decrease) will reduce the capacity of element. For the proposed device, the use of hardened spring washers (Belleville) is an effective option. These washers are widely used in mechanical devices such as pressure vessels.

3.2.3 Tension Element Design

Another possible design for a fuse element is the use of tension failure instead of compression failure. Figure 3.10 shows a conceptual schematic for a possible design, in which a mechanism converts compressive load into tensile resistance. The fuse like tension element should be replaced after its failure. The fuse of the tension element can be a high strength wire or a metal rod or similar material. One advantage of the tension element is the fact that controlling tension failure can be relatively easy compared to compression failure. Accurately designed, tested, and manufactured tension elements can be used to ensure accurate and predictable capacity. One major difference between tension and compression elements is the type of their failure modes. Compression elements typically have a brittle and sudden failure with no or very limited post-failure deformations. On the other hand, tension elements usually have flexible failure (yield) with considerable post-failure deformations.
3.2.4 Jagging and Releasing Element Design

The jagging and releasing concept as shown in Figure 3.11 is another possible design for a fuse-like SIWIS element. In this case, while the whole system is subjected to an axial compression load, a flexible or rigid type of jag releasing mechanism in the perpendicular direction can be used to act as fuse. One advantage of this mechanism is the fact that there is no failing component and the released element only needs to be re-set.

Figure 3.10: Conceptual schematics of SIWIS element using tension element design

Figure 3.11: Conceptual schematics of SIWIS element using jagging and releasing element design
3.3 Summary and Concluding Remarks

The seismic Infill Wall Isolator Subframe (SIWIS) system is proposed as an alternative approach for masonry infill walls construction. This system consists of subframes to be attached to the structural frame. The infill wall then is to be constructed within the subframe similar to regular tight fit masonry wall construction. The subframe consists of two vertical members and one horizontal member. The vertical members consist of two sandwiched light-gage steel plates with a “fuse” element between them. Within the subframe member at top of the wall and in open spaces of the subframe vertical elements, a flexible filler material can be used to provide the required sound and fire protections. The out-of-plane stability of the infill wall is provided through specific support to the side and top subframe members.

Several conceptual designs are proposed for an SIWIS element including: compression disk, friction device, tension element, and jagging and releasing element. The compression disk design consists of a pipe as the support, a disk as the failing component, a seat disk between pipe and failing disk, a rod for exerting force, and an end plate. The disk part is the failing component and needs to be replaced after its failure. For the failing disk, three different options are proposed including: concrete disk, steel disk and epoxy, and lumber disk. In friction design for SIWIS element, steel plates and brass shims can be sandwiched together by a bolt. Once the sliding friction force between steel and brass plates is reached, they slide either in translation or rotation. In tension element design, a tension yielding failure is implemented instead of compression failure as for the disk. Finally, a flexible or rigid type of jag releasing mechanism can be used to act like a fuse for SIWIS element.

The proposed SIWIS system can potentially improve the seismic performances of masonry infill walls. This system initially uses the beneficial effects of infill walls during minor-to-moderate earthquakes and wind load, but, ultimately isolates them from the structural frame during damaging earthquakes for their safety. Different grades (e.g., low, medium, high) for the “fuse” element can be used based on the type and predicted strength and stiffness properties of the infill wall. This system can be used for various types of masonry infill walls including: walls with or without openings, full or partial infills, walls with different masonry units, and steel or reinforced concrete buildings.
Chapter 4

NON-LINEAR FINITE ELEMENT MODELING

4.1 Overview of Analytical Program

In order to further understanding of the behavior of an infilled frame equipped with the SIWIS system, nonlinear finite element modeling schemes were developed and validated for analysis of bare and infilled steel frames with and without SIWIS elements. For more meaningful comparison and verification purposes, initially, a single-bay single-story steel frame with tightly fitted infill wall that has been the subject of past experimental and analytical studies was considered. Three different finite element modeling schemes used by other researchers to model this case study were found to be useful. After the preliminary validation of the modeling scheme for the single-bay single-story frame, finite element models for a typical two-bay three-story steel frame were developed to study the use of SIWIS system in multi-bay multi-story frames. The two models were subjected to pushover and cyclic analyses under different load-control and displacement-control parameters. Moreover, a parametric study was conducted on these models to investigate the influences of different conditions in their responses. Finally, a finite element model was developed for the scaled two-bay three-story test frame in order to compare the numerical results with the experimental observations and accomplish the final validation for the finite element modeling schemes.

4.2 Types of Analysis Used in this Study

In the Finite Element Method (FEM), structures are approximated as assemblages of discrete finite elements connected at nodal points, where displacements are sought. The element stiffness matrix can be developed with use of the strain-displacement and the stress-strain relationships. Applying equilibrium, compatibility, and constitutive relationships, one can form the partial differential equations, which should be solved using numerical analysis techniques in
order to determine unknown nodal displacements. Depending on the behavior and expected response, one can use linear and nonlinear analyses techniques. Nonlinearities arise due to inelastic constitutive response or large deformations. The first type is referred to as material nonlinearity and the second type as geometrical nonlinearity. One of the main sources of nonlinearity in analysis of the masonry infilled frames arises from the material nonlinearity associated with the frame members or its connections and infill wall components. Another source of nonlinearity is a contact problem, which is usually due to the variable length of contact between the frame and the infill wall throughout the loading history. An additional source of nonlinearity may arise in masonry infill wall material due to cracking and crushing.

In the finite element models developed in this study, both types of mentioned nonlinearities were considered. Material nonlinearities include: nonlinear moment-rotation and force-deformation responses of steel frame connections, equivalent infill wall struts, tie down steel rebar, and SIWIS element. The sources of geometrical nonlinearities include: contact between infill wall and frame and large deformations. Static analyses were conducted on highly nonlinear models with the use of ANSYS finite element commercial program (ANSYS 6.1, ANSYS Inc., 2002). The ANSYS program uses the “Newton-Raphson” procedure for nonlinear analysis, in which the stiffness matrix is updated at every equilibrium iteration. In this method, the load is subdivided into a series of load increments. Before each solution, ANSYS evaluates the out-of-balance load vector, which is the difference between the applied load and the restoring forces. Then, the program conducts a linear analysis and checks for solution convergence. If convergence criteria are not met, the out-of-balance load vector is re-evaluated, the stiffness matrix is updated, and a new solution is achieved. This iterative procedure continues until the solution converges.

In the analyses performed in this study, when convergence was not obtained, smaller load increments were used. A number of convergence-enhancement and recovery features such as “line-search”, “arc-length”, “automatic load stepping”, and “bisection”, were also employed to help the problem to converge. The “line-search” feature of ANSYS multiplies the calculated displacement increment by a program-calculated scale factor, whenever a stiffening response is detected. The “arc-length” method causes the Newton-Raphson equilibrium iterations to converge along an arc, thereby often preventing divergence problems, even when the slope of the load-deflection curve becomes zero or negative. “Automatic time stepping” feature adjusts the time step size to result in better balance between accuracy and economy (cost of run time). “Bisection” feature provides a means of automatically recovering from a convergence failure.
This feature will cut a time step size in half whenever equilibrium iterations fail to converge and automatically restart from the last converged substep. If the halved time-step again fails to converge, bisection will again cut the time step size and restart, continuing the process until convergence is achieved or until the minimum time-step size is reached. The above information is obtained from ANSYS 6.1 Manual (ANSYS 2002).

4.3 Types of Finite Elements Used in this Study

In this section a brief description of the ANSYS finite elements that have been used for modeling different components of the bare steel frame and infilled steel frame with and without the SIWIS system is given. Five different types of finite elements from the ANSYS element library were used for modeling. The BEAM3 element was used to model the two-dimensional elastic frame members including columns and beams. The four-node PLAIN42 element was used to represent the elastic infill masonry wall. In order to simulate the infill wall-frame interaction, the CONTACT12 element was used. The nonlinear spring COMBIN39 element was employed in order to model diagonal struts replacing the masonry infill wall. The same element was used to model the beam-column joint as a rotationally nonlinear spring. For modeling SIWIS elements either the combination CONBIN40 element or the nonlinear COMBIN39 element was considered. The main characteristics of the elements, which were used in this study, are explained next.

4.3.1 The 2D Elastic Beam BEAM3

The BEAM3 element is a uniaxial element with compression, tension, and bending capabilities. This element has three degrees of freedom (DOF) at each node, translations in the nodal x and y directions and rotation about the nodal z-axis. Figure 4.1 shows the geometry, coordinate system, and the node locations for the BEAM3 element. The element input data includes two nodes, the cross-sectional area, the area moment of inertia, the depth or height, and the material properties. The basic solution output associated with this element includes nodal displacements, element forces, and element moments in the element coordinate system. This element was used for modeling of the steel frame members including columns and beams.
4.3.2 The 2D Structural Solid PLAIN42

The PLAIN42 element is a 2D element for use in solid structures. This element can be used either as a plane element (plane stress or plane strain) or as an axisymmetric element. The element is defined by four nodes each with two degrees of freedom, namely, translation in the nodal x and y directions. Figure 4.2 shows the geometry, node locations, and the coordinate system for this element. The element is defined by four nodes, a thickness (for the plane stress option only) and the orthotropic material properties corresponding to the element coordinate directions. The solution output associated with this element includes nodal displacements, elastic strains and stresses, and principal elastic strains and stresses. This element with a plane stress option was used for modeling the masonry wall.
4.3.3 The Nonlinear Spring COMBIN39

The COMBIN39 element is a unidirectional element with nonlinear generalized force-deformation (or moment-rotation) capability in one, two, or three dimensions. The longitudinal option is a uniaxial compression-tension element with up to three DOFs at each node, translations in the nodal x, y, and z directions. No bending or torsion capability is available in this option. The rotational option of the element is purely rotational three DOFs at each node, rotations about the nodal x, y, and z-axis. No bending or axial loads are considered in this option. The element has large deformation capability.

The geometry, node locations, and the coordinate system for this element are shown in Figure 4.3. The input data for this element includes two node points and a generalized force-deformation (or moment-rotation) curve. The solution output includes nodal displacements, relative displacement and element forces, with additional element output for reversed loading.

In the study conducted here, the element with a longitudinal option and two DOFs at each node was used for modeling the diagonal struts in infilled steel frame models and for modeling tie down steel rebar. This element was also used for modeling the SIWIS elements with “flexible-failure” behaviors. This element with rotational option was used for spring modeling of beam-column connections as well.

![Figure 4.3: COMBIN39 element](image-url)
4.3.4 The 2D Point-to-Point Contact Element CONTACT12

The CONTACT12 element represents two surfaces which may maintain or break physical contact and may slide relative to each other. The element has the capability of supporting only compression in the direction normal to the surface and shear in the form of Coulomb friction in the tangential direction. The element has two DOFs at each node, namely, translations in the nodal x and y directions. The element is capable of modeling the gap specification as well. A specified stiffness is defined for the normal and the tangential directions, which acts when the gap is closed and not sliding. This element was used to simulate the infill wall-frame interaction in the model of infilled steel frame with the SIWIS system.

The element is defined by two nodes, an angle $\theta$ which defines the interface, and a normal and a tangential (shear) stiffness $K_N$ and $K_S$, respectively. The element is also given a friction coefficient $\mu$, and an initial element status option (START), which defines the initial condition of the gap and the sliding direction. If the interface is closed and sticking, $K_N$ and $K_S$ are used in the gap resistance and sliding resistance, respectively. If the interface is closed but sliding, $K_N$ is used in the gap resistance and the friction force of $\mu F_N$ is used for the sliding resistance. When the normal direction force $F_N$ is negative, the interface remains in contact and responds as a linear spring. When the normal force becomes positive, contact is broken and no force is transmitted. In the tangential direction, for negative force $F_N$ and the absolute value of the tangential force $F_S$ less than $\mu |F_N|$, the interface sticks with response as a linear spring in the tangential direction. For negative force $F_N$ and $F_S = |\mu |F_N|$, sliding occurs. All these relations can be visualized by referring to Figure 4.4.

This element may have only one of three available conditions, closed and stuck, closed and sliding, or open as explained next.

1. If the interface is closed and stuck, then:

$$F_S < \mu |F_N|$$

and the normal force $F_N$ and the sliding force $F_S$ are given by Equations (4.2) and (4.3):
\[ F_N = K_N (u_{N,i} - u_{N,j} - \delta) \]  \hspace{1cm} (4.2)

\[ F_S = K_S (u_{S,i} - u_{S,j} - u_O) \]  \hspace{1cm} (4.3)

In these equations, \( u_N \) refers to the displacement in the normal direction, \( u_S \) refers to the displacement in the tangential direction, \( \delta \) is the interface gap, and \( u_O \) is the distance that nodes \( i \) and \( j \) have slid with respect to each other.

2. If the interface is closed and sliding, then:

\[ F_S = \mu |F_N| \]  \hspace{1cm} (4.4)

3. If the interface is open, then there is no contact between \( i \) and \( j \) nodes.

The solution output for the CONTACT12 element includes nodal displacement and the normal force in the interface. Additional element outputs such as the relative displacement in the tangential direction and the tangential force can be obtained. The two nodes of the element may be coincident and the interface orientation by the angle \( \theta \) measured from the global X-axis as shown in Figure 4.5. As explained and shown in Figure 4.4, the CONTACT12 is a nonlinear element and therefore iterative solution is required. Furthermore, since the energy lost in sliding cannot be recovered, the load should be applied incrementally (gradually).

Figure 4.4: Force-deformation response for the CONTACT12 element (ANSYS 2002)
4.3.5 The Combination Element COMBIN40

The COMBIN40 element is a combination of a spring-slider and damper in parallel, coupled to a gap in series. A mass can be assigned to one or both nodes. The element has only one degree of freedom (DOF) at each node, either a nodal translation, rotation, pressure, or temperature. The mass, spring, slider, damper, and/or the gap may be removed from the element for specific purposes. This combination element is shown in Figure 4.6. The input data for this element includes two springs $K_1$ and $K_2$, a damping coefficient $C$, a mass $M$, a gap size GAP, and a limiting sliding force FSLIDE. A spring constant of zero, a damping coefficient of zero, or a GAP value of zero will remove these capabilities from the element. A positive GAP results in the existence of a gap, while a negative value introduces an initial interface. If FSLIDE is zero, the sliding capability is removed, which results in a rigid connection. A “break-away” feature is available to allow the spring stiffness $K_1$ to drop to zero once its force ($F_1$) reaches a limiting force of $|FSLIDE|$. The limit is input as $-|FSLIDE|$ for this purpose and it is applicable to both tensile breaking and compression crushing. A “lock-up” feature is also available to remove the gap opening capability once the gap has closed.

The force-deformation response for the element is shown in Figure 4.7. If an initial gap of zero is introduced, the element responds as a spring-damper-slider having both tension and compression capabilities. If the gap is not introduced as zero, the element responds as follow: when the spring experiences compression (force $F_1+F_2$ is negative), the gap remains closed and
the element responds as a spring-damper parallel combination. Once the spring 1 force (F1) increases beyond the FSLIDE value, the element slides and the F1 component of the spring force remains constant. If the FSLIDE is input with a negative value, the stiffness drops to zero and there is no resisting F1 spring force. If the spring experiences tension (force F1+2 is positive), the gap opens and no force is transmitted. The element has only one DOF.

The solution output associated with this element includes nodal displacements with additional element output such as the amount of sliding, force in each spring, relative displacement of each spring, and element status. This element with its spring, break-away feature, and gap specification was used for modeling of the SIWIS elements with “brittle-failure” and “friction-failure” behaviors.

Figure 4.6: COMBIN40 element (ANSYS 2002)

Figure 4.7: Force-deformation response for CONBIN40 element (ANSYS 2002)
4.4 Creation of Modeling Schemes

4.4.1 Selection of Infilled Steel Frame Case Study

A literature review conducted revealed that one of the infill wall systems reviewed by Dawe and Seah (1989) to be a good choice for this purpose since in the works they reviewed, a series of full-scale experiments have also been conducted. The finite element modeling approaches used by other researchers as presented in the literature review (Section 2.4) have also been found to be useful for comparison purposes in the study reported here.

The model creation and validation plan consisted of initially modeling the bare steel frame, then adding brace elements as per the Seah (1998) method (single-diagonal strut model) and the El-Dakhakhni et al. (2003) method (three-diagonal strut model), and finally, developing a model for the infilled steel frame with SIWIS elements. First, a brief review of the experimental program of the selected case study is presented.

4.4.2 Experimental Program of Case Study

Twenty-eight full-scale monotonically loaded single-bay single-story steel frames infilled with concrete block masonry walls were tested at the University of New Brunswick-Canada during the 1980s. The tests were conducted by McBride (1984), Yong (1984), Amos (1985) and Richardson (1986). The experimental program examined a broad spectrum of characteristics in order to isolate those that may have significant effect on the strength, stiffness, and behavior of infilled frame structures.

The typical test setup used for the experimental program is shown in Figure 4.8. The steel frames had the dimensions of 142 in. long by 110 in. high, while the masonry infill walls consisted of 16x8x8 in. concrete blocks placed in running bond confined by a steel frame, which was fabricated using W10x39 columns and a W8x31 roof beam and double W12x35 supporting bottom beam with rigid connections. Detailed descriptions of the specimens and their characteristics are available elsewhere (Dawe and Seah 1989). The frame was subjected to an in-plane load at the top generated by a 1800 KN loading jack.

One specimen, referred to as Specimen WD7 by Richardson (1986), which was classified as a standard specimen in the test series, was selected for this study. The specimen had a concrete block masonry infill wall with standard horizontal joint reinforcement and constructed within the
steel frame with no gaps and ties between the wall ends and the webs of the columns. The masonry prism for this specimen had a compressive strength ($f'_{m}$) of 3680 psi. The steel and masonry wall material properties and steel frame section properties used in the finite element models are summarized in Tables 4.1 and 4.2, respectively. This specimen was loaded to complete failure. Figure 4.9 shows the experimental load-deflection relation for Specimen WD7, along with that of the frame with no infill wall (bare frame).

![Test setup for Specimen WD7 (Dawe & Seah 1989)](image)

Table 4.1: Steel and masonry material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>29000</td>
<td>0.30</td>
</tr>
<tr>
<td>Masonry</td>
<td>2340</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Table 4.2: Steel frame section properties

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Area (in.$^2$)</th>
<th>Moment of Inertia (in.$^4$)</th>
<th>Plastic Modulus (in.$^3$)</th>
<th>Plastic Moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>W10X39 (Y-Y)</td>
<td>11.5</td>
<td>45</td>
<td>17.2</td>
<td>751</td>
</tr>
<tr>
<td>Top Beam</td>
<td>W8X31 (X-X)</td>
<td>9.13</td>
<td>110</td>
<td>30.4</td>
<td>1327</td>
</tr>
<tr>
<td>Bottom Beam</td>
<td>2W12X35 (X-X)</td>
<td>20.6</td>
<td>570</td>
<td>102.4</td>
<td>4471</td>
</tr>
</tbody>
</table>
4.4.3 Modeling of Bare Steel Frame

The ANSYS model shown in Figure 4.10 idealizes the bare steel frame, where 4 BEAM3 elements represented the frame and 4 nonlinear COMBIN39 elements represented the beam-to-column joints. Both coincident nodes of the column and beam at the joint were forced to have the same horizontal and vertical displacement using the DOF COUPLING provided by ANSYS. The trilinear moment-rotation relationship used for joints as proposed by El-Dakhakhni et al. (2003) is shown in Figure 4.11. Monotonically increasing displacement was applied for this model at the top corner of the frame. The resulting load-deflection relation is shown in Figure 4.12 along with the test results.

As can be seen from Figure 4.12, the ANSYS model results accurately match the available test results. The in-plane stiffness of the bare frame model (slope of the load-deflection curve) and its ultimate load are in good agreement with the test results. The reasons for such a good match include: first, the moment-rotation relation of springs in the model was calibrated in such a way to result in the same in-plane frame model stiffness as the experiment. An initial stiffness of 18.4 k/in. is the result for the bare frame. Second, the failure of the actual bare frame occurs by development of four plastic hinges in four beam-column connections, which are represented by four springs in the model. Using a simple plastic analysis on this mechanism, one
can calculate the ultimate load of the system as Equation (4.5), in which, $H$ is the height of frame, and $M_{pj}$ is the plastic moment capacity of joint. Replacing $H = 110$ in. and $M_{pj} = 751$ kips-in., an ultimate load of 27.3 kips results. This is very close to the experimental result, which is 27.2 kips. The plastic moment of the joint is the minimum plastic moment capacity of the beam or column sections.

$$F_{ult} = 4 \left( \frac{M_{pj}}{H} \right)$$  \hspace{1cm} (4.5)

Figure 4.10: ANSYS model for bare steel frame

Figure 4.11: Trilinear moment-rotation response for joints  
(El-Dakhakhni et al. 2003)
4.4.4 Modeling of Infilled Steel Frame

First, the single-diagonal method proposed by Seah (1998) and briefly presented in Section 2.4.2 was used for modeling the infilled steel frame. A nonlinear diagonal compression strut was added to the previous bare steel frame model. But as noted before in Section 2.4.2, nonlinear elements modeling the frame joints were removed to make them hinged connections as seen in Figure 4.13 (a). The simplified four-linear force-deformation curve suggested by Seah (1998) was considered for the force-deformation response of the COMBIN39 element representing the diagonal strut. The force-deformation curve for the equivalent diagonal strut representing the infilled steel frame was obtained based on the panel dimensions and the material properties of the steel frame and masonry infill wall as shown in Figure 4.14.

Next, the proposed method by El-Dakhakhni et al. (2003) presented in Section 2.4.3 was used. For this purpose, three nonlinear diagonal compression struts were added to the bare steel frame model. The developed ANSYS model is shown in Figure 4.13 (b). The force-deformation responses for the three struts were calculated with the use of equations proposed by El-Dakhakhni et al. (2003) based on the geometry and material properties of the case study, Specimen WD7, and shown in Figure 4.15.
Figure 4.13: ANSYS models for infilled steel frame; (a) single-diagonal strut model, (b) three-diagonal strut model

Figure 4.14: Force-deformation response for diagonal strut of single-diagonal strut model of infilled steel frame (Seah 1998)

Figure 4.15: Force-deformation responses for: (a) upper and lower struts, (b) middle strut of the three-diagonal strut model of infilled steel frame (El-Dakhakhni et al. 2003)
Monotonically increasing displacement was applied to these two models. The resulting load-deflection relations along with the experiment results are shown in Figure 4.16. The responses of these two models depend highly on the assumed force-deformation relation for their diagonal struts. As can be seen from Figure 4.16, the responses of the single-diagonal strut model and the three-diagonal strut model are, respectively, four-linear and trilinear, which are related to the four-linear and trilinear response introduced for their struts as shown in Figures 4.14 and 4.15. The three-diagonal strut model is able to predict the initial stiffness fairly well, while the single-diagonal strut model gives a relatively lower stiffness. The resultant stiffness of two models is 270 kips/in. and 160 kips/in., respectively. The ultimate load of the system resulting from the two models is found to be 121 kips and 98 kips, respectively. The actual tested infilled frame’s stiffness and ultimate load were 260 kips/in. and 111 kips, respectively (Richardson 1986). It should be pointed out that one reason for the lower stiffness and ultimate load results for single-diagonal strut model relative to three-diagonal strut model as well as the experimental results is because conservative force-deformation responses for the diagonal brace was used, which was obtained from a series of tables suggested by Seah (1998) for design purposes. In fact, the results of the single-diagonal strut modeling scheme can be considered to give conservative stiffness and strength properties of an infilled steel frame for design purposes.

Figure 4.16: Load-deflection relations for infilled steel frame
4.4.5 Modeling of Infilled Steel Frame with the SIWIS System

The development of the ANSYS model for infilled steel frame with an SIWIS system is presented in this section. The modeling of three main components of an SIWIS system including steel frame, masonry infill wall, and SIWIS element is described in the following. The developed ANSYS model is shown in Figure 4.19.

4.4.5.1 Modeling of Steel Frame

The same bare steel frame developed in Section 4.4.3 was used for the frame. Additionally, two nonlinear springs in the columns were added at the locations of the connection of the SIWIS elements to consider the possibilities of formation of plastic hinges in these locations. The moment-rotation response of these springs is similar to the springs used for the column section as shown in Figure 4.11. A total of 33 BEAM3 elements and 6 COMBIN39 elements were used.

4.4.5.2 Modeling of Masonry Infill Wall

Elastic 6x6-in. PLAIN42 element with plane stress option was used to model the masonry infill wall. CONTACT12 element was used to simulate the wall-frame interaction at the bottom and the two bottom corners (sides) of the wall panel. In this modeling scheme, in order to prevent uplifting of the wall panel, vertical elements (a pair of steel rebars) tie the top of the wall to the bottom beam on each side. The two bottom corners of the wall are assumed to lean against the columns for shear transfer. This system configuration shown in Figure 4.19 is one possible modeling scheme, which is meant to be used in this analytical study. In actual construction, other details can be used for shear (slip prevention) and rotational (tie-down) stabilities of masonry wall panel. For instance, steel rebars need not be used for tie-down; instead, the two vertical members of SIWIS subframe at two sides can be detailed for preventing uplift of the wall panel. However, for the discussion in this study, rebars are assumed to be used for tie-down.

The SIWIS masonry wall’s two steel rebars located on two sides of the wall were modeled by the COMBIN39 element assigning tension only bilinear steel behavior shown in Figure 4.17. A total of 396 PLAIN42 elements, 27 CONTACT12 elements, and 2 COMBIN39 elements were used to model the masonry wall panel.
4.4.5.3 Modeling of SIWIS Element

Several designs for an SIWIS element were introduced and explained in Chapter 3 including; concrete disk, steel disk with epoxy, lumber disk, tension element, frictional device, and jagging and releasing element. In this analytical study, based on the test results or assumed behavior (force-deformation responses), the SIWIS element is categorized into three main groups including; (a) “Brittle-Failure” for concrete disk, steel disk with epoxy, lumber disk, and jagging and releasing element, (b) “Flexible-Failure” for tension element and elements with yielding type failure, and (c) “Friction-Failure” for friction type devices. The finite element modeling of these three types of SIWIS elements is subsequently explained.

For modeling “brittle-failure” SIWIS element, the CONBIN40 element was used. In order to make the element work only in compression, a very small value was used for the GAP specification of the element. A gap of 0.0001 in. was considered for this purpose. Large values for gap may result in loss of accuracy of the model, while too small values may result in a numerical instability and divergence of the solution. A stiffness of 500 kips/in. was assumed for spring 1 (K1). The results of tests conducted on three different compression disk designs for SIWIS elements, one based on concrete disk (Section 5.4), one based on steel disk and epoxy (Section 5.5), and one based on lumber disk (Section 5.6) were the basis for this value of the stiffness of the SIWIS element. For the three representative disks that were compared in Section 5.6.6, initial axial stiffness of about 130 kips/in., 350 kips/in., and 120 kips/in. were, respectively, determined. It is assumed that in the actual fuse product (SIWIS element), which will be shop manufactured, the stiffness of the SIWIS element will be somewhat higher than laboratory test.
conditions. For instance, the adjustment and settling (sitting) of SIWIS element components will not occur in the actual product. For this reason, it was decided to consider an axial stiffness of 500 kips/in. for SIWIS element.

The “Break-Away” feature of the COMBIN40 element was employed to model the breaking of the element and drop the element’s force to zero once it reaches the capacity limit. As the concept and method of the SIWIS system were explained in Chapter 3, the breaking load of the SIWIS element is determined based on the strength of the infill wall. Since the main purpose of the SIWIS system is to save the masonry wall from cracking, the capacity of the SIWIS element needs to be limited to the cracking capacity of the wall panel with an adequate safety margin. For the case study (Specimen WD7), the major cracking load of the infill wall was found to be about 85 kips (Figure 4.9, Richardson 1986). Considering a safety factor of about 4, a limiting capacity of 20 kips is the result for the SIWIS element here. This value was specified for the FSLIDE option. A negative value for this parameter will result in a drop to zero when the force reaches the capacity, while a positive value will result in its yielding and a constant force equal to the capacity.

For modeling the “flexible-failure” element, the nonlinear CONBIN39 element with its longitudinal option was used. Simplified three-linear behavior was considered. Only negative values were introduced for force and deformation couples to release the element’s tension resistance and make it compression only. The same initial stiffness of 500 kips/in. and capacity of 20 kips that were used for “brittle-failure” case, were employed here. It is assumed that the element yields at 20 kips load and 0.04 in. deflection and fails at 20 kips load and 0.16 in. deflection.

For modeling the “friction-failure” element, the COMBIN40 element with a positive value of 20 kips for FSLID was used. The use of a positive value for FSLIDE in the “break-away” feature results in a constant load after reaching the capacity. The force-deformation response associated with the 20 kips capacity SIWIS element with three different behaviors is shown in Figure 4.18. At the same location of the SIWIS element, another COMBIN40 element is used as a gap element to model the possible second contact between the masonry wall and frame once the clearance between them is overcome.
Figure 4.18: Force-deformation responses for SIWIS element; (a) “Brittle-Failure”, (b) “Flexible-Failure”, (c) “Friction-Failure”

Figure 4.19: ANSYS model for infilled steel frame with SIWIS elements
4.5 Single-Bay Single-Story Case Study

In the previous section, the finite element modeling schemes were developed for bare frame, frame with infill wall, and frame with SIWIS system and the analysis results of bare frame and infilled frame were compared to available experimental results. In this section, pushover analysis of the single-bay single-story case study equipped with SIWIS elements using three different types of SIWIS elements is performed. For this purpose, a single monotonically increasing displacement is applied at the top joint.

4.5.1 Analysis Results for Model with “Brittle-Failure” SIWIS Element

The response of the detailed finite element model of infilled frame with “brittle-failure” SIWIS element is shown in Figure 4.20 along with experimental results of bare frame and infilled frame. The response of the infilled frame with SIWIS elements is shown in larger scale in Figure 4.21 along with the bare frame response. In order to observe the effects of varying SIWIS element capacity in response of the system, three different grades for SIWIS elements with capacities of 20 kips, 2X20=40 kips, and 3X20=60 kips are considered and shown in Figure 4.20.

As can be noted in the finite element model shown in Figure 4.19, SIWIS elements are used on both sides of the wall finite element model. For monotonic loading results shown in Figures 4.20 and 4.21, only the SIWIS element on the loaded side will be active (is in compression). As can be seen from Figures 4.20 and 4.21, two stages can be distinguished in the response of infilled frame with SIWIS elements. The first stage or phase is before the breaking of the SIWIS element and the second is after its breaking. These two phases in response are discussed next.

At the first phase shown by line OA in Figure 4.21, the compression side SIWIS element transfers the lateral load to the masonry infill wall. Therefore, the wall participates in the lateral resisting resulting in the highest stiffness for whole system including the steel frame and the masonry wall. At point A on the curve, the failure of the compression side fuse-like SIWIS element occurs at 20 kips in-plane load and corresponding 0.1 in. deflection. Since the capacity of the SIWIS element is 20 kips, one can conclude that for the studied case, almost the whole in-plane load transfers to the masonry wall through SIWIS element. This is because of the high in-plane stiffness of the wall panel relative to that of the steel frame.
The second phase of the response is presented by path BC. As can be seen from Figure 4.21, after the failure of the SIWIS element, at point A, the load-deflection response follows that of the bare frame (path BC) after a load drop (line AB) in the lateral load level. In the second phase, the wall is disengaged from the frame and the bare frame is the only resisting element. This results in lower in-plane stiffness for the whole system in the second phase relative to the first phase.

The two-phase behavior of the single-bay single-story infilled steel frame with SIWIS system and schematic moment diagrams of the frame along with failure mechanism are illustrated in Figure 4.22. As can be seen from this figure, at the first stage of the response, the wall-frame interaction affects the frame moment diagram mainly at its connecting locations to the compression side SIWIS element and to the tensioned steel rebar in the left side (Figure 4.22a). This is because of the fact that the frame is subjected to the concentrated compressive and tensile forces in these two locations, respectively. At the second phase, the frame’s moment diagram is similar to the moment diagram of a bare frame. However, the left column’s support to the wall panel in the bottom part affects the moment diagram (Figure 4.22b). The failure of the infilled steel frame with SIWIS system occurs by development of four plastic hinges in the frame from which two plastic hinges are located at the connection of the top beam to the columns, one plastic hinge is located at the connection of the bottom beam to the right column, and the forth plastic hinge located at the support point of the left column to the panel. Using a plastic analysis on the mechanism shown in Figure 4.22(c), one can calculate the ultimate load of the system as follows:

\[
F_{ult} = 2 \frac{M_{pl}}{H} + 2 \frac{M_{pl}}{H} = 2 \frac{751}{110 - 12} + 2 \frac{751}{110} = 29 \text{ kips}
\]  

(4.6)

The calculated ultimate load is the same as the ultimate load resulting from the computer analysis (Figures 4.20 or 4.21). The ultimate load of the bare frame is found to be 27.2 kips (Richardson 1986). Therefore, the ultimate load of the frame with the SIWIS system in this particular case is about 7% higher than the ultimate load of the frame with no SIWIS system.

It should be pointed out that another stage, Phase 3, may be developed in response if the gap between steel frame and the wall panel closes before the formation of plastic hinges on the frame and its failure. In this case, the connection between frame and the disengaged wall will be re-established and the wall will again take part in resistance. This will result in the postponing of failure of the whole system, which may occur either by development of new plastic hinges on the
frame or by development of cracks on the masonry wall and its failure. The first case will result in a ductile failure, while the second case will result in a brittle failure. Realistically, this third stage may not happen if enough clearance is predicted in the design of the SIWIS element and the subframe.

Figure 4.20: Load-deflection relations for single-bay single-story case study

The response of infilled frame with SIWIS elements can be compared to those of the bare frame and the infilled frame in Figure 4.20. It can be seen that the SIWIS element stiffens the system, but fails if in-plane load exceeds its capacity and after that, the response follows that of the bare frame. From Figure 4.20, it can be seen that the stiffness of infill frame with SIWIS system before failure of SIWIS element is about 195 kips/in. This stiffness is about 10 times that of the bare frame, but 75% of the stiffness of the frame with tightly fitted infill wall. The stiffness of the bare frame and the infilled frame are 18.4 kips/in. and 260 kips/in., respectively (Figure 4.20). In other words, the frame with SIWIS element will deflect 1/10 of that of the bare frame. For example, it can be seen from Figure 4.21 that at load level of 16 kips the story drift is 0.083
in. for the frame with an SIWIS element, while it is 0.88 in. for the bare frame. This considerable decrease in story drift can prevent damage to drift sensitive non-structural elements of the building in wind and minor-to-moderate earthquake events. After failure of the SIWIS element, there will not be an interaction between the frame and the infill wall (in the direction of applied load) unless the clearance in the SIWIS element is overcome by deflection of the frame. It should be noted that after the failure of the SIWIS element, which disengages the frame from the infill wall, the resistance to in-plane load comes from the stiffness of bare frame. In other words, upon the failure of the SIWIS element, the load-deflection curve follows that of the bare frame without any resistance from the infill wall.

![Load-Deflection Relation for Infilled Steel Frame with "Brittle-Failure" SIWIS Element](image)

**Figure 4.21:** Load-deflection relation for single-bay single-story model with “brittle-failure” SIWIS element

It can be seen from Figure 4.20 that the strength provided by SIWIS elements can be increased to desirable levels (i.e., 40 kips and 60 kips), although such a capacity should be kept reasonably below the wall cracking capacity. It can also be clearly seen from Figure 4.20 that the tightly fitted infill walls demonstrate higher ultimate load (capacity) than that of the frame with the SIWIS element. However, it should be pointed out that the main objective of the SIWIS
system alternative is to prevent the masonry wall and the frame from sustaining any damage. The conventional tightly fitted masonry infill walls are often very vulnerable to major cracks and even total collapse in moderate to strong earthquakes (Section 2.2). The tightly fitted infill wall with load-deflection relation shown in Figure 4.20 experienced major cracks at load level of 85 kips (Richardson 1986). On the other hand, the isolated infill wall with SIWIS element will not experience any failure (e.g., cracks) because of the fuse like performance of the SIWIS element. The impact of this advantage can be higher for infill walls of weaker but lighter material (e.g., hollow clay tile units or autoclaved aerated concrete blocks).

Figure 4.22: Behavior and failure mechanism of single-bay single-story infilled steel frame with SIWIS system

As mentioned before, there is a drop in in-plane load level after the failure point associated with breaking of the SIWIS element. From a practical point of view, this may be thought of to cause sudden shocks to the building structure. However, such a response can be compared with the developments of cracks in a concrete frame or a masonry wall under excessive loads. Such behavior during earthquake can be acceptable, particularly in light of the fact that it would come from a controlled failure aimed to prevent damage to the wall and frame. For commercial application of the SIWIS concept, there is further room for fine-tuning the design to minimize the adverse effects of sudden breakage of SIWIS elements. For example, a flexible component can be added to the SIWIS element to absorb any potential shock effects.
Moreover, it should be pointed out that the response shown in Figure 4.21 with a load drop at point A is associated with the introduced “brittle-failure” SIWIS element shown in Figure 4.18(a). This pure “brittle-failure” element is decided to be used in computer modeling as a simplified representative of SIWIS element with compression disk designs. However, it is observed in Chapter 5, for concrete, steel, and lumber disks tests, that there are limited post-peak responses after the peak loads. To observe the influence of these post peak stages in response to the single-bay single-story case study, “trilinear” and “multilinear” behaviors as shown in Figure 4.23 are assumed for the SIWIS element and introduced to the developed computer model. The resulting load-deflection relations are shown in Figures 4.24 and 4.25, respectively.

![Graph](image)

Figure 4.23: Assumed force-deformation responses for SIWIS element
(a) “trilinear” and (b) “multilinear”

As can be seen from these graphs, the sharp drop in load becomes smoother for these cases particularly for the “multilinear” case, which has longer post-peak response. It needs to be mentioned that due to the very high degree of nonlinearity of the model of these two cases, the analysis time extensively increases.
Figure 4.24: Load-deflection relation for single-bay single-story case study with “trilinear” response for SIWIS element

Figure 4.25: Load-deflection relation for single-bay single-story case study with “multilinear” response for SIWIS Element
4.5.2 Analysis Results for Models with “Flexible-Failure” and “Friction-Failure” Elements

The responses of the finite element model for infilled steel frame with “flexible-failure” and “friction-failure” SIWIS elements are, respectively, shown in Figures 4.26 and 4.27 along with results of the model with the “brittle-failure” SIWIS element and the bare frame test results.

As seen from Figures 4.26 and 4.27, the “friction-failure” case gives the highest strength for the system. The maximum load is 49 kips for the “friction-failure” case and 29 kips for other two cases. This is basically due to the fact that the masonry wall is participating in the in-plane resistance for the “friction-failure” case, while it is disengaged from the frame in the other two cases. However, due to the specific mechanism of the “friction-failure” SIWIS element, the transferring load does not increase beyond 20 kips. Thus, the wall is still in the safe zone.

The response of the “flexible-failure” case is generally similar to the response of “brittle-failure” case. The main difference between them is in the failure points of SIWIS elements. For the “flexible-failure” case, the failure points occur at higher in-plane load and deflection than in the “brittle-failure” case. This can be considered to be the main benefit of the “flexible-failure” case over “brittle-failure” system.

From a deflection point of view, it can be observed that after the failure point, the floor deflection in “friction-failure” case is much less than the floor deflection in the other two cases (at a specific load level). For example, after failure of the SIWIS element at a load of 25 kips, the deflections are 0.36, 1.33, and 1.33 in. for “friction-failure”, “flexible-failure”, and “brittle-failure” cases, respectively. However, they have the same paths before the failure of the SIWIS element.
Figure 4.26: Load-deflection relation for single-bay single-story case study with “flexible-failure” response for SIWIS Element

Figure 4.27: Load-deflection relation for single-bay single-story case study with “friction-failure” response for SIWIS Element
4.6 Two-Bay Three-Story Case Study

4.6.1 Case Study Description

In this section, the use of the SIWIS system in a typical two-bay three-story steel frame is discussed. The same modeling approaches described for the single-bay single-story case study were used. The geometry and the member section of the steel frame are shown in Figure 4.28. The panel sizes and their material properties were considered to be the same as the single-bay single-story case study. This decision was made because the experimental results of Specimen WD7 were available for comparison purposes. For the steel frame of the two-bay three-story case, however, heavier frame members were considered. All panels were 144 in. long by 111 in. high and the infill panels consisted of 8x8x16 in. concrete blocks placed in running bond within the frame, which was fabricated using W12x53 columns and W10x30 beams with rigid connections.

![Figure 4.28: Two-bay three-story case study description](image)

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Area (in.²)</th>
<th>Moment of Inertia (in.⁴)</th>
<th>Plastic Modulus (in.³)</th>
<th>Plastic Moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>W12X53</td>
<td>15.6</td>
<td>95.8</td>
<td>29.1</td>
<td>1455</td>
</tr>
<tr>
<td>Beams</td>
<td>W10X30</td>
<td>8.84</td>
<td>170</td>
<td>36.6</td>
<td>1830</td>
</tr>
</tbody>
</table>
4.6.2 Creation of Models

ANSYS models were developed for predicting the behavior for three cases including bare steel frame, infilled steel frame, and infilled steel frame with an SIWIS system. The bare steel frame was first modeled using nonlinear beam-column joints. Then, two different macro models of single-diagonal strut and three-diagonal strut models were considered for infilled frame. Finally, a nonlinear finite element model was developed for infilled steel frame with an SIWIS system.

The developed ANSYS model for the bare steel frame is shown in Figure 4.29(a), in which 17 BEAM3 elements represented the frame and 32 COMBIN39 elements represented the nonlinear joints. Two trilinear moment-rotation relationships, one for beam sections and one for column sections, were used for joints as shown in Figure 4.30.

For modeling the infilled steel frame using single-diagonal strut method, a nonlinear diagonal compression strut was added to each panel of the bare steel frame model (Figure 4.29b). For this model, unlike the model for the single-bay single-story case study, a conservative curve (for design purposes) was not considered for diagonal strut response. Instead, the results of the single-bay single-story infilled steel frame test presented in Figure 4.9 were used to generate it. The simplified force-deformation response generated for the diagonal strut is shown in Figure 4.31. Three nonlinear diagonal compression struts were added to each panel of the bare steel frame model to develop the three-diagonal strut model for infilled steel frame as shown in Figure 4.29(c). The trilinear force-deformation responses of the three struts were generated as shown in Figure 4.32.

Finally, the ANSYS model for infilled steel frame with the SIWIS system was developed as shown in Figure 4.33. The same finite elements and modeling method as for single-bay single-story model were used. A total of 110 BEAM3 elements for the frame, 50 COMBIN39 elements for nonlinear joints, 606 PLAIN42 elements for the masonry wall, 108 CONTACT12 elements for wall and frame connection, 12 COMBIN39/40 elements for SIWIS elements, 12 COMBIN40 elements for gap modeling, and 12 COMBIN39 elements for the steel rebars were used.
Figure 4.29: ANSYS models for two-bay three-story case study; (a) bare steel frame, (b) single-diagonal strut method, and (c) three-diagonal strut method

Figure 4.30: Moment-rotation response for joints

Figure 4.31: Force-deformation response for diagonal strut of single-diagonal strut model
4.6.3 Analysis Results for Model with “Brittle-Failure” SIWIS Elements

Incremental horizontal in-plane displacement was applied to the four developed models at the 3rd floor level. The resulting load-deflection relations for all models are plotted in
Figure 4.34. The failure of the bare steel frame occurs by a plastic mechanism at the ultimate load of 66 kips. In the single-diagonal strut model of infilled steel frame, the model reaches its maximum capacity at about 220 kips load and 3 in. deflection. There is an initial peak at 170 kips load and corresponding 1.3 in. deflection, which is the initiation of major crack/damage of the masonry wall. The load level of 170 kips for major cracking of the infill wall is reasonable. The major cracking load of the masonry wall in the single-bay single-story case study was found to be about 85 kips (Richardson 1986). Therefore, a value of about two times for major cracking load of two panels in the two-bay three-story case study is a reasonable expectation. The three-diagonal strut model for the infilled steel frame resulted in an ultimate load of 302 kips and corresponding deflection of 2.5 in.

![Load-Deflection Relations for Two-Bay Three-Story Model](image)

Figure 4.34: Load-deflection relation for two-bay three-story case study

Similar to the single-bay, single-story case study, three different grades of SIWIS elements with capacities of 20 kips, 20x2=40 kips, and 20x3=60 kips were considered for comparison purposes. The results for the model with SIWIS elements of 20 kips capacity as compared with the bare steel frame response are shown with a larger scale in Figure 4.35. As can be seen from Figure 4.35, four stages can be distinguished in the response of infilled frame with SIWIS system. The four-phase behavior of the system is schematically illustrated in Figure 4.36.
The first stage (or phase) is before breaking of any SIWIS elements (Figure 4.36a). Figure 4.38(b) shows the second phase after breaking of the 3rd story’s SIWIS elements (in both panels). The third phase occurs after the breaking of the SIWIS elements of the 2nd story (Figure 4.36c) and, finally, the fourth phase happens after the breaking of the 1st story’s SIWIS elements (Figure 4.36d). The sequence of SIWIS element failure is also included as an insert in Figure 4.35. The load and corresponding top floor’s deflection for each failure point are 36.3 kips and 0.25 in., 33.8 kips and 0.87 in., and 31.9 kips and 1.48 in., respectively.

Similar to the single-bay single-story case study, there are load drops in response after each failure point associated with breaking of SIWIS elements of each story. After the failure of all the SIWIS elements, the load-deflection response follows that of the bare frame. A fifth phase may be observed if the clearance between the frame and wall overcome before the plastic hinge formations and failure of the whole system. The failure of the system occurs at load level of 78 kips by development of 16 plastic hinges in the frame of which 8 are located in column sections in the beam-column joints, 2 in beam sections in beam-column joints, and finally 6 plastic hinges in the location of masonry wall-column connection.

![Load-Deflection Relations for Infilled Steel Frame with "Brittle-Failurer" SIWIS Elements](image)

**Figure 4.35**: Load-deflection relation for two-bay three-story infilled steel frame with “brittle-failure” SIWIS elements
The response of the frame with SIWIS system can be compared with the responses of bare frame and infilled frame in Figure 4.35. It can be seen that the SIWIS elements stiffen the system before their failures. From the diagrams shown it can be seen that in the first phase of the response, all masonry walls take part in in-plane resistance of the structural system resulting in the highest stiffness of 145 kips/in. In the second, third, and forth phases of the response, the masonry walls of the 3rd, 2nd, and the 1st story are disengaged from structural system resulting in the stiffness of 39 kips/in., 22 kips/in., and 16 kips/in., respectively. The stiffness of the system in the first phase of response is about 11 times the stiffness of the bare steel frame and in the fourth phase, it is 23% higher. Therefore, the story drifts in the frame with SIWIS elements is about 10% and 81% of the story drifts in the bare frame at the 1st and 4th phases, respectively.

![Figure 4.36: Behavior and failure mechanism of two-bay three-story infilled steel frame with SIWIS system](image)

4.6.4 Analysis Results for Models with “Flexible-Failure” and “Friction-Failure” Elements

The response of the two-bay three-story infilled steel frame with “brittle-failure” SIWIS elements was studied in the previous section. The responses of the model with “flexible-failure” and “friction-failure” SIWIS elements are shown in Figure 4.37 along with the response of the model with “brittle-failure” SIWIS elements.

As can be seen from this figure, the “friction-failure” case has the highest ultimate load of about 120 kips. For “brittle-failure” and “flexible-failure” cases, it is 78 kips. This is basically due to the fact that the masonry walls are contributing to the in-plane resistance of the system in the “friction-failure” case, while they are disengaged from the frame in the other two cases. For the “friction-failure” case, the transferring load from frame to wall is theoretically constant equal to
the capacity of the SIWIS element (20 kips here). Thus, although the wall is still participating in
the in-plane resistance, it is still safe.

One advantage of the “flexible-failure” case over “brittle-failure” case is the level of
loads and deflections associated with the breaking of SIWIS elements of the three stories. They
are slightly higher for model with “flexible-failure” SIWIS elements.

From a deflection point of view, the “friction-failure” case has significantly lower floor
deflections (and story drifts), in comparison to the other two cases. This is due to the stiffness
contribution of masonry infill walls in this case. In other two cases, the failure of SIWIS elements
isolated the masonry walls from the frame. Another advantage of the “friction-failure” case over
the other two cases is the fact that there is no load drop in the response.

![Load-Deflection Relation for Infilled Steel Frame with Different SIWIS Element Behaviors](image)

Figure 4.37: Load-deflection relation for two-bay three-story infilled steel frame
with different responses for SIWIS elements

4.6.5 Load Control Analysis

In previous analyses, displacement control loading was considered in order to study the
behavior of the frame with and without SIWIS elements including post-peak responses
(e.g., stiffness degradation and strength deterioration). Displacement control loading is used for
most experimental tests in order to collect more detailed data, since it can better characterize the
response (load-deflection relation over serviceability and ultimate limit states). However, in a real earthquake and for design purposes, load control is a more realistic representation. For these reasons, the two-bay three-story model was subjected to a load control parameter simulating induced earthquake loads. With the use of equivalent static force method, in-plane loads with ratios of 3:2:1 were considered at the 3rd, 2nd, and 1st floor levels, respectively (i.e., $F/2$, $F/3$, and $F/6$). This loading parameter was introduced to the finite element model of the two-bay three-story frame in incremental fashion, and the resulting load-deflection relation is shown in Figure 4.38 along with the sequence of SIWIS element failures in the three stories.

Similar to the displacement control analysis, four phases were observed in the response of the model for load control parameters. The four-phase behavior is schematically shown in Figure 4.39. The first phase is before breaking of any SIWIS elements (Figure 4.39a). The second phase is after the breaking of the 1st story’s SIWIS elements (Figure 4.39b). The third phase occurs after the breaking of the SIWIS elements of the 2nd story (Figure 4.39c) and, finally, the fourth phase occurs after the breaking of the 3rd story’s SIWIS elements (Figure 4.39d). After the failure of all compression side SIWIS elements (left sides), the load-deflection response follows that of the bare frame. A fifth phase may be observed if the clearance between the frame and wall is overcome before the failure of the whole system.

As can be seen from Figures 4.38, the use of the SIWIS system gives considerable advantages to the frame by appreciably reducing in-plane deflections and, thus, story drifts. For example, with reference to Figure 4.38, the deflection of the frame with the SIWIS system, 0.16 in., 0.77 in., 2.20 in., and 3.38 in., respectively, corresponds to the lateral load levels of 30 kips (in the first phase), 42 kips (in the second phase), 60 kips (in the third phase), and 70 kips (in the forth phase). The lateral deflection of the frame without the SIWIS system (bare frame) corresponds to the same lateral loads, 1.79 in., 2.52 in., 3.57 in., and 4.38 in., respectively. Based on these results, at these four load levels, the in-plane deflection of the frame with the SIWIS system is about 9%, 31%, 62%, and 77% of the bare frame.

As explained and shown in Figure 4.39, the sequence of failure of SIWIS elements occurs from the 1st story to the 3rd story in the load control case. This is basically because of larger story shear force in lower stories relative to upper stories coupled with using the same capacity for SIWIS elements in all three stories (20 kips). This sequence of failure of SIWIS elements can result in development of a soft story, which is undesirable for the structure. A preferable sequence of failure of SIWIS elements can be obtained by specifying SIWIS elements with different capacities (different grades) for different elevations. For instance, if the capacity of the SIWIS
elements in the first story is sufficiently higher than that of the second story, the 2nd story’s SIWIS elements will fail before the 1st story’s, which will prevent the formation of a first soft story and so on. The load control analyses are further used in Chapter 6 in order to develop the design approaches for SIWIS system in wind and earthquake loads.

![Load-Deflection Relations for Infilled Steel Frame with "Brittle-Failure" SIWIS Elements (Load Control)](image)

Figure 4.38: Load-deflection relation for two-bay three-story infilled steel frame with “brittle-failure” SIWIS elements (Load Control)

![Behavior and failure mechanism of two-bay three-story infilled steel frame with SIWIS system (Load Control)](image)

Figure 4.39: Behavior and failure mechanism of two-bay three-story infilled steel frame with SIWIS system (Load Control)
4.7 Further Analytical Investigations on SIWIS System

For a better understanding of the behavior of the SIWIS system, its response to different parameters and conditions are analytically investigated in this section. The story drifts and the possibilities of gap closing phenomena are evaluated for the two-bay three-story case study. Beam-column connections with different rigidity levels, such as pinned, semi-rigid, and fully-rigid, are considered. The effects of strength of frame members including beams and columns are investigated. The influences of the locations of SIWIS elements in elevation are studied. Finally, SIWIS elements with varying stiffnesses are considered. In the analytical studies conducted in this section, only “brittle-failure” response for SIWIS elements is considered.

4.7.1 Story Drifts and Gap Closing Mechanism

The load-deflection relations presented in previous sections for the two-bay three-story case study were based on the deflection at the 3rd floor level. In an infilled frame with SIWIS elements, after the masonry walls are disengaged from the frame by failure of the fuse elements, it is possible that a second contact between frame and the disengaged wall occurs if the clearance between them is overcome. Therefore, in order to eliminate the gap closing mechanism, enough clearance between the frame and wall panel should be provided. In this section, the story drifts of the two-bay three-story case study are investigated to observe the possibilities of gap closing.

Figure 4.40 shows the lateral deflections of the three floors of the two-bay three-story case study subjected to displacement control load. In this graph, the horizontal axis represents the in-plane load applied to the frame and the vertical axis represents the in-plane deflection of the floor. The following equations were used to calculate the story drifts for the three stories of the case study:

\[
\begin{align*}
\delta_{10} &= \Delta_1 - \Delta_0 = \Delta_1 \\
\delta_{21} &= \Delta_2 - \Delta_1 \\
\delta_{32} &= \Delta_3 - \Delta_2
\end{align*}
\] 

(4.7)

In this equation, \(\Delta_1\), \(\Delta_2\), and \(\Delta_3\) are deflection of the 1st, 2nd, and 3rd floor levels, respectively, and \(\delta_{10}\), \(\delta_{21}\), and \(\delta_{32}\) are story drifts of the 1st, 2nd, and 3rd stories, respectively. The resultant story drifts versus the applied lateral load are shown in Figure 4.41.
Floors Load-Deflection Relation for Two-Bay Three-Story Infilled Steel Frame with SIWIS System

Load (kip) vs. Deflection (in.)

- 1st Floor (green dashes)
- 2nd Floor (blue dashes)
- 3rd Floor (purple dashes)

Figure 4.40: Floors load-deflection relation for two-bay three-story infilled steel frame with SIWIS system

Story Drifts for Two-Bay Three-Story Infilled Steel Frame with SIWIS System

Load (kip) vs. Story Drift (in.)

- 1st Story (green dashes)
- 2nd Story (blue dashes)
- 3rd Story (purple dashes)

Figure 4.41: Story drifts for two-bay three-story infilled steel frame with SIWIS system
It can be seen from Figure 4.41 that for this typical two-bay three-story frame, the story drifts of the three stories are limited to about 1.8 in. before the curves approach the plastic region at an in-plane load of approximately 70 kips and are limited to about 3.0 in. when the frame fails in load level of about 78 kips. As a result, a minimum gap (clearance) of 1.8 in. should be provided in SIWIS element to prevent gap closing and secondary contact between the frame and the wall. If the gap closes, the wall will be subjected to stress after initial isolation.

### 4.7.2 Frames with Pinned or Semi-Rigid Connections

The behavior of infilled steel frame with SIWIS system with pinned and semi-rigid connections is investigated and compared to previously studied frames with rigid connections.

#### 4.7.2.1 Single-Bay Single-Story Case Study

The typical moment-rotation relation used for the beam-column connections of the frame is shown in Figure 4.42. The slope of the first part of this three-linear relationship is representative of the rotational stiffness (or the level of rigidity) of the connection. With reference to Figure 4.42, for a typical joint, the stiffness is defined as the Equation (4.8), in which, $M_{pl}$ is the plastic moment of joint in kip-in., and $\phi_{el}$ is the elastic yielding rotation of joint in radian.

$$\text{Joint Stiffness} = K_j = M_{pl} / \phi_{el}$$  \hspace{1cm} (4.8)

The stiffness of the joint should be calibrated in such a way that the lateral stiffness of the bare frame model matches that of the actual bare frame, which can be obtained either experimentally or by performing an elastic analysis. For the single-bay single-story case study with rigid connection, a value of 0.0005 radians was obtained (El-Dakhakhni et al 2003), which results in a stiffness of 1500000 kips-in./radian for the joint. In this section, different values for $\phi_{el}$ were considered to account for different stiffnesses for the connections. The considered $\phi_{el}$ values along with corresponding stiffness for connections of single-bay single-story frame are calculated and listed in Table 4.4. These connections were introduced to the developed ANSYS model for the infilled steel frame with SIWIS system and the results of the analysis are shown in Figure 4.43.
Table 4.4: Connection stiffness properties for single-bay single-story case study

<table>
<thead>
<tr>
<th>Case</th>
<th>$\phi_{el}$ (radian)</th>
<th>$K_j$ (kip-in./radian)</th>
<th>Representing Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0005</td>
<td>1500000</td>
<td>Rigid Frame</td>
</tr>
<tr>
<td>2</td>
<td>0.005</td>
<td>150000</td>
<td>Semi-Rigid Frame 1</td>
</tr>
<tr>
<td>3</td>
<td>0.01</td>
<td>75000</td>
<td>Semi-Rigid Frame 2</td>
</tr>
<tr>
<td>4</td>
<td>0.02</td>
<td>37500</td>
<td>Semi-Rigid Frame 3</td>
</tr>
<tr>
<td>5</td>
<td>0.05</td>
<td>15000</td>
<td>Semi-Rigid Frame 4</td>
</tr>
<tr>
<td>6</td>
<td>75</td>
<td>10</td>
<td>Pinned Frame</td>
</tr>
</tbody>
</table>

Figure 4.42: Three-linear moment-rotation response for joints

![Moment (kips-in.)](image)

Figure 4.43: Load-deflection relation for single-bay single-story infilled steel frame with SIWIS system with different connection rigidity

![Load-Deflection Relation for Single-Bay Single-Story Infilled Steel Frame with SIWIS System (Different Connection Rigidity)](image)
As can be seen from Figure 4.43, by increasing joint stiffness, the in-plane stiffness of the frame increases both before and after the breaking of the SIWIS element. For instance, the rigid frame, with $\phi_{el}=0.0005$, has stiffness of 195 kips/in. and 19 kips/in. before and after the breaking point (but before failure), while they are 120 kips/in. and 7 kips/in. for semi-rigid frame with $\phi_{el}=0.05$, respectively.

By increasing the joint stiffness, the SIWIS element breaking point is moving to the left and top side of the curve, which means an increase in the load and a decrease in the deflection. The breaking load and deflection are, respectively, 17 kips and 0.18 in. for pinned connection frame and 19.2 kips and 0.1 in. for full-rigid connection frame, respectively. The ultimate load of the frame for all cases, except pinned connection, is 29 kips. However, the yielding point for the more stiff frames happens at higher load and lower deflection. The ultimate load of frame with pinned connections is 7.7 kips associated with a different mechanism of failure as subsequently explained. The typical mechanism for a frame with SIWIS system was shown in Figure 4.22. The behavior and failure mechanism of a pinned frame is shown in Figure 4.44. As can be seen from this figure, the ultimate state for a frame with pinned connections occurs by development of only one plastic hinge in the left column in the location of support to the masonry wall. The ultimate load can be accurately predicted by a simple plastic analysis which results in the following:

$$F_{ult} = \frac{M_{pj}}{H-h} = \frac{751}{110-12} = 7.7 \text{ kips}$$

(4.9)

![Diagram](image)

Figure 4.44: Behavior and failure mechanism of single-bay single-story infilled steel frame with SIWIS system with pinned connections
4.7.2.2 Two-Bay Three-Story Case Study

The effects of the stiffness of connections for the two-bay three-story case study are examined in this section. Similar to the single-bay single-story frame, different values for $\phi_{el}$ were considered as listed in Table 4.5. The analytical results of the ANSYS models associated with these $\phi_{el}$ values are shown in Figure 4.45. As can be seen from Figure 4.45, similar to the single-bay single-story case, by increasing the stiffness of the connections, the SIWIS elements breaking points (i.e., the 1st point corresponding to the 3rd story’s SIWIS elements, the 2nd point corresponding to the 2nd story’s SIWIS elements, and the 3rd point corresponding to the 1st story’s SIWIS elements breaking) were moved to the left and top of the curve by means of a decrease in deflection and an increase in load level. For instance, the failure load and deflection of the 1st point are 31.7 kips and 0.44 in. for the pinned connection and 37.6 kips and 0.26 in. for the full-rigid connection frame, respectively. The differences become greater for the 2nd and the 3rd points.

Table 4.5: Connection stiffness properties for two-bay three-story case study

<table>
<thead>
<tr>
<th>Case</th>
<th>$\phi_{el}$ (radian)</th>
<th>Joint Stiffness for Columns (kip-in./radian)</th>
<th>Joint Stiffness for Beams (kip-in./radian)</th>
<th>Representing Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0001</td>
<td>14550000</td>
<td>18300000</td>
<td>Rigid Frame</td>
</tr>
<tr>
<td>2</td>
<td>0.005</td>
<td>291000</td>
<td>366000</td>
<td>Semi-Rigid Frame 1</td>
</tr>
<tr>
<td>3</td>
<td>0.01</td>
<td>145500</td>
<td>183000</td>
<td>Semi-Rigid Frame 2</td>
</tr>
<tr>
<td>4</td>
<td>0.02</td>
<td>72750</td>
<td>91500</td>
<td>Semi-Rigid Frame 3</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>14</td>
<td>18</td>
<td>Pinned Frame</td>
</tr>
</tbody>
</table>

The ultimate load for all cases except pinned connection is the same, which is 78 kips. The ultimate load of the pinned frame is 29.6 kips and occurs in a mechanism similar to the single-bay single-story frame with pinned connections. The failure mode of the pinned frame is shown in Figure 4.46. As can be seen from this figure, the ultimate state for the pinned frame occurred by development of two plastic hinges in two columns of the left panel of the first story in the locations where columns lean against masonry walls. The ultimate load can be accurately predicted by simple plastic analysis as:

$$F_{ult} = 2 \frac{M_{pl, column}}{H - h} = 2 \frac{1455}{111 - 12} = 29.4 \text{kips}$$

(4.10)
Figure 4.45: Load-deflection relation for two-bay three-story infilled steel frame with SIWIS system with different connection rigidity (stiffness)

Figure 4.46: Behavior and failure mechanism of two-bay three-story infilled steel frame with SIWIS system with pinned connections
4.7.3 Frames with Different Member Strengths

4.7.3.1 Single-Bay Single-Story Case Study

In this section, the effects of strength of frame members (columns and beams) on the response of the system are investigated. Besides the case study frame with W12X53 columns and W10X30 beam sections, two stronger frames with heavier columns and beam sections and two weaker frames with lighter columns and beam sections were considered. The section properties of the frame members including section, area, moment of inertia, plastic modulus, and plastic moment are listed in Table 4.6. These cases were introduced to the computer model and the resulting load-deflection relations are illustrated in Figures 4.47.

As can be seen from Figure 4.47, the in-plane stiffness of the system and its ultimate load increase by increasing the strength of frame sections. The breaking points also occurred in higher load and lower deflection.

![Load-Deflection Relation for Single-Bay Single-Story Infilled Steel Frame with SIWIS System (Different Frame Strengths)](image)

Figure 4.47: Load-deflection relation for single-bay single-story infilled steel frame with SIWIS system with different frame strengths
Table 4.6: Column and beam sections properties for single-bay single-story case study

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Area (in.²)</th>
<th>Moment of Inertia (in.⁴)</th>
<th>Plastic Modulus (in.³)</th>
<th>Plastic Moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>W10X22</td>
<td>6.49</td>
<td>11.4</td>
<td>6.1</td>
<td>305</td>
</tr>
<tr>
<td></td>
<td>W10X30</td>
<td>8.84</td>
<td>16.7</td>
<td>8.84</td>
<td>442</td>
</tr>
<tr>
<td></td>
<td>W10X39</td>
<td>11.5</td>
<td>45.0</td>
<td>17.2</td>
<td>860</td>
</tr>
<tr>
<td></td>
<td>W10X49</td>
<td>14.4</td>
<td>93.4</td>
<td>28.3</td>
<td>1415</td>
</tr>
<tr>
<td></td>
<td>W10X60</td>
<td>17.6</td>
<td>116</td>
<td>35.0</td>
<td>1750</td>
</tr>
<tr>
<td>Beams</td>
<td>W8X10</td>
<td>2.96</td>
<td>30.8</td>
<td>8.87</td>
<td>444</td>
</tr>
<tr>
<td></td>
<td>W8X15</td>
<td>4.44</td>
<td>48.0</td>
<td>13.6</td>
<td>680</td>
</tr>
<tr>
<td></td>
<td>W8X31</td>
<td>9.13</td>
<td>110</td>
<td>30.4</td>
<td>1520</td>
</tr>
<tr>
<td></td>
<td>W8X40</td>
<td>11.7</td>
<td>146</td>
<td>39.8</td>
<td>1990</td>
</tr>
<tr>
<td></td>
<td>W8X58</td>
<td>17.1</td>
<td>228</td>
<td>59.8</td>
<td>2990</td>
</tr>
</tbody>
</table>

4.7.3.2 Two-Bay Two-Story Case Study

Similar to the single-bay single-story case, two stronger frames (heavier columns and beams sections) and two weaker frames (lighter columns and beams sections) were introduced to the ANSYS model. The section properties of columns and beams used in this section are listed in Table 4.7. The results of the analyses are shown in Figure 4.48.

Table 4.7: Column and beam sections properties for two-bay three-story case study

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Area (in.²)</th>
<th>Moment of Inertia (in.⁴)</th>
<th>Plastic Modulus (in.³)</th>
<th>Plastic Moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>W12X30</td>
<td>8.79</td>
<td>20.3</td>
<td>9.56</td>
<td>478</td>
</tr>
<tr>
<td></td>
<td>W12X45</td>
<td>13.2</td>
<td>50</td>
<td>19.0</td>
<td>950</td>
</tr>
<tr>
<td></td>
<td>W12X53</td>
<td>15.6</td>
<td>95.8</td>
<td>29.1</td>
<td>1455</td>
</tr>
<tr>
<td></td>
<td>W12X65</td>
<td>19.1</td>
<td>174</td>
<td>44.1</td>
<td>2205</td>
</tr>
<tr>
<td></td>
<td>W12X87</td>
<td>25.6</td>
<td>241</td>
<td>60.4</td>
<td>3020</td>
</tr>
<tr>
<td>Beams</td>
<td>W10X12</td>
<td>3.54</td>
<td>53.8</td>
<td>12.6</td>
<td>630</td>
</tr>
<tr>
<td></td>
<td>W10X22</td>
<td>6.49</td>
<td>118</td>
<td>26.0</td>
<td>1300</td>
</tr>
<tr>
<td></td>
<td>W10X30</td>
<td>8.84</td>
<td>170</td>
<td>36.6</td>
<td>1830</td>
</tr>
<tr>
<td></td>
<td>W10X45</td>
<td>13.3</td>
<td>248</td>
<td>54.9</td>
<td>2745</td>
</tr>
<tr>
<td></td>
<td>W10X60</td>
<td>17.6</td>
<td>341</td>
<td>74.6</td>
<td>3730</td>
</tr>
</tbody>
</table>
Similar behavior as observed in response of the single-bay single-story frame can be seen for the two-bay three-story frame. By increasing the strength of the frame, the breakings of SIWIS elements happened in lower deflections and higher loads and the whole system demonstrated higher ultimate load and higher stiffness for all phases.

![Load-Deflection Relation for Two-Bay Three-Story Infilled Steel Frame with SIWIS System (Different Frame Strengths)](image)

Figure 4.48: Load-deflection relation for two-bay three-story infilled steel frame with SIWIS system with different frame strengths

### 4.7.4 Effects of Location of SIWIS Element

The influences of the location of the SIWIS element in the response of two-bay three-story case study are examined in this section. The vertical position of the SIWIS element (i.e., vertical distance from the center of the SIWIS element to the top of the masonry wall) was varied. Four positions (elevations) including right at the corner of the wall (d=0 in.), one foot below the top of the wall, which is the standard case (d=12 in.), two feet below (d=24 in.), and three feet below (d=36 in.) were considered. The load-deflection relations for these cases are illustrated in Figures 4.49. As can be seen from this figure, by lowering the location of the SIWIS elements the stiffness of the frame decreases. This is especially visible for the 1st phase of response before breaking of any SIWIS elements. Lower position of SIWIS elements result in
higher breaking loads as well. This observation leads to the conclusion that higher location for SIWIS elements is preferred. However in practice, because of the dimensions of the SIWIS elements, some room is required to locate the SIWIS elements. Therefore, the practical case is decided to be the 2nd case with d=12 in.

![Load-Deflection Relation for Two-Bay Three-Story Infilled Steel Frame with SIWIS System (Different SIWS Element Location)](image)

Figure 4.49: Load-deflection relation for two-bay three-story infilled steel frame with SIWIS system with varying location for SIWIS element

### 4.7.5 Effects of Stiffness of SIWIS Element

Effects of stiffness of SIWIS element on the response of the two-bay three-story case study are discussed in this section. Four different values of 50, 100, 500, and 1000 kips/in. were considered for SIWIS element stiffness. The resulting load-deflection relations are shown in Figure 4.50.

As can be seen from Figure 4.50, lower stiffness for SIWIS element results in lower in-plane stiffness for whole system. The ultimate load is the same for all cases, as expected. However, the failure points occur in higher loads and deflections for the case with lower SIWIS element stiffness. The deflection and load associated with the first failure point and the in-plane stiffness of the system at the first phase are listed in Table 4.8 for comparison purposes. As can be
seen from this table, the stiffness of SIWIS elements significantly affects the response of the system including the stiffness of the system and failure points.

![Load-Deflection Relation for Infilled Steel Frame with SIWIS System with Different Stiffness for SIWIS Elements](image)

**Figure 4.50:** Load-deflection relation for two-bay three-story infilled steel frame with SIWIS system with different stiffness for SIWIS elements

<table>
<thead>
<tr>
<th>Stiffness of SIWIS Elements (kips/in.)</th>
<th>Deflection of the 1st Failure (in.)</th>
<th>Load of the 1st Failure (kips)</th>
<th>Stiffness of System at the 1st Phase (kip/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>1.38</td>
<td>55.9</td>
<td>41</td>
</tr>
<tr>
<td>100</td>
<td>0.75</td>
<td>45.6</td>
<td>61</td>
</tr>
<tr>
<td>500</td>
<td>0.26</td>
<td>37.7</td>
<td>145</td>
</tr>
<tr>
<td>1000</td>
<td>0.20</td>
<td>36.4</td>
<td>182</td>
</tr>
</tbody>
</table>
4.8 Finite Element Modeling of Test Frame

In this section, a finite element model is created for the two-bay three-story steel test frame using the finite element modeling schemes developed in this chapter. The main purpose is to compare the experimental observations with the numerical simulation and perform final validation of the modeling schemes. First, a bare steel frame model including columns, beams, and bracing elements was developed. Then, the masonry walls and SIWIS elements were added to the model of the bare frame to develop the model for infilled steel frame with SIWIS system. The scaled two-bay three-story test frame and its experimental program is comprehensively described in Section 5.7.

The columns’ and beams’ properties used in the modeling are summarized in Table 4.9. BEAM3 elements were used for the columns and beams. For modeling the diagonal bracing elements of the frame, the nonlinear COMBIN39 element with its longitudinal option was used. The results of a rod pulling test were used to define the force-deformation response for the bracing elements. The multi-linear force-deformation response shown in Figure 4.51 was considered for the diagonal bracing elements in the finite element model.

The developed ANSYS model for test infilled steel frame with SIWIS system is shown in Figure 4.52. Elastic PLAIN42 and CONTACT12 elements were used to simulate the masonry wall and wall-frame interaction. For modeling SIWIS element using a lumber disk, the nonlinear CONBIN39 element was used. The simplified six-linear responses shown in Figure 4.53 were considered for SIWIS elements using lumber disks with different thicknesses. The results of the lumber disk test presented in Section 5.6 were used to generate these responses. The validation of this model is presented in Section 5.7.

![Figure 4.51: Simplified multi-linear force-deformation response for diagonal bracing elements](image)

Figure 4.51: Simplified multi-linear force-deformation response for diagonal bracing elements
Table 4.9: Column and beam section properties for two-bay three-story test frame

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Area (in.²)</th>
<th>Moment of Inertia (in.⁴)</th>
<th>Plastic Modulus (in.³)</th>
<th>Plastic Moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>TUBE 5X5X3/8</td>
<td>6.58</td>
<td>22.8</td>
<td>11.2</td>
<td>515</td>
</tr>
<tr>
<td>Beams</td>
<td>TUBE 5X5X3/8</td>
<td>6.58</td>
<td>22.8</td>
<td>11.2</td>
<td>515</td>
</tr>
</tbody>
</table>

Figure 4.52: ANSYS model for test infilled steel frame with SIWIS elements

Figure 4.53: Simplified six-linear force-deformation responses for SIWIS elements
4.9 Summary and Concluding Remarks

A nonlinear two-dimensional finite element modeling scheme was developed for analysis of infilled steel frames equipped with SIWIS elements. First, a single-bay single-story steel frame with a tightly fitted infill wall that has been the subject of past experimental and analytical studies was considered. Once the modeling scheme for the single-bay single story frame was preliminarily validated by comparing its results with the available test results, finite element models for a typical two-bay three-story steel frame were similarly developed to study the use of SIWIS system in multi-bay multi-story frames. Then, the two models were subjected to pushover and cyclic analyses under different parameters and conditions. To further expand understanding about the system, a parametric study was conducted on these two models. Finally, a finite element model was developed for the scaled two-bay three-story test frame in order to compare the analytical results with the experimental observations, which led to the validation of the modeling schemes. The nonlinear finite element modeling scheme created for infilled steel frames with SIWIS elements is a general modeling method and can implement different behaviors for their components, particularly SIWIS element, such as “brittle-failure”, “flexible-failure”, “friction-failure”, “multilinear-curve” and so on.

The finite element modeling results confirm that the concept of the proposed SIWIS system works as a “seismic isolation” system for infilled frames to save the infill wall and the frame from any damage. The “fuse” type SIWIS element, first, utilizes the beneficial stiffness effects of the infill wall up to a predefined point to reduce the frame drift. Then, it completely isolates the infill wall from the frame or limits the participation of infill walls in in-plane resistance.

Based on the number of stories and the number of failures of SIWIS elements, several phases (stages) can exist in the response of a frame with an SIWIS system. In-plane stiffness of the frame decreases by progressive failure of SIWIS elements and phases. After failure of all of SIWIS elements, the response follows that of a bare frame. The initial in-plane stiffness of the frame with an SIWIS system can be considerably higher than that of a bare frame. This will significantly reduce the frame story drift, which is desirable for drift sensitive components of the building. There are load drops in the in-plane load level after each failure point associated with breaking of the SIWIS elements of each story, which can be compared with the developments of cracks in concrete or masonry structures. Such behavior during an earthquake is considered acceptable, particularly in light of the fact that it would come from a controlled failure aimed to
prevent damage to the wall and frame. SIWIS elements with more ductile responses demonstrate smoother in-plane resistance load drops, while those with sudden brittle failure result in more sudden load drops.

The response of the SIWIS system is influenced by characteristics of the SIWIS element and frame, particularly by their stiffness and strength properties. By increasing connection stiffness, the in-plane stiffness of the system increases, as expected, and the SIWIS elements break under higher frame load and lower in-plane deflection. By increasing the strength of the frame, SIWIS elements fail under lower frame deflections and higher loads and the whole system demonstrated higher ultimate load and higher stiffness. Lower stiffness for the SIWIS element results in lower in-plane stiffness for whole system. However, the ultimate load is the same, as expected. The failure of SIWIS elements occur under higher frame loads and deflections for less stiff SIWIS elements.

The SIWIS system can be used for semi-rigid and pinned frames as well. This system has considerably more beneficial effects in less stiff frames such as pinned and moment-resisting frames than in more stiff frames such as braced frames or frames that include shear walls. It is possible that a secondary contact between the frame and the disengaged wall occurs if the clearance between them is overcome. Therefore, sufficient gap between masonry wall and frame should be specified. The gap size depends on the level of induced earthquake loads and frame stiffness and strength properties as well as the design of SIWIS element.
Chapter 5

EXPERIMENTAL PROGRAM

5.1 Overview of Experimental Program

An extensive experimental program was planned and carried out in the Building Envelopes Research Laboratory (BERL) of the Architectural Engineering Department at Pennsylvania State University to primarily test the concept of the proposed SIWIS system. The experimental results were also used to provide the information needed to further confirm the validity of the finite element models.

The main tests included a series of static tests on a scaled two-bay three-story steel frame. A total of 11 tests were conducted on the frame with and without infill walls and with and without SIWIS elements. Prior to these main tests, several other experimental tasks were conducted as described next. Two single plain masonry walls identical to those walls used in the two-bay three-story frame were pushed until failure occurred in order to investigate their behaviors and failure modes. The ultimate load (strength) of the walls was obtained and used to determine proper capacities for SIWIS elements for the frame tests. Another experimental task was pulling tests on three rods of different materials and base connections identical to those of the test frame. The performance of connection was evaluated and the most suitable rod was selected for the diagonal member of the test frame. The performance of three different designs for SIWIS element including concrete disk, steel disk and epoxy, and lumber disk was comprehensively investigated. For the concrete disk option, first, 4 concrete disks were preliminarily tested. After verification of the idea, a total of 24 concrete disks with 8 different thicknesses were tested. For the steel disk and epoxy option, 4 tests were performed in 2 configurations including tension case and shear case. Finally, for the lumber disk design, first, 12 lumber disks from three lumber types were tested. Then, a parametric test was conducted on a total of 24 lumber disks with 6 different thicknesses. Results of these SIWIS element tests led to the design of disks for frame tests. The experimental tests and their primary and secondary objectives are summarized in Table 5.1. The details of the experimental program are presented for each test in this chapter.
<table>
<thead>
<tr>
<th>Test Name</th>
<th>Test Description</th>
<th>Primary Objective</th>
<th>Secondary Objectives</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Masonry Wall Test</td>
<td>2 pushover tests on 2 single brick wall specimens</td>
<td>Measure the in-plane strength of a typical masonry wall specimen</td>
<td>- Test appropriateness of wall’s configuration and connections</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Study behavior and failure mode of wall</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Provide information for masonry wall strength prediction (empirical method)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Measure in-plane stiffness of wall</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Evaluate the controllability on loading history using available test facilities</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Measure in-plane stiffness of wall</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Evaluate the controllability on loading history using available test facilities</td>
</tr>
<tr>
<td>Rod Pulling Test</td>
<td>3 pulling tests on 3 different rods</td>
<td>Select the most suitable rod for main frame test diagonal member</td>
<td>- Measure tensile strength of rods</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop tensile force-deflection relations</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Test performance of designed pinned connections for main frame test</td>
</tr>
<tr>
<td>SIWIS Element (SE) Test using Concrete Disk</td>
<td>Preliminary test: Compression test on 4 concrete disks</td>
<td>Test the concept of compression concrete disk design</td>
<td>- Measure capacity of disks</td>
</tr>
<tr>
<td></td>
<td>Expanded test: Compression test on 24 concrete disks with 8 different thicknesses</td>
<td>Parametric study on capacity of concrete disk in SIWIS element</td>
<td>- Study behavior and failure modes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Evaluate performance of PVC support</td>
</tr>
<tr>
<td>SIWIS Element Test using Steel Disk and Epoxy</td>
<td>4 compression tests on 2 tension and 2 shear cases</td>
<td>Test concept of steel and epoxy application in SIWIS element</td>
<td>- Measure strength and stiffness properties</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Study behavior and failure modes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Select the better case (tension or shear)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop load-deflection relations</td>
</tr>
<tr>
<td>SIWIS Element Test using Lumber Disk</td>
<td>Preliminary test: Compression test on 12 disks with 3 different lumber types</td>
<td>Test idea of using lumber disk in SIWIS element</td>
<td>- Measure strength and stiffness properties</td>
</tr>
<tr>
<td></td>
<td>Expanded test: Compression test on 24 hard Maple disks with 6 different thicknesses</td>
<td>Parametric study on capacity of lumber disk in SIWIS element</td>
<td>- Evaluate behavior and failure modes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Select the most appropriate lumber type</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop load-deflection relations</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop rod strain-load relations</td>
</tr>
<tr>
<td>Two-Bay Three-Story Frame tests</td>
<td>Bare frame: 3 fully braced &amp; 1 half braced frame tests</td>
<td>Measure the in-plane stiffness of bare frame</td>
<td>- Develop load-deflection relationships</td>
</tr>
<tr>
<td></td>
<td>SIWIS braced frame: 4 tests on fully braced with 2 SIWIS element patterns and 1 test on half braced frame</td>
<td>Test the concept of SIWIS system and validate FE models</td>
<td>- Calibrate/verify FEM computer models</td>
</tr>
<tr>
<td></td>
<td>SIWIS pinned frame: 2 tests on 2 SIWIS element patterns</td>
<td>Compare performances of infilled frame with and without SIWIS system</td>
<td>- Test frame setup and configurations</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Study responses of system</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>- Investigate and compare responses to different SIWIS element patterns</td>
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<td></td>
<td></td>
<td>- Calibrate/verify FEM computer models</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop load-deflection relationships</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Observe performances of masonry walls</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Observe and study performance of system in pinned frames</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Develop load-deflection relationships</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Calibrate/verify FEM computer models</td>
</tr>
</tbody>
</table>
5.2 Single Masonry Wall Test

5.2.1 Introduction

In the proposed SIWIS system, the masonry infill wall needs to be isolated from the frame in strong earthquakes in order to prevent damage to the wall panel including cracking and collapse. Therefore, the SIWIS element needs to be designed in such a way to fail well before the masonry wall panel reaches its capacity. For the two-bay three-story frame test, single layer of masonry brick walls were constructed as infill walls. This experiment was planned and conducted to mainly evaluate the in-plane strength of these masonry brick walls.

5.2.2 Materials

Two masonry wall specimens constructed using solid clay bricks with actual dimensions of 7-5/8x3-1/2x2-1/4 in. as shown in Figure 5.1 were tested. An angle section was used to prevent the base sliding of the wall panel. A vertical system was designed to tie the top of the wall panel to the base and prevent the uplift of the masonry wall panel. The two masonry wall specimens were randomly chosen from a total of twelve identical masonry wall specimens that have been constructed with the same material and at the same time by two expert masons. The rest of the walls were used in the two-bay three-story frame tests. A general description of a typical wall specimen is shown in Figure 5.2, which shows a 9-course running bond masonry wall with four bricks in each course resulting in a total height of 24-1/4 in., a total length of 32 in., and an actual thickness of 3-1/2 in. Type “N” general purpose mortar was used. Figure 5.3 shows a photo of a typical wall panel. Three 5-course masonry prisms were also constructed and tested in order to measure the compressive strength of masonry material.

![Figure 5.1: Solid clay brick (left: actual dimensions, right: photo)]
Two 30 in.-long 6x4x½ angle sections were used; one to prevent the base sliding of the wall panel and the other one to attach the vertical tie system to the base. These two angle sections were bolted to the base foundation each with two 1-3/8 in. anchor bolts. Two 36 in.-long ½”-13
threaded steel rods with high strength material of Grade B7 were used as vertical elements to tie the top of the wall panel to the base angle section. This grade material meets ASTM A193 and is normally made from AISI 4140 alloy steel with a minimum tensile strength of 125,000 psi. These two threaded steel rods were bolted to a plate assembly at the top including one 7x6x1/2 in. and one 7x2x1/2 in. plates. At the bottom, each steel rod was bolted to a smaller 4 in.-long 4x1-3/4x1/2 angle section, and this smaller angle section was itself bolted to the vertical leg of the base angle section by two ½”-13 bolts. The vertical tie system was employed only in one side of the wall where the wall panel had a tendency to uplift due to the in-plane load at the top (loading side). The steel plates were all Carbon C1018 steel and the angle sections were A36 steel. The ½”-13 bolts and nuts were all Grade 5.

5.2.3 Equipment

The main equipment used in this experiment included; a testing platform, a hydraulic loading jack and pump, and load control and data acquisition systems as explained next.

5.2.3.1 Testing Platform

A testing platform consisting of a reinforced concrete foundation and a steel support (reaction) frame was designed and constructed. The test platform and reactor frame are schematically shown in Figure 5.4. The steel support (reaction) frame consists of two arms each including a W14x211 column section and a W14x211 inclined section. Two arms are connected with two W10x88 horizontal beams at two different elevations. The support frame is constructed on a strong reinforced concrete foundation with a height of 36 in. (3 ft.), width of 60 in. (5 ft.), and length of 216 in. (18 ft.). A hydraulic jack can be connected to a vertical W12x190 section, which is connected to the two horizontal members. Four different locations were considered for two different loading jacks. A photo of the testing platform is shown in Figure 5.5. The design drawings and some photos of construction of this testing platform are in Appendix C.

5.2.3.2 Hydraulic Loading Cylinder and Pump

An RRH-606 type hollow plunger double acting (pushing and pulling) cylinder and a pump from ENERPAC Company was used in this experiment to exert in-plane pushing load. The
jack has a maximum capacity of 140 kips (64 tons) in pushing and 92 kips (42 tons) in pulling. The hydraulic jack has a maximum stoke length of 6.5 in.

![Diagrammatic view and plan of test platform](Image)

5.2.3.3 Load Control and Data Acquisition Systems

The loading history was controlled by the amount of hydraulic pressure in the cylinder. A manual two-bottom, push and pull (advance and retract), controller (joystick) was used to control the hydraulic pressure (Figure 5.6). A pressure transducer was connected to the cylinder to
measure the pressure of the hydraulic flow. Signals from the pressure transducer and two external LVDTs were sent to a computer and were monitored during the test by Lab-View, a computer program. The station used for this experiment including the controller and the computer is shown in Figure 5.7.

Figure 5.5: Photo of test platform

Figure 5.6: Manual hydraulic pressure controller
5.2.4 Test Setup, Instrumentation, and Loading

The design of test setup is shown in Figure 5.8. As explained before, one angle section was used to prevent base sliding (left angle) and another angle section was used as base support for the vertical tie system (right angle). Two smaller angle sections bolted to the right angle section were used to support the two steel rods. A concentric load was applied at the centroid of the wall thickness at the assumed elevation of the SIWIS element at the right side, which was 4 in. below the top of the wall. An extended arm mechanism consisting of a heavy bolt and a ¾ in. thick circular steel plate with 4 in. diameter was attached to the stroke of the cylinder (1) to reach to the wall, (2) to distribute the load over a larger area on the wall, and (3) to accommodate the installation of two external LVDTs. At the left bottom corner of the wall, between left angle section and the masonry wall panel, a 1/16 in. thick rubber was inserted to prevent direct contact between the steel angle section and the masonry wall and reduce the possibility of stress concentration on the masonry wall, which could ultimately lead to premature crushing failure of the masonry. The photo of the typical test setup is shown in Figure 5.9.

The in-plane movements of the stroke of the loading cylinder were measured using two LVDTs. The two LVDTs were attached to the horizontal center line of a large plate (instrumentation plate) at equal distances from its center. The instrumentation plate was attached
to the stroke of the loading cylinder at its center. The in-plane deflection of the stroke of the loading cylinder was taken to be the average of the recorded displacements by the two LVDTs. The two LVDTs were firmly held in horizontal position using two magnetic holders. The magnetic holders were themselves attached to the steel reaction frame. As mentioned before, a pressure transducer was used to record the pressure of the hydraulic flow. Figure 5.10 shows a photo of the top of the hydraulic cylinder where the two LVDTs and pressure transducer can be seen.

It was planned to use incrementally increasing load in this experiment. The loading history was manually controlled by controlling the flow of the hydraulic pressure with the use of a 2-button (push and pull) joystick. The loading histories are presented in Section 5.2.5.2.

Figure 5.8: Diagrammatic view of test setup design
Figure 5.9: Photo of typical test setup

Figure 5.10: Photo of top view of loading cylinder and instrumentation
5.2.5 Results and Discussions

5.2.5.1 Tests Observations and Failure Modes

Two masonry wall specimens were tested under pushing in-plane load. Both wall specimens failed in brittle manner right after formation of the first cracks. There were no preliminary or early cracks and all cracks were developed simultaneously. Figures 5.11 and 5.12 show the failure modes of the two wall specimens, respectively. For more clarity, the crack patterns are schematically shown in Figure 5.13. As can be seen from these figures, the failure mode for both wall specimens was diagonal shear failure. The main diagonal cracks extended from the point where the pushing load was applied (top right) to the opposite corner where the base angle section had supported the wall panel (bottom left). In both specimens, the cracks passed both mortar joints and bricks. However, the majority of cracks were located in mortar joints including horizontal (bed) and vertical (head) joints. Figure 5.12 shows that the upper part of the wall Specimen-2 was thrown to the left side after complete separation from the rest of the wall. No signs of crushing were observed in compression zones of the masonry wall specimens.

The vertical system including threaded steel rods and their top and bottom connections performed well. No damage or failure was observed in the threaded steel rods and connections. As can be seen from Figure 5.11, the vertical steel rods experienced small bending due to the movement of the top plate assembly to the left side, which was due to the presence of friction between the top plate and the brick surfaces. It can be seen from Figure 5.11 that the angle section also experienced some bending due to the high vertical pulling load of the rods. However, these tensile and bending forces were not high enough to cause plastic deformations in either threaded rods or the angle sections, and after releasing the load they returned to their original states. The maximum resisting loads (capacities) of the two masonry wall specimens are listed in Table 5.2 along with the duration of each test (the time of peak load). The ultimate load of the masonry wall Specimens-1 and 2 were about 22.3 kips and 24.3 kips, respectively. An average of 23.3 kips is considered for the in-plane strength of the typical masonry wall specimen.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Time (second)</th>
<th>Capacity (kip)</th>
<th>Standard Deviation (kip)</th>
<th>Coefficient of Variation (CV%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen-1</td>
<td>199</td>
<td>22.3</td>
<td>1.4</td>
<td>6.1</td>
</tr>
<tr>
<td>Specimen-2</td>
<td>103</td>
<td>24.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 5.11: Failure mode of wall Specimen-1

Figure 5.12: Failure mode of wall Specimen-2

Figure 5.13: Crack patterns of two wall specimens
5.2.5.2 Load-Deflection Relations

The load-deflection relation is shown in Figure 5.14 for wall Specimen-1. The load and deflection histories for wall Specimen-1 are also shown in Figures 5.15 and 5.16, respectively. Because of a technical problem, no displacement data were obtained from the test on wall Specimen-2. Thus, for wall Specimen-2 only loading history is included in Figure 5.15. The load-deflection curve in Figure 5.14 shows a nearly linear relation. The slope of this curve, which represents the in-plane stiffness of the masonry wall, is about 45 kips/in. It was intended to apply constant loading rate. However, the loading histories in Figure 5.15 show some jaggedness in the curves, which illustrates the difficulty of keeping the loading rate constant with the use of a manual controller. Furthermore, there is a significant difference between the durations of the two tests. The two wall specimens reached their capacities in 199 and 103 seconds, respectively. This again can be attributed to some degree to the difficulty of having accurate control over the loading history using manual controller. However, it should be mentioned that regardless of the issues related to the loading rate, the static test results (e.g., failure load) obtained in this experiment are valid.

Figure 5.14: Load-deflection relation for wall Specimen-1
Figure 5.15: Load-time relation (loading history) for wall specimens

Figure 5.16: Deflection-time relation for wall Specimen-1
5.2.5.3 Masonry Prism Test

Three 5-course stack bond masonry prisms were constructed and tested according to ASTM C1314-02, which resulted in a specified compressive strength of 3600 psi. The complete report of the masonry prism test is included in Appendix D.

5.2.6 Concluding Remarks

Based on the results of the conducted tests, the following conclusions are highlighted:

1. The two masonry wall specimens presented a nearly linear response followed by a brittle and sudden shear failure.
2. The failure mode of the two masonry wall specimens was diagonal shear failure. No signs of masonry crushing were observed.
3. Maximum loads (strengths) of 22.3 and 24.3 kips resulted from the two masonry wall specimens with an average of 23.3 kips.
4. An in-plane stiffness of about 45 kips/in was measured for wall Specimen-1.
5. The wall test setup was satisfactory. The vertical system including the threaded steel rods and their bolted connections to the top plate and bottom angle sections performed well.
6. An average of 3600 psi resulted from the specified compressive strength of masonry by testing three masonry prisms.
7. Accurate control over the loading rate was found to be a challenge due to the use of a manual controller.

5.3 Rod Pulling Test

5.3.1 Introduction

For the two-bay three-story steel frame, the beam-column and base connections of test frame were designed to be hinged and diagonal elements were considered in each of the six panels to provide stability and necessary in-plane stiffness for the frame. One practical and
An economical option for these brace elements was the use of steel or plastic rods. An experiment was planned and conducted to evaluate this concept as explained in this section.

Three different rods including; threaded steel rod, threaded plastic rod, and plain plastic rod were subjected to pulling tests. The rods were attached to a special connection, which was designed to simulate the connection between diagonal braces and a beam-column joint in the steel frame. The design of the test setup is schematically shown in Figure 5.17 and a photo of its connection is shown in Figure 5.18.

A total of four steel plates, two angles, one steel bar (pin), and one coupling nut were used to create the connection for each test. A thick steel plate bolted to the bottom ram of the load frame provided a relatively rigid base. Two angle sections were bolted to the base plate on their horizontal legs and to two plates (edge plates) on their vertical legs. A high strength steel bar was used as a pin between the two edge steel plates and a middle steel plate, which was used to connect the testing rod specimen to the steel bar pin. The configuration shown in Figures 5.17 and 5.18 creates a pinned connection, where the rod assembly is restrained at the support against displacement, but is free to rotate around the pin. A coupling nut welded to middle plate is used to connect the rod specimen to the connection.

Figure 5.17: Schematics of test setup design
5.3.2 Materials

Three rods with different materials and diameters were used for this experiment. A total of four steel plates, three coupling nuts (one for each test), and one steel bar pin were used for connection. These pieces are explained next.

A 48 in.-long 3/8”-16 high strength threaded steel rod of Grade B7 was one considered option. This grade material meets ASTM A193 and is normally made from AISI 4140 alloy steel with a minimum tensile strength of 125,000 psi. The second rod was 48 in.-long ½”-13 threaded plastic. This plastic fiberglass material has a minimum Rockwell hardness of R199 and minimum tensile strength of 70,000 psi. The third rod was a 52 in.-long ¾ in. plain plastic rod. Using die tools, a thread of 10 threads/in. was created at the two ends of this plain rod to lengths of 3 in. This opaque white plastic fiberglass material has a minimum tensile strength of 30,000 psi lengthwise and 7,000 psi crosswise. These three rods are shown in Figure 5.19.

The base plate was a thick 10x6x1-1/2 in. plate to provide a relatively rigid base. The two 3x3x1/2 angles were connected to the two edge 6x6x3/8 in. plates through their vertical legs and to the base plate through their horizontal legs. The middle plate was 6x3x1/2 in. steel plate. The pin was a 12 in.-long 1-1/2 in.-diameter steel bar (rod) made from high strength stress relieved 1144 alloy. All bolts were ⅝”-13 Grade 5 as were the nuts. A total of three Grade 2 coupling nuts,
one for each rod specimen, were used including; a ¼"-10 for plain plastic rod, a ½"-13 for threaded plastic rod, and a 3/8"-16 for threaded steel rod. The 3/8"-16 and ½"-13 coupling nuts are shown in Figure 5.20.

![Three rod specimens](image)

**Figure 5.19:** Three rod specimens (left: 3/8”-16 threaded steel rod, middle: ¾”-10 partially threaded plastic rod, ½”-13 right: fully threaded plastic rod)

![Coupling nuts](image)

**Figure 5.20:** 3/8”-16 and ½”-13 coupling nuts

### 5.3.3 Equipment

The main equipment used in this experiment included; a testing load frame and a data acquisition system. The tension test was performed on a MTS servo-hydraulic load frame. The loading fixtures consisted of two stainless steel rods. They were attached to the upper frame of the machine equipped with a load cell at the top and to the actuator at the bottom. The load cell had a capacity of 100 kips. The displacement of the bottom piston was measured using an internal ± 5.0 in. LVDT. An MTS 458 controller with 458.91 microprofiler was used to control the load frame. Signals from the internal LVDT and the load cell were utilized by an MTS 458.2 Micro
Console to adjust the flow of hydraulic fluid. Lab-View computer program was employed to monitor and record the deformation from the LVDT and the load frame load cell.

5.3.4 Test Setup, Instrumentation, and Loading

The tension arrangement of the test is shown in Figure 5.21. The rod specimen was held in a vertical position. It was attached to the bottom connection by a coupling nut. At the top of rod specimen, a reduced coupling nut was used to adapt the rod specimen to the top ram of the load frame. As mentioned before, the total axial deformation of the system (rod specimen and connection) was measured using the LVDT by recording the vertical movement of the load frame piston. The internal LVDT had a $\pm 5.0$ in. stroke length. The applied load was also measured using the 100-kip capacity load cell. Incrementally increasing load in the form of displacement was considered in this experiment. MTS 458.91 microprofiler was employed to control loading history. 0.02% of LVDT stroke length was to be applied in one second. This results in a loading rate of 0.001 in./sec.

![Figure 5.21: Tension test setup on MTS load frame](image)
5.3.5 Results and Discussion

5.3.5.1 Tests Observations and Failure Modes

Three tension tests were conducted on the three rod specimens including a threaded steel rod, a threaded plastic rod, and a plain plastic rod with only the ends threaded. The two plastic rods failed on their threads at the top reduced coupling nuts, thus, the strengths of the rods were not fully reached. The threads of the rods failed and the rod slid inside the reduced coupling nut. The failure modes and failed parts of the three specimens are shown in Figure 5.22.

For the steel rod, the failure occurred by yielding of the rod in the middle of its length. The necking phenomenon can be clearly seen in Figure 5.22. The threads in the steel rod performed well. The test results showed that the designed connection had the required strength. No deficiencies were observed in any parts of the connection. The pin bar, which was subjected to heavy bending moment and shear forces, performed well. There were no bearing or tear out failures in the 3/8 in. and ½ in. thick plates, which were representing walls of the tube sections of the two-bay three-story steel frame and diagonal brace’s connector plate, respectively. No damage was observed in the threads of the coupling nuts or the steel rod, which demonstrates the adequacy of strength of their steel threads. However, the threads of plastic rods failed. Finally, the weld between coupling nut and the connector plate performed well.

Figure 5.22: Failure modes of three rod specimens (left: steel rod, middle: ½” plastic rod, right: ¾” plastic rod)
5.3.5.2 Load-Deformation Relations

The resulting load-deformation relations for the three tests are plotted in Figure 5.23. In this figure, the horizontal axis presents the total deformation of the system, which was measured by the movement of the piston of the testing load frame and recorded by the internal LVDT. The vertical axis is the measured tension load by the load cell at the top ram of the load frame.

As can be seen in Figure 5.23, the steel rod test resulted in a nearly bi-linear response with initial stiffness of about 28 kips/in. and stiffness of about 1 kips/in. after yielding. The steel rod started yielding at about 7.2 kips tension load and 0.3 in. axial deformation. As can be seen from Figure 5.23, at the tension load of about 2.6 kips and deformation of 0.1 in., there was a small slide close to 0.01 in., which can be because of a limited thread slide or adjustment of connection pieces. The steel rod failed at tension load of about 8.5 kips and corresponding axial deformation of about 1.2 in. The fully threaded \( \frac{1}{2}''\)-13 rod test also showed a nearly linear response as shown in Figure 5.24 with stiffness of about 15 kips/in. Unlike the steel rod test, for this test the rod prematurely failed in its threads, and the failure was of a brittle type. The threads slid inside the top reduced coupling nut at a load level of about 1.7 kips and deformation of 0.13 in. Similar to steel test, there was an early limited slide in load and deformation of 1.1 kips and 0.09 in., respectively. For the plain \( \frac{3}{4} \) in. plastic rod with only two ends threaded, the response was linear with stiffness of about 26 kips/in. up to 5 kips load and 0.2 in. axial deformation. Then, apparently, there was an early thread failure and slide between deformations of 0.2 in. and 0.28 in., which resulted in a total slide of 0.08 in. At this point, a new thread engagement was formed and the final failure occurred at a load of about 7 kips and deformation of about 0.44 in. in a brittle manner. The initial stiffness and the ultimate resistance of the three tests are summarized in Table 5.3.

<table>
<thead>
<tr>
<th>Test</th>
<th>Initial Stiffness (kip/in.)</th>
<th>Resistance (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8” Steel Rod</td>
<td>28</td>
<td>8.5</td>
</tr>
<tr>
<td>1/2” Plastic Rod</td>
<td>15</td>
<td>1.7</td>
</tr>
<tr>
<td>3/4” Plastic Rod</td>
<td>26</td>
<td>7</td>
</tr>
</tbody>
</table>
5.3.5.3 Theory

The theoretical strength and initial stiffness of a rod can be estimated as Equations (5.1) and (5.2), respectively, in which, $D$ and $L$ are the diameter and length of the rod specimen; $n$ is the number of threads per inch; $\sigma_i$ is the tensile strength of the rod specimen material, and $E$ is the Modulus of elasticity of the rod specimen material.

\[
P_u = \frac{\pi}{4} \left( D - \frac{0.9743}{n} \right)^2 \sigma_i
\]
\[
K_i = \frac{EA}{L} = \frac{E\pi}{4L} \left( D - \frac{0.9743}{n} \right)^2
\]

The tensile strength of the three rod materials are available, while for steel only modulus of elasticity is available. By substituting the material properties and dimensions into Equations (5.1) and (5.2), the following values result for the ultimate strength and initial stiffness:
For threaded steel rod:

\[ P_u = \frac{\pi}{4} \times \left( \frac{3}{8} - \frac{0.9743}{16} \right)^2 \times 125 = 9.7 \text{ kips} \]

\[ K_e = \frac{29000\pi}{4 \times 54} \left( \frac{3}{8} - \frac{0.9743}{16} \right)^2 = 41.6 \text{ kips/in.} \]

For threaded plastic rod:

\[ P_u = \frac{\pi}{4} \times \left( \frac{1}{2} - \frac{0.9743}{13} \right)^2 \times 70 = 9.9 \text{ kips} \]

For plain plastic rod:

\[ P_u = \frac{\pi}{4} \times \left( \frac{3}{4} - \frac{0.9743}{10} \right)^2 \times 30 = 10.0 \text{ kips} \]

These theoretical values are compared with the test results in Table 5.3. It can be seen that there are some differences between them. The sources of these differences are discussed subsequently for each test. The initial test stiffness of the steel rod is about 67% of the theoretical value. The reason for this lower initial stiffness is due to presence of deformations in other parts of the system such as the bending in the pin bar and adjustment deformation (setting deformation) of connection parts. The test results for tensile strength of the steel rod is about 88% of the calculated value. The fact that the steel rod was not perfectly aligned in the test setup coupled with possibility of the presence of some bending moment and particularly torsion can be the sources of this lower tensile strength. The steel rod was slightly forced at the top connection in attaching to the loading cell at the top ram. For the plastic rods, the failure occurred in the threads and thus, the full strength of rod material was not reached. Therefore, Equation (5.1) is not applicable for these two tests.

5.3.6 Concluding Remarks

Based on the results of this experimental program, the following conclusions are reached:

1. The designed connection, which represented the connection between diagonal bracing rods and beam-column joints, performed well.
2. The weld between coupling nuts and connector plates provided necessary strength.
3. The pin steel bar showed enough strength for bending moment and shear forces.
4. The two plastic rods demonstrated poor performances by failure in their threads.
5. The steel rod was chosen to be the best case among the three rod specimens.
6. The threads in the steel rod and coupling nuts performed well and the failure occurred by yielding of the rod material in its middle length, not in threads.
7. Bi-linear behavior is assumed for the diagonal steel rod to be used in the computer modeling of the two-bay three-story test frame.

5.4 SIWIS Element Test Using Concrete Disk

5.4.1 Introduction

A preliminary experimental test was planned and conducted to test the concept of compression concrete disk as the failing component in an SIWIS element. A SIWIS element including three separate parts; a steel rod for exerting force, a concrete disk intended to fail under a specific load and an open cylindrical polyvinylchloride (PVC) piece as the support was proposed. For preliminary SIWIS element tests, four concrete disks with different disk and notch thickness were tested. The whole system worked well as a fuse element for the SIWIS element, and therefore, the proposed concept was verified. The complete report and results of the preliminary experimental test using a concrete disk is detailed in Appendix E. Following the preliminary test, an experimental parametric study was planned and conducted to more comprehensively study the behavior and capacity of concrete disks of different thicknesses. Figure 5.24 shows the schematic and photo of an SIWIS element using a concrete disk that was used in this experiment. In this section, the expanded test program on SIWIS element using concrete disks is presented.

5.4.2 Materials

Concrete disk specimens were molded using Quickrete. A steel pipe, a steel seat disk, a steel end plate, and a threaded steel rod were used in this experiment as explained next.
5.4.2.1 Concrete Disks

A total of 8 different thicknesses of 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, and 2.50 in. were considered for concrete disk specimens. Three disk specimens were tested for each thickness, which is constituted a “set”. Thus, a total of 24 disks in 8 sets were tested. All disk specimens were molded using the same concrete mixture. Molds were constructed using a PVC pipe, a plastic end cap, and a wood rod. A single wood rod with a 1 in. diameter and 3/8 in. height was used, which resulted in a uniform notch for all concrete disks specimens. The plastic caps were tightly attached to the PVC pipes. This created a closed end for molds and eliminated leakage of concrete mix while casting. After casting the concrete disks, they were kept wet for about 1 week and after 4 weeks they were stripped from the molds. A typical disk specimen is shown in Figure 5.25.

Figure 5.25: A 1.50 in. concrete disk specimen
5.4.2.2 Steel Rod, Pipe, Seat Disk, and End Plate

A 1 in. diameter fully threaded steel rod with length of 2-3/4 in. was used to locate in the notch on the top of the concrete disk and exert the compression load on the center of the disk. A 1-5/8 in. long steel pipe with an outside diameter of 4 in. and wall thickness of 1/8 in. was used as support piece. A hollow thin steel disk was used as the seat for the concrete disk. This seat disk, which itself sat on the steel pipe, provided a larger support area for the concrete disk to eliminate punching and penetration of the concrete disk into the steel pipe. A 3x3x1 in. steel plate to be attached to the end of steel rod was used. The center of this plate was drilled and tapped for 1”-14 screw size to accommodate the attachment to the threaded rod. Another advantage of this threaded system is that the whole length of the SIWIS element can be adjusted by rotating the rod. However, sufficient thread engagement between end plate and rod should be provided to eliminate thread failure. Photos of these pieces are shown in Figure 5.26.

Figure 5.26: Steel rod, steel pipe, steel seat disk, and steel end plate
5.4.3 Equipment

Compression tests were performed on an Instron-1350 servo-hydraulic load frame as shown in Figure 5.27. The loading fixtures consisted of two stainless steel rods. They were attached to the upper frame of the machine equipped with a load cell at the top and to the actuator at the bottom. The load cell had a capacity of 20 kips. The displacement of bottom piston was measured using a ±2.5 in. internal linear variable differential transformer (LVDT).

An MTS 458 controller with 458.91 microprofiler was employed to control the load frame. Signals from the internal LVDT and the load cell were utilized by MTS 458.2 Micro Console to adjust the flow of hydraulic fluid. A 16 channel 2100 system signal conditioner/amplifier was used to calibrate strain gages. The strain data from three strain gages, the deformation from LVDT, and the load from the load cell were monitored during the test by a Lab-View computer program. The controller, the channel, and the computer are shown in Figure 5.28.

Figure 5.27: Instron 1350 load frame and compression test setup
5.4.4 Test Setup, Instrumentation, and Loading

The compression arrangement of the test is shown in Figure 5.27. The SIWIS element was held in vertical position with the pipe piece located on bottom ram of the load frame. All SIWIS element pieces should be placed in the center of testing load frame. The wide flange sections bolted to test table, seen in this photo, were used for different experiments.

The total axial deformation of the SIWIS element was measured using the internal LVDT by recording the vertical movement of the load frame's bottom piston. The internal LVDT had a ±2.5 in. stroke length. The applied load was also measured using the 20-kip capacity load cell attached to the top ram. Two electrical resistance strain gages (ERSG) were attached to two opposite sides of the steel rod in its middle height in order to record its longitudinal strains. Locations of these gages were locally sanded to create flat surfaces. However, these gages were damaged after performing 5 tests. Therefore, strain data are available for only 5 disk specimens.

Incrementally increasing load in the form of displacement controlled was used in this test. The MTS 458.91 microprofiler was used to control loading history. Eight percent of the LVDT stroke length was considered for application in one second. This resulted in a loading rate of 0.002 in./sec (0.08% x 2.5 in. = 0.002 in.). This loading rate was chosen to be consistent with the assumed loading rate in the main two-bay three-story frame test.

Figure 5.28: Load control and data acquisition systems
5.4.5 Results and Discussion

5.4.5.1 Test Observations and Failure Modes

Compression tests were conducted on the 24 concrete disk specimens in 8 sets, each set including 3 identical disk specimens. All 3 disk specimens of each set showed relatively similar behaviors. The development of cracks and failure modes are, representatively, shown for one concrete disk specimen of 1.00 and 2.00 in. sets in Figures 5.29 to 5.30, respectively.

Two different behaviors were observed; one for disk specimens 1.50 in. and thinner and one for disk specimens 1.75 in. and thicker. The failure mode of the first group (sets 0.75, 1.00, 1.25, and 1.50 in.) was combination of two basic failures as subsequently explained. One failure mode was a circular sloped fracture starting from the edges of the bottom of notch on the top of the disk, where the steel rod pushed, and ending at the edges of the steel seat disk on the bottom of the disk. This fracture mode let the steel rod to go through the disks, in punching mode. The second failure mode was the fracture of the whole disk into 3 to 5 main parts. This failure mode can be seen in Figure 5.29.

In disk specimens 1.75, 2.00, 2.25, and 2.50 in., the failure mode was different. First, slopped cracks were developed at an angle of about 45 degrees starting from the notch of the concrete disk where the steel rod pushed. However, due to the greater thicknesses of disks and their geometry, these cracks ended at the surfaces of disks where the disk reached the edges of the steel seat disk. By increased displacement, these cracked parts fell away and the remainder of concrete disk looked like a cone as can be seen in Figure 5.30. At this stage, the steel rod compressed the cone shape concrete disk, similar to a concrete cylinder compression test.

Figure 5.29: Typical failure mode of 1.00 in. disk specimens
5.4.5.2 Capacity Results

The disk specimens’ ultimate loads (capacities) are summarized in Table 5.4 and showed in charts in Figure 5.31. As can be seen from Table 5.4, for sets 1.25, 2.25, and 2.50 in., coefficients of variations were under 10% while for the rest of the sets it was larger than 15%. For sets 0.75 in. and 2.00 in., it was 32.5% and 24.1%, respectively, which is an indication of non-uniform and inconsistent statistical results. The relatively large coefficient of variation in these tests can be due to the fact that the concrete and its construction may not provide a well homogenized and uniform material in thin castings like in these concrete disk specimens.

The average capacity was calculated to be 400, 880, 1330, 2320, 4830, 6820, 7610, and 9170 lbs for disk specimens with thicknesses of 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, and 2.50 in., respectively. The capacity of disk specimens increased by thickness as expected. The average capacities are shown in comparison fashion in charts in Figure 5.32. In order to observe the influences of the thicknesses of disk specimens in their capacities as well as the distribution of test results, for all twenty four disk specimens the capacity-thickness relation is shown in Figure 5.33. In this graph, the best fitting linear and polynomial curves are included. As can be seen from these two figures, there is not a good linear relationship between capacity of disk specimen and its thickness for all ranges of thicknesses and the polynomial relationship is a better representation. As mentioned before, the behavior and failure modes were different for disks thicker than 1.50 in. and those thinner or equal to 1.50 in. For these two groups, capacity-thickness relation is separately presented in Figure 5.34, which shows fairly well linear fits for these two groups.
Table 5.4: Capacities of concrete disk specimens

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Total Thickness (in)</th>
<th>Effective Thickness (in)</th>
<th>Capacity (lb)</th>
<th>Average (lb)</th>
<th>Standard Deviation (lb)</th>
<th>Coefficient of Variation (CV%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75 # 1*</td>
<td>0.75</td>
<td>0.375</td>
<td>530</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75 # 2</td>
<td>0.75</td>
<td>0.375</td>
<td>390</td>
<td>400</td>
<td>130</td>
<td>32.5</td>
</tr>
<tr>
<td>0.75 # 3</td>
<td>0.75</td>
<td>0.375</td>
<td>270</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00 # 1*</td>
<td>1.00</td>
<td>0.625</td>
<td>740</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00 # 2</td>
<td>1.00</td>
<td>0.625</td>
<td>1060</td>
<td>880</td>
<td>164</td>
<td>18.6</td>
</tr>
<tr>
<td>1.00 # 3</td>
<td>1.00</td>
<td>0.625</td>
<td>840</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.25 # 1</td>
<td>1.25</td>
<td>0.875</td>
<td>1340</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.25 # 2</td>
<td>1.25</td>
<td>0.875</td>
<td>1320</td>
<td>1330</td>
<td>14</td>
<td>1.1</td>
</tr>
<tr>
<td>1.25 # 3</td>
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<td>0.875</td>
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</tr>
<tr>
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<td>1.125</td>
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</tr>
<tr>
<td>1.50 # 2**</td>
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<td>1.125</td>
<td>2550</td>
<td>2320</td>
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</tr>
<tr>
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<td>1.375</td>
<td>4220</td>
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<td></td>
<td></td>
</tr>
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<td>5810</td>
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</tr>
<tr>
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<td>2.00</td>
<td>1.625</td>
<td>8460</td>
<td>6820</td>
<td>1642</td>
<td>24.1</td>
</tr>
<tr>
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<td>1.625</td>
<td>6910</td>
<td></td>
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</tr>
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<td>1.875</td>
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<td>1.875</td>
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<td>2.50</td>
<td>2.125</td>
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<td></td>
<td></td>
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<td>2.50 # 2</td>
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<td>2.125</td>
<td>9220</td>
<td>9170</td>
<td>761</td>
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<td>2.50</td>
<td>2.125</td>
<td>8380</td>
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</tr>
</tbody>
</table>

Note: * No rubber was used for these disk specimens, ** No data was available for this disk specimen.

Figure 5.31: Capacities of disk specimens
Figure 5.32: Specified (average) capacities of concrete disk specimens

Figure 5.33: Capacity-thickness relation for disk specimens
5.4.5.3 Load-Deflection Relations

The resulting load-deflection relations for the twenty-four tests are separately plotted for each set in graphs in Figure 5.35. These graphs show that although the three disk specimens of each set had relatively similar responses, they were not identical and did not afford well matched curves.

The difference between the responses of the two groups can be clearly seen in the load-deflection relations in Figure 5.36. For the first group (set 0.75 to 1.50 in.), the maximum load reached when the first major crack occurred. Thereafter, there were local load drops and increases with lower amplitudes relative to the first major crack. On the other hand, for the second group, the maximum load reached after the first major crack took place. Thus, the concrete disks in this group presented higher strength after the first major crack. These higher strengths were associated with the compression strength of coned shape concrete.
Figure 5.35: Load-deflection relation for disk specimens
5.4.5.4 Stiffness of SIWIS Element

The axial load-deflection relations presented in the previous section exhibit relatively linear initial starts followed by occurrences of cracks and several drops and increases in resisting load. The slope of the initial linear stage was calculated from load-deflection curves and summarized in Table 5.5 and shown in the charts of Figure 5.36. As can be seen from Table 5.5, the coefficients of variations were under 10% (except for the 0.75 in. disk specimens). This indicates that the initial stiffness of the SIWIS element in this experiment is relatively consistent. The initial stiffness was increased by greater thickness of the concrete disk as expected. The average initial stiffness is shown in comparison fashion in charts in Figure 5.37. In order to observe the effects of thickness of disk specimens in initial stiffness as well as the distribution of test results, for all disk specimens (except those with no rubber used in their tests), the initial stiffness-thickness relation is shown in Figures 5.38. As can be seen from this figure, there is a relatively well defined linear relationship between initial stiffness and thickness of the disk specimen.

![Figure 5.36: Initial stiffness of disk specimens](image-url)
Table 5.5: Initial stiffness of SIWIS element

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Total Thickness</th>
<th>Effective Thickness</th>
<th>Initial Stiffness</th>
<th>Average</th>
<th>Standard Deviation</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(in.)</td>
<td>(in.)</td>
<td>(kip/in.)</td>
<td></td>
<td>(kip/in.)</td>
<td>CV%</td>
</tr>
<tr>
<td>0.75 # 1*</td>
<td>0.75</td>
<td>0.375</td>
<td>40</td>
<td>19</td>
<td>4.2***</td>
<td>22.3***</td>
</tr>
<tr>
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<td>0.375</td>
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<td>0.375</td>
<td>16</td>
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</tr>
<tr>
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<td>52</td>
<td>3.5***</td>
<td>6.7***</td>
</tr>
<tr>
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<td>56</td>
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<td></td>
</tr>
<tr>
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</tr>
<tr>
<td>1.25 # 1</td>
<td>1.25</td>
<td>0.875</td>
<td>55</td>
<td>52</td>
<td>4.2</td>
<td>8.2</td>
</tr>
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<td>0.875</td>
<td>49</td>
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<tr>
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<td>1.25</td>
<td>0.875</td>
<td>NA**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50 # 1*</td>
<td>1.50</td>
<td>1.125</td>
<td>78</td>
<td>75</td>
<td>NA***</td>
<td>NA***</td>
</tr>
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</tr>
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<td>1.375</td>
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<td>9.7</td>
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<td>1.375</td>
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</tr>
<tr>
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<td>2.00</td>
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<td>1.875</td>
<td>109</td>
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<td></td>
<td></td>
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<tr>
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<td>2.25</td>
<td>1.875</td>
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<tr>
<td>2.25 # 3</td>
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<td>1.875</td>
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<td></td>
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<td>2.125</td>
<td>132</td>
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<td></td>
</tr>
<tr>
<td>2.50 # 2</td>
<td>2.50</td>
<td>2.125</td>
<td>128</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.50 # 3</td>
<td>2.50</td>
<td>2.125</td>
<td>127</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* No rubber was used for these disk specimens.
** No data was available for this disk specimen.
*** Only those disks with rubber were considered.
Figure 5.37: Average initial stiffness of concrete disk sets

Figure 5.38: Stiffness-thickness relation for disk specimens
5.4.5.5 Rod Strain-Load Relations

Two strain gages were used to monitor the longitudinal strains along the steel rod. A typical axial strain versus axial load curve is illustrated in Figure 5.39 for one disk specimen. As can be seen from this graph, there are differences between histories of two strains in two opposite sides of the rod, which is an indication of the presence of some bending moment or distortion in the steel rod. Even in some cases, the steel rod experienced tension in one side and compression in the opposite side, which was hard to predict.

Also it can be seen from all graphs that the slopes of these strain-load curves were changed several times throughout the test. This indicates that the direction of the bending of the rod changed during the test by progressive cracking and fracture of concrete disk. The two strains of the steel rod can be expressed by the following equations:

\[
\varepsilon_1 = \left(\frac{P}{A} + \frac{M}{S}\right)\frac{1}{E} \\
\varepsilon_2 = \left(\frac{P}{A} - \frac{M}{S}\right)\frac{1}{E}
\]

(5.3)  
(5.4)

In these equations, \(\varepsilon_1\) and \(\varepsilon_2\) are strains in two opposite sides of the steel rod; \(P\) and \(M\) are the axial force and the bending moment perpendicular to the axial direction of the rod; \(E\) is the modulus of elasticity, and \(A\) and \(S\) are the cross sectional area and modulus of steel rod, respectively. By averaging these two strains, the bending moment terms cancel out and the following two equations are obtained.

\[
\varepsilon = \frac{\varepsilon_1 + \varepsilon_2}{2} = \frac{P}{EA} \\
\text{Slope of } \varepsilon - P \text{ curve} = \frac{\varepsilon}{P} = \frac{1}{EA}
\]

(5.5)  
(5.6)

The average strain was calculated and included in the graph in Figure 5.39. As can be seen from this figure, the average strain-load relation was quite linear, which is to be expected due to the linear response of the steel rod. From the available strain data, the average slope of the strain-load curve was approximated and listed in Table 5.6. For comparison purposes, the theoretical value for this slope was also calculated using Equation (5.6) and listed in Table 5.6. A
cross sectional area of 0.63 in$^2$ and an assumed modulus of elasticity of 29000 ksi were considered. It can be seen from Table 5.6 that there is good agreement between theoretical and experimental values.

Table 5.6: Slope of $\varepsilon - P$ curves for steel rod

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Test $\varepsilon - P$ slope (1/lb x 10$^{-6}$)</th>
<th>Test Average (1/lb x 10$^{-6}$)</th>
<th>Standard Deviation (1/lb x 10$^{-6}$)</th>
<th>Coefficient of Variation</th>
<th>Theoretical $\varepsilon - P$ slope (1/lb x 10$^{-6}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75 # 1</td>
<td>0.053</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00 # 1</td>
<td>0.058</td>
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<td>1.50 # 1</td>
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<td>0.00324</td>
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</tr>
<tr>
<td>1.50 # 2</td>
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<td></td>
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</tr>
<tr>
<td>1.50 # 3</td>
<td>0.053</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.39: Rod strain-load relation in disk specimen 1.50 # 1
5.4.6 Concluding Remarks

Based on the results of this experimental testing program, the following conclusions are highlighted:

1. Disk specimens of each set with the same thickness demonstrated relatively similar behaviors and failure modes.
2. Two types of behavior and failure modes were observed for two groups of disks. For disk specimens with 0.75, 1.00, 1.25, and 1.50 in., the failure mode was punching of steel rod through the disk coupled with the fracture of the whole disk into 3 to 5 main pieces. In disk specimens with thicknesses of 1.75, 2.00, 2.25, and 2.50 in., first, slopped cracks fractured the disk, which resulted in a remaining cone shape. Then, the coned shape concrete crashed under pure compression load.
3. Disks in Group 1 (1.50 in. and thinner) reached their strengths when the first major crack occurred, while in disks in Group 2 (1.75 in. and thicker) the ultimate load was reached after the first major crack took place.
4. The average capacity of disk specimens was 400, 880, 1330, 2320, 4830, 6820, 7610, and 9170 lbs for sets 0.75 to 2.50 in. thickness, respectively. Relatively high coefficients of variation resulted from most sets, which is an indication of inconsistent statistical results.
5. There was a relatively good linear relation between capacity and thickness of disk specimens in each group separately, but not as whole.
6. Average initial stiffness of 19, 52, 52, 75, 90, 103, 113, and 129 kips/in. was calculated for sets 0.75 to 2.50 in., respectively.
7. There was a relatively well linear relation between initial stiffness of the SIWIS element and the thickness of the concrete disk.
8. It was observed that the steel rod experienced a bending moment besides axial compression. The axial strain-load relations were linear as expected and their slopes were consistent and in good agreement with the theory.
5.5 SIWIS Element Test Using Steel Disk and Epoxy

5.5.1 Introduction

An alternative design for SIWIS element is to replace the concrete disk with a steel disk assembly in the form of two steel disks attached by epoxy, in which the epoxy joint will likely control the failure of the fuse element. A steel rod for exerting force and a steel pipe as the support were used. Two different configurations for steel disk assembly were considered including: tension case and shear case. The idea of using steel disk and epoxy is schematically shown in Figure 5.40.

![Conceptual schematics of SIWIS element using steel disk and epoxy](image)

Figure 5.40: Conceptual schematics of SIWIS element using steel disk and epoxy

5.5.2 Theory

The SIWIS element is axially loaded by pushing the steel rod into notch on top of the steel disk assembly, which sits on the thin steel pipe. All three parts experience stress and deformation. In this section, theoretical equations are developed to estimate strength and stiffness properties of the SIWIS element and its components.

The SIWIS element in this experiment is designed so that its ultimate state is likely to occur by failure of the adhesive joint which connects the two steel disks. The epoxy joint experiences tension stress in the tension case and shear stress in the shear cases (Figure 5.40). Assuming failure in the adhesive joint, one can predict the ultimate load for each case by the
following equations, in which, \( D_1 \), \( D_2 \), \( h \) are the dimensions of the steel disk assembly as shown in Figure 5.40; \( \sigma \) and \( \tau \) are the tensile and shear strengths of epoxy, respectively.

\[
P_{U1} = \frac{\pi (D_1^2 - D_2^2)}{4} \sigma_i \quad (5.7)
\]

\[
P_{U2} = \frac{\pi (D_1 + D_2)}{2} h \tau \quad (5.8)
\]

The axial deformation of the steel rod and steel pipe can be predicted by simple linear equations. However, prediction of the deformation of the steel disk assembly in the direction of the load due to the basically unknown behavior of the adhesive joint is difficult. Flexibility/rigidity level, normal and shear stiffness, and possible elongation or sliding of joint before its complete failure are some of the unknown factors. For the rod and pipe, the following equations are developed to estimate their axial deformations in the direction of the applied load:

\[
\delta_{TH}^{ROD} = \frac{PL}{EA} = \frac{P_iL_R}{E_i \left( \frac{\pi D_R^2}{4} \right)} = \frac{4P_iL_R}{\pi E_i D_R^2}
\]

\[
\delta_{TH}^{PIPE} = \frac{PL}{EA} = \frac{P_iL_p}{E_i \left( \frac{\pi D_{p2}^2 - \pi D_{p1}^2}{4} \right)} = \frac{4P_iL_p}{\pi E_i (D_{p2}^2 - D_{p1}^2)}
\]

In these two equations, \( D_R \) and \( L_R \) are, respectively, the diameter and length of the rod; \( D_{p1} \) and \( D_{p2} \) are, respectively, the outer and inner diameters of the pipe; \( L_p \) is the length of the pipe; and \( E_i \) is the modulus of elasticity of the steel.

As mentioned before, it is difficult to develop an empirical equation to predict the deformation of the steel disk assembly. However, in this section, linear theory of plates is used to approximate it for the shear case. It is assumed that the whole disk assembly is isotropic and homogeneous steel. Also a uniform thickness of \( t \) is assumed. For a simple support circular plate, partially loaded in its center (Figure 5.41), the following equation is available in literature for its deformation at the center:
\[
w = \frac{3P_c D^4 (1 - \nu^2)}{256 E_s t^3} \left[ (4 - 5 \beta^2 + 4(2 + \beta^2) \ln \beta) \beta^2 + 2 \frac{\kappa_2}{1 + \nu} \right] \quad (5.11)
\]

\[
P_0 \frac{\pi D_{LOAD}^2}{4} = P_U \quad (5.12)
\]

\[
\kappa_2 = \left[ 4 - (1 - \nu) \beta^2 - 4(1 + \nu) \ln \beta \right] \beta^2 \quad (5.13)
\]

\[
\beta = \frac{D_{LOAD}}{D_{DISK}} \quad (5.14)
\]

By applying \( \beta = 0.25 \) to these equations, the following equation is obtained:

\[
\delta_{TH}^{DISK} = 0.0136 \frac{P_U D^4}{E_s t^3} \quad (5.15)
\]

The total axial deformation of SIWIS element consists of four components including the axial deformation of the steel rod, the axial deformation of the steel pipe, the out of plane deformation of the steel disk assembly, and an adjustment deformation. The adjustment deformation comes from tightening and setting of the SIWIS element parts in a test procedure, particularly in the beginning. The total axial deformation of the SIWIS element is given by the following equation:

\[
\delta_{TOTAL} = \delta_{ROD} + \delta_{PIPE} + \delta_{DISK} + \delta_{ADJUSTMENT} \quad (5.16)
\]

The equivalent axial stiffness of SIWIS element is determined by dividing the ultimate load and corresponding axial deformation as follow:

\[
K_e = \frac{P_U}{\delta_{TOTAL}} \quad (5.17)
\]

Figure 5.41: Idealized steel disk assembly for shear case
5.5.3 Materials

The SIWIS element which was used for this experiment consisted of three parts including a steel rod, a steel pipe, and a steel disk assembly. A simple steel rod of 3 in. length, 1 in. diameter was used as shown in Figure 5.42. The steel pipe used as support had a height of 3 in., outside diameter of 4 in., and inside diameter of 3.75 in as shown in Figure 5.42. A total of four steel disks (two pairs) were used for this experiment. The two steel disks of each pair were attached with an epoxy to form two steel disk assemblies for tension and shear cases. Dimensions of these steel disks are shown in Figures 5.43 and 5.44 for the tension case and the shear case, respectively. Photos of the four steel disks and formed two disk assemblies are shown in Figures 5.45 and 5.46. A relatively high strength epoxy was required for the SIWIS element in this experiment. A product of 3M Company, “DP-460 Off-White,” was chosen for this experiment. This Scotch Weld Duo-Pak epoxy is a high performance, two-part epoxy adhesive offering high shear and peel adhesion, and a very high level of durability.
Figure 5.44: Dimensions of steel disks for shear case
(lef) End steel disk, (right) Middle steel disk

Figure 5.45: Steel disks and assembly for tension case

Figure 5.46: Steel disks and assembly for shear case
Four steel disk assemblies were tested; two for the tension case and two for the shear case. First, two steel disk assemblies were formed, cured, and tested to load control. Then their steel disks were cleaned by removing remaining particles of epoxy. The same four steel disks were reattached to form two other steel disk assemblies which this time were tested under displacement control load. The test number, load type, and curing time are listed in Table 5.7.

Table 5.7: Steel disk assemblies tests

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<th>Disk Assembly</th>
<th>Test Order</th>
<th>Loading Type</th>
<th>Curing Time (hr)</th>
</tr>
</thead>
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<td>Load Control</td>
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</tr>
<tr>
<td></td>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>Displacement Control</td>
<td>22</td>
</tr>
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<td>Load Control</td>
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<tr>
<td></td>
<td>4&lt;sup&gt;th&lt;/sup&gt;</td>
<td>Displacement Control</td>
<td>66</td>
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5.5.4 Equipment

The main equipment used in this experiment included; a testing load frame, a data acquisition system, and a 3M EPX applicator and accessories. The same testing load frame and data acquisition system that were used for the SIWIS element test using a concrete disk is used for this experiment (Section 5.4.3). 3M™ Scotch-Weld™ EPX Applicator and Accessories were used to attach two steel disks to form the steel disk assemblies. This system included the following four equipment elements including; EPX applicator 37 ml, EPX 2:1 plunger assembly, EPX mixing nozzle, and DP-460 Off-White cartridge (Figure 5.47).

Figure 5.47: 3M EPX applicator and accessories
(Top: cartridge, bottom: applicator, plunger, mixing nozzle)
5.5.5 Test Setup, Instrumentation, and Loading

The compression arrangement of the test is shown in Figure 5.48. The SIWIS element was located in the vertical position (the steel pipe on bottom and the steel rod on top) between top and bottom cylinder rams of the testing frame. Three electrical resistance strain gages (ERSG) were used to record the strains on the SIWIS element parts. The longitudinal strain of the steel rod in its middle height, the longitudinal strain on the outside of the steel pipe in its middle height, and the diagonal strain of the bottom of the steel disk assembly were recorded. The total axial deformation of the SIWIS element was measured using the internal LVDT by recording the vertical movement of the load frame piston. The applied load was also measured using the 20-kip capacity load cell.

Incrementally increasing loads through both load and displacement were used in this experiment. A MTS 458.91 microprofiler was employed to control loading history. One percent of the load cell capacity and 0.04 percent of the LVDT stroke length were specified to be applied in one second for load control and displacement control tests, respectively. This resulted in a loading rate of 200 lbs/sec and 0.001 in./sec for load control and displacement control cases, respectively.

Figure 5.48: Compression test setup
5.5.6 Results and Discussion

5.5.6.1 Tests Observations and Failure Modes

Four tests including: tension case-load control, shear case-load control, tension case-displacement control, and shear case-displacement control were conducted. The test observation and failure modes are explained next for each case.

**Tension Case – Load Control:** This specimen had a brittle failure that occurred in the epoxy joint and the two steel disks were separated. The failure occurred at load of 8380 lbs and axial deformation of 0.0769 in. The test lasted 42 seconds. The surfaces of the two steel disks of the failed specimens are shown in Figure 5.49. As can be seen, some particles of epoxy were left on each separated steel disk after the test.

![Surfaces of failed steel disks of tension case-load control](image)

**Shear Case – Load Control:** This specimen had a relatively higher capacity but lower deformation than the previous case. The steel disk assembly had a brittle failure after 53 seconds at 10500 lbs axial load and 0.0427 in. axial deformation. Figure 5.50 shows the failed disk specimens. By careful study of the surfaces and the epoxy layer of this specimen, it was observed that there were numerous voids in the thin layer of the epoxy between the two steel disks. This was basically due to the inappropriate procedure of applying epoxy in this case. An attempt was made to prevent this event for the steel disk specimen for shear case-displacement control using a very thin steel knife. In order to estimate the void areas, unattached surfaces of the end steel disk were marked using a blue marking pen. The surfaces of the middle and end steel disks were
coincident to ensure that the correct and exact location of voids were marked. Figure 5.51 shows the two half parts of the end steel disk with unattached areas marked in blue. It was estimated that about 70% of the surface was attached, which resulted in a reduction of 30% in the capacity. This might also affect the stiffness as well.

![Unattached Areas](image1)

**Figure 5.51:** Unattached surfaces of end steel disk of shear case-load control test

**Tension Case – Displacement Control:** Similar to the previous tests, brittle failure was observed in the epoxy tensile joint in this test. The failure occurred after 108 seconds at 4830 lbs axial load and 0.1074 in. deformation. Figure 5.52 shows the failed specimen and separated surfaces of the two steel disks.

In this test, the capacity of the steel disk assembly was about only 60% of the tension case-load control, while the epoxy and its area were the same for both cases. The type of load and its rate were different for two tests. In the load control case, a load rate of 200 lbs/sec. was
applied, while in the displacement control case, an equivalent load rate of 45 lbs/sec. (4830 lbs maximum load applied in 108 seconds) was used. From a displacement rate view, the load control test had an equivalent displacement rate of 0.0018 in./sec., while the displacement control test had a rate of 0.001 in./sec. Furthermore, the steel disk assembly in the displacement control test had a lower curing time of just 22 hours, while the curing time for the steel disk assembly of the load control test was 96 hours. The recommended curing time for this epoxy in order to reach its full strength is 24 hours at room temperature.

**Shear Case – Displacement Control:** In this last test, the final failure did not occur at the maximum load, unlike the previous three tests. After peak load, the steel disk assembly showed significant post-peak resisting until the test was stopped at a deformation of about 0.65 in. The maximum recorded load was 11140 lbs at a deformation of 0.0317 in. after 32 seconds. The failure mode is shown in Figure 5.53. With reference to this figure, it is clearly seen that adhesion failure occurred and the middle steel disk slid into the end steel disk.
5.5.6.2 Load-Deflection Relations

The resulting load-deflection relations for all four tests are plotted in Figure 5.54. Since the maximum load and the corresponding deflection are the most important properties of the SIWIS element, the equivalent axial stiffness of the system is defined to be the tangent of the line connecting the failure point to the origin as introduced by Equation (5.17). The results of this equation are listed in Table 5.8 for all cases. As can be seen from Figure 5.54, the stiffness of the system was increased by applied load for tension cases until failure occurred. For instance, the initial and ultimate stiffness of the tension case-load control test were about 24 kips/in. and 220 kips/in., respectively. For tension case-displacement control test, they were about 10 kips/in. and 180 kips/in., respectively. This behavior of epoxy is similar to the behavior of most types of rubber and plastic materials when subjected to tension load. These materials become stiffer when subjected to tension, and elongation occurs until failure. Figure 5.54 shows that shear cases had higher capacity and higher initial stiffness. The initial and middle stiffnesses of the shear case-load control test were about 70 kips/in. and 380 kips/in., respectively. For shear case-displacement control test, they were about 150 kips/in. and 500 kips/in., respectively.

Figure 5.54: Axial load-deflection relations for four cases
Table 5.8: Strength and stiffness results of disk tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Load (lb)</th>
<th>Deformation (in.)</th>
<th>Equivalent Stiffness (kip/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension-Load</td>
<td>8380</td>
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<tr>
<td>Tension-Displacement</td>
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<td>Shear-Load</td>
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<tr>
<td>Shear-Displacement</td>
<td>11140</td>
<td>0.0317</td>
<td>350</td>
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</tbody>
</table>

5.5.6.3 Strength of Epoxy

The average tensile strength of the epoxy for tension cases and the average shear strength of the epoxy for shear cases were calculated by Equations (5.7) and (5.8), respectively, and listed in Table 5.9.

Table 5.9: Strength results of epoxy

<table>
<thead>
<tr>
<th>Test</th>
<th>Tensile Strength (psi)</th>
<th>Shear Strength (psi)</th>
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</thead>
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<td>Tension-Displacement</td>
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<td>Shear-Load</td>
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<td>0.0427</td>
</tr>
<tr>
<td>Shear-Displacement</td>
<td>NA</td>
<td>0.0317</td>
</tr>
</tbody>
</table>

5.5.7 Concluding Remarks

The following main conclusions are highlights of this experimental study:

1. The whole system worked well as a “rigid-brittle” element for the SIWIS system, and therefore the proposed concept of using steel disk and epoxy was verified.
2. Shear case disk assembly with shear configuration for adhesive epoxy was the better case for an SIWIS element. It had the highest capacity of 11140 lbs and the highest equivalent stiffness of 350 kips/in.
3. Other possible configurations of disk attachments with different dimensions should be tested with different epoxies to obtain the most optimized design for the SIWIS element.
5.6 SIWIS Element Test Using Lumber Disk

5.6.1 Introduction

The use of a lumber disk as the failing component in SIWIS element is one of the proposed options (Figure 5.55). The appropriateness of using a lumber disk in SIWIS element was confirmed through a preliminary test, in which three lumber species, red oak, poplar, and hard maple, were considered. Based on test results, hard Maple was concluded to be the best choice. Appendix F presents the test program and its results for the preliminary test. Thereafter, an experimental study was planned and conducted to more extensively study the performance of hard maple disks with different thicknesses. In this section, the expanded test program is reported and its results are discussed.

![Figure 5.55: SIWIS element using lumber disk](image)

5.6.2 Materials

Lumber disks cut from hard maple were used for this experiment. The same steel pipe support, steel seat disk, and steel end plate that were used in the concrete test (Section 5.4.2.2) were used in this experiment as well. A 1-3/4 in. half threaded steel rod with 1.5 in. long threaded size of 1”-14 and 1.25 in. long plain rod with diameter of 0.88 in. as shown in Figure 5.56 was
used. The purpose of the upper threaded part was to accommodate an effective attachment to the end plate. The lower part of the rod was turned down to eliminate any engagement between the rod’s threads and the lumber disk when the rod punches through disk.

Six different thicknesses of 0.50, 0.75, 1.00, 1.25, 1.50, and 1.75 in. were considered for lumber disks. Four disk specimens were tested for each thickness, called a “set”. Thus, a total of 24 disk specimens in 6 sets were tested. All disks were cut from the same lumber type and board using 4-1/8 in. diameter hole-saws. This resulted in an average diameter of 3.93 in. for disk specimens. All disk specimens are shown in Figure 5.57. The weight of each disk was measured and listed in Table 5.10. The density properties of disk specimens are also calculated and included in this table. An overall average of 0.0265 lb/in³ (with small coefficient of variations of less than 3% as listed in Table 5.10) was concluded for the density of hard maple lumber used in this experiment.

Figure 5.56: Half threaded steel rod

Figure 5.57: Lumber disk specimens (from left: 0.50 in. set to right: 1.75 in. set)
<table>
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<th>Disk Specimen</th>
<th>Thickness (in)</th>
<th>Weight (gr)</th>
<th>Weight (lb)</th>
<th>Volume (in³)</th>
<th>Density (lb/in³)</th>
<th>Average Density (lb/in³)</th>
<th>Standard Deviation (lb/in³)</th>
<th>Coefficient of Variation (CV %)</th>
<th>Capacity (lb)</th>
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<td>1.75 # 3</td>
<td>1.75</td>
<td>259.7</td>
<td>0.5725</td>
<td>21.23</td>
<td>0.02697</td>
<td></td>
<td></td>
<td></td>
<td>8610</td>
<td>8890</td>
<td>660</td>
<td>7.4</td>
</tr>
<tr>
<td>1.75 # 4</td>
<td>1.75</td>
<td>256.0</td>
<td>0.5644</td>
<td>21.23</td>
<td>0.02659</td>
<td></td>
<td></td>
<td></td>
<td>8450</td>
<td>8890</td>
<td>660</td>
<td>7.4</td>
</tr>
</tbody>
</table>
5.6.3 Equipment, Test Setup, Instrumentation, and Loading

The main equipment used in this experiment included; a testing load frame and a data acquisition system. The same equipment that was used for the concrete disk test (Section 5.4.3) was employed in this experiment as well. The compression arrangement of the test is shown in Figure 5.58. The SIWIS element was held in vertical position with the pipe piece located on the bottom ram of the load frame. All SIWIS element pieces were placed in the center of the testing load frame. The wide flange sections bolted to test table were used for different experiment. The same instrumentation and loading rate as the concrete disk test (Section 5.4.4) were used.

Figure 5.58: Compression test setup
5.6.4 Results and Discussion

5.6.4.1 Test Observations and Failure Modes

Compression tests were conducted on the 24 lumber disk specimens in 6 sets, each set including 4 identical (equal thickness) disk specimens. All 4 disk specimens of each set showed similar behavior. Figure 5.59 shows typical crack development and failure mode of the 1.50 in. lumber disk set, representatively.

In all tests, first, the steel rod started to punch through the center of the disk specimen. The load showed continued increase during this stage. By increased displacement, the 1st major crack was developed in a longitudinal direction on the bottom side of the disk specimen due to bending of the disk specimen. This vertical crack was in the direction of the minor axis of the disk specimen and was parallel to the direction of the grain in the wood structure. Formation of the 1st major crack caused a drop in the resisting load level. By progressive displacement, the width of longitudinal crack was increased and new longitudinal cracks in both bottom and top sides of disk specimen were developed. At this stage, after the load drop due to the 1st major crack, the resistances of the disk specimens were again increased but with a lower rate compared to the beginning of the test. This regain in resistance can be due to confining action that the lumber disk specimen experienced. The disk specimen tended to expand diagonally, but the edges of steel seat disk limited this expansion. The tests were continued and the disk specimens experienced some horizontal and inclined cracks at the same location of the 1st major cracks. By increased displacement, a middle strip of disk specimen with widths of about 1 in. in at the top and 1.5 to 2 in. at the bottom started to separate from the rest of the disk. This strip was formed under a combined action of the steel rod punching and the lumber disk bending. The formation of additional vertical, inclined, and horizontal cracks led to further separation of the middle strip. The tests were stopped after load values dropped to around 500 lbs. The failure mechanism was a combination of rod punching and middle strip separation along grain directions followed by bending failure of the strip in its major direction (parallel to grain direction).
Figure 5.59: Typical failure mode of a 1.50 in. disk specimen
5.6.4.2 Capacities of Disk Specimens

The disk specimens’ ultimate loads (capacities) are summarized in Table 5.10 and shown in charts in Figure 5.60. As can be seen from this table, the coefficients of variations were under 10% and varied from 2.9% to 9.0%, which is an indication of consistent statistical results. As can be seen, the average capacity was 1990, 3660, 4870, 6450, 8850, and 8890 lbs for disk specimens with thicknesses from 0.50 to 1.75 in., respectively. The capacity of the disk specimen increased by thickness as expected. However, the capacities of disk specimens with thicknesses of 1.75 in. were close to those with 1.50 in. thickness. The average capacities were shown in comparison fashion in charts in Figure 5.61. In order to observe the influences of thickness of disk specimen in its capacity as well as the distribution of test results, the capacity-thickness relation is shown in Figure 5.62 for all 24 disk specimens. As can be seen from Figure 5.62, there is a fairly well linear relationship between capacity of disk specimen and its thickness.

![Capacities of Disk Specimens](image)

Figure 5.60: Capacity of lumber disk specimens
Figure 5.61: Specified (average) capacities of lumber disk sets

Figure 5.62: Capacity-thickness relation for lumber disk specimens
5.6.4.3 Load-Deflection Relations

The resulting load-deflection relations for the twenty four tests are plotted in Figures 5.63 to 5.68 for each set.

Figure 5.63: Load-deflection relation for 0.50 in. disk specimens

Figure 5.64: Load-deflection relation for 0.75 in. disk specimens
Figure 5.65: Load-deflection relation for 1.00 in. disk specimens

Figure 5.66: Load-deflection relation for 1.25 in. disk specimens
Figure 5.67: Load-deflection relation for 1.50 in. disk specimens

Figure 5.68: Load-deflection relation for 1.75 in. disk specimens
5.6.5 Effects of Moisture Content and Temperature in Lumber Disk Strength

Wood contains moisture in two forms of “free water” in the cell cavities and “absorbed water” in the capillaries of the cell wall. When the moisture of green wood starts to drop in the seasoning process, the cell walls remain saturated until the free water completely evaporates. This point at which the cell walls begin to lose their moisture is called the fiber saturation point. When moisture content of wood drops below the fiber saturation point, strength of wood increases rapidly and shrinkage begins.

Temperature has reverse effects on the strength of wood. The strength of wood increases when wood cooled below ordinary temperature and decreases when heated.

5.6.6 Concluding Remarks

Based on the results of this experimental testing program, the following conclusions are highlighted:

1. All of the lumber disk specimens demonstrated similar behaviors and failure modes, which were also similar to those in the preliminary test.
2. The tests on specimens of each set with the same thickness resulted in relatively consistent capacities and similar load-deflection relations.
3. The average capacity of disk specimens were 1990, 3660, 4870, 6450, 8850, and 8890 lbs for sets from 0.50 to 1.75 in. thickness, respectively.
4. For all lumber disk sets, the coefficients of variations of capacities were under 10%, which is an indication of consistent test results.
5. There was a relatively consistent linear relation between capacity and thickness of lumber disk specimens.

5.6.7 Comparison between Different Compression Disk Tests

Three different compression disk designs including; concrete disk, steel disk, and lumber disk were experimentally tested and investigated. In order to compare the performances of these three options, one disk specimen of each design was selected. Those disk specimens were selected that had closer capacity results. They are 2.50#1 concrete disk, shear-case steel disk, and
1.50#1 lumber disk (hard Maple). The load-deflection relations for these three disks are shown in Figure 5.69.

As can be seen from Figure 5.69, an SIWIS element with a concrete disk and a steel disk are stiffer than the element with lumber disk. The element with the steel disk is stiffer than the element with the concrete disk. The element with lumber disk demonstrates more deformation and, thus, more ductility compare to the other two cases. After peak load, the elements with steel and concrete disks demonstrated a relatively high drop in resisting load, while in the element with the lumber disk, the resisting load deteriorated at a slower rate. The presence of sudden high load drop in the fuse element can have undesirable effects (serviceability) on the building. Based on this discussion, it was decided to use lumber disks for the main test frame’s SIWIS elements.

**Figure 5.69: Load-deflection relations for three different SIWIS element designs; concrete disk, steel disk and epoxy, and lumber disk**
5.7 Test Program of Two-Bay Three-Story Frame

5.7.1 Introduction

The concept of SIWIS system was experimentally investigated through a testing program presented in this section. A scaled two-bay three-story steel frame with infill walls equipped with SIWIS elements with different grades was subjected to a series of pushover static tests. The results of single wall test, rod pulling test, and SIWIS element tests, which were presented in the previous sections of this chapter, were used to design and conduct the frame test program. A total of 11 static tests in 3 series were conducted on the scaled two-bay three-story steel frame including: 4 tests on bare frame, 5 tests on frame with SIWIS elements, and finally 2 tests on pinned frame. In all tests, incrementally increasing displacement was applied at the third floor level (displacement control). The main objective of this test was to test and verify the concept of the proposed SIWIS system as well as to validate the developed finite element modeling schemes through comparing experimental observations with the numerical predictions.

5.7.2 Frame Design and Construction

A general description of the test frame with infill walls and SIWIS elements is schematically shown in Figure 5.70. A scale ratio of about 1 to 4 was considered for the frame based on the available space and loading facilities. The story heights were all 31 in. and the bay lengths were all 49 in. from centerlines of beams and columns. For all columns and beams, Tube 5x5x3/8 section was considered. This resulted in a clear story height and length of 26 in. and 44 in., respectively. Brick masonry walls identical to those walls that have been used for the single wall test (Section 5.2) were used for the six bays of the test frame. The walls had a height of 24-1/4 in. and a length of 32 in. This resulted in a horizontal gap of 6 in. between the columns and wall for locating SIWIS elements and a vertical gap of 1-3/4 in. between top of the wall and bottom of the top beam. Figure 5.71 shows a photo of the test frame.

It was decided to design the test frame with pinned beam-to-column and column-to-column connections to have a more feasible experimental simulation for adequate floor movements as well as to study the performance of the SIWIS system in both stiff and flexible frames (braced and pinned frames). At each connection, beams and columns were connected together by a pin bar (pinned connection). At the base, each of the first story’s columns
was connected to two angle sections by a pin bar and the angle sections were rigidly connected to the base plate by weld. For each pinned connection, a 1-1/2 in. diameter pin bar (plain rod) with stress relieved 1144 steel was used. This steel type meets ASTM A311 and has minimum yield strength of 95 ksi. The frame was designed in such a way that all beams and columns connected in a joint can independently and freely rotate about the pin rod. Figure 5.72 shows a typical pinned connection between beam and column and a rigid connection between the base angle section and base plate (Axis 1-A). During manufacturing, an attempt was made to have relatively tight but movable (rotatable) connections. As can be seen from Figure 5.72, in order to reduce the friction forces, thin washer shims were placed between those members of the frame that intend to rotate to reduce the friction surface area. Moreover, lubricating grease was used between plates and between pin bar and plates for all connections. Two photos of this connection are shown in Figure 5.73.

Diagonal X-bracing elements were designed for each bay of the test frame to provide required in-plane stiffness for the frame. Based on the rod pulling test results (Section 5.3), 3/8”-16 steel threaded rod with Grade B7 material (high tensile strength of 125 ksi) was considered for the diagonal bracings. Four diagonal bracing elements were used for each bay, two in each side. They were connected to the pin bars of the beam-column joints by 1-3/4 in. long coupling nuts welded to 6x4x1/4 in. plates. The design of the diagonal bracing element is shown in Figure 5.74. Each diagonal brace consisted of one right-hand threaded rod and one left-hand threaded rod connected to each other by a special coupling nut. This special coupling nut with right-hand threads in one end and left-hand threads in the other end, allows the joining of coupled rods by a single turn of the nut. Each thread continues halfway into the nut. This design made it possible to easily assemble and disassemble the diagonal braces for different configurations. The other advantage of this design was in providing effective control and adjustment over the axial tightening or pre-tension load of the diagonal rods.
Figure 5.70: Schematic design of scaled two-bay three-story test frame
Six masonry wall panels constructed using solid clay bricks identical to those that have been tested in the single wall test (Section 5.2) were used. A 5-in. long 4x4x3/4 in. angle section welded to the beam was used in each side of the supporting beam to prevent base sliding of the wall panel. A vertical system including two ½"-13 Grade B7 threaded rods and top 9x6x1/2 in. plate was designed to tie the top of the wall panel to the bottom beam and prevent the rotation of the masonry wall panel (uplift). Each vertical rod was supported by bolts to a 5x3x1/2 in. angle section, which was welded to the supporting beam. At the two bottom corners of some of the wall panels, between angle sections and the masonry wall panel, either a 1/16 in. thick rubber or a
¼ in. thick wood shim was inserted to fill the gap between the wall and the angle sections in order to provide tight connections between them.

Figure 5.72: Schematic design of a typical connection (Axis 1-A)
Figure 5.73: Photos of a typical connection (Axis 1-A)

Figure 5.74: Schematic design of a diagonal bracing element
5.7.3 SIWIS Elements

The compression design option using hard maple lumber disk was chosen for SIWIS elements in this experimental program as explained in Section 5.6.6. Three different grades were considered for SIWIS elements. These grades were chosen based on the results of the SIWIS element test using lumber disk (Section 5.6). The specified capacities and thicknesses of these SIWIS elements are listed in Table 5.11. Similar pieces as those used for the SIWIS element test were used. A 1-3/4 in. long, 4 in. diameter, and 1/8 in. wall thick steel pipe was used as the support piece. A steel disk was machined as a seat disk for lumber disks. A half threaded steel rod with 1”-14 thread size in its one end was used. Three different lengths of 2.25, 2.50, and 3.00 in. were considered to accommodate the placement of lumber disks with different thicknesses. A 3x3x1 in. steel end plate with 1”-14 thread size hole in its center was used to distribute the compression load over larger area on the masonry wall.

The design of SIWIS elements in this experimental program and its photo are shown in Figures 5.75 and 5.76, respectively. The steel seat disk was welded to a steel pipe and this assembly was itself welded to the column. The steel end plate was attached to the masonry walls using high strength 3M DP-190 epoxy. This epoxy can be used for both metal and masonry and requires a curing time of one week. The SIWIS element should be tightly fitted inside the gap between the wall panels and frame. Thus, the end steel plate and half threaded steel rod were designed in such a way that they allow the whole length of the SIWS element to vary from 5.50 in. to 6.50 in. This design permitted extending or shortening the SIWIS element in the gap in case they are slightly smaller or larger than 6 in. This design also made it easy to replace failed lumber disks with new ones after each test.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Lumber Disk Thickness (in.)</th>
<th>Specified Capacity (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade 1</td>
<td>0.50</td>
<td>1990</td>
</tr>
<tr>
<td>Grade 2</td>
<td>0.75</td>
<td>3660</td>
</tr>
<tr>
<td>Grade 3</td>
<td>1.00</td>
<td>4870</td>
</tr>
</tbody>
</table>
Figure 5.75: Schematic design of SIWIS element and its placement

Figure 5.76: A photo of SIWIS element
5.7.4 Test Series and Objectives

The conducted tests on the scaled two-bay three-story frame are summarized in Table 5.12. Before proceeding with any test, the movements of the frame were evaluated. As mentioned before it was intended to design a pinned frame; however, there can be some resistance due to presence of friction between plates and between pin bar and plates. To check the magnitude of this resistance, the frame without diagonal bracing and SIWIS elements was subjected to in-plane displacements. It was observed that the magnitude of this initial resistance was about 100-200 lbs, which is negligible.

In SIWIS braced frame test series (SBFT), the diagonal bracing elements were in place and all masonry walls were engaged by SIWIS elements. All of the diagonal bracing elements were used in tests SBFT1, SBFT2, SBFT3 and SBFT4, while only half of them (one bay) were used in test SBFT5. Two different patterns were considered for the SIWIS elements in this test series. In the first pattern, which was considered for tests SBFT1, SBFT2, and SBFT5, Grade 3, 2, and 1 SIWIS elements were used in the 1st, 2nd, and 3rd stories, respectively. In the second pattern, which was considered for tests SBFT3 and SBFT4, Grade 1 SIWIS elements were used in all three stories. The main purposes of these tests were to study and observe the performance of the frame with SIWIS elements and to validate the developed finite element model. In test SBFT5, only one bay in each story was braced to study the influence of less stiff frame in response as well as to further verify the validation of the finite element model. In bare frame test series (BFT), only the frame (frame with diagonal bracings but without SIWIS elements) was subjected to an in-plane load. The main purposes of this test were to measure the in-plane stiffness of the bare frame, provide comparable information with the results of the frame with SIWIS elements (SBFT), and to validate the finite element model. Finally, SIWIS pinned frame tests (SPFT) were carried out to observe the performance of the SIWIS system in the pinned frame and to examine the accuracy of the finite element model in predicting pinned frame response. No diagonal bracing elements were used in these tests. Two patterns for SIWIS element capacities in the three stories were used. In test SPFT1, Grade 3, 2, and 1 SIWIS element were used in the 1st, 2nd, and 3rd stories, respectively. In test SPFT2, Grade 3 SIWIS elements were used for all three stories. Diagrammatic views of all tests series are shown in Figure 5.77.
Table 5.12: Tests on two-bay three-story frame

<table>
<thead>
<tr>
<th>Test</th>
<th>Description</th>
<th>Description</th>
<th>SIWIS Element Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; Story</td>
</tr>
<tr>
<td>BFT1</td>
<td>Bare Frame Test</td>
<td>No SIWIS Elements, with all Braces</td>
<td>NA</td>
</tr>
<tr>
<td>BFT2</td>
<td>Bare Frame Test</td>
<td>No SIWIS Elements, with all Braces</td>
<td>NA</td>
</tr>
<tr>
<td>BFT3</td>
<td>Bare Frame Test</td>
<td>No SIWIS Elements, with all Braces</td>
<td>NA</td>
</tr>
<tr>
<td>BFT4</td>
<td>Bare Frame Test</td>
<td>No SIWIS Elements, with half Braces</td>
<td>NA</td>
</tr>
<tr>
<td>SBFT1</td>
<td>SIWIS Braced Frame Test</td>
<td>All SIWIS Elements, with all braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SBFT2</td>
<td>SIWIS Braced Frame Test</td>
<td>All SIWIS Elements, with all braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SBFT3</td>
<td>SIWIS Braced Frame Test</td>
<td>All SIWIS Elements, with all braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SBFT4</td>
<td>SIWIS Braced Frame Test</td>
<td>All SIWIS Elements, with all braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SBFT5</td>
<td>SIWIS Braced Frame Test</td>
<td>All SIWIS Elements, with half Braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SPFT1</td>
<td>SIWIS Pinned Frame Test</td>
<td>All SIWIS Elements, No Braces</td>
<td>Grade 1</td>
</tr>
<tr>
<td>SPFT1</td>
<td>SIWIS Pinned Frame Test</td>
<td>All SIWIS Elements, No Braces</td>
<td>Grade 3</td>
</tr>
</tbody>
</table>

Figure 5.77: Diagrammatic views of two-bay three-story frame test series
5.7.5 Instrumentation

The schematic of instrumentation for a typical test is shown in Figure 5.78. The in-plane horizontal movements of all three floor levels were recorded using six potentiometers. Two potentiometers were used for each story. The floor deflection was taken as the average measured deflections by the two potentiometers. The spring of each potentiometer was attached to a small hook bolt that was installed at each end of the pin bar of the frame joint in Axis C (loading end). The six potentiometers were supported to the reaction frame (zero deflection) by a truss type system, which was constructed using aluminum L-shape sections.

Figure 5.78: Design of test instrumentation
For SIWIS elements, two electrical strain gages (EG) were installed on two opposite sides of each half threaded steel rod to record the longitudinal strain history. These strain values were used to obtain the axial force histories for the SIWIS element as well as its failure point (the failure of the lumber disk). A total of 12 strain gages were used for the compression side SIWIS elements. A pressure transducer was used to record the pressure of the hydraulic flows for the loading cylinder. The values of recorded pressure were then converted into load values by multiplying them in appropriate pressure to load ratios of the used loading cylinder. Figure 5.79 shows a photo of loading cylinder, attached pressure transducer, and a potentiometer.

**Figure 5.79: Loading cylinder, potentiometer, and pressure transducer**

### 5.7.6 Equipment

The main equipment used in this experiment included; the testing platform, hydraulic loading jacks and pump, load control system, and data acquisition system. The loading history was controlled by the amount of the hydraulic pressure through the cylinder. A manual two-button, push and pull (advance and retract), controller was used to control the hydraulic pressure. The loading jack was connected to a hydraulic pump by two hoses. A hydraulic flow
controlling valve was placed in the path of the advance hose between the pump and the loading jack to provide better control for applying hydraulic pressure by manual two-button controller. Figure 5.80 shows the pump and manual controller that were used in this experiment.

A total of 12 strain gages, 6 potentiometers, and 1 pressure transducers were used. Two 10 channel 2100 system signal conditioner/amplifiers connected together were used to control the strain gages. Two data acquisition boards were used, one for strain gages and one for potentiometers and pressure transducer. Signals from this instrumentation were sent to two computers by the two data acquisition boards and were monitored during the test by Lab-View computer program. The station used for this experiment including channels, two data acquisition boards, and two computers is shown in Figure 5.81.
For an effective and proper attachment of the stroke of the loading jack to the frame, a load connection device mainly including a heavy bolt and nut, three thick plates, and two short pin bars were designed and constructed as can be seen in Figure 5.79. The bolt was tightly attached to the stroke of the loading cylinder. On the other side of the bolt, a 1-1/4 in. thick plate was attached by a heavy nut. Two 2-1/2 in. long and 1-1/2 in. diameter pin bars were welded to the two sides of the plate to provide pinned connection similar to those in the frame joints. Two identical ¾ in. thick plates with holes for 1-1/2 in. pin bars were used to connect the pin bar of the frame (joint) to the pin bar of the loading device. This system created an appropriate pinned connection between the loading cylinder and the frame.

5.7.7 Loading

In-plane loads in the form of incrementally increasing displacement were applied at the third floor level in these experiments. Relatively slow loading rate was used to allow careful observation of system performance and the failure progression of the lumber disks. Displacement
control loading is used for most experimental tests in order to collect more detailed data, since it can better characterized the response. It was decided to use displacement control mainly in order to make it possible to observe and monitor the post-crack and post-peak responses, such as stiffness degradation and strength deterioration. As observed in SIWIS element tests using lumber disk (Section 5.6), there are substantial post-crack and post-peak responses in the form of resistance drops and regains and stiffness changes. In order to capture these responses, it is necessary to conduct displacement control loading. On the other hand, load control conditions are usually used for strength tests, in which the ultimate resistance of a system is the most important parameter to be measured. Incrementally increasing displacement (displacement control) will provide stable crack growth of lumber disks, while incrementally increasing load (load control) will cause total failure of a lumber disk once it cracked and its capacity reached. Most test standards recommend that the whole test to be conducted under displacement control loading or initially started with load control and then switched to displacement control when stiffness changes are anticipated.

Based on the above discussion, displacement control loading was considered in this part of the study. However, the response of the system for load control conditions is investigated in Chapter 4 through finite element modeling. Furthermore, for design purposes of the system, load control parameters are considered in Chapter 6.

5.7.8 Tests Results and Discussions

5.7.8.1 Bare Frame Tests (BFT)

A total of four tests were performed on the bare frame as listed in Table 5.12. Tests BFT1, BFT2, and BFT3 were carried out on the frame with all diagonal bracings in place, while test BFT4 was performed on the frame with only one bay braced in each story. As mentioned before, incrementally increasing displacement was applied with the use of the manual controller. An attempt was made to have a relatively constant loading rate with the use of manual controller. The applied loading histories are shown in Figure 5.82 for tests BFT1 to BFT3. In order to eliminate yielding of any diagonal bracing rods, the tests were stopped at about 25 kips. It can be seen from Figure 5.82 that the tests took about 6 to 7 minutes to reach to the 3rd floor deflection of about 1.20 in. This resulted in an average loading rate of about 0.0035 in./sec. The resulting load-deflection relation is shown in Figure 5.83 for all tests. In this graph, the horizontal axis
represents the 3\textsuperscript{rd} floor level’s in-plane deflection and the vertical axis represents the measured load. It can be seen from Figure 5.83 that the response of the three tests are very close to each other. The response of the frame was linear, as expected, and thus, the loading history did not affect the load-deflection relation. Also, as expected, at the beginning of the tests, there was a region where the frame demonstrated very low stiffness. The reason for this observation, which was before about 0.20 in. deflection, was the fact that the different elements particularly diagonal bracings and connection parts of the frame were tightening and setting. After a deflection of about 0.20 in., the frame obtained its real stiffness and exhibited a very linear response. An average in-plane stiffness of about 23 kips/in. resulted for the full bare frame from Figure 5.83. In order to observe the response of all three floors, all floor levels’ deflection histories are shown in Figure 5.84. It can be seen from this figure that the bare frame demonstrated nearly linear deflection distribution for the three floor levels. At any specific time, the ratios of deflections of the 1\textsuperscript{st}, 2\textsuperscript{nd}, and 3\textsuperscript{rd} floor levels were nearly 1:2:3. This is expected, since all three stories have equal height and the same diagonal bracing elements.

The load-deflection relation for test BFT4, the frame with one bay braced, is also included in Figure 5.84. An in-plane stiffness of about 11.6 kips/in. was calculated for this test. This is about half of the stiffness of the full frame, as anticipated.

![Figure 5.82: Bare frame test (BFT) loading history](image)
Tests BFT: Load-Deflection Relation

Figure 5.83: Bare frame test (BFT) load-deflection relation

Tests BFT1 to BFT3: Floor Levels’ Deflection Histories

Figure 5.84: Bare frame test (BFT) floors’ deflection histories
5.7.8.2 FE Model Validation for Bare Frame Tests (BFT)

The development of finite element models for the two-bay three-story test frame was presented in Section 4.8. Incrementally increasing displacement was applied for the developed model of the bare steel frame, and the resulting load-deflection relation is shown in Figure 5.85 along with the experimental results. As can be seen from this figure, the analytical model very closely captured the test results. There is a very close match between the stiffness of experimental and analytical curves. For fully braced frame (tests BFT1, BFT2, and BFT3), it is about 23 kips/in. and 22.6 kips/in. for the test and computer model, respectively. For the half braced frame (test BFT4), it is about 11.6 kips/in. and 11.4 kips/in., respectively.

![Tests BFT: Load-Deflection Relation](image)

Figure 5.85: Bare frame test (BFT) load-deflection relation

5.7.8.3 SIWIS Braced Frame Tests (SBFT)

A total of five tests were conducted on the braced frame with SIWIS elements in two different configurations. Tests SBFT1 and SBFT2 were conducted on the fully braced frame with Grade 3, 2, and 1 SIWIS elements in the 1st, 2nd, and 3rd stories, respectively. Tests SBFT3 and SBFT4 were also performed in the fully braced frame, but with Grade 1 SIWIS elements in all
three stories. Test SBFT5 was carried out on the half braced frame with Grade 3, 2, and 1 SIWIS elements for the 1st, 2nd, and 3rd stories, respectively. Similar to the bare frame tests, incrementally increasing displacement was applied at the 3rd floor level. The applied loading histories are shown in Figure 5.86. A slower loading rate was used for these tests compare to bare frame tests for more accurate test observation and data collection purposes. It can be seen from Figure 5.86 that tests SBFT1 to SBFT5 took about 30, 19, 16, 8, and 10 minutes to reach to the 3rd floor deflection of about 2.12, 1.72, 1.18, 1.20, and 1.54 in., respectively. The tests were stopped at these times to eliminate yielding of diagonal bracing rods of the frame. This resulted in an average loading rates of about 0.0013, .0015, 0.0013, 0.0028, and 0.0026 in./sec for the five tests, respectively.

![Tests SBFT: Loading History](image)

Figure 5.86: SIWIS braced frame test (SBFT) loading history

To comprehensively study the response and behavior of the frame with SIWIS elements, the experimental observations and results for test SBFT1 is discussed in detail in this section. For the rest of the tests, only load-deflection relations and SIWIS element failure loads are presented and other results are enclosed in Appendix G.
5.7.8.3.1 SIWIS Braced Frame Test 1 (SBFT1)

The resulting load-deflection relation for test SBFT1 is shown in Figure 5.87. The strain histories for each rod of the 6 compression side SIWIS elements were recorded by 2 electrical gages during the test. These test strain data, \( \varepsilon_1 \) and \( \varepsilon_2 \), were converted to force data, \( F_{SE} \), using the following equation:

\[
F_{SE} = \left( \frac{\varepsilon_1 + \varepsilon_2}{2} \right) E_s A_{rod}
\]  \hspace{1cm} (4.18)

In Equation (4.18), \( E_s \) is the modulus of elasticity of rod steel material (assumed 29000 ksi), and \( A_{rod} \) is the cross sectional area of rod (0.61 in.\(^2\)). The force histories of the 6 rods are plotted in graphs in Figure 5.88. In these graphs, the vertical axis represents the axial force of the rod of the SIWIS element and thus, the axial force of the SIWIS element as well and the horizontal axis represents the in-plane deflection of the frame at the 3\(^{rd}\) floor level, which was measured using two potentiometers. With reference to the test observations and Figures 5.87 and 5.88, the response of the system is subsequently explained for test SBFT1.

As can be seen from Figure 5.87, similar to bare frame tests, there is a region at the beginning of the test where the system demonstrated lower stiffness due to setting deformations and connections tightening (Point A). After this region, the system showed linear response with an in-plane stiffness of about 26 kips/in. Figure 5.87 shows that at deflection levels of about 0.50 to 0.70 in., the slope of the load-deflection curve started to decrease slightly (Point B). This observation can be attributed to the start of rods punching through lumber disks and formation of initial cracks in lumber disks. This can be verified by referring to Figure 5.88, which shows that SIWIS elements of the 3\(^{rd}\) and 2\(^{nd}\) stories experienced some load drops in the mentioned deflection region. These resistance drops are attributed to the formation of cracks in the lumber disks of these SIWIS elements. The slope of the load-deflection curve (stiffness of the system) in Figure 5.87 continued to decrease slightly by increased applied displacement. This is due to progressive cracks growth in lumber disks and the rods punching through them.

At the load and deflection levels of about 22000 lbs and 1.02 in., respectively, there is a resistance load drop of about 900 lbs. This load drop, which is shown by Point C in Figure 5.87, is attributed to the failure of the 3\(^{rd}\) story’s SIWIS element in the left panel. This can be clearly seen in Figure 5.88 as well, which shows the SIWIS element of the 3\(^{rd}\) story in the left panel.
reached its capacity of about 2000 lbs, after which its resistance dropped significantly (about 400 lbs). Next, the SIWIS element of the 3rd story in the right panel reached its capacity in the frame load and deflection level of about 27000 lbs and 1.35 in., respectively. Similarly, there is a resisting load drop of about 600 lbs in this stage as shown as Point D in Figure 5.87. The failure of the lumber disk of this SIWIS element can be concluded from Figure 5.88 as well. It can be seen from Figure 5.88 that the lumber disk reached its capacity of about 2000 lbs and immediately its resistance dropped to about 500 lbs.

![Test SBFT 1: Load-Deflection Relation](image)

**Figure 5.87: Test SBFT1 load-deflection relation**

By increasing the applied displacement, the failure of the lumber disk of the 2nd story’s left panel occurred at frame load and deflection of about 30900 lbs and 1.67 in., respectively (Point E). Next, the lumber disk of the 2nd story’s right panel failed at frame load and deflection of about 31500 lbs and 1.78 in., respectively (Point F). Figure 5.87 shows frame load drops of about
600 lbs and 2000 lbs in Points E and F, respectively. These results are in close agreement with the graphs in Figure 5.88, which show that in these two points (Point E and Point F), the lumber disks of the 2nd story in the left and right panels reached their capacities of about 3600 lbs and 3800 lbs followed by lumber disk resistance drops of about 2600 lbs and 2800 lbs, respectively.

It was decided to stop the test in Point G in the frame load and deflection of about 33400 lbs and 2.12 in., respectively, to eliminate potential yielding of diagonal rods. At this stage, the lumber disks of the 1st floor, which were of higher grade (Grade 3) compared to the lumber disks of the 3rd story (Grade 1) and the 2nd story (Grade 2), had developed initial cracks and the rod had punched through the disks about 1/16 to 1/8 in. However, the lumber disks had not reached their capacities and thus, they had not failed yet. This can be seen in Figure 5.88, which shows that the SIWIS elements of the 1st story were still resisting with no substantial resistance drop after Point F until the end of the test.

The rod punching through lumber disks and their cracking are shown in Figure 5.89 for the left bay SIWIS elements, representatively. The right side photos were taken at the end of the test, while the left side photos were taken from the same SIWIS elements at an earlier time. Photos of the failed lumber disks taken after they were extracted from the SIWIS elements are shown in Figures 5.90 and 5.91. As can be seen from these figures, after the rod punched through the lumber disks of the 2nd and 3rd stories, the middle strip was separated from the rest of the disk and then the middle strip fractured in bending. This failure mode is similar to those observed in the SIWIS element test (Section 5.6). As mentioned before, the 1st story’s lumber disks did not fail. Figures 5.89 to 5.90 confirm this and show that the rod started punching through these lumber disks and they only developed initial major cracks in the form of a diagonal crack on back side of the disks.

The SIWIS element design that was used in this experiment demonstrated satisfactory performances. The design included; steel pipe (welded to the column), steel seat disk (welded to the pipe), lumber disk, steel end plate (attached to the wall by epoxy), and a steel rod (bolted to the end plate). The thread design in the rods and end plate made it very easy to insert or replace lumber disks with different thicknesses. This design also allowed effective tightening of the SIWIS element between the wall panel and the frame after the placement of lumber disk. The steel pipe permitted the rod and failed lumber disk to freely displace inside its hollow space in order to effectively disengage the masonry wall from interaction with the frame. This observation can be seen in Figure 5.89 for the 3rd story’s SIWIS element. The end steel plate, which was designed to distribute the compressive force of the SIWIS element over larger area on the
masonry wall panel, performed well. No signs of damages were observed in the masonry walls at the locations of end plate.

Figure 5.88: Tests SBFT1 and SBFT2 SIWIS elements force histories
Figure 5.89: Failure modes of the SIWIS elements in test SBFT1

(Left: former time, right: later time)
Figure 5.90: Failure mode of all SIWIS elements in test SBFT1 (front side)
(Left: left panel, right: right panel)

Figure 5.91: Failure mode of all SIWIS elements in test SBFT1 (back side)
(Left: left panel, right: right panel)
All masonry walls were carefully inspected during and after the test. There were no signs of any damage such as cracks or crushing. This performance was expected, since the SIWIS elements were designed in such a way that they fail well before the reaching the masonry walls’ cracking load. The vertical systems that tied the top of the wall panels to the bottom beams including threaded steel rods and their top and bottom connections provided a good performance. No damage or failure was observed in the threaded steel rods and their connections. The steel angle sections, which were used to prevent the sliding of masonry wall panels, provided effective support.

One of the advantages of the proposed SIWIS system is the increased in-plane resistance of the frame by added resistance from masonry walls. However, SIWIS elements will limit the masonry walls’ resistance sharing to a certain degree to eliminate damage to the walls. In order to observe the beneficial effects of masonry walls in this test, the resistances provided by masonry walls are calculated and shown in graphs in Figures 5.92 to 5.94 for the 1st to 3rd stories, respectively, along with the resistance of the bracings. The resistance of masonry wall in each panel is taken to be equal to the SIWIS element force attached to the wall under consideration (available in Figure 5.88). Figures 5.92 to 5.94 show that considerable resistances were provided by masonry walls, particularly in the 1st story, in which higher grade SIWIS elements were used compared to the 2nd and 3rd stories. For instance, for the 1st story, at the frame deflection of 0.50 in., the bracings and masonry walls provided about 5500 and 5000 lbs resistances, respectively. Thus, masonry walls supplied nearly equal resistance as did the bracings. In other words, masonry walls increased the resistance of the frame by nearly 100%. By increased deflection and formation of cracks in lumber disks, the share of the resistance of masonry walls relative to bracings was decreased. For example, at the deflection of 2.00 in. after cracking of lumber disks of the 1st story’s SIWIS elements, the bracings and masonry walls resisted about 26400 and 6100 lbs, respectively.

The 3rd story’s masonry walls provided the least resistance due to their lower SIWIS elements’ grades compared to the other two stories. It should be mentioned that as observed in the SIWIS element test using lumber disk (Section 5.6 and Appendix F) a thicker lumber disk had higher capacity and higher axial stiffness as well. It can be seen from Figure 5.94 that even after the failure of the lumber disks in the 3rd story, the masonry walls still provided some resistance. This is due to the fact that lumber disks demonstrated a quantity of post-peak resistance (Figure 5.88). In other words, the SIWIS element design using lumber disks do not completely isolate the masonry walls from the frame after reaching their capacities.
Figure 5.92: Test SBFT1 resisting load (1st Story)

Figure 5.93: Test SBFT1 resisting load (2nd Story)
It was explained in Chapter 4 for the analytical response of the frame with SIWIS elements that, ideally, depending on the number of SIWIS element failure points, there can be several phases in the response of the system. As explained, each phase (except Phase 1) starts with failure of an SIWIS element associated with a load drop followed by a decrease in the in-plane stiffness of the system. Based on this discussion, phase behavior can be specified for the response of test SBFT1 shown in Figure 5.87. The first phase (or stage) was before failure of any SIWIS elements (Path AC). The second phase occurred after failure of the SIWIS element of the 3rd story in the left panel (Path CD). The third phase took place after failure of the 3rd story’s SIWIS element in the right panel (Path DE). Similarly, the fourth and fifth phases happened after failure of the 2nd story’s SIWIS element in the left and right panels, respectively (Paths EF and FG). Since, the test was stopped before failure of SIWIS elements of the 1st story, further phases were not captured. As expected, after the failure point associated with failure of each SIWIS element, there was a load drop. Also, the in-plane stiffness of the system decreased for progressive phases. The first phase had the highest stiffness and the last phase had the lowest. However, the scale of load drops and stiffness decreases shown in Figure 5.87 for test SBFT1 are

Figure 5.94: Test SBFT1 resisting load (3rd Story)
somewhat smaller than those demonstrated in Chapter 4 for analytical models. The main reasons for these observations are discussed next.

There are differences between the SIWIS elements force-deformation responses considered in Chapter 4 and those used in the experimental tests. In Chapter 4, relatively high axial stiffness was considered for the “brittle-failure” responses of SIWIS elements with no or very limited post-peak resistance, while the lumber disks that were used in the experiment showed lower stiffness with considerable post-peak resistance. Moreover, moment resisting frame was considered in Chapter 4, while braced frame was used for experimental tests. The higher in-plane stiffness of the braced frame, relative to the moment resisting frame, decreases the effects of load drops and stiffness changes induced by SIWIS elements. It is, of course, clear that the fact that much lower grade SIWIS elements (capacity and stiffness) were used in the scaled tests compared to more realistic ones considered in analytical models in Chapter 4, thereby escalating the mentioned differences.

5.7.8.3.2 SIWIS Braced Frame Tests 2 to 5 (SBFT2 to SBFT5)

The load-deflection relations for tests SBFT2 to SBFT5 subjected to loading histories presented in Figure 5.86 are shown in Figures 5.95 to 5.98, respectively. The SIWIS elements force histories and other floor levels’ deflection histories are enclosed in Appendix G for these tests. Similar to test SBFT1, with reference to the load-deflection relations and SIWIS elements force histories for each test, the SIWIS element failure points are investigated and included in Figures 5.95 to 5.98. For all frame tests with SIWIS elements, the failure points of SIWIS elements in each story and each bay are summarized in Table 5.13 along with the initial in-plane stiffness of the frame.

Test SBFT2 was similar to test SBFT1 with replaced new lumber disks after their failure. The main purpose of this repeat test was to observe the consistency of test results. The load-deflection relations of the two tests are also shown together in Figure 5.99. The two tests resulted in close load-deflection relations; however, there were some differences. The two curves nearly coincide up to the first failure point. It can be seen from the load-deflection relations that the initial stiffness of the system in tests SBFT1 and SBFT2 was 26 and 26.2 kip/in., respectively. In both tests, the SIWIS element of the 3rd story in the left panel failed first at very close load level. It failed at load level of 22000 and 22100 lbs for tests SBFT1 and SBFT2, respectively. The SIWIS element in the right panel failed next in the two tests as well, but at different load levels. It
failed at load levels of 27000 and 23000 lbs for tests SBFT1 and SBFT2, respectively. In test SBFT1, the SIWIS elements of the 2nd story at the left and right panels failed next with close load levels of 30900 and 31500 lbs, respectively. In test SBFT2, they also failed next at load levels of 30700 and 30600 lbs, respectively. Thus, the failure loads of the 2nd story’s SIWIS elements were very close in the two tests.

The main reason for the observed differences in the response of tests SBFT1 and SBFT2 can be attributed to the fact that the lumber disks that were used for the two tests were not realistically identical. It was observed in SIWIS element tests (Section 5.6) that lumber disks with the same thickness had slightly varying force-deformation responses. The force-deformation responses of the SIWIS elements presented in Figure 5.88 also verifies that although the two lumber disks with the same grade had close responses, they were not identical.

![Test SBFT2: Load-Deflection Relation](image)

Figure 5.95: Test SBFT2 load-deflection relation
Next, tests SBFT3 and SBFT4 were carried out to study the response of the system with the same grade SIWIS elements in all three stories. It can be seen from Table 5.13 and Figures 5.96 and 5.97 that in these two tests, all of the SIWIS elements in the three stories failed close together. The majority of the SIWIS elements failed between 23000 and 26000 lbs. The lowest failure load was 19000 lbs in SBFT4 for the 2nd story’s SIWIS element in the right panel and the highest one was 26000 lbs in SBFT3 for the 1st story’s two SIWIS elements and the 3rd story’s right SIWIS element. The initial stiffness of the system was 25.1 and 25.2 kips/in. for tests SBFT3 and SBFT4, respectively.
Next, test SBFT5 was carried out to observe the response of the system with lower frame in-plane stiffness. Figure 5.98 and Table 5.13 show that in this test only the 3rd story’s SIWIS elements, which were of lower Grade 1, failed. They failed at the same time at a load level of 12500 lbs. An initial stiffness of 15.5 kips/in. resulted from this test.
Test SBFT5: Load-Deflection Relation

![Graph showing load-deflection relation for Test SBFT5](

Figure 5.98: Test SBFT5 load-deflection relation

### Table 5.13: SIWIS frame tests results versus FE model results

<table>
<thead>
<tr>
<th>Tests</th>
<th>Initial in-plane Stiffness (kip/in.)</th>
<th>Frame Load Corresponding to SIWIS Elements Failure (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3rd Story Left Bay</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test</td>
</tr>
<tr>
<td>SBFT1</td>
<td>26.0</td>
<td>26.0</td>
</tr>
<tr>
<td>SBFT2</td>
<td>26.2</td>
<td>26.0</td>
</tr>
<tr>
<td>SBFT3</td>
<td>25.1</td>
<td>25.0</td>
</tr>
<tr>
<td>SBFT4</td>
<td>25.2</td>
<td>25.0</td>
</tr>
<tr>
<td>SBFT5</td>
<td>15.5</td>
<td>15.6</td>
</tr>
<tr>
<td>SPFT1</td>
<td>6.5</td>
<td>7.3</td>
</tr>
<tr>
<td>SPFT2</td>
<td>10.4</td>
<td>9.8</td>
</tr>
</tbody>
</table>

Notes: ¹ TS: Test stopped, ² NF: Not failed
5.7.8.4 FE Model Validation for SIWIS Braced Frame Tests (SBFT)

The developed finite element model for the test frame, which was presented in Section 4.8, was modified for each test based on the SIWIS element capacity pattern and bracing configurations and subjected to incrementally increasing displacement to simulate tests SBFT1 to SBFT5. The resulting load-deflection relations are shown in Figure 5.99 for tests SBFT1 and SBFT2, Figure 5.100 for tests SBFT3 and SBFT4, and Figure 5.101 for test SBFT5 along with the experimental results. The finite element model (FE Model) results for the failure of SIWIS elements and initial stiffness are also included in Table 5.13.

As can be seen from these figures, the finite element model quite closely predicted the test load-deflection curve. As listed in Table 5.13, the initial stiffness of the FE model curves is very close to the initial stiffness of the test curves. The FE model and test curves nearly coincided for the majority of the responses particularly up to the failure of the SIWIS elements.

Another important comparison between numerical and test results is sequence and load level of failure of SIWIS elements. For tests SBFT1 and SBFT2, the FE model predicts that the two SIWIS elements of each story occur simultaneously. The sequence of failure in both FE model and test results are from top story to bottom story. It should be remembered that the two tests were stopped before the failure of the 1st story’s SIWIS elements occurred. The FE model predicts that the failure of the SIWIS elements of the 3rd story occurs, first, at load level of 22800 lbs, then the failure of SIWIS elements of the 2nd story happens at load level of 32600 lbs, and finally, the failure of SIWIS elements of the 1st story occurs at load level of 36000 lbs. These load levels are fairly close to test results. The ratio between test failure loads and FE model failure loads vary from 0.95 to 1.18 for test SBFT1 and from 0.94 to 1.01 for test SBFT2.

For tests SBFT3 and SBFT4, the FE model predicts that the SIWIS elements of the 1st and 2nd stories fail first at load level of 24600 lbs followed by failure of the SIWIS elements of the 1st story at load level of 24700 lbs. This is comparatively close to the test results. It can be, practically, assumed that all SIWIS elements fail simultaneously. The ratio between the test and FE model failure loads vary from 0.87 to 1.06 for test SBFT3 and from 0.77 to 1.01 for test SBFT4. It can be seen from table 5.13 that the initial in-plane stiffness of the frame are almost the same in FE model and the two tests. It was 25.1 and 25.2 kips/in. for tests SBFT3 and SBFT4, respectively, and the FE model resulted in 25.0 kips/in. frame stiffness.

As can be seen from Figure 5.101, the FE model captures test SBFT5 results closely as well. The initial stiffness of the system is 15.5 and 15.6 kips/in. in the test and FE model,
respectively. The FE model predicts that only the SIWIS elements of the 3\textsuperscript{rd} story fail as was the case in the test. The FE model results in a failure load level of 13000 lbs and the test resulted in a load level of 12500 lbs.

![Tests SBFT1 and SBFT2: Load-Deflection Relation](image1)

Figure 5.99: Tests SBFT1 and SBFT2 load-deflection relation

![Tests SBFT3 and SBFT4: Load-Deflection Relation](image2)

Figure 5.100: Tests SBFT3 and SBFT4 load-deflection relation
For a more conclusive FE model verification, the load-deflection relations are developed for all three floor levels for the tests and FE model and shown in Figures 5.102 to 5.104. In these graphs, the vertical axis represents the measured load values at the 3rd floor level and the horizontal axis characterizes the three floor levels’ in-plane deflections. It can be seen from these figures that the FE model can closely capture the deflections of the other two floor levels as well. As mentioned before, in the conducted tests, the loadings were limited to eliminate the yielding of diagonal rods. However, in all graphs, the FE model results are extended beyond the test results.

As observed, there is fairly consistent agreement between the test results and FE model results. The potential source of the observed minor differences is subsequently discussed. In the computer model, identical elements and properties were considered for the two panels in the three stories. The same force-deformation responses were assumed for the lumber disks in both panels (same grade). However, it was observed in the SIWIS element test (Section 5.6) that there can be considerable differences between the stiffness and strength properties of lumber disks with the same thickness (grade). In fact, the results of the experimental tests also showed that there were sizeable differences between the force histories of the two SIWIS elements of the two panels of each story (Figure 5.88). Besides differences between lumber disks, differences in the horizontal and vertical alignments of steel rods of the SIWIS elements and the amount of tightening of rods can contribute to the differences. Differences between the diagonal elements in the six panels.
particularly the amount of their tightening (pre-tensioning load) can affect their stiffness and forces during the test. An additional possible source of the observed differences can be attributed to the fact that in the actual experiment, because of existing of slight gaps between some of the masonry wall panels and angle sections in the two sides of the beam, either a wood shim or a thin rubber was inserted. The placement of much softer shims (wood or rubber) as compared to masonry or steel material can affect the in-plane stiffness of the panel.

The existence of these actual differences between panels can affect their in-plane stiffness, which ultimately can influence the distribution of load and load path in the system. For instance, if the initial stiffness of either diagonal bracing, masonry wall, or SIWIS element of one panel is higher than the other panel, the stiffer panel will experience higher loads (absorb higher loads). This will cause transferring of higher forces to the SIWIS elements and as a result to the lumber disk in the stiffer panel. The consequence is that the lumber disk of the less stiff panel will fail slightly later as observed in some of the conducted tests. On the other hand, in the computer model, the two panels in each story have similar and identical stiffness and strength properties and the two lumber disks of each story have similar force histories and as a result, they fail simultaneously. Obviously, besides the mentioned reasons, simplification of numerical modeling (e.g., simplified force-deformation responses of different elements, material constitutions, and so on) can cause some differences between analytical and test results.

![Tests SBFT1 and SBFT2: Load Deflection Relations](image)

Figure 5.102: Tests SBFT1 and SBFT2 floor level’s load-deflection relations
Figure 5.103: Tests SBFT3 and SBFT4 floor level’s load-deflection relations

Figure 5.104: Test SBFT5 floor level’s load-deflection relations
5.7.8.5 SIWIS Pinned Frame Tests (SPFT)

Two tests were conducted on the pinned frame (with no diagonal bracings) with SIWIS elements in two different patterns. In test SPFT1, Grade 3, 2, and 1 SIWIS elements were considered for the 1st, 2nd, and 3rd stories, respectively. Test SPFT2 was carried out on frame with Grade 3 SIWIS elements for all three stories. Similar to previous tests, incrementally increasing displacement was applied at the 3rd floor level. The applied loading histories are shown in Figure 5.105 for the two tests. As can be seen from Figure 5.105, tests SPFT1 and SPFT2 took about 6 and 15 minutes to reach to a 3rd floor deflection of about 1.23 and 2.60 in., respectively. This resulted in an average loading rates of about 0.0032 and 0.0029 in./sec for the two tests, respectively. The resulting load-deflection relations are shown in Figure 5.106 and 5.107 for tests SPFT1 and SPFT2, respectively. The SIWIS elements force histories and the floor levels’ deflection histories are included in Appendix G. The SIWIS elements failure loads are listed in Table 5.13 along with the initial stiffness of the system.

Figure 5.105: SIWIS pinned frame tests (SPFT) Loading history
Both tests showed similar overall responses. In both tests, only the SIWIS elements of the 3rd story failed, after which the resistance of the system almost dropped to zero. This was expected, since after the failure of the SIWIS elements in the 3rd story, there is no in-plane resisting element such as diagonal bracings. Test SPFT1 resulted in initial stiffness of 6.5 kips/in., while it was 10.4 kips/in. in test SPFT2. The difference between the two in-plane stiffnesses shows that the axial stiffness of the lumber disk, and as a result, the SIWIS element has a significant influence in overall stiffness of the system.

In test SPFT1, the two SIWIS elements of the 3rd story failed simultaneously at load level of 3250 lbs. In test SPFT2, the right side SIWIS element of the 3rd story failed first at load level of 7500 lbs, and then, the left side SIWIS element failed at load level of 5700 lbs. As mentioned, after failure of SIWIS elements of the 3rd story, the resisting load significantly dropped. At this stage, the 3rd story’s frame (beams and columns) freely deformed by increased applied displacement. The deformed shape of the test frame is shown in Figure 5.108 for test SPFT2. As can be seen from this figure, the SIWIS elements of the 2nd and 1st stories did not fail and thus, the presence of masonry wall stiffness eliminated the free movements of the 2nd and 1st floor levels. Figure 5.109 shows closer view of the 3rd story’s SIWIS element after its failure. It can be seen from this figure that the steel rod had punched through lumber disk and had moved nearly 2 in. inside the steel pipe. This observation demonstrates good performance for the SIWIS element and shows that after failure of the SIWIS element, the frame can freely deform without major interaction with the masonry wall panel. The floor levels’ deflection histories are shown in Figures 5.110 and 5.111 for tests SPFT1 and SPFT2, respectively. These figures confirm that after the failure of the 3rd story’s SIWIS elements, the in-plane deflection of the 3rd floor level increased by the applied displacement, while the deflections of the 1st and 2nd floor levels decreased or remained unchanged.
Figure 5.106: Test SPFT1 load-deflection relation
Figure 5.107: Test SPFT2 load-deflection relation
Figure 5.108: Frame deflection in test SPFT2 after failure of SIWIS element

Figure 5.109: Punching through and failure of SIWIS element in test SPFT2
Figure 5.110: Test SPFT1 floor level’s deflection histories

Figure 5.111: Test SPFT2 floor level’s deflection histories
5.7.8.6 FE Model Validation for SIWIS Pinned Frame Tests (SPFT)

For tests SPFT1 and SPFT2, the test results are compared with the FE model results in this section. The test and FE model load-deflection relations are shown in Figure 5.112 for both tests. As can be seen from this figure, the FE model predicted the test results fairly closely. However, the prediction is not as good as the previous tests (tests BFT and SBFT). The initial stiffness of the tests were 6.5 and 10.4 kips/in., while they were 7.3 and 9.8 kips/in. in the FE model. In test SPFT1, the two SIWIS elements of the 3rd story failed at load level of 3250 lbs. They failed at load level of 3000 lbs in FE model. The FE model predicted that in test SPFT2, the SIWIS elements of the 3rd story fail simultaneously at load level of 7200 lbs. While, test results showed that the right side SIWIS element of the 3rd story failed first at load level of 7500 lbs followed by failure of the left side SIWIS element at 5700 lbs. The discussion about the sources and reasons for differences between the FE model and test results for test SBFT series are true for these two tests as well. Further, since there is no bracing element in these tests, the response of the pinned frame is greatly affected with the response of the SIWIS element (the force-deformation of lumber disk). It can be seen from Figure 5.112 that the overall response of the FE model is very similar to the assumed simplified force-deformation response of the lumber disk as presented in Section 4.8 (Figure 4.53).

The FE model predicted the same response for the system in both tests. It predicted that after the failure of the 3rd story’s SIWIS elements, the 3rd floor level deforms freely with no in-plane resistance. This can be clearly seen in Figure 5.113, which shows the deformed shape of the FE model for test SBFT2.
Figure 5.112: Test SPFT1 and 2 load-deflection relations

Figure 5.113: Frame deflection in test SPFT2 after failure of SIWIS element (ANSYS FE Model)
5.7.9 Concluding Remarks

Based on the results of this experimental testing program, the following highlighted and core conclusions are accomplished:

1. The experimental results showed that an SIWIS system works well as a “seismic isolation” system for infilled frames to save the infill wall from any damage. The “fuse” type SIWIS elements, first, use the beneficial stiffness and strength effects of the masonry wall up to a predefined point, after which they limit the participation of masonry walls in in-plane resistance.

2. The compression design for SIWIS element using pipe, seat disk, lumber disk, rod, and end plate demonstrated good performance. It provided a rigid link between frame and wall followed by their disengagement, after which adequate room was supplied for independent frame movements. The design also provided easy and practical maneuver and accessibility.

3. Frames with SIWIS elements showed an initial linear response followed by stiffness degradation due to progressive cracking of lumber disks of SIWIS elements. There were drops in resisting load levels after failure of each SIWIS element.

4. The response of the frame with SIWIS elements is significantly affected by the stiffness and strength properties of the SIWIS elements.

5. The beneficial effects of the SIWIS system are greater in more flexible frames (i.e., pinned and moment-resisting frames) compared to more stiff frames (i.e., braced frames and frames with shear walls).

6. Besides moment-resisting frames, the SIWIS system can be effectively used for pinned frames as well.
7. The two identical tests (tests SBFT1 and SBFT2 and tests SBFT3 and SBFT4) did not give fully consistent results (i.e., load-deflection curve, SIWIS element force histories, and SIWIS element failure load and sequence). However, they were reasonably close.

8. The masonry wall configuration was satisfactory. The vertical tie-down and base slide preventing systems presented good performance.

9. The developed finite element models captured the test results fairly closely as summarized below:
   a. The FE model load-deflection curve was fairly close to the test curve for bare frame and SIWIS braced frame tests and it was acceptably close for SIWIS pinned frame tests.
   b. The FE model predicted the in-plane stiffness of bare frame tests within 2%, SIWIS braced frame tests within 2%, and SIWIS pinned frame tests within 11%.
   c. The FE model predicted the failure sequence of SIWIS elements in elevation; it predicted that the two SIWIS elements of each story will fail simultaneously. However, this is unlikely for all cases, and it was not the case in some tests.
   d. The failure loads of the SIWIS elements were predicted by FE model for majority of them within 6%.
   e. The FE model predicted the failure mechanism of SIWIS pinned frame tests.
   f. The FE model fairly closely captured the 2nd and 3rd floor levels’ load-deflection relations as well as the 1st floor level.
6.1 Strength of Masonry Walls in SIWIS System

6.1.1 Introduction

The proposed SIWIS system is intended to prevent masonry infill walls from experiencing any damage in strong wind and earthquake activities. A fuse-like element should always disengage the masonry wall from the frame well before the wall reaches its damaging state. For the purpose of SIWIS system design, the damage level can be defined to be either initial cracks or major cracks. The initial and major cracking loads of a masonry wall can be approximated to be about 40% and 70% of its ultimate load, respectively (e.g., Abrams 1992 and Tomazevic 1999). Thus, the ultimate load (strength) of a masonry wall in SIWIS system needs to be predicted. The ultimate load of an unreinforced masonry wall is the minimum of its shear and flexural capacities. It is often preferred to eliminate the flexural failure of a masonry wall by providing sufficient strength for a vertical element that ties the wall (usually top) to the bottom. In this section, with the use of empirical equations, the in-plane strength of masonry wall in an SIWIS system is estimated. For this purpose, first, the empirical equations discussed in the literature review are used to predict the strength of a single wall test specimen. Then, those empirical equations with better estimation are considered to develop a series of expressions for in-plane strength of a masonry wall in an SIWIS system.

6.1.2 Application of Empirical Equations for Test Wall Specimen

Two wall specimens were tested in the experimental program (Section 5.2). They failed in diagonal shear mode at maximum loads of 22.3 and 24.3 kips. Thus, an average of 23.3 kips was considered for the in-plane strength of the testing wall specimen.
6.1.2.1 Compressive and Tensile Strength of Masonry

The compressive test on three masonry prisms resulted in a compressive strength of $f'_{m} = 3600$ psi for masonry. The tensile strength of masonry is required for use in some of the empirical equations. In the case of a lack of test results, the tensile strength of masonry can be approximated based on the compressive strength of masonry. The Uniform Building Code (UBC 1997) specifies the tensile strength as follows:

$$f_t = 2.5 \sqrt{f'_{m}} \quad \text{For Partially-Grouted} \quad (6.1a)$$

$$f_t = 4.0 \sqrt{f'_{m}} \quad \text{For Fully-Grouted} \quad (6.1b)$$

Alternatively, according to Tomazevic (1999), based on data analysis on a large number of test results, the ratio between the tensile and compressive strength of masonry varies from 0.03 to 0.09 with an overall average of 0.05. Thus, according to Tomazevic (1999), the tensile strength of masonry can be approximated as follows:

$$f_t = 0.05 f'_{m} \quad (6.2)$$

By applying these two approaches, tensile strengths of $f_t = 240$ psi and $f_t = 180$ psi, respectively, result for the masonry material of the case under study. Since, the tensile strength of masonry is needed for use in the empirical expression proposed by Tomazevic and Lutman (1988), the second value of $f_t = 180$ psi, which was proposed by the same author, is adopted here.

6.1.2.2 Average Compressive Stress ($\sigma_c$)

In the setup used for the masonry wall in an SIWIS system in the experimental program conducted in this study, the axial compressive load $N$ (tie down load) depends on the applied horizontal load $V$. Figure 6.1 shows the free body diagram of the masonry wall panel. With reference to this figure, by taking moment about the bottom-left corner, Equation (6.3) is obtained for the axial load $N$, in which, $l$, $l'$, $h$, and $h'$ are dimensions shown in Figure 6.1.
Assuming \( l' = 0.05l \sim 0.10l \) and \( h' = 0.05h \sim 0.10h \) and substituting \( \sigma_v = N/A \) in Equation (6.3), Equation (6.4) results for the average axial compressive stress:

\[
\sigma_v = \frac{h}{lA} V
\]

Figure 6.1: Masonry wall panel subjected to in-plane load

### 6.1.2.3 Shear Strength of Test Wall Specimen

The empirical equations reviewed in Section 2.4 are considered here for in-plane shear strength prediction of the test wall specimen. Since the cohesion and the coefficient of friction are not available for the wall material, those equations based on friction theory [Eurocode 6 (1996) and Magenes and Calvi (1997)] were not considered here. The following parameters were considered for the masonry wall test specimen.

- \( L = 32 \) in. (length of wall)
- \( l = 28 \) in. (distance between tie down element and the opposite corner of wall)
- \( h = 20.25 \) in. (height of loading point)
- \( t = 3.5 \) in. (thickness of wall)
- \( A = Lt = 32 \times 3.5 = 112 \) in\(^2\) (horizontal cross sectional area of wall)
\[ A_{sh} = \rho_h = \rho_v = 0 \] (unreinforced masonry wall)

\[ r = \frac{h}{L} = \frac{20.25}{32} = 0.63 \] (aspect ratio, Equation (2.6))

\[ b = 1.0 \] (shear stress distribution factor, Equation (2.26))

\[ \psi' = 1.0 \] (cantilever walls)

\[ \alpha_v = \psi' r = 1.0 \times 0.63 = 0.63 \] (shear ratio, Equation (2.20))

\[ C_d = 2.8 - 1.6\alpha_v = 2.8 - 1.6(0.63) = 1.792 \] (Equation (2.34))

\[ \sigma_v = \frac{h}{lA} V = \frac{20.25}{28 \times 112} V = 0.0065V \] (average compressive stress, Equation (6.4))

A summary of calculations are presented next for each empirical equation.

**Fattal (1993), Equation (2.7a):**

\[ V = (0.012 f_m + 0.2\sigma_v) A = (0.012 \times 3600 + 0.2 \times 0.0065V) \times 112 = 4838 + 0.1456V \]

\[ \Rightarrow \quad V = 5.7 \quad \text{kips} \]

**Shing et al. (1990), Equation (2.11):**

\[ V = (0.0018\sigma_v + 2) A \sqrt{f_m} = (0.0018 \times 0.0065V + 2) \times 112 \sqrt{3600} = 0.0786V + 13440 \]

\[ \Rightarrow \quad V = 14.6 \quad \text{kips} \]

**Shing et al. (1993), Equation (2.16):**

\[ V = 0.04 f_m \left( 1 - \frac{4.5\sigma_v A_n}{f_m} + 0.25\sigma_v A_n \right) \]

\[ V = 0.04(3600) \left( 1 - \frac{4.5(0.0065V)}{3600} \times 112 + 0.25 \times 0.0065V \times 112 \right) \]

\[ \Rightarrow \quad V = 19716 \sqrt{1 - \frac{V}{123077}} \quad \Rightarrow \quad V = 18.2 \quad \text{kips (trial and error)} \]

**Tomazevic and Lutman (1988), Equation (2.25):**

\[ V = A \left( \frac{f_t}{b} \right) \left( 1 + \frac{\sigma_v}{f_t} \right) = 112 \left( \frac{180}{1.0} \right) \left( 1 + \frac{0.0065V}{180} \right) = 20160 \left( 1 + \frac{V}{27690} \right) \]

\[ \Rightarrow \quad V = 28.8 \quad \text{kips (trial and error)} \]
Guiqiu et al. (1997), Equation (2.23):
\[
\psi = 0.96 - 0.68 \log(r) = 0.96 - 0.68 \log(0.63) = 1.096 \\
V = f'_m \left[ 0.02 + 0.88 \left( \frac{\sigma_c}{f'_m} \right) - 0.9 \left( \frac{\sigma_c}{f'_m} \right)^2 \right] \psi A \\
V = 3600 \left[ 0.02 + 0.88 \left( \frac{0.0065V}{3600} \right) - 0.9 \left( \frac{0.0065V}{3600} \right)^2 \right] \times 1.096 \times 112 \\
\Rightarrow V = 26.6 \text{ kips (trial and error)}
\]

UBC (1997), Equation (2.31):
\[
V = C_d A_e f'_m = 1.792 \times 112 \sqrt{3600} \quad \Rightarrow \quad V = 12.0 \text{ kips}
\]

\[
V = c_3 (4 - 1.75 \alpha_v) A_e f'_m + 0.25 A_o \sigma_c \\
V = 1.0 (4 - 1.75(0.63)) \times 112 \sqrt{3600} + 0.25 \times 112 \times 0.0065 V \\
V = 19471 + 0.182 V \quad \Rightarrow \quad V = 23.8 \text{ kips}
\]

6.1.2.4 Flexural Strength of Test Wall Specimen

Figure 6.2 shows the free body diagram of the masonry wall panel along with the strain distributions that were considered in the experimental test. With reference to this figure, the flexural resistance of the test masonry wall is calculated as subsequently explained. The forces equilibrium equation in vertical direction results in the following:

\[
C = T \quad \Rightarrow \quad \kappa f'_m \tau a = A_s f_y 
\]

(6.5)

Two ½”-13 steel threaded rods were used in this test (one in each side). The effective area of a single threaded rod is calculated by Equation (6.6), in which, \( D \) is the nominal diameter of rod and \( n \) is the number of threads per inch.

\[
A_{ei} = \frac{\pi}{4} \left( D - \frac{0.9743}{n} \right)^2
\]

(6.6)
For a single $\frac{1}{2}$-13" threaded rod, Equation (6.6) results in the following effective area:

$$A_{e} = \frac{\pi}{4} \left( 0.5 - \frac{0.9743}{13} \right)^2 = 0.142 \text{ in}^2$$

Assuming $\kappa = 0.85$ and $a = 0.85c$ and substituting the steel rod properties into Equation (6.5), the following is resulted:

$$c = \frac{0.142 \times 2 \times 125000}{0.85(3600)(3.5)(0.85)} = 3.9 \text{ in.}$$

![Figure 6.2: Flexural capacity calculation of the test wall specimen](image)

Assuming linear strain distribution along the horizontal cross section of the wall panel as shown in Figure 6.2 and taking $\varepsilon_m = 0.003$, yielding of steel threaded rods is checked as follows:

$$\varepsilon_s = \frac{(l-c)}{c} \varepsilon_m$$  \hspace{1cm} (6.7)

$$\varepsilon_s = \frac{(28-3.9)}{3.9} \times 0.003 = 0.018 > \varepsilon_y = \frac{f_y}{E_s} = \frac{125}{29000} = 0.004$$  Thus steel rods yield.
The nominal flexural resistance of the masonry wall is calculated as follows:

\[ M_n = A_f f_y (l - \frac{a}{2}) \quad (6.8) \]

\[ M_n = 0.142 \times 2 \times 125000 \left( 28 - \frac{0.85 \times 3.9}{2} \right) / 1000 = 935 \text{ kips-in.} \]

Finally, the in-plane flexural resistance of the wall is calculated using Equation (2.40) as follows:

\[ V_{flexural} = \frac{M_n}{\psi'h} = \frac{935}{1.0 \times 20.25} = 46.2 \text{ kips} \]

### 6.1.2.5 Results and Discussion

The resulting in-plane resistances of the test masonry wall specimen including shear resistance, flexural resistance, and test result are summarized in Table 6.1. In the second and third rows of this table, the strengths and the ratio between calculated strengths to test result are listed, respectively. It can be seen from this table that the flexural strength of the wall is about two times the shear strength, which is preferred. It is usually desirable to use the full capacity of masonry material and eliminate flexural failure of masonry walls.

<table>
<thead>
<tr>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear</td>
<td>5.7</td>
<td>14.6</td>
<td>18.2</td>
<td>26.6</td>
<td>28.8</td>
<td>12.0</td>
<td>23.8</td>
<td>46.2</td>
<td>23.3</td>
</tr>
<tr>
<td>Ratio</td>
<td>(0.24)</td>
<td>(0.63)</td>
<td>(0.78)</td>
<td>(1.14)</td>
<td>(1.24)</td>
<td>(0.52)</td>
<td>(1.02)</td>
<td>NA</td>
<td>(1.00)</td>
</tr>
</tbody>
</table>

The test and calculated shear resistances are also shown in charts in Figure 6.3. As can be seen from Figure 6.3, the Fattal (1993) and UBC (1997) equations resulted in highly conservative values. The Fattal (1993) equation was mainly developed to estimate the shear resistance of reinforced masonry walls and thus, it is not considered here for unreinforced masonry walls. Although the UBC (1997) equation was entitled as “nominal shear strength” of masonry walls, but it was basically developed for design purposes. Furthermore, the UBC (1997) equation lacks...
terms related to the resistance of the axial compressive stress. Therefore, it is predictable that this equation may result in overly conservative values. The Shing et al. (1990) equation also resulted in lower resistance compared to the test result. However, the Shing et al. (1993) equation gave better results. The other three equations including Tomazevic and Lutman (1988), Guiqiu et al. (1997), and NEHRP (2000) & MSJC (2002) predicted the test results fairly well.

![Masonry Wall In-Plane Shear Strength](image)

**Figure 6.3:** In-plane shear strength of masonry wall test specimen

### 6.1.3 Development of Expressions for SIWIS Wall Shear Strength

Four of the empirical equations that predicted the shear strength of the test masonry wall specimen within 25% margin of the test result are selected here for further consideration. The objective is to use these four approaches and develop new expressions for direct estimation of shear strength for a masonry wall in an SIWIS system. For this purpose, the axial compressive stress in Equation (6.4) is substituted in the four selected expressions and the resulting equations are solved for the shear strength \( V \). The final results are subsequently presented.
**Expression 1:** (Based on the Shing et al. (1993) empirical equation)

\[ V_{\text{shear}} = Af_n^* X_1 \]  
\[ X_1 = \frac{0.04}{(1 - 0.25\beta)^2} \left[ \sqrt{0.0081\beta^2 + (1 - 0.25\beta)^2} - 0.09\beta \right] \]

**Expression 2:** (Based on the Guiqiu et al. (1997) empirical equation)

\[ V_{\text{shear}} = Af_n^* X_2 \]  
\[ X_2 = \frac{1}{1.8\beta^2} \left[ 0.88\beta - \frac{1}{\psi} + \sqrt{\left(0.88\beta - \frac{1}{\psi}\right)^2 + 0.072\beta^2} \right] \]

**Expression 3:** (Based on the Tomazevic and Lutman (1988) empirical equation)

\[ V_{\text{shear}} = \frac{\beta Af_t}{2b^2 \left[ 1 + \sqrt{1 + \frac{4b^2}{\beta^2}} \right]} \]

If the tensile strength of masonry \( f_t \) is approximated based on the compressive strength of masonry \( f_n^* \) applying the two approaches mentioned before, one according to UBC (1997) and one according to Tomazevic (1999), the following are resulted:

**Expression 3a:** \( f_t = 0.05f_n^* \): According to Tomazevic (1999)

\[ V_{\text{shear}} = Af_n^* X_{3a} \]  
\[ X_{3a} = \frac{\beta}{40b^2} \left[ 1 + \sqrt{1 + \frac{4b^2}{\beta^2}} \right] \]

**Expression 3b:** \( f_t = 2.5\sqrt{f_{m}^*} \): According to UBC (1997) for partially-grouted masonry

\[ V_{\text{shear}} = A\sqrt{f_{m}^*} X_{3b} \]  
\[ X_{3b} = \frac{1.25\beta}{b^2} \left[ 1 + \sqrt{1 + \frac{4b^2}{\beta^2}} \right] \]
Expression 3c: \( f_r = 4\sqrt{f_m'} \): According to UBC (1997) for fully-grouted masonry

\[
V_{shear} = A\sqrt{f_m'}X_{3c} \quad (6.14a)
\]

\[
X_{3c} = \frac{2\beta}{b^2} \left[ 1 + \sqrt{1 + \frac{4b^2}{\beta^2}} \right] \quad (6.14b)
\]

Expression 4: (Based on the NEHRP (2000) & MSJC (2002) empirical equation)

\[
V_{shear} = A\sqrt{f_m'}X_{4}\quad (6.15a)
\]

\[
X_{4} = \frac{(4 - 1.75\alpha_v)}{1 - 0.25\beta} \quad (6.15b)
\]

Note: \( \alpha_v \) needs not to be taken greater than 1.0; \( X_{4_{max}} = 6 \) for \( \alpha_v < 0.25 \), and \( X_{4_{max}} = 4 \) for \( \alpha_v < 1.0 \), \( \varphi' = 1.0 \) for a cantilever wall (the case of wall in SIWIS system). In these expressions, dimensionless parameter \( \beta \) is defined as:

\[
\beta = \frac{h}{l} \quad (6.16)
\]

These expressions are developed in customized format in such a way that they are multiplication of a factor \( (X_i, i = 1, 2, 3a, 3b, 3c, \text{ and } 4) \) by either the term \( A\sqrt{f_m'} \) or the term \( Af_m'. \) \( X_1, X_2, \) and \( X_{3a} \) are dimensionless factors while \( X_{4b}, X_{3c}, \) and \( X_{4} \) have \( \text{psi} \) units.

6.1.4 Direct Strength Assessment Graphs

For simplification and direct assessment of shear strength of unreinforced masonry walls in an SIWIS system, graphs are developed for the four introduced expressions and shown in Figures H.1 to H.12 in Appendix H. In these figures, the horizontal axis presents the geometry related dimensionless parameter of \( \beta = \frac{h}{l} \) varying from 0.0 to 2.0 and the vertical axis gives the value of \( X_i \) factors. For the graphs except those for Expression 1, eight different values varying from 0.65 to 1.00 with 0.05 increments are considered for the \( l/L \) ratio. Expression 1 results in
the same values for all $l/L$ ratios. As an example, the strength of test masonry wall specimen is obtained using these graphs as follows:

$$\beta = \frac{h}{l} = \frac{20.25}{28} = 0.72$$

$$\frac{l}{L} = \frac{28}{32} = 0.875$$

$$Af_m'' = 112 \times 3600 = 403200 \text{ lbs}$$

$$A\sqrt{f_m''} = 112 \times \sqrt{3600} = 6720 \text{ in}^2 \sqrt{\text{psi}}$$

$X_1 = 0.0456$ (Figure H.1)

$X_2 = 0.065$ (Figure H.3)

$X_{3a} = 0.713$ (Figure H.5)

$X_4 = 3.53 \sqrt{\text{psi}}$ (Figure H.11)

Expression 1: $V = X_1 Af_m'' = 0.0456 \times 403200 / 1000 = 18.4 \text{ kips}$

Expression 2: $V = X_2 Af_m'' = 0.065 \times 403200 / 1000 = 26.2 \text{ kips}$

Expression 3a: $V = X_{3a} Af_m'' = 0.0713 \times 403200 / 1000 = 28.7 \text{ kips}$

Expression 4: $V = X_4 A\sqrt{f_m''} = 3.53 \times 6720 / 1000 = 23.7 \text{ kips}$

These capacity values, which were obtained using the developed graphs for Expressions 1, 2, 3a, and 4, can be compared with the results in Table 6.1. As expected, they are very close to the capacities obtained from empirical equations proposed by Shing et al. (1993), Guiqiu et al. (1997), Tomazevic and Lutman (1988), and NEHRP (2000) & MSJC (2002), respectively. Expression 4 results in very close capacity to the test capacity (23.3 kips) compared to the other three expressions. Expression 1 and 3a give the lowest and highest values, respectively. These can be used as the lower and higher limits for the in-plane capacity of a masonry wall in SIWIS system.
6.2 SIWIS Element Capacity Arrangement

6.2.1 Introduction

In a typical multi-story building, there are larger story shear forces in lower stories relative to upper stories in an earthquake or wind. Thus, as observed in numerical analysis in Chapter 5, if the same capacities are used for SIWIS elements in all stories, those in lower stories will fail first. This sequence of failure of SIWIS elements can result in development of a soft story, which can be unfavorable for the structure. A desirable sequence of failure of SIWIS elements can be obtained by specifying SIWIS elements with different capacities (grades) in elevation. In this section, a method is developed to design an SIWIS elements capacity arrangement in multi-bay multi-story buildings.

6.2.2 Development of Design Method

A multi-bay multi-story frame subjected to design seismic loads is shown in Figure 6.4. The story shear force is the sum of applied loads at and above the story under consideration as Equation (6.17), in which, $V_{ai}^{al}$ is the applied story shear force; $F_j$ is the applied lateral load at the story level of $i$, and $n$ is the number of stories.

$$V_{ai}^{al} = \sum_{j=i}^{n} F_j$$  \hspace{1cm} (6.17)

On the other hand, the story shear resistance, $V_{i}^{rs}$, is the sum of the resistances of masonry infill walls integrated with SIWIS elements, $V_{i}^{sc}$, the resistance of frame, $V_{i}^{fr}$, and the resistance of bracings, $V_{i}^{br}$, as Equation (6.18).

$$V_{i}^{rs} = V_{i}^{sc} + V_{i}^{fr} + V_{i}^{br}$$  \hspace{1cm} (6.18)

The share of resistance of each term is related to their relative stiffness as follows:
\[ V_{ij}^{sc} = \sum_{j=1}^{m} V_{ij}^{sc} = \sum_{j=1}^{m} \left[ K_{ij}^{sc} \right] \]  
(6.19)

\[ V_{ij}^{fr} = \sum_{j=1}^{m} V_{ij}^{fr} = \sum_{j=1}^{m} \left[ K_{ij}^{fr} F_{ij} \right] \]  
(6.20)

\[ V_{ij}^{br} = \sum_{j=1}^{m} V_{ij}^{br} = \sum_{j=1}^{m} \left[ K_{ij}^{br} B_{ij} \right] \]  
(6.21)

In these equations, \( C_{ij} \), \( F_{ij} \), and \( B_{ij} \) are the SIWIS element capacity, the frame resistance, and the bracing resistance for story \( i \) and bay \( j \), and \( m \) is the number of bays. Furthermore, \( K_{ij}^{sc} \), \( K_{ij}^{fr} \), and \( K_{ij}^{br} \) are, respectively, the in-plane stiffnesses of the masonry wall through an SIWIS element, the in-plane stiffness of the frame, and the in-plane stiffness of the bracing for story \( i \) and bay \( j \). Finally, \( \sum K_j \) is the sum of the stiffnesses of all members in story \( i \) as shown in Equation (6.22).

\[ \sum K_i = \sum_{j=1}^{m} \left( K_{ij}^{sc} + K_{ij}^{fr} + K_{ij}^{br} \right) \]  
(6.22)

For a typical building structure, the in-plane stiffness of each member including masonry wall, frame, and bracing can be calculated as subsequently explained for each member. For a cantilevered masonry wall, which is the case in an SIWIS system, deflection at the free end due to in-plane load \( P \) at the free end (Figure 6.5) is given by the following equation:

\[ \Delta_{wall} = \frac{Ph^3}{3E_m I} + \frac{1.2Ph}{AE_v} \]  
(6.23)

In Equation (6.23), \( h \) is the height of the location of load \( P \); \( A \) is cross-sectional area of wall; \( I \) is moment of inertia of the wall in the direction of bending; \( E_m \) and \( E_v \) are, respectively, the modulus of elasticity and rigidity for masonry. In Equation (6.23), the 1.2 factor is the shape coefficient for a rectangular wall. By substituting \( E_v = 0.4E_m \), \( A = tL \), and \( I = tL^3/12 \) in Equation (6.23), in which \( t \) and \( L \) are the thickness and length of wall, the following equation results for the stiffness of wall, \( K_{wall} \):
\[ K^{wall} = \frac{E_m t}{4 \left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right)} \]  \hspace{1cm} (6.24)

The modulus of elasticity of masonry, \( E_m \), can be estimated based on the compressive strength of masonry, \( f'_m \). According to UBC (1997):

\[ E_m = 750 f'_m \leq 3,000,000 \text{ psi} \]  \hspace{1cm} (6.25)

MSJC (2002) specifies different values for clay and concrete masonry as below:

\[ E_m = 700 f'_m \] \hspace{1cm} for clay masonry \hspace{1cm} (6.26a)

\[ E_m = 900 f'_m \] \hspace{1cm} for concrete masonry \hspace{1cm} (6.26b)

The masonry wall and SIWIS element are coupled together in serial fashion, and thus, the in-plane stiffness of the masonry wall through SIWIS elements is given by Equation (6.27), in which, \( K^{se} \) is the axial stiffness of the SIWIS element.

\[ K^{se} = \frac{K^{wall} K^{se}}{K^{wall} + K^{se}} \]  \hspace{1cm} (6.27)

Figure 6.5 shows two cases for the frame column: one for fixed-end column and one for free-end column (free to rotate). For these two cases, the in-plane stiffness of the column can be determined by the following equations:

\[ K^{fr} = \frac{12 E_s I}{h^3} \] \hspace{1cm} for fixed-end \hspace{1cm} (6.28a)

\[ K^{fr} = \frac{3 E_s I}{h^3} \] \hspace{1cm} for free-end \hspace{1cm} (6.28b)

For a panel with diagonal tension-only, concentrically X bracing (Figure 6.5), Equation (6.29) gives the in-plane stiffness, in which, \( A \) is the cross section area of the bracing element; \( L \) is its length, and \( \theta \) is the horizontal angle of the bracing.
\[ K^{br} = \frac{E_i A}{L} \cos^2(\theta) \] (6.29)

With the distribution of the required story shear forces in elevation \( (V_i^{sd}) \) known, to achieve a desirable failure sequence of SIWIS elements in elevation, the SIWIS elements’ capacities should be specified in such a way that the ratios of the provided story shear resistance \( (V_i^{ns}) \) to required story shear forces \( (V_i^{sd}) \) match the preferred failure sequence. The order of this ratio in elevation determines the order of failure of SIWIS elements. Since it is desirable for SIWIS elements to fail from top stories to the bottom, these ratios should be in increasing order from top to bottom stories. To illustrate this design method, three design examples are considered in the next section.
Figure 6.5: In-plane stiffness of masonry wall (top), frame column (middle), and frame bracing (bottom)
6.3 Frame Design Considerations

In the proposed SIWIS system, there is interaction between the frame and the wall panel at the locations of SIWIS element, tie-down element, and compression toe of the wall. These interactions affect the bending moment, axial, and shear forces of the frame. After breaking of the SIWIS element, there will be no interaction between the frame and the wall, after which the frame will act as a bare frame. The effects of SIWIS system on the frame before breaking of the fuse-like element and the necessary frame checks are discussed in this section. A practical frame design method is introduced that can be used in commercial analysis and design software.

6.3.1 Bending Moment and Shear Force

Figure 6.6 shows an infilled frame equipped with an SIWIS system subjected to in-plane load from left to right. The compression side SIWIS element (left side) transfers load from the frame to the masonry wall panel. As a result, in the location of the SIWIS element, the structural frame (column) experiences a concentrated horizontal compression force of $C$. The vertical element that ties the top of the wall to the bottom beam (left side) applies a vertical tension force of $T$ to the bottom beam. At the other side of the bottom beam, the wall applies vertical compressive force to the beam (compression toe in bottom right) and horizontal compression force to the beam-column joint.

Figure 6.7(a) shows the qualitative bending moment and shear force diagrams of a bare frame under an in-plane load. The bending moment and shear force diagram induced by the presence of an SIWIS device is also shown in Figure 6.7(b). The response of the frame is
considered to be linear before the failure of SIWIS elements, thus, with the use of superposition principle, the resulting forces of the frame are the sum of the forces of the bare frame and those induced by the SIWIS system as shown in Figure 6.7(c). In Figure 6.7, for simplification, the distributed compression stress at the compression toe of the wall is substituted by a concentrated force. For analysis and design purposes of a structural frame with an SIWIS system, it is proposed to substitute the presence of the SIWIS system by considering its reaction on the frame as new loading cases. The procedure is explained in details in a four-story building example in Section 6.4.5.

![Figure 6.7: Influence of SIWIS system in frame forces](image)

**Figure 6.7: Influence of SIWIS system in frame forces**

**6.3.2 Design Checks for Concentrated Forces**

Another design consideration for the frame is checking the column and beam for concentrated forces. The column experiences concentrated compressive force at the location of the SIWIS element and the beam is subjected to concentrated tensile force at the location of the
attachment of the tie-down element. The beam is also subjected to compressive force at the compression toe of the wall. This section presents a brief review of necessary checks and requirements of AISC-LRFD (2001) design code for concentrated forces on a steel frame. More complete and detailed information is available in the AISC-LRFD (2001) design code.

6.3.2.1 Local Flange Bending

The flange of the column or beam should be sufficiently rigid so that it will not buckle due to compressive or tensile force (Figure 6.8a). The nominal strength of the flange, $\phi R_n$, is given by Equation (6.30), in which, $t_f$ is the thickness of flange, and $F_{yf}$ is the specified yield stress of the flange. If the applied load exceeds the strength of the flange, transverse stiffeners are required.

$$\phi R_n = (0.9)(6.25t_f^2 F_{yf})$$

(6.30)

6.3.2.2 Local Web Yielding

The concentrated compressive or tensile force may cause local yielding of the web (Figure 6.8b). The nominal strength of the web is determined as follows:

- If the force is applied at a distance greater than the member depth from the end of the member, then:
  $$\phi R_n = (1.0)(5k + N)F_{yw} t_w$$
  (6.31a)

- If the force is applied at a distance equal or less than the member depth from the end of the member, then:
  $$\phi R_n = (1.0)(2.5k + N)F_{yw} t_w$$
  (6.31b)

In these equations, $t_w$ is the web thickness; $N$ is the length of bearing; $k$ is the distance from the outer face of the flange to the web toe of the fillet, and $F_{yw}$ is the specified yield stress of the web. In the case that the strength of the web is lower than the applied force, either a pair of transverse stiffeners or a doubler plate should be provided.
6.3.2.3 Web Crippling

When the concentrated compressive force is applied in the plane of the web, crippling of web may occur (Figure 6.8c). The nominal strength of the web to prevent web crippling is determined as follows:

- If the force is applied at a distance equal or less than the member depth from the end of the member, then:

\[
\phi R_w = (0.75)(135t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \]  

(6.32a)

- If the concentrated force is applied at a distance less than \( \frac{d}{2} \) from the end of the member, then:

\[
\phi R_w = (0.75)(68t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \text{ for } \frac{N}{d} \leq 0.2 \]  

(6.32b)

\[
\phi R_w = (0.75)(68t_w^2 \left[ 1 + \left( \frac{4N}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \text{ for } \frac{N}{d} > 0.2 \]  

(6.32c)

In these equations, \( d \) is the depth of the member. A transverse stiffener, a pair of transverse stiffeners, or a doubler plate are required, when strength is insufficient.

6.3.2.4 Sidesway Web Buckling

When the concentrated compressive force is applied to a laterally braced compression flange, the web is subjected to compression and the tension flange may buckle (Figure 6.8d). The sidesway buckling of the web can be prevented, if a proper lateral bracing system or stiffeners are provided at the load point. The design strength of the web is given as follows:

- If the compression flange is restrained against rotation and \( \frac{h}{t_w} \leq 2.3 \), then:

\[
\phi R_w = (0.85) \frac{C_f t_w^3 t_f}{h^2} \left[ 1 + 0.4 \left( \frac{h}{t_w} \right)^3 \right] \]  

(6.33a)
If the compression flange is not restrained against rotation and \( \frac{h}{t_w} \leq 1.7 \), then:

\[
\phi R_n = (0.85) \frac{C_r t_w^3 t_f}{h^2} \left[\frac{0.4 \left( \frac{h}{t_w} \right) \left( \frac{h}{l/b_f} \right)^3}{3}\right]
\]  

(6.33b)

In these equations, \( l \) is the largest laterally unbraced length along the flange at the point of the load; \( b_f \) is the flange width, and \( h \) is the clean distance between flanges. If the strength of the web given by the above equations is less than the applied force, local lateral bracing should be used at the flange, or a pair of transverse stiffeners or a doubler plate should be provided.

6.3.2.5 Compression Buckling of the Web

When concentrated compressive forces are applied to both flanges of a member at the same location, the slenderness ratio of the web should be limited to avoid the possibility of buckling. The strength of the member for web buckling is given by the following equation:

\[
\phi R_n = (0.90) \frac{4100 t_w^3 \sqrt{F_{yw}}}{h}
\]  

(6.34)

If the strength of the member is exceeded, either a single transverse stiffener, or pair of transverse stiffeners, or a doubler plate needs to be provided.

Figure 6.8: Steel member failure subjected to concentrated force
6.4 Design Examples

Three design examples are considered in this section to illustrate the application of an SIWIS system design procedure as well as to demonstrate the performances of an SIWIS system. First, an eight-story building is considered in Los Angeles, CA for observing the performance of an SIWIS system in a high earthquake zone. Then, the same eight-story building is considered in New York, NY with low seismic but high wind activity. Then, the use of an SIWIS system in a typical twenty-story building in Los Angeles, CA is evaluated. Finally, a four-story building with different bay and story sizes and partial infill walls are considered. Through the four-story building example, general practical methods are presented for: (1) SIWIS elements design and (2) analysis and design of the frame. The seismic and wind design forces are calculated using ASCE-7 (2002) assuming normal conditions, e.g., Seismic Use Group I, Site Class C, and R=8 (special steel moment resisting frame system) for eight and twenty-story buildings, and R=4.5 (intermediate steel moment resisting frame system) for four-story building for seismic, and Exposure B for wind.

6.4.1 Eight-Story Building, Los Angeles, CA

An eight-story office building as shown in Figure 6.9 is considered. The plan area is 80 feet by 75 feet and the story heights are all 12 feet. A total load of 150 pounds per square foot is considered for all levels. The longitudinal direction of this building with 4 bays is considered here. A masonry infill wall of 8 inches thickness and a masonry tensile strength of 125 psi is considered for all panels (medium wall).

![Figure 6.9: Eight-story office building example](image)
Expression 3 (Equation 6.11) was considered here for in-plane strength estimation of masonry walls. The initial cracking load of the masonry wall panel (taken as 40% of the ultimate resistance) is considered to be the limiting criterion. If, for design purposes, a safety factor of SF is considered, Equation (6.35) is obtained from Expression 3 for the capacity of SIWIS elements.

\[
C_{\text{max}} = \frac{0.2 \beta A_f}{\text{SF} \cdot b^2} \left[ 1 + \sqrt{1 + \frac{4b^2}{\beta^2}} \right]
\]  

(6.35)

A safety factor (SF) of 1.25 is chosen in this example. Equation (6.35) results in a maximum capacity of 90 kips for the SIWIS elements that is equivalent to the initial cracking load of the wall. The seismic and wind story shear forces for one longitudinal frame are calculated and summarized in Table 6.2 (2nd and 3rd columns). Since the seismic story shear forces are larger than the wind story shear forces in this example, the SIWIS elements are designed based on seismic loads and then checked for wind loads.

For a typical steel building, the resistance term related to the frame is often negligible, since the shear stiffness of the steel frame columns is small compared to the shear stiffness of the masonry wall panels. Thus, in Equation (6.18) the term related to resistance of the frame is ignored. Since, there are no bracing members in this example and the same walls (size, stiffness, and strength) are used, Equations (6.18) and (6.19) give the following simplified equation:

\[
\sum_{j=1}^{m} C_{ij} = V_i^{\text{se}} = V_i^{\text{exe}}
\]  

(6.36)

In order to determine a suitable capacity distribution for SIWIS elements in elevation, the ratio of seismic story shear force for each story, \( V_{i\text{se}} \), to the above story is calculated and listed in Table 6.2 (4th column). For instance, the ratio of 1.22 results for the 5th story by dividing the 5th story shear force (343 kips) by the 6th story shear force (282 kips). These ratios are used here to determine the distribution of SIWIS element capacities in elevation to achieve a desirable failure sequence. Since the preferred sequence of failure of SIWIS elements occurs from the top to bottom in a building, the ratios of SIWIS element capacities, \( V_i^{\text{exe}} \), in elevation should exceed the calculated ratios of story shear forces listed in Table 6.2 (4th column). The appropriate capacities of SIWIS elements are determined and listed in Table 6.2 (5th column) as subsequently explained.
Since the maximum considered wall strength is 90 kips, a capacity of 90 kips is assumed for the 1st story’s panels to give a capacity not larger than the masonry infill wall. The maximum capacity for the 2nd story’s SIWIS elements can be determined by dividing the considered capacity of SIWIS elements in the 1st story, which is 90 kips, by the calculated story shear ratio for the 1st story, which is 1.02. This results in a maximum capacity of 88 kips. Thus, if the capacity of SIWIS elements in the 2nd story is taken to be slightly smaller than 88 kips, they will fail after the SIWIS elements of the 1st story. A capacity of 85 kips is assumed for the 2nd story’s SIWIS elements. For the 3rd story, the maximum capacity is 85 kips/1.04 = 81 kips, thus a capacity of 75 kips is assumed. This procedure is continued and the results are listed in Table 6.2 (5th column). This completes the selection of the required SIWIS element capacities. However, to verify the design, the sequence of failure of SIWIS elements in elevation is checked next.

<table>
<thead>
<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>Seismic Story Shear Ratio</th>
<th>SE* Capacity (kip)</th>
<th>Story SE Shear Capacity (kip)</th>
<th>Story SE Capacity /Seismic Story Shear</th>
<th>Story SE Capacity /Wind Story Shear</th>
<th>SE Failure Sequence</th>
<th>SE/Wall Capacity</th>
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<td>0.81</td>
<td>7</td>
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</table>

* Note: SE stands for SIWIS element.

The sum of SIWIS elements capacities for each story is calculated by multiplying the capacity of each element (5th column) by the number of bays in each story as listed in Table 6.2 (6th column). This is in the case of using the same capacity for SIWIS elements in all bays of each story. The ratio of story SIWIS elements capacity (6th column) to the story shear force (2nd column for seismic and 3rd column for wind) is calculated and listed in Table 6.2 (7th column for seismic and 8th column for wind). The sequence of failure of SIWIS elements in elevation can be easily determined by comparing the order of these ratios as included in Table 6.2 (9th and 10th columns). As can be seen, for seismic load, the sequence is from top to bottom, as expected. For
wind load, all of these ratios (8th column) are greater than 1.00 and this means that none of SIWIS elements will fail under the wind load used. It can be seen from Table 6.2 that for this eight-story building example located in Los Angeles, the used SIWIS elements fail under the design seismic loads. The 8th story’s SIWIS elements fail first at 36 percent of the total seismic design load. The SIWIS elements of the 1st story fail last at 81 percent of the total seismic design load. Thus, if a weaker earthquake with effects less than 36 percent of the design earthquake occurs, none of the SIWIS elements will fail. The story capacity of SIWIS elements (6th column) are graphically compared with seismic and wind load story shears (2nd and 3rd columns) in Figure 6.10.

Figure 6.10: Story shear forces for eight-story building in Los Angeles, CA

In the last column in Table 6.2, the ratio of SIWIS element capacity to the masonry infill wall capacity (90 kips) is calculated for each story. These ratios show that the beneficial effects of the masonry walls are being effectively used. For instance, under design seismic loads, the walls of the 8th, 4th, and 1st stories become isolated at their 11, 72, and 100 percent capacities, respectively. It can be concluded for this example that wind load and low-to-minor earthquakes will not cause the SIWIS elements to fail. In moderate-to-strong earthquakes, some of the upper stories SIWIS elements will fail and they will all fail in a strong earthquake event. For instance, it can be predicted from Table 6.2 that if an earthquake with magnitude (effects) of 70 percent of the design earthquake occurs, only the SIWIS elements of top five stories (the 8th, 7th, 6th, 5th, and
4th stories) will fail. This example demonstrated that the proposed SIWIS system can act like a “smart system” to save the masonry walls from cracks and the frame from premature failure in winds and earthquakes. At the same time, this system can use the beneficial effects of masonry walls when and where it is proper and safe.

6.4.2 Eight-Story Building, New York, NY

In order to evaluate the performance of SIWIS system in areas with stronger wind loads, the same eight-story building is next assumed to be located in New York. Using the ASCE-7 (2002) building standard, seismic and wind story shear forces are calculated and summarized in Table 6.3. First, the same design (SIWIS element capacities) is considered for this example. The ratios of story capacity to story shear load are calculated and included in Table 6.3 (7th column for seismic and 8th column for wind). As can be seen from this table, due to lower seismic design loads for this case (New York) compared with the previous case (Los Angeles), none of the SIWIS elements fail under either wind or seismic design load.

Special care should be taken into account when none of SIWIS elements fail under seismic design load. The seismic story shear forces listed in Tables 6.2 and 6.3 are calculated based on the assumption of a fundamental period for moment resisting steel frames that are not enclosed or adjoined by more rigid components (masonry wall) and can deflect freely. The integration of masonry walls and frames will result in a stiffer structure and lower fundamental period and ultimately will result in higher seismic loads. To check the appropriateness of SIWIS elements design for the second case (New York), where the masonry walls will not be isolated from the frame under the applied in-plane loads in Table 6.3, the seismic design loads are re-calculated based on ASCE-7 (2002) by taking into account the integration of walls and frame (leading to smaller period and larger story shear) and the results are summarized in Table 6.4.

As can be seen from Table 6.4, higher seismic story shear forces result. However, none of the SIWIS elements fail. For this example, one may conclude that it is not necessary to use an SIWIS system in this example and the masonry walls can be traditionally integrated with the frame. This is correct and the designer may decide to choose this solution. However, it is always safe to use these fuse-like SIWIS elements. For instance, it is always possible that a wind or earthquake could activity cause forces higher than those considered in the codes. Furthermore, the designer can alternatively decide to use lighter masonry walls, which can lower the construction cost while maintaining structural performance, as considered next.
If 6 in. masonry walls with a lower tensile strength of 75 psi are considered (light wall), using Equation (6.35) a maximum capacity of 40 kips results for these lighter wall panels. Thus, the capacity of SIWIS elements needs to be limited to 40 kips. The same procedure, as explained, is applied and the SIWIS elements with capacities listed in Table 6.5 (5th column) are chosen. It can be seen from Table 6.5 that the lighter masonry walls can reach their 100, 63, and 13 percent capacities in the 1st, 4th, and 8th stories, respectively, with an overall average of 57 percent. This table shows that for this design, none of SIWIS elements fail as well. However, this final design with lighter masonry walls and safety SIWIS element is recommended. A graphic comparison of story SIWIS capacities and seismic and wind shear loads is also shown in Figure 6.11.

Table 6.3: Eight-story building design example, New York, NY (higher period)

<table>
<thead>
<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>Wind Story Shear Ratio</th>
<th>SE* Capacity (kip)</th>
<th>Story SE Capacity/Seismic Story Shear</th>
<th>Story SE Capacity/Wind Story Shear</th>
<th>SE Failure Sequence</th>
<th>SE /Wall Capacity</th>
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</table>

* Note: SE stands for SIWIS element.

Table 6.4: Eight-story building design example, New York, NY (lower period)

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<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>Wind Story Shear Ratio</th>
<th>SE* Capacity (kip)</th>
<th>Story SE Capacity/Seismic Story Shear</th>
<th>Story SE Capacity/Wind Story Shear</th>
<th>SE Failure Sequence</th>
<th>SE /Wall Capacity</th>
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* Note: SE stands for SIWIS element.
Table 6.5: Eight-story building design example, New York, NY (lower period, light walls)

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<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>Wind Story Shear Ratio</th>
<th>SE* Capacity (kip)</th>
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<td>80</td>
<td>1.9</td>
<td>2.2</td>
</tr>
<tr>
<td>4</td>
<td>48</td>
<td>47</td>
<td>1.32</td>
<td>25</td>
<td>100</td>
<td>2.1</td>
<td>2.1</td>
</tr>
<tr>
<td>3</td>
<td>53</td>
<td>59</td>
<td>1.25</td>
<td>30</td>
<td>120</td>
<td>2.3</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>56</td>
<td>71</td>
<td>1.21</td>
<td>35</td>
<td>140</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>1</td>
<td>58</td>
<td>84</td>
<td>1.18</td>
<td>40</td>
<td>160</td>
<td>2.8</td>
<td>1.9</td>
</tr>
</tbody>
</table>

* Note: SE stands for SIWIS element.

Figure 6.11: Story shear forces for eight-story building in New York, NY (higher period, light walls)
6.4.3 Twenty-Story Building, Los Angeles, CA

A twenty-story office building with special moment resisting steel frames located in Los Angeles as shown in Figure 6.12 is considered. The plan area is 120 feet by 100 feet and the story heights are all 12 feet. A total load of 120 pounds per square foot is considered for all levels. The longitudinal direction of this building with 6 bays is considered in this example. Similar to the eight-story building example, masonry infill walls of 8 inches thickness and masonry tensile strength of 125 psi is considered for all panels. Thus, the limit for capacity of SIWIS elements is 90 kips.

Figure 6.12: Twenty-story office building example

The wind and seismic design forces are calculated based on ACSE-7 (2002) and the same procedure as in the previous examples is applied. The SIWIS elements with capacities listed in
Table 6.6 (5th column) are chosen. In order to make it feasible and more practical, a total of 10 different grades (capacities) are considered for SIWIS elements. Thus, the two consecutive stories have the same grade SIWIS elements. It can be seen from Table 6.6 that for this twenty-story building example, the designed SIWIS elements fail under the design seismic loads. The 20th and 19th story’s SIWIS elements fail first at 40 percent of the total seismic design load. The SIWIS elements of the 1st and 2nd story fail last at 63 percent of the design seismic load. Therefore, if a weaker earthquake with effects less than 40 percent of the design earthquake occurs, none of the SIWIS elements will fail. It can be predicted from Table 6.6 that if an earthquake with amplitude of 50 percent of the design earthquake takes place, the SIWIS elements of the top eleven stories (from the 11th to 20th stories) will fail, afterward new SIWIS elements must be installed in these stories.

Table 6.6: Twenty-story building design example, Los Angeles, CA

<table>
<thead>
<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>Seismic Story Shear Ratio</th>
<th>SE* Capacity (kip)</th>
<th>Story SE Shear Capacity (kip)</th>
<th>Story SE Capacity /Seismic Story Shear</th>
<th>Story SE Capacity /Wind Story Shear</th>
<th>SE Failure Sequence</th>
<th>SE/Wall Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>116</td>
<td>7</td>
<td>NA</td>
<td>10</td>
<td>60</td>
<td>0.40</td>
<td>6.1</td>
<td>1</td>
<td>NA</td>
</tr>
<tr>
<td>19</td>
<td>221</td>
<td>20</td>
<td>1.91</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>316</td>
<td>32</td>
<td>1.43</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>401</td>
<td>45</td>
<td>1.27</td>
<td>25</td>
<td>150</td>
<td>0.42</td>
<td>4.0</td>
<td>2</td>
<td>NA</td>
</tr>
<tr>
<td>16</td>
<td>477</td>
<td>58</td>
<td>1.19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>541</td>
<td>70</td>
<td>1.14</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>14</td>
<td>604</td>
<td>83</td>
<td>1.11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>656</td>
<td>95</td>
<td>1.09</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>700</td>
<td>107</td>
<td>1.07</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>738</td>
<td>119</td>
<td>1.05</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>769</td>
<td>130</td>
<td>1.04</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>9</td>
<td>795</td>
<td>142</td>
<td>1.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>816</td>
<td>153</td>
<td>1.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>832</td>
<td>164</td>
<td>1.02</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>844</td>
<td>175</td>
<td>1.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>853</td>
<td>185</td>
<td>1.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>859</td>
<td>195</td>
<td>1.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>862</td>
<td>204</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>864</td>
<td>213</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>864</td>
<td>222</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Note: SE stands for SIWIS element.
As can be seen from Table 6.6, none of SIWIS elements fails under the design wind loads. The story capacity of SIWIS elements (6th column) are graphically compared with seismic and wind load story shear (2nd and 3rd columns) in Figure 6.13. In the last column in Table 6.6, the ratios of SIWIS element capacity to the masonry infill wall capacity show that the beneficial effects of the masonry walls are being effectively used. For example, under the design seismic loads, the walls of the 20th, 10th, and 1st stories disengage from the frame at their 11, 78, and 100 percent capacities, respectively.

Figure 6.13: Story shear forces for twenty-story building in Los Angeles, CA
6.4.4 Four-Story Building, Los Angeles, CA

A four-story building as shown in Figure 6.14 is considered. The structural system of this building located in Los Angeles, CA includes an intermediate steel moment resisting frame system (R=4.5). The plan area is 75 feet by 55 feet and the story heights are 15 feet for the first story and 12 feet for the next three stories. The north-south direction of this building with 4 bays is considered in this example (Frame 1 & 2). As shown in Figure 6.28, the fourth story of this frame has only three bays. All panels of this frame have infill walls except the first story's second panel (between Axis B and C), which is the entrance to the building. The other three panels of the first story have full infill walls, while the next three stories are partially infilled. A total load of 150 pounds per square foot is considered for all levels. Masonry infill walls of 8 inches thickness and masonry compressive strength of 2500 psi is considered for all panels. The main purpose of this example is to show the design procedure in varying conditions such as story height, panel lengths, and panels with no or partial infill walls.

![Figure 6.14: Four-story building example](image-url)
With the use of ASCE-7 (2002), seismic and wind design loads are calculated for this building and the resulting design loads and the story shear forces are shown in Figure 6.15 for Frame 1. Since the design seismic forces are larger than the design wind forces, the design will be conducted for seismic and then will be checked for wind. For this four-story frame, since the sizes of walls are not the same in all bays, the simplified design procedure presented for the two previous examples cannot be used. However, the same concept explained in Section 6.1 will be used for design purposes as presented in this section.

First, the capacity of the masonry wall in each panel is estimated with the use of Equation (6.35). In this equation, the tensile strength of masonry is assumed to be 5% of the compressive strength of masonry (Section 6.1.2.1) resulting in a tensile strength of 125 psi. The resulting capacities for masonry walls are included in Figure 6.15 for each panel. As an example, the capacity of the wall in the second story and the first bay is calculated as follows:

\[
\beta = \frac{h}{l} = \frac{7' - 1'}{16.5'} = 0.364
\]

\[
C_{\text{max}} = \frac{0.2}{1.25} \times \frac{0.364(17.5 \times 12 \times 8)(125)}{(1.0)^2} \left[ 1 + \sqrt{1 + \frac{4(1.0)^2}{(0.364)^2}} \right] = 80 \text{ kips}
\]

Next, the stiffness of each wall needs to be determined. The total deflection of the frame in each floor level is the sum of the deflection of the wall and the deflection of the column above.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Wind</th>
<th>Seismic</th>
<th>Seismic</th>
<th>Wind</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>136</td>
<td></td>
<td></td>
<td>136</td>
</tr>
<tr>
<td>8</td>
<td>113</td>
<td></td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>7</td>
<td>77</td>
<td></td>
<td></td>
<td>326</td>
</tr>
<tr>
<td>8</td>
<td>40</td>
<td></td>
<td></td>
<td>367</td>
</tr>
</tbody>
</table>

Figure 6.15: Applied seismic and wind loads and wall capacities

First, the capacity of the masonry wall in each panel is estimated with the use of Equation (6.35). In this equation, the tensile strength of masonry is assumed to be 5% of the compressive strength of masonry (Section 6.1.2.1) resulting in a tensile strength of 125 psi. The resulting capacities for masonry walls are included in Figure 6.15 for each panel. As an example, the capacity of the wall in the second story and the first bay is calculated as follows:

\[
\beta = \frac{h}{l} = \frac{7' - 1'}{16.5'} = 0.364
\]

\[
C_{\text{max}} = \frac{0.2}{1.25} \times \frac{0.364(17.5 \times 12 \times 8)(125)}{(1.0)^2} \left[ 1 + \sqrt{1 + \frac{4(1.0)^2}{(0.364)^2}} \right] = 80 \text{ kips}
\]

Next, the stiffness of each wall needs to be determined. The total deflection of the frame in each floor level is the sum of the deflection of the wall and the deflection of the column above.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Wind</th>
<th>Seismic</th>
<th>Seismic</th>
<th>Wind</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>136</td>
<td></td>
<td></td>
<td>136</td>
</tr>
<tr>
<td>8</td>
<td>113</td>
<td></td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>7</td>
<td>77</td>
<td></td>
<td></td>
<td>326</td>
</tr>
<tr>
<td>8</td>
<td>40</td>
<td></td>
<td></td>
<td>367</td>
</tr>
</tbody>
</table>

Figure 6.15: Applied seismic and wind loads and wall capacities

First, the capacity of the masonry wall in each panel is estimated with the use of Equation (6.35). In this equation, the tensile strength of masonry is assumed to be 5% of the compressive strength of masonry (Section 6.1.2.1) resulting in a tensile strength of 125 psi. The resulting capacities for masonry walls are included in Figure 6.15 for each panel. As an example, the capacity of the wall in the second story and the first bay is calculated as follows:

\[
\beta = \frac{h}{l} = \frac{7' - 1'}{16.5'} = 0.364
\]

\[
C_{\text{max}} = \frac{0.2}{1.25} \times \frac{0.364(17.5 \times 12 \times 8)(125)}{(1.0)^2} \left[ 1 + \sqrt{1 + \frac{4(1.0)^2}{(0.364)^2}} \right] = 80 \text{ kips}
\]

Next, the stiffness of each wall needs to be determined. The total deflection of the frame in each floor level is the sum of the deflection of the wall and the deflection of the column above.
the wall (along the free length of the column). The in-plane stiffness associated with these two
deflections are calculated according to Equations (6.24) and (6.28) for the wall and column,
respectively. UBC (1997) recommendation (Equation 6.25) was used to estimate the modulus of
elasticity of the masonry as:

\[ E_m = 750 f'_{m} = 750(2500) = 1,875,000 \text{ psi} \]

The in-plane stiffness of wall, \( K_{sc} \), and column, \( K_{fr} \), are calculated for each panel and the
total stiffness for the panel, \( K' \), is calculated as:

\[ K' = \frac{1}{\frac{1}{K_{sc}} + \frac{1}{K_{fr}}} \]  
(6.37)

The stiffness in the 2\textsuperscript{nd} story and the first bay is calculated here as an example:

\[ K_{sc} = \frac{187500 \times 8}{4\left(\frac{6}{17.5}\right)^3 + 3\left(\frac{6}{17.5}\right)} / 1000 = 12600 \text{ kips/in.} \]

\[ K_{fr} = \frac{12 \times 29000000 \times 2660}{(12 - 6) \times 12} / 1000 = 2480 \text{ kips/in.} \]

\[ K' = \frac{1}{\frac{1}{12600} + \frac{1}{2480}} = 2070 \text{ kips/in.} \]

Figure 6.16 shows the diagrammatic design procedure and the summary of calculations
for this example. The design procedure, which is subsequently explained, is a general method and
can be used for any multi-bay multi-story building with partial and full infill walls in some or all
panels and different story and bay sizes. The calculation related to each panel is included in the
panel under consideration in Figure 6.16. The top part of this figure presents the parameters and
formulations used in each panel. The wall, frame, and total stiffness are included in the top part of
each panel. The ratio of the resulting stiffness of each panel to the total stiffness of the story is
calculated as Equation (6.38) and listed in parenthesis for each panel (Step 1).

\[ K' = \frac{K'}{\sum_{\text{story}} K'} \]  
(6.38)
In the lower part of each panel in Figure 6.16, there are two columns. The first column presents the capacity of the SIWIS element, in which the first number, \( C_1 \), (top number) is the capacity of the wall (maximum allowed capacity of SIWIS element) as the first trial, the second number, \( C_2 \), (middle number) is the second trial, and the third number, \( C_3 \), (bottom number) is the third trial (the final design) for SIWIS element capacity. The second column represents the story shear resistance associated with the trial SIWIS element capacity.

The design of SIWIS element’s capacity starts from the first story and continues to the top story. For the first story, first, for each panel, the maximum allowable capacity of SIWIS element is considered as the first trial, \( C_1 \). The story shear resistance associated with this capacity SIWIS element, \( V^1 \), is calculated as the following equation and listed as the first number of the second column (Step 2).

\[
V^1 = \frac{C_1}{k'}
\]  

(6.39)

In the next step, the second trial for story shear, \( V^2 \), is selected to be the minimum of (1) \( V^1 \) of the four panels of the first story and (2) the story design shear force. The capacity of the SIWIS element associated with this story shear is calculated as Equation (6.39) and listed as the second trial, \( C_2 \), for each panel (Step 3).

\[
C_2 = V^2 \times K'
\]  

(6.40)

The third trial of SIWIS element capacity, \( C_3 \), is chosen as the final design. For this purpose, it can be considered that the grades of SIWIS elements are increasing in increments of 5 kips. Thus, if a capacity of 48 kips is calculated for \( C_2 \), it is recommended to take \( C_3 \) as 50 kips. Similarly, the story shear associated with this capacity is calculated as (Step 3):

\[
V^3 = \frac{C_3}{k'}
\]  

(6.41)

The smallest value between \( V^3 \) of the four panels of each story (should be relatively close to each other) is the final story shear resistance. Last, the ratio of the story shear resistance to story shear force is calculated (\( \alpha \)) and listed in the top left side of each story (Step 5).
This procedure is applied for the rest of the stories from bottom (the 2nd story) to top (the 4th story). The only difference between the procedure for the first story and the other stories is that in Step 2, \( V^2 \) is the smallest value between: (1) \( V^j \) of the panels; (2) the story design shear force, and (3) \( V^* \) as defined in Equation (6.42), in which, \( V^D_j \) is the story design shear force, \( \alpha_{i-1} \) is the ratio of story shear resistance to story shear force of the lower story, and \( \beta_i \) is a desirable coefficient that control the sequence of the failure of SIWIS elements in elevation. It is recommended to select a number between 0.85 and 0.95.

\[
V^*_i = \alpha_{i-1} \times \beta_i \times V^D_i
\]  

(6.42)

Figure 6.16: Calculations for four-story building example
The designed SIWIS elements are listed in Table 6.7 (4th to 7th columns). The $\alpha$ ratios (9th column) show that the sequence of failure is from top to bottom as expected. It is 0.54, 0.60, 0.64, and 0.68 for the 4th, 3rd, 2nd, and 1st stories, respectively. The wind $\alpha$ ratios (10th column) show that none of the SIWIS elements fail under wind load. The story capacity of SIWIS elements (8th column) are graphically compared with seismic and wind load story shear (2nd and 3rd columns) in Figure 6.17.

Table 6.7: Four-story building design example, Los Angeles, CA

<table>
<thead>
<tr>
<th>Story</th>
<th>Seismic Story Shear (kip)</th>
<th>Wind Story Shear (kip)</th>
<th>SIWIS Element Capacity (kip)</th>
<th>Story SE Capacity/Seismic Story Shear</th>
<th>Story SE Capacity/Wind Story Shear</th>
<th>SE Failure Sequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>136</td>
<td>4</td>
<td>20 20 35 NA</td>
<td>73</td>
<td>0.54</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>250</td>
<td>12</td>
<td>30 30 60 30</td>
<td>150</td>
<td>0.60</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>326</td>
<td>19</td>
<td>50 50 60 50</td>
<td>208</td>
<td>0.64</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>367</td>
<td>27</td>
<td>100 NA 55 100</td>
<td>250</td>
<td>0.68</td>
<td>4</td>
</tr>
</tbody>
</table>

* Note: SE stands for SIWIS element.

Figure 6.17: Story shear forces for four-story building in Los Angeles, CA
6.4.5 Frame Design Considerations for Four-Story Building

In this section, a practical method is introduced for analysis and design of frames with an SIWIS system. The four-story building is considered as an example to illustrate the details of the procedure. For this purpose, as explained in Section 6.3.1, concentrated forces are applied to the frame in each panel as representative of reaction forces of masonry walls and SIWIS elements. SAP2000 Version 8 computer commercial program (SAP2000 V.8 2002) is selected to model the four-story building. The reason for using SAP2000 is that this program is widely used in offices for analysis and design of structures.

First, four load cases representing concentrated loads induced by SIWIS elements and masonry walls of the four stories are considered as shown in Figure 6.18 along with design seismic load case. In order to observe and compare the effects of an SIWIS system in frame forces, the bending moment and shear forces of all members are graphically shown in Figures 6.19 to 22 with identical scale. It can be seen that the presence of SIWIS system introduces bending moments and shear forces in all members, particularly in columns and beams of the story under consideration. The effects of an SIWIS system located in lower stories are larger than the effects of those in higher stories. This is basically due to higher breaking loads for SIWIS elements (grades) in lower stories.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Seismic Load Case Ratio</th>
<th>1st Story SE Load Case Ratio</th>
<th>2nd Story SE Load Case Ratio</th>
<th>3rd Story SE Load Case Ratio</th>
<th>4th Story SE Load Case Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.54</td>
<td>0.54 = 0.79</td>
<td>0.54 = 0.84</td>
<td>0.54 = 0.90</td>
<td>0.54 = 1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.60</td>
<td>0.60 = 0.88</td>
<td>0.60 = 0.94</td>
<td>0.60 = 1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>0.64</td>
<td>0.64 = 0.94</td>
<td>0.64 = 1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>0.68</td>
<td>0.68 = 1.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

* Note: SE stands for SIWIS element.
Next, four load combinations as listed in Table 6.8 are defined and introduced to the model to observe the resulting frame forces. The $\alpha$ ratios that were calculated in Section 6.4.4 for this four-story building are used in these load combinations to obtain the correct ratios for each load case. For example, the first load combination is related to the time when the SIWIS elements of the 4th story are going to fail in 54% of total seismic load ($\alpha_4$ ratio). In this time, the SIWIS elements of the 3rd story are holding 90% of their capacities ($\alpha_4/\alpha_3 = 0.54/0.60 = 0.90$) and so on. The forth load combination is related to the time when the SIWIS elements of the 4th, 3rd, and 2nd stories have already failed and the SIWIS elements of the 1st story are going to fail in 68% of total seismic load ($\alpha_1$ ratio).

The new load combinations are added to the SAP2000 model of the building and the analysis results, including bending moments and shear forces, are shown in Figures 6.24 to 6.27 for the four load combinations, respectively. Comparison between these figures and Figure 6.19 indicates that for this particular example, the magnitude of bending moments and shear forces due to presence of SIWIS systems (load combinations 1 to 4) are lower than those due to seismic design load. However, all beams and columns need to be checked for new load combinations to ensure their design. To check the design of columns and beams, the “Steel Frame Design” feature of SAP2000 computer program is employed here. From available design codes, AISC-ASD 89 design code is selected. First, the frame subjected to seismic design load (Figure 6.19) is checked and the resulting stress ratios are shown in Figure 6.28. Next, the four load combinations (Figure 6.24 to 6.27) are considered and the resulting stress ratios for columns and beams are shown in Figure 6.29. It can be seen that all members are acceptable and do not need stronger sections. Also it can be seen that the stress ratios of all members for the four load combinations are lower than those for design seismic load. Thus, for this four-story building example, it can be concluded that the presence of the SIWIS system will not cause any column or beam to fail and the design based on design seismic load is sufficient.
Figure 6.18: Seismic load and story’s SIWIS elements loads on frame

Figure 6.19: Bending moment and shear force diagrams under seismic load case
Figure 6.23: Bending moment (left) and shear force (right) diagrams under the 1st, 2nd, 3rd, and 4th story SIWIS elements load case.
Figure 6.24: Bending moment and shear force diagrams under load combination 1
(None of SIWIS elements are failed)

Figure 6.25: Bending moment and shear force diagrams under load combination 2
(4th story’s SIWIS elements are failed)

Figure 6.26: Bending moment and shear force diagrams under load combination 3
(4th and 3rd stories’ SIWIS elements are failed)
Figure 6.27: Bending moment and shear force diagrams under load combination 4
(4th, 3rd, and 2nd stories’ SIWIS elements are failed)

Figure 6.28: Steel design results (stress ratio) for seismic load
Next, the deflection results for four floor levels are read from the output file and summarized in Table 6.9. In this table, four times are considered for the four load combinations representing the failure of each story’s SIWIS elements. For example, Time 1 is equal to the 54% of seismic load when the SIWIS elements of the fourth story are going to fail first. It can be seen from this table that the SIWIS system significantly reduces floor deflections. For instance, the deflection of the fourth floor in Time 1 and 4 is, respectively, 0.58 in. and 1.64 in. for frame with SIWIS system and 1.72 in. and 2.16 in. for bare frame. This observation is expected and in agreement with the findings in previous chapters as an advantage of the SIWIS system.

Table 6.9: Floor deflections for four-story building (in.)

<table>
<thead>
<tr>
<th>Floor Levels</th>
<th>Time 1 (0.54)</th>
<th>Time 2 (0.60)</th>
<th>Time 3 (0.64)</th>
<th>Time 4 (0.68)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Frame with SE*</td>
<td>Bare Frame</td>
<td>Frame with SE*</td>
<td>Bare Frame</td>
</tr>
<tr>
<td>4th floor</td>
<td>0.58</td>
<td>1.72</td>
<td>0.84</td>
<td>1.91</td>
</tr>
<tr>
<td>3rd floor</td>
<td>0.43</td>
<td>1.41</td>
<td>0.52</td>
<td>1.57</td>
</tr>
<tr>
<td>2nd floor</td>
<td>0.21</td>
<td>0.94</td>
<td>0.23</td>
<td>1.04</td>
</tr>
<tr>
<td>1st floor</td>
<td>0.04</td>
<td>0.49</td>
<td>0.05</td>
<td>0.54</td>
</tr>
</tbody>
</table>

*Note: SE stands for SIWIS Element.
6.4.6 Design Checks for Concentrated Forces

The design checks presented for steel frames in Section 6.3.2 are applied for the four-story building example. The first story’s column in Axis A with W14X211 section subjected to compressive concentrated force of 100 kips is considered here.

Local Flange Bending:

$$\phi R_u = (0.9)(6.25t_f^2F_{yf}) = (0.9)(6.25(1.56)^2(50) = 685 \text{ kips}$$

Local Web Yielding:

Member depth = $d = 15.72$ in.
Location of applied load = 6 ft = 72 in. > $d$
Assume a length of bearing $N = 4$ in.

$$\phi R_u = (1.0)(5k + N)F_{yw}t_w = (1.0)(5(2.25) + 4)(50)(0.98) = 747 \text{ kips}$$

Web Crippling:

$$\phi R_u = (0.75)(135t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right] \frac{F_{yw}t_f}{t_w}$$

$$= (0.75)(135(0.98)^2\left[1 + 3\left(\frac{4}{15.72}\right)\left(\frac{0.98}{1.56}\right)^{1.5}\right] \frac{50(1.56)}{0.98} = 1200 \text{ kips}$$

Sidesway Web Buckling:

$$\frac{h}{t_w} = \frac{15.72 - 2 \times 2.25}{15 - 2.5} \times 12 \times 15.8 = 1.2 < 1.7 \text{ Thus, needs to be checked.}$$

$$\phi R_u = (0.85)\frac{C_t t_w^3 t_f}{h^2} \left[0.4\left(\frac{h}{t_w}\right)^3\right]$$

$$= (0.85)(480000)(0.98)^3(1.56)\left[0.4(1.2)^3\right] = 3290 \text{ kips}$$

Compression Buckling of the Web:

$$\phi R_u = (0.90)\frac{4100t_w^3}{h} \sqrt{F_{yw}} = (0.90)\frac{4100(0.98)^3 \sqrt{50}}{15.72 - 2 \times 2.25} = 2180 \text{ kips}$$
6.5 Summary and Concluding Remarks

Practical design approaches were developed for the SIWIS system including: design and prediction of the in-plane strength of masonry walls in SIWIS system through the developed empirical expressions; design the capacity arrangement for SIWIS elements in multi-bay multi-story frame buildings, and design checks for structural frames. The design approaches were then applied to three typical building examples including: a low-rise (four-story), a mid-rise (eight-story), and a high-rise (twenty-story) building in two locations including: Los Angeles, California (high seismic but low wind zone), and New York, New York (low seismic but high wind zone).

A design approach is developed according to the distribution of induced in-plane loads on different in-plane load resisting elements of the frame based on their relative stiffnesses. This design approach can specify appropriate capacity distribution for SIWIS elements both in elevation and in bays. Desirable failure sequences for SIWIS elements can be achieved to eliminate development of soft-story mechanisms. The proposed process can be computerized and included into the structural analysis and design programs.

In typical low-to-mid rise buildings in low seismic zones, the SIWIS elements usually do not fail. Thus, the designer may choose to use traditionally integrated masonry infill walls. However, it is always safe to use the “fuse-like” SIWIS elements to ensure safety during unforeseen wind and earthquake events. Alternatively, the designer can specify lighter masonry walls with an SIWIS system, which can lower the construction cost while maintaining the structural performance. In typical low rise buildings in high seismic zones, some or all of the SIWIS elements may fail. In typical mid-to-high rise buildings, in high seismic zones, usually all of SIWIS elements fail. High wind loads and minor-to-moderate earthquakes may cause failure of some of the SIWIS elements in typical mid-to-high rise buildings.

The in-plane flexural failure of a masonry wall in the SIWIS system can be, preferably, eliminated by providing adequate strength for the vertical tie-down element. The empirical expressions developed and graphically presented (for practical design purposes) to predict the in-plane shear strength of wall resulted in good agreement with the test results.

In the proposed SIWIS system, there is an interaction between the frame and the wall panel, which can affect the bending moment, axial, and shear forces of the frame. In order to check and ensure the frame for this interaction in analysis and design stages, it is proposed to substitute the presence of the SIWIS system by its reaction of the frame as a new loading case.
Chapter 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

Infill walls have displayed poor seismic performance in past earthquakes. Many buildings with infilled frames have suffered damage to their infill walls or confining structural frames. This dissertation has introduced and developed a third option, entitled Seismic Infill Wall Isolator Subframe (SIWIS) System, for infill walls, in which conventional designers have had two options: tight fit design, or completely isolated design. The new option introduced intends to involve the infill wall in in-plane load resistance of the structure by using its beneficial stiffness and strength effects under wind loads and in minor to moderate earthquakes in order to reduce induced story drifts. However, it will perform as a structural “fuse” to prevent any damage to the infill wall or the structural frame and minimize life-safety hazards during potentially damaging earthquakes. The designer will have control over the performance of the infill wall by designing the SIWIS elements with specifically desirable capacities.

To achieve the stated goals of this dissertation, to further understanding of the behavior, and finally to develop design approaches for the proposed system, a research program has been carried out. The study involves five main phases as subsequently summarized.

The first phase, described in Chapters One and Two, reviewed the common practice of masonry infill walls and studied their in-plane seismic performances and problems. It was discussed that neither of the two approaches implemented (isolated and tight fit infill walls) has been very successful and satisfactory so far. In the case of isolated infill walls, providing effective out-of-plane stability for the wall as well as acoustic and fire protection of the separation gaps can be very difficult. In the case of tight fit construction, masonry infill walls have been long known to affect stiffness, strength, and post-peak response of the infilled frame structures, which is not usually accounted for in analysis and design. Despite this fact, most building codes still do not have strict and clear provisions for seismic design and construction of infill walls. Modern seismic codes disregard or very briefly refer to the effects of infill walls. The consequence, of
course, is that the infill walls continue to be vulnerable to damage in earthquakes, as shown in Appendix A. Damage to these walls can result in significant economic loss and human casualties even if the structural frame is only lightly damaged. The most common damages include; cracking and total collapse of infill walls, soft story, short column, and column shear failure. Phase One of the study highlighted the essential need for a solution.

In the second phase of study, the SIWIS alternative was introduced in Chapter Three. The solution proposed consists of using subframes to be attached to the structural frame. The infill wall then is to be constructed within the subframe just like usual tight fit masonry wall construction. The subframe consists of two vertical members, which consist of two sandwiched light-gage steel plates with a “fuse” element between them, and one horizontal member. Within the subframe member at top of the wall, which will not have fuse elements, and in open spaces of the subframe vertical elements, there can be a flexible filler material to provide the needed sound insulation and fire-resistance. Through specific support to the side and top subframe members, the out-of-plane stability of the infill wall is provided. Several designs were conceptually proposed for an SIWIS element, which is the centerpiece in the proposed SIWIS system, including: (1) compression disk, (2) friction device, (3) tension element, and (4) jagging and releasing element. The compression disk design, which was the focus of this dissertation, consists of a pipe as the support, a disk as the failing part, a seat disk between pipe and failing disk, a rod for exerting force, and an end plate are proposed. The disk component is the failing component and needs to be replaced after its failure from the exertion of a certain compression force. For the failing disk, three different options are proposed including: (1) concrete disk, (2) steel disk and epoxy, and (3) lumber disk.

In the third phase of the study presented in Chapter Four, a numerical modeling for the analysis of the infilled steel frame equipped with SIWIS elements was created. Nonlinear finite element modeling schemes were developed for bare steel frame, infilled steel frame, and infilled steel frame with SIWIS elements. First, a single-bay single-story steel frame with a tightly fitted infill wall that has been the subject of past experimental and analytical studies was considered. Three different behaviors were considered for an SIWIS element including; “brittle-failure”, “flexible-failure”, and “friction-failure”. Once the modeling schemes for the single-bay single story frame were preliminarily validated by comparing numerical results with the available test results, finite element models for a typical two-bay three-story steel frame were similarly developed to study the use of the SIWIS system in multi-bay multi-story frames. Then, the two models were subjected to pushover and cyclic analyses under different conditions. To further
expand understanding about the system, a parametric study was conducted on these two models. Finally, a finite element model was developed for the scaled two-bay three-story test frame in order to compare the analytical results with the experimental observations, which led to the final and significant validation of the modeling schemes.

In the forth phase, presented in Chapter Five, an experimental testing program was completed to test the concept of an SIWIS system and observe the behavior of the proposed system. First, two preliminary tests including: a single wall test and a rod pulling test were conducted to provide necessary information for designing of the main tests. In the single wall test, two brick masonry wall panels were loaded to failure under a static in-plane pushover load. In the rod pulling test, three different rods including; 1/2”-13 threaded plastic rod, ¾ in. plain plastic rod, and 3/8”-16 threaded steel rod were subjected to tension tests. Second, the three options for compression disk design for SIWIS element were tested to study their performance and select the most suitable case for the main tests. After a preliminary test on four concrete disk specimens, a total of twenty four disk specimens with eight different thicknesses were tested. For the steel disk option, four compression tests in two cases, shear-case, and tension-case were conducted. First, for the lumber disk option, three lumber species including: poplar, hard maple, and red oak, were tested. The strength results led to the selection of hard maple as the better type. Subsequently, a total of twenty four hard maple lumber disk specimens in six different thicknesses were tested. The lumber disks’ failure mechanisms were a combination of rod punching through the center of the disk specimen and middle strip separation along the lumber grain directions followed by strip bending failure in the strong direction (parallel to grain direction). Due to the better performance and more consistent test results, the lumber disk was determined to be the more suitable case for the main tests. Third, the use of the SIWIS system in a scaled two-bay three-story steel frame was experimentally tested. For this purpose, initially, a testing platform including reinforced concrete foundation and steel reaction frame was designed and constructed. The test program included eleven static pushover tests in three main forms including: bare frame, infilled braced frame with SIWIS elements, and infilled pinned frame with SIWIS elements. Three different grades for SIWIS elements using hard maple disks with different thickness were used.

In the fifth phase of the study, as presented in Chapter Six, practical design approaches were developed for the SIWIS system. Design approaches include: design and predict the in-plane strength of masonry wall in SIWIS system through the developed empirical expressions; design the capacity arrangement for SIWIS elements in multi-bay multi-story frame building, and design checks for the confining structural frame. The design approaches were applied to three
typical building examples including; a low-rise (four-story), a mid-rise (eight-story), and a high-rise (twenty-story) building in two locations including: Los Angeles, CA (high seismic but low wind zone), and New York, NY (low seismic but high wind zone). The performance and advantages of the proposed system are evaluated for these three buildings.

7.2 Conclusions

The literature, analytical, and experimental studies reported in this dissertation lead to the following core and highlighted conclusions:

1. In the case of tight-fit construction, which is the common practice, masonry infill walls are very vulnerable to damage in strong earthquakes. The infill walls’ damage, which often result due to lack of clear and strict code provisions for analysis and design, lead to the continuation of significant economic loss and human casualties.

2. Completely isolated masonry infill walls have demonstrated better performance in strong earthquakes than tight-fit infill walls. However, they suffer from a lack of convenient and satisfactory solutions for gap fire protection, gap acoustic insulation, and wall out-of-plane stability, as well as not employing rather heavy infill wall in a more efficient way.

3. The proposed SIWIS system, as the third option, potentially improves the seismic performance of masonry infill walls. This “smart” system, initially, uses the beneficial effects of infill walls during minor-to-moderate earthquakes, but, ultimately isolates them from the structural frame during damaging earthquakes for their safety.

4. The developed nonlinear finite element modeling schemes predicts the test results and observations fairly well for both the case study selected from the literature (single-bay single-story frame, Richardson 1986) and the tests conducted on the two-bay three-story steel frame.

5. The nonlinear finite element modeling scheme created for infilled steel frames with an SIWIS system is a general modeling method and can implement different behaviors for their components, particularly SIWIS element, such as “brittle-failure”, “flexible-failure”, “friction-failure”, “multilinear-curve” and so on.
6. The analytical and experimental results confirm that the concept of the proposed SIWIS system works as a “seismic isolation” system for infilled frames to save the infill wall and the frame from any damage. The “fuse” type SIWIS element, first, utilizes the beneficial stiffness and strength effects of the infill wall up to a predefined point, after which it isolates them or limits the participation of infill walls in in-plane resistance.

7. The initial in-plane stiffness of a frame with SIWIS elements can be significantly higher than that of the bare frame. In other words, the frame with SIWIS elements deflects appreciably less than the bare frame before failure of the SIWIS element. The proportion depends on the design strength and stiffness of the SIWIS element and the stiffness properties of the infill wall.

8. For a frame with an SIWIS system, based on the number of stories and the number of SIWIS element failures, several phases can exist in the response. The in-plane stiffness of the frame with SIWIS elements decreases by progressive phases until it reaches the stiffness of the bare frame in the last phase after all SIWIS elements have failed. The response of the frame with the SIWIS system is significantly affected by the stiffness and strength properties of the SIWIS elements.

9. There are load drops in the in-plane load level after each failure point associated with breaking of the SIWIS elements of each story, which can be compared with the development of cracks in concrete or masonry structures. Such behavior during an earthquake is considered acceptable, particularly in light of the fact that it would come from a controlled failure aimed to prevent damage to the wall and frame. For commercial application of the SIWIS concept, there is further room for fine-tuning the design to minimize the adverse effects of sudden breakage of SIWIS elements.

10. The beneficial effects of the SIWIS system are considerably greater in more flexible frames (i.e., pinned and moment-resisting frames) as compared to stiffer frames (i.e., braced and shear wall frames).

11. The proposed design approaches can determine the appropriate capacity (grade) arrangement for SIWIS elements in elevation and in bays (with different geometry and conditions) to achieve a desirable sequence of failure of SIWIS elements and eliminate development of soft-story mechanisms. The proposed process can be easily computerized and included into the finite element programs used in structural analysis and design.
12. The in-plane flexural failure of a masonry wall in the SIWIS system can be, preferably, eliminated by providing adequate strength for the vertical tie-down element. The empirical expressions developed and graphically presented (for practical design purposes) to predict the in-plane shear strength of wall resulted in good agreement with the test results.

13. In typical low-to-mid rise buildings in low seismic zones, the SIWIS elements may not fail. Thus, the designer may decide to consider traditionally integrated masonry infill walls. However, it is always safe to use these “fuse-like” SIWIS elements to ensure safety during unforeseen wind and earthquake events. Furthermore, the designer can alternatively decide to use lighter masonry walls with an SIWIS system, which can lower the construction cost while maintaining the structural performance.

14. In typical low rise buildings in high seismic zones, some or all of the SIWIS elements may fail. For typical mid-to-high rise buildings in high seismic zones, all of the SIWIS elements will usually fail. High wind loads may cause the failure of some or all of the SIWIS elements in high rise buildings.

15. The tests conducted on compression disk design (concrete, steel, and lumber) verified the concept of a “fuse-like” SIWIS element. Desirable stiffness and strength properties can be obtained for the SIWIS element by different disk geometry and material properties. The proposed compression disk option using pipe, seat disk, failing disk, rod, and end plate is an efficient and promising design. This design provides a rigid link between frame and wall followed by their disengagement, after which adequate room is available for independent frame movements. Easy and practical maneuverability and accessibility are other advantages of this design. Other designs such as a friction device, tension element, and jagging and releasing element can be alternatively considered.

7.3 Recommendations for Future Work

The development of an SIWIS system is ongoing and additional research is needed in order to formulate a manufactured ready-to-implement building product. It is, however, believed
that this research provides a milestone in the effort towards development of this innovative system. The following key recommendations for future work are suggested:

1. The static pushover verification of an SIWIS system is analytically and experimentally obtained in this study as the first step. It is recommended that the dynamic behavior of the SIWIS system be studied by developing a dynamic time history finite element model or by upgrading the developed static finite element model. Performing dynamic tests such as high frequency loadings (use of actuators) or a shake table will be a considerable contribution. Moreover, cyclic tests can be conducted to further study the static response of the system.

2. There is extended room for further research on an SIWIS element, the centerpiece of the system, including the following:
   a. The studied compression disk design is an effective and efficient option. However, other options such as tension element, friction device, jagging and releasing element, and other possible alternatives need to be studied and compared to direct to the final product.
   b. The development and testing of an SIWIS element in this dissertation are aimed at testing the “fuse-like” concept to be used in the scaled test frame. For the actual product, higher capacities may be needed. To accomplish such capacities, it is suggested that studies use larger elements such as, the parallel combination of several elements, and other alternative designs.
   c. It was conceptually determined that hard maple lumber disk is the better option for the disk component among other disk options. This decision, which was based on limited tests, was made for the subsequent frame tests. It is appropriate to more comprehensively study the short and long term performances of lumber disk including effects of moisture content, drying, temperature, and so on.

3. The proposed SIWIS system can be used for new designs as well as for existing buildings in “seismic retrofit/upgrades”. A research program is encouraged to develop a retrofit procedure for masonry infilled frames with the use of the SIWIS system.

4. The details of out-of-plane stability of masonry walls in the SIWIS system need to be developed. The performance of the design should be experimentally demonstrated and verified through dynamic or static tests.
5. After completion and finalizing of the structural details of the system, the non-structural and architectural concerns need to be addressed such as the following:
a. The details of the SIWIS subframe and its connection to the structural frame need to be developed. The design of the subframe should take into account the architectural appearance of the system.
b. Research is needed to discover a suitable flexible filler material for open spaces of the subframe to provide effective and proper sound insulation and fire protection. Mono-Lasto-Meric, foamed polyurethane, and Declon-156 are some examples of a long lasting soft material, a sound isolator material, and a fire protection material, respectively. It should be mentioned that the subframe itself can notably facilitate in these regards, but may not be sufficient.
c. An aesthetic study of the SIWIS system in a building can be performed including the ability to integrate with windows, doors, and other architectural components, finishing materials, etc.

6. The design approaches of the SIWIS system developed in this research for a multi-bay multi-story building can be easily computerized and included the finite element programs used in structural analysis and design.

7. More literature, analytical, and experimental studies are encouraged to develop a generalized method for prediction of the strength of masonry walls with different conditions, such as, walls with openings, walls with different masonry units (i.e., concrete masonry blocks, clay bricks, thin wall hollow clay tiles, autoclaved aerated blocks, etc.), walls with different construction methods, and so on.

8. The main focus of this dissertation is on masonry infilled steel frames. It is believed that the findings of this research are applicable for masonry infilled reinforced concrete frames as well. However, analytical and perhaps experimental confirmations of this subject are desirable.

9. The proposed SIWIS system is classified as “seismic isolation”, a category of “passive control” of structures. With the use of the SIWIS idea, another interesting system that can be classified as “energy dissipation”, as passive control, is proposed. Energy dissipater devices such as metallic dampers, friction dampers, and visco or visco-elastic dampers can be used instead of the “fuse” SIWIS element. The results of a preliminary cyclic computer analysis that show this concept are included in Appendix B.
Appendix A

PHOTOS OF PAST EARTHQUAKE PERFORMANCES
OF MASONRY INFILL WALLS

A total of 49 photos in 34 figures mostly from recent earthquakes were selected to
demonstrate seismic performances of infill walls and infilled frames. The locations and dates of
earthquakes are listed in Table A.1 for each figure along with their sources. The permission to use
these photos has been obtained by author.

Table A.1: Photos of past earthquake performances of infill walls

<table>
<thead>
<tr>
<th>Figure</th>
<th>Earthquake Location</th>
<th>Earthquake Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 to A8</td>
<td>Arequipa, Peru</td>
<td>June 23, 2001</td>
<td>EERI ¹</td>
</tr>
<tr>
<td>A9 to A10</td>
<td>Seattle, WA, USA</td>
<td>February 28, 2001</td>
<td>Web ²</td>
</tr>
<tr>
<td>A11</td>
<td>Gujarat, India</td>
<td>January 26, 2001</td>
<td>EERI</td>
</tr>
<tr>
<td>A12 to A19</td>
<td>Izmit, Turkey</td>
<td>August 17, 1999</td>
<td>NISSE ³</td>
</tr>
<tr>
<td>A20 &amp; A21</td>
<td>El-Quindio, Colombia</td>
<td>January 25, 1999</td>
<td>EERI</td>
</tr>
<tr>
<td>A22</td>
<td>Adana-Ceyhan, Turkey</td>
<td>June 27, 1998</td>
<td>SGEB ⁴</td>
</tr>
<tr>
<td>A23</td>
<td>Adana-Ceyhan, Turkey</td>
<td>June 27, 1998</td>
<td>EERI</td>
</tr>
<tr>
<td>A24 &amp; A25</td>
<td>Nazca, Peru</td>
<td>November 12, 1996</td>
<td>EERI</td>
</tr>
<tr>
<td>A26</td>
<td>Northridge, CA, USA</td>
<td>January 17, 1994</td>
<td>NISSE</td>
</tr>
<tr>
<td>A27</td>
<td>Irpinia, Italy</td>
<td>1987</td>
<td>Web ⁵</td>
</tr>
<tr>
<td>A28 &amp; A29</td>
<td>Mexico City, Mexico</td>
<td>September 19, 1985</td>
<td>Web ⁵</td>
</tr>
<tr>
<td>A30</td>
<td>Mexico City, Mexico</td>
<td>September 19, 1985</td>
<td>NISSE</td>
</tr>
<tr>
<td>A31 to A34</td>
<td>Managua, Nicaragua</td>
<td>December 23, 1972</td>
<td>NISSE</td>
</tr>
</tbody>
</table>

Note:
¹ EERI: Earthquake Engineering Research Institute: (http://www.eeri.org/earthquakes/earthquakes.html)
² Web: (http://www.amre.com/content/home/news/nisqually_quake.htm)
³ NISSE: National Information Service for Earthquake Engineering, University of California, Berkeley: (http://nisee.berkeley.edu/images/servlet/EqiisListQuake)
⁴ SGEB: Swiss Society of Earthquake Engineering and Structural Dynamics (http://www.sgeb.ch/missionen/Adana.pdf)
⁵ Web: (www.gfz-potsdam.de/pb5/pb53/projekt/ems/eng/guide/illustrations/illustrations.htm)
Figure A.1: In this failed column, there is no special reinforcement in the short column area with wide spacing of transverse reinforcement.

Figure A.2: In this short column, even #3 @ 4 in. transverse rebars with did not prevent heavy damage.
Figure A.3: Both masonry infill wall and column were damaged in this school building.

Figure A.4: In this typical campus building, approximately 60 short columns damages were observed.
Figure A.5: This 3-story building in University Jorge Basadre campus experienced moderate damage to the beam-column corner joints due to formation of compression strut in the strong masonry infill panel.
Figure A.6: In this school, which was designed using the most recent Peruvian code, the masonry infill walls were separated from the confining reinforced concrete frames by an elastomeric material. This separation prevented damages to the infill wall as well as the formation of short columns mechanism.
Figures A.7: In this reinforced concrete building with light fit reinforced masonry infill walls, walls were heavily damaged.

Figure A.8: This municipality building suffered severe damage to masonry walls and to the short columns lacking enough transverse reinforcement.
Figure A.9: This reinforced concrete building suffered diagonal cracks in masonry infill panels.
Figure A.10: Typical diagonal crack was formed in an unreinforced masonry pier of a steel frame building.

Figure A.11: Left: A typical construction in Ahmedabad, which consists of reinforced concrete moment resisting frame structures with unsymmetrical reinforced brick or stone infill panels, collapsed. Middle: A 5-story reinforced concrete frame building with stone infill masonry walls suffered short column failures with insufficient transverse reinforcement. Right: A severe failure of corner column at the ground floor of a 3-story reinforced concrete frame building due to unsymmetrical stone infill masonry walls.
Figure A.12: These 5-story building (top) and 4-story building (bottom) under construction suffered nearly total collapse of their hollow clay tile masonry infill walls.
Figure A.13: This 13-story residential building with reinforced concrete structure suffered partial collapse of its pilaster and hollow clay tile infill walls.
Figure A.14: The first two stories of this building failed completely due to soft story mechanism while the upper three stories lightly damaged.
Figures A.15: In these two 6-story buildings, infill masonry walls of lower stories suffered heavy damages due to higher demands on the moment frame-infill wall system.

Figure A.16: Infill walls fell down in this food processing plant causing severe damages to steel penthouse.
Figure A.17: In the first story of this building, the front side was open, while the other faces were infill by masonry walls. This irregular placement of infill masonry walls caused the first-story columns to be heavily damaged and the whole building was nearly collapsed with no feasible repair design.
Figure A.18: After failure of two first stories masonry infill walls of this 6-story building, soft story mechanism was developed and the two first stories totally collapsed. The infill masonry walls and frames in the third and fourth stories (the first and second stories of the collapsed building in this Figure) suffered heavy damage.

Figure A.19: This 3-story reinforced concrete frame building experienced total collapse of its brick infill walls in the 2nd story and damage of the columns.
Figure A.20: Unreinforced masonry infill walls of this municipal building suffered major cracks and the reinforced concrete structure was moderately damaged.

Figure A.21: In this church building, unreinforced masonry infill walls and spires severely cracked and damaged while the spire's reinforced concrete columns remained standing.
Figure A.22: this 2-story carpet company with reinforced concrete frame and masonry infill walls suffered major damage in the first story. Reinforced concrete columns experienced a brittle shear failure (short column) instead of forming a more ductile frame mechanism with plastic hinges at beam-column connections (bottom) and in the fully infilled panel, the masonry wall cracked diagonally (top).
Figure A.23: In this hospital building, masonry infill walls cracked and separated from the frame.
Figure A.24: Infilled hollow brick walls suffered heavy damages in this 3-story school building with reinforced concrete frame.

Figure A.25: Development of short column mechanism in this 2-story school building with reinforced concrete structure and partial masonry infill walls was resulted.
Figure A.26: This residential building with reinforced concrete frame suffered major diagonal cracks in the masonry infill walls.

Figure A.27: In this reinforced concrete frame building, many exterior masonry infill walls were collapsed and some of the beam-column joints were also heavily damaged.
Figure A.28: This tower with reinforced concrete structure suffered heavy damages and cracks to the non-structural masonry infill walls. The repair of these damages, which can require evacuation of building and temporary shutdown of its normal operation, can be very expensive and process.
Figure A.29: In this building with reinforced concrete structure, masonry infill walls were damaged and heavily cracked. In such a situation, most of infill walls often need to be completely replaced with new walls within panels.
Figure A.30: In these two buildings, masonry infill walls were heavily damaged. The fall of heavy masonry infills can cause a serious life hazard.

Figure A.31: This church building suffered major damages to its unreinforced brick masonry walls.
Figure A.32: Partial masonry infill walls created a short column mechanism for these columns of these two 2-story reinforced concrete buildings.
Figure A.33: This 5-story reinforced concrete building experienced significant damages to the unreinforced masonry infill walls.
Figure A.34: The First National City Bank of New York tower with reinforced concrete structure suffered extensive damages to the unreinforced masonry infill walls.
Appendix B

CYCLIC MODELING AND ANALYSIS

B.1 Cyclic Modeling Using ANSYS

In this Appendix, ANSYS finite element models for both single-bay single-story and two-bay three-story case studies are subjected to cyclic loadings. Cyclic displacements were applied to models using the “File Load Set” option of the ANSYS solution method. It is not allowed to introduce negative slope at either force-deformation (or moment-rotation) or stress-strain relations by ANSYS in a cyclic analysis. Therefore, the negative parts of these relations were approximated by a zero slope in a cyclic analysis. This approximation will limit the cyclic response to be correct up to the stages where the slope of the load-deflection curve becomes negative as shown in Figure B.1. This approximation, however, will not affect the failure points of SIWIS elements and the phase behavior of the system, since these responses occur before the load-deflection relations reach to the negative slope region.

Figure B.1: Typical cyclic load-deflection relation and ANSYS approximation
B.2 Cyclic Analysis of Single-Bay Single-Story Case Study

The same ANSYS models developed for pushover loading (Chapter 4) were used for cyclic loading by modified moment-rotation and force-deformation relations as explained in Section B.1. The cyclic displacement patterns shown in Figures B.2 to B.4 were considered for the models of bare steel frame, infilled steel frame, and infilled steel frame with SIWIS elements, respectively. A total of 10 cycles with increments of 0.25 in. were considered for the bare steel frame. A total of 6 cycles with increments of 0.25 in. were used for infilled steel frame models. For cyclic loading of infilled steel frame with SIWIS element model, a total of 10 cycles with two different increments were considered. Small increments of 0.05 in. were considered for the first 2 cycles, while increments of 0.25 in. were used for the next 8 cycles. The first 2 small cycles were used to capture the response of the model before breaking of SIWIS element. The results of analyses are shown in Figures B.5 to B.8.

![Cyclic Loading Pattern Used for Bare Steel Frame Model](image)

Figure B.2: Cyclic displacement load pattern for single-bay single-story bare steel frame

Comparison of the cyclic responses and pushover responses in Figure B.8 shows that the pushover response well envelopes the cyclic response. The load and deflection values of the breaking point of the SIWIS element are the same for both cyclic and pushover responses. It is, of course, clear that in the cyclic response both SIWIS elements of the panel (left and right) failed, while in the monotonic response just the compression side (left) SIWIS element failed, and there
is no load on the tension side (right) SIWIS element. In other words, in the pushover response there is one failure point, whereas in the cyclic response there are two failure points. In order to be able to follow the cyclic response and behavior clearly, the responses associated with the 10 cycles of infilled steel frame with SIWIS elements are separately shown in Figure B.9.

Figure B.3: Cyclic displacement load pattern for single-bay single-story infilled frame

Figure B.4: Cyclic displacement load pattern for single-bay single-story infilled steel frame with SIWIS system
Figure B.5: Cyclic load-deflection relation for single-bay single-story bare steel frame

Figure B.6: Cyclic load-deflection relation for single-bay single-story infilled steel frame using single-diagonal strut model
Figure B.7: Cyclic load-deflection relation for single-bay single-story infilled steel frame using three-diagonal strut model

Figure B.8: Cyclic load-deflection relation for single-bay single-story infilled steel frame with “brittle-failure” SIWIS element
By studying the graphs shown in Figure B.9, one can see that the responses to the first and second cycles are before the failure of the SIWIS element and thus in the first phase of the response. In the third cycle, the left SIWIS element fails. Then in the reversed displacement, the right SIWIS element fails. Therefore, in the third cycle transition from the first phase to the second phase occurs. The responses of the rest of the cycles follow that of the bare frame and are parts of Phase 2. Observation reveals that for the first 8 cycles ($\delta = \pm 1.5$ in.), the response is linear and the curves pass the origin (zero load and deflection), while for the next two cycles, response enters the nonlinear range and, therefore, the curves do not pass the origin.

Figure B.9: Cyclic load-deflection relation for single-bay single-story infilled steel frame with SIWIS system for 10 cycles
B.3 Cyclic Analysis of Two-Bay Three-Story Case Study

The same types of modifications that were applied for single-bay single-story pushover models to develop the cyclic models were considered for the two-bay three-story models as well. For example, the force-deformation responses used for cyclic analysis of infilled steel frame using three-diagonal strut method are shown in Figure B.10.

The applied cyclic displacements are shown in Figures B.11 to B.13. A total of 10 cycles with increments of 1.00 in. were considered for bare steel frame. For the two models of infilled steel frame, a total of 12 cycles with 0.25 in. increments were used. For infilled steel frame with SIWIS elements, a total number of 15 cycles with 0.25 in. increments for the first 6 cycles and 0.5 in. increments for the next 9 cycles were considered as shown in Figure B.13. Small increments were used for the first 6 cycles to be able to capture the breaking points of the SIWIS elements. For all cases, the horizontal displacement was applied at the 3rd floor level. The resultant cyclic load-deflection relations are illustrated in the Figures B.14 to B.16.

Figure B.10: Force-deformation response for: (a) upper and lower struts and (b) middle strut for cyclic analysis of infilled steel frame using three-diagonal strut method

Comparing the cyclic and monotonic responses from Figure B.17, shows that the pushover response well envelopes the cyclic response. The load and deflection values of the breaking points of the SIWIS elements are the same for both cyclic and pushover responses. In the cyclic analysis, all 12 SIWIS elements (2 in each panel) reached failure, while in the pushover analysis only 6 compression side (left) SIWS elements failed. In other words, the pushover response creates three SIWIS element failure points, while the cyclic response creates six failure points. The sequence of the failure of the SIWIS elements in elevation is the same for two loading
Similar to the single-bay single-story case, in order to be able to clearly investigate and follow the cyclic response and observe the sequence of failure of SIWIS elements, the responses associated with the 15 cycles are separately shown in Figure B.18.

Figure B.11: Cyclic displacement load pattern for two-bay three-story bare steel frame

Figure B.12: Cyclic displacement load pattern for two-bay three-story infilled steel frame
Figure B.13: Cyclic displacement load pattern for two-bay three-story infilled steel frame with SIWIS system

Figure B.14: Cyclic load-deflection relation for two-bay three-story bare steel frame
Figure B.15: Cyclic load-deflection relation for two-bay three-story infilled steel frame using single-diagonal strut model

Figure B.16: Cyclic load-deflection relation for two-bay three-story infilled steel frame using three-diagonal strut model
Referring to Figure B.18, one can investigate the sequence of the failure of the SIWIS elements. The response to the first cycle is before the failure of any SIWIS element and thus is part of first phase of the response. In the second cycle, the 2 left SIWIS elements of the 3rd story’s panels fail. Then, in the reversed displacement of the second cycle, the 2 right SIWIS elements of the 3rd story’s panels fail. Therefore, in the second cycle transition from the first phase to the second phase occurs. The third cycle’s response is all in the second phase. In Cycle 4, failure of the SIWIS elements of the 2nd story’s panels occurs. First, the 2 left elements and then the 2 right elements fail. Cycle 4 is the transition between Phase 2 and Phase 3. Cycle 5 is part of Phase 3. The failure of the 1st story’s SIWIS elements happens in Cycle 6 by failure of 2 left side elements followed by the failure of 2 right side elements. In this cycle, the response is transferred from its third phase to the fourth phase. The responses in the rest of the cycles follow that of the bare frame and are parts of Phase 4. The response is almost linear up to Cycle 11 ($\delta = \pm 2.5$ in.), after which it enters the nonlinear region. The stiffness of the frame system (slope of the load-deflection curve) is almost the same for the pushing and pulling (positive and negative displacements) responses for all cycles.
Figure B.18: Cyclic load-deflection relations for two-bay three-story infilled steel frame with SIWIS system for 15 cycles
B.4 Cyclic Analysis of Single-Bay Single-Story Case Study Using Friction Damper (FD) SIWIS Element

A preliminary cyclic analysis of the single-bay single-story case study with 20-kip capacity compression only “friction-damper” SIWIS elements is presented in this appendix. The cyclic displacement pattern shown in Figure B.19 with a total of 10 cycles is applied to the developed finite element model. Lateral displacements of 0.05, 0.10, 0.25, 0.50, 0.75, 1.00, 1.25, 1.50, 1.75, and 2.00 in. are considered for 10 cycles, respectively. Positive direction is from left side of the frame to the right side.

![Figure B.19: Applied cyclic displacement history](image)

The load-deflection relation is shown in Figure B.20. For clarity purposes, the response of Cycle 8 is separated and shown in Figure B.21. For the same purpose, the responses of other cycles are also shown separately in Figure B.22. The hysteretic loops of FDs’ forces are shown in Figure B.23 and B.24 for left FD and right FD, respectively. The responding load history is shown in Figure B.25. Finally, the histories of forces of FDs are shown in Figure B.26. With reference to these figures, the general response and behavior of the system is explained next.
During the first cycle, the response of the structure is linear and the maximum load is 16.1 kips. Since this load level is lower than the friction capacity of the FD element, there is no sliding in the FD and the wall is rigidly linked to the frame. In the next cycles, the FD force reaches 20 kips, and thus, it slides.

General response to a single cycle is representatively shown for Cycle 8 in Figure B.20. During the pushing part of the cycle (Path ADF), when positive displacement is applied (from left to right), only the left side FD is active and during the pulling part (Path A’D’F’), only the right side FD is active. This is due to the fact that the connections between the wall panel and the FDs are designed to transfer only compression load and not tension load. This behavior can be also observed in the hysteretic loops of FDs’ forces in Figures B.23 and B.24, where the left side FD’s loop is located in the first quarter and the right side FD’s loop is located in the second quarter in a Cartesian system. A full tension-compression FD will have its loop in all four quarters.

When the FD is compressed between the frame and the wall (Paths BCD and B’C’D’), it acts as a semi-rigid link and transfers force from the frame to the wall. During this phase, when the force reaches the FD’s capacity of 20 kips, the FD starts sliding (Points B and B’). During the slide of the FD (Paths CD and C’D’), it maintains transfer of a constant load of 20 kips from frame to wall. Upon the reversal of direction of load, the FD stops sliding (Points D and D’) and thus, the wall panel is disengaged from the frame. By progressive reverse side displacement load (Paths DEF and D’E’F’), a gap is developed between the FD and the wall panel. During the following cycle due to existence of this gap, there is no instant connection between the FD and the wall panel and thus, initially, there is no force in the FD (Paths AB and A’B’). During this part of the response, the bare frame is the only resisting element. By progressive displacement load and upon overcoming the pre-developed gap (in previous cycle), the FD is re-activated and transfers the load from the frame to the wall (Paths BCD and B’C’D’). This re-activation, which results in an increase in lateral stiffness of whole system upon the closing of gap (Points B and B’), is the source of the so-called “pinching effect” as representatively shown in Figure B.24 for Cycle 9. The active periods of FDs and the amount of their slides can be observed in Figure B.26. A similar type of response is repeated for other cycles as well.
Figure B.20: Cyclic load-deflection relation for single-bay single-story frame

Figure B.21: Response of the frame to Cycle 8
Figure B.22: Separated load-deflection relation for each cycle
The input energy to the system and energy dissipated by the structure for each cycle is shown in charts in Figure B.27. The dissipation of energy has two sources including friction dampers, which dissipate energy in the form of friction force, and steel frame, which dissipates energy in the form of nonlinear (plastic) deformations (i.e., plastic hinges). Distribution of this dissipated energy among these two sources is shown in Figure B.28. The input energy, total dissipated energy, and dissipated energy by an FD are measured and representatively shown in Figure B.22 for Cycle 8, 10, and 6, respectively. The steel frame contributes to the energy dissipation capacity only after Cycle 8. Friction dampers add to the energy dissipation capacity of
the structure from Cycle 2, when their slides commence. Charts in Figure B.28 show that for this particular cyclic loading history, FDs and frame have dissipated nearly equal amounts of energy. The conclusions, therefore, is that the two friction dampers have increased 100% (or have doubled) the energy dissipation capacity of the structure in this example.

Figure B.25: Frame in-plane load response history

Figure B.26: Friction Dampers force histories
Figure B.27: Input energy and dissipated energy

Figure B.28: Dissipated energy by frame and FDs
Appendix C

TEST PLATFORM:
STRUCTURAL DRAWINGS AND PHOTOS

C.1 Structural Drawings

Structural design drawings of test platform including reinforced concrete foundation and steel reaction frame are included in Appendix C.1.

C.2 Photos

A total of 14 photos including; 10 photos of construction and 4 photos of completed test platform (reaction frame) were selected and included in Appendix C.2.
TEST PLATFORM

STRUCTURAL DRAWINGS

THE PENNSYLVANIA STATE UNIVERSITY
ARCHITECTURAL ENGINEERING DEPARTMENT

DESIGN & DRAWINGS:
MOHAMMAD ALIAARI

FEBRUARY 2003
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MATERIALS:

CONCRETE:
MIN. COMPRESSIVE STRENGTH = 6000 psi

STEEL REINFORCEMENT:
LATERAL:
STEEL GRADE 60
(MIN. YIELD STRENGTH = 60000 psi)

STIRRUPS:
STEEL GRADE 40
(MIN. YIELD STRENGTH = 40000 psi)

MIN. 7.0 in. FOR #11
TEST FRAME’S COLUMN TO COLUMN CONNECTION DETAIL

W14X211—W14X211 CONNECTION

VIEW "A"

SCALE 5/8" = 1’
SECTION A-A FROM S-10

SCALE 5/8" = 1'

END PLATE 72X15.8X1 3/4"

SCALE 5/8" = 1'
W14X211–W10X88 CONNECTION

VIEW "A"

SECTION A–A

END PLATE

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W14X211–W10X88–W12X190 CONNECTION

SCALE 5/8" = 1'

TEST FRAME’S
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SHEET: 13 OF 15
DATE: FEB. 2003
W14X211—BASE CONNECTION
SCALE 5/8" = 1'

SECTION A - A
SCALE 5/8" = 1'

Testing Frame’s Column Base Plate Connection Detail

Drawing No.: S-14
Unit: Inch
Scale: 5/8" = 1'
Sheet: 14 of 15
Date: Feb. 2003
W12X190 TO W10X88 CONNECTION

W12X190 TO CYLINDER CONNECTION

4-1\(\frac{1}{4}\)" BOLTS
A325-N BOLTS

STIFFENER 1\(\frac{1}{2}\)" PLATE
(REF. 5-13)

HOLES FOR 1\(\frac{1}{4}\)" BOLTS
SIXTEEN HEAD OR SLEEVE

Sheet Title: TEST FRAME'S LOADING JACK
CONNECTION DETAILS

Drawing No.: S-15
Unit: INCH
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Date: FEB. 2003
Figure C.1: Stirrups reinforcements of foundation

Figure C.2: Installation of longitudinal and stirrups reinforcements
Figure C.3: Foundation anchorage bolt

Figure C.4: Completion of foundation reinforcement installation
Figure C.5: Placement of foundation anchorage bolts

Figure C.6: Details of foundation reinforcement and anchorage bolts
Figure C.7: Inspection of foundation reinforcement (Author)

Figure C.8: Placement of base plate anchorage bolt by template
Figure C.9: Formwork of concrete foundation

Figure C.10: Curing fresh concrete by wet bags
Figure C.11: A view of steel reaction frame

Figure C.12: Details of column-to-base plate connection
Figure C.13: Details of column-to-loading member connection

Figure C.14: Details of column-to-inclined member connection
Appendix D

MASSONRY PRISM TEST

Three 5-course stack bond masonry prisms were constructed by the same masons and at the same time as the brick wall specimens. The actual dimensions of these three prisms are listed in Table D.1. The prism test was conducted on these three prisms according to the standard test for compressive strength of masonry prisms, ASTM C1314-02. The main objective of this test was to calculate the specified compressive strength of masonry walls’ material.

The typical masonry prism compression test setup is shown in Figure D.1. A MTS loading machine with a 100-kip capacity load cell was used for the prism test. Two thick hardened metal blocks were used in top and bottom for bearing purposes. Both top and bottom plates had a larger area than the prism and, thus covered the whole prism bearing areas and the upper platen was spherically seated as recommended by ASTM C1314-02. Each prism was placed on the lower platen with both centroidal axes of the prism aligned with the center of thrust of the testing machine. Then the spherically seated upper platen was brought to bear on the prism and the movable part of the upper platen was rotated to obtain a uniform seating. A compression loading rate of 0.001 in./sec was applied. The axial load and displacement histories were recorded using the internal load cell and LVDT of the MTS loading machine.

The failure modes of the three prisms are shown in Figures D.2 to D.4, respectively. As can be seen from these figures, the type of failure was a combination of cone splitting and face-shell separation. In all prisms, the failure occurred on both sides and in nearly the whole height of the prisms. The maximum compression loads are listed in Table D.2. The masonry compressive strength of each prism is calculated by dividing each prism’s maximum load sustained by the cross-sectional area of that prism and included in Table D.2. The correction factor for each prism is taken from Table 1 of ASTM C1314-02 based on the height to thickness ratio of that prism and the compressive strength of the masonry for each prism is specified as listed in Table D.2. Specified compressive strengths of 3950, 2840, and 3960 psi resulted from the three prisms, respectively. By averaging these three strengths, a specified compressive strength of 3600 psi is considered for masonry material of the wall panels. The compression
load-displacement relations are shown in Figure D.5 for the three masonry prisms. The test took 134, 139, and 156 seconds for the three masonry prisms, respectively.

Table D.1: Masonry prisms dimension properties

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Figure D.1: Masonry prism test setup on MTS load frame

Table D.2: Masonry prism test result

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<th>Correction Factor</th>
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Figure D.2: Failure mode of Prism-1

Figure D.3: Failure mode of Prism-2
Figure D.4: Failure mode of Prism-3

Figure D.5: Compression load-deformation relations for the three masonry prisms
E.1 Introduction

An experimental test was planned and conducted to mainly test the concept of a compression concrete disk as a failing component in an SIWIS element. An SIWIS element including three separate parts; a steel rod for exerting force, a concrete disk intended to fail under certain load and an open cylindrical polyvinylchloride (PVC) piece as the support was proposed. The steel rod’s tip is positioned in the notch on the concrete disk. The concrete disk is positioned on a cap on the PVC support, which has a thicker wall on the seat for the concrete disk.

E.2 Experimental Test

Four concrete disk specimens were molded using Quickrete concrete mix, which is a ready-to-mix material included gravel, cement, and sand. These four disk specimens with four different notch and total thicknesses are shown in Figure E.2. The external diameter of all disks was 2¾ in. on the bottom flat side. The disk specimens’ thickness properties including total, notch, and effective thicknesses and weights are listed in Table E.1. They were 14-days old at time of testing. For the support part, a PVC piece shown in Figure E.2 was chosen. This piece was an open cylinder with a notch at the top with a thicker lower part as support for the concrete disk. The height of the piece was 2 in., outside and inside diameters of 3.50 in. and 2.375 in., respectively. A plain steel rod shown in Figure E.2, with 3.94 in. length, and 1 in. diameter was used. The main equipment used for performing this experiment was a small hydraulic apparatus (Figure E.3) that can exert tension or compression forces up to 7850 lbs. A hydraulic cylinder was used for compression test (pushing up). A hand pump supplied the hydraulic pressure. For the compression test, the piston rod was extended and the pressure hose was attached to the bottom of the cylinder. The pressure was measured by a pressure gage with a dial graduated in pounds per square inch (psi). The equivalent load (in pounds) was read directly from the appropriate scale of
the pressure gage dial. The maximum pressure was 10,000 psi (equivalent to 7850 pounds force) on the gage. The compression arrangement of the apparatus used for this experiment is shown in Figure E.3.

Table E.1: Concrete disk specimens properties and capacity results

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Weight (lb)</th>
<th>Thickness (in.)</th>
<th>Capacity (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Total</td>
<td>Notch</td>
</tr>
<tr>
<td>A</td>
<td>0.44</td>
<td>0.875</td>
<td>0.313</td>
</tr>
<tr>
<td>B</td>
<td>0.50</td>
<td>1.000</td>
<td>0.250</td>
</tr>
<tr>
<td>C</td>
<td>0.56</td>
<td>1.125</td>
<td>0.313</td>
</tr>
<tr>
<td>D</td>
<td>0.58</td>
<td>1.125</td>
<td>0.375</td>
</tr>
</tbody>
</table>

Figure E.2: Concrete disk specimens, PVC support, and steel rod

Figure E.3: Hydraulic apparatus and compression test setup
E.3 Results and Discussion

Disk specimens A, B, C, and D failed at compression loads of 1700, 2300, 2800, and 2900 lbs, respectively. Figure E.4 plots the capacity of disk specimens versus their thicknesses. Although the capacity of concrete disks increase with thickness, it is obvious from Figure E.4 that the capacity is related to both effective thickness and total thickness.

In all tests, the steel rod penetrated the concrete disk. Figures E.5 to E.7 show multiple photos of the failed concrete disks. The failure modes of tested specimens showed two basic failures as schematically illustrated in Figure E.8. The failure modes resulted from a combination of two kinds of cracks and fractures. One failure mode was a circular sloped fracture starting from the edges of the bottom of notch on the top of disk, where the steel rod pushed, and ending at the edges of the PVC support on the bottom of the disk. This fracture mode let the steel rod to go through, like punching. The resistance of this fracture is related only to the effective thickness of the disk. The second failure mode was the fracture of the whole disk into 4 or 5 main parts. The resistance of this fracture mode is related to whole thickness of disk and not just to the effective one.

![Concrete Disk Capacity - Thickness Graph](image)

Figure E.4: Concrete disk capacity versus thickness
Figure E.5: Failed concrete Disk A (left) and Disk B (middle and right)

Figure E.6: Failed concrete Disk C

Figure E.7: Failed concrete Disk D (left: top side, right: bottom side)
E.4 Conclusion

Based on the results of this test, the following conclusions can be made:

1. The whole system worked well as a fuse element for the SIWIS element, and therefore, the proposed concept was verified.
2. The four concrete disk specimens showed similar behavior and failure modes. Failure mode was a combination of punching through and fracture of the whole disk into 4 or 5 main parts.
3. The four disk specimens failed in a very brittle manner at load levels of 1700, 2300, 2800, and 2900 lbs, respectively. The capacity of the concrete disk is related to both total thickness and effective thickness.
4. The PVC piece performed well as a support piece in this test. No damage or deficiencies were seen during and after the test.
Appendix F

SIWIS ELEMENT PRELIMINARY TEST
USING LUMBER DISK

F.1 Introduction

The use of a lumber disk is one proposed design for the fuse component in the compression disk design for an SIWIS element. A preliminary experimental program was designed and carried out to test the concept of using a lumber disk as explained in this appendix.

F.2 Experimental Program

Three different lumber types including poplar, red oak, and hard white maple were used for this experiment. Hard white maple has a cream to light-reddish-brown colored heartwood, with thin white sapwood tinged slightly with reddish brown. The wood is heavy, strong, and stiff and has high resistance to shock. For poplar, the sapwood is creamy white and may be streaked, with the heartwood varying from pale yellowish brown to olive green. The green color in the heartwood will tend to darken on exposure to light and turn brown. The wood has a medium to fine texture and is straight-grained. Poplar has a comparatively uniform texture. For red oak, the heartwood resembles other oaks with a biscuit to pink color, but has a reddish brown tinge. It is mostly straight grained and coarse textured, with a less attractive figure than white oak due to smaller rays.

Four disk specimens of poplar, five disk specimens of red oak, and three disk specimens of hard maple were used resulting in a total of 12 tests. All disks for each lumber type were cut from the same lumber piece using a 4-1/8 in. diameter hole saw as shown in Figure F.1, which resulted in an average diameter of 3.93 in. for all lumber disks. The poplar and hard maple disk specimens had a thickness of 1-3/8 in. while the Red Oak disk specimens were 1-1/8 in. thick. The 12 disk specimens are shown in Figure F.1. The weight of each disk specimen was measured...
and listed in Table F.1. The density of each disk specimen is also calculated and included in Table F.1. As can be seen from this table, the poplar and hard maple disk specimens have very consistent densities with coefficient of variations less than 1%. It is higher and about 11% for red oak disk specimens. This is due to existence of a denser wood structure in some areas of the lumber plate, from which these disks were cut. For example, it can be seen from Figure F.1 that one of the red oak disk specimens (the 2\textsuperscript{nd} from top) had a denser (seen darker in photo) part relative to the rest of the disks. The same steel pipe support, steel seat disk, steel end plate, and steel threaded rod that were used for concrete disk test were used in this experiment.

The main equipment used in this experiment included: a testing load frame and a data acquisition system. The same equipment used for concrete disk test were employed in this experiment as well. Two electrical resistance strain gages were attached to two opposite sides of the steel rod in its middle height in order to record its longitudinal strains. Locations of these gages were locally sanded to create flat surfaces. However, these gages were damaged after performing 4 tests. Therefore, strain data are available for only 4 disk specimens. Incrementally increasing load in the form of displacement with rate of 0.002 in./sec. was used in this test. An MTS 458.91 microprofiler was used to control loading history.

Table F.1: Thickness and density properties of lumber disk specimens

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Thickness (in.)</th>
<th>Weight (lb)</th>
<th>Density (lb/in.(^3))</th>
<th>Average (lb/in.(^3))</th>
<th>Standard Deviation (lb/in.(^3))</th>
<th>Coefficient of Variation (CV%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poplar #1</td>
<td>1-3/8</td>
<td>0.3230</td>
<td>0.0194</td>
<td>0.0193</td>
<td>0.0001</td>
<td>0.6</td>
</tr>
<tr>
<td>Poplar #2</td>
<td>1-3/8</td>
<td>0.3219</td>
<td>0.0193</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Poplar #3</td>
<td>1-3/8</td>
<td>0.3194</td>
<td>0.0192</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poplar #4</td>
<td>1-3/8</td>
<td>0.3239</td>
<td>0.0194</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Red Oak #1</td>
<td>1-1/8</td>
<td>0.3757</td>
<td>0.0275</td>
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<td></td>
<td></td>
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<tr>
<td>Red Oak #2</td>
<td>1-1/8</td>
<td>0.2963</td>
<td>0.0217</td>
<td>0.0243</td>
<td>0.0027</td>
<td>11.3</td>
</tr>
<tr>
<td>Red Oak #3</td>
<td>1-1/8</td>
<td>0.3102</td>
<td>0.0227</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Red Oak #4</td>
<td>1-1/8</td>
<td>0.2837</td>
<td>0.0208</td>
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<tr>
<td>Red Oak #5</td>
<td>1-1/8</td>
<td>0.3333</td>
<td>0.0244</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Maple #1</td>
<td>1-3/8</td>
<td>0.4378</td>
<td>0.0263</td>
<td>0.0262</td>
<td>0.0001</td>
<td>0.4</td>
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<tr>
<td>Maple #2</td>
<td>1-3/8</td>
<td>0.4385</td>
<td>0.0263</td>
<td></td>
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<td></td>
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<tr>
<td>Maple #3</td>
<td>1-3/8</td>
<td>0.4352</td>
<td>0.0261</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure F.1: Lumber disk specimens (left: hard maple, middle: red oak, right: poplar) and a typical 4-1/8 in. hole saw

F.3 Results and Discussion

F.3.1 Test Observations and Failure Modes

Twelve tests were conducted on the twelve lumber disk specimens. Figures F.2, F.3 and F.4 show the progressive cracks and failure modes of Poplar-1, Red Oak-8, and Maple-11 disk specimens, respectively. All disk specimens showed relatively similar behavior. The overall behaviors and failure modes were similar to those explained in the expanded test (Section 5.6.4). The failure mechanism was a combination of rod punching and middle strip separation along the grain direction followed by bending failure of the strip in its major direction.

F.3.2 The 1st Major Crack Loads and Capacities

The disk specimens’ resulting ultimate loads (capacities) are summarized in Table F.2 along with their 1st major crack loads. The capacities and the 1st major crack loads are also shown in charts in Figures F.5 and F.6 respectively. The hard maple disk specimens had the highest capacity. The poplar, red oak, and hard maple disk specimens resulted in an average capacity of 3980, 4490, and 8850 lbs, respectively. The poplar and hard maple disk specimens had relatively
consistent capacities with coefficient of variations of 5.5% and 6.6%. The red oak disk specimens resulted in a coefficient of variation of 16% for capacities of disks. As can be seen the Red Oak-1 disk specimen, which was denser than other red oak disks, had capacity of 5390 lbs that is 20% more than the average capacity for red oak disks.

Figure F.2: Failure mode of Poplar #1 disk specimen

Figure F.3: Failure mode of Red Oak # 8 disk specimen
Table F.2: Strength properties of lumber disk specimens

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>The 1st Major Crack (lb)</th>
<th>Average (lb)</th>
<th>SSD (lb)</th>
<th>CV (%)</th>
<th>Strength (lb)</th>
<th>Average (lb)</th>
<th>SSD (lb)</th>
<th>CV (%)</th>
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<tbody>
<tr>
<td>Poplar #1</td>
<td>4015</td>
<td>3940</td>
<td>293</td>
<td>7.4</td>
<td>4015</td>
<td>3980</td>
<td>219</td>
<td>5.5</td>
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<td>Poplar #2</td>
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<td></td>
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<td>Poplar #3</td>
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<td>3670</td>
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<td></td>
</tr>
<tr>
<td>Poplar #4</td>
<td>4060</td>
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<td></td>
<td></td>
<td>4060</td>
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<td></td>
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<tr>
<td>Red Oak #1</td>
<td>2765</td>
<td>3280</td>
<td>539</td>
<td>16.4</td>
<td>5390</td>
<td>4490</td>
<td>716</td>
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<td>4070</td>
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<td></td>
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<tr>
<td>Red Oak #3</td>
<td>4130</td>
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<td></td>
<td></td>
<td>4425</td>
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<td>Maple #1</td>
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<td>8020</td>
<td>1239</td>
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<td>9000</td>
<td>8850</td>
<td>584</td>
<td>6.6</td>
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<td>Maple #2</td>
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<td>9350</td>
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<tr>
<td>Maple #3</td>
<td>7840</td>
<td></td>
<td></td>
<td></td>
<td>8210</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
F.3.3 Load-Deflection Relations

The load-deflection relations for the twelve tests are plotted in Figure F.7. As the behavior of disk specimens was explained in the previous section, the occurrence of the 1st major crack and ultimate stages can be seen in the load-deflection curves. For the poplar disk
specimens, the ultimate stage occurred at 0.41-0.48 in. deflection with an average of 0.45 in. and 3670-4180 lbs load with an average of 3980 lbs. For Poplar disk specimens except Poplar-3, the 1st major crack was the same as the ultimate stage. For Poplar-3, the ultimate stage happened slightly after the 1st major crack. The red oak disk specimens exhibited stiffer behavior than Poplar disk specimens. The deflection and load values of the 1st major crack were 0.04-0.18 in. with an average of 0.11 in. and 2765-4130 lbs with an average of 3280 lbs, respectively. The ultimate stage occurred at about 0.19-0.30 in. deflection with an average of 0.25 in. and 3575-5390 lbs load with an average of 4490 lbs. The hard maple disk specimens had the highest ultimate loads. For the maple disk specimens, the 1st major crack took place at 0.21-0.34 in. deflection with an average of 0.29 in. and at load level of 6875-9335 lbs with an average of 8015 lbs. The ultimate stage occurred at the deflections of 0.49-0.51 in. with 0.50 in. average and at the load of 8210-9350 lbs with an average of 8850 lbs.

![Load - Deflection Relation](image)

Figure F.7: Load-deflection relation for 12 disk specimens

**F.3.4 Strength-Density Relations**

Although all disk specimens of each lumber type were cut from the same piece of lumber plate, they had slightly different densities. This is basically due to the fact that the lumber does not have an ideal homogenous structure. The denser lumbers often result in higher capacity and
stiffness. In order to observe the effects of density, the lumber disks’ strengths (capacities) versus densities are plotted for the three types of lumbers in Figures F.8 to F.10. In each graph, a linear relation is examined for strength-density curves from which the red oak and maple disk specimens’ results show fairly good fits. If Poplar-3 is ignored, the poplar disk specimens also result in a good linear strength-density relation. For comparison purposes, the average strength and density values for the three lumber types are shown in charts in Figure F.11. As can be seen, red oak had the highest density and maple had higher density than poplar. However, Maple had the highest strength and Poplar had the lowest one. It should be mentioned that the red oak disk specimens had lower thickness of 1-1/8 in. compared to maple and poplar disk specimens, which had thicknesses of 1-3/8 in. Thus, it can be that larger differences exist between red oak and poplar disk specimens’ capacities, if the same thicknesses were used for both.

![Figure F.8: Strength of poplar disk specimens versus density](image)

![Figure F.9: Strength of red oak disk specimens versus density](image)
F.3.5 Rod Strain-Load Relations

Two strain gages were used to record the longitudinal strains along the steel rod. The strain gages were lost after performing 4 tests. The axial rod strain versus axial load is, representatively, illustrated in Figure F.12 for Red Oak-1. As can be seen from this figure, the two strain gages recorded different strain values during the test. This indicates that there was a relatively heavy bending moment or distortion in the steel rod.
Similar to the procedure explained in Section 5.4.5.5 for the concrete disk test, the average rod strains are calculated and included in Figure F.12. As expected, it can be seen from these figures that average rod strain is linear for all cases. The slope of $\varepsilon - P$ is calculated and listed in Table F.3 along with the theoretical $\varepsilon - P$ slopes, which were calculated using Equation (5.6). An average of $0.0534\times10^{-6}$ 1/lb was calculated for the slope of $\varepsilon - P$ curves. This can be compared with the average slope of $0.055\times10^{-6}$ 1/lb that was the result in Section 5.4.5.5 for the concrete disk test.

Table F.3: Slope of $\varepsilon - P$ curves for steel rod

<table>
<thead>
<tr>
<th>Disk Specimen</th>
<th>Test $\varepsilon - P$ slope (1/lb x10^{-6})</th>
<th>Test Average (1/lb x10^{-6})</th>
<th>Standard Deviation (1/lb x10^{-6})</th>
<th>Coefficient of Variation</th>
<th>Theoretical $\varepsilon - P$ slope (1/lb x10^{-6})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poplar # 1</td>
<td>0.0536</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Red Oak # 1</td>
<td>0.0540</td>
<td>0.0534</td>
<td>0.00064</td>
<td>1.2</td>
<td>0.055</td>
</tr>
<tr>
<td>Red Oak # 4</td>
<td>0.0535</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maple # 2</td>
<td>0.0525</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

Figure F.12: Rod strain-load relation for Red Oak #1
F.3.6 Selection of Lumber Type

The important criteria that need to be considered for the selection of the best lumber type among the three test lumber types include; high capacity, high capacity to density ratio, and consistency of test results. Based on these criteria, maple is determined to be the best case.

F.4 Conclusions

The following conclusions are highlighted based on the results of this preliminary test:

1. The idea of using lumber disk as the failing component (fuse) in an SIWIS element is promising.
2. The lumber disks’ failure mechanism were a combination of rod punching through the center of disk specimen and middle strip separation along lumber grain directions followed by strip bending failure in its strong direction (parallel to grain direction).
3. The disk specimens of each lumber type demonstrated a relatively similar behavior, failure modes, and load-deflection relations.
4. There was a relatively good linear relation between strength and density of disk specimens for each lumber type especially for red oak and maple disk specimens.
5. Maple disk specimens had the highest strength of 8850 lbs and polar disk specimens had the lowest strength of 3980 lbs. red oak disk specimens were in between with an average of 4490 lbs.
6. The steel rod’s demonstrated linear strain-load curve agrees well with the theoretical results.
7. Maple lumber was determined to be the best option among the three lumber types for application in an SIWIS element.
This appendix presents the expanded experimental tests results of the two-bay three-story steel frame and includes the followings:

<table>
<thead>
<tr>
<th>Test</th>
<th>Results</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBFT1</td>
<td>The strain histories of SIWIS elements’ rods</td>
<td>G1</td>
</tr>
<tr>
<td></td>
<td>Three floor levels’ deflection histories</td>
<td>G12</td>
</tr>
<tr>
<td>SBFT2</td>
<td>The strain histories of SIWIS elements’ rods</td>
<td>G2</td>
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<tr>
<td></td>
<td>Three floor levels’ deflection histories</td>
<td>G13</td>
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<tr>
<td>SBFT3</td>
<td>The strain histories of SIWIS elements’ rods</td>
<td>G3</td>
</tr>
<tr>
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<td>SIWIS element’s force histories</td>
<td>G8</td>
</tr>
<tr>
<td></td>
<td>Three floor levels’ deflection histories</td>
<td>G14</td>
</tr>
<tr>
<td>SBFT4</td>
<td>The strain histories of SIWIS elements’ rods</td>
<td>G4</td>
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<td></td>
<td>SIWIS element’s force histories</td>
<td>G8</td>
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<td>Three floor levels’ deflection histories</td>
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<td>The strain histories of SIWIS elements’ rods</td>
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<td>SIWIS element’s force histories</td>
<td>G9</td>
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<td>Three floor levels’ deflection histories</td>
<td>G16</td>
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<tr>
<td>SPFT1</td>
<td>The strain histories of SIWIS elements’ rods</td>
<td>G6</td>
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<td>SIWIS element’s force histories</td>
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<td>The strain histories of SIWIS elements’ rods</td>
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<td></td>
<td>SIWIS element’s force histories</td>
<td>G11</td>
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</table>
Figure G.1: The strain histories of SIWIS elements’ rods (SBFT1)
Figure G.2: The strain histories of SIWIS elements’ rods (SBFT2)
Figure G.3: The strain histories of SIWIS elements’ rods (SBFT3)
Figure G.4: The strain histories of SIWIS elements’ rods (SBFT4)
Figure G.5: The strain histories of SIWIS elements’ rods (SBFT5)
Figure G.6: The strain histories of SIWIS elements’ rods (SPFT1)
Figure G.7: The strain histories of SIWIS elements’ rods (SPFT2)
Figure G.8: SIWIS elements’ force histories (SBFT3 and 4)
Figure G.9: SIWIS elements’ force histories (SBFT5)
Figure G.10: SIWIS elements’ force histories (SPFT1)
Figure G.11: SIWIS elements’ force histories (SPFT2)
Figure G.12: Three floor levels’ deflection histories (SBFT1)

Figure G.13: Three floor levels’ deflection histories (SBFT2)
Figure G.14: Three floor levels’ deflection histories (SBFT3)

Figure G.15: Three floor levels’ deflection histories (SBFT4)
Figure G.16: Three floor levels’ deflection histories (SBFT5)
Appendix H

DIRECT MASONRY WALL STRENGTH
ASSESSMENT GRAPHS

For simplification and direct assessment of shear strength of unreinforced masonry walls in SIWIS system, graphs are developed for the four introduced expressions in Section 6.1 and shown in Figures H.1 to H.12 in this appendix. In these graphs, the horizontal axis presents the geometry related dimensionless parameter of $\beta = \frac{h}{l}$ varying from 0.0 to 2.0 and the vertical axis gives the value of $X_i$ factors. For the graphs except those for Expression 1, eight different values varying from 0.65 to 1.00 with 0.05 increments are considered for the $l/L$ ratio. Expression 1 results the same values for all $l/L$ ratios.

Expression 1: \[ V = X_1 Af_m \]
Expression 2: \[ V = X_2 Af_m \]
Expression 3a: \[ V = X_3 Af_m \]
Expression 3b: \[ V = X_3 A \sqrt{f_m} \]
Expression 3c: \[ V = X_3 A \sqrt{f_m} \]
Expression 4: \[ V = X_4 A \sqrt{f_m} \]
Figure H.1: Expression 1 \( (0.0 < \beta = h/l < 1.0) \)
Figure H.2: Expression 1 \((1.0 < \beta = h/l < 2.0)\)
Expression 2: \( V_{\text{shear}} = A f_m X_2 \)

Figure H.3: Expression 2 (\( 0.0 < \beta = h/l < 1.0 \))
Figure H.4: Expression 2 \( (1.0 < \beta = h/l < 2.0) \)
Figure H.5: Expression 3a ($0.0 < \beta = h/l < 1.0$)
Figure H.6: Expression 3a (1.0 < \beta = h/l < 2.0)
Figure H.7: Expression 3b ($0.0 < \beta = h/l < 1.0$)
Figure H.8: Expression 3b \((1.0 < \beta = h/l < 2.0)\)
Figure H.9: Expression 3c ($0.0 < \beta = h/l < 1.0$)
Figure H.10: Expression 3c ($1.0 < \beta = h/l < 2.0$)
Figure H.11: Expression 4 \((0.0 < \beta = h/l < 1.0)\)
Figure H.12: Expression 4 \( V_{\text{shear}} = A(f'_{m})^{0.5} X_4 \)

\[ 1.0 < \beta = h/l < 2.0 \]
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