This dissertation presents a probabilistic fragility assessment for piping systems to mitigate the seismic hazards. The percentage of construction cost for nonstructural components such as mechanical and electrical equipments, medical equipments, and piping systems is over 80% of the total cost in critical facilities. This research is conducted under the NEES Grand Challenge Project “Simulation of the Seismic Performance of Nonstructural Systems”. It focuses on characterizing the seismic performance of threaded Tee-joints in actual piping system and incorporating the experimentally observed behavior in a system level piping analysis with the purpose of evaluating piping fragilities. A key step in this process is analyze the experimentally observed moment-rotation relationship of threaded Tee-joints and use it to characterize the appropriate limit-state. A nonlinear finite element model for the threaded Tee-joint, validated against the experimental results, is then incorporated in the complete piping system model in order to facilitate system level piping analysis and fragility assessment. More specifically, the research presented in this dissertation focuses on evaluating fragility for the limit-state characterized to represent “First-Leakage” failure at the threaded Tee-joint. Finally, the effect of building performance on the piping fragility is also evaluated by considering a high rise 20-story and a low rise 5-story building. Differences in piping fragility are evaluated for piping located in buildings that remain linear as well as that exhibit significant nonlinear behavior. Variation in fragility due to location of piping at different floor levels is also explored.
Seismic Fragility of Piping System

by
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2012

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Chair of Advisory Committee
DEDICATION

To My Family
BIOGRAPHY

Bu Seog Ju was born in March 5th, 1976 in Tae-An, Korea. He received a Bachelor of Science in Civil Engineering at KyungHee University in February, 2002, and decided to study abroad in the United States. He started his Master’s program at North Carolina State University in August 2005. He received the Master of Civil Engineering in Structural Engineering and Mechanics in May 2007. Upon graduation, he decided to continue his studies at Civil Engineering-Structural Engineering. Currently he is pursuing a Ph.D. degree in Civil Engineering-Structural Engineering and Mechanics under the direction of Dr. Abhinav Gupta. While he was in graduate school, he served as a board member of Korean Student Association (KSA) in 2006-2007, and was the 35th president of KSA in 2010-2011. He is a student member of Korean-American Scientists and Engineers Association (KSEA). Bu Seog has a job offer from the department of Civil Engineering at Gangneung-Wonju National University in Korea, so he is going back to Korea after graduation in December, 2011.
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PART I

INTRODUCTION
1. INTRODUCTION

In critical facilities typically hospitals and school buildings, structural and nonstructural components must remain operational or functional after an earthquake. Nonstructural components for maintaining the operation of the buildings are defined as architectural components, piping systems, mechanical and electrical equipments, and building contents. In recent years, the economic loss due to failures of nonstructural components has been much more considerable than that due to failure of structural systems in most seismic events. It has been observed that the nonstructural components are much more fragile than the structural components during an earthquake. In some cases, the failure of nonstructural components has also resulted in injuries and loss of life.

Nonstructural damage, especially in hospital buildings, has been reported through the past earthquakes. For example, during the 1971 San Fernando Earthquake, 4 of the 11 major medical facilities in the area had extensive damage to nonstructural systems (Wasilewski, 1998) and during the 1994 Northridge Earthquake, 3 hospital buildings in the area were shut down and 915 patients evacuated. The Olive View Hospital which was retrofitted for withstanding earthquake forces after the 1971 San Fernando Earthquake was also shut down for a week because of damage to nonstructural components (Ayres, 1998). The economic loss due to failure of nonstructural components was more than 50% of total loss. According to Fleming (1998), many instances of damage to sprinkler piping system in hospital buildings were observed during the Northridge earthquake. Few failures of sprinkler piping system were caused by the differential movement between sprinkler heads and ceiling
system and by the failures of screwed joints or threaded Tee-joints in the fire protection piping systems.

A reduction in the damage to nonstructural components or an improvement the performance of nonstructural components has emerged as a key area of research in recent years. This research focuses on understanding the seismic performance of piping systems that are typical of hospital and/or office buildings. More specifically, it is targeted on evaluating piping fragilities by characterizing the piping performance using results from experimental test on piping components such as threaded Tee-joints, validating analytical models using experimental results, and then conducting simulations for complete piping system configurations to evaluate system fragilities.

2. BACKGROUND

2.1 ENGINEERING DEMAND PARAMETERS FOR NONSTRUCTURAL COMPONENTS (Bachman et. al, 2004)

The primary goal of ATC-58 project is to improve the next generation performance based design guidelines associated with the identification of EDPs (Engineering Demand Parameters) characterizing the response of the structure and EDP_{NS} (Nonstructural Engineering Demand Parameters) specifying the response of the nonstructural components and contents.

As the current Nonstructural Engineering Demand Parameters are not incorporated
in the nonlinear dynamic response of the actual building or actual components itself, the new

generation $E_{DPS}$ should establish the significant correlation between the $E_{DPS}$ and limit

states. Depending upon such complexities, the next $E_{DPS}$ are to be classified into two basic
categories. The first classification of $E_{DPS}$ is linked to building responses such as peak floor
accelerations, peak velocity of floor, and inter story drift and the second classification of

$E_{DPS}$ is related to the evaluation of secondary response parameters like the inelastic rotation
of a pipe joint.

The fragility functions, based on the relationship between $E_{DPS}$ and the probability

of reaching or exceeding various limit states at given levels, would deal with the $E_{DPS}$
identified for the above issues. In order to generate the fragility curves, the definition of the
damage states such as no damage, leakage, and loss of function must be well defined.

2.2 TESTING SPRINKLER-PIPE SEISMIC-BRACE COMPONENTS (Malhotra et. al, 2003)

The current design codes and standards such as Uniform Building Code (UBC, 1997),
have prescribed the allowable values of the seismic load in the bracing components of fire
protection piping system. However, the various components of pipe braces can fail in low-
cycle fatigue. The number of load cycles for design of hangers and seismic braces in lateral
and longitudinal directions has not been specified in the current design codes.

Unlike Antaki and Guzy (1998), tests performed by Malhotra et. al. (2003) focused
on determining the number of load cycles due to seismic loads on bracing components and evaluated their seismic strength and the cyclic behavior. The test results indicated considerable stiffness and strength degradation of the brace components under the cyclic loading conditions. Also, the failure modes in the monotonic test certainly differed from the failure modes in cyclic conditions. They also proposed that the tests of the friction based and non-friction based components be performed at different frequencies to obtain a conservative load rating.

2.3 CODE BASED EVALUATION OF SEISMIC FORCE LEVELS FOR HOSPITAL EQUIPMENT (Horne and Burton, 2003)

The design forces specified in California Building Code (CBC, 2001) and IBC (2000) have included various new items such as component ductility, location in the structure, and site specific acceleration. The design of structural system is related to resonance of the structure with earthquake frequencies. However, no dynamic interaction between the equipment and structure is considered.

Horne and Burton (2003) suggested that current design codes for nonstructural anchorage reflect a relationship between the fundamental periods of equipment and the supporting structure. In this case, revised design forces defined by the general average amplification response can apply to static, dynamic, and any individual modal response. Also, the equipment design forces can also be more realistically associated with dynamic performance rather than the semi-arbitrary distinction of flexible or rigid. According to the
results based on linear elastic analysis, friction at supports and yielding of equipment tend to flatten out the plots of the amplification versus the period and result in smaller amplification factors, but this analytical study needs to be verified for the nonlinear cases.

2.4 SEISMIC BEHAVIOR OF WELDED HOSPITAL PIPING SYSTEMS (Maragakis, Itani, and Goodwin, 2003)

The purpose of this research was to understand the seismic behavior of welded hospital piping system and to evaluate their seismic capacity as well as identify critical locations. Maragakis, Itani, and Goodwin (2003) conducted shake table experiments on cable-braced and unbraced welded hospital piping. The results indicated that the cable-braced systems considerably reduced the displacement responses. On the other hand, the acceleration responses were not affected by the addition of braces.

3. RESEARCH OBJECTIVES

The primary objective of this research is to evaluate system-level seismic fragility of nonstructural systems such as piping by incorporating experimentally obtained characteristics of piping components in a system level simulation. A related objective is to characterize the experimentally evaluated performance levels of piping components in terms of certain engineering response parameters(s). Then, develop a non-linear finite element model of the piping component for incorporation in system-level piping models. Finally, use system-level
simulations to evaluate piping fragilities associated with different scenarios.

4. PROPOSED RESEARCH

The specific tasks needed to accomplish the objectives of this research are characterized in the followings:

4.1 BEHAVIOR OF EXISTING HOSPITAL PIPING

- Understand the basis of existing guidelines and design codes such as NFPA-13 and SMACNA.
- Study the configuration of an actual hospital piping for a hospital in California.
- Evaluate if the support spacing in the actual hospital piping satisfy NFPA-13 and SMACNA requirements. If not, add supports / bracings in accordance with NFPA-13 and SMACNA.
- Use linear time history analysis of the modified piping system to further evaluate if piping responses are with acceptable range. If not, modify the bracing system further. The purpose of this exercise is to evaluate the fragility of an actual hospital piping in which supports/bracing system satisfies the existing guidelines of NFPA-13 and SMACNA.
4.2 COMPONENT BEHAVIOR-LABORATORY TESTS OF TEE-JOINTS CONDUCTED AT UB

- Study the data from the laboratory tests on various Tee-joints conducted at University at Buffalo (UB).
- Estimate the force-deflection and moment-rotation relationships at threaded Tee-joints of fire protection piping system for both the monotonic and cyclic loading tests.
- Study the data for different materials and different diameter piping Tee-joints.

4.3 CHARACTERIZE THE LIMIT-STATES FOR FAILURE AT THE THREADED TEE-JOINTS

- Study the limit-states used to characterize the performance of piping components in ASME Section III BPV&P code (Gerdeen, 1979).
- Characterize the failure in terms of an “unacceptable performance” – typically represented by the first leakage.
- Characterize the limit-states for the first leakage at the threaded Tee-joints of the piping system in terms of an engineering demand parameter.
- If possible, relate the engineering demand parameter used for characterizing the Tee-joint performance to a definition of limit-state that is similar to the ASME BVP&P code (Gerdeen, 1979).
4.4 MODEL THE BEHAVIOR OF PIPING COMPONENT USING FE ANALYSIS AND RECONCILIATION WITH THE EXPERIMENTAL TESTS

- Develop a non-linear finite element model of the threaded Tee-joints using OpenSees. This model will be based on the experimental test results.
- Reconciliation of FE analysis results for monotonic load cases by comparison with experimental data.
- Reconciliation of FE analysis results for cyclic load cases by comparison with experimental data.

4.5 EVALUATE SEISMIC FRAGILITY IN ACTUAL PIPING SYSTEM

- Incorporate the non-linear FE model for the Tee-joint at a single “critical” location in the actual piping system considered in this study. The “critical” location can be identified based on location of maximum moment and rotation among all Tee-joints from a linear time history analysis. The purpose of including only a single location is to study the problem in steps by increasing complexity incrementally. To begin with, consider the non-linear model based on monotonic loading case.
- Conduct multiple time history analyses of the piping system with non-linear Tee-joint using 75 real earthquake records. Calculate the piping fragility for the case of monotonic loading based single non-linear Tee-joint location.
- Consider non-linear model for the Tee-joint at additional locations incrementally.
- Generate the fragility of piping system of the case of the multiple possible failure
locations. In this process, study the changes in the fragility of the single-location case when failure can occur at more than single location.

4.6 STUDY THE EFFECT OF STRENGTHENING/RETROFITTING A GIVEN TEE-JOINT

• Often, it is required that the “most-probable” failure location identified in a fragility analysis be strengthened/retrofitted for improved seismic performance. The purpose of this task is to evaluate the effect of such strengthening on the changes in the fragility associated with failures at other possible locations.

4.7 EVALUATE SEISMIC FRAGILITY USING CYCLIC TEST RESULTS

• Incorporate the FE model for non-linear moment-rotation loading-unloading curves based on cyclic test data into the actual piping system at a single critical location.

• Evaluate the piping fragility failure at a single location for this case.

• Compare the fragility with that calculated using the monotonic case.

4.8 PIPING FRAGILITY EVALUATION: INTERACTION WITH BUILDING PERFORMANCE

• Model multi-storied (5 and 20 story) moment frame building system using
OpenSees to understand the effect of dynamic interaction between the piping and the building systems on the piping fragilities.

- For each floor, obtain the acceleration time histories through linear/nonlinear analyses using various earthquake records from multiple events, all normalized to the same PGA.
- Consider the interaction of linear and nonlinear building systems with the piping system to evaluate the effect of building performance on piping fragility.
- Compare the piping fragility in nonlinear multi-story moment frame building systems with that in linear multi-story moment frame building systems.
5. ORGANIZATION

This dissertation is comprised of three main parts each of which corresponds to a different manuscript. The first manuscript (part II) of this dissertation describes the review of existing literature on real-life failures, seismic design guidelines, and experimental tests for piping system in critical building systems. This manuscript is being considered for publication as an MCEER Technical Report.

The part III of this dissertation presents a framework for evaluating system-level fragility of piping system and its components by incorporating experimentally validated nonlinear Tee-joint model in the piping systems. This manuscript will be submitted for possible publication in ASME’s Journal of Pressure Vessel Technology.

The manuscript in Part IV of this study focuses on evaluating system-level fragility of the piping system by considering the effect of nonlinearity in high-rise buildings. This manuscript will be submitted for possible publication in ASME’s Journal of Pressure Vessel Technology.

The last part of this dissertation summarizes the conclusions of seismic piping fragility evaluation framework presented in the three manuscripts mentioned above. In addition, it gives the recommendation for future research in this area.
REFERENCES


PART II

REVIEW OF EXISTING LITERATURE ON REAL-LIFE FAILURES, SEISMIC DESIGN GUIDELINES, AND EXPERIMENTAL TESTS FOR PIPING SYSTEMS IN BUILDINGS

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1. REVIEW OF DAMAGE OF NONSTRUCTURAL SYSTEM IN RECENT EARTHQUAKES

1.1 ECONOMIC LOSS

Nonstructural components that make up a considerable part of the building construction cost in offices, hotels, and hospital buildings, are (a) piping systems, (b) ceilings building contents, and (c) mechanical and electrical equipment. Damage to these systems can result in a major economic loss, injuries, and loss of life in hospital buildings. Figure 2-1 presents the typical ratios of investment between structural and nonstructural components in different kinds of buildings (Whittaker and Soong, 2003). According to this data, the nonstructural elements tend to comprise over 80% of the total construction cost of the structure. The economic loss due to nonstructural systems can be significant during an earthquake. The 1994 Northridge earthquake resulted in a total direct economic loss of $23.3 billion of which $18.8 billion went to repair or replacement of damaged structural and nonstructural systems. Furthermore, over 50% of the $18.8 billion corresponded to nonstructural component damage alone (Kircher, 2003). During the 2001 Nisqually Earthquake, the damage to nonstructural components resulted in a total of $2 billion economic loss (Filiatrault et. al., 2005). Also the recent 8.8 magnitude Chile Earthquake of Feb. 27th, 2010, caused severe damage to structural and nonstructural units and the total estimation of economic loss of structural and nonstructural components was about $30 billion.
1.2 PIPING FAILURES

Due to the high cost associated with the repair and reinstallation of the piping systems, the seismic performance evaluation and design of nonstructural systems has received significant attention in recent years. Damage during an earthquake could increase the risk of fire hazard, leakage of water and other effluents or even loss of life. Due to the prevalence of significant higher mode effects, the complexity of the failure mechanisms is enhanced. Moreover, the common damage/failure patterns have been identified to exist at more than one location in a piping system. Figure 2-2 to Figure 2-11 shows schematic
representations of typical failure mechanisms of various kinds of nonstructural systems in power plants and hospitals.

Figure 2-2: Unanchored Tanks – Slide & Twist on Saddles

Figure 2-3: Unanchored Flat Bottom Tanks-Slide and Rock
Figure 2-4: Threaded Pipe Coupling - Leakage due to Bending

Figure 2-5: Piping Lift off Shallow Saddles
Figure 2-6: Sprinkler Pipe–Sways and Impacts Suspended Ceiling

Figure 2-7: Suspended Header–Stiff Branch Breaks
Figure 2-8: HVAC Heater—Sways and Ruptures the Copper Tube

Figure 2-9: C-Clamp, which Relies on Friction, may Slide
Figure 2-10: Underside Weld-Shearing

Figure 2-11: Spring Support–Slides from under the Pipe
1.2.1 THE SAN FERNANDO EARTHQUAKE IN 1971

The 1971 San Fernando earthquake was the turning point in seismic design of buildings and nonstructural components. Due to the failures observed in this earthquake, OSHPD started requiring that the hospital buildings remain fully operational following an earthquake. Figure 2-12 and Figure 2-13 show the failures of piping systems during the 1971 San Fernando Earthquake.

![Failure of Central Plant Pipe (Gates, 2005)](image)

Figure 2-12: Failure of Central Plant Pipe (Gates, 2005)
1.2.2 THE NORTHRIDGE EARTHQUAKE IN 1994

The 1993 Northridge Earthquake resulted in the greatest economic loss to building structures, highways and bridges in United States history. During the Northridge earthquake, 51 people were killed and over 9,000 people were injured. Major highways collapsed and 9 hospitals were shut-down in Los Angeles area. A total of 2,500 beds were lost for usage in the hospitals. There were serious damages to nonstructural components, especially the piping systems. Inside the buildings, water lines were broken, and most hospital buildings suffered from significant water damage due to failure of water chilled and hot water pipe lines. For example, the Olive View Hospital had no structural damage, but the hospital was closed because of water damage (Ayres and Phillips, 1998). Failures were observed in several
different piping systems such as HVAC systems, sprinkler piping systems, and water piping systems. The major reason of the damage to fire sprinkler piping systems has been identified as the excessive vertical acceleration. Vertical acceleration led to impact of piping with ceiling and caused the failures in fire sprinkler piping systems. Another reason for significant damage in sprinkler systems was attributed to bracing type and brace spacing of pipe lines. Unbraced pipelines less than 1 inch diameter experienced widespread failures (Filiatraul et al., 2001). Figure 2-14 illustrates a typical failure of fire sprinkler piping system in the Northridge Earthquake (Miranda, 2004). Figure 2-15 gives an example of water damage due to failure of water pipe lines (Miranda, 2004) and Figure 2-16 shows the failure of fire sprinkler head due to vertical acceleration.
Figure 2-15: Water Damage due to Failure of Pipe Lines (Miranda, 2004)

Figure 2-16: Failure of Fire Sprinkler due to Vertical Acceleration
Besides, according to Fleming (1998), 9 hospitals in the area suffered damage due to water by failure of fire sprinkler, HVAC, and domestic water systems (Fleming, 1998). The examples given below describe the failures observed in sprinkler piping systems (Fleming, 1998).

- Cedars-Sinai Medical (0.4g) – Sprinklers on 1-inch lines crossing a seismic separation on floors 4 through 8 were activated by striking other building components. (Fleming, 1998).
- Holy Cross Medical Center (0.4g) – Short drops (6 to 10 inches long) failed at threaded Tee-joints (Fleming, 1998).
- Santa Monica Hospital Medical Center (0.6g) – 1-inch line failed at a Tee-joint by impact against a duct. (Fleming, 1998).

1.2.3 THE NISQUALLY EARTHQUAKE IN 2001

The 2001 Nisqually Earthquake was the largest earthquake in the history of the state of Washington and caused wide spread damage in the Puget Sound area of Washington State. The Nisqually Earthquake injured about 400 people. Fortunately, there was no fatality related directly to the earthquake. The economic loss due to nonstructural component damage was approximately $2 billions. There were also the damage to piping systems such as the failure of water line and chilled water line on the fourth floor of the Kent Regional Center (Filiatrault et. al., 2001). During the earthquake, 75-mm diameter of water pipe line failed at the mechanical room of the roof of a hotel, and the unstable water tank was shifted
about 150-mm on the floor. Figure 2-17 shows the ruptured pipe of the supply water line of
the tank in the hotel (Filiatrault et. al., 2001).

![Figure 2-17: Failure of Piping System (Filiatrault et. al., 2001)](image)

1.2.4 THE CHILE EARTHQUAKE IN 2010

The 2010 Chile Earthquake of magnitude 8.8 caused severe structural and
nonstructural damage in all type of buildings.

Especially, this strong earthquake influenced most of hospital buildings in central
south region. 4 hospitals were not able to operate and 12 hospitals lost almost 75% function
of the building system. Most of loss in hospitals was caused by damage to nonstructural
components such as suspended ceilings, light fixtures, and fire sprinkler piping system.
Figure 2-18 shows that a pipe supporting system became unrestrained from its anchor or bracing. It moved about 90 degrees and because of this relative motion, the connecting pipes failed and slipped away from the joint, as shown in Figure 2-19.

Figure 2-18: Failure of Piping System (Photo: G. Mosqueda, 2010)
2. GUIDELINES FOR SEISMIC DESIGN OF FIRE PROTECTION SPRINKLER PIPING SYSTEM

2.1 NATIONAL FIRE PROTECTION ASSOCIATION (NFPA-13, 2007)

The NBFU-13 (“The National Board of Fire Underwriters for the Installation of Sprinkler Equipments”) was the first set of guidelines addressing the seismic protection for fire protection sprinkler piping systems:

- The 1950 edition of NFPA-13 contained very few guidelines regarding damage during earthquakes
• The 1951 edition of NFPA-13 included standards for longitudinal and lateral bracing, flexible coupling on risers, and maximum slenderness ratio of bracing components.

• The 1991 edition of NFPA-13 reorganized the standards of sprinkler installation and inscribed the need to constrain branch systems where movement could damage sprinklers through impact against other building features. (Fleming, 1998).

• The 1999 edition of NFPA-13 considered the need to change all subject areas of the placements, spacing, location, bracing, and type of sprinklers to protect damage due to earthquakes and included the installations of underground piping system.

The 2002 and 2007 editions considered irregular ceiling systems and seismic bracing criteria of the applicable requirements for standards and design of fire sprinkler systems.

The maximum distances between the hangers of piping systems as prescribed in NFPA-13(2007) are listed in Table 2-1. The seismic design requirements of hanging, bracing, and restraint of piping systems as given in NFPA-13 (2007) are given below:

1. Where water-based fire protection systems are required to be protected against damage from earthquakes, hangers shall also meet the requirements.

2. C-type claps used to attach hangers to the building structure in areas subject to earthquakes shall be equipped with a restraining strap.

3. The restraining strap shall be listed for use with a C-type clamp or shall be a steel strap of not less than 16 gauge thickness and not less than 1 in. wide for pipe diameters 8 in. or less and 14 gauge thickness and not less than 1 1/4 in. wide for pipe diameters greater than 8 in.

4. The restraining strap shall wrap around the beam flange not less than 1 in.
5. A lock nut on a C-type clamp shall not be used as a method of restraint.

6. A lip on a “C” or “Z” purlin shall not be used as method of restraint.

7. Where purlins or beams do not provide an adequate lip to be secured by a restraining strap, the strap shall be through-bolted or secured by a self-tapping screw.

8. Power-driven fasteners shall not be used to attach braces to the building structure, unless they are specifically listed for service in resisting lateral loads in areas subject to earthquakes.

9. In areas where the horizontal force factor exceeds 0.5$W_p$, power-driven studs shall be permitted to attach hangers to the building structure where they are specifically listed for use in areas subject to earthquakes.

10. Longitudinal sway bracing spaced at a maximum of 80ft on center shall be provided for feed and cross mains.

11. Longitudinal braces shall be allowed to act as lateral braces if they are within 24 in. of the centerline of the piping braced laterally.

12. The distance between the last brace and the end of the pipe shall not exceed 40 ft.

13. The horizontal force, $F_{pw}$, acting on the brace shall be taken as $F_{pw} = C_p W_p$, where $C_p$ is the seismic coefficient (Table 2-2) utilizing the short period response parameter $S_s$ obtained from the authority having jurisdiction or from seismic hazard maps.

14. Where the authority having jurisdiction does not specify the horizontal seismic load, the horizontal seismic force acting on the braces shall be determined as specified with $C_p = 0.5$. 

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Table 2-1: Maximum Distance between Hangers (NFPA-13, 2007)

<table>
<thead>
<tr>
<th>Size (in)</th>
<th>1</th>
<th>1.25</th>
<th>1.5</th>
<th>2</th>
<th>2.5</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel (ft)</td>
<td>12</td>
<td>12</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Threaded light wall steel (ft)</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Copper tube (ft)</td>
<td>8</td>
<td>10</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>15</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>CPVC (ft)</td>
<td>6</td>
<td>6-6</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>10</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Table 2-2: Seismic Coefficient Table (NFPA-13, 2007)

<table>
<thead>
<tr>
<th>$S_s$</th>
<th>$C_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 0.33$</td>
<td>0.31</td>
</tr>
<tr>
<td>0.50</td>
<td>0.40</td>
</tr>
<tr>
<td>0.75</td>
<td>0.43</td>
</tr>
<tr>
<td>0.95</td>
<td>0.50</td>
</tr>
<tr>
<td>1.00</td>
<td>0.52</td>
</tr>
<tr>
<td>1.25</td>
<td>0.60</td>
</tr>
<tr>
<td>1.50</td>
<td>0.71</td>
</tr>
<tr>
<td>2.00</td>
<td>0.95</td>
</tr>
<tr>
<td>2.40</td>
<td>1.14</td>
</tr>
<tr>
<td>3.00</td>
<td>1.43</td>
</tr>
</tbody>
</table>
Although, the main hospital structure may not be structurally damaged during an earthquake, the nonstructural components such as sheet metal ducts, piping, and conduit systems, equipment etc. are vulnerable to severe damage. Therefore, the nonstructural systems must be designed to adequately resist the lateral forces during the earthquake. Following the 1971 San Fernando earthquake, the state of California required that the hospital buildings remain fully functional during after an earthquake. The California Office of Statewide Health Planning and Development (OSHPD) approved the design guidelines for bracing of piping and duct systems as specified by SMACNA in 1976. SMACNA has added several seismic restraints such as transverse and longitudinal bracing types over the decades.

SMACNA provides the design guidelines and seismic restraint detailing for piping and duct systems. The bracing types for ducts or piping systems is determined based on the desired Seismic Hazard Level (SHL). The base seismic coefficient in SMACNA is based on the 1997 Uniform Building Code (UBC) guidelines and is given by:

\[
\frac{F_p}{W_p} = \frac{a_p \times C_a \times I_p}{R_p}.
\]

Where,

- \(F_p\): Seismic Force
- \(W_p\): Weight of the Item to be Braced
- \(a_p\): Component Amplification Factor
$C_0$: Seismic Coefficient

$I_\rho$: Occupancy Category

$R_\rho$: Component Response Modification Factor

Figure 2-20: Seismic Hazard Level (SHL) (SMACNA, 2003)

Figure 2-20 plots the base seismic coefficients as a function of relative height of the bracing for various SHLs. A particular SHL is a designation that implicitly combines several building codes, seismic zones, and importance factors into a single quantity for determining appropriate restraints. SHL A is the most severe because of the likely earthquake strength and because of the severity of consequences if the building becomes unusable. SHL C is the least...
severe and has the lowest requirements (SMACNA-Appendix E, 1998).

Table 2-3 illustrates the bracing, support types and maximum load capacity of a 4-inch pipe using Figure 2-20 according to SHL prescribed by SMACNA. Table 2-4 gives the maximum load capacity for each of the support type as prescribed.

Table 2-3: Brace Spacing and Support Types (SMACNA, 2003)

<table>
<thead>
<tr>
<th>SHL</th>
<th>Pipe Size (in)</th>
<th>Max. Wt. (lb/ft)</th>
<th>Brace Spacing</th>
<th>Support Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Trans.(ft)</td>
<td>Level 1</td>
</tr>
<tr>
<td>SHL-A</td>
<td>4</td>
<td>16.3</td>
<td>40</td>
<td>D</td>
</tr>
<tr>
<td>SHL-B</td>
<td>4</td>
<td>16.3</td>
<td>40</td>
<td>C</td>
</tr>
<tr>
<td>SHL-C</td>
<td>4</td>
<td>16.3</td>
<td>40</td>
<td>A</td>
</tr>
<tr>
<td>SHL-AA</td>
<td>4</td>
<td>16.3</td>
<td>40</td>
<td>E</td>
</tr>
</tbody>
</table>

Table 2-4: Maximum Load Capacity (SMACNA, 2003)

<table>
<thead>
<tr>
<th>SUPPORT TYPE</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOAD (lb)</td>
<td>1040</td>
<td>1415</td>
<td>1586</td>
<td>2020</td>
<td>2870</td>
<td>4600</td>
<td>7040</td>
<td>9240</td>
</tr>
</tbody>
</table>
According to Table 2-3 and Table 2-4, the maximum transverse brace spacing for a 4-inch pipe is 40 ft and the maximum longitudinal brace spacing for a 4-inch pipe is 80 ft. The level 2 represents the critical facilities such as schools and hospital buildings. Therefore, the maximum load capacities range from 1040 (lb) to 4600 (lb) depending upon the seismic hazard level (SHL).

3. REVIEW OF EXPERIMENTAL TESTS OF FIRE PROTECTION PIPING SYSTEMS

3.1 INTRODUCTION

In facilities like the hospitals, the fire sprinkler piping systems are one of the most critical systems that must continue operating during earthquakes. The locations most susceptible to damage in these piping components are the joints and support hangers/systems. The behavior of threaded and grooved joints in the piping are of special interest as leakage and rupture is generally caused at these points under the actions of extreme lateral loads or earthquakes.

Therefore, experimental tests have been conducted to study the stiffness and failure modes at the threaded and grooved joints. Two different tests, based on national standards such as NFPA-13, were carried out - a) Static Test and b) Dynamic Tests (Antaki and Guzy, 1998).
3.2 STATIC TEST OF FIRE PROTECTION PIPING SYSTEMS

Figure 2-21 illustrates test layout of a four-point bending test of a simply supported pipe. A total of 16 specimens were tested that consisted of 2-inch and 4-inch schedule 40 carbon steel pipe with flexible and rigid types of grooved coupling and threaded joints at mid-span. The test was performed with water pressure at 150 psi.

![Figure 2-21: Static Four-Point Bend Test of Grooved Coupling and Threaded Joints (Antaki and Guzy, 1998)](image)

3.2.1 STATIC TEST OF GROOVED COUPLING SYSTEMS

Test data in terms of Load (lb) - Deflection (in) curves were obtained up to the first
leakage point of the grooved coupling. Different failure modes (major versus minor leaking) were observed in the 2-inch and 4-inch pipe coupling systems respectively. The 4-inch pipe coupling system was much stiffer than 2-inch pipe coupling system. However, the 4-inch pipe coupling system suffered a loss of stiffness upon increase in displacement leading to a partial fracture occurring at coupling joints (Figure 2-22). Also, the rotations at the coupling point were significantly larger for flexible coupling of 2-inch and 4-inch grooved coupling system.

![Figure 2-22: Deflection 2-inch Flexible Grooved Coupling (Antaki and Guzy, 1998)](image)
Figure 2-23: Load-Deflection 2-inch Rigid Grooved Coupling (Antaki and Guzy, 1998)

Figure 2-24: Load-Deflection 4-inch Flexible Grooved Coupling (Antaki and Guzy, 1998)
Table 2-5 describes the rotational stiffness and moment capacities of flexible and rigid coupling joint system.
3.2.2 STATIC TEST OF THREADED JOINTS

The static test of threaded joints of 2-inch schedule 40 threaded carbon steel pipe was conducted in the same manner as grooved coupling. A total of 4 specimens were tested with threaded joints at mid-span. Figure 2-27 shows the load-deflection curve obtained in these tests. However, 3 specimens broke down due to rupture at the first exposed thread and one specimen failed by stripping of the engaged threads (Antaki and Guzy, 1998)

<table>
<thead>
<tr>
<th></th>
<th>Stiffness (lb-in / rad)</th>
<th>Moment Capacity (kips-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-inch Rigid or Flexible</td>
<td>$1.0 \times 10^5$ to $3.0 \times 10^5$</td>
<td>12 to 30</td>
</tr>
<tr>
<td>4-inch Rigid or Flexible</td>
<td>$1.2 \times 10^6$ to $2.4 \times 10^6$</td>
<td>24 to 96</td>
</tr>
</tbody>
</table>
3.2.3 DYNAMIC TEST OF GROOVED COUPLING AND THREADED JOINTS

The dynamic test was conducted to understand the cyclic/seismic performance of grooved coupling and threaded joint system. A total of 16 grooved coupling joints and 4 threaded joints specimens were tested on a shake table. Each specimen consisted of a 6 foot long pipe filled with water at 150 psi (same as the static test). The test setup is shown in Figure 2-28. The excitation level was increased gradually. The leakage of the grooved coupling was observed at 70% of the capacity and the failure due to bending was similar to that observed in the static test (Figure 2-29). However, the leakage of the threaded joints was observed at 50% and 25% of the capacity.

Figure 2-28: Dynamic Test Specimens of the Coupling System and Thread Joints (Antaki and Guzy, 1998)
Figure 2-29: Failure Mode of Grooved Coupling by Dynamic Test (Antaki, 2004)
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PART III

FRAGILITY ANALYSIS OF THREADED TEE-JOINT CONNECTIONS IN HOSPITAL PIPING SYSTEMS

Bu Seog Ju, Sashi Kanth Tadinada, and Abhinav Gupta
FRAGILITY ANALYSIS OF THREADED TEE-JOINT CONNECTIONS IN HOSPITAL PIPING SYSTEMS

Bu Seog Ju, Sashi Kanth Tadinada, and Abhinav Gupta

ABSTRACT

The cost of damage to the non-structural systems in critical facilities like nuclear power plants and hospitals can exceed 80% of the total cost of damage during an earthquake. Studies assessing damage from the 1974 San Fernando and 1994 Northridge earthquakes reported a widespread failure of non-structural components like sprinkler piping systems (Ayer and Phillips, 1998). The failure of piping systems led to leakage of water and subsequent shut-down of hospitals immediately after the event. Consequently, probabilistic seismic fragility studies for these types of structural configurations have become necessary to mitigate the risk and to achieve reliable designs.

This paper proposes a methodology to evaluate seismic fragility of threaded Tee-joint connections found in typical hospital floor piping systems. Numerous experiments on threaded Tee-joints of various sizes subjected to monotonic and cyclic loading conducted at University of Buffalo indicate that the "First Leak" damage state is observed predominantly due to excessive flexural deformations at the Tee-joint section. The results of the monotonic and cyclic loading tests help us evaluate the following characteristics for a given pipe size and material:

(i) Maximum allowable value of rotational deformation at the Tee-joint section to prevent
"First Leak" damage state

(ii) The force-displacement and moment-rotation relationships at the Tee-joint section

A non-linear finite element model for the Tee-joint system is formulated and validated with the experimental results. It is shown that the Tee-joint section can be satisfactorily modeled using non-linear rotational springs. The system-level fragility of the complete piping system corresponding to the “First Leak” damage state is determined from multiple time-history analyses using a Monte-Carlo simulation accounting for uncertainties in demand.
1. INTRODUCTION

The total installation and construction cost of non-structural elements in any critical facility like a hospital or a nuclear power plant is almost 80% of the total cost (Ritherman, 2009). Furthermore, damage to the non-structural systems in hospitals comprise a significant proportion of the total economic loss incurred in the event of an earthquake. During the 1994 Northridge earthquake, 85% of the total $7.4 billion damage is attributed to non-structural systems (Kircher, 2003). The Olive View Hospital had to be shut down soon after the 1994 Northridge earthquake due to water damage caused by failure of sprinkler systems (Ayer & Phillips, 1998). Similarly, during the 1971 San Fernando earthquake, 4 of 11 medical facilities in the area incurred significant economic losses due to damaged non-structural components (Wasilewski, 1998). Damage to components such as fire protection piping system, Heating, Ventilating, and Air Conditioning (HVAC), and water piping systems have resulted in direct economic loss and injuries or loss of life in many seismic events.

In recent years, many engineers have recognized the need to address the problem in the design stages such that the nonstructural components remain operational or functional after an earthquake. Antaki and Guzy (1998) conducted static and dynamic tests of the fire protection piping systems designed in accordance with National Fire Protection Association (NFPA-13). The objective of tests was to identify the stiffness, failure modes, and limit states for leakage of threaded pipe joints and grooved coupling systems that are commonly used in the piping systems. ATC-58 (2004) has highlighted the need for a performance-based design based on statistical approaches in nonstructural systems. Consequently, probabilistic seismic
fragility studies for these systems can be vital in mitigating risk and achieving reliable designs.

A fragility curve describes the relationship between a ground motion intensity parameter like Peak Ground Acceleration (PGA) and the corresponding probability of failure as characterized by a specified limit-state or exceedence criteria. The concept of fragility has been used extensively for over more than a decade in the seismic probabilistic risk assessment of nuclear power plant structural and nonstructural systems. In the nuclear power plant industry, the most commonly used approach to evaluate structural fragilities is the lognormal model based on design factors of safety (Kennedy et al, 1980; EPRI, 1994) which assumes a lognormal distribution for the acceleration capacity of the structure. The “acceleration capacity” of a structure is defined by the level of PGA that the structure can safely withstand without any damage and is equal to the Design Basis Earthquake (DBE) level of PGA multiplied by a Capacity Factor ($F_c$). The median and the logarithmic standard deviations of $F_c$ are calculated from design code equations, past studies, experience data and to quite an extent on expert judgment.

The widely used lognormal model of structural fragility cannot be directly extended to the piping and secondary systems due to the lack of sufficient existing fragility data. Furthermore, characterization of the structural performance in terms of an “acceleration capacity” is quite simplistic and in many cases far from the “true performance” especially as the higher mode effects and mass interaction with the supporting structure tends to be significant in piping systems. Many researchers have evaluated seismic fragilities using experimental data either independently or in conjunction with experimentally validated finite
element models (Lupoi et al, 2006). Consideration of experimental data is essential in a
fragility assessment primarily for the purpose of characterizing the structural performance in
terms of an appropriate “limit-state”. Almost all the studies have focused on evaluating the
seismic fragilities of structural components or sub-systems. Limitations in conducting large-
scale experiments are a key obstacle in the evaluation of system-level fragilities. This is
particularly true in the case of piping-systems.

In this paper, we present the results of a detailed study that focuses on:

- Characterizing the performance “limit-state” of threaded Tee-joints in a fire sprinkler
  piping system using experimental results from monotonic and cyclic testing of Tee-
  joint components.

- Developing an equivalent non-linear finite element model of the Tee-joint component
  such that the results reconcile with the experimentally observed behavior.

- Incorporating the non-linear Tee-joint model in an actual hospital piping system
  model for the purpose of conducting a system level analysis and evaluate system level
  fragilities.

- Evaluating system-level fragilities for failure at different locations of the same Tee-
  joint component within the piping system.

- Often, it is suggested that the excessive computational effort needed in a non-linear
  system-level analysis can be reduced by using linear analysis of the piping system to
  evaluate earthquake input at the location of the Tee-joint. Then, conduct a separate
(decoupled) non-linear analysis of the Tee-joint to determine the fragility. In this study, we compare the fragility evaluated from a system-level analysis with that evaluated from the corresponding decoupled analysis of the non-linear Tee-joint.

2. DESCRIPTION OF EXPERIMENTAL TESTS OF THREADED TEE-JOINT

Many assemblies and sub-assemblies in piping systems like connections and linkages often exhibit complex nonlinear behavior that is quite difficult to model or predict using simple finite element models. An accurate FE model for a connection like a threaded piping joint often requires dense meshing using special contact elements thus making the problem computationally ineffective for fragility estimations. In such cases, simple monotonic and cyclic tests can provide valuable insights into the behavior of these connections and their typical damage states. Data from such tests can also help us formulate simpler but experimentally validated nonlinear finite element models that describe the seismic response of the connections with sufficient accuracy.

Laboratory tests were conducted by University at Buffalo, State University of New York (UB) on various kinds of Tee-joint component of sprinkler piping systems (Dow, 2010). Both, monotonic as well as cyclic loading were considered. Two types of piping material, Black Iron and CPVC, were considered. A total of three different diameter pipe components, 2 inch, 1 inch and ¾ inch, were included. In this paper, a detailed discussion is provided on the experimental setup and results for only the 2 inch diameter Black Iron threaded piping
Figure 3-1 shows the actual test set-up for determining the force-displacement and moment-rotation relationships of the threaded Tee-joint connection under monotonic and cyclic loading conditions. Figure 3-1 also gives the schematic of this setup along with the direction of loading and the measurement locations. As seen in this figure, the loading was applied as an in-plane force loading at the bottom of the Tee-joint. The test-setup was erected in such a way that the Tee-joint can be displaced in both forward and reverse directions along the “bottom of the T”, an axis perpendicular to the center of the Tee-joint, as shown in Figure 3-1. Also, the ends of the pipes connected to the flange of the Tee-joint were simply supported. The failure was characterized in terms of “first-leakage” and if observed in terms of “pipe-fracture.” To detect the “first-leakage” point, the piping component arrangement was connected to a water hose that carried water at standard city pressure. Also, ten potentiometers were used to record the data. One monotonic test and three cyclic tests were conducted for each type of piping component (Dow, 2010).

Figure 3-2 gives the force-displacement curve as measured in the case of monotonic loading test. The “first-leakage” point is depicted by the vertical line at 1.5189 inch displacement. As seen in this figure, a leakage caused a vertical drop in the force-displacement curve at this value of displacement. The moment-rotation curves as measured at the two flanges, right and left ends, are shown in Figure 3-3 and 3-4. As in the case of force-displacement curve, the moment-rotation curve at the right-end of the Tee-joint flange also exhibits a vertical drop in Figure 3-3 at the first-leakage, corresponding to a rotation of 0.0814 radians. However, Figure 3-4 for the moment-rotation curve at the left-end of the Tee-
joint flange does not exhibit a similar vertical drop. The rotation corresponding to the first-leakage point is identified in this curve by a vertical line. This difference between the left-end and the right-end behavior exists because the first-leakage occurred at the right-end as shown in Figure 3-5. The reasons for the first-leakage are identified as bending of pipe ends and the corresponding slippage and rupture of pipe threads.

The same setup was used to conduct three different cyclic loading tests for each type of Tee-joint. The loading protocol used in these tests is shown in Figure 3-6. The corresponding force-displacement and moment-rotation curves are shown in Figure 3-7 to 3-9. The first-leakage points are also shown on these figures. It can be observed that the Tee-joint undergoes several cycles before accumulating enough rotations that cause a leakage. Table 3-1 lists the moments and rotations at the instance of “First Leak” damage state in the three cyclic tests of 2 inch Black Iron piping Tee-joint. It must be noted that the first-leakage was detected in cyclic tests at rotations in the range of 0.014 - 0.017 radians which are much smaller than the corresponding first-leakage detected in monotonic tests at rotations of 0.07 - 0.08 radians. This data implies that the behavior of the threaded connection under cyclic loading is more critical than that under monotonic loading. Next, we use the experimental data and observations to identify an engineering demand parameter that is subsequently used to characterize the performance of threaded Tee-joint in terms of a limit-state corresponding to first-leakage.
3. DEFINITION OF THE “FIRST LEAKAGE” LIMIT STATE

In order to evaluate the structural fragility, it is necessary to characterize the performance of a structural component or system in terms of a limit state. The limit-state is representative of a formal criterion corresponding to the observed failure/damage. Based on the experimental results discussed above, it can be identified that rotations at the flanges of the Tee-joint as observed from the cyclic tests are a good choice for engineering demand parameter to characterize the limit-state corresponding to the first-leakage instance. While the test data provides a good insight into the absolute values of rotations corresponding to first-leakage instance for a 2-inch diameter Black Iron pipe, it is not prudent to use these values (or a distribution derived from these values) for characterizing the capacity because the absolute values are likely to change significantly due to changes in material and/or geometrical properties of the threaded Tee-joint. For example, relatively smaller changes in the initial stiffness (slope of the moment-rotation curve) can result in a relatively larger variation in absolute value of rotations corresponding to the first leakage. Therefore, it is necessary to characterize the performance of the Tee-joint using a limit-state that can represent the behavior in a more general form. To do so, we compare the experimental results presented above with the criterion that forms the basis of design guidelines specified in the ASME (2004) for nuclear power plant piping systems. It must be noted that unlike the piping system in hospitals and office buildings that have threaded joints, the studies related to nuclear power plant piping systems are conducted on piping systems that have welded joints.

of a piping component such as a pipe-bend (elbow) or a Tee-joint in terms of a “Plastic Collapse” moment. The “Plastic Collapse” moment is calculated from the moment-rotation curve as shown in Figure 3-10. It is defined as the ordinate where a straight line drawn at an angle of $\phi$ with respect to the moment-axis intersects the moment-rotation curve. Angle $\theta$ is defined as the initial slope of the moment-rotation curve with respect to the moment-axis. The ASME design equations use this definition of “Plastic Collapse” moment to characterize the performance in terms of the moment limit-state (not a rotation based limit state). The intent behind the definition of a limit state based on moment capacity is to design for safety against the formation of a plastic hinge. Since the pipe joints in the nuclear power plant piping are welded, safety against the formation of a plastic hinge is the primary objective. On the contrary, safety against “leakage” is the primary objective in a threaded pipe joint of the types encountered in hospital and office building piping systems.

Next, we compare the “first-leakage” rotations recorded in the experimental test of threaded Tee-joints (given in Table 3-1) with the ASME definition of the failure point on the moment-rotation curve. Figure 3-11 plots the 3 first-leakage rotations from the 3 cyclic loading tests along with the moment-rotation relationship observed at the left end of the Tee-joint. Interestingly, all the 3 “first-leakage rotations” lie between the lines $\phi = 2\theta$ and $\phi = 2.5\theta$ where $\theta$ is the initial slope with respect to the moment-axis. This observation leads us to believe that the ASME criteria can be used in an alternative way, i.e. the “first-leakage” failure can be conservatively characterized in terms of a limiting rotation which corresponds to a point on the moment-rotation curve where the $\phi = 2\theta$ line intersects the
moment-rotation curve. In fact, the argument can be extended further to characterize failure limit-states for more severe damage states. For example, one may argue that if $\varphi = 2\theta$ represents minor damage (“first leakage”) then a larger rotation limit-state such as $\varphi = 3\theta$ represents severe damage. While there is no experimental basis, in this paper we consider two additional limit states simply for illustration purposes. We consider $\varphi = 2.5\theta$ to characterize “moderate damage” and $\varphi = 3\theta$ to characterize “severe damage”. The purpose of considering these additional limit states is to evaluate the relative changes in fragility curves for these three limit-states. Next, we discuss the details of the nonlinear finite element model that is created to represent the nonlinear behavior of this Tee-joint pipe component and to include it in the model of the complete piping system.

4. FINITE ELEMENT MODEL, VERIFICATION, AND FRAGILITY ESTIMATION

In order to calculate the fragility of the complete piping system (system-level fragility), it is necessary to create a nonlinear finite element model of the Tee-joint and incorporate it in the model of the complete piping system. Furthermore, the finite element model must reconcile with the results obtained from the experimental tests. Often, researchers create detailed finite element models with dense meshing and contact elements to reproduce the behavior observed in test data. Such a model while accurate can be computationally impractical for incorporation into a system-level piping analysis especially
if multiple locations in the piping system can have the non-linear Tee-joint. Therefore, we developed a simple model of the Tee-joint as shown in Figure 3-12. In this model, we incorporate two non-linear rotational springs whose characteristics are specified based on the moment-rotations relationship observed in the test data. The branch pipes connected to the either side of the flanges are supported by a hinge at the end to allow rotations at the end. The loads are applied as an axial load at the bottom of the Tee-joint for cyclic and monotonic tests, respectively, as shown in Figure 3-6 and Figure 3-13.

Figure 3-14 compares the moment-rotation curves at the location of rotational springs from finite element model to that obtained from the experimental tests for the monotonic load case. As seen in the figure, the two sets of curves are almost identical until the point of failure i.e. first leakage.

As we can see from Figure 3-11 and Table 3-1, the first leak failure rotations from monotonic and cyclic tests are significantly different. In order to evaluate the seismic fragility of the Tee-joint connection, it is necessary to use the finite element model that is validated with the more critical loading tests, i.e. the cyclic tests. Next, we obtain the nonlinear moment-rotation relationships of the two rotational springs used in the finite element model for the cyclic load case.

Detailed analysis of the moment-rotation curve observed indicates a gradual degradation of stiffness in all cyclic tests. In order to characterize the hysteretic behavior of the Tee-joint under cyclic loading, the "Pinching4" material model from the OpenSees Finite Element Package (Mazzoni et al, 2006) was used. The "Pinching4" material of OpenSees models a uniaxial material that represents a "pinched" load-deformation response and
exhibits degradation under cyclic loading. The strength and stiffness degradation properties are prescribed in the "Pinching4" model using a set of parameters denoting stiffness degradation, reloading stiffness degradation and strength degradation. Figure 3-15 shows a schematic definition of the OpenSees Pinching4 Uniaxial Material Model. Figure 3-16 compares the moment-rotation relationship of the left spring between Pinching4 model simulation with the experimental test under the cyclic loading given in Figure 3-6.

Next, we present the details of the piping system model. The representative piping system considered in this study is an actual fire sprinkler piping system taken from a hospital in California. The hospital piping system shown in Figure 3-17 consists of main piping runs along 4 sections with a total of 64 branches in all. The main runs of the piping system comprise of 2-inch and 4-inch diameter pipes whereas the branches comprise of pipes with diameters smaller than those of the main pipe runs. This system is supported by unbraced single hangers, transverse braced hangers and longitudinal braced hangers at various locations. The details of actual support locations are not provided in this paper for brevity. There are 4 anchors at the ends of the four main piping system runs. Since all the supports of the piping system are connected to the same ceiling, they all experience the same earthquake input. The natural frequencies for this piping system for the first 8 modes are given in Table 3-3. Ideally, the earthquake input exhibited by the piping through its supports is an amplified earthquake motion filtered through the building to which the piping is connected. Typically, the amplification is higher on higher floors and the filtering changes the characteristic of earthquake motion. The degree of amplification and nature of modification due to filtering in a time history is different for different buildings which cannot be generalized. Therefore in
this study, we consider free-field earthquake ground motions applied directly to piping supports. This situation is representative of a piping system in low rise buildings (less than 4 or 5 stories tall). The effect of building’s own characteristics and its interaction with the piping system introduces additional complexities which are not considered in this paper. The effects of these building characteristics on the piping fragility can be considered after the preliminary study presented in this paper is completed. Therefore, the results presented in this paper are quite valuable for piping systems in low rise buildings including hospitals, office spaces, hotels, schools, etc.

Nonstructural fragilities are functions that relate the probability that a nonstructural component will exceed a certain limit-state as a function of the Engineering Demand Parameter (EDP) (Bachman et. al., 2004). Equation (1) gives the expression for fragility at a Peak Ground Acceleration (PGA) level of $\lambda$.

$$P_f(\lambda) = P[\theta \geq \theta_{\text{lim}} \mid PGA = \lambda]$$  

(1)

The structural fragility is usually estimated empirically by conducting multiple nonlinear time history analyses of the structure for various ground motions

$$P_f(\lambda) = \frac{\sum_{i=1}^{N} 1(\theta_{i,\lambda} \geq \theta_{\text{lim}} \mid PGA = \lambda)}{N}$$  

(2)

In equation (2), $\theta_{i,\lambda}$ is the maximum rotation from $i^{th}$ earthquake time history analysis at a PGA level of $\lambda$ and $1(.)$ is the indicator function.

Next, we present a detailed discussion on the results for evaluating the system-level
5. RESULTS: SYSTEM FRAGILITY USING VALIDATED FE MODEL FOR MONOTONIC LOAD CASE

To begin with, we calculate fragility at the location of 2-inch Black Iron threaded Tee-joint(s) by incorporating the experimentally validated FE model for monotonic load case as discussed in section 4 above. Since monotonic load case gives moment-rotation curve for only loading path and does not provide the behavior for unloading path, we use the widely accepted practice in which the same moment-rotation curve is used for loading as well as unloading conditions.

5.1 FAILURE AT SINGLE LOCATION

In order to study the system-level fragility for the complete piping system, we begin with a simple case in which the moment-rotation non-linearity in the threaded Tee-joint connection is considered at only a single critical location in the complete piping system, i.e. a single location is considered for failure within the piping system. The complexity of multiple failure locations is introduced later and discussed in subsequent sections in this manuscript. Figure 3-18 shows the configuration of the system considered. In Figure 3-18, the main piping system from Figure 3-17 is modeled and the non-linear threaded Tee-joint connection
is a part of the 2 inch branch piping sprinkler system as shown. The particular location of the 2 inch branch piping sprinkler system shown in the figure is selected for detailed fragility evaluation based the results from a linear time history analysis of the complete piping, i.e. the critical location is determined as the point of maximum displacements and rotations from linear time history analyses on the main piping.

The first mode frequency of the complete piping system is 1.82 Hz. A total of 75 real earthquake records normalized to the same PGA, (Gupta and Choi, 2005) are chosen to model the uncertainty in ground motion. Table 3-4 list the earthquake records considered in this study. Non-linear analyses were conducted by normalizing each of the 75 earthquake records at many PGA levels from 0.5g to 3.0g at an interval of 0.1g and the fragility was estimated at each PGA level using Equation (2). The fragility curves corresponding to the three damage states specified in Table 3-2 are given in Figure 3-19. As seen in this figure, the piping system is relatively more fragile for the case of $\varphi = 2\theta$ and the fragility curve shifts to the right (less fragile) for the other two limit states of $\varphi = 2.5\theta$ and $\varphi = 3\theta$.

**5.2 FAILURE AT MULTIPLE LOCATIONS**

After calculating the preliminary fragility estimates for the case of failure at single location, we introduce the possibility of failure at two locations. The second location is also selected in a similar manner as before, i.e. the location of a 2-inch threaded Tee-joint that has the next largest displacements and rotations as observed from the linear analysis as well as from the non-linear analysis with a single non-linear Tee-joint. The purpose is to study
whether the fragility curves calculated in the previous section exhibit any change due to the presence of additional non-linearity in the piping system. A related objective is to evaluate the relative difference in the fragility curves for failure at the two locations. The two critical nonlinear Tee-joint locations where failure is considered to be possible in this study are shown in Figure 3-20. The same set of earthquake records are considered as before. The system fragility curves for the case of first-leakage limit state \( (\phi = 2\theta) \) at locations 1 and 2 are shown in Figure 3-21, respectively. It can be seen that the fragility curve for failure at Location 1 remains unchanged from that shown in Figure 3-21. This indicates that the presence of additional non-linearity at the second most critical location in the piping system does not have any meaningful effect on the fragility estimate for failure at Location 1. Furthermore, it can be seen that the probability of failure at location 1 is about 20 percent greater than the probability of failure at location 2 throughout the range of PGA values of interest.

Finally, we introduce additional complexity by considering the possibility of failure at a third location in the piping system. One may argue that it might be more productive to introduce non-linearity at all possible threaded Tee-joint locations. However, it must be noted that greater locations of non-linearity increases the computation time for a single time-history analysis significantly. The total computational effort needed to conduct 75 time histories at each level of PGA is non-trivial. Therefore, we consider only 3 locations for possible failure in the piping system as shown in Figure 3-22. The system-level fragilities for failure at all the three locations are shown in Figure 3-23. It can be seen that the fragility curves for failures at locations 1 and 2 remain unchanged. In fact, failure at location 3 is not initiated until PGA
value of 4.8g which is much larger than the PGA value of about 3g where the fragility curve for both locations 1 and 2 reached a probability of failure equal to unity. Therefore, it can be concluded that failure at a third or additional location does not necessarily need to be considered in the estimation of system-level piping fragility.

5.3 EFFECT OF STRENGTHENING AT LOCATION 1

Often, it is desired that the “most-probable” failure location identified in a fragility analysis be strengthened/retrofitted for improved seismic performance. The purpose of this task is to evaluate the effect of such strengthening on the changes in the fragility associated with failures at other possible locations. As shown in Figure 3-24, the piping system model with non-linearity at two locations is considered but modified such that the threaded Tee-joint connection at location 1 is assumed to remain linear. The system-level fragility at location 2 as shown in Figure 3-25 modified by strengthening at location 1 remains unchanged. The reason for this observation was investigated further. As shown Figure 3-26 the fundamental mode shape of the piping system does not excite the complete piping system but a localized region around the Tee-joint at location 1. Similarly, the response at location 2 is governed by a localized mode at that location which is the second mode of the piping system as shown in Figure 3-27. Therefore, the response at a particular location in a piping system is typically governed by a local mode and not a global mode shape and hence the observation that strengthening at location 1 does not alter the fragility associated with failure at location 2.
6. SEISMIC FRAGILITY OF THE THREADED TEE-JOINT AT LOCATION-1 BASED ON CYCLIC FE MODEL

Next, we consider the non-linear FE model of the Tee-joint for which the moment rotation curve is reconciled with the results obtained from the cyclic tests, i.e. using the nonlinear model described earlier in section 4. The purpose is to evaluate the change, if any, in fragility curves due to consideration of validated FE models based on monotonic test versus those based on cyclic test. No other change was made and the same input earthquake records are considered. The system-level fragility curves corresponding to the three damage states at location 1 are given Figure 3-28. Furthermore, Figure 3-29 compares the fragility curve for “first-leakage” case obtained from this analysis with that obtained from the previous analysis for the monotonic test case. It can be seen that the fragility curves evaluated using FE model validated using the monotonic tests give higher fragility estimates. These differences can have a meaningful impact on the design of a piping system. The primary reason for this difference is the degree of stiffness degradation and energy dissipation in each cycle for the two different cases of moment-rotation curves. It may be argued that an FE model validated using cyclic test results gives more realistic estimates of fragility curves. For the case of multiple failure locations, the fragility curves are given in Figure 3-30. The natures of differences are similar to those observed in earlier case. The fragilities at both locations are much lower in the case of FE model validated for cyclic test data when compared to the corresponding fragilities at the two locations in the case of FE model validated for monotonic test data as is given in Figure 3-21.
7. SEISMIC FRAGILITY OF THE DECOUPLED THREADED TEE-JOINT SYSTEM AT LOCATION 1 BASED ON CYCLIC FE MODEL

It can be seen that the evaluation of the fragility curves in the proposed framework requires significant computational resources and time. Often, engineers simplify the process by decoupling the branch from the main piping and evaluating the fragility of the branch system by a separate analysis using the displacement time histories at the points of connection. Such a procedure is justified if there is no interaction between the relatively heavy main piping and the relatively light branch piping. In this section, we examine the effects of decoupling the branch piping on the fragility estimates of threaded Tee-joint connection. The fragility in the decoupled case is evaluated as follows:

**Step 1:** Analyze the main piping by linear time history analysis using the 75 earthquakes all normalized to unit PGA. Record the displacement time-history for each of the 75 analyses at the point A where the branch piping system is connected to the main piping system.

**Step 2:** Scale each linear displacement time histories appropriately and apply these displacement histories to the finite element model of the Tee-joint shown in Figure 4. Evaluate the maximum rotations in each case.

**Step 3:** Evaluate the fragility at each PGA level using Equation (2).

The fragility estimates corresponding to the “first-leak” damage state from the coupled and the decoupled cases are compared in Figure 3-31. As can be seen in this figure, the fragility estimates are significantly different in the two cases and that the decoupled
analysis yields highly conservative fragility values i.e. predicts the system to be more fragile. The primary reason for the differences in the two curves is that there exists a mass interaction and tuning between the main and branch piping systems. Ignoring these effects, leads to an incorrect estimation of the fragility.

8. SUMMARY AND CONCLUSIONS

The cost of damage to the non-structural systems and components can exceed 80% of the total damages during an earthquake. Many existing studies and ATC-58 (2004) highlight the need for performance based design and probabilistic approaches to mitigate risk and to achieve reliable designs. This paper focuses on evaluating the structural seismic fragilities of threaded T-joint connection in sprinkler piping systems.

Often, threaded connections in piping tend to exhibit complex nonlinear behavior under dynamic loading. Simple laboratory tests of the threaded connections under monotonic and cyclic loading can give us insights into the behavior and the typical damage patterns. Test data can be used to define the limit states of damage as well as obtain nonlinear force-displacement and moment-rotation relationships at locations around the connection. This can be used to create a relatively simple non-linear finite element model for seismic fragility evaluations.

The University at Buffalo, SUNY conducted laboratory tests on various Tee-joint components of sprinkler piping systems. The tests indicate that the "First Leak" damage state is observed predominantly due to excessive flexural deformations at the Tee-joint section. We
use the monotonic and cyclic loading test data to characterize the limit state corresponding to “First Leak” for a 2 inch black iron threaded Tee-joint connection and to develop a simple nonlinear finite element model describing the Tee-joint behavior.

The nonlinear FE model developed is then used in conjunction with the main line of a typical hospital floor piping to obtain system-level fragilities corresponding to the first leak damage state. A total of 75 real normalized earthquake records are used to describe the uncertainty in ground motion histories. In addition, the system fragility is evaluated by considering two different types of threaded Tee-joint connections i.e. monotonic FE model and cyclic FE model. The fragility curves are generated for failure at a single critical location as well as multiple possible failure locations. The seismic fragility curves of the Tee-joint system at each location are significantly different for the two different FE models considered. Consequently, it is suggested that the threaded Tee-joint connection based on cyclic FE model be used to evaluate the system-level fragility at each critical location.

It is also illustrated that the system fragilities corresponding to the same limit state is significantly different when evaluated from a decoupled analysis of just the Tee-joint assembly component. Therefore, mass interaction and tuning between the main line piping and the branch piping can have a considerable effect on seismic fragilities.
ACKNOWLEDGEMENTS

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Table 3-1: Maximum Rotation(s) and Moment(s) at “First Leak” in Cyclic Tests

<table>
<thead>
<tr>
<th>Cyclic Test #</th>
<th>Rotaion (rad)</th>
<th>Moment (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0171</td>
<td>22,192</td>
</tr>
<tr>
<td>2</td>
<td>0.0152</td>
<td>24,646</td>
</tr>
<tr>
<td>3</td>
<td>0.0142</td>
<td>24,347</td>
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</table>

Table 3-2: Multiple Damage States Considered for Fragility Assessment

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Definition</th>
<th>$\theta$ (radians)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor Damage (First Leak)</td>
<td>$\varphi = 2\theta$</td>
<td>0.0135</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>$\varphi = 2.5\theta$</td>
<td>0.0175</td>
</tr>
<tr>
<td>Severe Damage</td>
<td>$\varphi = 3\theta$</td>
<td>0.0217</td>
</tr>
</tbody>
</table>
Table 3-3: The Natural Frequencies for Hospital Piping System with 64 Branch Systems

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Mode</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.65</td>
<td>5</td>
<td>1.53</td>
</tr>
<tr>
<td>2</td>
<td>0.84</td>
<td>6</td>
<td>1.71</td>
</tr>
<tr>
<td>3</td>
<td>1.28</td>
<td>7</td>
<td>2.08</td>
</tr>
<tr>
<td>4</td>
<td>1.52</td>
<td>8</td>
<td>2.082</td>
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Table 3-4: 75 Selected Seismic Events

<table>
<thead>
<tr>
<th>No.</th>
<th>Date</th>
<th>Seismic Events, Location</th>
<th>Component</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>May 18 1940</td>
<td>Imperial Valley El-Centro</td>
<td>S00E S90W</td>
<td>0.89</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>0.55</td>
</tr>
<tr>
<td>3</td>
<td>July 21 1952</td>
<td>Kern County, Pasadena, Caltech-Athenaeum</td>
<td>S00E S90W</td>
<td>0.12</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>0.14</td>
</tr>
<tr>
<td>5</td>
<td>July 21 1952</td>
<td>Kern County, Taft, Lincoln School Tunnel</td>
<td>N21E S69E</td>
<td>0.40</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>0.46</td>
</tr>
<tr>
<td>7</td>
<td>July 21 1952</td>
<td>Kern County, Santa Barbara Court House</td>
<td>N42E S48E</td>
<td>0.23</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td>0.33</td>
</tr>
<tr>
<td>9</td>
<td>July 21 1952</td>
<td>Kern County, Hollywood Storage, Basement</td>
<td>S00W N90E</td>
<td>0.14</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td>0.11</td>
</tr>
<tr>
<td>11</td>
<td>March 22 1957</td>
<td>San Francisco Golden Gate Park</td>
<td>N10E S80E</td>
<td>0.21</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td>0.27</td>
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<tr>
<td>13</td>
<td>March 10 1933</td>
<td>Long Beach Vernon CMD Building</td>
<td>S08W N82W</td>
<td>0.34</td>
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<tr>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td>0.39</td>
</tr>
<tr>
<td>15</td>
<td>December 30 1934</td>
<td>Lower California, El-Centro Imperial Valley</td>
<td>S00W S90W</td>
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</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td>0.47</td>
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<tr>
<td>17</td>
<td>October 31 1935</td>
<td>Helena, Montana Carrol Collage</td>
<td>S00W S90W</td>
<td>0.37</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td>0.37</td>
</tr>
<tr>
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<td>April 13 1949</td>
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PART IV

PIPING FRAGILITY EVALUATION: INTERACTION WITH BUILDING PERFORMANCE

Bu Seog Ju and Abhinav Gupta
ABSTRACT

Many recent studies have emphasized the need for improving seismic performance of nonstructural systems in critical facilities in order to reduce the damage as well as to maintain continued operation of the facility after an earthquake. This paper is focused on evaluating system-level seismic fragility of the piping in a representative high rise and a low rise building. Piping fragilities are evaluated by incorporating the nonlinear finite element model of a threaded Tee-joint that is validated using experimental results. The emphasis in this study is on evaluating the effects of building performance on the piping fragility. The differences in piping fragility due to the nonlinearities in building are evaluated by comparing the fragility curves for linear frame and nonlinear fiber models. It is observed that as nonlinearity in the building increases with increasing value of peak ground acceleration, the floor accelerations exhibit a reduction due to degradation/softening. Consequently, the probabilities of failure increase at a slower rate relative to that in a linear fiber. It is also observed that a piping located at higher floor does not necessarily exhibits high fragilities, i.e. the fundamental building mode is not always the governing mode. Higher order building modes with frequencies closest to critical piping modes of interest contribute more significantly to the piping fragility. Within a particular building mode of interest, a good
indicator of the amplification at different floor levels can be obtained by the product of mode shape ordinate and modal participation factor. Piping fragilities are likely to be higher at floor levels at which this product has a higher value.

1. INTRODUCTION

Observations from earthquake related failures have emphasized the significance of improving the seismic performance of not only the primary structures such as buildings and bridges but also of the nonstructural secondary systems such as equipment and piping. In many cases, these secondary nonstructural systems do not pose a direct threat to life safety but these systems must remain functional to provide protection against secondary threats such as fires especially in critical facilities like hospital buildings, nuclear power plants, petrochemical and other industrial facilities. Over the past 2 decades or so, failure of nonstructural components has resulted in significant economic loss and closure of critical facilities even though the primary structural components and buildings did not exhibit any damage. For example, during the 1994 Northridge earthquake, the Olive View Medical Center had suffered no structural damage but the hospital was shut down due to the water leakage from fire protection sprinkler piping system and chilled water piping system (Reitherman et al. 1995). In terms of the economic impact, the construction costs associated with the nonstructural systems/components can be as high as $103 million out of the $170 million total construction cost of a hospital, approximately 60% of the total construction cost (Horne and Burton, 2003).
In recent years, some studies have been conducted to evaluate and improve the seismic performance of nonstructural components such as piping systems, mechanical equipment, and HVAC systems. Shinozuka and Masri (2003) conducted seismic risk assessments of water systems for fire suppression and electrical power systems in hospital buildings. The focus of this analysis was to estimate the annual probability of failure subjected to the earthquake ground motions and to provide optimal strengthening or retrofitting by integrating nonstructural component fragility enhancement and seismic response of structural system. Miranda and Taghavi (2003) evaluated seismic demands on acceleration sensitive rigid and flexible secondary systems in critical facilities. In particular, this study investigated that frequency content of earthquake ground motions and dynamic characteristics of the primary structural systems such as the fundamental period which can influence the acceleration demands along the building height significantly.

The dynamic characteristics of building structural systems and nonstructural components/contents such as piping systems can be quite different. Most of the building response can be contained in the fundamental mode whereas the response of piping systems is highly dependent upon higher order modes because piping modes are typically local modes, i.e. a fundamental mode which is local to a certain part of the piping system may have no influence on the performance/response at a location far from this localized zone.

The objective of this research is to develop the system-level fragility of piping systems in critical facilities by considering the effect of nonlinearities in the buildings on which the piping systems are supported. The purpose is to evaluate the effects of filtering the earthquake motion through a typical high rise building and through a typical low-rise
building on the piping fragility curve. Piping fragilities are evaluated for buildings that remain linear as well as for buildings that exhibit significant nonlinear behavior. A related objective is to evaluate the changes in fragility curve for piping located at different elevations of the buildings. The piping system considered for the purpose of this study is same as that described in detail in Part-III of this dissertation.

2. DESCRIPTION OF BUILDINGS CONSIDERED

For the purpose of this study, we consider a 20 story building that is representative of a typical high rise building and a 5 story building that is representative of the low rise building. Figure 4-1 shows the single bay idealized 5 and 20 story reinforced concrete moment resisting frame building configurations. These buildings are based on the building design details provided in Wood et al. (2009). Both buildings are assumed to have same footprint of 150 ft by 120 ft. A total of 5 bays exist in each direction such that the width of longitudinal bay is 360 in, that of transverse bay is 288 in, and that of story height is 144 in. Both the buildings are assumed to be fixed at their base. The analysis is conducted only for the longitudinal direction of the prototype building. For the simplicity in analysis, only a single bay is analyzed. A live load of 0.348 psi is assumed, which is in accordance with ICC (2006). In addition to the self-weight of the members and a 0.95 in thick two-way floor slab, a superimposed dead load of 0.1389 psi is also considered. The reinforced concrete considered for the design of this building is conventional concrete with a 28-day unconfined compressive strength of 5 ksi for the beams and a range of 5 – 10 ksi for the columns. The
concrete weight is assumed to be 1.0 psi. Reinforcing steel is taken as Grade 60 with a design yield tensile strength of 60 ksi.

According to Wood et al. (2006), the design of the buildings is governed by IBC (ICC, 2006) and ACI 318-08 (ACI, 2008). The IBC 2006 design code is used to evaluate estimates of the base shear, period, and lateral force distribution. The general concrete design and detailing is done as per ACI 318-08 with particular emphasis on Chapter 21. The strong column-weak beam philosophy is adopted for the frame design, i.e. the sum of the moment capacity at the columns is designed to be at least 20% greater than the sum of the moment capacity at the beams. Both the building designs are governed by seismic loading. For simplicity in the detailing of the beams and columns, the reinforcing steel is taken to be in one layer. The shear reinforcement is provided using the double leg #5 stirrups. The confinement spacing resulted in shear reinforcement providing a minimum lateral pressure of 9% of the target \( f'_{c} \) (Englekirk, 2003) within the assumed plastic hinge zone. The compression reinforcement is considered to be equal to the tension reinforcement. A single longitudinal reinforcement layer is considered on each side for column detailing. Table 4-1 provides the design details of the various beams and columns in the buildings. The range of longitudinal reinforcing steel ratios \( \rho \) for the columns and beams is 1.1 – 2.7%. Additional building design details can be found in Wood et al. (2009).

### 2.1 NUMERICAL MODELING OF NONLINEAR FIBER MODEL

Numerical simulation of the nonlinear fiber models is performed using the OpenSees
(2010) platform. As described in Wood et al. (2009), two dimensional lumped mass idealizations are developed and considered for the prototype buildings. The damping ratio for all the modes is taken as 5%. Several material models such as concrete and steel are available in OpenSees platform and various types of element models for linear and nonlinear analyses can be also used. In this study, to develop the fiber section, concrete02 material (Figure 4-2) is adopted as a uniaxial model with tensile strength and linear tension softening. For steel section, steel02 model (Figure 4-3) named as Giuffre-Menegotto-Pinto steel material with isotropic strain hardening is used (Mazzoni et al. 2006). The building models are discretized using the BeamWithHinges element (Scott and Fenves, 2006). This element is selected as it can be integrated with nonlinear fiber sectional discretization and has demonstrated good performance for members anticipated to undergo nonlinearity as well as softening or degradation (Scott and Fenves, 2006). The BeamWithHinges element also eliminates the nonobjective curvature response due to its sensitivity to the number of integration points (Coleman and Spacone, 2001). The element is developed as a force-based, lumped plasticity, zero-volume line element with two different sections, namely, a fiber section at each end, which represents the plastic hinge over a discrete length $L_p$ and an interior linear elastic section (Melo et al. 2011) as shown in Figure 4-4. In the development of the fiber section in the end regions, the linear tension strength concrete (concrete02 model) and the Menegotto-Pinto model (steel02 model) are used. The effects of confinement are considered using the model of Mander et al. (1988). The plastic hinge lengths ($L_p$) of the beam and column elements are specified using the Paulay and Priestley (1992) analytical model.
2.2 NUMERICAL MODELING OF LINEAR FRAME MODEL

For comparing the effect of building nonlinearity on piping fragility with the fragility curves evaluated in the corresponding building that remains linear, we also consider a modified design of the building described above. To do so, the numerical model discretization for the fiber sections of linear frame models is designed with an elastic uniaxial material based on nonlinear fiber sections. The elasticBeamColumn element in OpenSees is used to characterize the linear response of the frame sections for beams and columns under earthquake ground excitations. The damping in the linear frame model, however, is considered to be same as that in the nonlinear fiber model, i.e. 5% modal damping ratio in all the modes.

3. PIPING SYSTEM LAYOUT

The piping system, shown in Figure 4-5, is same as that considered and described earlier in the Part III of this dissertation. The piping system, originally designed as per NFPA-13(2007) and SMACNA (2003), is selected from a hospital building in California. This piping system was made up of 2, 3, and 4 inch diameter schedule 40 black iron pipe and the system is braced by transverse/longitudinal hangers and anchors at the ends of the pipes. Additional piping system design detail can found in the manuscript given in Part- III of this dissertation.
4. MODAL CHARACTERISTICS OF UNCOUPLED BUILDING AND PIPING SYSTEMS

Table 4-2 gives the natural frequencies and mass participation in the first 5 modes of the 20 story nonlinear and linear frame models whereas Table 4-3 gives the corresponding values for the 5 story nonlinear and linear frame models. It can be seen that the first 4 modes of each building model account for more than 90% of the total mass participation. The fundamental natural frequencies for the 20-story and the 5-story linear frame models are 0.53Hz are 2.62Hz, respectively. The frequencies for the 20 and 5 story nonlinear fiber models are relatively less, i.e. 0.48Hz and 2.42Hz, respectively. Table 4-4 gives the natural frequencies for the first 6 modes of the piping systems. It can be seen that the frequencies of the first and the second modes in the piping system are 1.82Hz to 3.142Hz, respectively. Therefore, the fundamental piping system mode is close to, and therefore interacts most with, the second mode of the 20 story building models. Similarly, the second piping system mode interacts most with either the third or the fourth modes of the 20 story building models. Since the 5 story building models are much stiffer than the 20 story building models, the fundamental mode of the 5 story building model interacts with both the fundamental and the second mode of the piping system.

Figure 4-6 to 4-7 shows the first four mode shapes of 5-story and Figure 4-8 to 4-9 that of the 20-story linear and nonlinear fiber models, respectively. The mode shapes for the first 2 modes of the piping system are shown in Figure 4-10. As can be seen in this figure, the fundamental piping mode is a local mode that is located far away from the location of the 2-
inch diameter branch piping with the threaded Tee-joint which is the location of interest for evaluating piping fragility. However, the second piping mode shape shown in Figure 8 occurs in the region of the 2-inch diameter branch piping. Therefore, the second piping mode and its interaction with the corresponding building modes are expected to contribute significantly to the piping fragility.

5. FINITE ELEMENT MODEL OF THE THREADED TEE-JOINT

As discussed in detail in the manuscript presented in Part-III of this thesis, the system-level fragility for this piping is determined by incorporating experimentally validated finite element model of the nonlinear behavior at the threaded Tee-joint within the complete piping system model. As explained in Part-III, the 2 inch diameter black iron pipe threaded Tee-joint is modeled in OpenSees platform by using two nonlinear rotational springs. The moment-rotation curves for the nonlinear springs are obtained based on the component level experiments conducted on several of this threaded Tee-joint at University at Buffalo (Dow, 2010). The equivalent rotational spring model used to model this behavior is shown in Figure 4-11. The Pinching4 uniaxial material model available in OpenSees is used to model the moment-rotation behavior for the Tee-joint. This model is capable of considering unloading/reloading, stiffness degradation, and strength degradation under cyclic loading. Figure 4-12 compares the simulation using the Pinching4 model with the experimentally evaluated moment-rotation curve. As can be seen from this figure, the Pinching4 model is adequate to represent the experimentally evaluated behavior.
6. SEISMIC FRAGILITY EVALUATION

Seismic fragility of structural and nonstructural systems is the conditional probability of its failure for a given level of engineering design parameter such as Peak Ground Acceleration (PGA) (Sundararajan, 1995). Seismic fragility can be described by means of a family of fragility curves reflecting the uncertainty in the parameter values and in the models. The fragility curves are developed on the premise that the component could fail in any of the potential failure modes (Sundararajan, 1995).

The first step in the evaluation of seismic fragility curves is to characterize the limit state for the purpose of defining failure associated with critical components. In this study, the limit state for the “first-leakage” of the 2 inch black iron piping system is characterized based on the alternative form of the ASME’s “Twice the Elastic Slope” criteria (ASME Section III BVP & P Code, Gerdeen, 1979) as described in detail in the Part-III of this dissertation. Based on this criterion, the limiting rotation characterizing the “first-leakage” would correspond to 0.0135 radians, i.e. \( \theta_{\text{lim}} = 0.0135 \). The fragility values for a given value of the engineering demand parameter \( \lambda \) can be calculated as per Equation (1) given below.

\[
P_f(\lambda) = P[\theta \geq \theta_{\text{lim}} \mid PGA = \lambda]
\]  

(3)

In a simulation based approach, multiple earthquake records normalized to the same value of engineering demand parameter are used to conduct the nonlinear time history analysis of the complete system. The Conditional probability of failure \( P_f \) is modeled as a function of inelastic rotation as
\[
P_J(\lambda) = \frac{\sum_{i=1}^{N} I(\theta_{i,\lambda} \geq \theta_{\text{lim}} | \text{PGA} = \lambda)}{N}
\]

Where,

\( \theta \) = Plastic functional failures (Rotations at Tee-joint)

\( \theta_{\text{lim}} \) = Limit state capacity of Tee-joint piping system (0.0135 radians)

\( \lambda \) = Scale factor of Peak Ground Acceleration

\( N \) = Number of earthquake records

In order to evaluate seismic fragility of piping system in this study, a total of 22 earthquake records are considered. These ground motions are taken from PEER-NGA (2009). These records are selected such that an uncertainty in the nature of earthquakes based on their fault mechanism is accounted for in these ground motions. The records correspond to earthquakes resulting from a variety of fault mechanisms such as normal, reverse, and strike-slip fault. Furthermore, the magnitude of selected earthquakes is greater than 6.0. The seismic events that contribute to these earthquakes are geographically distributed, i.e. earthquakes that occurred in the North and South America, Asia, Middle East, and Europe. Table 4-5 gives the detailed characteristics of the ground motions considered in this study. Figure 4-13 to 4-17 gives the spectra for each of the 22 earthquake ground motions as well as the mean spectrum curve.
7. PIPING FRAGILITY EVALUATION: INTERACTION WITH BUILDING PERFORMANCE

7.1 GENERATION OF FLOOR ACCELERATION TIME HISTORIES

In order to consider the effect of building performance on the piping fragility, we conducted time history analyses of each building model for all individual 22 ground motion records and evaluated acceleration time histories at various floors. To do so, the ground motions are first normalized to a given value of PGA. Therefore, 22 time history analyses of the building models are needed for each normalized value of PGA considered for generating the fragility curve, i.e. for each value of $\lambda$ in Equation (2).

In 20-story building model, the floor accelerations time histories are obtained from the ground motion records normalized to the same PGA levels at 5th, 10th, 15th, and 20th floors, respectively. For the 5-story building system, the floor acceleration time histories are evaluated at each of the 5 floors. A noticeable uncertainty in the process of generating floor motions is related to the fact that the range of PGA in which the piping fragility curve goes from zero to unity is not known a-priori. For example, let us consider that the ground time histories are normalized to a PGA value of 0.3g and the corresponding floor time histories are evaluated by nonlinear analyses of the building model. The floor time histories are then used to conduct nonlinear analyses of the piping system. Once all these analyses are completed, one may find that none of the time histories resulted in a piping failure even if significant nonlinearity existed in the building system model. Consequently, all these
analyses would not result in generating even a single point on the fragility curve. Therefore, the complete process of generating the piping fragility curve by considering building performance can be computationally quite intensive. In this study, we used a preliminary iterative procedure to determine that a variation of PGA from 0.2g to 3.0g is sufficient to evaluate the complete fragility curve for the piping system in the various building models. In order to better understand the effect of building performance on the piping system fragility, we evaluated floor spectra for each case. For brevity, we have included the mean spectra at various floor levels for only a single case of normalization equal to 1.0g, i.e. the case for which each ground motion record is normalized to a PGA of 1.0g. Figure 4-18 gives the mean spectra at various floor levels of the nonlinear 20 story building model including the mean spectra at the ground level. Similarly, Figure 4-19 gives the spectra for the linear 20 story building, Figure 4-20 for the nonlinear 5 story building, and Figure 4-21 for the linear 5 story building, respectively. Each of these figures contains two vertical lines. These lines correspond to the frequencies of the first two piping system modes. Before proceeding further with the evaluation of piping fragility, we examine the degree of nonlinearity in the building models. This is particularly important because we consider both the linear and the nonlinear fiber models to emphasize the effect of building nonlinearity. Figures 4-22 to 4-30 give selected moment-rotation curves for at three different floor levels, 1st, 10th, and 20th floors, in the 20 story nonlinear fiber model. Figure 4-31 gives the story drift ratios for all the earthquake records. As seen in this figure, the building model exceeds the limiting story drift of 0.02 (UBC, 1997) for only 4 of the 22 earthquakes considered.
7.2 PIPING FRAGILITY FOR 20-STORY BUILDING MODELS

The piping fragility for “first-leakage” at the threaded Tee-joint location in the 2 inch black iron pipe is shown in Figure 4-32 for the nonlinear fiber model. As seen in this figure, fragility curves are presented at different floor levels and also compared to the fragility curve evaluated by directly using the ground motions which is representative of the piping fragility curve evaluated without considering the effect of building performance, i.e. at the ground level. As seen in this figure, the piping fragility is significantly higher due to dynamic interaction with the building. However, it must be noted that the piping fragility does not increase with increasing floor level. Even though the relative value of piping fragility are quite high for a system located on the 20th floor, the piping fragilities for a system located on 15th floor are about the smallest among the four floor levels considered in this study.

Observations similar to those noted above can be made for the fragility curves in the case of the 20-story linear frame model. Figure 4-33 gives the piping fragility curves at different floor levels for the 20-story linear frame model case and also compare them to those for the ground level case. Once again, the piping fragilities do not necessarily increase with an increase in the floor elevation.

At first, the observation that the fragility at 15th floor is about the least among all the floor levels considered appears intriguing. However, it can be explained as follows. While much of the building performance is governed by the fundamental building mode, the piping performance is governed by the tuning of the building and the piping modes. As observed by the values of frequencies for the building models and the piping models, the fundamental
mode of the piping system is nearly tuned to the second mode of both the nonlinear and the linear 20-story building models. Consequently, the relative contribution of the second building mode is much greater than that of the fundamental building mode even though the mass participation of the fundamental building mode is much greater. Once the significance of the second building mode is established, the differences in the fragilities at various floor levels can be explained by examining the product of the mode shape ordinate ($\phi$) and the modal participation factor ($\gamma$) at each floor in the second mode. In the case of nonlinear fiber model, the product of gamma ($\gamma$) phi ($\phi$) is equal to 0.6470 at 20th floor, 0.0159 at 15th floor, 0.4654 at the 10th floor, and 0.3685 at the 5th floor, respectively. It is quite evident that the amplification at 15th floor is about the smallest among the four floor levels and, therefore, the piping fragilities at the 15th floor are relatively less. Similarly, the products of gamma($\gamma$) and phi($\phi$) in the case of linear 20 story building are -0.6298, -0.0030, 0.4673, and 0.3830 at the 20th, 15th, 10th, and 5th floor levels, respectively, indicating a similar trend. It must be noted that this observation is quite specific to the 20 story building model considered in this study and should not be considered as a generic behavior.

Next, the piping fragility curves for the cases of linear frame and nonlinear fiber models are compared in Figures 4-34 to 4-37 for various floor levels. As seen in these figures, the piping fragilities are relatively higher in the case of linear frame models at the 20th floor and the 5th floor. In general, the piping fragility in a linear frame model can be relatively higher because the frequencies of the nonlinear fiber model modes are smaller than the corresponding values for the linear frame model modes due to the softening/degradation of
beams and columns. Consequently, the linear frame model modes have greater degree of tuning with the piping system modes thereby leading to higher piping fragilities. It must be noted that it is quite possible in some cases for a nonlinearity in the building to increase the degree of tuning with the piping system modes of interest which in turn would lead to higher fragilities for piping system. In contrast to the observations made for the 20th and the 5th floors, the piping fragilities at the 10th and the 15th floors in a linear frame model are relatively lower in certain regions especially at lower PGA values. The primary reason for such an observation is related to the degree of degradation at lower PGA values. As the PGA values increase, the degree of degradation in both the building and the piping increases. Therefore, the degree of tuning between the two individual systems changes continually at different PGA levels and influences the evolution of the fragility curve.

7.3 PIPING FRAGILITY FOR 5-STORY BUILDING MODELS

Figures 4-38 and 4-39 give the piping fragility curves in 5-story linear frame and nonlinear fiber models, respectively. The fragility curves for the piping system are developed at each floor in the 5-story linear frame and nonlinear fiber model. As seen in this figure, the piping fragility increases with increasing floor level. This is so because the 5-story linear frame and nonlinear fiber models are relatively stiffer with a natural frequency of the fundamental mode equal to 2.63Hz and 2.42Hz, respectively. Therefore, both the first and the second mode of the piping system interact with the fundamental building mode in each case. The amplification in the fundamental building mode increases with an increase in the floor
elevation. This can be seen in the values of the product of the mode shape ordinate ($\phi$) and the modal participation factor ($\gamma$) at each floor level in the fundamental mode. The values of this product are equal to 0.1722, 0.5005, 0.8372, 1.1140, and 1.3074 at the 1st, 2nd, 3rd, 4th, and 5th floor of the linear frame model, respectively. The corresponding values for the nonlinear fiber model are 0.1721, 0.4982, 0.8345, 1.1129, and 1.3093, respectively.

8. CONCLUSIONS

This paper presents the results of a detailed study on the evaluation of piping system fragility curves by incorporating the nonlinear moment-rotation characteristics of a 2-inch threaded Tee-joint of a black iron pipe in the system level piping model. The particular focus is on the evaluating the effect of interaction between the building and the piping on the piping fragility. The effect of building behavior is included by considering low rise 5-story and high rise 20-story building models. For each of the two building models, piping fragilities are evaluated by considering the building to exhibit significant nonlinearity in the beams and columns as well as by considering the buildings to remain linear for each of the 22 real ground motion records. In order to consider the effect of nonlinearity in building model, the ground motion records are taken from relatively larger magnitude earthquakes (greater than 6) that are recorded in different parts of the world. It is observed that the fundamental building mode is not necessarily critical contributing mode in the evaluation of piping fragilities. Same is true for the fundamental piping system mode. Since piping system modes are local in nature, the piping mode that causes excitation around the location of the
critical component such as the threaded Tee-joints governs the piping performance. Furthermore, the building mode with frequency closest to the governing piping system mode influences the piping system fragility more critically. It is also observed that the piping system fragilities can be higher in a linear frame model whose modes have greater degree of tuning with the piping system modes. However, in some cases the piping system fragility can be higher for the nonlinear fiber models if the nonlinearity increases the degree of tuning between the piping and the building modes. The piping fragility does not necessarily increases with the increasing floor levels in a high rise building. The product of modal participation factor and the mode shape ordinate in the most critical building mode gives a good estimate of the relative amplification at different floor levels and consequently the relative increase or decrease in the piping fragility. Finally, it must be noted that as the degree of degradation in a nonlinear fiber model increases with increasing values of the peak ground accelerations, the effect of tuning between the building and the piping modes can either increase or decrease. Such a variation in the degree of tuning leads to a corresponding change in the piping fragility.
ACKNOWLEDGEMENTS

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Components in Hospitals,” *Proceedings of Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities*, ATC-29-2, Newport Beach, California


Table 4-1: Description of Building Design (Woods et al. 2009)

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Beam</th>
<th>Column</th>
<th>Column</th>
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<tr>
<td></td>
<td>b (in)</td>
<td>h (in)</td>
<td>Strength $f_c$’(ksi)</td>
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<tr>
<td>Level 01 to Level 10</td>
<td>32.00</td>
<td>32.00</td>
<td>5.00</td>
</tr>
<tr>
<td>Level 11 to Level 15</td>
<td>30.00</td>
<td>30.00</td>
<td>5.00</td>
</tr>
<tr>
<td>Level 16 to Level 20</td>
<td>24.00</td>
<td>28.00</td>
<td>5.00</td>
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Table 4-2: Frequencies and Mass Participation in 20-Story Building Models

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<th>Nonlinear Fiber Model</th>
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<td>Frequency (Hz)</td>
<td>Mass Participation (%)</td>
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Table 4-3: Frequencies and Mass Participation in 5-Story Building Models

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<td>40.17</td>
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<td>Mode</td>
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<td>----------------</td>
<td></td>
</tr>
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Table 4-5: Characteristics of Ground Motion Records Used

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<th>Magnitude (Mw)</th>
<th>PGA (g)</th>
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</tr>
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<td>Canyon Country, USA</td>
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<td>Bolu, Turkey</td>
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<td>0.8224</td>
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<td>Hector</td>
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<td>Delta</td>
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<td>PGA (g)</td>
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Figure 4-1: RC Moment Frame Building Configurations
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Fundamental Mode Shape

Second Mode Shape
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Figure 4-15: Response Spectra for the Input Ground Motions Selected
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Figure 4-24: Moment-Rotation Curves at the 1st Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
Figure 4-25: Moment-Rotation Curves at the 10th Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
Figure 4-26: Moment-Rotation Curves at the 10th Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
Figure 4-27: Moment-Rotation Curves at the 10th Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
Figure 4-28: Moment-Rotation Curves at the 20th Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
Figure 4-29: Moment-Rotation Curves at the 20th Floor in 20-Story Nonlinear Building Model, PGA Normalized to 1.0g
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Figure 4-37: Seismic Fragilities of Piping System at the 5th Floor in 20-Story Linear and Nonlinear Building Models
Figure 4-38: Seismic Fragilities of Piping System in 5-Story Linear Building Model
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PART V

SUMMARY AND CONCLUSIONS
1. SUMMARY AND CONCLUSIONS

This dissertation presents the results of a detailed study that focuses on the evaluation of system level fragilities in the piping system with threaded Tee-joints. The two main aspects of this study focus on (a) developing a framework to incorporate the lessons learned from component level testing of threaded Tee-joint into a system level piping analysis, and (b) the effect of building performance on the piping fragilities. The summary and main conclusions for each part are given below.

(i) FRAGILITY OF PIPING SYSTEMS WITH THREADED TEE-JOINTS

The cost of damage to the non-structural systems and components can exceed 80% of the total damages during an earthquake. Many existing studies and ATC-58 (2004) highlight the need for performance based design and probabilistic approaches to mitigate risk and to achieve reliable designs. This study focuses on evaluating the seismic fragilities of threaded T-joint connection in sprinkler piping systems.

Often, threaded connections in piping tend to exhibit complex nonlinear behavior under dynamic loading. Simple laboratory tests of the threaded connections under monotonic and cyclic loading can give us insights into the behavior and the typical performance characteristics. Test data can be used to characterize the limit states as well as obtain nonlinear force-displacement and moment-rotation relationships at locations around the connection. This can be used to create a relatively simple non-linear finite element model for
seismic fragility evaluations.

The University at Buffalo, SUNY conducted laboratory tests on various threaded Tee-joint components of sprinkler piping systems. The tests indicate that the "First Leak" damage state is observed predominantly due to excessive flexural deformations at the Tee-joint section. We use the monotonic and cyclic loading test data to characterize the limit state corresponding to “First Leak” for a 2 inch black iron threaded Tee-joint connection and to develop a simple nonlinear finite element model describing the Tee-joint behavior.

The nonlinear FE model developed is then used in conjunction with the main line of a typical hospital floor piping to obtain system-level fragilities corresponding to the first leak damage state. A total of 75 real normalized earthquake records are used to describe the uncertainty in ground motion histories. In addition, the system fragility is evaluated by considering two different types of threaded Tee-joint connections i.e. monotonic FE model and cyclic FE model. The fragility curves are generated for failure at a single critical location as well as multiple possible failure locations. The seismic fragility curves of the Tee-joint system at each location are significantly different for the two different FE models considered. Consequently, it is suggested that the threaded Tee-joint connection based on cyclic FE model be used to evaluate the system-level fragility at each critical location.

It is also illustrated that the system fragilities corresponding to the same limit state is significantly different when evaluated from a decoupled analysis of just the Tee-joint assembly component. Therefore, interaction and tuning between the main line piping and the branch piping can have a considerable effect on seismic fragilities.
(ii) PIPING FRAGILITY EVALUATION: INTERACTION WITH BUILDING PERFORMANCE

This paper presents the results of a detailed study on the evaluation of piping system fragility curves by incorporating the nonlinear moment-rotation characteristics of a 2-inch threaded Tee-joint of a black iron pipe in the system level piping model. The particular focus is on the evaluating the effect of interaction between the building and the piping on the piping fragility. The effect of building behavior is included by considering low rise 5-story and high rise 20-story building models. For each of the two building models, piping fragilities are evaluated by considering the building to exhibit significant nonlinearity in the beams and columns as well as by considering the buildings to remain linear for each of the 22 real ground motion records. In order to consider the effect of nonlinearity in building model, the ground motion records are taken from relatively larger magnitude earthquakes (greater than 6) that are recorded in different parts of the world. It is observed that the fundamental building mode is not necessarily critical contributing mode in the evaluation of piping fragilities. Same is true for the fundamental piping system mode. Since piping system modes are local in nature, the piping mode that causes excitation around the location of the critical component such as the threaded Tee-joints governs the piping performance. Furthermore, the building mode with frequency closest to the governing piping system mode influences the piping system fragility more critically. It is also observed that the piping system fragilities can be higher in a linear building model whose modes have greater degree of tuning with the piping system modes. However, in some cases the piping system fragility
can be higher for the nonlinear building models if the nonlinearity increases the degree of
tuning between the piping and the building modes. The piping fragility does not necessarily
increases with the increasing floor levels in a high rise building. The product of modal
participation factor and the mode shape ordinate in the most critical building mode gives a
good estimate of the relative amplification at different floor levels and consequently the
relative increase or decrease in the piping fragility. Finally, it must be noted that as the degree
of degradation in a nonlinear building model increases with increasing values of the peak
ground accelerations, the effect of tuning between the building and the piping modes can
either increase or decrease. Such a variation in the degree of tuning leads to a corresponding
change in the piping fragility.
2. RECOMMENDATIONS FOR FUTURE RESEARCH

The research presented in this dissertation is a first step towards exploring the subject of system-level piping fragilities in buildings and other critical facilities. Based on the experience gained in conducting this research, a few recommendations for future work in this area are summarized below.

- Additional verification and validation work is needed for threaded Tee-joints of different diameters. Experimental data from the tests performed at UB for other pipe diameters should be used to conduct these additional verification studies.

- University at Buffalo is currently in the process of completing laboratory testing of a sub-branch piping system at two different floor levels connected with a riser. This sub-system consists of threaded Tee-joint connections for different diameter pipes. A comparison of FE model for this system with the experimental results and subsequent evaluation of piping fragility will be quite instrumental in improving the confidence of this work.

- Preliminary testing of subsystems at UB have shown that ceiling tile systems attached to the sprinkler heads can influence the performance of piping systems. Therefore, future research should consider the interaction of ceiling system and sprinkler heads within the sub-branch system to evaluate the effect of ceiling system on piping fragility.
• Evaluate the fragility of sub-branch piping system based on the multiple possible failure locations.

• A coupled analysis of the building and piping systems along with non-classical damping should be considered.

• It would be good to explore the development of simpler methodology for evaluating approximate but reasonable estimates of piping fragility without actually conducting a detailed nonlinear simulation using multiple time history analyses.