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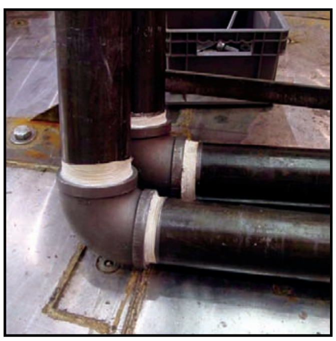


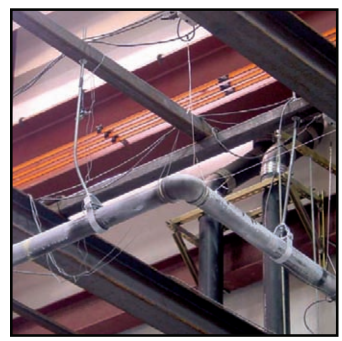

Technical Report MCEER-07-0013

September 4, 2007

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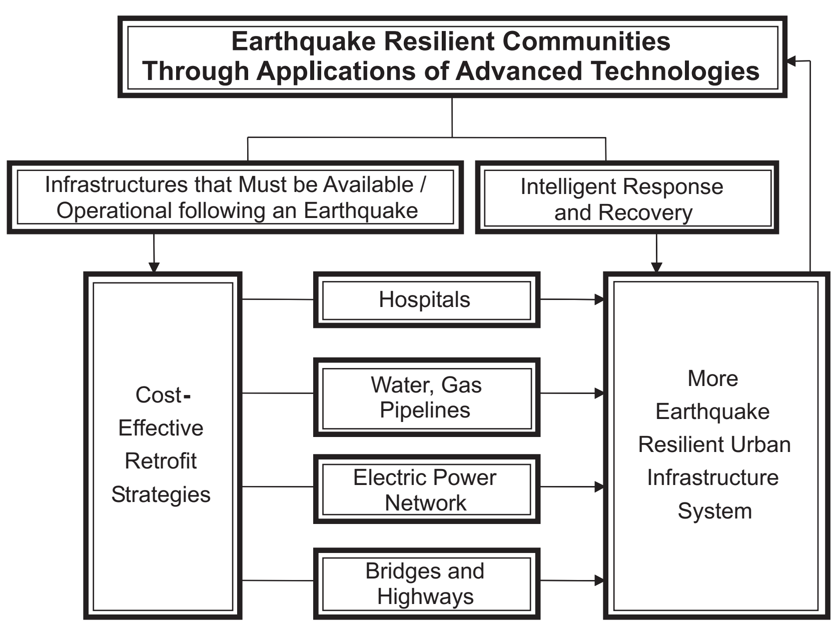
Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies: the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

MCEER's NSF-sponsored research objectives are twofold: to increase resilience by developing seismic evaluation and rehabilitation strategies for the post-disaster facilities and systems (hospitals, electrical and water lifelines, and bridges and highways) that society expects to be operational following an earthquake; and to further enhance resilience by developing improved emergency management capabilities to ensure an effective response and recovery following the earthquake (see the figure below).



A cross-program activity focuses on the establishment of an effective experimental and analytical network to facilitate the exchange of information between researchers located in various institutions across the country. These are complemented by, and integrated with, other MCEER activities in education, outreach, technology transfer, and industry partnerships.

This report describes an experimental research program conducted on hospital piping systems. The piping systems included typical valves, water heaters, and a heat exchanger modeled after a typical subassembly in a California hospital. The objectives were to understand the seismic behavior of typical braced and unbraced welded and threaded hospital piping systems, identify their drift capacities and failure modes, and provide data for use in calibration purposes in future analytical studies. The systems rested on a shake table and were hung from a stationary frame that rested on the lab floor. Two piping systems were developed, with identical geometries but different connection details. One had welded connections, while the other had threaded connections. Both systems were tested with and without seismic bracing. The seismic bracing used was a cable-style bracing commonly used in seismic applications. The braced and unbraced welded systems were subjected to story drifts up to 4.34% with no damage, which exceeds the 1997 UBC requirements on drift. Both the braced and unbraced threaded systems began to leak at 2.17% and 1.08%, respectively, and failed at 4.34%. Thus, they did not meet the 1997 UBC requirement on drift without sustaining damage.

ABSTRACT

Two piping subassemblies with identical geometries were tested in the Large Scale Structures Laboratory at the University of Nevada, Reno. The experimental piping subassembly was modeled after a typical subassembly in a California hospital. The two subassemblies differed only by their connection details. One subassembly had welded connections; the other had threaded connections. The welded and threaded subassemblies were then tested with and without seismic bracing. The braced welded subassembly was subjected to story drifts up to 4.34% with no damage. The braced threaded subassembly began to leak at 2.17% and failed at 4.34%. The unbraced welded subassembly was subjected to story drifts up to 4.34% with no damage. The unbraced threaded subassembly began to leak at 1.08% story drift. The welded piping subassemblies exceed the 1997 UBC requirements on drift. The threaded piping systems do not meet the 1997 UBC requirement on drift without sustaining damage.

ACKNOWLEDGEMENTS

The authors thank Bruce Dillon for his knowledge in constructing piping systems and for his enthusiasm, Patrick Laplace for his assistance, and help with instrumentation, and Paul Lucas and Patrick Laplace for their help in the lab with instrumentation.

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SECTION 1
INTRODUCTION

1.1 Background

The operation of an essential facility, such as a hospital, after an earthquake relies heavily on its nonstructural components. The nonstructural components of a building are those systems, parts, or elements that are not part of the structural load-bearing system. Examples of nonstructural components include elevators, architectural partitions, ceiling and piping systems. These systems are susceptible to damage during earthquakes. Any damage to these systems can cause substantial economic loss as well as functional loss of the building's systems.

Specifically, the behavior of hospital piping systems during earthquakes has been poor. The improper bracing of systems, the use of non ductile components and the lack of understanding of the behavior of these systems have all contributed to the poor performance of hospital piping systems during past earthquakes.

The focus of this report is to describe experimental research conducted on hospital piping systems at the University of Nevada, Reno. The following sections describe the observed damage due to past earthquakes and the codes that govern the seismic design of piping systems. A comprehensive literature review is also presented.

1.2 Behavior of Piping Systems During Previous Earthquakes

The following sections will present a brief overview of the performance of piping systems during the Northridge, Kobe and Nisqually earthquakes. Damage reports compiled after these earthquakes are discussed.

1.2.1 1994 Northridge Earthquake

The economic losses due to the earthquake damage of nonstructural components is significant. Repair or replacement costs of non residential buildings after the Northridge earthquake was $6.3 billion and only $1.1 billion was due to structural damage (Kircher 2003). Immediately after the earthquake, 88% of the hospital beds in the damage area (13 hospitals) had to be evacuated due primarily to nonstructural damage such as water damage, elevator failure, etc. (Ayres and Phillips 1998).

Damage reports are an effective way of learning from past damage to buildings and their contents. Numerous reports have been published by state and private agencies. A few of these damage reports are summarized below. To compile these reports, teams of engineers go into the affected area and catalogue the damage to nonstructural components. The teams also talk with hospital officials to understand some of the secondary effects such as repair/replacement costs and hospital downtime.

Office of Statewide Health and Planning Development (OSHPD) commissioned Ayres & Ezer Associates and Hillman Biddison & Loevenguth (Ayres et al. 1996) to collect and analyze water damage data from 13 hospitals damaged during the 1994 Northridge Earthquake. The objectives were to document failures in fire sprinkler, heating, ventilating and air conditioning and domestic water systems. The scope of the study also included identifying deficiencies and recommending corrective measures.

The most significant finding of the study was the extensive failures in heating hot water line connections to unbraced duct mounted reheat coils. Damage to fire sprinkler systems caused by differential movements were also found to be a major cause of water damage. Acceleration time histories obtained from different types of buildings were analyzed. By determining what the actual acceleration levels were in buildings during the earthquake, the authors determined that design force levels were not unconservative.

A report prepared by John A. Martin & Associates, Inc., compiled hundreds of strong ground motion and building response accelerograms that were retrieved from stations throughout the Los Angeles area after the 1994 Northridge earthquake (John A. Martin & Associates 1997). There were a total of 20 buildings included in this report. The buildings included two hospitals, numerous office buildings and hotels, as well as a parking garage.

Along with the recorded accelerograms, the code specified values for natural period design base shears and drift indices were calculated. Two code values were calculated: one value corresponding to the code in use at the time of the building design and the other was based on the 1994 UBC. These values were then compared with the earthquake records. Also included in the report are pictures and descriptions of damage from each of the sites. See also references (Ayres and Phillips 1998), (Naeim 1997) and (Pandya 1995).

1.2.2 1995 Kobe Earthquake

A report published by OSHPD detailed the damage to Kobe area hospitals after the 1995 Kobe earthquake. The Kobe earthquake caused 193 of the 222 hospitals in the Hyogo Prefecture to undergo some level of damage. Of those 193, 13 suffered major damage or partial collapse. Most of the hospitals suffered only minor structural damage but damage to nonstructural components was much more widespread.

Numerous rooftop mounted water tanks or their piping connections failed flooding the upper floors of the facility. The water supply that was left in the building, drained quickly, leaving the hospital with no water. Of the affected hospitals, two thirds experienced suspension of hot and tap water service. Because water service was severed during the earthquake, hand washing and sterilizing were limited in operating rooms.

1.2.3 2001 Nisqually Earthquake

A report published by PEER (Filiatrault et al. 2001), summarizes damage to nonstructural components in the 2001 Nisqually earthquake as well as numerous other earthquakes. A chapter is devoted to the codes that govern nonstructural components. The National Earthquake Hazards Reduction Program (NEHRP), Structural Engineers Association of California (SEAOC) Blue Book, 1997 Uniform Building Code, and the 2000 International Building Code are all discussed. A literature review on relevant analytical and experimental works is given. Finally, a computerized database of extensive amounts of literature on nonstructural components is discussed. PEER developed the database which includes reconnaissance reports, past research and many specific requirements published by different organizations.

Some of the damage discussed includes a 75 mm diameter pipe that broke in a building causing 3000 liters of water in a storage tank to flood the building. An unsecured water tank shifted causing a water supply line to rupture. Of the $2 billion loss resulting from the Nisqually earthquake a large portion was associated with nonstructural components.

1.3 Development of Seismic Design Specifications for Piping

The following text provides a brief overview of the current design methods and codes that govern piping systems and all other nonstructural components. The 1997 UBC (ICBO 1997), 2000 IBC (ICC 2000) and the 1997 NEHRP (ICBO 1997; ATC 1997) codes are briefly summarized.

1.3.1 1997 UBC

The Uniform Building Code (UBC) is the code used primarily in the U.S., west of the Mississippi River. The UBC is updated about every 3 years. As with other codes, the UBC is constantly evolving to better estimate the forces on nonstructural components. For example, the 1988 UBC took into account that an earthquake had different effects on flexible or flexibly mounted nonstructural components. Also, versions of the UBC prior to the 1997 UBC did not require a seismic force that varied along the height of a building.

The 1997 UBC calculates forces on nonstructural components with the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{1.1}$$

with

$$0.7C_a I_p W_p \le F_p \le 4.0C_a I_p W_p \tag{1.2}$$

where:

F_p = total design lateral seismic force
a_p = in structure component amplification factor, ranges from 1.0 to 2.5 (1997 UBC Table 16-O)
C_a = seismic coefficient (1997 UBC Table 16-Q)
R_p = component response modification factor, ranges from 1.0 to 4.0 (1997 UBC Table 16-O)

I_p = importance factor, ranges from 1.0 to 1.5 (1997 UBC Table 16-K)
h_x = element or component attachment elevation with respect to grade
h_r = structure roof elevation with respect to grade
W_p = weight of the component

1.3.2 2000 IBC and 1997 NEHRP

The 1997 NEHRP was developed by the U.S. Building Seismic Safety Council. The 2000 IBC adopted the design seismic force and displacement equation of the 1997 NEHRP guidelines. (Filiatrault et al. 2001)

The 1997 NEHRP calculate forces on nonstructural components with the following formula:

$$Fp = \frac{0.4 a_p S_{DS} W_p}{\frac{R_p}{I_p}} \left(1 + 2\frac{z}{h}\right) \tag{1.3}$$

with

$$0.3 S_{DS} I_p W_p \le F_p \le 1.6 S_{DS} I_p W_p \tag{1.4}$$

where:

F_p = seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution
a_p = in structure component amplification factor, ranges from 1.0 to 2.5
S_{DS} = the design spectral response acceleration for short periods
I_p = component importance factor, ranges from 1.0 to 1.5
z = height in structure at point of attachment of component
h = average roof height of structure relative to the base elevation
R_p = component response modification factor which ranges from 1.0 to 5.0
W_p = component operating weight

1.3.3 Seismic Bracing

In order for piping systems to withstand the code derived forces, seismic restraints must be designed to absorb the required force demands. The following sections briefly describe the types restraints used on piping systems. The two primary types of seismic restraints (also referred to as bracing) for piping systems are cable and solid braces. Both types have a vertical hanger rod (also referred to as support rod) and both limit the lateral deflection of the pipe.

1.3.3.1 Cable Restraints

Cable bracing restrains the deflection of the pipe by utilizing cables that take tension only. Since the cables take only tension, two cables are necessary to brace the pipe. The cable is generally some kind of prestretched high strength cable. (see figure 1-1)

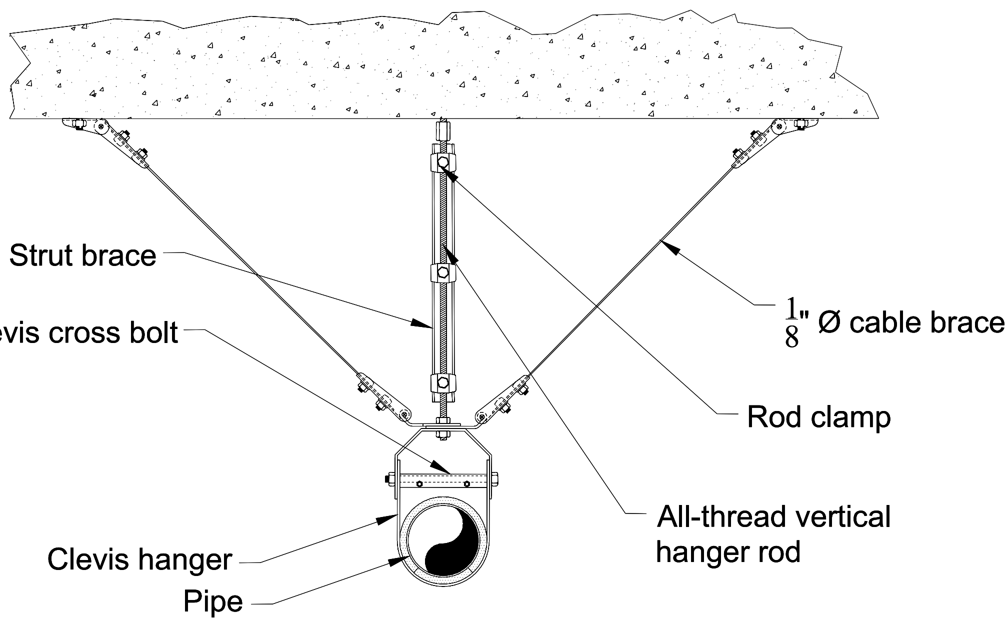


Figure 1-1 Typical Transverse Cable Brace

In a seismic event, cable bracing causes the vertical support rod to be in compression, regardless of the direction of the seismic force (Lama 1998). This may cause the vertical support rod to buckle. Bracing manufacturers require an angle or strut to be clamped to the vertical hanger rod to prevent buckling. The clamps combined with the angle or strut reduces the unbraced length of the vertical hanger rod thereby increasing the compression capacity of the rod.

Since the cables do not take compression, a hook may be used to connect the brace to the clevis. Hooking devices cannot be used on solid seismic bracing due to the fact that the solid cable brace, when in compression could break the hook free. Another distinct advantage of cable bracing is that the cables can be cut to the approximate length and adjusted easily during installation.

1.3.3.2 Solid Restraints

Solid bracing consists of a single steel member, generally an angle or 12 gauge channel strut, installed at the hanger rod connection to the system up to the structure at an angle between 30 and 60 degrees from horizontal. One advantage of this system is that it needs access to the structure on only one side. (see figure 1-2)

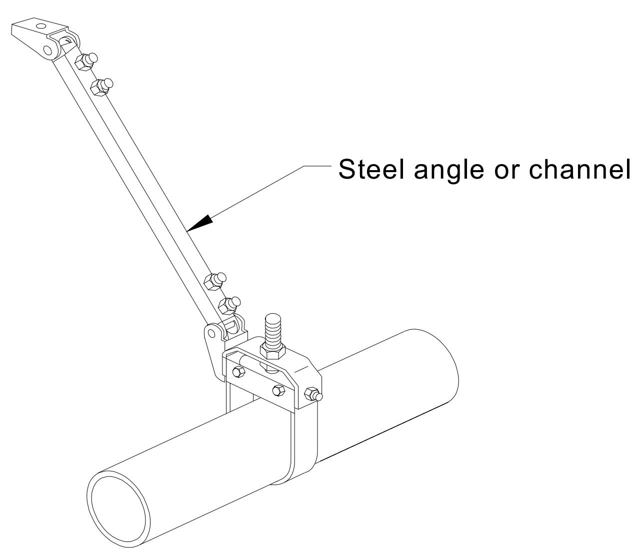


Figure 1-2 Typical Transverse Solid Brace

There are two disadvantages to using solid bracing. First, the length of the brace must be limited in order to effectively resist compression loads without buckling. Also, when the solid brace is in compression, the support rod is forced into tension. This tension may cause pullout failure at the connection point, and is most crucial when support rods are anchored to concrete slabs (Lama 1998).

1.3.3.3 Wall and Floor Penetrations as Brace Locations

Wall and floor penetrations might be appropriate brace locations for ductwork, but piping generates much higher loads in a more concentrated area which may damage most walls. Light framed walls required for area separation, smoke barriers or other fire or life safety related functions should not be used to brace piping. In all other cases, the structural engineer of record should determine if a wall may act as a sway brace for piping (Tauby et al. 1999).

1.4 Literature Review

Very little research has been done on entire piping systems. One of the few studies that did deal with an entire system was performed at the University of California Berkeley. There, researchers completed a study of two alternative seismic restraints to snubbers, which are commonly used in nuclear power plants (Nims 1991). The research objectives were to find an alternative seismic restraint device that did not have the maintenance problems and associated costs of the snubbers, and study the effects of the new devices on pipe forces, stresses and behavior.

The two alternate types of seismic restraints tested were a gap device and a frictional energy dissipating device. Two frames of different natural frequencies and a piping system that ran between the two frames were constructed on top of a shake table. The dynamic properties of the framing system were determined prior to erecting the piping system. The snubbers were tested on the system first in order to provide a standard that the other devices could be tested to. All of the devices were tested with a series of synthetic and recorded motions.

Although the piping system introduced high frequency accelerations into the frame, the piping system did not significantly affect the frame response. The source of the high frequency acceleration was the experimental devices rattling in the rigid supports. The researchers found that the high frequencies did influence the piping and devices where they attached to the frames. The seismic restraints were also found to introduce high frequencies into the piping system.

In 1999, Tauby et al. published a practical guide to seismic restraint. The authors of this guide give design and analysis examples of all types of seismic restraint (Tauby et al. 1999). Also included are guidelines to bracing piping systems.

Each straight run of the piping system should be braced with a minimum of two transverse braces installed perpendicular to the piping and one longitudinal brace installed parallel to the piping. Transverse braces should be located at the final support of each run of pipe with two supports. If the distance between the braces exceeds the maximum transverse brace spacing, additional transverse braces should be located to limit the brace spacing to the maximum transverse brace spacing. A longitudinal brace should be located on each straight run of the pipe. If the length of the pipe run exceeds the maximum longitudinal brace spacing, additional longitudinal braces should be located on the pipe run while limiting the brace spacing to the maximum longitudinal brace spacing. A transverse brace located within the maximum offset length of the pipe can provide limited longitudinal bracing for the straight run of pipe around a 90° turn, or elbow. The maximum offset length is based on the maximum stress of the pipe and a maximum ¼ in (6 mm) deflection of the offset of the pipe with welded, brazed, or groove-fitted joints. Vertical drops from horizontal runs of piping to the equipment require a transverse brace at the final support location before the pipe drop. Avoid bracing a pipe to separate portions of the structure that may act differently in response to an earthquake. For example, do not connect a transverse brace to a wall and a longitudinal brace to a floor or roof at the same brace location.

In 2000, researchers at Hiroshima University in Japan investigated the acceleration amplification of nonstructural systems mounted on floors of buildings (Marsantyo et al. 2000). The researchers used a three degree of freedom structure with and without a suspended pendulum isolation (SPI) system. The structure was subjected to four different ground excitations and the floor responses were recorded. A filing cabinet was then placed on the shake table surface and subjected to the floor responses recorded from the first stage of testing.

It was found that the dynamic responses of nonstructural systems are strongly influenced by the response of the floors to a ground excitation as well as characteristics of the ground motion. Specifically, they found indications that amplified responses of nonstructural components are more dependent on the frequency content of the ground excitations than they are on the strength level of the ground motion.

T. Chiba, R. Koyanagi, N. Ogawa and C. Minowa investigated the behavior of a large scale piping system (Chiba et al. 2000). The research included multiple support excitations.

Numerous computational methods were investigated. These included direct integration (Newmark b, Wilson q), modal superposition, modal time history, complex frequency response, multiple response spectra and uniform response spectrum methods.

The experimental portion of their work was on a piping system that consisted of 4 and 8 inch diameter pipe, about 70 feet long. The system had 13 elbows, one tee, one reducer, three simulated valves and three welded flanges. The system was filled with water and supported by four separate shake tables. The motions used included the 1940 El Centro N-S and an artificial signal.

The analytical and measured frequencies of the system matched. The modal time history results were compared with the time history traces and were found to be in agreement. The multiple response spectra method was investigated because this is the method used in the design of piping systems with multiple support excitations. The multiple response spectra method has good correlation to the maxima observed in the time history analysis.

The researchers found that discrepancies arose between the measured and calculated results given by the multiple response spectra method near the support points; when the model amplitudes are zero.

Some studies focused on the mechanical properties of pipe. One such project was undertaken to characterize the damping properties of pipe undergoing high levels of plastic deformations (Otani et al. 2000). This was done by observing the response of a 2-D piping model during high levels of excitation. The vibration energy of the model and the plastic energy dissipation were calculated.

The damping ratio was obtained from the ratio of those two values. This actual damping ratio was compared with the equivalent damping ratio calculated from the maximum response.

The researchers also calculated damping from the moment - deflection angle curves of the elbows. Using this relationship, the damping can be found from the hysteresis loop of a single cycle.

The researchers found that the damping calculated from analysis, test results or from energy dissipation of the elbows were all similar. They also found an equation for the system energy dissipation based on displacement response. A method using analysis results, response displacement and the equivalent damping ratio from the dissipation energy to find the appropriate damping value to predict the maximum response was presented.

In 2000, Adams et al. published a paper on a subjective method of evaluating piping systems. The authors of this paper discussed the methodologies to perform seismic evaluations and seismic verification walkdowns of a piping system, specifically in the Monticello Nuclear Generating Power Plant (Adams et al. 2000). A walkdown evaluation compares the subject piping systems to piping systems which have actually experienced strong motion earthquakes (experience data). This was different from the historically used practice of computer analysis. The seismic walkdown procedure includes the following objectives:

- Documentation of the most common causes of seismic damage or operational difficulties in facilities that contain structures, systems, equipment and components similar to those in nuclear stations.
- Credible definition of the threshold of seismic motion for various types of documented earthquake damage and shake table tests.
- Identification of structures, system equipment and components that are not typically damaged in shake table tests that subject the components to earthquakes that are larger than design basis earthquakes for nuclear stations and other facilities. This data provides insights to the actual seismic design margin.

- Development of seismic integrity criteria that can credibly predict the performance of structures, system equipment and components in future earthquakes.

The authors outlined that the reasons for local failures in piping systems can be due to the following:

- Relatively low piping flexibility in regions of relatively large displacements where piping is attached to building structures, massive equipment or other piping.
- Low piping ductility associated with the use of cast iron, PVC or other low-ductility materials.
- Threaded piping joints or other regions of reduced cross section with sharp corners susceptible to fatigue, ratchet cracking or rupture when subject to cyclic seismic loads.
- Regions of degraded pipe caused by corrosion or erosion.
- Weak joints associated with friction type connections or repairs which result from poor welding.
- Failure of piping associated with loss of non-ductile pipe supports.

Although not an exact method, the seismic walkdown procedure can be effective since experience data states that only 0.01 percent of the total piping at risk of seismic failure has failed since 1952.

Another paper that focused on brace components was one published by Malhotra et al. in 2003. For this paper, researchers set out to determine the number of cycles for which a component must resist its rated capacity (Malhotra et al. 2003). A series of tests were performed that illustrated the cyclic behavior of brace components and a test protocol was developed in order to determine the seismic strength of brace components.

The researchers subjected a test setup to an input motion that occurred during the Northridge earthquake at the roof of a building. They recorded the load taken by the brace and then found the deformation history. Using this history they found a uniform amplitude deformation history that caused the same damage as the non-uniform deformation history. The number of cycles of the uniform amplitude deformation history that the system should withstand was found by proportioning the number of uniform amplitude deformation history cycles for the 6.7Mw Northridge up to a 7.2Mw earthquake. The number of cycles was found to be 15.

The researchers found that components that derive their strength from friction between brace-pipe and sprinkler-pipe appear to exhibit lower strength at higher frequencies. Therefore, friction based components should be tested at a high frequency. Components that did not derive their strength from friction exhibit greater strength at higher frequencies. These components could be tested at a slow rate to obtain a conservative estimate of cyclic strength.

1.4.1 Damage Databases

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) has compiled an extensive Microsoft Access database on earthquake damage to nonstructural components (Kao et al. 1999). The records date back to the Alaska earthquake of 1964 and include entries from 52 other earthquakes up through 1999. There are over 2900 entries of nonstructural component damage from the 52 earthquakes included. A user may sort the database by earthquake, equipment affected or type of building affected.

The authors of the database consider it as a living document. As more information from past earthquakes becomes available, the database will expand; and as data is gathered from future earthquakes it will be added to the database.

The Pacific Earthquake Engineering Research Center (PEER) expanded upon the MCEER database from 1999 (Taghavi and Miranda 2003). The researchers at PEER improved upon the MCEER database in one way by expanding the database to include a wider range of information about nonstructural components. Information on fragility curves as well as costs and losses of nonstructural components were added to the database. Costs of over 200 typical nonstructural components commonly used in buildings, cost analysis of 23 different types of buildings and cost functions of nonstructural components for different damage states were all added.

PEER also changed the way data is retrieved from the database. PEER made the database as a self executable, standalone program unlike the Microsoft Access platform used by MCEER. Searching and forming queries with this standalone application gives the user more flexibility and ease in extracting information.

1.5 Objectives and Scope

The primary objectives of this research are to:

- Understand the seismic behavior of a typical braced and unbraced welded and threaded hospital piping systems
- Identify the drift capacities and failure modes of the braced and unbraced welded and threaded hospital piping system
- Provide data for use in calibration purposes in future analytical studies

There are numerous configurations of piping subassemblies. This research was conducted on one typical subassembly. The results of this work, when applied to other configurations, should be applied with engineering judgment.

In order to achieve these objectives, typical welded and threaded hospital piping systems were subjected to earthquake excitation with and without seismic bracing. The piping systems were modeled after a subassembly at the University of California Davis medical center. The subassembly included typical valves, water heaters, and a heat exchanger. The size of the pipes were 3 inch and 4 inch. The subassemblies rested on a shake table and were hung from a stationary frame that rested on the lab floor. Although this experiment does not represent the actual boundary conditions and anchor point input motions which would be applied to the piping subsystem during an earthquake, it allows the study of the response to interstory motions and it provides an economical set up that can be used to estimate drift capacities. The experiments were carried out in the Rogers Wiener Large Scale Structures Laboratory at the University of Nevada, Reno.

SECTION 2
EXPERIMENTAL INVESTIGATION

2.1 Introduction

This chapter describes the experimental specimens and components used in the research program. In consultation with OSHPD engineers, the experimental hospital piping system was modeled after a system in the University of California, Davis Medical Center. The system was modified slightly to accommodate the dimensions and geometry restrictions of the shake table facility.

The following subassemblies were tested in the experimental protocol:

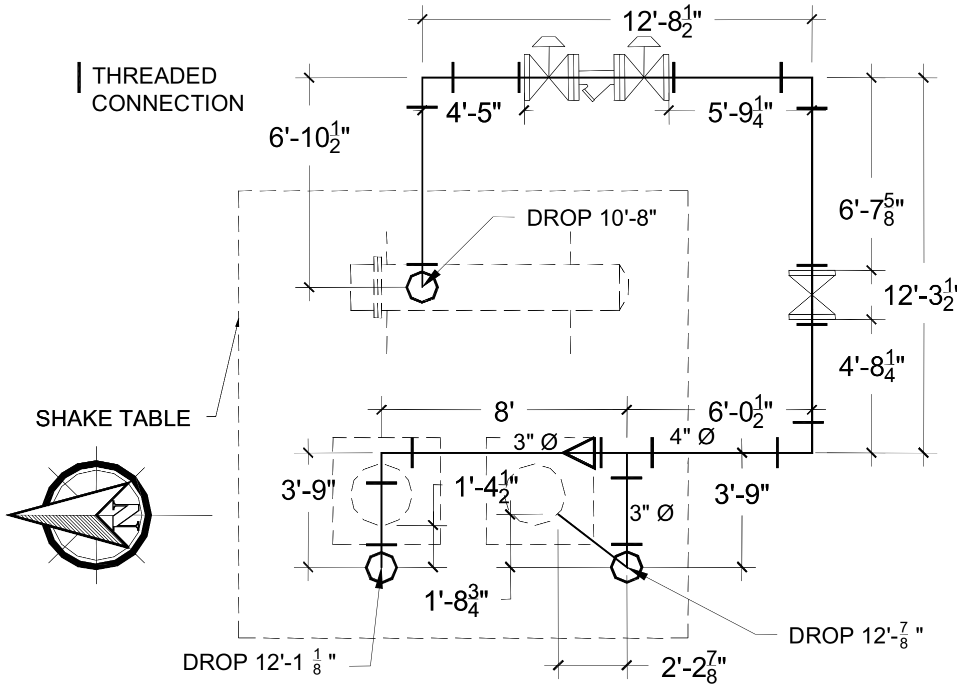
- Welded braced
- Welded unbraced
- Threaded braced
- Threaded unbraced

These subassemblies will be described in the following pages.

2.2 Experimental Piping Subassembly Specimen

Two geometrically identical systems were tested, one with welded connections and one with threaded connections. The connection details will be discussed in a later section.

Figure 2-1 through figure 2-3 illustrate the plan and elevation views of the subassemblies. The system was made up of approximately 100 feet (30.48 m) of 3 inch (7.62 cm) and 4 inch (10.16 cm) diameter schedule 40 ASTM A53 Grade A black steel pipe. The yield stress of this material is 30 ksi (207 MPa) and the ultimate stress in tension is 48ksi (331 MPa). The system includes two water heaters, one simulated heat exchanger, one y-strainer (61 lbs/27 kg), one check valve (80lbs/36 kg) and two gate valves (83 lbs/37kg). The water heaters were connected to the system through a 4 bolt flanged connection as shown in figure 2-4. The heat exchanger and all of the valves were connected to the pipes through an 8 bolt flanged connection, seen in figure 2-5. The system was filled with room temperature water prior to the experiments. The only pressure in the system was the hydrostatic pressure caused by the water.



igure 2-1 Plan View of Threaded Piping Subassembly

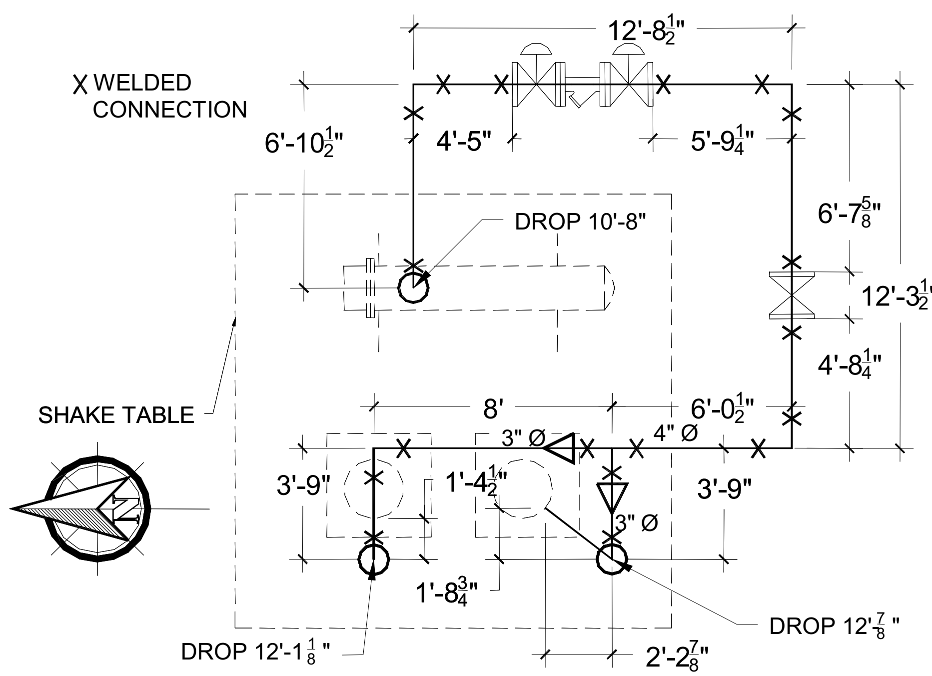


Figure 2-2 Plan View of Welded Piping Subassembly

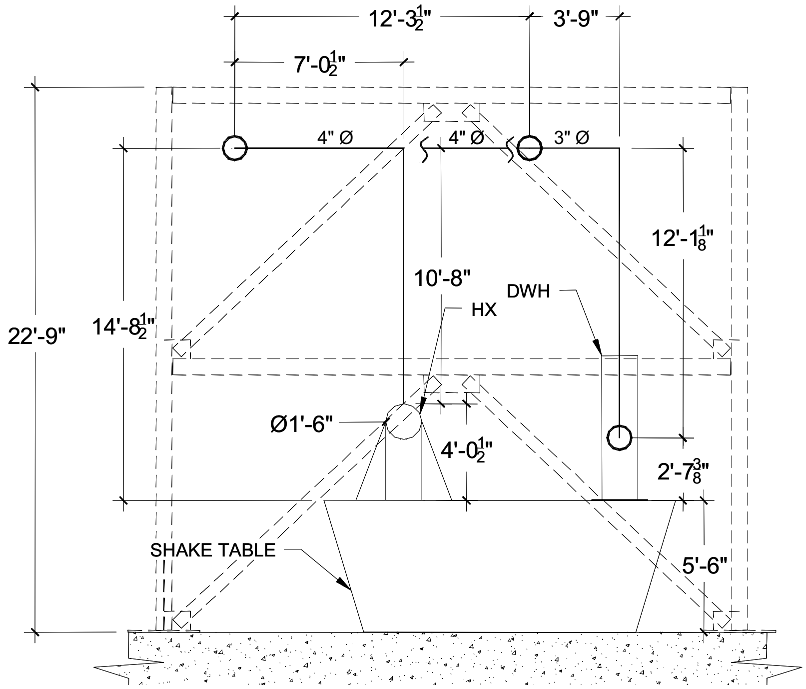


Figure 2-3 North Elevation of Piping Subassembly

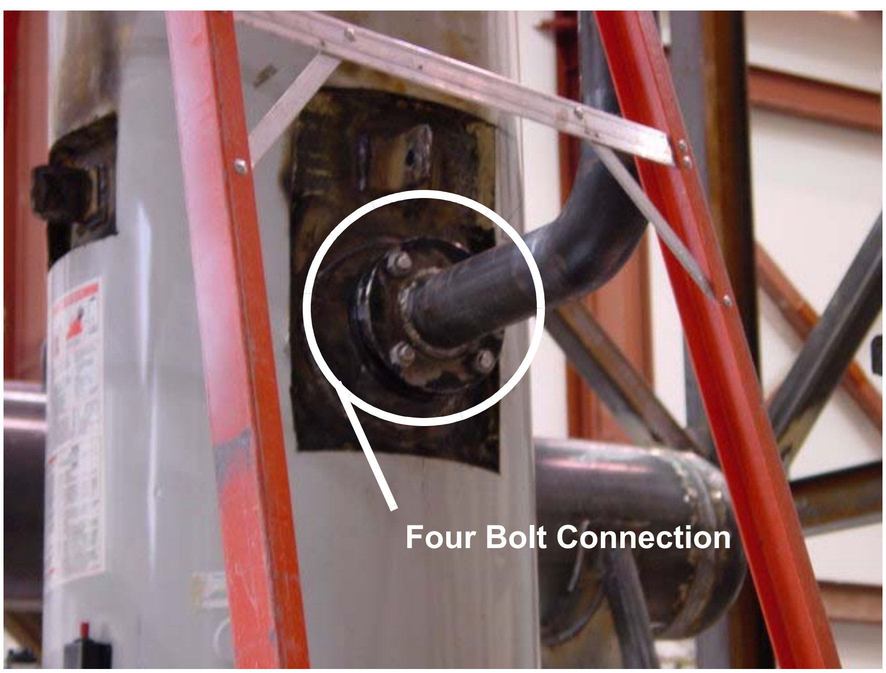


Figure 2-4 Four Bolt Flanged Connection to Water Heaters

Figure 2-5 Eight Bolt Flanged Connection to Valves and Heat Exchanger

2.2.1 Connection Details of Welded and Threaded Piping Subassemblies

2.2.1.1 Layout

One difference in the layout of the two systems is the tee connection configuration. In the welded system, the tee connection was made with a 4 inch tee and two 4 inch to 3 inch concentric reducers. (See figure 2-9) In the threaded system, the tee connection was made with a reducing tee and one 4 in to 3 in concentric reducer. (See figure 2-6)

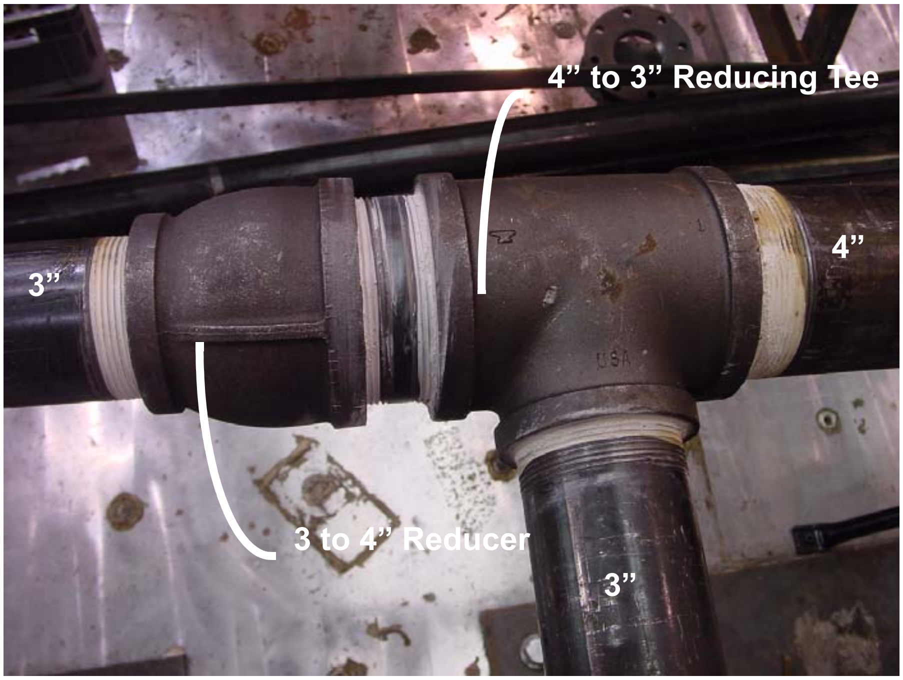


Figure 2-6 Threaded Tee Connection

2.2.1.2 Welded Subassembly

All of the elbow to pipe connections were welded using a shielded metal arc welding process. The root material was E6010 and the fill material was E7018. A joint before welding can be seen in figure 2-7. Each joint was prepared with a 65°, 3/32 inch (0.24 cm) length root and a 3/32 inch (0.24 cm) landing. A copy of the welding procedure can be found in Appendix A. A table of the welding procedure and joint schematic are provided in table 2-1. A joint seen after one pass and with a partial fill completed is shown in figure 2-8, and the finished product can be seen in figure 2-9.

Figure 2-7 Joint Before Welding

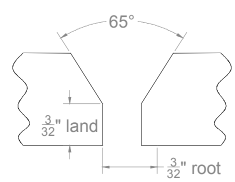
		Filler Metals		Current				Joint Details
Pass or Weld Layer (s)	Process	Class	Diam.	Type and Polarity	Amps or Wire Feed Speed	Volts	Travel Speed	
1 Root	SMAW	E6010	1/8"	DCEP	75-130	NA	NA	
2 thru all fill	SMAW	E7018	3/32"	DCEP	70-100	NA	NA	

Table 2-1 Welding Procedure

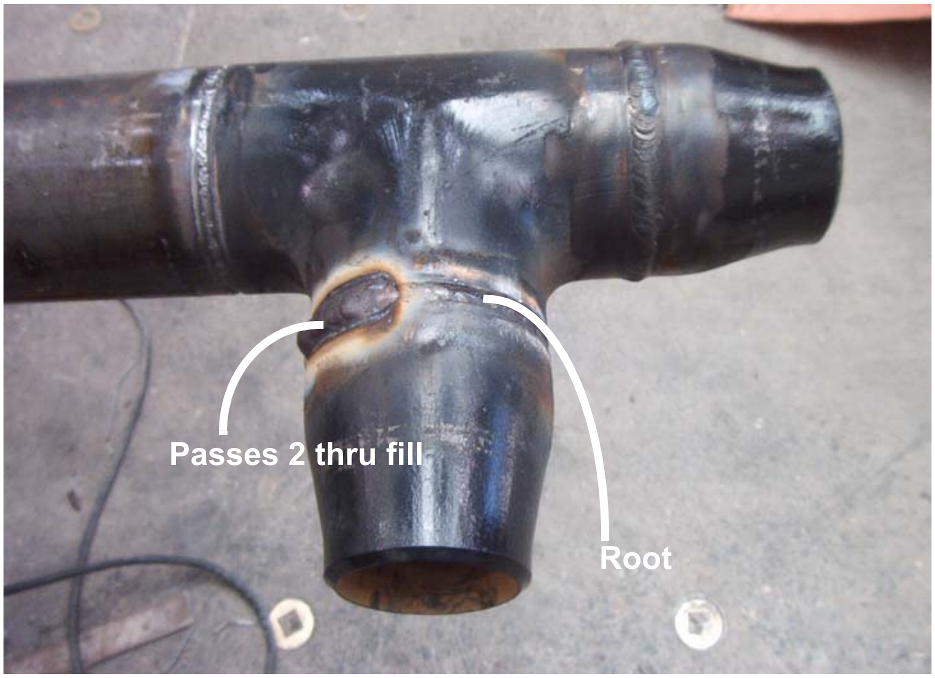


Figure 2-8 Welded Tee Connection After One Pass

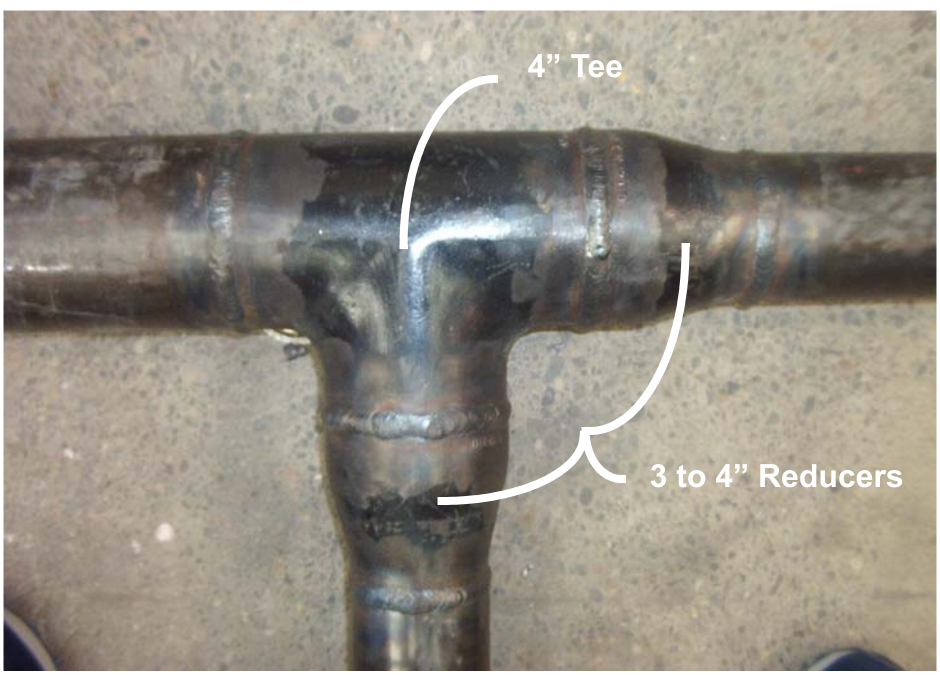


Figure 2-9 Completed Welded Tee Connection

All fittings including flanges and elbows were forged steel. Forged steel is highly weldable, unlike the cast iron fittings used in threaded systems. Forged steel fittings were used in lieu of cheaper cast iron fittings due to permit welded connections. All elbows were long radius elbows.

2.2.1.3 Threaded Subassembly

The fittings on the threaded system were all cast iron. These fittings are used in threaded systems because they are cheaper than forged steel fittings. The ends of each pipe were threaded according to table 2-2. Once threaded, the pipe ends were covered with a water proofing dope. While the dope was still wet, the pipe was twisted into the fitting and again sealed with dope. The dope and fittings can be seen in figure 2-10 and figure 2-6. The pipe is secured with enough torque to engage all of the threads in the fitting.

Nominal Pipe Size (inches)	Schedule 40 Wall Thickness (inches)	Depth of Thread Cut (inches)	Remaining Wall After Threading (inches)	Amount of Pipe Wall Lost
1/4	0.088	0.0594	0.0286	67.5%
3/8	0.091	0.0594	0.0316	65.3%
1/2	0.109	0.0721	0.0369	66.1%
3/4	0.113	0.0721	0.0409	63.8%
1	0.133	0.0846	0.0484	63.6%
1 - 1/4	0.140	0.0846	0.0554	60.4%
1 - 1/2	0.145	0.0846	0.0604	58.3%
2	0.154	0.0846	0.0694	54.9%
2 - 1/2	0.203	0.1150	0.0880	56.7%
3	0.216	0.1150	0.1010	53.2%
3 - 1/2	0.226	0.1150	0.1110	50.9%
4	0.237	0.1150	0.1220	48.5%
5	0.258	0.1150	0.1430	44.6%
6	0.280	0.1150	0.1650	41.1%
8	0.322	0.1150	0.2070	35.7%

Table 2-2 Pipe Threading Information

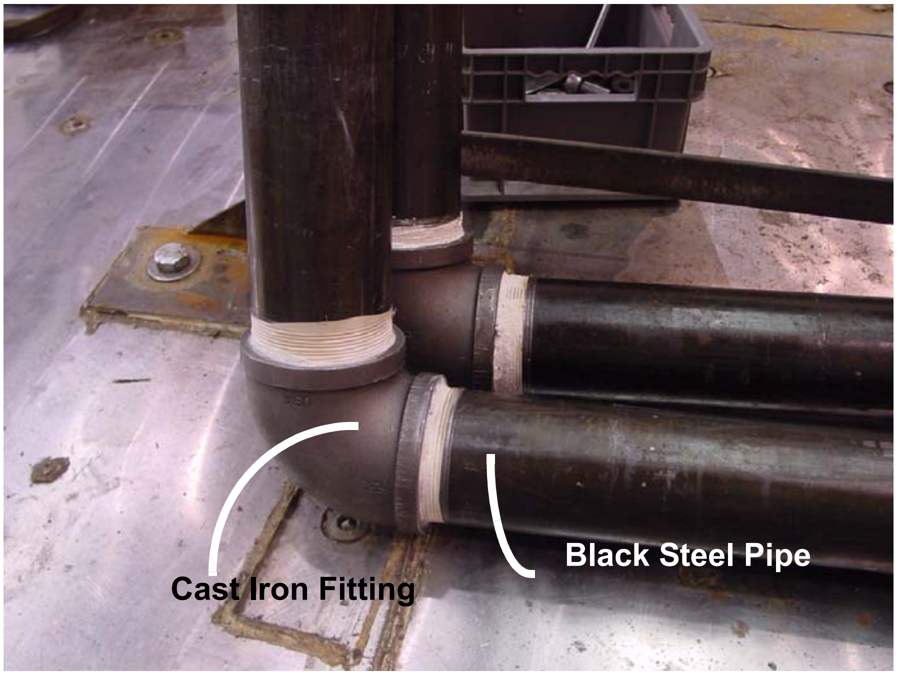


Figure 2-10 Threaded Elbow Connection

2.2.2 Bracing Layout and Components

2.2.2.1 Braced Subassembly

There were seven brace points and four hanger points for the braced experiments. Figure 2-11 illustrates the bracing layout. The layout was designed by Mason Industries staff. In California, bracing manufactures undergo a preapproval process with OSHPD. This preapproval process allows companies to make standardized design tables that make designing bracing systems an easy process for engineers. Knowing a site specific acceleration, the diameter of the pipe and the contents of the pipe, an engineer can go into the tables and pull out what kind of brace is required as well as the spacing of the braces. A sample of these design tables can be seen in Appendix C. (Mason Industries 2004)

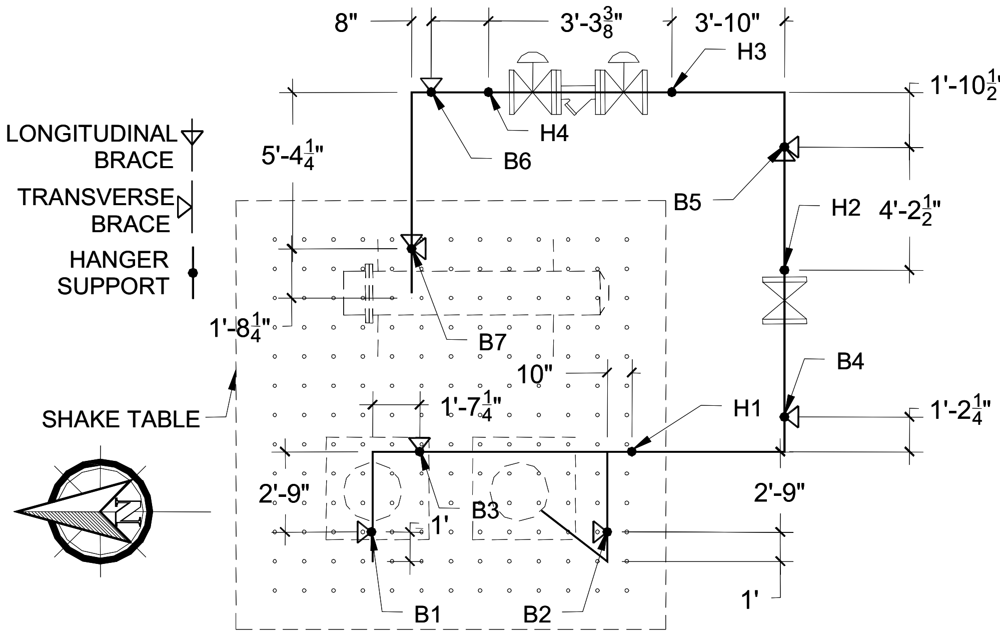


Figure 2-11 Bracing Layout

The cables were made of 1/8 inch (0.3175 cm) diameter prestretched galvanized 7x19 aircraft grade steel. The vertical hanger rods were of two sizes: 5/8 inch (1.59 cm) diameter all-thread galvanized steel rod for supporting the 4 inch (10.16 cm) diameter pipe and 1/2 inch (1.27 cm) diameter all-thread galvanized steel rod for supporting the 3 inch (7.62 cm) diameter pipe. The vertical hanger rods were braced continuously along their length with 1 5/8 inch (4.13 cm) square, 12 gauge strut. The detail of both longitudinal and transverse braces can be seen in figure A.1 and figure A.2. A picture of a typical transverse brace can be seen in figure 2-12.

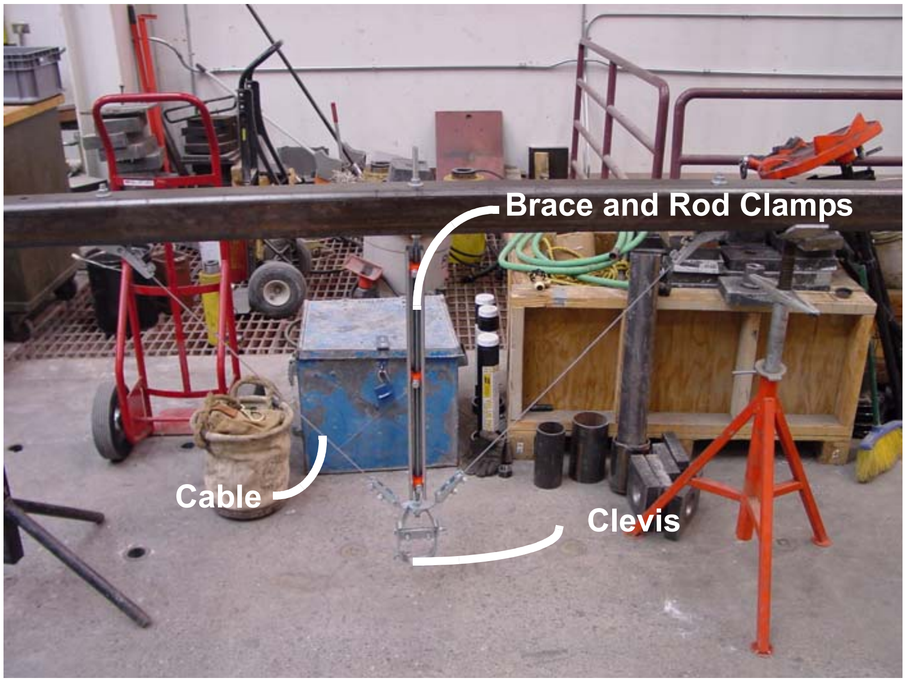


Figure 2-12 Typical Transverse Brace

2.2.2.2 Unbraced Subassembly

The unbraced system geometry and materials were the same as the braced system, except that the cable braces, the strut that braced the all-thread vertical rod and the clevis cross braces were removed. This left only the vertical hanger rods as supports. A picture of a typical unbraced hanger support can be seen in figure 2-13.

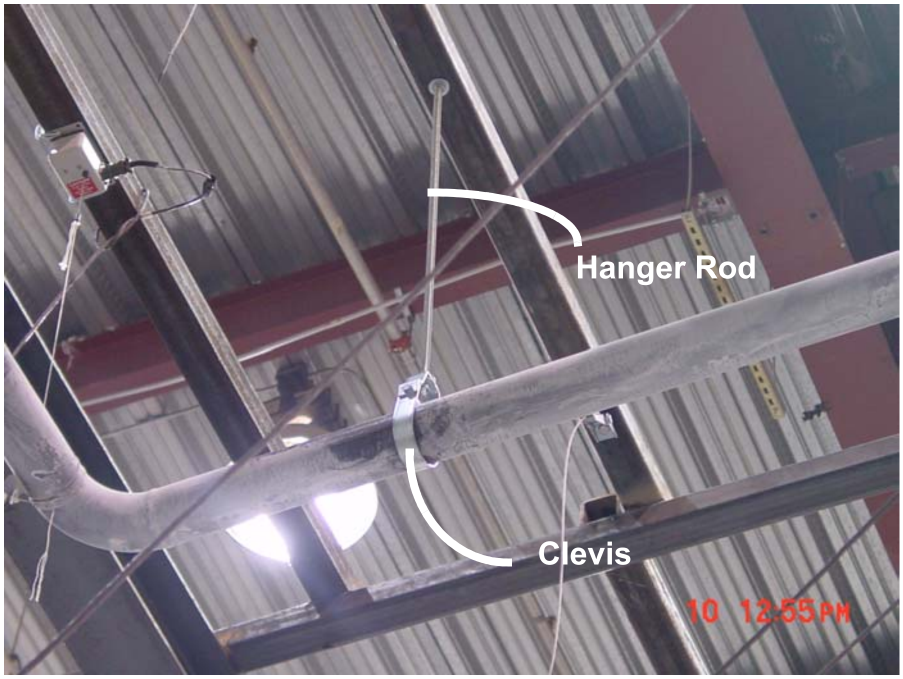


Figure 2-13 Typical Unbraced Support

2.3 Experimental Setup

The water heaters and the heat exchanger were anchored to the shake table and the pipes were braced and hung from a stationary frame, which rested on the lab floor, as shown in figures 2-14 - 2-16. The fixed frame allowed the piping system to be subjected to extreme relative displacements; story drifts in excess of 8%. The water heaters were braced on the table (see figure 2-17) to avoid premature failure of the piping system due to excessive rigid body motion of the water heaters. The system was painted with a white wash to aid in observing cracks and leakage.

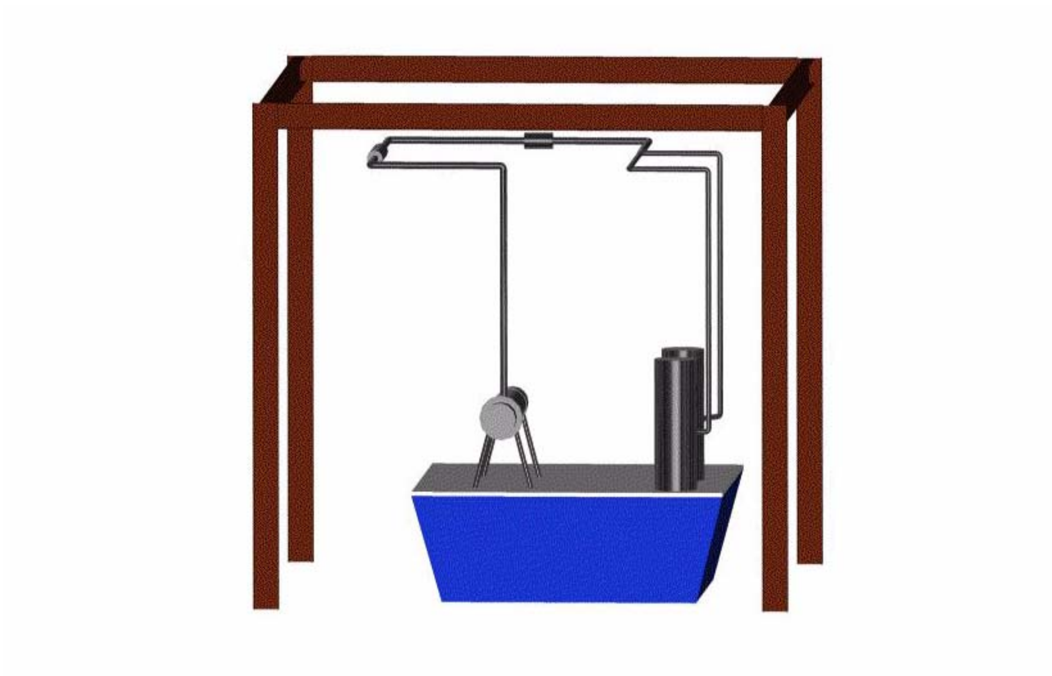


Figure 2-14 Experimental Setup

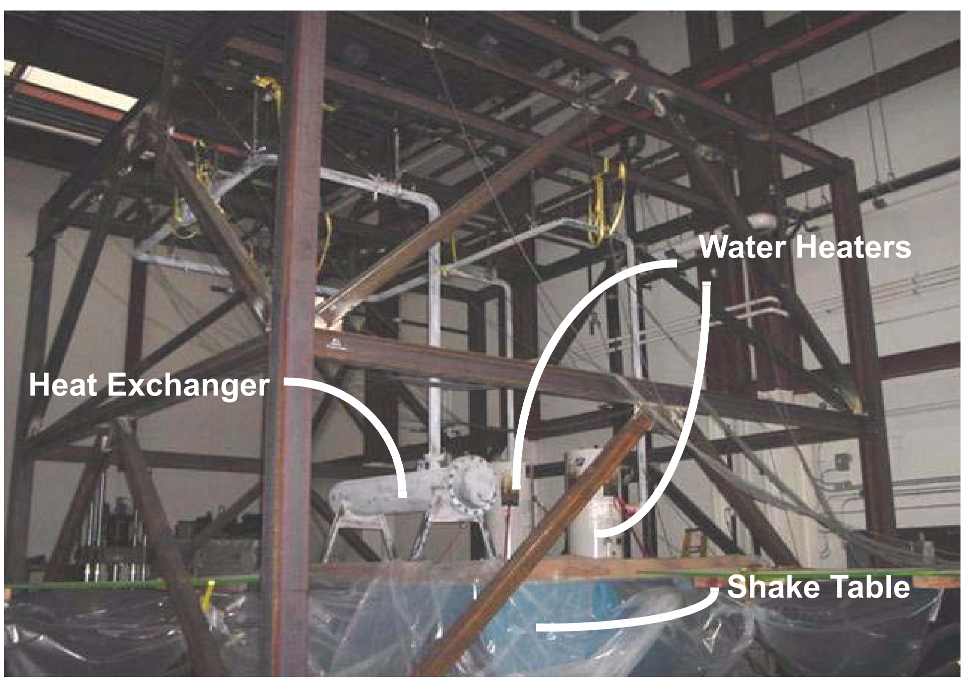


Figure 2-15 Experimental Setup

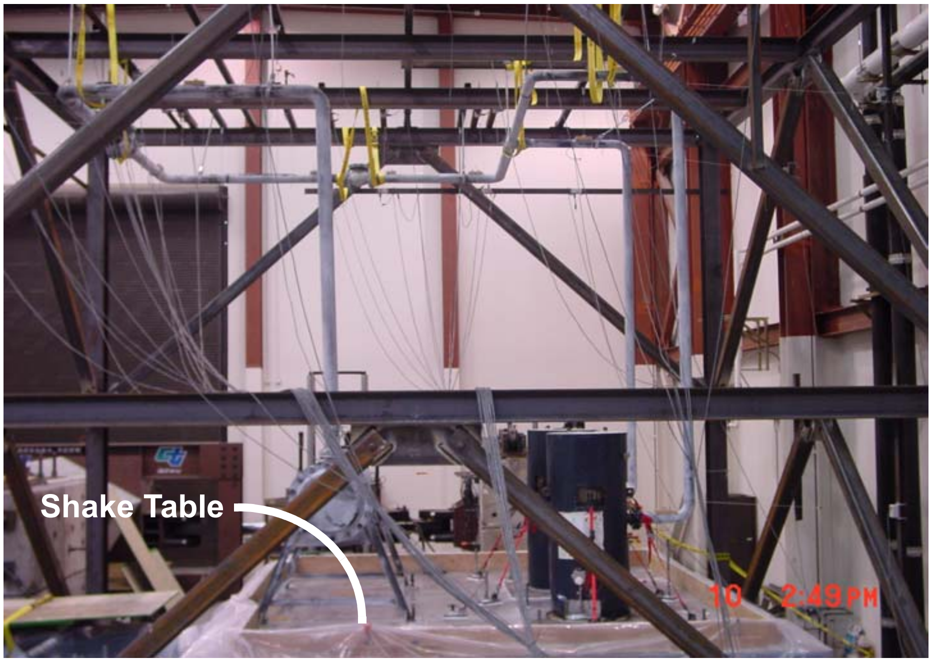


Figure 2-16 North View of Experimental Setup

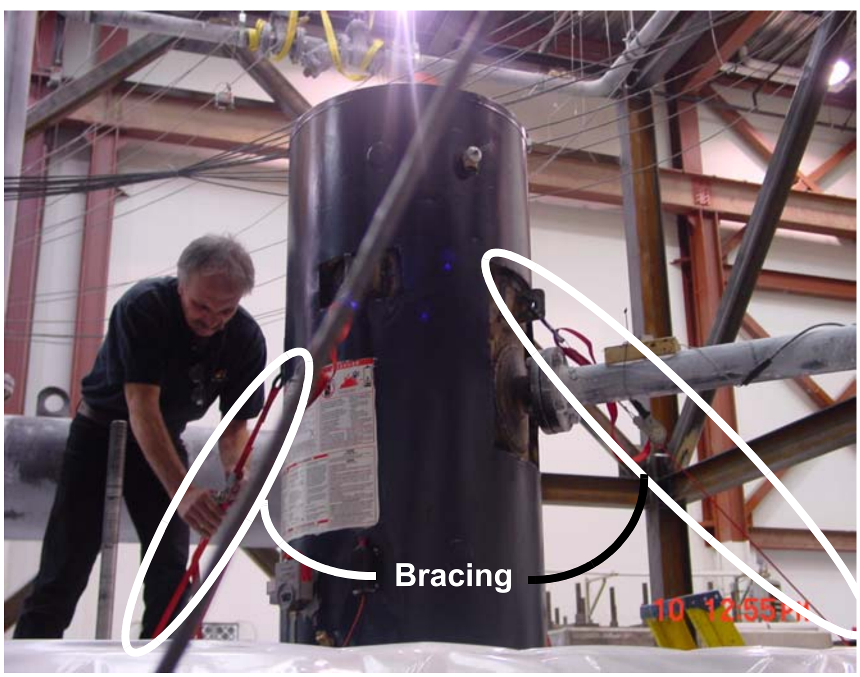


Figure 2-17 Bracing Water Heater to Table

2.4 Instrumentation Plan

The instrumentation plan, seen in figures 2-18 and 2-19, was developed for the experiment to accurately capture the physical response of the system. The callout numbers on the instrumentation plan will be used when referencing instruments in this document. The instrumentation consisted of 29 Celesco (±20 inch [±51 cm] stroke) displacement transducers and 16 Kinemetrics (±4g) accelerometers. The displacement transducers were anchored to the stationary frame that sat outside of the shake table. This permitted direct measurements of absolute displacements.

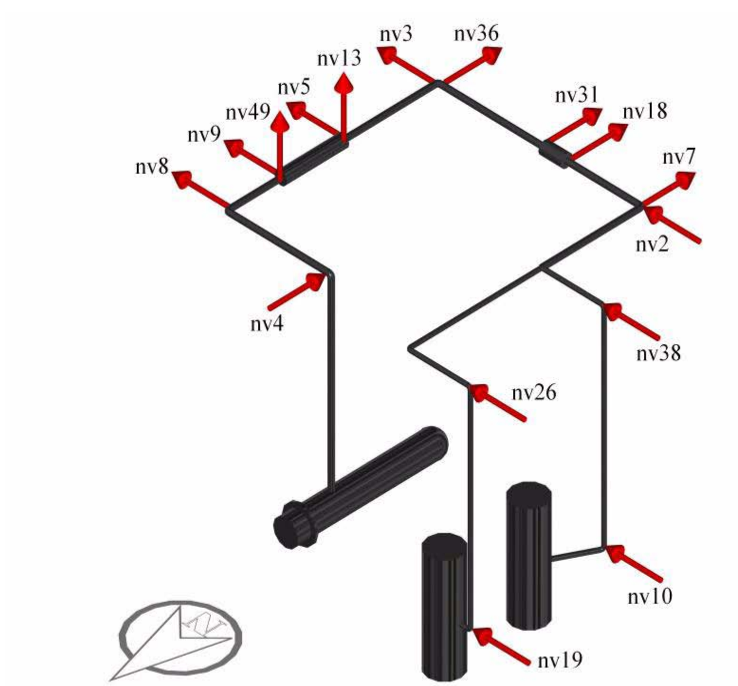


Figure 2-18 Accelerometer Instrumentation Plan

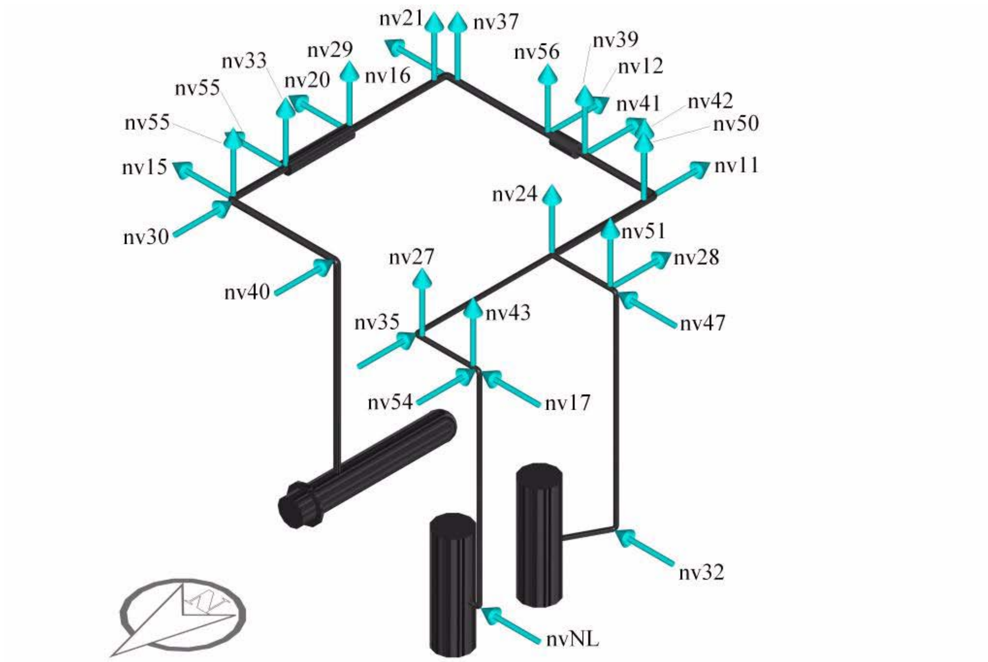


Figure 2-19 Linear Transducer Instrumentation Plan

For the threaded set of experiments, strain gauges were mounted on the vertical hanger rods. The strain gauge layout can be seen in figure 2-21. A typical installation can be seen in figure 2-20. By taking the average of the two signals, the bending strains were cancelled and the axial strains were captured. These axial strains can be used to determine the amount of axial force being transmitted through the vertical hanger rod.

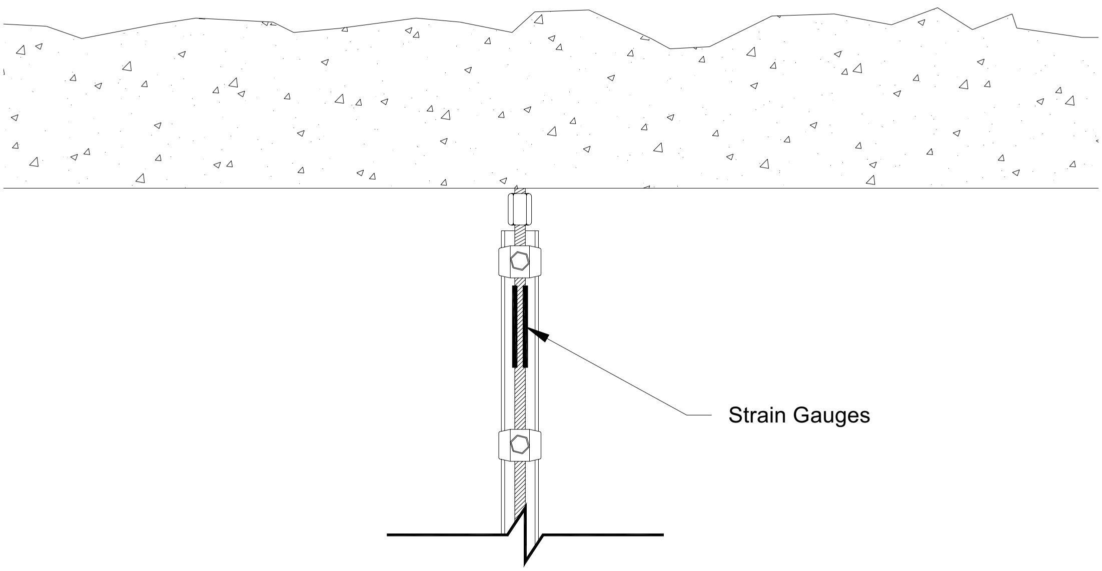


Figure 2-20 Strain Gauge Layout Plan

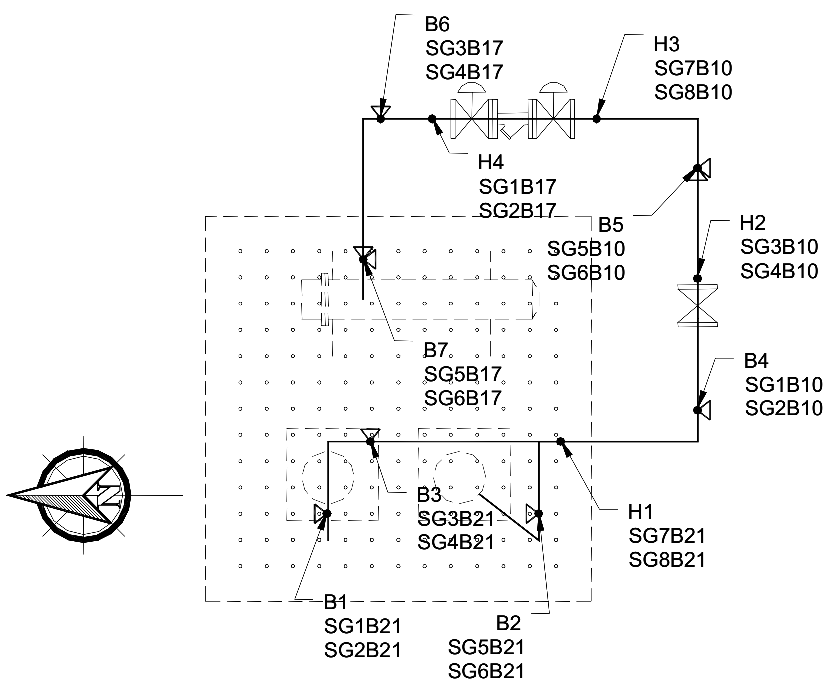


Figure 2-21 Strain Gauge Layout Plan

2.5 Input Motion Generation

This section will discuss the development of the input motions and the experimental protocol. The system was tested to meet the ICBO AC156 Acceptance Criteria for Seismic Qualification Testing of Nonstructural Components (ICBO 2000). AC156 can be found in Appendix B. AC156 specifies a required response spectrum (RRS) for both the 2000 IBC and the 1997 UBC. According to AC156, the component being tested must be subjected to a synthetic input motion that conforms to the RRS. In this experiment, the system was subjected to increasing levels of the synthetic input motions up to the full motion.

2.5.1 Required Response Spectrum

To derive the RRS, AC156 begins with Equation 2.1, which is the total design lateral force as specified in Equation 32-2 from the 1997 UBC.

$$F_p = \frac{a_p C_a}{R_p / I_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{2.1}$$

where:

F_p = total design lateral seismic force
a_p = in structure component amplification factor, ranges from 1.0 to 2.5 (1997 UBC Table 16-O)
C_a = seismic coefficient (1997 UBC Table 16-Q)
R_p = component response modification factor, ranges from 1.0 to 4.0 (1997 UBC Table 16-O)
I_p = importance factor, ranges from 1.0 to 1.5 (1997 UBC Table 16-K)
h_x = element or component attachment elevation with respect to grade
h_r = structure roof elevation with respect to grade
W_p = weight of the component

AC156 assumes that the ratio (R_p/I_p) represents the allowable inelastic energy absorption capacity of the equipment's force-resisting system and is considered to be a design reduction factor. Therefore, AC156 sets (R_p/I_p) equal to 1.

AC156 then defines flexible and rigid equipment. For equipment with natural frequencies less than 16.7 Hz, the equipment is considered flexible ($a_p = 2.5$), which corresponds to the strong portion of the RRS. For equipment with natural frequencies greater than 16.7 Hz, the equipment is considered rigid ($a_p = 1$), which corresponds to the zero period acceleration.

Using these two definitions and dividing both sides of Equation 2.1 by the weight of the component, Equations 2.2 and 2.3 are derived.

$$A_{FLX} = 2.5C_a\left(1 + 3\frac{h_x}{h_r}\right) < 4C_a \tag{2.2}$$

$$A_{RIG} = C_a\left(1 + 3\frac{h_x}{h_r}\right) \tag{2.3}$$

where:

A_{FLX} = horizontal spectral acceleration for flexible equipment
A_{RIG} = the horizontal spectral acceleration for rigid equipment

For this research, a C_a of 0.66 was calculated. This is based off of soil type S_D, seismic source type A and near source factor N_a of 1.5. The ratio h_x/h_r was set to 1, representing equipment mounted at the roof of a building. Setting h_x/h_r to 1 also represents a worst case scenario. Using these factors, A_{FLX} and A_{RIG} both become 2.64g.

The final RRS is given in figure 2-22. The response spectrum is calculated at 5% damping.

2.5.2 SIMQKE and RSCTH

Two motions were generated in order to subject the system to a variety of different excitations. The programs Simulated Earthquake Motions Compatible with Prescribed Response Spectra, "SIMQKE I", (Gasparini and Vanmarcke 1976) and Response Spectrum Compatible Time

Histories, “RSCTH”, (Halldorsson et al. 2002) were used to develop the synthetic input motions used in the experimental protocol. AC156 requires that the synthetic input motion have a build, hold and decay envelope of 5, 15 and 10 seconds, respectively. The motion must be non-stationary broadband random excitations having an energy content from 1 to 33 Hz (0.03 to 1 seconds).

The motion that is produced from SIMQKE meets all of the demands of AC156 while the RSCTH motion meets all but the build, hold and decay requirement. The SIMQKE acceleration and displacement response spectrums, and FFT can be seen in figures 2-22 - 2-24.

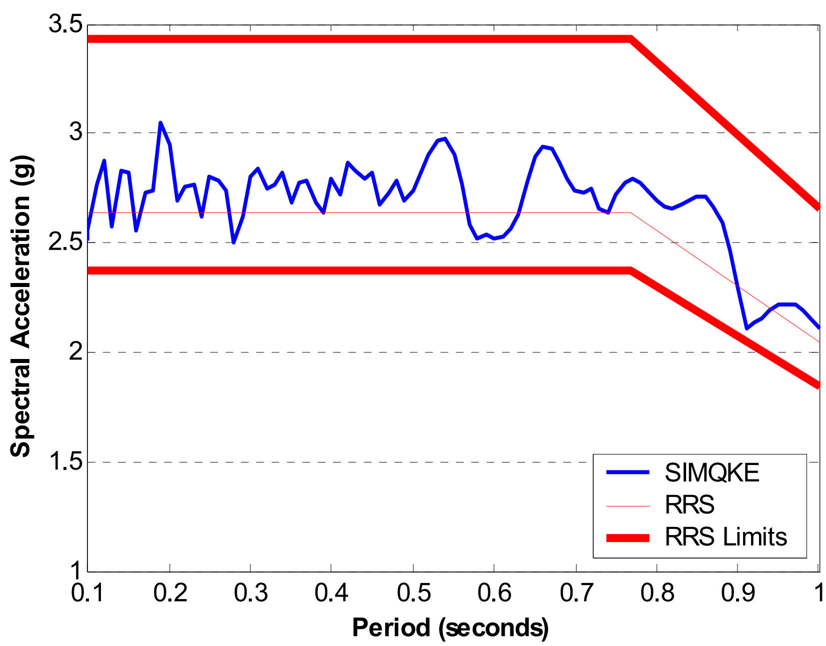


Figure 2-22 Required Response and Generated Input Motion Response Spectra

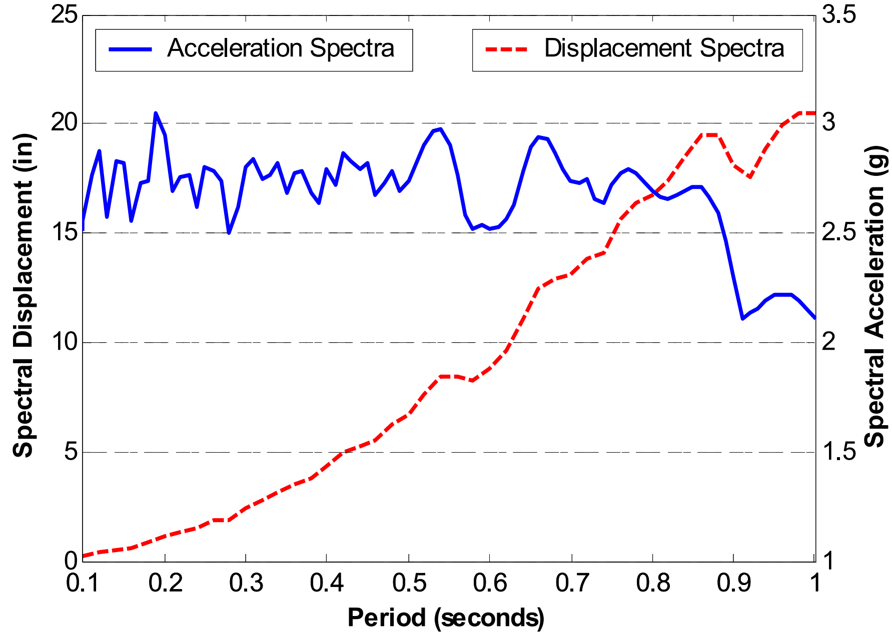


Figure 2-23 SIMQKE Input Motion Response Spectra

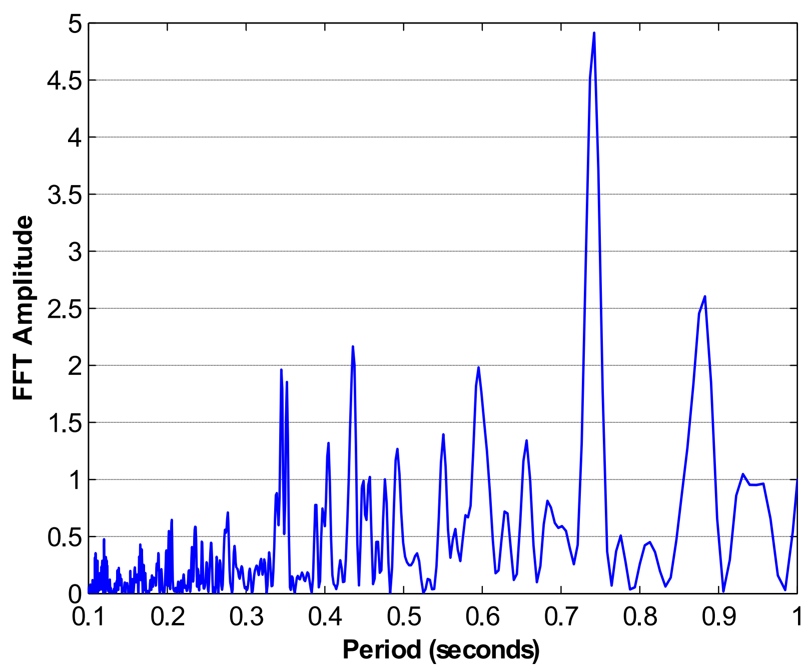


Figure 2-24 SIMQKE Input Motion FFTs

For acceleration, velocity and displacement time histories of the SIMQKE input motion used in the experimental protocol, see figures 2-25 - 2-27. For acceleration, velocity and displacement time histories of the RSCTH input motion used in the experimental protocol, see figures 2-28 - 2-30.

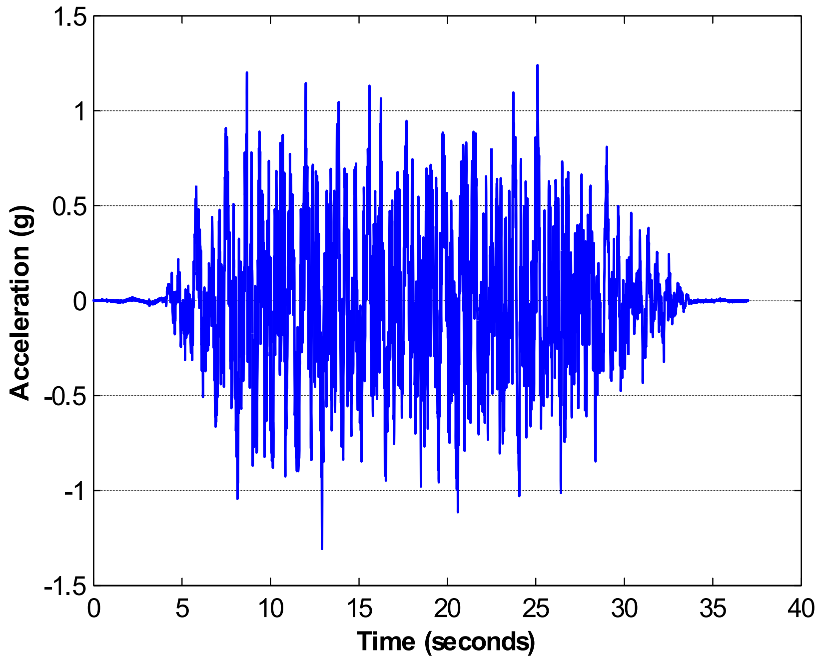


Figure 2-25 SIMQKE Input Motion Acceleration Time History

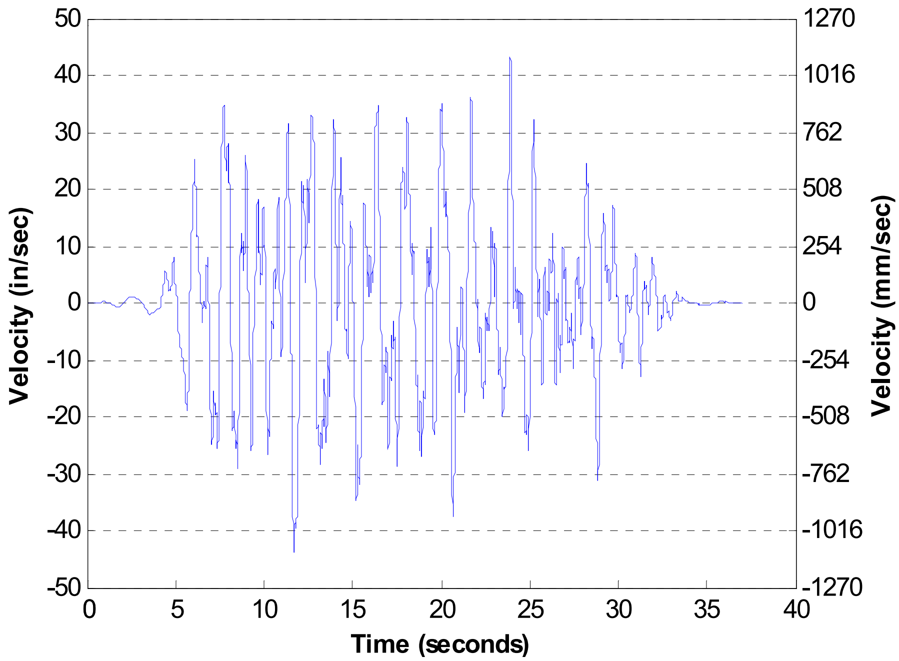


Figure 2-26 SIMQKE Input Motion Velocity Time History

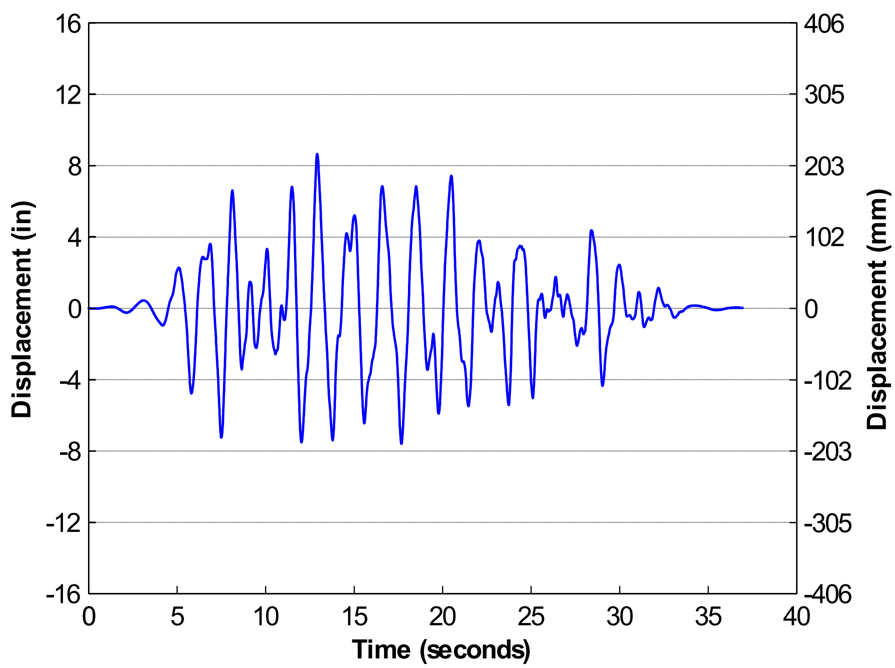


Figure 2-27 SIMQKE Input Motion Displacement Time History

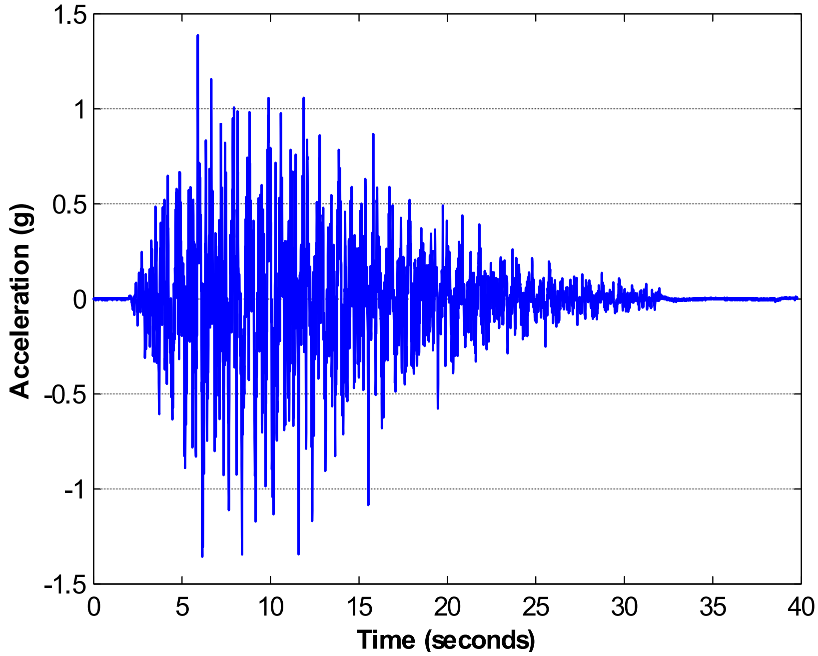


Figure 2-28 RSCTH Input Motion Acceleration Time History

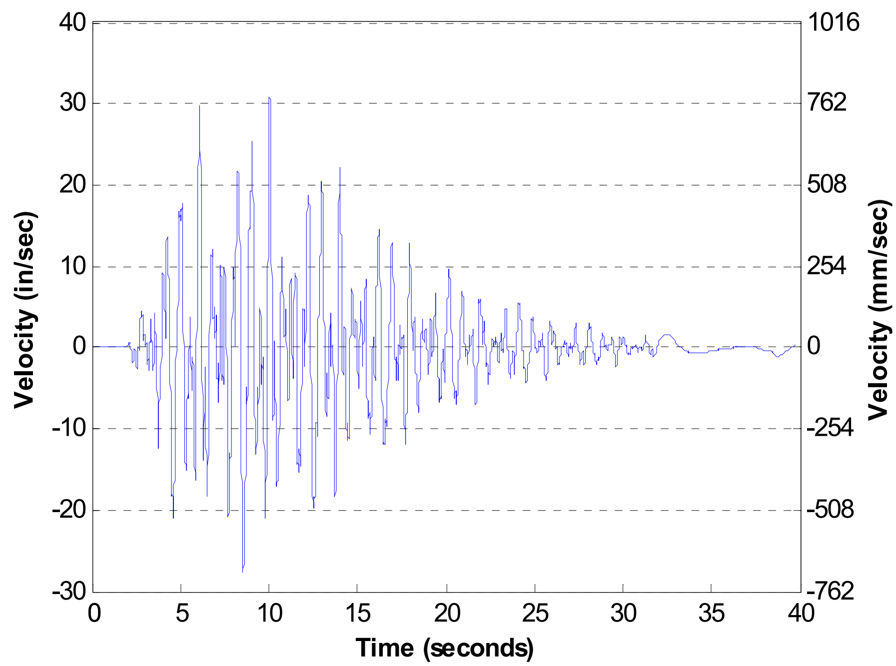


Figure 2-29 RSCTH Input Motion Velocity Time History

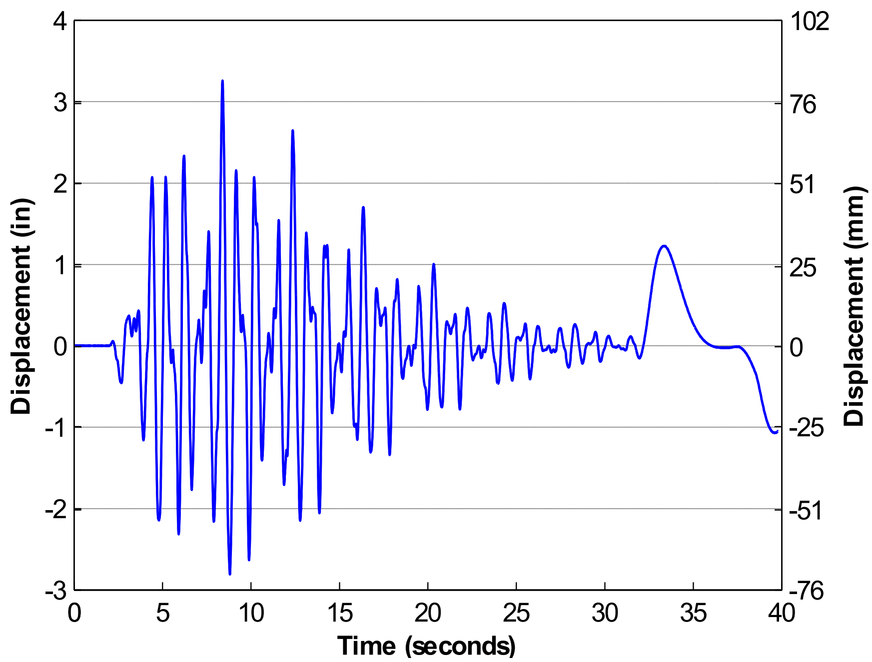


Figure 2-30 RSCTH Input Motion Displacement Time History

2.6 Experimental Protocol

Tables 2-3 - 2-10 illustrate the experimental protocol for all subassemblies. The experimental protocol started out with 5% of the full strength input motion. The system was subjected to that 5% motion in the East/West, North/South, and biaxially (equal components in East/West and North/South). The motions were incremented in steps of 5% up to 50% where the motion was then incremented by 10%. The rest of this thesis will discuss only the SIMQKE input motion as it was the primary motion used in the experiments.

Number	Condition	Motion Used	Strength(%)	Direction
1	Braced - Welded	SIMQKE	5	EW
2	Braced - Welded	SIMQKE	5	NS
3	Braced - Welded	SIMQKE	5	Biax
4	Braced - Welded	SIMQKE	10	EW
5	Braced - Welded	SIMQKE	10	NS
6	Braced - Welded	SIMQKE	10	Biax
7	Braced - Welded	Sweep	100	BiaxSweep - A
8	Braced - Welded	Sweep	100	EWSeep -B
9	Braced - Welded	Sweep	100	NSSweep -C
10	Braced - Welded	SIMQKE	15	EW
11	Braced - Welded	SIMQKE	15	NS
12	Braced - Welded	SIMQKE	15	Biax
13	Braced - Welded	RSCTH	5	EW
14	Braced - Welded	RSCTH	5	NS
15	Braced - Welded	RSCTH	5	Biax
16	Braced - Welded	RSCTH	10	EW
17	Braced - Welded	RSCTH	10	NS
18	Braced - Welded	RSCTH	10	Biax
19	Braced - Welded	RSCTH	15	EW
20	Braced - Welded	RSCTH	15	NS
21	Braced - Welded	RSCTH	15	Biax
22	Braced - Welded	SIMQKE	15	EW REDO
23	Braced - Welded	SIMQKE	15	NS REDO
24	Braced - Welded	SIMQKE	15	Biax REDO
25	Braced - Welded	SIMQKE	20	EW
26	Braced - Welded	SIMQKE	20	NS - LONG
27	Braced - Welded	SIMQKE	20	Biax
28	Braced - Welded	SIMQKE	25	EW
29	Braced - Welded	SIMQKE	25	NS
30	Braced - Welded	SIMQKE	25	Biax
31	Braced - Welded	SIMQKE	30	EW
32	Braced - Welded	SIMQKE	30	NS
33	Braced - Welded	SIMQKE	30	Biax
34	Braced - Welded	SIMQKE	35	EW
35	Braced - Welded	SIMQKE	35	NS
36	Braced - Welded	SIMQKE	35	Biax
37	Braced - Welded	SIMQKE	40	EW

able 2-3 Experimental Protocol for Specimen I

Number	Condition	Motion Used	Strength(%)	Direction
38	Braced - Welded	SIMQKE	40	NS
39	Braced - Welded	SIMQKE	40	Biax
40	Braced - Welded	SIMQKE	45	EW
41	Braced - Welded	SIMQKE	45	NS
42	Braced - Welded	SIMQKE	45	Biax
43	Braced - Welded	SIMQKE	50	EW
44	Braced - Welded	SIMQKE	50	NS
45	Braced - Welded	SIMQKE	50	Biax
46	Braced - Welded	RSCTH	50	EW - A
47	Braced - Welded	RSCTH	50	NS - B
48	Braced - Welded	RSCTH	50	Biax - C
49	Braced - Welded	SIMQKE	60	EW
50	Braced - Welded	SIMQKE	60	NS
51	Braced - Welded	SIMQKE	60	Biax
52	Braced - Welded	SIMQKE	70	EW
53	Braced - Welded	SIMQKE	70	NS
54	Braced - Welded	SIMQKE	70	Biax
55	Braced - Welded	SIMQKE	80	EW - MOD
56	Braced - Welded	SIMQKE	80	NS - MOD
57	Braced - Welded	SIMQKE	80	Biax - MOD
58	Braced - Welded	SIMQKE	90	EW
59	Braced - Welded	SIMQKE	90	NS
60	Braced - Welded	SIMQKE	90	Biax
61	Braced - Welded	SIMQKE	100	EW
62	Braced - Welded	SIMQKE	100	NS
63	Braced - Welded	SIMQKE	100	Biax
64	Braced - Welded	SIMQKE - Peter	100	EW
65	Braced - Welded	7 inch circle	100	NA
66	Braced - Welded	8 inch circle	100	NA
67	Braced - Welded	9 inch circle	100	NA
68	Braced - Welded	10 inch circle	100	NA

Table 2-4 Experimental Protocol for Specimen I, cont'd

Number	Condition	Motion Used	Strength(%)	Direction
69	Unbraced - Welded	SIMQKE	10	EW
70	Unbraced - Welded	SIMQKE	10	NS
71	Unbraced - Welded	SIMQKE	10	Biax
72	Unbraced - Welded	Sweep	100	NS - A
73	Unbraced - Welded	Sweep	100	EW - B
74	Unbraced - Welded	SIMQKE	15	EW
75	Unbraced - Welded	SIMQKE	15	NS
76	Unbraced - Welded	SIMQKE	15	Biax
77	Unbraced - Welded	SIMQKE	20	EW
78	Unbraced - Welded	SIMQKE	20	NS
79	Unbraced - Welded	SIMQKE	20	Biax
80	Unbraced - Welded	SIMQKE	25	EW
81	Unbraced - Welded	SIMQKE	25	NS
82	Unbraced - Welded	SIMQKE	25	Biax
83	Unbraced - Welded	SIMQKE	30	EW
84	Unbraced - Welded	SIMQKE	30	NS
85	Unbraced - Welded	SIMQKE	30	Biax
86	Unbraced - Welded	SIMQKE	35	EW
87	Unbraced - Welded	SIMQKE	35	NS
88	Unbraced - Welded	SIMQKE	35	Biax
89	Unbraced - Welded	SIMQKE	40	EW
90	Unbraced - Welded	SIMQKE	40	NS
91	Unbraced - Welded	SIMQKE	40	Biax
92	Unbraced - Welded	SIMQKE	45	EW
93	Unbraced - Welded	SIMQKE	45	NS
94	Unbraced - Welded	SIMQKE	45	Biax
95	Unbraced - Welded	SIMQKE	50	EW
96	Unbraced - Welded	SIMQKE	50	NS
97	Unbraced - Welded	SIMQKE	50	Biax
98	Unbraced - Welded	RSCTH	50	EW - A
99	Unbraced - Welded	RSCTH	50	NS - B
100	Unbraced - Welded	RSCTH	50	Biax - C
101	Unbraced - Welded	SIMQKE	60	EW
102	Unbraced - Welded	SIMQKE	60	NS
103	Unbraced - Welded	SIMQKE	60	Biax
104	Unbraced - Welded	SIMQKE	70	EW
105	Unbraced - Welded	SIMQKE	70	NS

Table 2-5 Experimental Protocol for Specimen II

Number	Condition	Motion Used	Strength(%)	Direction
106	Unbraced - Welded	SIMQKE	70	Biax
107	Unbraced - Welded	SIMQKE	80	EW
108	Unbraced - Welded	SIMQKE	80	NS
109	Unbraced - Welded	SIMQKE	80	Biax
110	Unbraced - Welded	SIMQKE	90	EW
111	Unbraced - Welded	SIMQKE	90	NS
112	Unbraced - Welded	SIMQKE	90	Biax
113	Unbraced - Welded	SIMQKE	100	EW
114	Unbraced - Welded	SIMQKE	100	NS
115	Unbraced - Welded	SIMQKE	100	Biax
116	Unbraced - Welded	SIMQKE - v2	100	EW
117	Unbraced - Welded	SIMQKE - v2	100	NS
118	Unbraced - Welded	SIMQKE - v2	100	Biax
119	Unbraced - Welded	RSCTH	100	EW
120	Unbraced - Welded	RSCTH	100	NS
121	Unbraced - Welded	RSCTH	100	Biax

Table 2-6 Experimental Protocol for Specimen II, cont'd

Number	Condition	Motion Used	Strength(%)	Direction
122	Braced - Threaded	SIMQKE	5	EW
123	Braced - Threaded	SIMQKE	5	NS
124	Braced - Threaded	SIMQKE	5	Biax
125	Braced - Threaded	SIMQKE	10	EW
126	Braced - Threaded	SIMQKE	10	NS
127	Braced - Threaded	SIMQKE	10	Biax
128	Braced - Threaded	Sweep	100	Biax
129	Braced - Threaded	Sweep	100	EW
130	Braced - Threaded	Sweep	100	EW
131	Braced - Threaded	Sweep	100	NS
132	Braced - Threaded	SIMQKE	15	EW
133	Braced - Threaded	SIMQKE	15	NS
134	Braced - Threaded	SIMQKE	15	Biax
135	Braced - Threaded	RSCTH	5	EW
136	Braced - Threaded	RSCTH	5	NS
137	Braced - Threaded	RSCTH	5	Biax
138	Braced - Threaded	RSCTH	10	EW
139	Braced - Threaded	RSCTH	10	NS
140	Braced - Threaded	RSCTH	10	Biax
141	Braced - Threaded	RSCTH	15	EW
142	Braced - Threaded	RSCTH	15	NS
143	Braced - Threaded	RSCTH	15	Biax
144	Braced - Threaded	SIMQKE	20	EW
145	Braced - Threaded	SIMQKE	20	NS
146	Braced - Threaded	SIMQKE	20	Biax
147	Braced - Threaded	SIMQKE	25	EW
148	Braced - Threaded	SIMQKE	25	NS
149	Braced - Threaded	SIMQKE	25	Biax
150	Braced - Threaded	SIMQKE	30	EW
151	Braced - Threaded	SIMQKE	30	NS
152	Braced - Threaded	SIMQKE	30	Biax
153	Braced - Threaded	SIMQKE	35	EW
154	Braced - Threaded	SIMQKE	35	NS
155	Braced - Threaded	SIMQKE	35	Biax
156	Braced - Threaded	SIMQKE	40	EW
157	Braced - Threaded	SIMQKE	40	NS
158	Braced - Threaded	SIMQKE	40	Biax

Table 2-7 Experimental Protocol for Specimen III

Number	Condition	Motion Used	Strength(%)	Direction
159	Braced - Threaded	SIMQKE	45	EW
160	Braced - Threaded	SIMQKE	45	NS
161	Braced - Threaded	SIMQKE	45	Biax
162	Braced - Threaded	SIMQKE	50	EW
163	Braced - Threaded	SIMQKE	50	NS
164	Braced - Threaded	SIMQKE	50	Biax
165	Braced - Threaded	RSCTH	50	EW
166	Braced - Threaded	RSCTH	50	NS
167	Braced - Threaded	RSCTH	50	Biax
168	Braced - Threaded	SIMQKE	60	EW
169	Braced - Threaded	SIMQKE	60	NS
170	Braced - Threaded	SIMQKE	60	Biax
171	Braced - Threaded	SIMQKE	70	EW
172	Braced - Threaded	SIMQKE	70	NS
173	Braced - Threaded	SIMQKE	70	Biax
174	Braced - Threaded	SIMQKE	80	EW
175	Braced - Threaded	SIMQKE	80	NS
176	Braced - Threaded	SIMQKE	80	Biax
177	Braced - Threaded	SIMQKE	90	EW
178	Braced - Threaded	SIMQKE	90	NS
179	Braced - Threaded	SIMQKE	90	Biax
180	Braced - Threaded	SIMQKE	100	EW

Table 2-8 Experimental Protocol for Specimen III, cont'd

Number	Condition	Motion Used	Strength(%)	Direction
181	Unbraced - Threaded	SIMQKE	10	EW
182	Unbraced - Threaded	SIMQKE	10	NS
183	Unbraced - Threaded	SIMQKE	10	Biax
184	Unbraced - Threaded	Sweep	100	EW
185	Unbraced - Threaded	Sweep	100	NS
186	Unbraced - Threaded	SIMQKE	15	EW
187	Unbraced - Threaded	SIMQKE	15	NS
188	Unbraced - Threaded	SIMQKE	15	Biax
189	Unbraced - Threaded	SIMQKE	20	EW
190	Unbraced - Threaded	SIMQKE	20	NS
191	Unbraced - Threaded	SIMQKE	20	Biax
192	Unbraced - Threaded	SIMQKE	25	EW
193	Unbraced - Threaded	SIMQKE	25	NS
194	Unbraced - Threaded	SIMQKE	25	Biax
195	Unbraced - Threaded	SIMQKE	30	EW
196	Unbraced - Threaded	SIMQKE	30	NS
197	Unbraced - Threaded	SIMQKE	30	Biax
198	Unbraced - Threaded	SIMQKE	35	EW
199	Unbraced - Threaded	SIMQKE	35	NS
200	Unbraced - Threaded	SIMQKE	35	Biax
201	Unbraced - Threaded	SIMQKE	40	EW
202	Unbraced - Threaded	SIMQKE	40	NS
203	Unbraced - Threaded	SIMQKE	40	Biax
204	Unbraced - Threaded	SIMQKE	45	EW
205	Unbraced - Threaded	SIMQKE	45	NS
206	Unbraced - Threaded	SIMQKE	45	Biax
207	Unbraced - Threaded	SIMQKE	50	EW
208	Unbraced - Threaded	SIMQKE	50	NS
209	Unbraced - Threaded	SIMQKE	50	Biax
210	Unbraced - Threaded	RSCTH	50	EW
211	Unbraced - Threaded	RSCTH	50	NS
212	Unbraced - Threaded	RSCTH	50	Biax
213	Unbraced - Threaded	SIMQKE	60	EW
214	Unbraced - Threaded	SIMQKE	60	NS
215	Unbraced - Threaded	SIMQKE	60	Biax
216	Unbraced - Threaded	SIMQKE	70	EW
217	Unbraced - Threaded	SIMQKE	70	NS

Table 2-9 Experimental Protocol for Specimen IV

Number	Condition	Motion Used	Strength(%)	Direction
218	Unbraced - Threaded	SIMQKE	70	Biax
219	Unbraced - Threaded	SIMQKE	80	EW
220	Unbraced - Threaded	SIMQKE	80	NS
221	Unbraced - Threaded	SIMQKE	80	Biax
222	Unbraced - Threaded	SIMQKE	90	EW
223	Unbraced - Threaded	SIMQKE	90	NS
224	Unbraced - Threaded	SIMQKE	90	Biax
225	Unbraced - Threaded	SIMQKE	100	EW
226	Unbraced - Threaded	SIMQKE	100	NS
227	Unbraced - Threaded	SIMQKE	100	Biax
228	Unbraced - Threaded	SIMQKE	100	EW
229	Unbraced - Threaded	SIMQKE	100	NS
230	Unbraced - Threaded	SIMQKE	100	Biax

Table 2-10 Experimental Protocol for Specimen IV, cont'd

SECTION 3

EXPERIMENTAL RESULTS

3.1 Introduction

This chapter presents the results of the experiments on hospital piping subassemblies. The component (threaded rod and cable) tests are discussed first, followed by the subassembly tests. Overall performance of each system is discussed followed by a presentation of selected instrument responses. Due to the large volume of data obtained, only the instrument that exhibited the maximum response from the maximum intensity input motions are presented. Some of the instrument maxima are summarized in tabular format. These instruments and their descriptions are presented in table 3-1. Only the results from the East/West and North/South 100% SIMQKE experiments are discussed in this chapter.

Name	Type	Orientation	Description
nv15	Displacement	EW	Second Elbow after HX
nv17	Displacement	EW	Above WH
nv20	Displacement	EW	Vertical on group of three valves
nv40	Displacement	NS	Above HX
nv41	Displacement	NS	Near single valve
nv54	Displacement	NS	Above WH
nv27	Displacement	Vertical	Third elbow after WH
nv21	Displacement	Vertical	Between groups of valves, on elbow
nv29	Displacement	Vertical	On group of three valves
nv8	Acceleration	EW	Second Elbow after HX
nv26	Acceleration	EW	Above WH
nv5	Acceleration	EW	Vertical on group of three valves
nv18	Acceleration	NS	Near single valve
nv4	Acceleration	NS	Above HX
nv13	Acceleration	Vertical	On group of three valves

Table 3-1 Selected Instruments

The results are organized by specimen and response. Instruments oriented parallel with the direction of the input motion are referred to as in parallel instruments, whereas the instruments oriented perpendicular to the input motion are referred to transverse. The dynamic characteristics, displacements and accelerations are discussed for all subassemblies in Appendix D. Forces in the hanger rods are discussed only for the threaded subassemblies.

All instrument time histories were recorded in reference to a stationary frame. Instrument displacement time histories, when parallel to the input motion, are occasionally presented as relative displacements. Relative displacements were calculated by subtracting the table displacement from the absolute displacement time history. Displacement time histories, when transverse, are given as absolute.

Forces in the hanger rods were obtained for only the threaded subassembly experiments. Forces in the hanger rods were computed based on the strains obtained during the system tests and the modulus of elasticity that was computed from the component tests. The strain gauges mounted on the threaded rods were used to calculate axial forces during the excitation. Using a Young's Modulus (E) value obtained from the rod tensile tests, and the strains obtained during the excitation, a stress was calculated by taking the average strain in the rod and multiplying it by E. From the stress, an axial force can be calculated by multiplying the stress by the net tensile area of the rod.

The strains in the rods obtained from the experiments are a minor portion of this research. The averaging of two strain gauges is not the most accurate method for obtaining stress, but were well suited to obtain a basic understanding of the forces in the rods.

Comparative plots are presented in order to evaluate the effects of bracing, fittings and direction of input motion. Drifts are used as an index of damage. Story drifts were calculated by taking the displacement of the table and dividing it by the distance from the table to point of attachment to the steel frame. This distance was 16.5 feet.

3.2 Component Tests

For the component tests, 3 - 5/8 inch diameter rods and 3 - 1/8 inch diameter cables were subjected to tensile tests. Displacement and force were measured. Strain was calculated based on the measured displacement and the measured gauge distance of the components in the tensile test machine. For the cable tests, stress was calculated based on a diameter of 1/8 inch. The cables were tested by threading the cable through the cable support and clamping a screw on top of it.

This is the way that the cable is mounted in the actual subassembly. All of the cables failed at the clamped portion of the cable. This indicates that the clamping effect produces the governing limit state in the failure of the cable.

Table 3-2 summarizes the cable results. Figure 3-1 shows the force/displacement curve for the cables. Stresses and strains were calculated based on the net tensile area of the cable and the gauge distance measured for each cable.

	Cable 1	Cable 2	Cable 3	Average
Displacement at Failure (inches)	0.423	0.491	0.3692	*0.428*
Maximum Force (kips)	1.538	1.67	1.24	*1.483*
Max Strain	0.016	0.0169	0.0128	*0.015*
Max Stress (ksi)	12.500	13.6	10.7	*12.267*

Table 3-2 Summary of Cable Tests

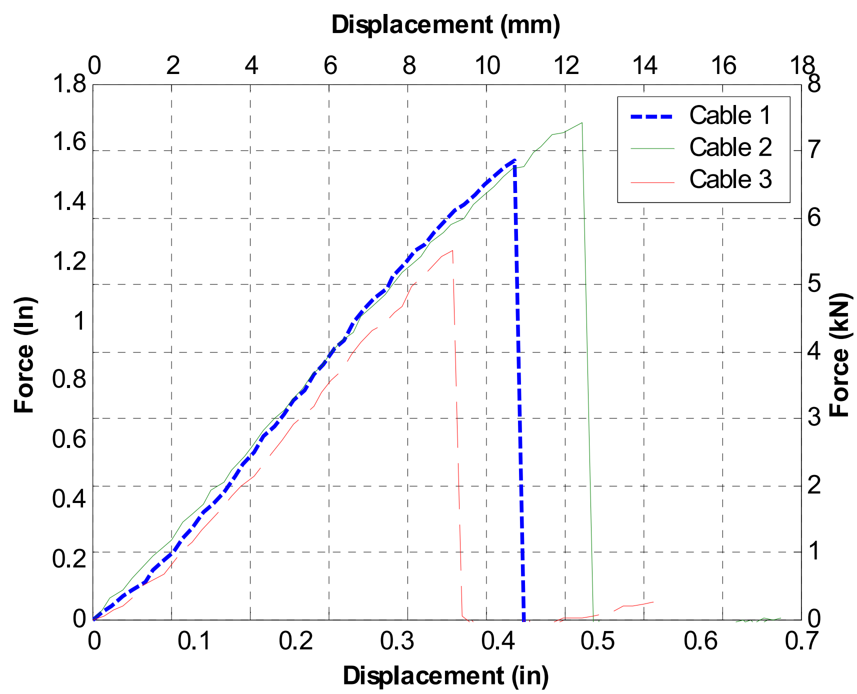


Figure 3-1 Cable Force/Displacement Curve

Table 3-3 summarizes the rod results that were tensile tested. Figure 3-2 shows the force/ displacement curve for the rods. Stresses and strains were calculated based on the net tensile area of the rod and the gauge distance measured for each rod.

	Rod 1	Rod 2	Rod 3	Average
Displacement at Failure (inches)	1.02	0.971	1.06	1.02
Maximum Force (kips)	19.5	18.7	19.3	19.2
Max Strain	0.034	0.0324	0.0353	0.0339
Max Stress (ksi)	86.1	82.5	85.6	84.7
E (ksi)	21866	21275	21732	21624

Table 3-3 Summary of Rod Tests

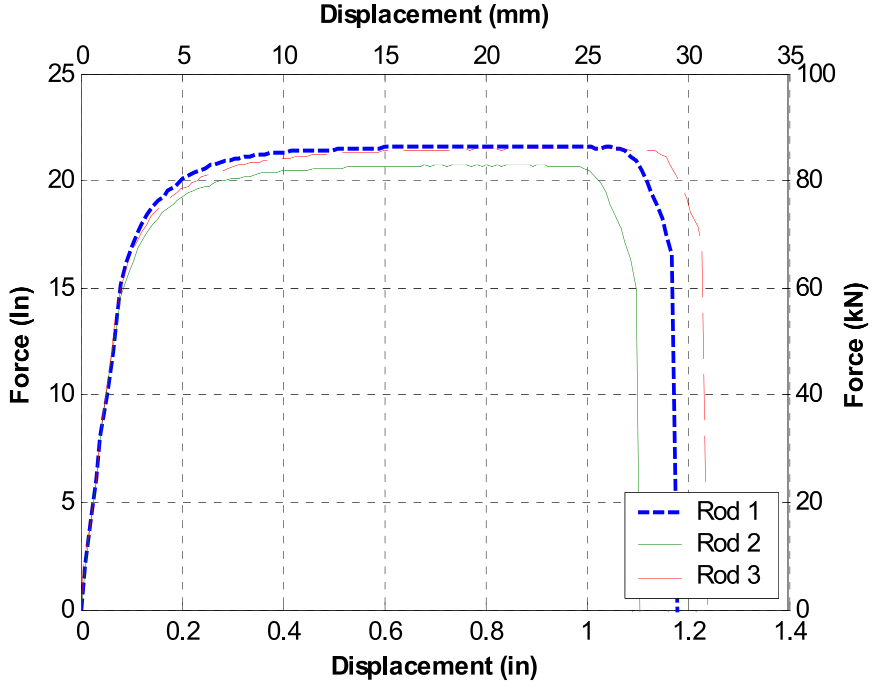


Figure 3-2 Rod Force/Displacement Curve

3.3 Welded Subassembly

The following sections present the experimental results from the braced and unbraced welded subassembly experiments. The welded braced subassembly was subjected to runs 1-68 from the experimental protocol (tables 2-3 and 2-4). The unbraced subassembly was subjected to runs 69-121 (tables 2-5 and 2-6) from the experimental protocol. A comparison of the two systems is made.

3.3.1 Welded Braced Subassembly

3.3.1.1 Displacement Response

For run 61, the maximum parallel displacement of an instrument mounted at the top of the piping system was 3.56 inches (90.4 mm); this occurred at instrument nv17. The maximum transverse displacement occurred at nv30 and was 1.82 inches (46.2 mm). The maximum vertical displacement was 0.99 inches (25.1 mm) and occurred at nv27. The time histories for these instruments can be seen in figures 3-3 - 3-5. The relative displacement time history for nv17 is shown in figure 3-6.

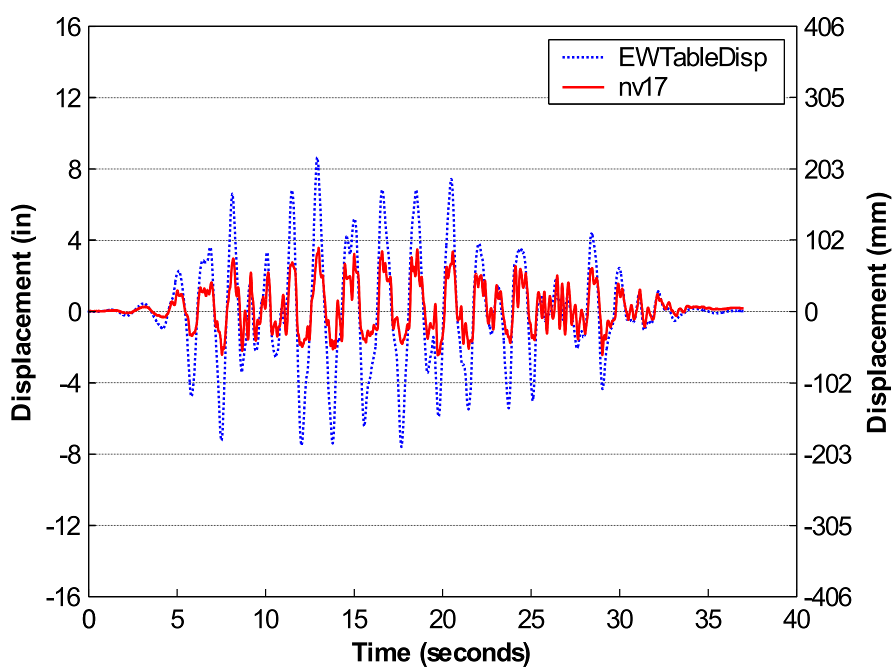


Figure 3-3 Run 61 - Absolute Displacement Time History of nv17

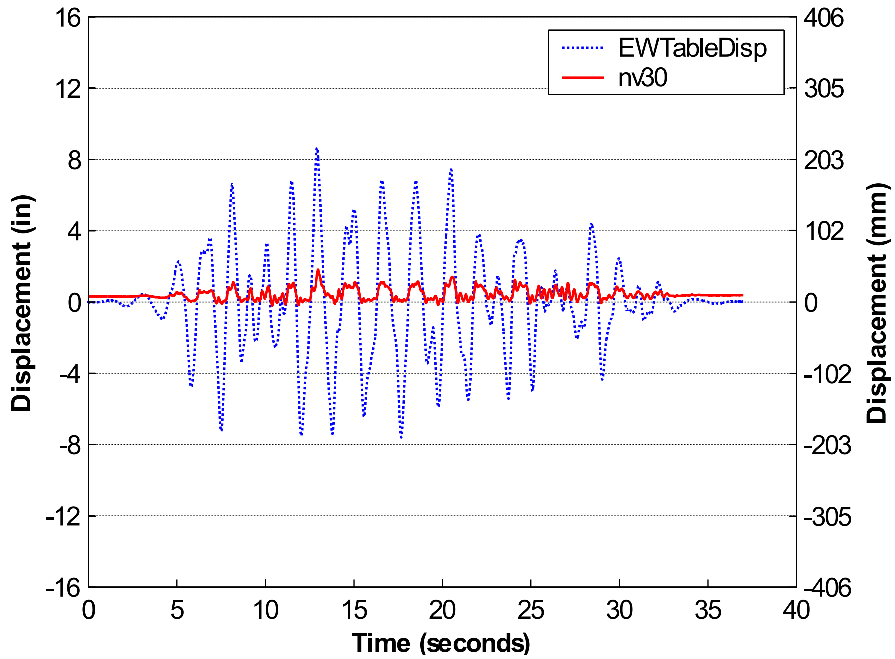


Figure 3-4 Run 61 - Absolute Displacement Time History of nv30

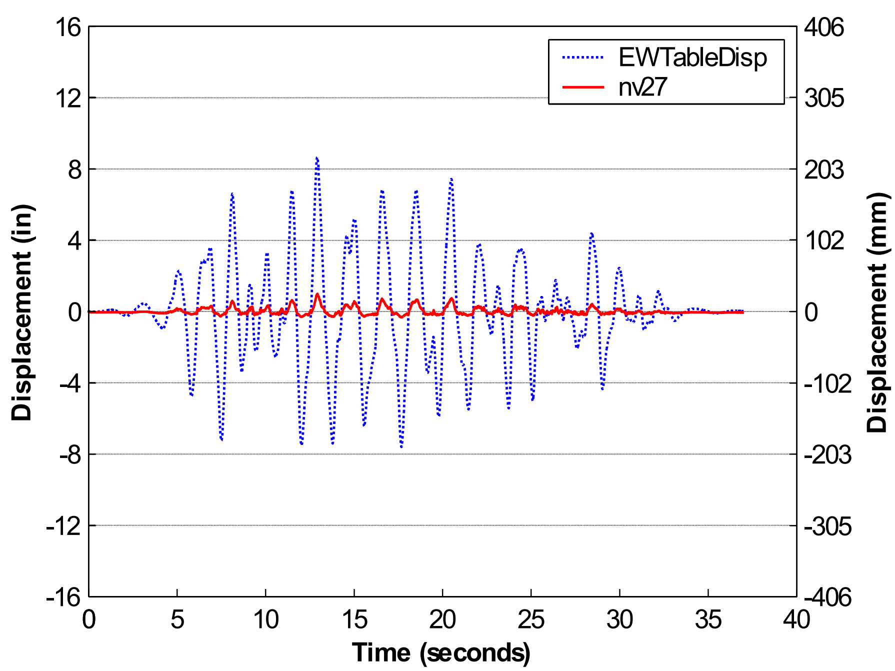


Figure 3-5 Run 61 - Absolute Displacement Time History of nv27

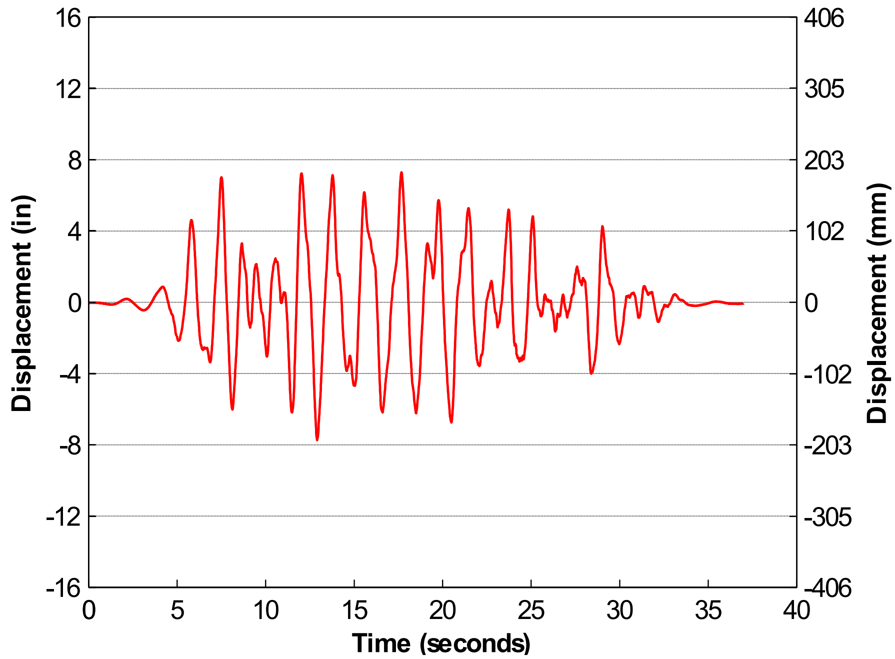


Figure 3-6 Run 61 - Relative Displacement Time History of nv17

For run 62, the maximum parallel displacement occurred at nv40 and was 3.03 inches (77 mm). The maximum transverse displacement was 1.25 inches (31.75 mm) and occurred at instrument nv15. The maximum vertical displacement was 0.43 inches (10.9 mm) and occurred at nv27. The time histories for these instruments can be seen in figures 3-7 - 3-9. The relative displacement time history for nv40 is shown in figure 3-10

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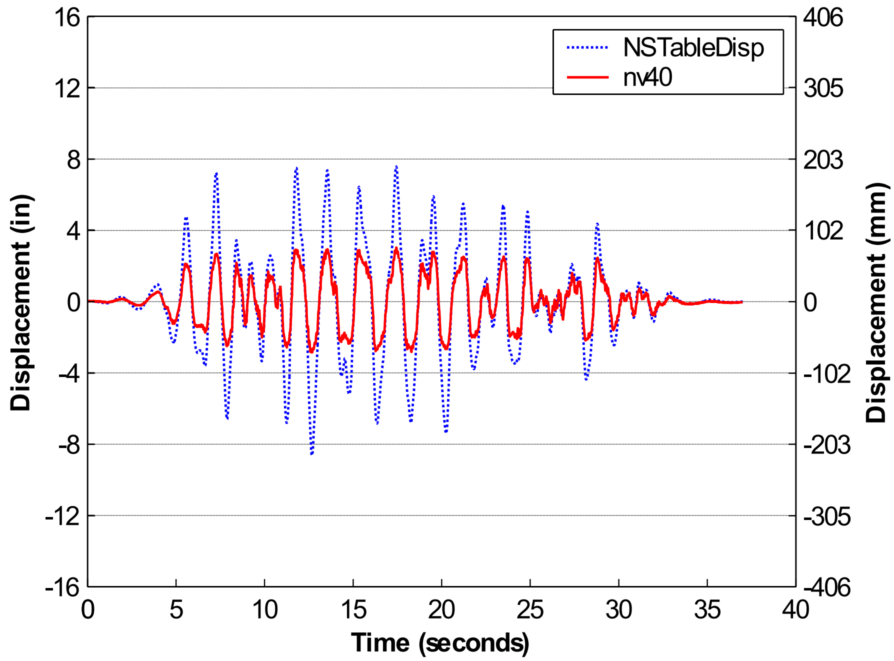

Figure 3-7 Run 62 - Absolute Displacement Time History of nv40

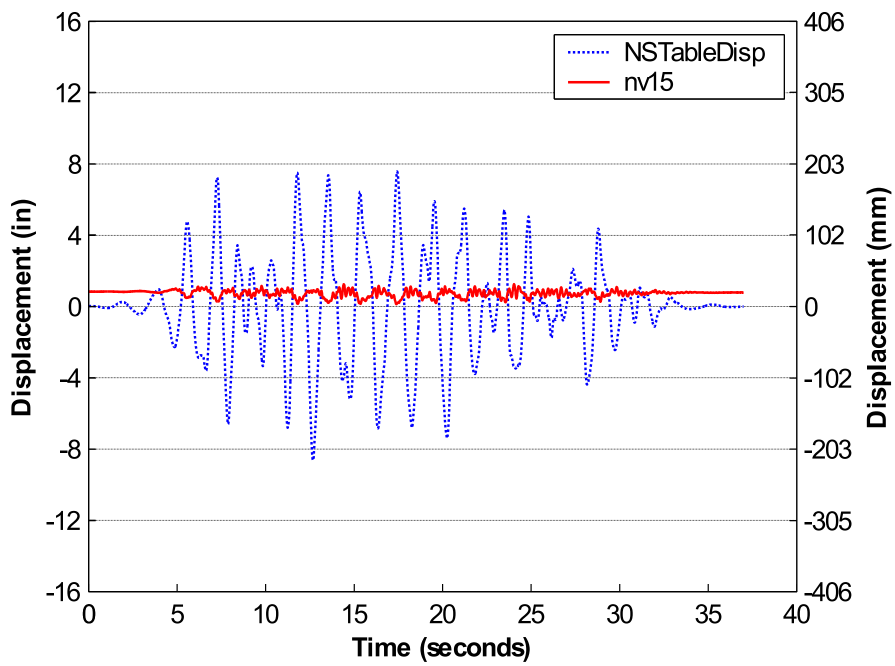

Figure 3-8 Run 62 - Absolute Displacement Time History of nv15

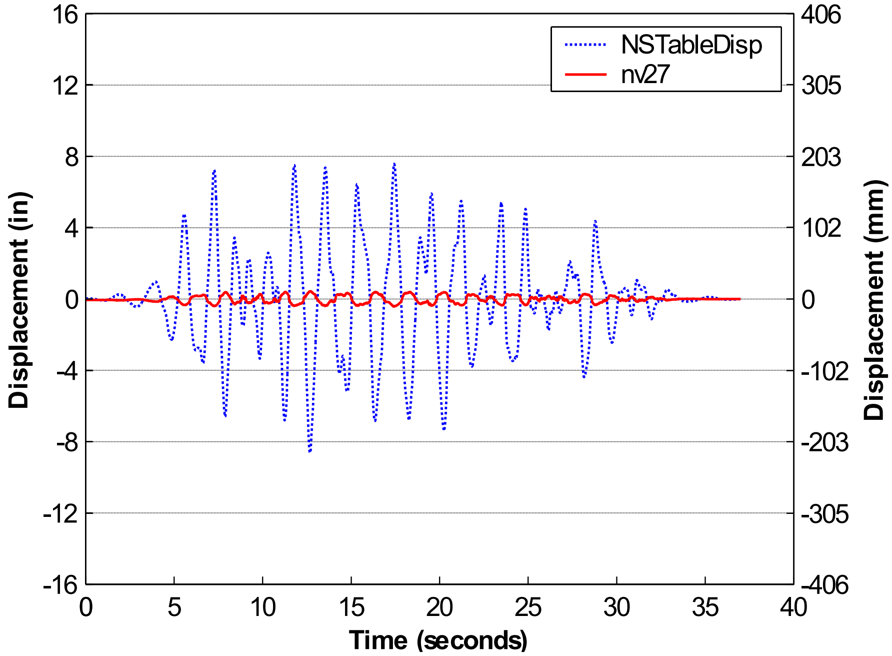

Figure 3-9 Run 62 - Absolute Displacement Time History of nv27

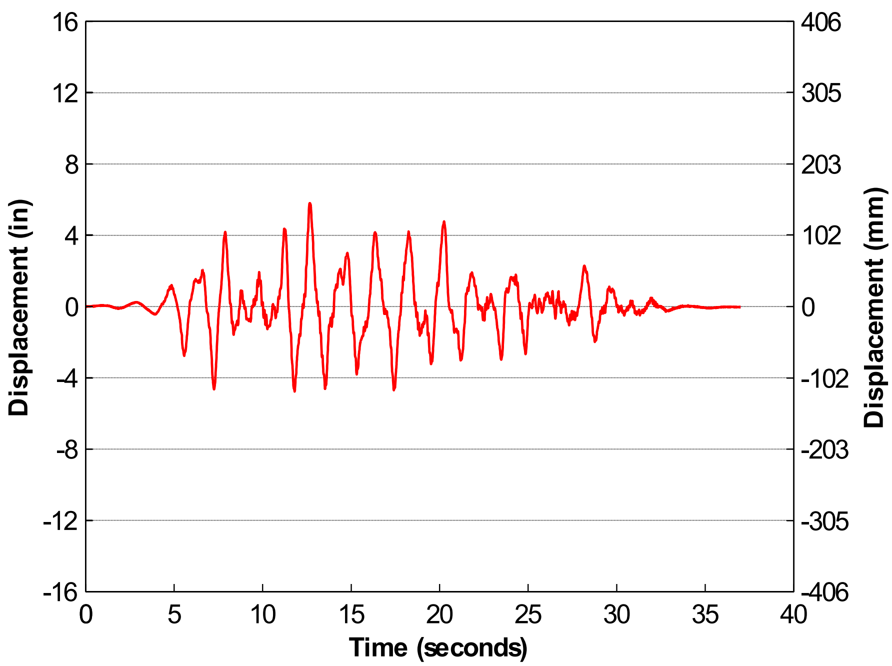

Figure 3-10 Run 62 - Relative Displacement Time History of nv40

Table 3-4 shows the maximum absolute displacements for the selected instruments during runs 61-63. Figures 3-11 and 3-12 illustrate the residual displacement of the subassembly after runs 61 and 62 respectively.

	Maximum Displacement (inches)		
Instrument Number	Run 61	Run 62	Run 63
nv20	1.84	0.78	1.32
nv17	3.56	0.98	2.75
nv15	3.22	1.25	2.58
nv40	1.00	3.03	2.56
nv41	0.80	1.90	1.47
nv54	0.73	1.65	1.89
nv27	0.99	0.43	0.85
nv21	0.21	0.22	0.19
nv29	0.14	0.09	0.11

Table 3-4 Maximum Displacements of Selected Instruments for Runs 61-63

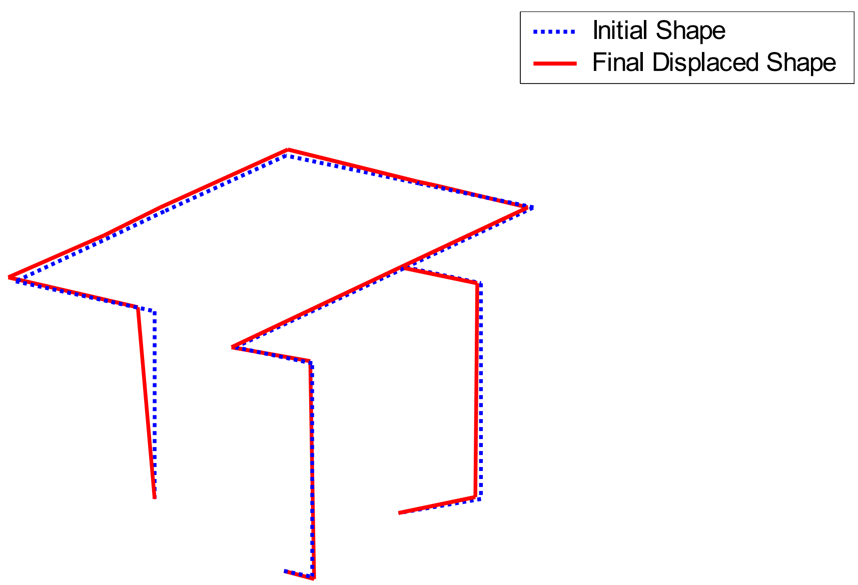

Figure 3-11 Final Displaced Shape After Run 61 (Scale Factor = 12)

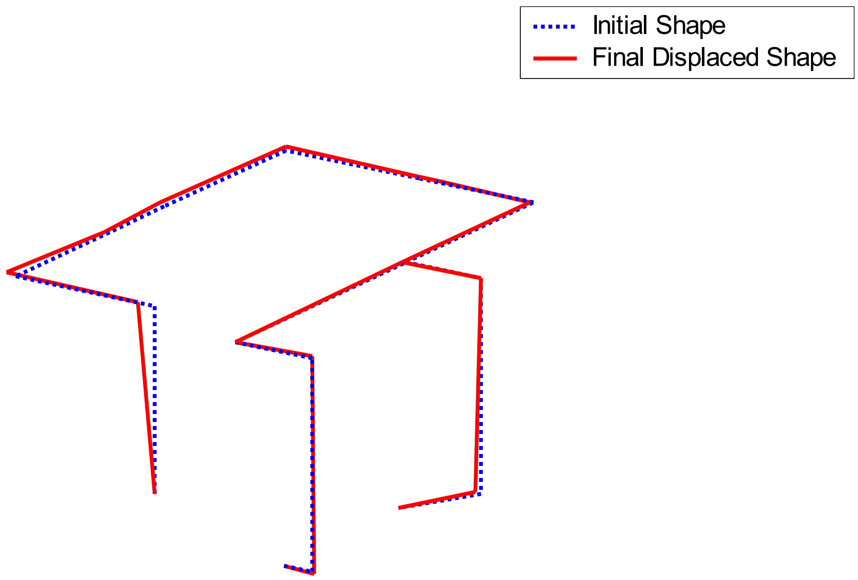

Figure 3-12 Final Displaced Shape After Run 62 (Scale Factor = 12)

3.3.1.2 Observations

At the end of run 63, there was no apparent damage to the subassembly. During run 64, brace B6 failed in the cable portion of the brace. Figure 3-13 shows a picture of the failure. Shortly after B6 failed, the longitudinal portion of brace B7 failed in the cable section. A picture of this failure can be seen in figure 3-14. This failure corresponds to a story drift of 4.34%.

Figure 3-13 Failure of Brace B6

Figure 3-14 Failure of Longitudinal Portion of Brace B7

Most braces were permanently offset at the end of the experiments. The offset at brace B5 can be seen in figure 3-15. Figure 3-16 illustrates the offset at B2 and that of hanger H1 can be seen in figure 3-17. The flanged connection joining the heat exchanger to the pipe began to leak during a 10 inch radius dynamic pushover. This leakage corresponds to a story drift ratio of 5%. The leak was caused by the gasket inside the two flanges being compressed. Once the gasket assumed its initial shape, the leak stopped.

Figure 3-15 Permanent Offset of Brace B5

Figure 3-16 Permanent Offset of Brace B2

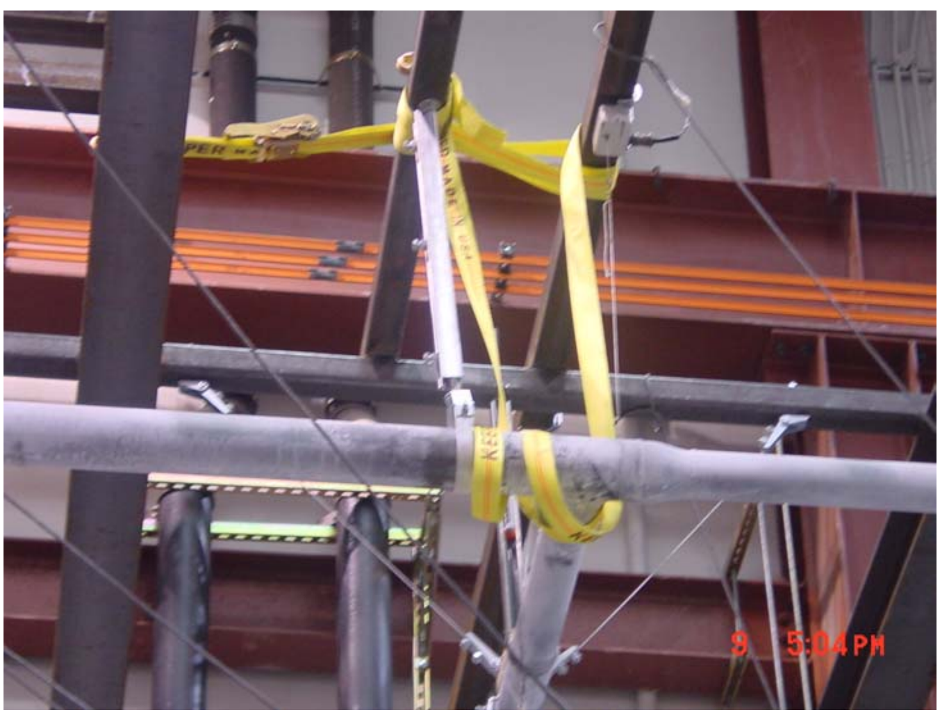

Figure 3-17 Permanent Offset of Hanger H1

3.3.2 Welded Unbraced Subassembly

3.3.2.1 Displacement Response

For run 113, the maximum parallel displacement of an instrument mounted at the top of the piping system was 9.65 inches (245 mm); this occurred at instrument nv17. The maximum transverse displacement occurred at nv30 and was 3.96 inches (101 mm). The maximum vertical displacement was 1.75 inches (44.4 mm) and occurred at nv55. The time histories for these instruments can be seen in figures 3-18 - 3-20. The relative displacement time history for nv17 is shown in figure 3-21.

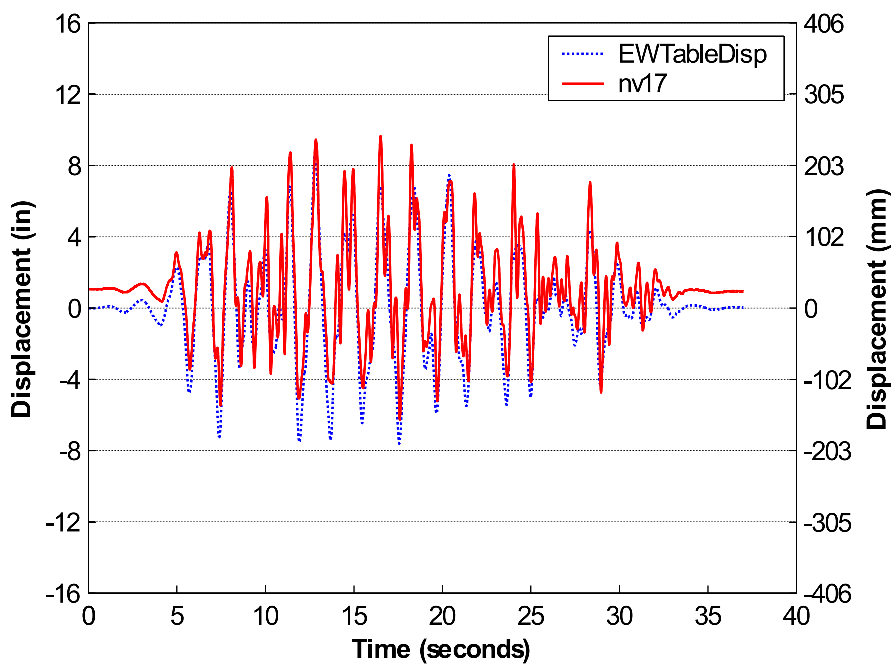

Figure 3-18 Run 113 - Absolute Displacement Time History of nv17

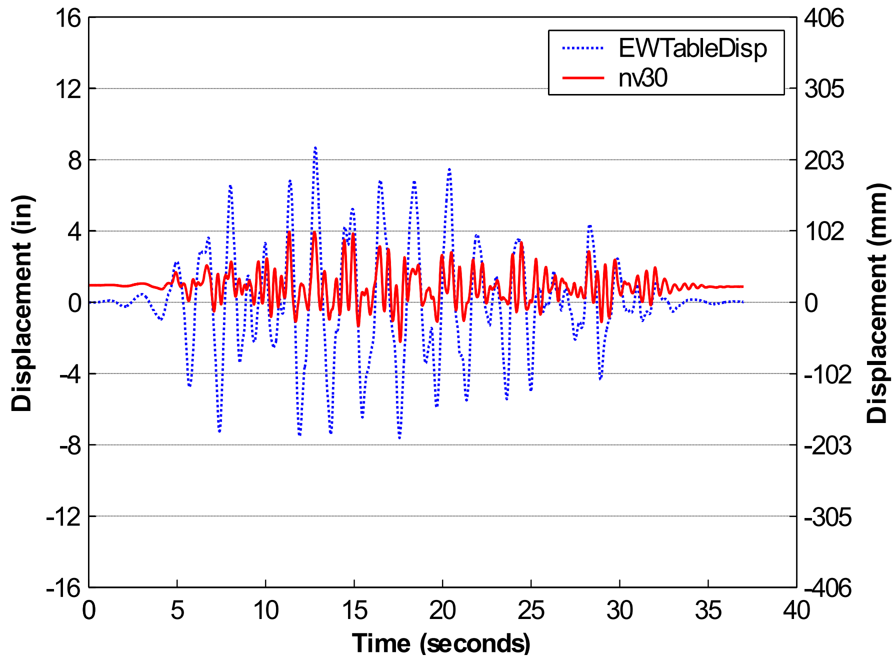

Figure 3-19 Run 113 - Absolute Displacement Time History of nv30

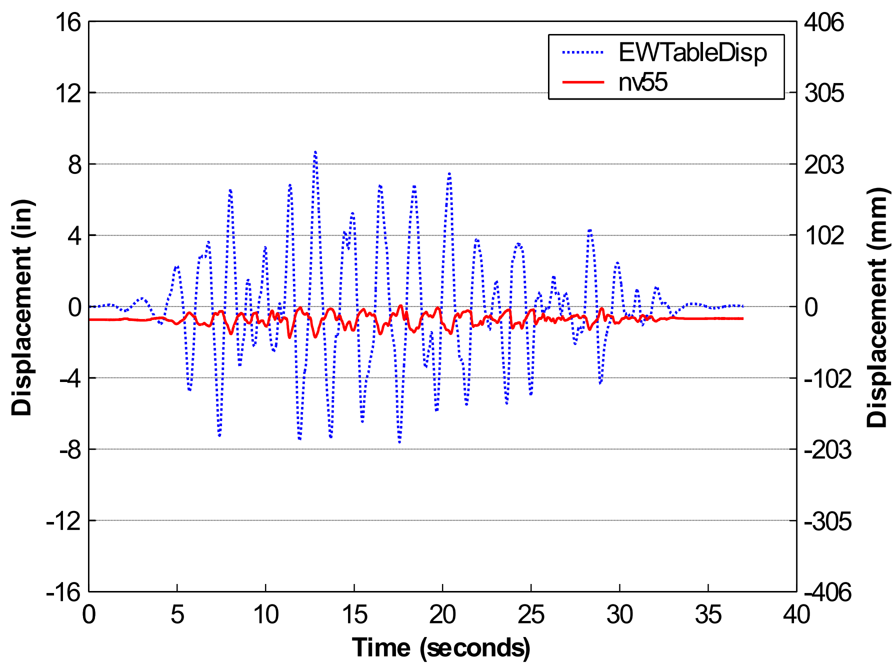

Figure 3-20 Run 113 - Absolute Displacement Time History of nv55

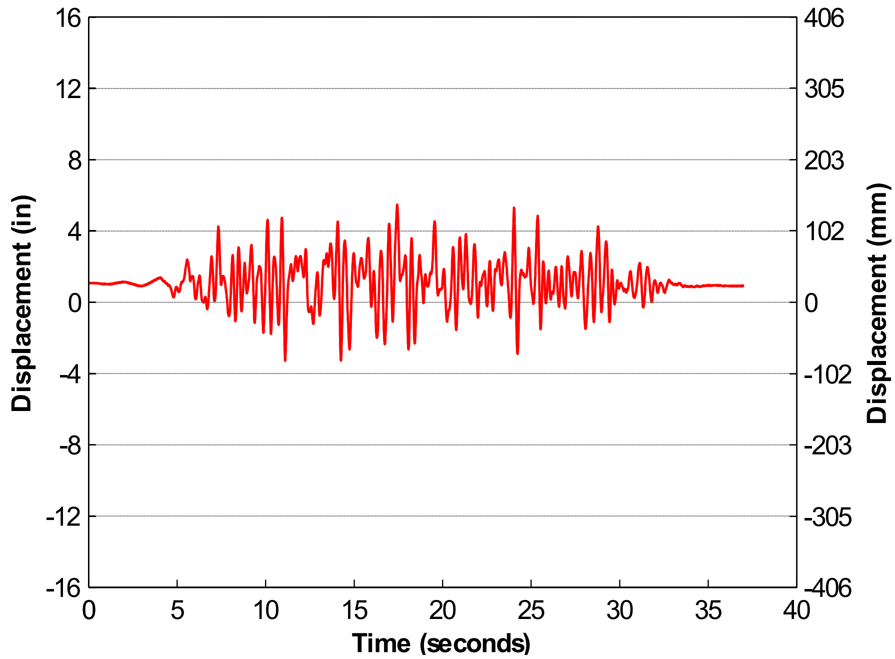

Figure 3-21 Run 113 - Relative Displacement Time History of nv17

For run 114, the maximum parallel displacement occurred at nv30 and was 8.80 inches (224 mm). The maximum transverse displacement was 3.79 inches (96.3 mm) and occurred at instrument nv15. The maximum vertical displacement was 1.65 inches (41.9 mm) and occurred at nv51. The time histories for these instruments can be seen if in figures 3-22 - 3-25. The relative displacement time history of instrument nv30 is shown in figure 3-25.

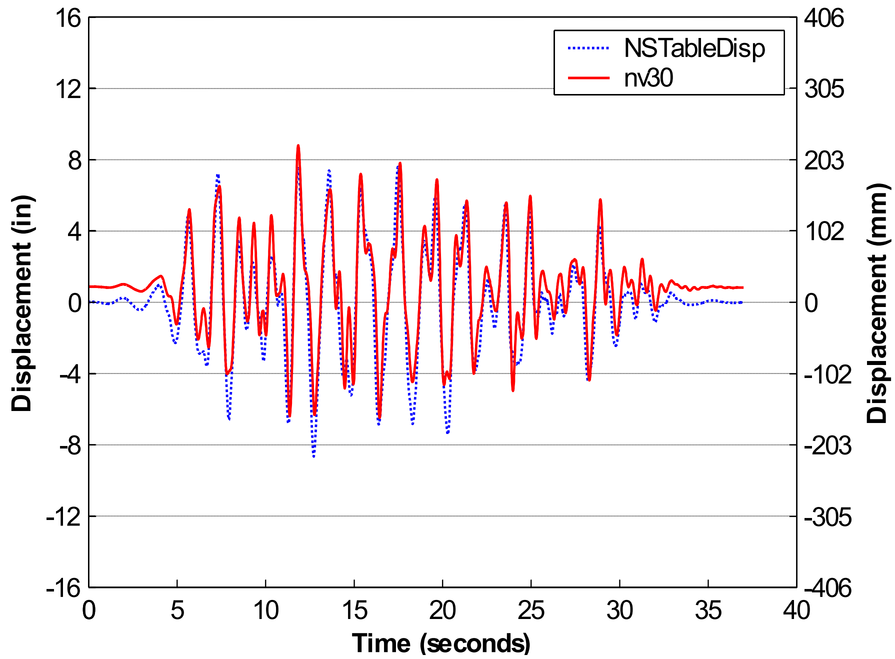

Figure 3-22 Run 114 - Absolute Displacement Time History of nv30

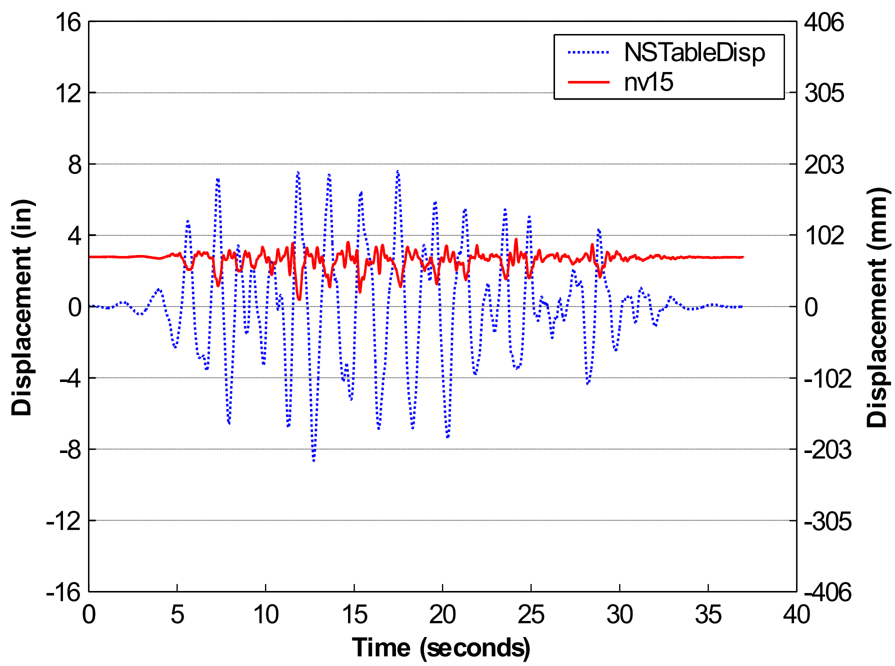

Figure 3-23 Run 114 - Absolute Displacement Time History of nv15

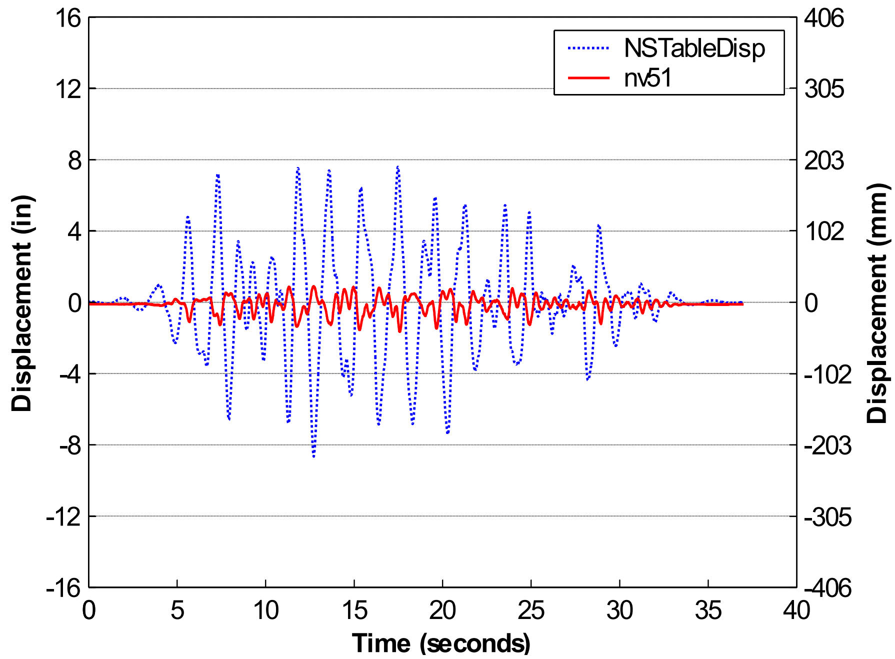

Figure 3-24 Run 114 - Absolute Displacement Time History of nv51

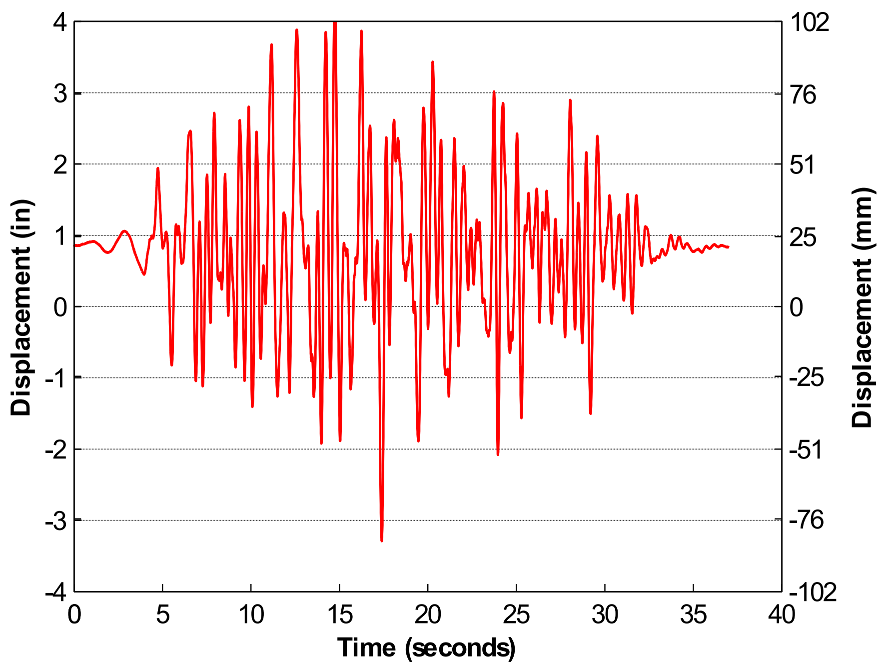

Figure 3-25 Run 114 - Relative Displacement Time History of nv30

Table 3-5 shows the maximum absolute displacements for the selected instruments during runs 113-115. Figures 3-26 and 3-27 illustrate the residual displacement of the subassembly after runs 113 and 114 respectively.

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	Maximum Displacement (inches)		
Instrument Number	**Run 113**	**Run 114**	**Run 115**
nv20	7.63	3.07	5.13
nv17	9.65	3.75	7.94
nv15	8.27	3.79	7.60
nv40	1.77	7.68	5.16
nv41	1.39	7.68	5.52
nv54	1.81	8.74	6.11
nv27	1.18	1.41	1.34
nv21	1.01	0.49	0.75
nv29	0.92	0.72	0.53

Table 3-5 Maximum Displacements of Selected Instruments for Runs 113-115

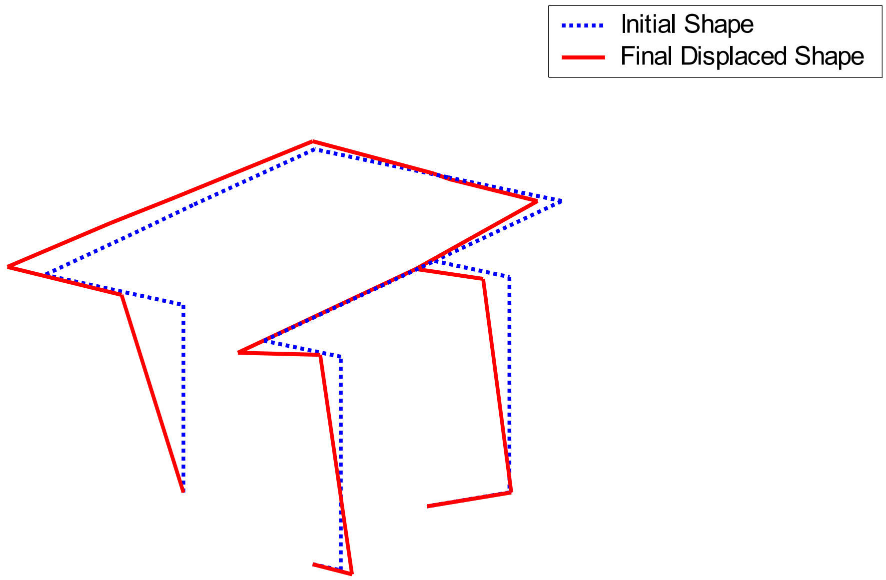

Figure 3-26 Final Displaced Shape After Run 113 (Scale Factor = 12)

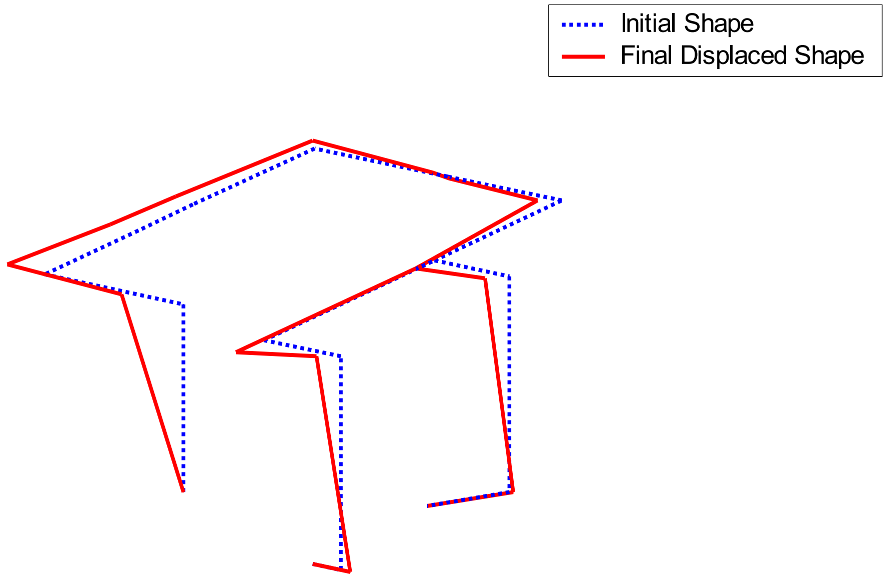

Figure 3-27 Final Displaced Shape After Run 114 (Scale Factor = 12)

3.3.2.2 Observations

Up to run 115, the subassembly had not experienced any damage. During run 115, hanger B2 failed. This corresponds to a story drift of 4.34%. A picture of the hanger failure can be seen in figure 3-28. The failure occurred right at the connection to the steel frame. After run 115, testing was continued without replacing the failed hanger rod. During run 121, hanger B1 failed in the same manner that B2 had failed in during run 115. Hanger B1 failure can be seen in figure 3-29. At the end of the experiments on this subassembly, many of the hangers had permanently displaced. One such displacement occurred at hanger B5, which was permanently displaced 6 to 8 inches (152 to 203 mm) from its original position, as seen in figure 3-30. The welded unbraced subassembly did not have any leaks at the end of the experiments.

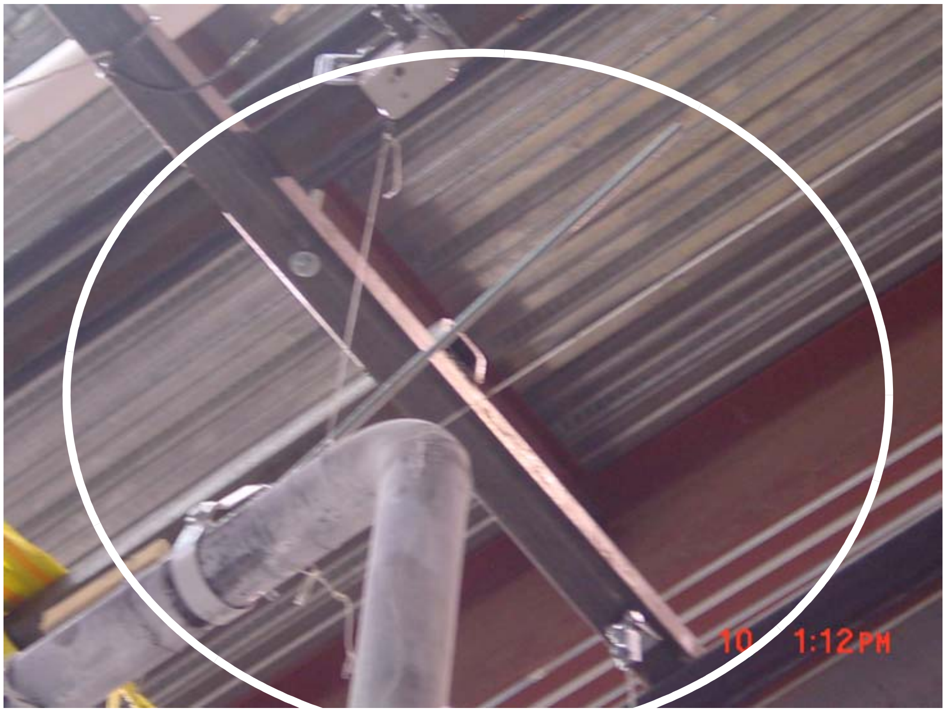

Figure 3-28 Failure of Hanger B2

Figure 3-29 Failure of Hanger B1

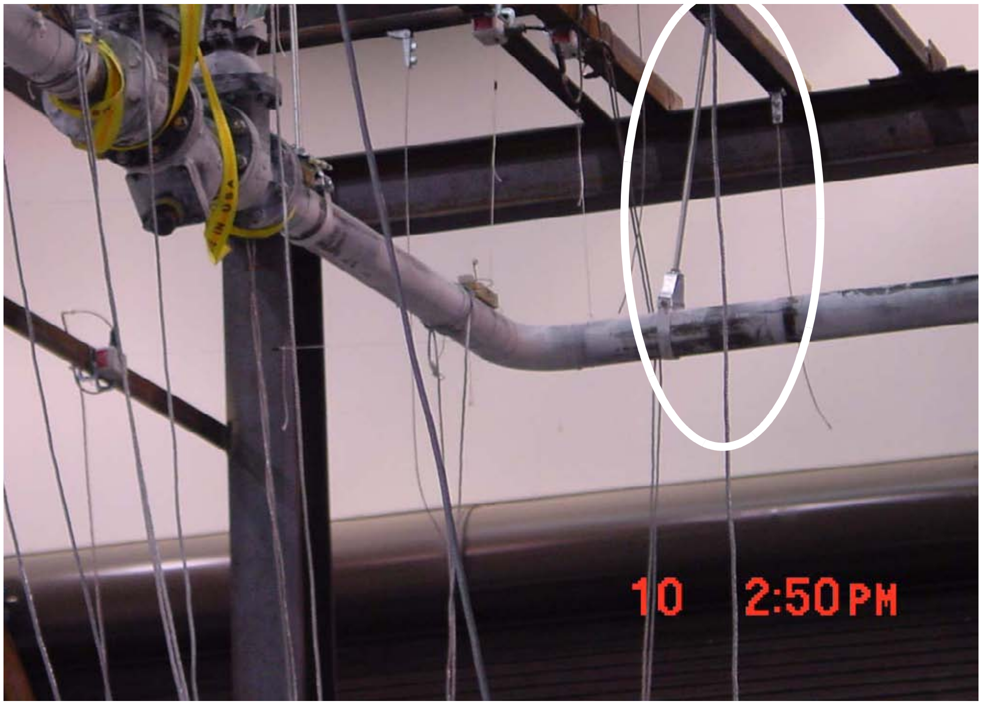

Figure 3-30 Permanent Offset of Hanger B5

3.3.3 Comparison of Welded Braced and Unbraced Subassemblies

This section will present a comparison between the responses of the welded braced and unbraced subassemblies. Existing plots are discussed and new plots are presented in order to make the comparison.

3.3.3.1 Displacement Comparison

The presence of braces makes the absolute displacement response of the braced subassembly less than the unbraced case. Figure 3-31 compares the absolute displacement time histories for instrument nv17 during runs 61 and 113. Figure 3-32 compares the absolute displacement time histories for instrument nv40 during runs 62 and 114. As seen, the absolute displacement for the braced case is less.

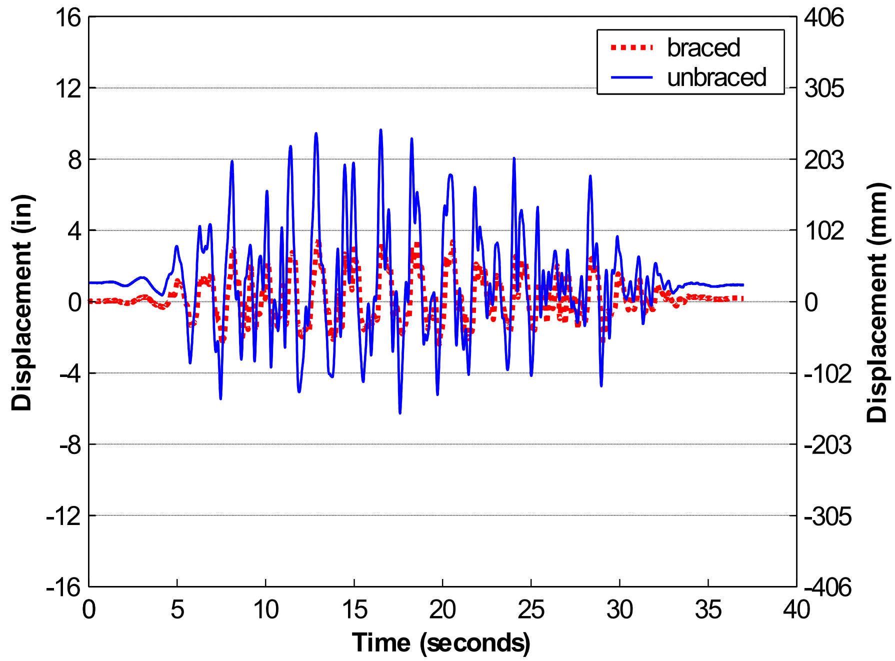

Figure 3-31 Welded Braced and Unbraced Displacement Response of nv17

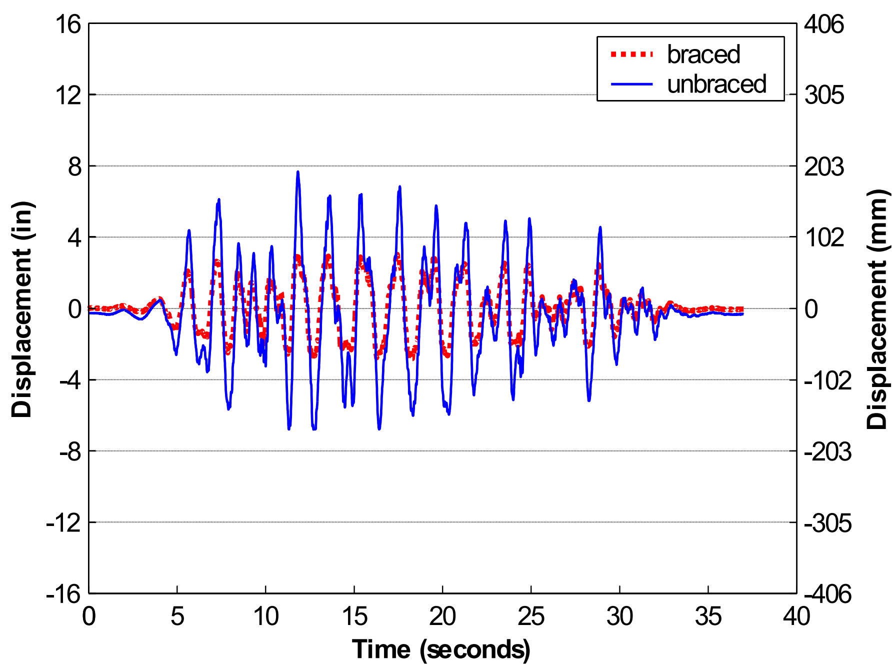

Figure 3-32 Welded Braced and Unbraced Displacement Response of nv40

Although the absolute displacement response decreases, the presence of braces results in higher relative displacements due to the fact that some points of the subassembly are braced and others are free to move. Figures 3-6 and 3-21 illustrate this phenomenon. Figures 3-6 and 3-21 illustrate the relative displacement time histories for instrument nv17 during runs 61 and 113. Figures 3-10 and 3-25 show the relative displacement time histories for instrument nv40 during runs 62 and 114. As shown, the braced relative displacement is higher than the unbraced relative displacement. The increased relative displacement can be detrimental to the fragility of the system. For instance, during the braced case, the subassembly began to leak at the heat exchanger flange; but when the subassembly was unbraced, no leaks formed.

The transverse response for the unbraced assembly was larger than the braced subassembly for all instruments. The unbraced transverse response was anywhere from 1.4 to 3.3 times as large as the braced case for the East/West excitation. Instrument nv11 had the largest increase in transverse response for East/West excitation. For the North/South excitation, instrument nv16 had the largest unbraced transverse response increase at 4.6 times the braced response.

Vertical displacement response also increased for every instrument when there were no braces present. Instrument nv29 had the largest increase for both East/West and North/South excitations with the unbraced response being 6.43 and 8.29 times larger, respectively.

Final offsets were also increased when there were no braces present on the subassembly. This is shown by comparing the run 113 and 114 responses with run 61 and 62 responses. The high initial value in the instrument response of runs 113 and 114 shows the large final offsets.

3.3.3.2 Observational Comparison

The unbraced subassembly was subjected to story drifts up to 4.34% and suffered no damage. The braced subassembly had no damage at 4.34% and minor leakage at 5% story drift.

3.4 Threaded Subassembly

The following sections present the experimental results from the braced and unbraced threaded subassembly experiments. The threaded braced subassembly was subjected to runs 122-180 (tables 2-7 and 2-8) from the experimental protocol. The unbraced subassembly was subjected to runs 181-230 (tables 2-9 and 2-10) from the experimental protocol. Displacements and observations are discussed for each subassembly. A comparison of the two subassemblies is made as well. Acceleration data and dynamic characteristics are presented in Appendix D.

3.4.1 Threaded Braced Subassembly

3.4.1.1 Displacement Response

The experiments were stopped after run 180 due to excessive leakage at the heat exchanger flange. Run 180 was the highest level of input motion that the threaded braced subassembly was subjected to. Therefore, only data from run 180 is plotted.

For run 180, the maximum parallel displacement of an instrument mounted at the top of the piping system was 3.79 inches (96.3 mm); this occurred at instrument nv17. The maximum transverse displacement occurred at nv30 and was 1.51 inches (38.4 mm). The maximum vertical displacement was 2.08 inches (52.8 mm) and occurred at nv27. The time histories for these instruments can be seen in figures 3-33 - 3-35. The relative displacement time history for nv17 is shown in figure 3-36.

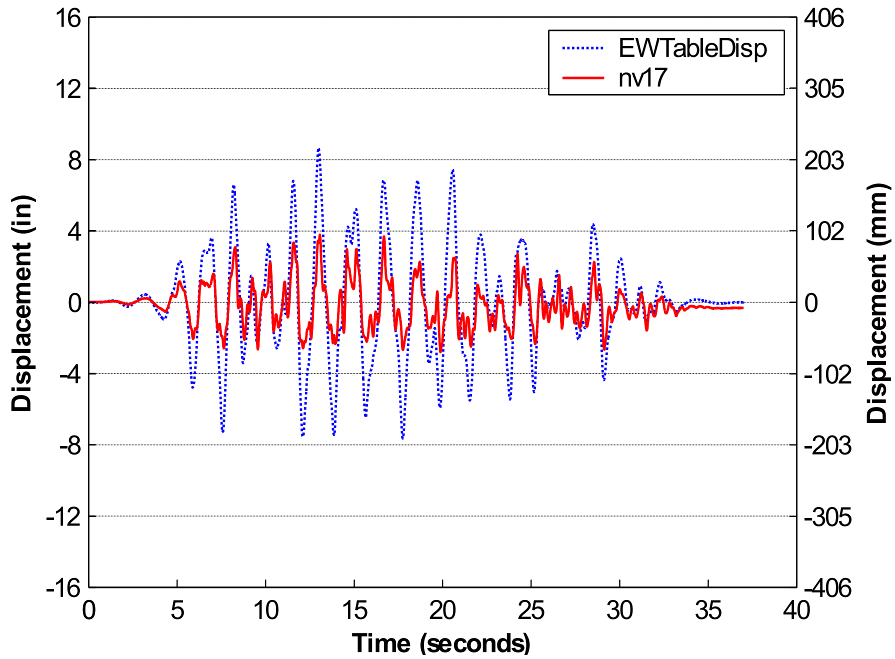

Figure 3-33 Run 180 - Absolute Displacement Time History of nv17

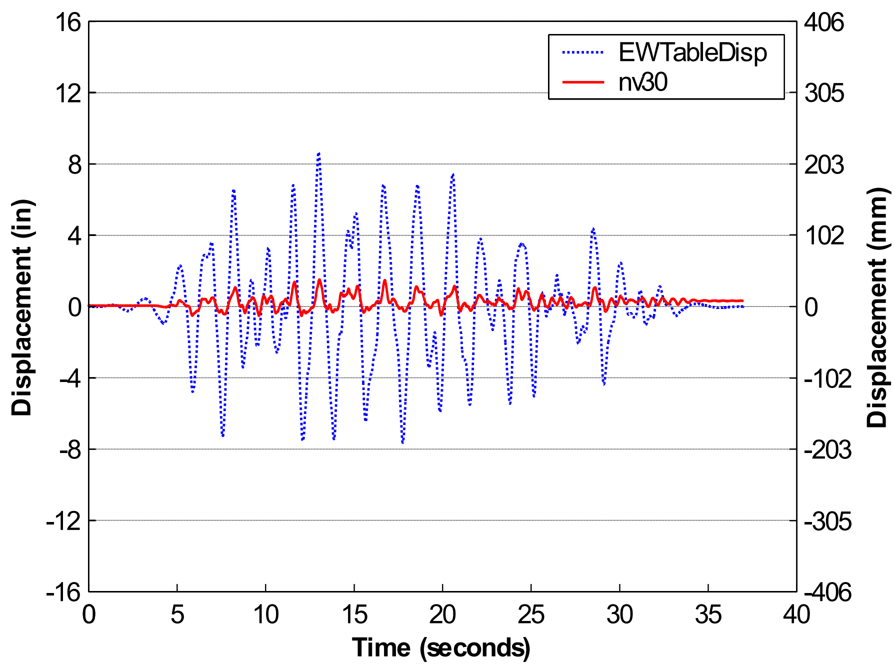

Figure 3-34 Run 180 - Absolute Displacement Time History of nv30

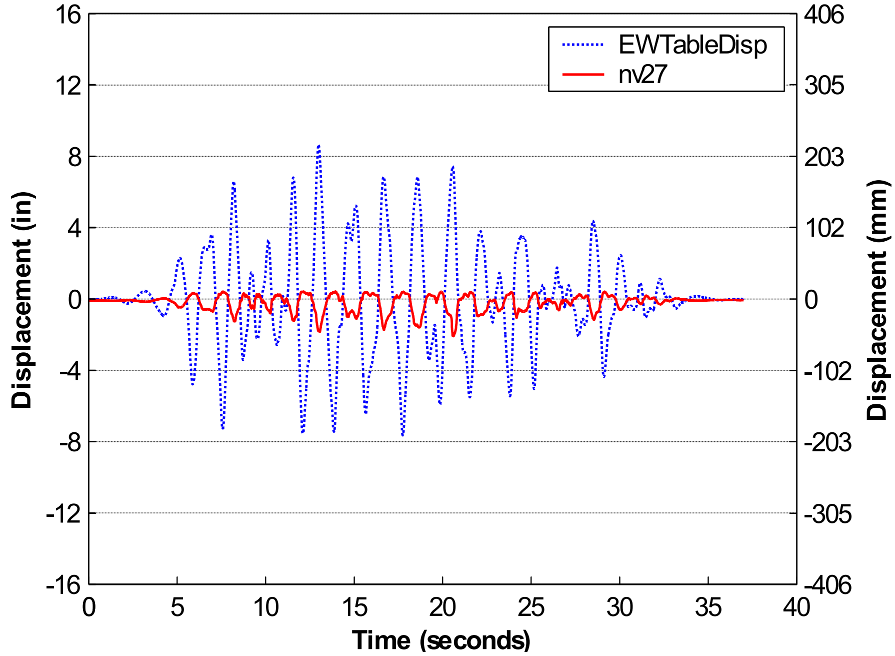

Figure 3-35 Run 180 - Absolute Displacement Time History of nv27

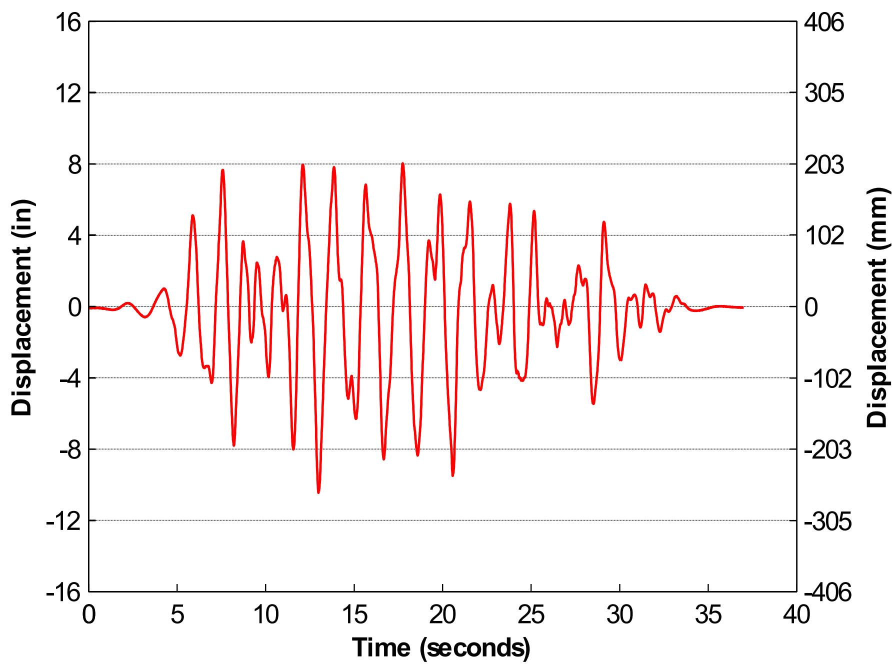

Figure 3-36 Run 180 - Relative Displacement Time History of nv17

Table 3-6 shows the maximum absolute displacements for the selected instruments during runs 177 - 180. Figure 3-39 illustrates the residual displacement of the subassembly after run 180.

	Maximum Displacement (inches)			
Instrument Number	**Run 180**	**Run 177**	**Run178**	**Run179**
nv20	1.23	1.28	0.36	0.91
nv17	3.79	3.49	0.69	2.78
nv15	2.25	2.01	0.59	1.86
nv40	1.33	0.88	2.49	2.06
nv41	0.72	0.65	1.06	0.72
nv54	1.37	1.38	1.28	1.46
nv27	2.08	1.35	0.32	1.11
nv21	0.38	0.41	0.08	0.15
nv29	0.23	0.19	0.10	0.08

Table 3-6 Maximum Displacements of Selected Instruments for Runs 177-180

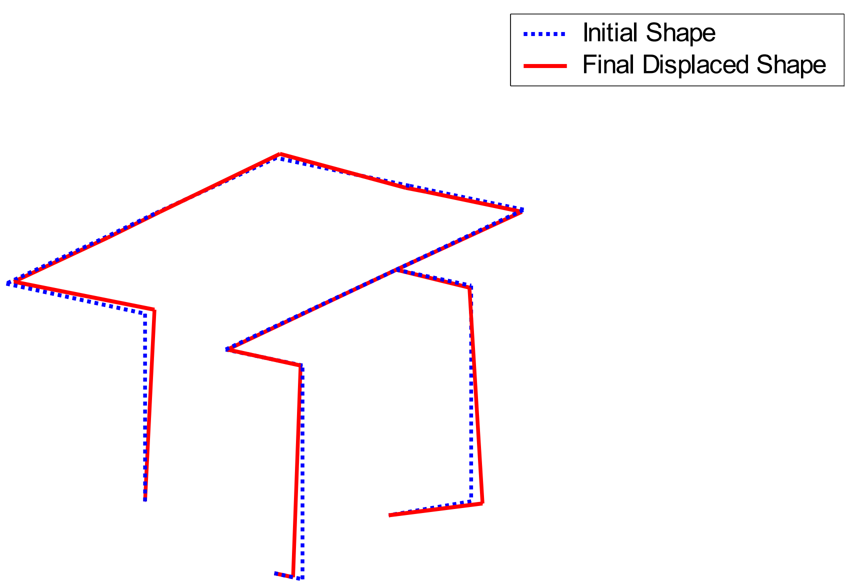

Figure 3-37 Final Displaced Shape After Run 177 (Scale Factor = 12)

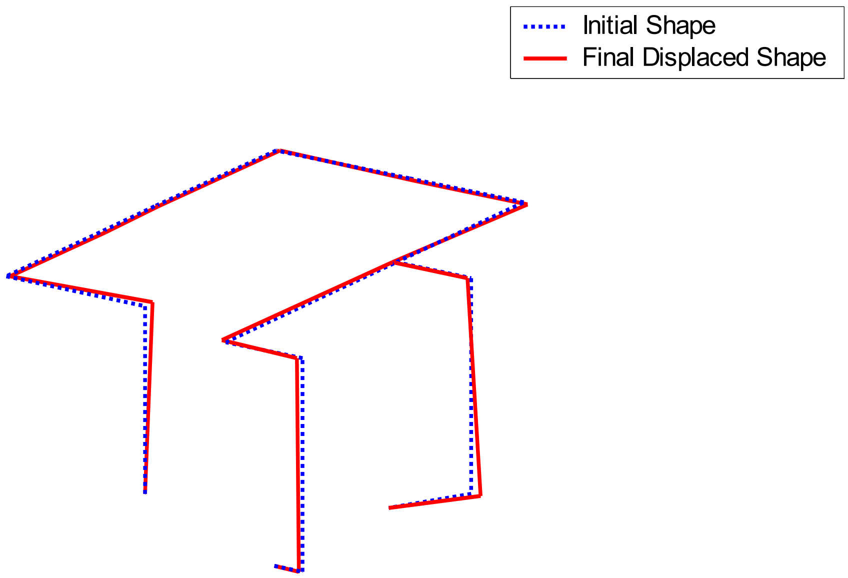

Figure 3-38 Final Displaced Shape After Run 178 (Scale Factor = 12)

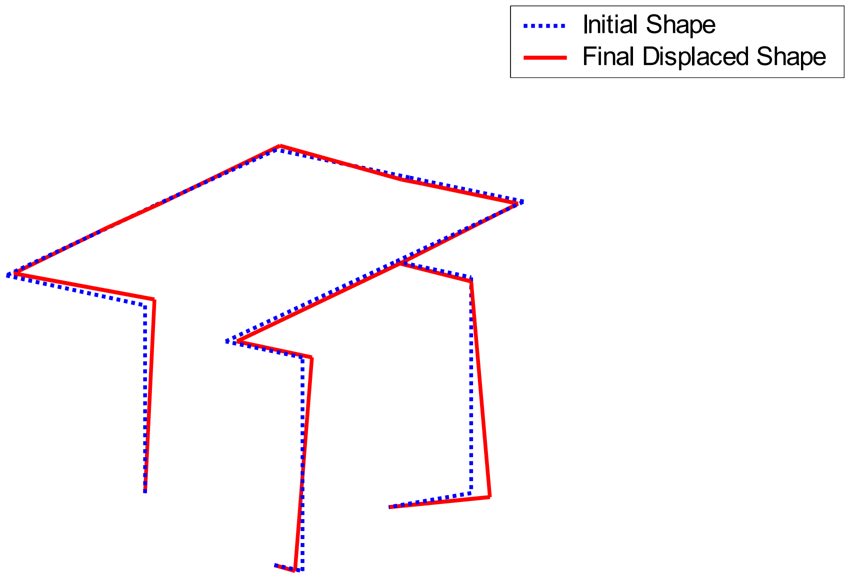

Figure 3-39 Final Displaced Shape After Run 180 (Scale Factor = 12)

3.4.1.2 Forces in Rods

Most of the rods stayed in the elastic range throughout the duration of the experiments for the threaded braced subassembly. Plots of stain and force versus time for run 180 are given for B1 and B2 (figure 2-21) in figures 3-40 - 3-41.

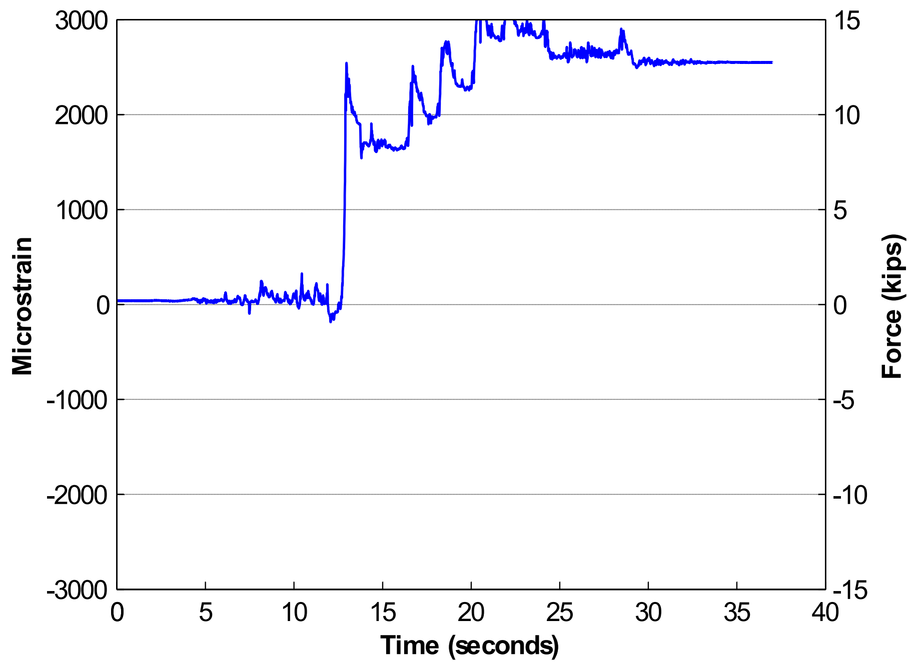

Figure 3-40 Run 180 - Strain and Force Time History for Rod B1

Rod B1 had the largest residual force at the end of run 180 with a residual strain of nearly 2500 microstrains. Initially, these results look false due to the fact that rod B2 does not experience the same level of force. But upon closer examination, this was most likely caused by the large vertical displacement that rod B1 experienced. Figure 3-42 shows the displacement time histories of nv27 and nv24. As seen, nv27 records a much larger vertical displacement, thereby causing a larger axial force in rod B2.

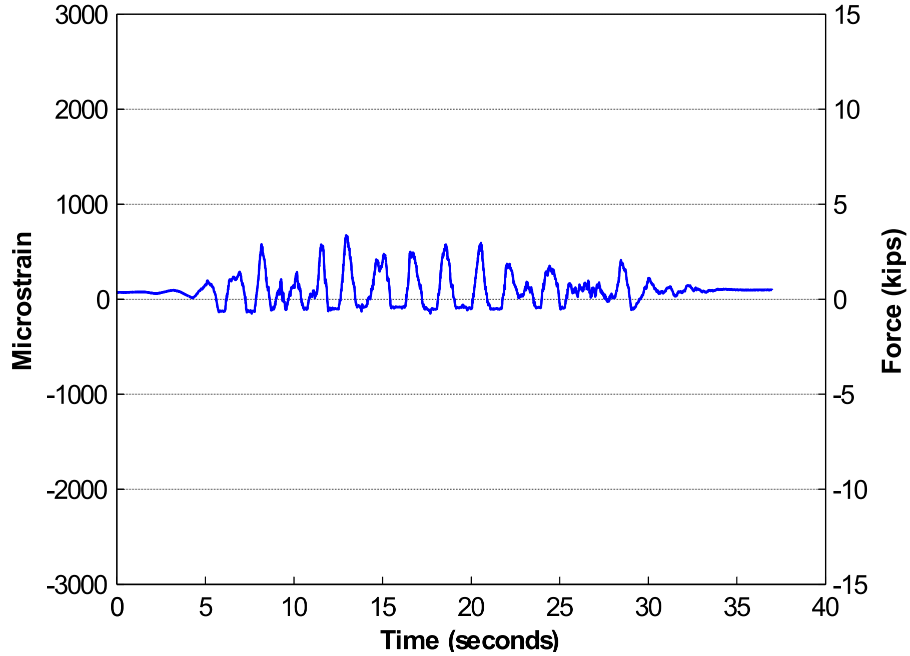

Figure 3-41 Run 180 - Strain and Force Time History for Rod B2

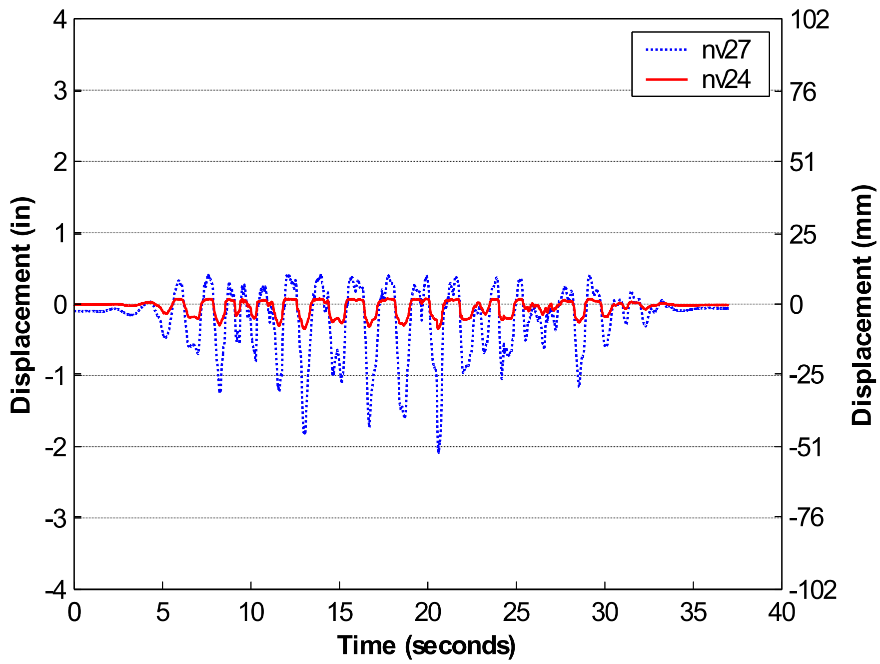

Figure 3-42 Run 180 - Displacement Time History of nv27 and nv24

3.4.1.3 Observations

The threaded braced subassembly was subjected to runs 122-180. At run 164, leaks began forming at the heat exchanger connection to the piping system. This corresponds to a story drift ratio of 2.17%. This initial leak can be seen in figure 3-43. At run 172, leaks began forming at the elbow above the heat exchanger, as shown in figure 3-44. During run 173, the heat exchanger connection began leaking steadily. At run 178, the water coming out of the connection was flowing freely. The tests were stopped at run 180 due to the volume of water coming out of the connection. This corresponds to a story drift ratio of 4.34%. Figure 3-45 illustrates the water flowing out of the connection.

After the tests were stopped, the heat exchanger connection was investigated and it was found that the cast iron flange used in the connection had fractured, as shown in figure 3-46. Once the pipe run was taken down for replacement, it was found that the flange had fractured through its entire cross section. This is illustrated in figure 3-47.

Figure 3-43 Initial Leak On Heat Exchanger Connection

Figure 3-44 Leak Above Heat Exchanger

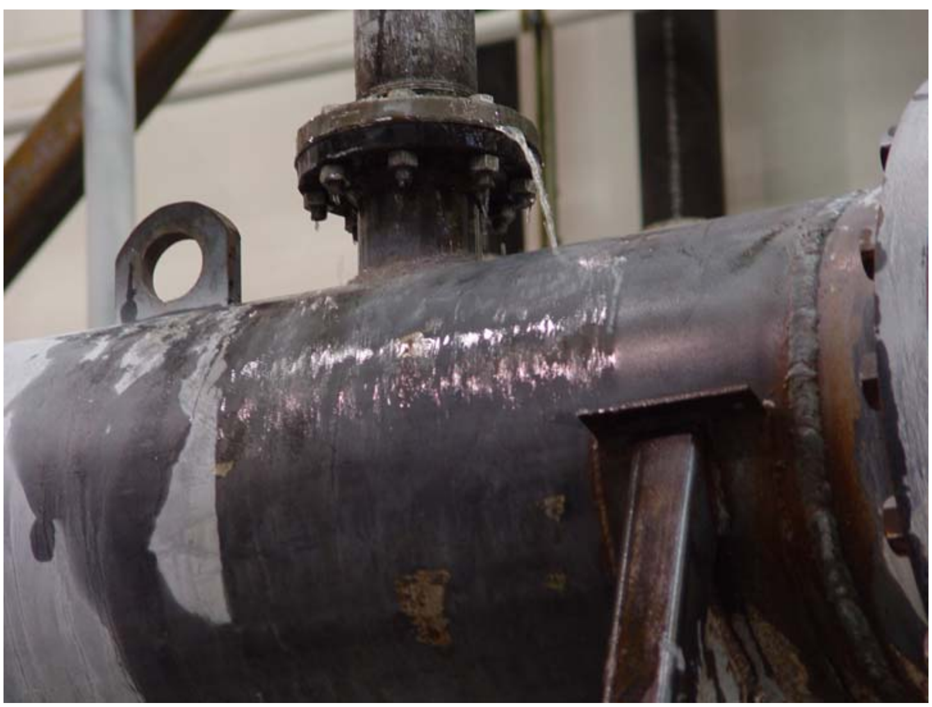

Figure 3-45 Water Flowing Out of Heat Exchanger Connection

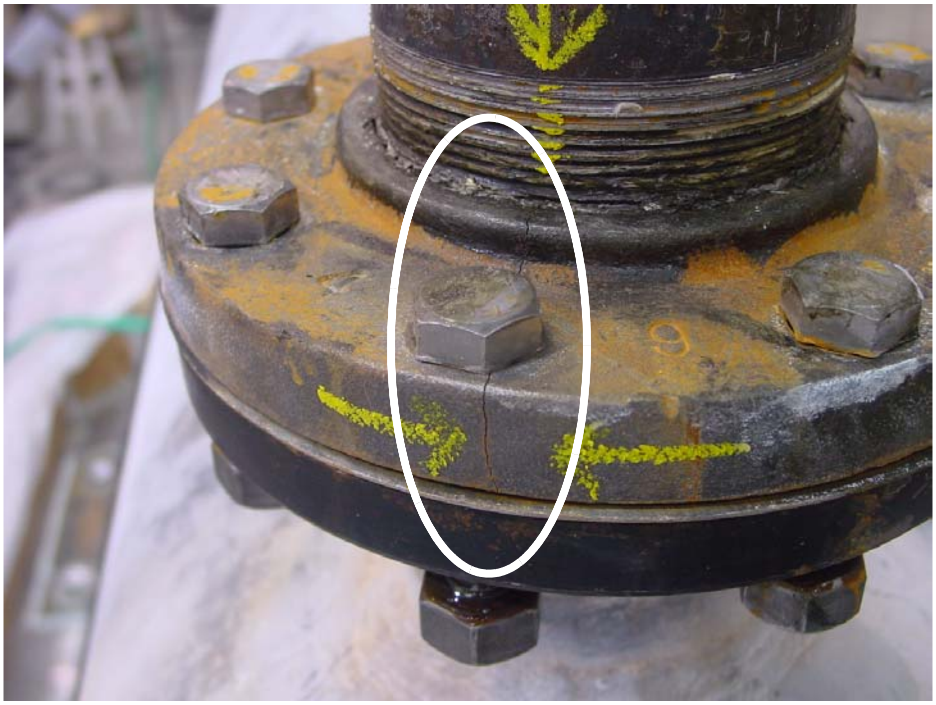

Figure 3-46 Fracture of the Flange

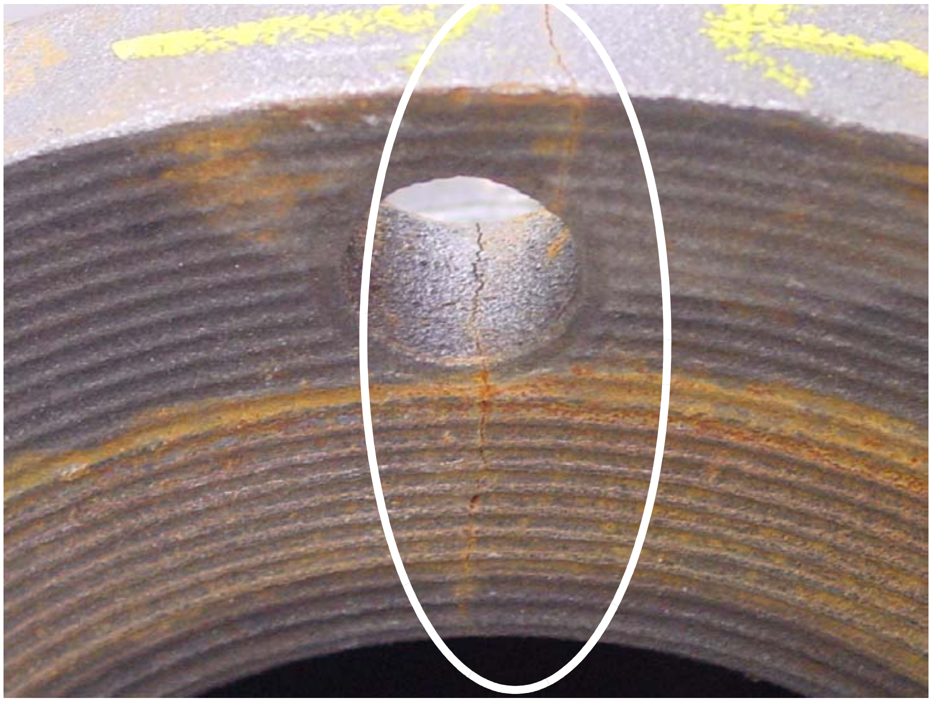

Figure 3-47 Fracture of the Flange

3.4.2 Threaded Unbraced Subassembly

3.4.2.1 Displacement Response

For run 225, the maximum parallel displacement of an instrument mounted at the top of the piping system was 9.81 inches (249 mm); this occurred at instrument nv17. The maximum transverse displacement occurred at nv30 and was 5.94 inches (151 mm). The maximum vertical displacement was 2.80 inches (71.1 mm) and occurred at nv27. The time histories for these instruments can be seen in figures 3-48 - 3-50. The relative displacement time history for nv17 is shown in figure 3-51.

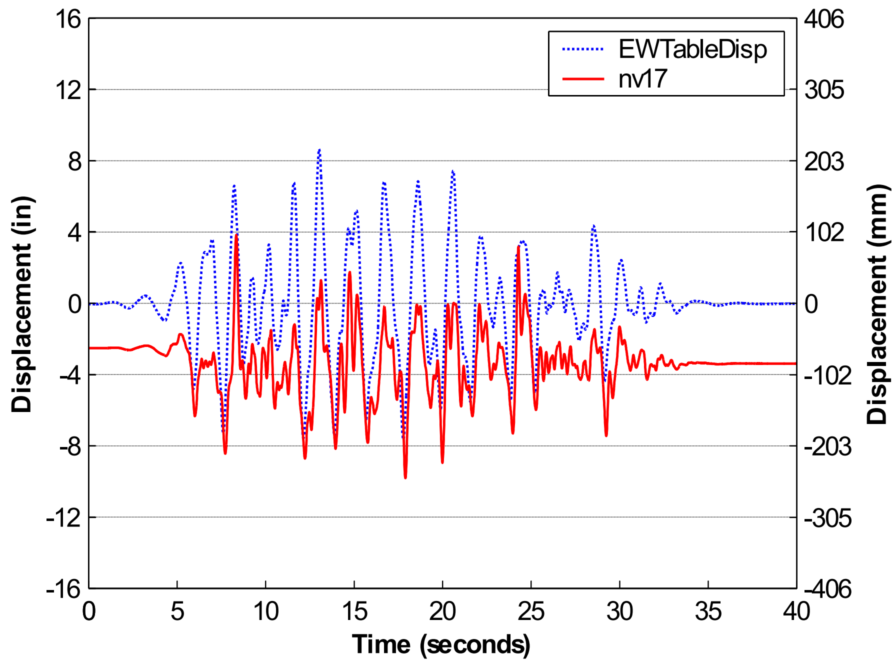

Figure 3-48 Run 225 - Absolute Displacement Time History of nv17

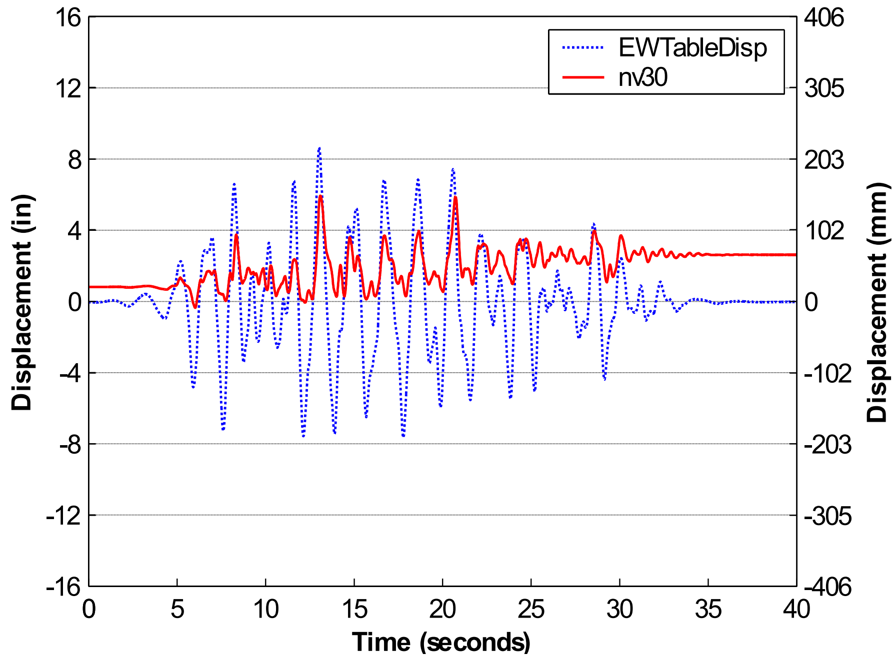

Figure 3-49 Run 225 - Absolute Displacement Time History of nv30

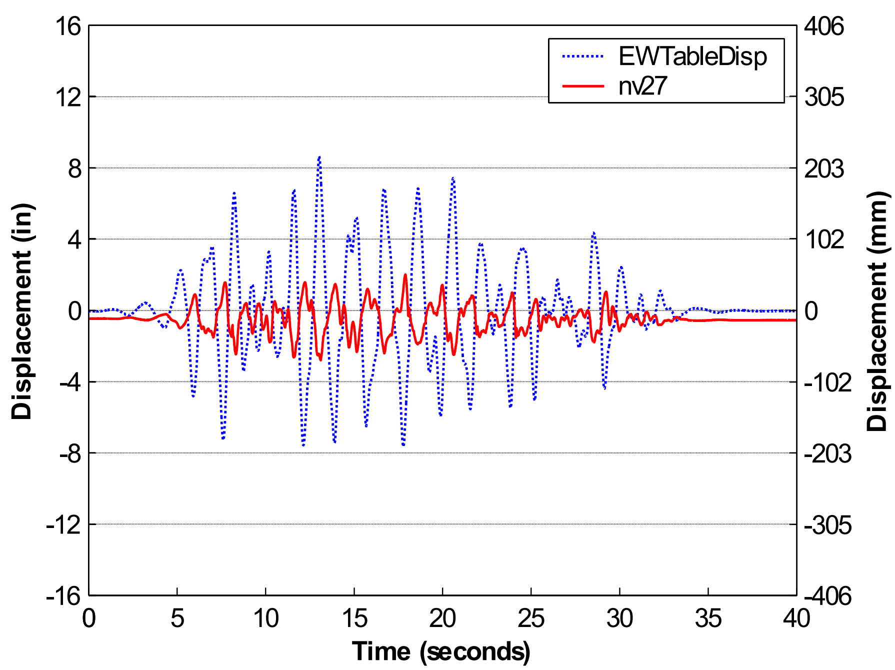

Figure 3-50 Run 225 - Absolute Displacement Time History of nv27

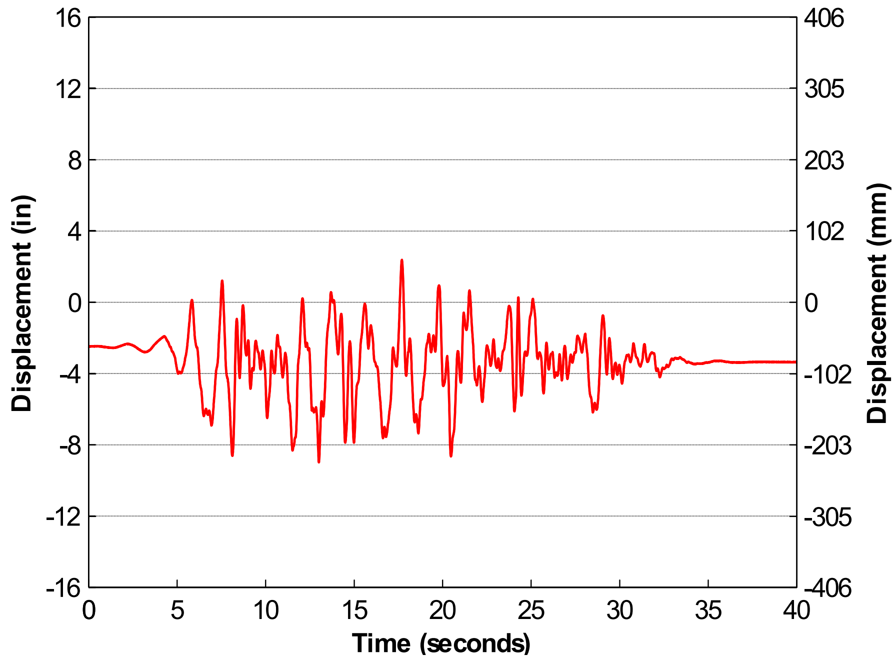

Figure 3-51 Run 225 - Relative Displacement Time History of nv17

For run 226, the maximum parallel displacement occurred at nv40 and was 5.41 inches (137 mm). The maximum transverse displacement was 4.32 inches (109.7 mm) and occurred at instrument nv16. The maximum vertical displacement was 1.76 inches (44.7 mm) and occurred at nv42.

The time histories for these instruments can be seen in figures 3-52 - 3-54. The relative displacement time history for nv17 is shown in figure 3-55.

Table 3-7 shows the maximum absolute displacements for the selected instruments during runs 225 through 227. Figures 3-56 and 3-57 illustrate the residual displacement of the subassembly after runs 225 and 226 respectively.

	Maximum Displacement (inches)		
Instrument Number	Run 225	Run 226	Run 227
nv20	6.77	4.80	4.70
nv17	9.81	4.58	7.90
nv15	7.86	2.02	6.34
nv40	1.44	5.67	4.64
nv41	1.27	4.71	3.50
nv54	4.28	5.95	6.37
nv27	2.80	1.39	2.46
nv21	0.48	0.58	0.24
nv29	0.31	0.36	0.23

Table 3-7 Maximum Displacements of Selected Instruments for Runs 225-227

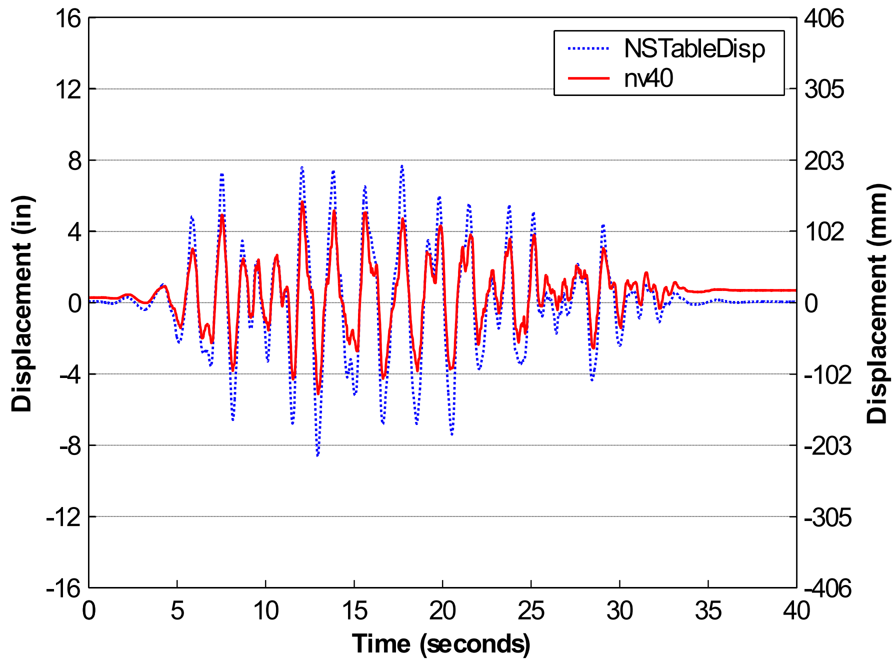

Figure 3-52 Run 226 - Absolute Displacement Time History of nv40

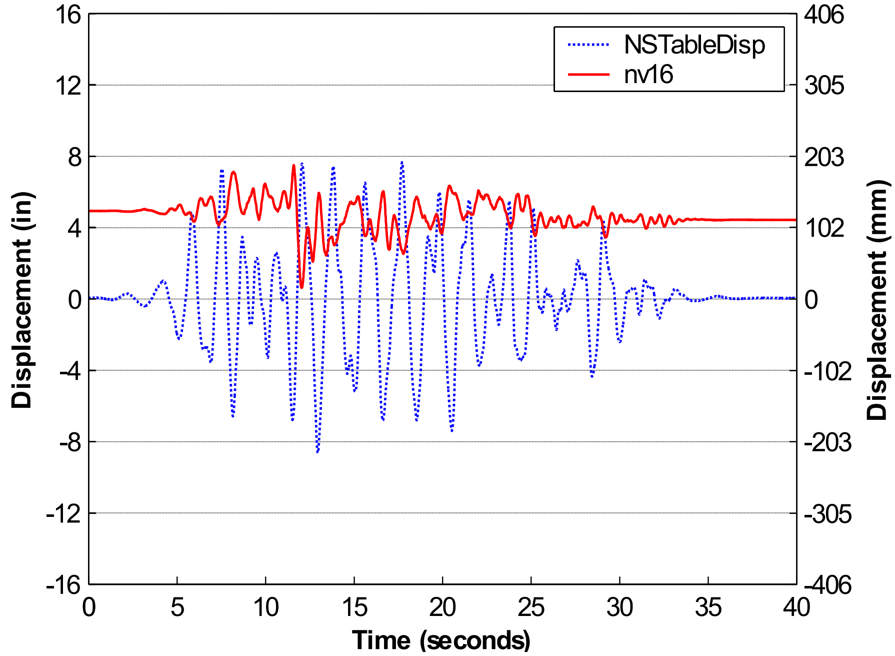

Figure 3-53 Run 226 - Absolute Displacement Time History of nv16

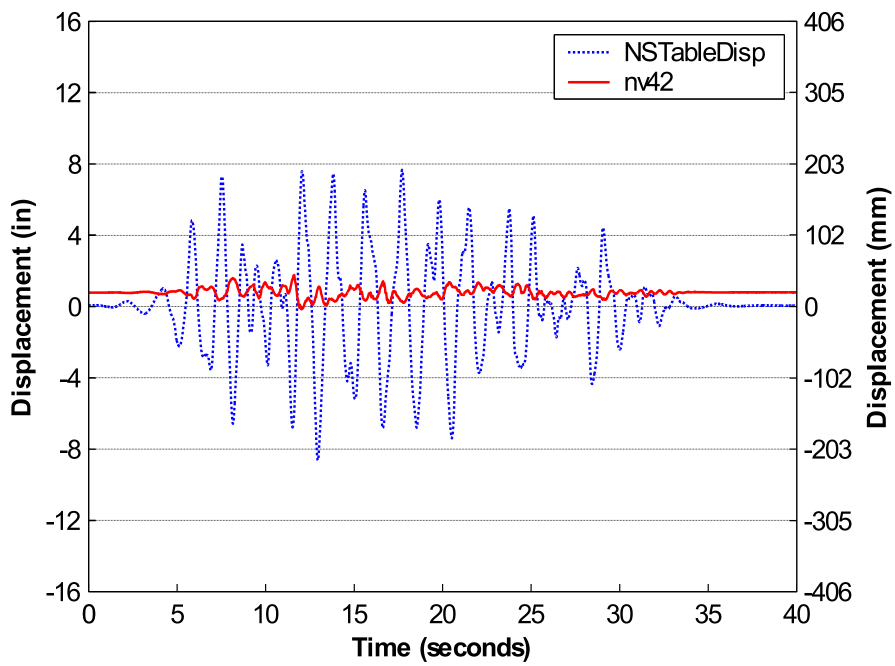

Figure 3-54 Run 226 - Absolute Displacement Time History of nv42

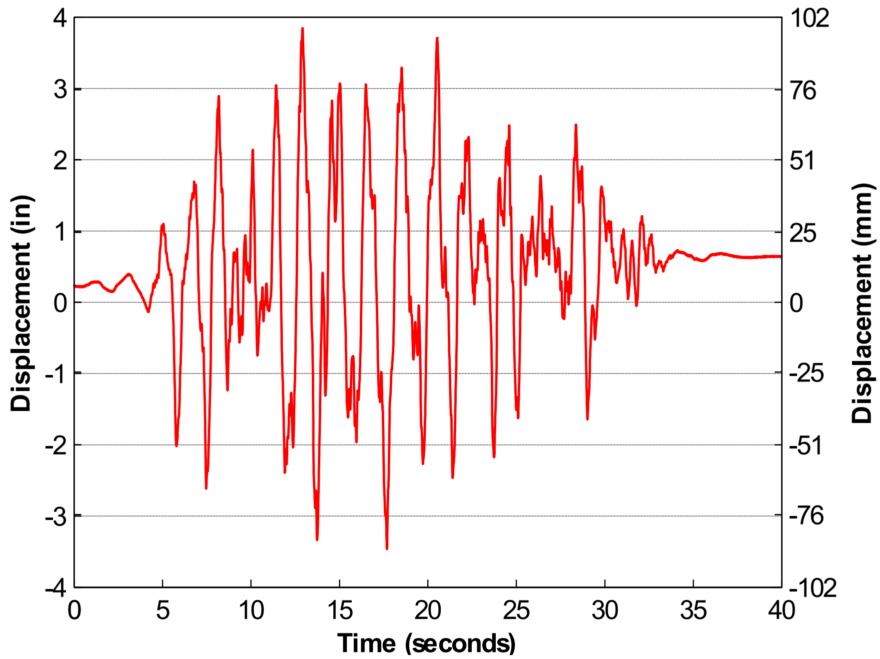

Figure 3-55 Run 226 - Relative Displacement Time History of nv40

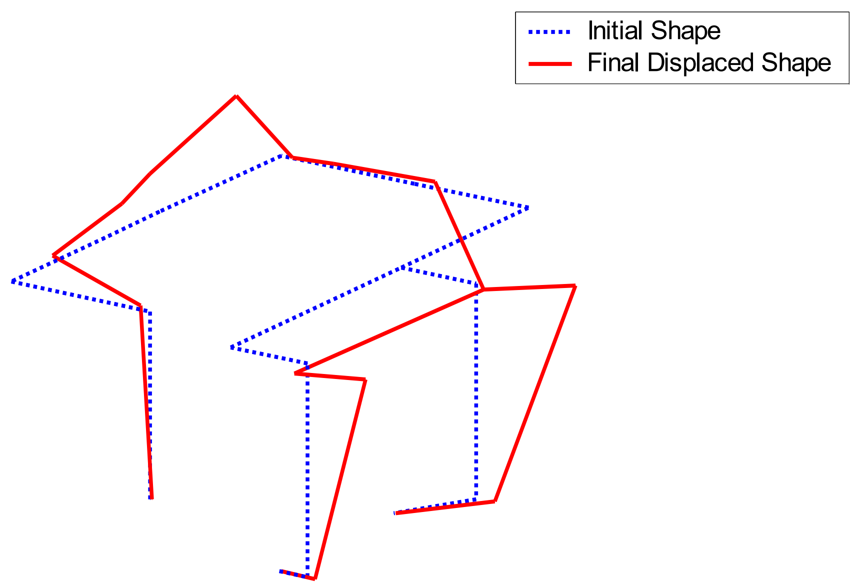

Figure 3-56 Final Displaced Shape After Run 225 (Scale Factor = 12)

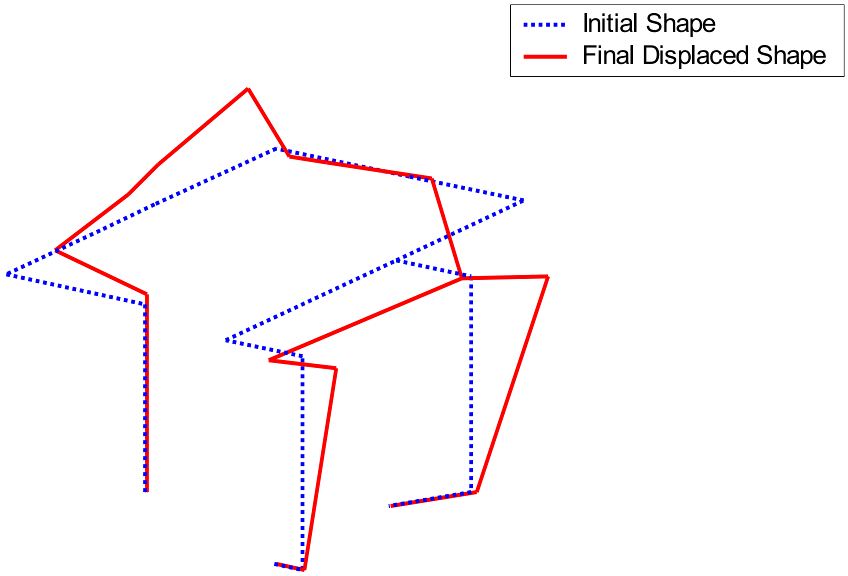

Figure 3-57 Final Displaced Shape After Run 226 (Scale Factor = 12)

3.4.2.2 Force in Rods

Most of the strain gauges were not taking accurate measurements at the time that the unbraced subassembly tests were conducted. The strain gauges on B5 and H3 (figure 2-21) were still functioning at the end of experiments. Figures 3-58 and 3-59 show the force and strain time histories of rods B5 and H3, respectively.

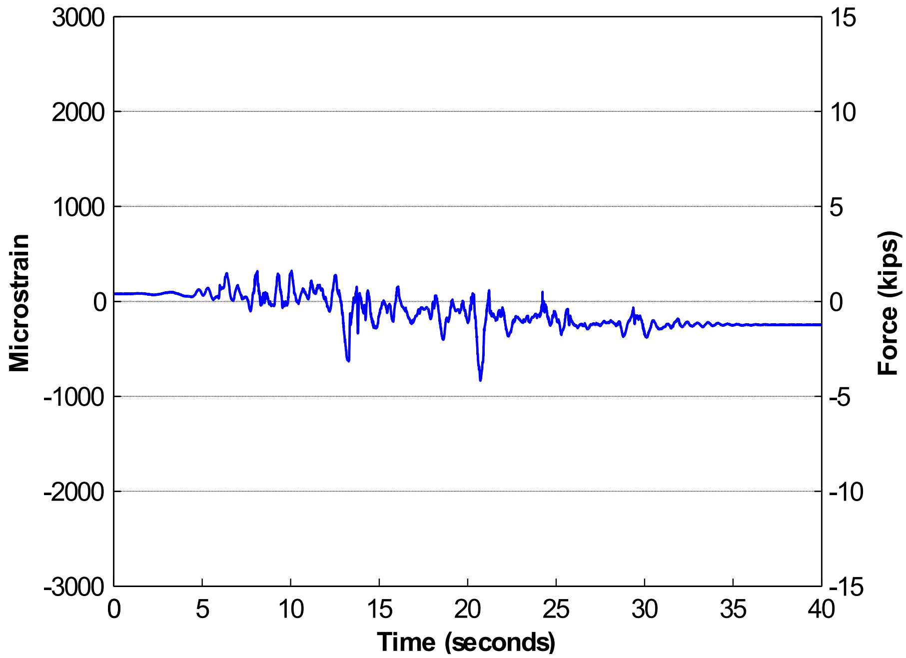

Figure 3-58 Run 225 - Strain and Force Time History for Rod B5

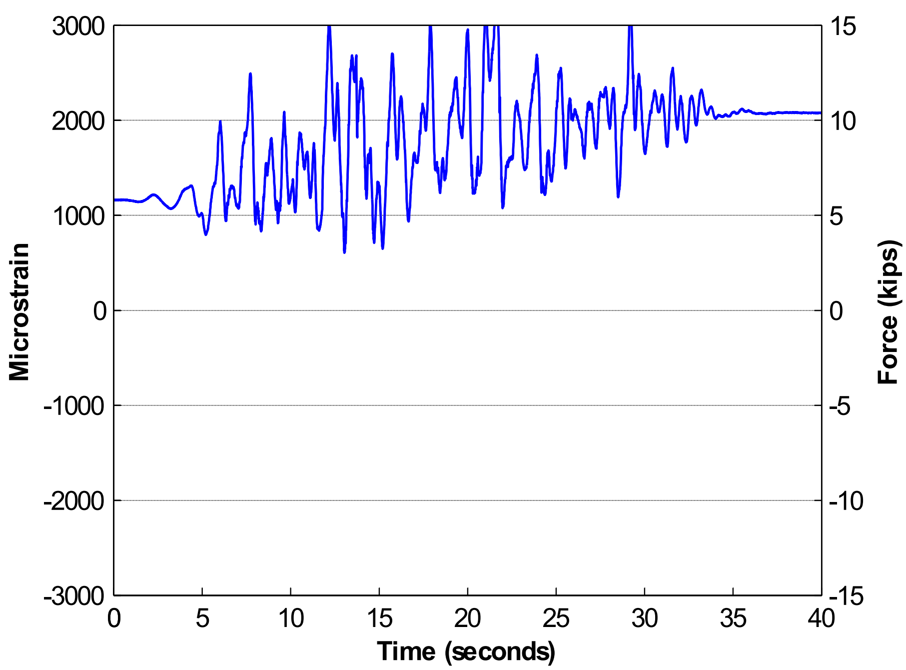

Figure 3-59 Run 225 - Strain and Force Time History for Rod H3

3.4.2.3 Observations

Leaks began forming at instrument nv36's location during run 192. This corresponds to a story drift of 1.08%. At run 202, leaks formed at instrument nv30's location. During run 205, hanger B3 buckled, as shown in figure 3-60. Leaks began to form on the heat exchanger during run 211 and during run 215, leaks formed at the elbow above the heat exchanger. This is a story drift of 2.17%. These two leaks were a steady dripping. See figures 3-61 and 3-62.

The threaded unbraced subassembly had no fracture in the heat exchanger connection flange at the end of all of the runs.

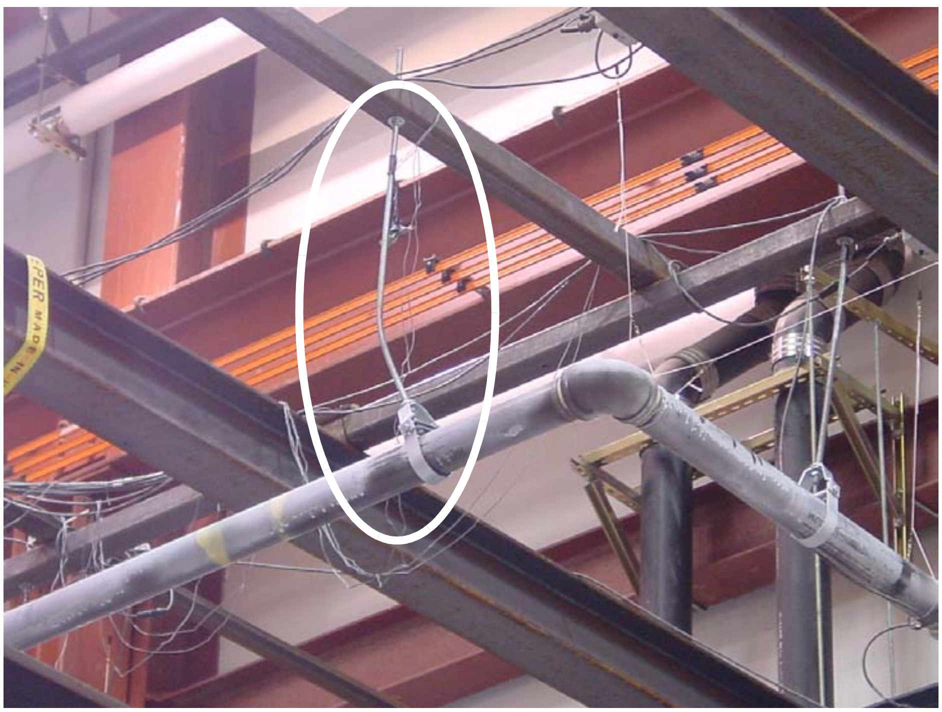

Figure 3-60 Buckled Hanger B3

Figure 3-61 Heat Exchanger Leakage

Figure 3-62 Elbow Above Heat Exchanger Leakage

3.4.3 Comparison of Threaded Braced and Unbraced Subassemblies

The following section will present a comparison between the threaded braced and unbraced subassemblies. Existing plots are discussed and new plots are presented in order to make the comparison. The experiments were halted after run 180 on the threaded braced subassembly due to extensive leakage at the heat exchanger flange. For comparative purposes, only the East/West excitations are discussed.

3.4.3.1 Displacement Comparison

The presence of braces makes the absolute displacement response of the braced subassembly always less than the unbraced case. Figures 3-36 and 3-51 illustrate the effect of the braces. Both figures show the displacement time history of nv17. The braced time history amplitude is much less than the unbraced time history.

The parallel displacement response of the unbraced case ranged from 2.59 times to 7.17 times the braced displacement. Instrument nv16 had the highest increase in displacement response.

The transverse response for the unbraced assembly was larger than the braced subassembly. The unbraced displacement response ranged anywhere from 1.09 times to 3.93 times the braced case.

Vertical response also increased for the unbraced case. The vertical unbraced response increase ranged from 1.26 times to 8.67 times the braced case. Instrument nv42 increased 8.67 times the braced case.

Final offsets in the system were large in the unbraced case. This can be seen by inspecting figures 3-63 and 3-64. Figure 3-63 shows the displacement time history of nv16 for the braced and unbraced cases. For comparative purposes, the initial displacement has been removed for clarity. As shown, there is a significant final offset in the position of the instrument. Figure 3-64 shows the displacement responses for instrument nv17 for the braced and unbraced cases. The final offset has not been removed to illustrate that there was a significant offset in the position of the instrument. This is shown illustratively in figures 3-39 and 3-56. Figure 3-56 shows the final displaced shape of the unbraced subassembly after run 225. As seen, the final displaced shape of the subassembly is much different than the final displaced shape of the braced subassembly, shown in figure 3-39.

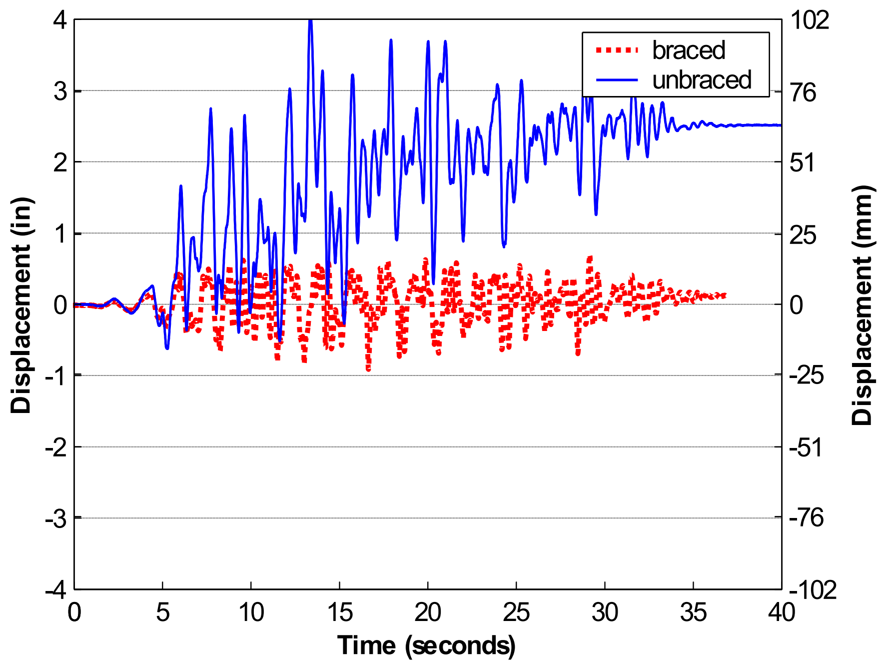

Figure 3-63 Threaded Braced and Unbraced Displacement Time History of nv16

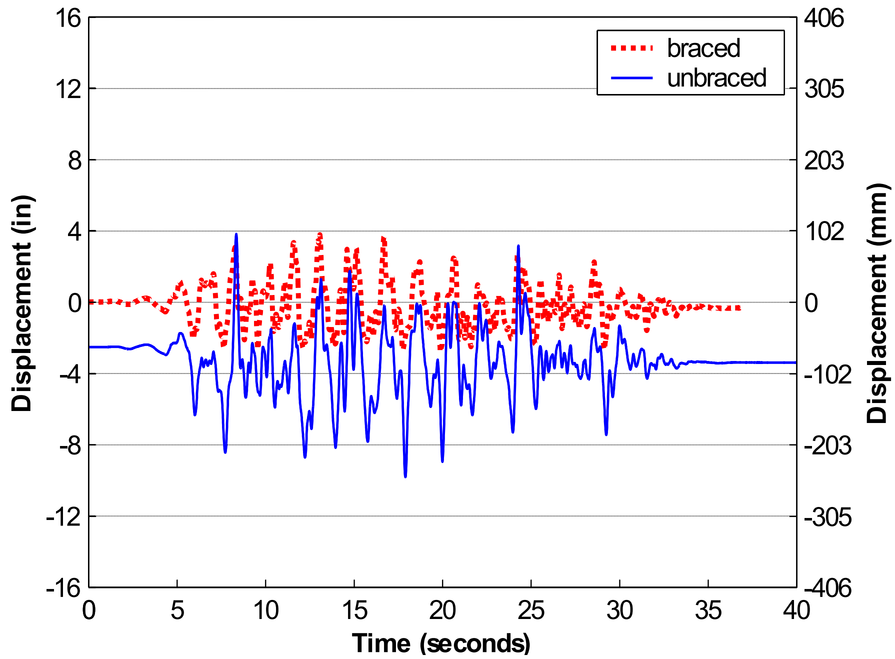

Figure 3-64 Threaded Braced and Unbraced Displacement Time History of nv17

The displacement responses for the threaded subassemblies for most of the instruments near the heat exchanger were out of phase with the table motion by 180 degrees. This can be seen in figures 3-65 and 3-66. Figure 3-67 shows the relative displacement time history of instrument nv15.

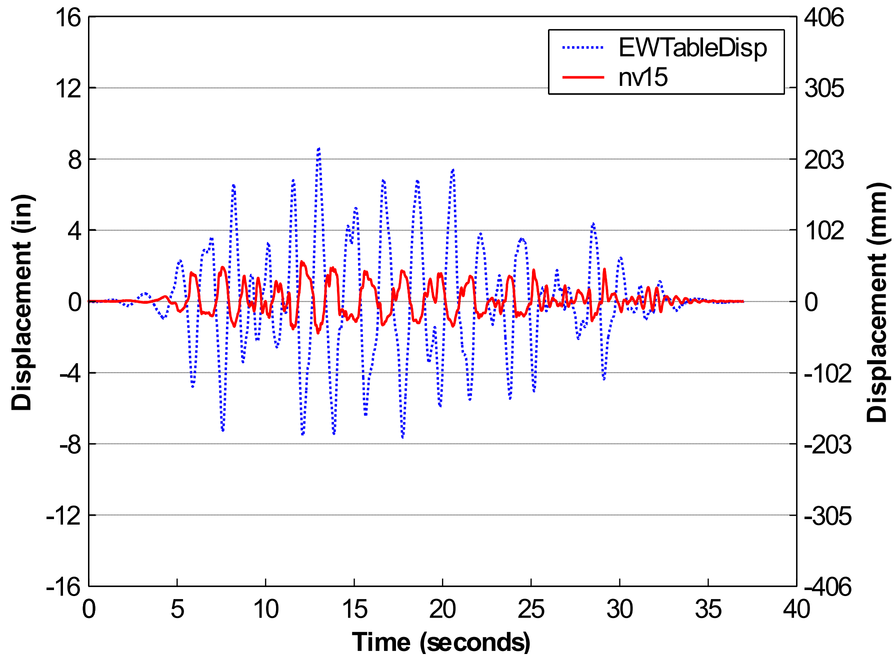

Figure 3-65 Run 180 - Absolute Displacement Time History of nv15

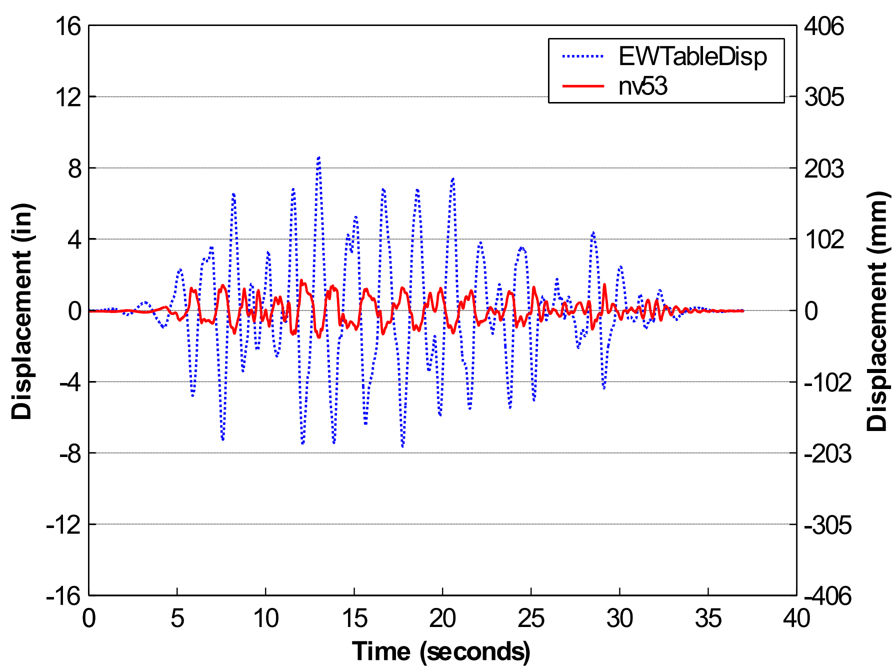

Figure 3-66 Run 180 - Absolute Displacement Time History of nv53

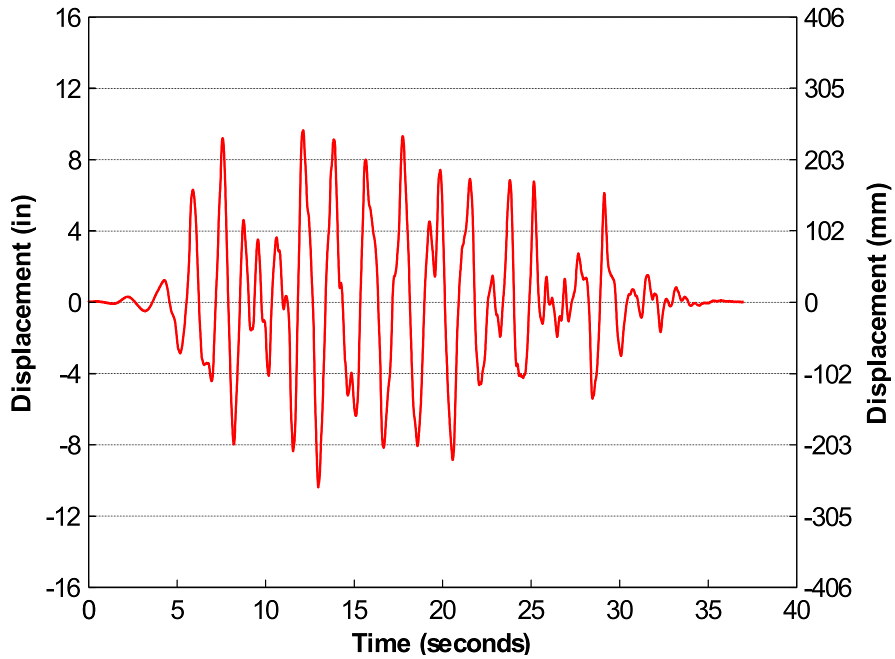

Figure 3-67 Run 180 - Relative Displacement Time History of nv15

3.5 Comparison of Welded Braced and Threaded Braced Subassemblies

This section compares the performance of the welded braced and threaded braced subassemblies. First, the braced subassemblies are discussed, followed by the unbraced subassemblies. For each comparison, displacements and overall performance are presented.

3.5.1 Observations

The welded system performed better than the threaded system. The welded subassembly was able to withstand 4.34% story drift with no damage. The threaded subassembly, however, began leaking at 2.17% and failed at 4.34%. Although the welded system did have some leakage at the heat exchanger flange, this was caused by the dynamic pushover and not a synthetic input motion. This leakage was due to a gasket being compressed between the two flanges and not pipe or flange fracture.

3.5.2 Displacement Response

The parallel displacement response for the braced welded subassembly ranged from 0.67 to 1.07 times the threaded response. The transverse displacement response for the braced welded subassembly ranged from 0.88 to 1.89 times the braced threaded response. The vertical displacement response for the braced welded system ranged from 0.71 to 2.11 times the braced threaded response. Figure 3-68 shows a comparative plot for nv17. In general, the displacement response for the braced systems were very similar.

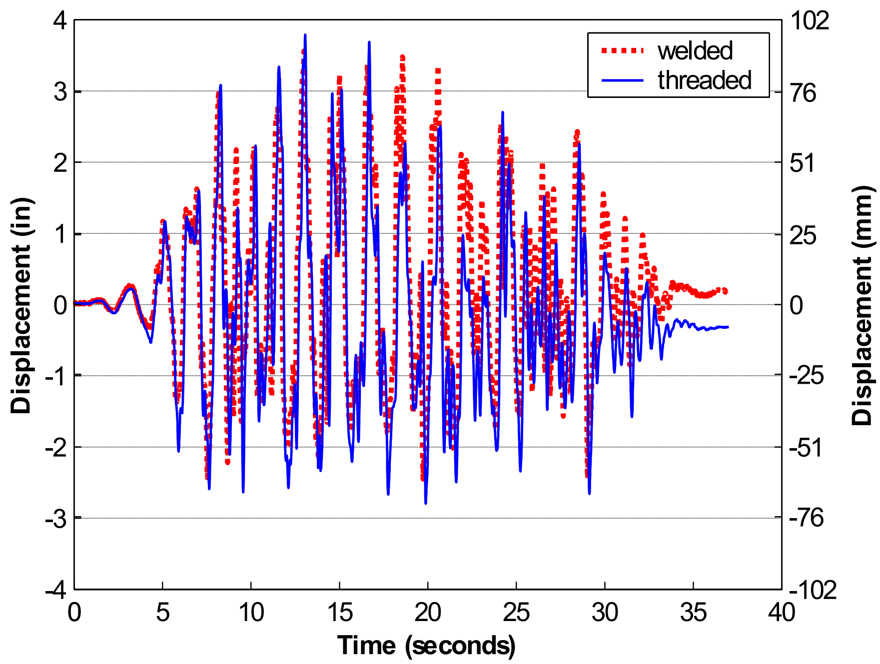

Figure 3-68 Braced Welded and Threaded Displacement Time History of nv17

3.6 Comparison of Welded Unbraced and Threaded Unbraced Subassemblies

This section compares the performance of the welded unbraced and threaded unbraced subassemblies. First, the braced subassemblies are discussed, followed by the unbraced subassemblies. For each comparison, displacements, accelerations and overall performance are presented.

3.6.1 Observations

Whereas the unbraced threaded subassembly experienced minor leaks, the unbraced welded subassembly suffered no leaks. The welded subassembly did see two vertical hanger rods fail. The threaded subassembly had no rods fail.

3.6.2 Displacement Response

For East/West excitation, the parallel displacement response for the unbraced welded subassembly ranged from 0.88 to 1.13 times the unbraced threaded response. The transverse displacement response for the welded subassembly ranged from 0.81 to 2.34 times the threaded response for East/West excitation. The vertical displacement response for the welded system ranged from 0.31 to 2.37 times the threaded response. Figure 3-69 shows a plot of nv17 for the threaded and welded unbraced cases. The initial offset has been removed from both tim histories for clarity.

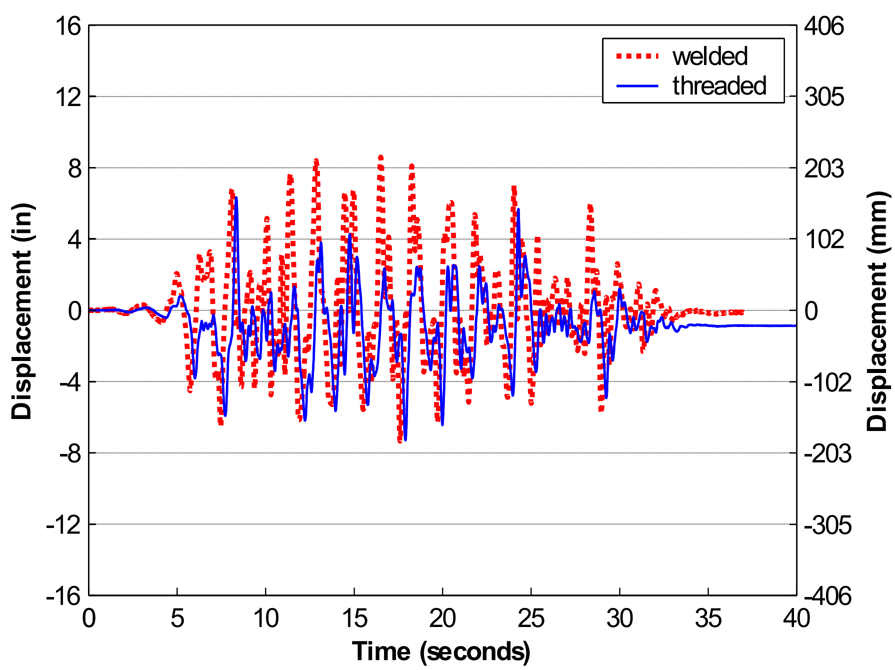

Figure 3-69 Unbraced Welded and Threaded Displacement Time History of nv17

For North/South, excitation the parallel displacement response for the welded subassembly ranged from 0.53 to 2.48 times the threaded subassembly response. The parallel displacement response for the unbraced welded subassembly was always greater than the unbraced threaded subassembly response. The welded response ranged from 0.61 to 0.73 times the unbraced threaded response.

3.7 Conclusions

This chapter described the experimental results found from conducting experiments on 4 full scale hospital piping systems. Comparative plots and tables were presented that summarized the experimental results.

The welded subassemblies performed much better than the threaded subassemblies. The welded subassemblies were able to withstand up to 4.34% story drift with no damage. The threaded subassemblies either had complete failure or leaking heavily at 4.34% story drift.

The ductility of the welded subassembly led to its superior performance over the threaded subassembly. The use of brittle, cast iron components in the threaded system lead to failure of the heat exchanger flange in the braced subassembly. There were also leaks in the threaded subassembly that can be attributed to the threaded components.

The large relative displacement also caused damage to the subassemblies. The threaded subassembly experienced a large amount of damage and leaks due to this relative displacement. The response of the subassembly being out of phase with the table motion by 180 degrees excerbated the relative displacements.

The displacement responses of the braced threaded and welded subassemblies were very similar. The use of braces effectively limited the in and transverse displacement response, but did not restrict the acceleration response. The acceleration response for the braced subassemblies were the same if not higher than the unbraced response.

SECTION 4
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

4.1 Summary

The main objective of this research was to identify the capacity characteristics of typical hospital piping subassemblies by subjecting them to varying levels of input motions. This objective was carried out by testing two subassemblies. One of the subassemblies used welded connections and the other used threaded connections. The welded connections used were forged steel. Forged steel components were necessary due to their superior weldability. The threaded connections were made out of cast iron. In threaded subassemblies, cast iron components are commonly used due to their lower cost over forged steel components. The subassemblies were hung from a stationary frame that sat outside of the table on the lab floor. The subassemblies were tested using a synthetic input motion derived from the ICBO AC156. The AC156 derives a response spectrum based off of the 1997 UBC. The computer programs SIMQKE and RSCTH were used to generate a synthetic input motion that conformed to this response spectrum. Both subassemblies were tested with and without seismic bracing. The seismic bracing used in the experiments was a cable style bracing commonly used in seismic applications.

4.2 Conclusions

- The braces were effective in limiting displacement response of the welded and threaded subassemblies.
- The braced welded subassembly performed better than the threaded subassembly. The welded subassembly was subjected to story drift up to 4.34% with no damage. The threaded subassembly began leaking at 2.17% drift and failed at 4.34%. The poor performance of the threaded subassembly is due to two reasons. The response of the threaded subassembly was out of phase with the table motion by 180 degrees. This caused higher relative displacements. Another reason for the poor performance of the threaded subassembly is the use of brittle, cast iron components.

- The unbraced welded subassembly performed marginally better than the unbraced threaded subassembly. The welded subassembly was subjected to 4.34% drift with no damage. The threaded subassembly began leaking at 1.08% drift.
- The 1997 UBC code limit on story drift is 2.5%. This means that the welded subassemblies meet code requirements with no damage. The threaded subassemblies could suffer significant damage at 2.5% drift.

4.3 Future investigation

- There are many different, commercially available bracing systems. Component and system testing would further expand the knowledge of piping subassemblies.
- The performed experiments produced a collection of results that can be used to correlate analytical models.
- More research should be done on other materials commonly used in hospitals.
- The experimental setup did not model multiple piping subassemblies, ceiling tiles and light fixtures. In a hospital, these systems occupy the same space. Studies should be done on how these systems interact and if this interaction produces different failure modes
- Vertical input motion should be studied. These relatively flexible systems might behave differently under vertical and horizontal excitation

SECTION 5

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APPENDIX A
BRACING TERMINOLOGY

1. Longitudinal brace - Restrains pipe displacement along its length. Requires a pipe clamp and a disruption in the insulation, but does not require a vertical hanger rod. A longitudinal brace should be located within 4 in (102 mm) of a hanger rod to prevent uplift due to horizontal seismic loads (Lama 1998). See figure A.1.

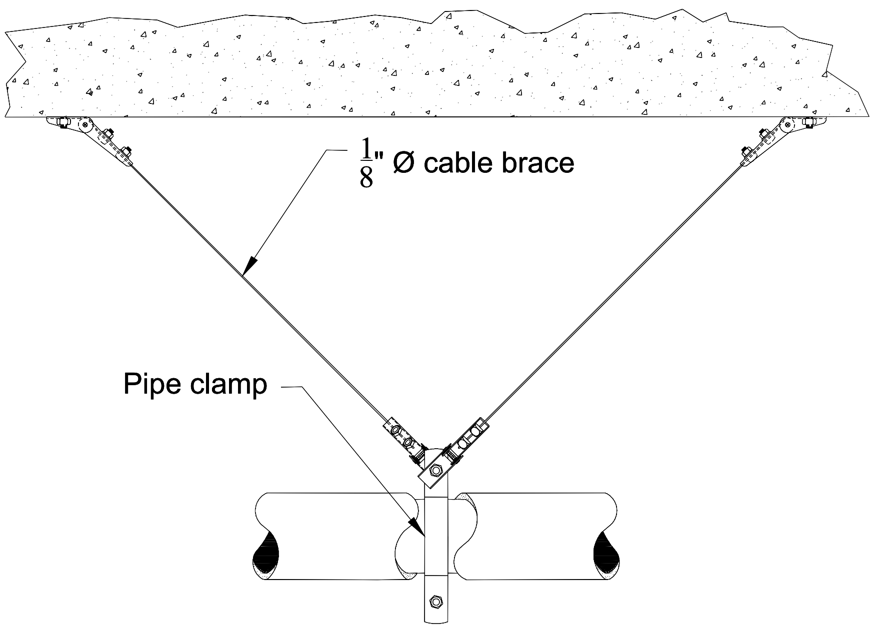

Figure A.1: Longitudinal Brace

2. Transverse brace - Restrains pipe displacement perpendicular to its length. Requires a clevis and vertical hanger rod. There is no disruption required in the insulation. See figure A.3.

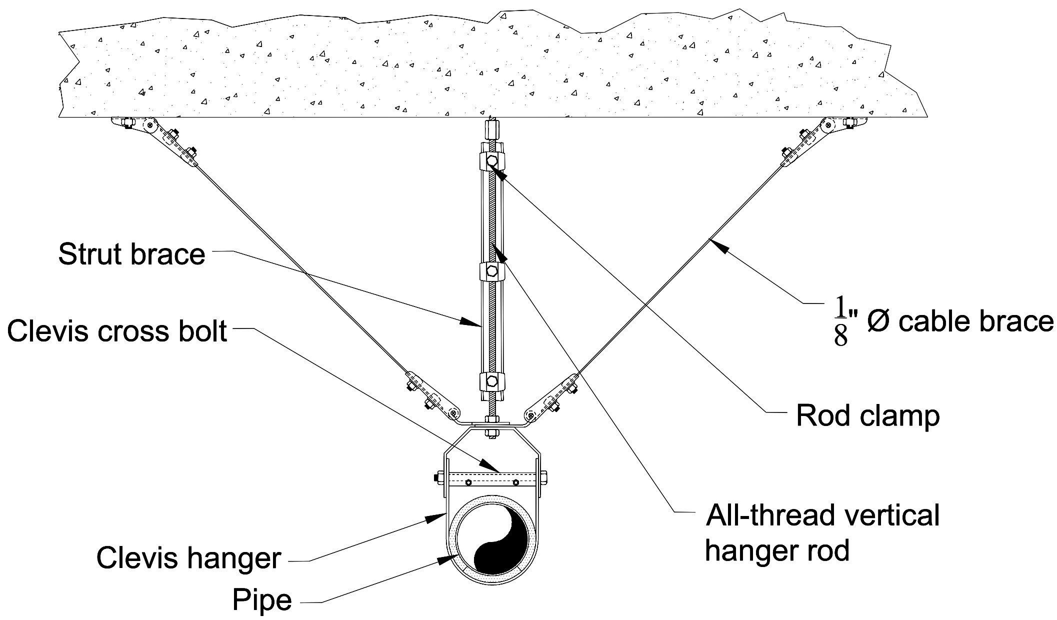

Figure A.2: Transverse Brace

3. Clevis - A saddle for the pipe to be supported from the vertical hanger rod. See figure A.3.

4. Clevis cross brace - A piece of hardware installed in the clevis to prevent the clevis

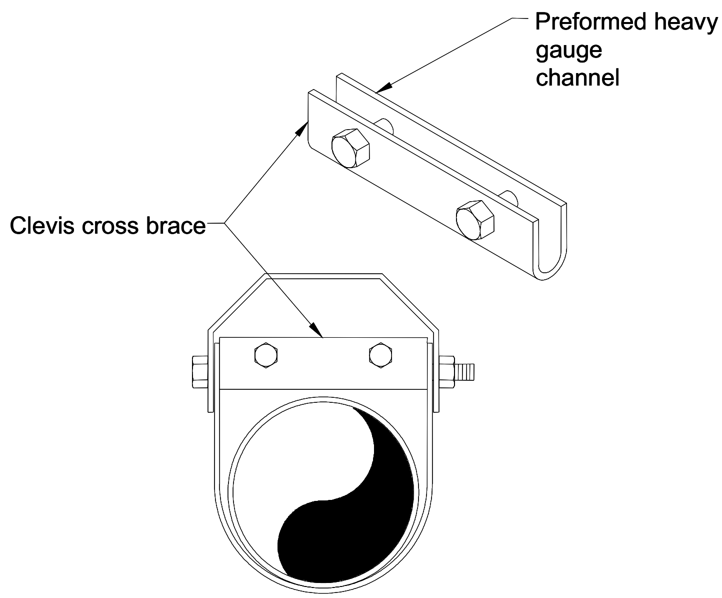

Figure A.3: Clevis Cross Brace

from buckling and distorting. See figure A.3.

5. Vertical hanger rod - Anchored to the superstructure and to the clevis. Supports the pipe vertically. See figure A.2.

6. Pipe clamp - Hardware installed around the pipe providing attachment for longitudinal brace. See figure A.1.

7. Rod clamp and strut brace -Increases the compression capacity of the vertical support rod by decreasing the unbraced length of the rod. See figure A.2.

APPENDIX B

AC156

ICBO Evaluation Service, Inc.
A subsidiary corporation of the International Conference of Building Officials
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ACCEPTANCE CRITERIA FOR SEISMIC QUALIFICATION TESTING OF NONSTRUCTURAL COMPONENTS

AC156

January 2000

(Effective February 1, 2000)

PREFACE

Evaluation reports issued by ICBO Evaluation Service, Inc. (ICBO ES), are based upon performance features of the *Uniform Building Code*™, ICBO *Uniform Mechanical Code*™ and related codes. Section 104.2.8 of the *Uniform Building Code* is the primary charging section upon which evaluation reports are issued. Section 104.2.8 reads as follows:

> The provisions of this code are not intended to prevent the use of any material, alternate design or method of construction not specifically prescribed by this code, provided any alternate has been approved and its use authorized by the building official.
>
> The building official may approve any such alternate, provided the building official finds that the proposed design is satisfactory and complies with the provisions of this code and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in suitability, strength, effectiveness, fire resistance, durability, safety and sanitation.
>
> The building official shall require that sufficient evidence or proof be submitted to substantiate any claims that may be made regarding its use. The details of any action granting approval of an alternate shall be recorded and entered in the files of the code enforcement agency.

The attached acceptance criteria has been issued to provide all interested parties with guidelines on implementing performance features of the codes. The criteria was developed and adopted following public hearings conducted by the Evaluation Committee and is effective on the date shown above. All reports issued or reissued on or after the effective date must comply with this criteria, while reports issued prior to this date may be in compliance with this criteria or with the previous edition. If the criteria is an updated version from a previous edition, solid vertical lines (▌) in the outer margin within the criteria indicate a technical change or addition from the previous edition. Deletion indicators (➧) are provided in the outer margins where a paragraph or item has been deleted if the deletion resulted from a technical change. This criteria may be further revised as the need dictates.

ICBO ES may consider alternate criteria, provided the proponent submits valid data demonstrating that the alternate criteria are at least equivalent to the attached criteria and otherwise meet the applicable performance requirements of the codes. Notwithstanding that a material, type or method of construction, or equipment, meets the attached acceptance criteria, or that it can be demonstrated that valid alternate criteria are equivalent and otherwise meet the applicable performance requirements of the codes, if the material, product, system or equipment is such that either unusual care in its installation or use must be exercised for satisfactory performance, or malfunctioning is apt to cause unreasonable property damage or personal injury or sickness relative to the benefits to be achieved by the use thereof, ICBO ES retains the right to refuse to issue or renew an evaluation report.

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ICBO Evaluation Service, Inc.

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ACCEPTANCE CRITERIA FOR SEISMIC QUALIFICATION TESTING OF NONSTRUCTURAL COMPONENTS

1.0 SCOPE AND PURPOSE

This criteria establishes minimum requirements for the issuance of ICBO Evaluation Service, Inc., evaluation reports on seismic qualification testing of non-structural systems as related to the seismic design requirements for architectural, mechanical, electrical and other nonstructural systems, components, and elements permanently attached to structures (hereinafter referred to as "equipment"), as specified in Section 1632 of the 1997 *Uniform Building Code*™ (hereinafter referred to as the "1997 code") and Section 1621 of the 2000 *International Building Code*™ (hereinafter referred to as the "2000 code").

2.0 NOMENCLATURE

2.1 1997 Code:

The following symbols and notations have the noted meanings in this document:

C_a = Seismic coefficient, as set forth in Table 16-Q of the 1997 code.

N_a = Near-source factor used in the determination of C_a in Seismic Zone 4, related to both the proximity of the building or structure to known faults having magnitudes and slip rates as set forth in Tables 16-S and 16-U of the 1997 code.

H_x = Equipment attachment elevation with respect to grade. For items at or below the base, H_x shall not be taken to be less than 0.0.

H_r = Building or structure roof elevation with respect to grade.

R_p = Equipment response modification factor. R_p represents the energy absorption capability of the equipment's structure and attachments, as set forth in Table 16-O of the 1997 code.

I_p = Equipment importance factor. I_p represents the greater of the life-safety importance factor of the component and the hazard exposure importance factor of the structure, as set forth in Table 16-K of the 1997 code.

2.2 2000 Code:

The following symbols and notations have the noted meanings in this document:

S_{DS} = Design spectral response acceleration at short periods, as determined in Section 1615.1.3 of the 2000 code.

z = Height in structure, with respect to grade, at point of attachment of equipment. For items at or below the base, z shall not be taken to be less than 0.0.

h = Average building/structure roof height relative to the base elevation.

R_p = Equipment response modification factor. R_p represents the energy absorption capability of the equipment's structure and attachments, as set forth in Table 1621.2 or 1621.3 of the 2000 code.

I_p = Equipment importance factor. I_p represents the greater of the life-safety importance factor of the component and the hazard exposure importance factor of the structure, as set forth in Section 1621.1.6 of the 2000 code.

2.3 Both 1997 Code and 2000 Code:

The following symbols and notations have the noted meanings in this document:

A_{RRS} = Spectral acceleration as calculated from design seismic forces, F_p, and equipment weight, W_p.
= F_p/W_p.

A_{FLX} = Horizontal spectral acceleration calculated for flexible equipment.

A_{RIG} = Horizontal spectral acceleration calculated for rigid equipment.

a_p = In-structure equipment amplification factor. The a_p represents the dynamic amplification of the equipment relative to the fundamental frequency of the building structure.

F_p = Horizontal seismic design force centered at the equipment's center of gravity, and distributed relative to the equipment's mass distribution.

W_p = Equipment operating weight.

3.0 DEFINITIONS

3.1 Attachments: The means by which equipment is secured or restrained by the seismic force resisting system of the building structure. Such attachments and restraints may include anchor bolting, welded connections and fasteners.

3.2 Biaxial Test: A dynamic test in which the test specimen is subjected to acceleration in one principal horizontal axis and the vertical axis simultaneously. The horizontal and vertical acceleration components are derived from two different input signals that are phase-incoherent.

3.3 Build-hold-decay (BHD): The build-hold-decay time interval envelope (5 ± 1 second, 15 ± 2, seconds and 10 ± 1 second, respectively) imposed on the drive signal of the shake table to simulate the nonstationary nature of an earthquake event.

3.4 Damping: An energy dissipation mechanism that reduces the amplification and broadens the vibratory response in the region of resonance. Damping is expressed as a percentage of critical damping.

3.5 Flexible Equipment: Component, including its attachments, having a fundamental period greater than 0.06 second (less than 16.67 Hz).

3.6 Equipment Force-resisting System: Equipment force-resisting systems are those members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between the equipment and the building structure. Equipment supports also transmit lateral forces and/or provide structural stability for the connected equipment.

3.7 Octave: The interval between two frequencies that have a frequency ratio of two.

3.8 One-third Octave: The interval between two frequencies that have a frequency ratio of $2^{1/3}$.

3.9 Required Response Spectrum (RRS): The response spectrum generated using the formulas and normalized spectra detailed in Section 6.5.1 of this acceptance criteria. The RRS constitutes a requirement to be met.

3.10 Rigid Equipment: A component, including its attachments, having a fundamental period less than or equal to 0.06 second (greater than or equal to 16.67 Hz).

3.11 Test Response Spectrum (TRS): The acceleration response spectrum that is developed from the actual time history of the motion of the shake table test.

3.12 Triaxial Test: A dynamic test in which the test specimen is subjected to acceleration in two principal horizontal axes and the vertical axis simultaneously. The two horizontal components and the vertical acceleration component are derived from three different input signals that are phase-incoherent.

3.13 Uniaxial Test: A dynamic test in which the test specimen is subjected to acceleration in one principal axis. The acceleration components are derived from a single input signal.

3.14 Unit Under Test (UUT): The equipment item to be qualification-tested.

3.15 Zero Period Acceleration (ZPA): The peak acceleration of motion time-history that corresponds to the high-frequency asymptote on the response spectrum. This acceleration corresponds to the maximum peak acceleration of the time history used to derive the spectrum. For the purposes of this acceptance criteria, the ZPA is assumed to be the acceleration response at 33.3 Hz.

4.0 REQUIRED INFORMATION

Sections 4.1 through 4.5 detail the necessary information to be provided for each UUT. Section 4 shall be a complete document, submitted by the UUT manufacturer or the manufacturer's representative.

4.1 Description: A general description of the UUT shall be provided, including the following items:

4.1.1 Description of the primary equipment product function.

4.1.2 Overall dimensions and weight of the UUT.

4.1.3 Restrictions or limitations on equipment use.

4.2 Seismic Parameters:

4.2.1 1997 Code:

The following seismic parameters, used to establish UUT seismic qualification test requirements, shall be provided:

H_x = Equipment attachment elevation with respect to grade.

H_r = Building or structure roof elevation with respect to grade.

C_a = Seismic coefficient.

I_p = Equipment importance factor.

4.2.2 2000 Code:

The following seismic parameters, used to establish UUT seismic qualification test requirements, shall be provided:

z = Equipment attachment elevation with respect to grade.

h = Average building/structure roof elevation with respect to grade.

S_{DS} = Spectral response acceleration at short period.

I_p = Equipment importance factor.

4.3 Functional and Operability Requirements: A listing and description shall be provided of the functional and operability equipment tests used to verify pre- or post-seismic-testing functional compliance.

4.4 Equipment Product Line Extrapolation and Interpolation: Testing every single configuration of a given equipment product line may not be feasible. Therefore, it may be necessary to select test specimens that adequately represent the entire equipment product line. The following criteria shall be used to establish UUT configuration requirements and represent an equipment product line (UUT configuration rationale shall be provided):

4.4.1 The UUT structure shall be similar to the major structural configurations being supplied in the product line. If more than one major structure is proposed, then these other configurations shall also be tested.

4.4.2 The configuration mounting the UUT to the shake table shall simulate mounting conditions for the product line. If several mounting configurations are used, they shall be simulated in the test.

4.4.3 The major subassembly components shall be included in the UUT. These components shall be mounted to the specimen structure at locations similar to those specified for proposed installations. The components shall be mounted to the structure using the same type of mounting hardware specified for proposed installations. Substitution of nonhazardous materials and fluids is permitted for verification of equipment or subassemblies that contain hazardous materials or fluids, provided the substitution does not reduce the functional demand on the equipment or subassembly.

4.4.4 The weight and mass distribution shall be similar to the typical weight and mass distribution of the equipment being represented. Weights equal to or heavier than the typical weight shall be acceptable.

4.4.5 Other equipment variations, such as number of units/components in production assemblies, indoor and outdoor applications, etc., shall also be represented by the test specimens.

4.5 Installation Instructions: Instructions shall include the following items:

4.5.1 Description of how the UUT will be installed in the field.

4.5.2 Description of how the UUT will be installed during the qualification test.

5.0 TESTING LABORATORIES AND REPORTS OF TESTS

5.1 Laboratories: Testing laboratories shall comply with the ICBO ES Acceptance Criteria for Laboratory Accreditation (AC89).

5.2 Reports and Product Sampling: Test reports and product sampling shall comply with the ICBO ES Acceptance Criteria for Test Reports and Product Sampling (AC85).

6.0 SEISMIC QUALIFICATION TEST PROCEDURE

6.1 Qualification Test Plan: The UUT shall be subjected to a seismic qualification test program, consisting of the following elements, to satisfy the intent of Section 1632 of the 1997 code and/or Section 1621 of the 2000 code for equipment:

- Pre-test Inspection and Functional Compliance Verification
- Resonance Search Tests
- Random Multifrequency Seismic Simulation Tests
- Post-Test Inspection and Functional Compliance Verification

All of the tests described in Section 6 shall be specified in the test plan. The test plan shall be a complete document.

6.2 Pre-test Inspection: Upon arrival at the test facility, the UUT shall be visually examined and results documented by the testing laboratory, to verify that no damage has occurred during shipping and handling.

6.3 Pre-test Functional Compliance Verification: Functional and/or operability tests, as specified in Section 4.3, shall be performed by the testing laboratory to verify pre-test functional performance. Functional testing could be performed at either the test facility or the UUT manufacturing facility. Test description and results shall be documented.

6.4 Seismic Simulation Test Setup: Seismic ground motion occurs simultaneously in all directions in a random fashion. However, for test purposes, uniaxial, biaxial or triaxial tests are allowed. If a uniaxial test is performed, the test shall be performed in three steps, with the UUT rotated after each step, such that all three principal axes of the UUT have been tested. If a biaxial test is performed, the test shall be performed in two steps, with the UUT rotated 90 degrees about the vertical axis for the second step.

6.4.1 Mounting: The UUT shall be mounted on the shake table in a manner that simulates the intended service mounting. The mounting method shall be the same as that recommended for actual service, and shall use the recommended bolt size, bolt type, bolt torque, configuration, weld pattern and type (if applicable), etc. The orientation of the UUT during the tests shall be such that the principal axes of the UUT are collinear with the axes of excitation of the shake table. A description of any interposing fixtures and connections between the UUT and the shake table shall be provided.

6.4.2 Monitoring: Sufficient vibration response monitoring instrumentation shall be used to allow determination of the applied acceleration levels in the principal horizontal and vertical axes of the shake table. Reference control accelerometers shall be mounted on the shake table at a location near the base of the UUT. Vibration response monitoring instrumentation shall also be used to determine the response of the UUT, at those points within the structure that reflect the UUT's response associated with its structural fundamental frequencies. Placement of the response sensors shall be at the discretion of the UUT manufacturer or the manufacturer's representative. The location and orientation of all vibration monitoring sensors shall be documented.

6.4.3 Resonance Search: A low-level (0.1 $\pm$.0.05 g peak input; a lower input level may be used to avoid equipment damage) single-axis sinusoidal sweep from 1 to 33 Hz shall be performed in each orthogonal UUT axis, to determine resonant frequencies. The sweep rate shall be two octaves per minute, or less, to ensure adequate time for maximum response at the resonant frequencies. Transmissibility plots of the in-line UUT response monitoring sensors shall be provided.

6.5 Multifrequency Seismic Simulation Tests:

6.5.1 Derivation of Seismic RRS: The equipment earthquake effects shall be determined for combined horizontal and vertical load effects. The required response spectra for the horizontal direction shall be developed based on the normalized response spectra shown in Figure 1, and the formula for total design horizontal force, F_p. The required response spectra for the vertical direction shall be developed based on two-thirds of the ground-level base acceleration. The seismic parameters specified in Section 4.2 shall be used to calculate the RRS levels as defined by A_{FLX} and A_{RIG}. The RRS shall be defined using a damping value equal to 5 percent of critical damping.

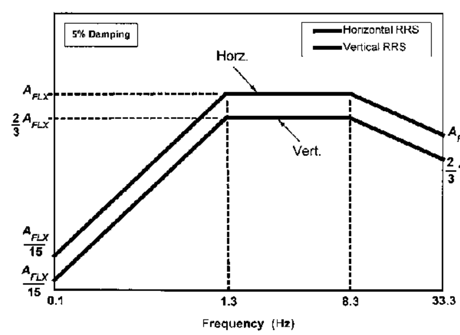


FIGURE 1—REQUIRED RESPONSE SPECTRUM, NORMALIZED FOR EQUIPMENT

6.5.1.1 1997 Code: The required response spectra for both horizontal and vertical directions shall be developed based on the formula for total design lateral force as specified in Equation (32-2) of the 1997 code:

$$F_p = \frac{a_p\ C_a}{\left(R_p/I_p\right)} \left(1 + 3\ \frac{H_x}{H_r}\right) W_p$$

The ratio (R_p / I_p) represents the allowable inelastic energy absorption capacity of the equipment's force-resisting system, and is considered to be a design reduction factor. During the seismic simulation test, the UUT will respond naturally to the excitation, and therefore (R_p / I_p) shall be set equal to 1, since the derived RRS controls the shaker table input motion. By definition, for frequencies less than 16.7 Hz, the equipment is considered flexible (a_p = 2.5), which corresponds to the strong portion of the RRS. For frequencies greater than 16.7 Hz, the equipment is considered rigid (a_p = 1.0), which corresponds to the ZPA. This results in two formulations that, when combined, define the horizontal RRS:

$$A_{FLX} = 2.5C_a\left(1 + 3\frac{H_x}{H_r}\right) \text{ and}$$

$$A_{RIG} = C_a\left(1 + 3\frac{H_x}{H_r}\right) \text{ and}$$

where:

A_{FLX} is limited to a maximum value of 4 C_a.

For vertical response, H_x may be taken to be 0.0 for all attachment heights.

6.5.1.2 2000 Code: The required response spectra for both horizontal and vertical directions shall be developed based on the formula for total design horizontal force, F_p, in accordance with either Section 6.5.1.2.1 or Section 6.5.1.2.2 of this acceptance criteria.

6.5.1.2.1 Building—Generic: When the building dynamic characteristics are not known or specified, the horizontal force requirements shall be as determined using Equation (1621.1.4-1) of the 2000 code:

$$F_p = \frac{0.4a_pS_{DS}}{(R_p/I_p)}\left(1 + 2\frac{z}{h}\right)W_p$$

The ratio (R_p / I_p) represents the allowable inelastic energy absorption capacity of the equipment's force-resisting system, and is considered a design reduction factor. During the seismic simulation test, the UUT will respond naturally to the excitation, and therefore (R_p / I_p) shall be set equal to 1.0, since the derived RRS controls the shaker table input motion. By definition, for frequencies less than 16.7 Hz, the equipment is considered flexible (a_p = 2.5), which corresponds to the strong portion of the RRS. For frequencies greater than 16.7 Hz, the equipment is considered rigid (a_p = 1.0), which corresponds to the ZPA. This results in two formulations that, when combined, define the horizontal RRS:

$$A_{FLX} = S_{DS}\left(1 + 2\frac{z}{h}\right) \quad \text{and}$$

$$A_{RIG} = 0.4\ S_{DS}\left(1 + 2\frac{z}{h}\right)$$

where:

A_{FLX} is limited to a maximum value of 1.6 S_{DS}.

For vertical response, z may be taken to be 0.0 for all attachment heights.

6.5.1.2.2 Building Specific: Equipment requirements may alternatively be defined using the results of a dynamic analysis procedure for the seismic design of a specific building per Section 1618 of the 2000 code. Response spectra constructed in accordance with Section 1615, using R = 1.0, shall be used in the analysis of the building structural system. Use of results of the dynamic analysis procedure yields the following parameters for A_{FLX} and A_{RIG}:

$A_{FLX} = 2.5\ A_x\ a_i$ and $A_{RIG} = A_x\ a_i$

where:

A_x is the torsional amplification factor per Section 1617.4.4.5 of the 2000 code and a_i is the acceleration at level i, obtained from the dynamic analysis procedure.

For vertical response, two-thirds of the response spectra used in the analysis of the building structural system may be used.

6.5.2 Derivation of Test Input Motion: To meet the required response spectra as defined in Section 6.5.1, the corresponding shake table drive signals shall be non-stationary broadband random excitations having an energy content of from 1 to 33 Hz. The drive signal composition shall be multiple-frequency random excitations, the amplitudes of which are adjusted either manually or automatically based on a maximum one-third-octave bandwidth resolution. The duration of the input motion shall be 30 $\pm$ 4 seconds, with the non-stationary character being synthesized by an input signal build-hold-decay envelope of 5 seconds, 15 seconds, and 10 seconds, respectively. Independent random signals that result from an aggregate of the maximum-one-third-octave narrowband signals shall be used as the excitation to produce phase-incoherent motions in the principal horizontal and vertical axes of the shake table.

6.5.3 Test Response Spectrum Analysis: The test response spectrum (TRS) shall be computed using either justifiable analytical techniques or response spectrum analysis equipment using the control accelerometers located at the UUT base per Section 6.4.2. The TRS shall be calculated using a damping value equal to 5 percent of critical damping. The TRS must envelop the RRS based on the maximum-one-third-octave bandwidth resolution over the frequency range from 1 to 33 Hz, or up to the shake table limits. The amplitude of each maximum-one-third octave bandwidth shall be independently adjusted in each of the principal axes until the TRS envelops the RRS, within the limitations of the test machine. The TRS should not exceed the RRS by more than 30 percent over the amplified frequency range. It is recommended that the TRS be computed with maximum-one-sixth-octave bandwidth resolution. Any acceleration-signal filtering performed within the range of analysis must be defined. The general requirement for the enveloping of the RRS by the TRS can be modified under the following conditions:

6.5.3.1 In those cases in which it can be shown by use of the resonance search in Section 6.4.3 that no resonance response phenomena exist below 5 Hz, it is required to envelop the RRS only down to 3.5 Hz. Excitation must continue to be maintained in the 1 Hz to 3.5 Hz range, within the limitations of the shake table.

6.5.3.2 When resonance phenomena exist below 5 Hz, it is required to envelop the RRS only down to 70 percent of the lowest frequency of resonance.

6.5.3.3 When the absence of resonance response phenomena below 5 Hz cannot be justified, the general requirement applies and the low-frequency enveloping should be maintained down to 1 Hz.

6.5.3.4 Under any circumstances, failure to envelop the RRS at or above 3.5 Hz must be justified.

In the performance of a test program, the TRS may, on occasion, not fully envelop the RRS. The general requirement for a retest may be exempted if the following criteria are met:

6.5.3.4.1 A point of the TRS may fall below the RRS by 10 percent or less, provided the adjacent one-sixth-octave points are at least equal to the RRS.

6.5.3.4.2 A maximum of five of the one-sixth octave analysis points may be below the RRS, as under the same constraints as noted in Section 6.5.3.4.1, provided they are at least one octave apart.

6.6 Post-test Inspection: The UUT shall be visually examined and results documented upon completion of the multifrequency seismic simulation tests performed in accordance with Section 6.5. The following conditions shall apply:

6.6.1 Attachments: Equipment design must ensure that the anchored UUT will not leave its mounting and cause damage to other building components or injury to personnel during the seismic event. Structural integrity of the equipment attachment system shall be maintained.

6.6.2 Equipment Force-resisting System: Equipment design must ensure that structural failure of the lateral force-resisting system of the UUT would not cause a safety hazard to life or limb. Structural damage, such as local yielding, to UUT force-resisting members is acceptable as long as the damage does not interfere with functional performance of the equipment and does not cause a life or limb safety hazard. Structural members and joints comprising the UUT force-resisting system shall be allowed minor fractures and anomalies, as long as a brittle fracture would not result from a subsequent load cycle (aftershock event) and cause a life hazard.

6.7 Post-test Functional Compliance Verification: Equipment being qualified must be capable of performing its intended functions after the seismic event. Functionality and/or operability tests, as specified in Section 4.3, shall be performed by the testing laboratory to verify post-test functional and operational performance. Functional testing may be performed at either the test facility or the UUT manufacturing facility. Test results shall be described and documented. Requirements of this section are satisfied if one of the following criteria is met:

6.7.1 For I_p = 1.0: At the completion of the seismic testing, the UUT does not pose a life or limb safety hazard due to collapse or due to major subassemblies becoming separated, and materials deemed to be hazardous have not been released into the environment in quantities greater than the exempted amounts listed in the code.

6.7.2 For I_p > 1.0: The equipment is deemed to be essential to the continued operation of a facility, and/or essential to maintaining critical life-support systems, and/or contains materials deemed to be hazardous, to humans or the environment, in quantities greater than the exempted amounts listed in the code. After completion of the seismic testing, the UUT shall satisfy the functional and operational tests specified in Section 4.3, and materials deemed to be hazardous shall not have been released into the environment in quantities greater than the exempted amounts listed in the code.

7.0 FINAL SUBMITTAL

The final submittal shall consist of a qualification test plan and test reports. Contents of the final submittal are described in the following subsections:

7.1 Qualification Test Plan: The accredited laboratory shall generate a test plan based on the UUT required information in Section 4. The test plan shall document the test procedure as specified in Section 6.

7.2 Test Report: The accredited laboratory shall report on the qualification testing performed in accordance with the qualification test plan. In addition to the information requested in Section 5, the test report must include the following information:

1. Identification of equipment being qualified.
2. Seismic parameters and derived RRS levels for equipment that is being qualified.
3. Testing equipment functionality/operability requirements.
4. Test facility location.
5. Testing equipment and calibration.
6. Equipment mounting details, including all interface connections.
7. Test data, including proof of performance, TRS plots, acceleration time histories of the shake table input motion, transmissibility plots, etc.
8. Test results and conclusions (including a statement on any anomalies, and justification that the equipment is still qualified).

8.0 REFERENCES

8.1 *Uniform Building Code*™, 1997 Edition. International Conference of Building Officials, Whittier, California.

8.2 *International Building Code*®, 2000 Edition. International Code Council, Falls Church, Virginia.

8.3 *ANSI/IEEE Standard 344-1987, IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations* (1987). The Institute of Electrical and Electronics Engineers, Inc.

8.4 *IEEE Standard 693-1997, IEEE Recommended Practice for Seismic Design of Substations* (1997). The Institute of Electrical and Electronics Engineers, Inc.

APPENDIX C

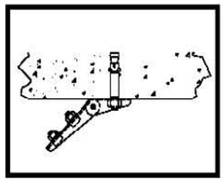
BRACING DESIGN TABLES

SEISMIC RESTRAINT GUIDELINES

INDIVIDUALLY SUPPORTED SYSTEMS

MINIMUM 3000 PSI STONE AGGREGATE CONCRETE SLAB

EXPANSION ANCHORS



Pipe Dia. (in.)	Maximum Weight per Foot (lbs.)	Maximum Brace Spacing (ft.) for Specified Seismic 'g' Loads * 0.25g Tran.	0.25g Long.	0.5g Tran.	0.5g Long.	1.0g Tran.	1.0g Long.	Option 1 SCB Size	Option 1 Cable Dia. (in.)	Option 2 SSB Size	Option 2 Minimum Brace Size (in.)	Concrete Anchors Quantity Req'd	Concrete Anchors Dia (in.)	Concrete Anchors Embed (in.)
1	2.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	2 1/4
1 1/4	3.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	2 1/4
1 1/2	4.5	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	2 1/4
2	6.2	50	80	40	80	40	68	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4 1/8
2 1/2	9.1	50	80	40	80	40	61	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4 1/8
3	12.1	50	80	40	80	40	46	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4 1/8
4	18.3	50	80	40	60	30	30	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4 1/8
5	26.6	50	80	40	42	21	21	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4 1/8
6	34.8	50	80	40	50	25	25	SCB-2	3/16	SSB-2	L3 x 3 x 1/4	1	5/8	5 1/8
8	55.1	50	80	40	52	26	26	SCB-2	3/16	SSB-2	L3 x 3 x 1/4	2	5/8	5 1/8
10	80.2	50	72	36	36	18	18	SCB-2	3/16	SSB-3	L3 x 3 x 1/4	2	5/8	5 1/8
12	109	50	80	40	40	20	20	SCB-3	1/4	SSB-3	L4 x 4 x 1/4	4	5/8	5 1/8
14	122	50	72	36	36	18	18	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5 1/8
16	150	50	56	28	28	14	14	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5 1/8
18	190	44	44	22	22	11	11	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5 1/8
20	214	50	72	36	36	18	18	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	4	5/8	5 1/8
24	289	50	52	26	26	13	13	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	4	5/8	5 1/8

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.67; for 2:1 brace angle, divide by 2.33.
Example: For 3" diameter pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 40/1.67 = 23 ft.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 12" diameter pipe, transverse brace spacing = 20/0.74 = 27 feet. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H1, H2 and H3 for the maximum seismic loads.

NOTES:

Anchorage is based on attachment with an ITW Ramset/Red Head Trubolt Wedge Anchor, ICBO Report ER-1372/1997, Table 7, 8 and 9, embedded headed stud or approved equal.
Where (2) anchors are specified, the SCB or SSB w/SLDB-2000 is required for attachment to structure. (Ref. Page H2 and X9)
Where (4) anchors are specified, the SCB or SSB w/SLDB-4000 is required for attachment to structure. (Ref. Page H3 and X10)
An increase in SCB or SSB size may be required to accommodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 12ga formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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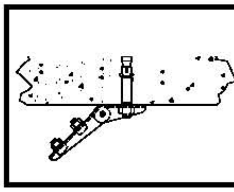
SEISMIC RESTRAINT GUIDELINES

INDIVIDUALLY SUPPORTED SYSTEMS

MINIMUM 20,680 kPa
STONE AGGREGATE CONCRETE SLAB

EXPANSION ANCHORS



Pipe	Maximum	Maximum Brace Spacing (m)						Option 1		Option 2		Concrete		
Dia.	Weight	for Specified Seismic 'g' Loads *							Cable		Minimum	Anchors		
	per Meter	0.25g		0.5g		1.0g		SCB	Dia.	SSB	Brace Size	Quantity	Dia.	Embed.
(mm)	(kg)	Tran.	Long.	Tran.	Long.	Tran.	Long.	Size	(mm)	Size	(mm)	Req'd	(mm)	(mm)
25	4.1	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	57
32	5.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	57
38	6.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	57
51	9.2	15.2	24.4	12.2	24.4	12.2	20.7	SCB-1	3	SSB-1	L51x51x6	1	13	105
64	13.5	15.2	24.4	12.2	24.4	12.2	18.6	SCB-1	3	SSB-1	L51x51x3	1	13	105
76	18.0	15.2	24.4	12.2	24.4	12.2	14.0	SCB-1	3	SSB-1	L51x51x3	1	13	105
102	27.3	15.2	24.4	12.2	18.3	9.1	9.1	SCB-1	3	SSB-1	L51x51x3	1	13	105
127	39.5	15.2	24.4	12.2	12.8	6.4	6.4	SCB-1	3	SSB-1	L51x51x3	1	13	105
152	51.8	15.2	24.4	12.2	15.2	7.6	7.6	SCB-2	5	SSB-2	L76x76x6	1	16	130
203	82.0	15.2	24.4	12.2	15.8	7.9	7.9	SCB-2	5	SSB-2	L76x76x6	2	16	130
254	119.3	15.2	21.9	11.0	11.0	5.5	5.5	SCB-2	5	SSB-3	L76x76x6	2	16	130
305	162.2	15.2	24.4	12.2	12.0	6.1	6.1	SCB-3	6	SSB-3	L102x102x6	4	16	130
356	181.5	15.2	21.9	11.0	11.0	5.5	5.5	SCB-3	6	SSB-4	L102x102x6	4	16	130
406	222.6	15.2	17.1	8.5	8.5	4.3	4.3	SCB-3	6	SSB-4	L102x102x6	4	16	130
457	282.7	13.4	13.4	6.7	6.7	3.4	3.4	SCB-3	6	SSB-4	L102x102x6	4	16	130
508	317.7	15.2	21.9	11.0	11.0	5.5	5.5	SCB-4	10	SSB-4	L102x102x6	4	16	130
610	430.1	15.2	15.8	7.9	7.9	4.0	4.0	SCB-4	10	SSB-4	L102x102x6	4	16	130

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.67; for 2:1 brace angle, divide by 2.33.
Example: For 76 mm dia. pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 12.2/1.67 = 7.3 m.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 305 mm dia. pipe, transverse brace spacing = 6.1/0.74 = 8.2 m. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H1, H2 and H3 for the maximum seismic loads.

NOTES:

Anchorage is based on attachment with an ITW Ramset/Red Head Trubolt Wedge Anchor, ICBO Report ER-1372/1997, Table 7, 8 and 9, embedded headed stud or approved equal.
Where (2) anchors are specified, the SCB or SSB w/SLDB-2000 is required for attachment to structure. (Ref. Page H2 and X9)
Where (4) anchors are specified, the SCB or SSB w/SLDB-4000 is required for attachment to structure. (Ref. Page H3 and X10)
An increase in SCB or SSB size may be required to accommodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 2.7 mm formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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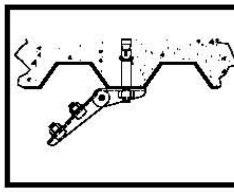
SEISMIC RESTRAINT GUIDELINES

INDIVIDUALLY SUPPORTED SYSTEMS

MINIMUM 3000 PSI LIGHTWEIGHT CONCRETE DECK

EXPANSION ANCHORS



Pipe Dia. (in.)	Maximum Weight per Foot (lbs.)	Maximum Brace Spacing (ft.) for Specified Seismic 'g' Loads * 0.25g Tran.	0.25g Long.	0.5g Tran.	0.5g Long.	1.0g Tran.	1.0g Long.	Option 1 SCB Size	Option 1 Cable Dia. (in.)	Option 2 SSB Size	Option 2 Minimum Brace Size (in.)	Concrete Anchors Quantity Req'd	Concrete Anchors Dia. (in.)	Concrete Anchors Embed. (in.)
1	2.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
1 1/4	3.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
1 1/2	4.5	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
2	6.2	50	80	40	80	40	60	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
2 1/2	9.1	50	80	40	80	40	41	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
3	12.1	50	80	40	62	31	31	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
4	18.3	50	80	40	40	20	20	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
5	26.6	50	56	28	28	14	14	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	3
6	34.8	50	64	32	32	16	16	SCB-2	3/16	SSB-2	L2 x 2 x 1/4	1	5/8	5
8	55.1	50	80	40	50	25	25	SCB-2	3/16	SSB-2	L3 x 3 x 1/4	2	5/8	5
10	80.2	50	68	34	34	17	17	SCB-2	3/16	SSB-3	L3 x 3 x 1/4	2	5/8	5
12	109	50	80	40	40	20	20	SCB-3	1/4	SSB-3	L4 x 4 x 1/4	4	5/8	5
14	122	50	72	36	36	18	18	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5
16	150	50	56	28	28	14	14	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5
18	190	44	44	22	22	11	11	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	4	5/8	5
20	214	50	52	26	26	13	13	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	4	5/8	5
24	289	36	36	18	18	9	9	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	4	5/8	5

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.68; for 2:1 brace angle, divide by 2.38.
Example: For 3" diameter pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 40/1.68 = 23 ft.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 12" diameter pipe, transverse brace spacing = 20/0.74 = 27 feet. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H4, H5 and H6 for the maximum seismic loads.

NOTES:

Anchorage is based on attachment with an ITW Ramset/Red Head Trubolt Wedge Anchor, ICBO Report ER-1372/1997, Table 7, 8 and 14, embedded headed stud or approved equal.
Where (2) anchors are specified, the SCB or SSB w/SLDB-2000 is required for attachment to structure. (Ref. Page H5 and X9)
Where (4) anchors are specified, the SCB or SSB w/SLDB-4000 is required for attachment to structure. (Ref. Page H6 and X10)
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 12ga formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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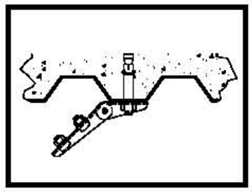
SEISMIC RESTRAINT GUIDELINES	INDIVIDUALLY SUPPORTED SYSTEMS MINIMUM 20,680 kPa LIGHTWEIGHT CONCRETE DECK EXPANSION ANCHORS



Pipe Dia. (mm)	Maximum Weight per Meter (kg)	Maximum Brace Spacing (m) for Specified Seismic 'g' Loads *						Option 1		Option 2		Concrete Anchors		
		0.25g		0.5g		1.0g		SCB Size	Cable Dia. (mm)	SSB Size	Minimum Brace Size (mm)	Quantity Req'd	Dia. (mm)	Embed. (mm)
		Tran.	Long.	Tran.	Long.	Tran.	Long.							
25	4.1	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	76
32	5.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	76
38	6.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	76
51	9.2	15.2	24.4	12.2	24.4	12.2	18.3	SCB-1	3	SSB-1	L51x51x3	1	13	76
64	13.5	15.2	24.4	12.2	24.4	12.2	12.5	SCB-1	3	SSB-1	L51x51x3	1	13	76
76	18.0	15.2	24.4	12.2	18.9	9.4	9.4	SCB-1	3	SSB-1	L51x51x3	1	13	76
102	27.3	15.2	24.4	12.2	12.2	6.1	6.1	SCB-1	3	SSB-1	L51x51x3	1	13	76
127	39.5	15.2	17.1	8.5	8.5	4.3	4.3	SCB-1	3	SSB-1	L51x51x3	1	13	76
152	51.8	15.2	19.5	9.8	9.8	4.9	4.9	SCB-2	5	SSB-2	L51x51x3	1	16	127
203	82.0	15.2	24.4	12.2	15.2	7.6	7.6	SCB-2	5	SSB-2	L76x76x6	2	16	127
254	119.3	15.2	20.7	10.4	10.4	5.2	5.2	SCB-2	5	SSB-3	L76x76x6	2	16	127
305	162.2	15.2	24.4	12.2	12.2	6.1	6.1	SCB-3	6	SSB-3	L102x102x6	4	16	127
356	181.5	15.2	21.9	11.0	11.0	5.5	5.5	SCB-3	6	SSB-4	L102x102x6	4	16	127
406	222.6	15.2	17.1	8.5	8.5	4.3	4.3	SCB-3	6	SSB-4	L102x102x6	4	16	127
457	282.7	13.4	13.4	6.7	6.7	3.4	3.4	SCB-3	6	SSB-4	L102x102x6	4	16	127
508	317.7	15.2	15.8	7.9	7.9	4.0	4.0	SCB-4	10	SSB-4	L102x102x6	4	16	127
610	430.1	11.0	11.0	5.5	5.5	2.7	2.7	SCB-4	10	SSB-4	L102x102x6	4	16	127

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.68; for 2:1 brace angle, divide by 2.38.
Example: For 76 mm dia. pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 12.2/1.68 = 7.3 m.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 305 mm diameter pipe, transverse brace spacing = 6.1/0.74 = 8.2 m. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H4, H5 and H6 for the maximum seismic loads.

NOTES:
Anchorage is based on attachment with an ITW Ramset/Red Head Trubolt Wedge Anchor, ICBO Report ER-1372/1997, Table 7, 8 and 14, embedded headed stud or approved equal.
Where (2) anchors are specified, the SCB or SSB w/SLDB-2000 is required for attachment to structure. (Ref. Page H5 and X9)
Where (4) anchors are specified, the SCB or SSB w/SLDB-4000 is required for attachment to structure. (Ref. Page H6 and X10)
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 2.7 mm formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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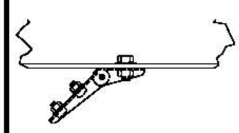
SEISMIC RESTRAINT GUIDELINES	INDIVIDUALLY SUPPORTED SYSTEMS STRUCTURAL STEEL BEAM OR MEMBER A307 BOLT OR 70XX WELD	

Pipe Dia. (in.)	Maximum Weight per Foot (lbs.)	Maximum Brace Spacing (ft.) for Specified Seismic 'g' Loads * 0.25g Tran.	0.25g Long.	0.5g Tran.	0.5g Long.	1.0g Tran.	1.0g Long.	Option 1 SCB Size	Option 1 Cable Dia. (in.)	Option 2 SSB Size	Option 2 Minimum Brace Size (in.)	Steel Bolt Qty. Req'd	Steel Bolt Dia. (in.)	Weld Size (in.)
1	2.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	1/8
1 1/4	3.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	1/8
1 1/2	4.5	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	1/8
2	6.2	50	80	40	80	40	68	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	1/8
2 1/2	9.1	50	80	40	80	40	75	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	1/8
3	12.1	50	80	40	80	40	56	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	1/8
4	18.3	50	80	40	74	37	37	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	1/8
5	26.6	50	80	40	50	25	25	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	1/8
6	34.8	50	80	40	80	40	41	SCB-2	3/16	SSB-2	L3 x 3 x 1/4	1	5/8	1/8
8	55.1	50	80	40	52	26	26	SCB-2	3/16	SSB-2	L3 x 3 x 1/4	1	5/8	1/8
10	80.2	50	72	36	36	18	18	SCB-2	3/16	SSB-3	L3 x 3 x 1/4	1	5/8	3/16
12	109	50	80	40	40	20	20	SCB-3	1/4	SSB-3	L4 x 4 x 1/4	1	3/4	3/16
14	122	50	72	36	36	18	18	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	1	3/4	3/16
16	150	50	56	28	28	14	14	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	1	3/4	3/16
18	190	44	44	22	22	11	11	SCB-3	1/4	SSB-4	L4 x 4 x 1/4	1	3/4	3/16
20	214	50	80	40	44	22	22	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	1	1 1/4	3/16
24	289	50	64	32	32	16	16	SCB-4	3/8	SSB-4	L4 x 4 x 1/4	1	1 1/4	3/16

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.5; for 2:1 brace angle, divide by 2.
Example: For 3" diameter pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 40/1.5 = 26 ft.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 12" diameter pipe, transverse brace spacing = 20/0.74 = 27 feet. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H7 to H12 for the maximum seismic loads.

NOTES:
Anchorage is based on attachment with a Standard ASTM A307 Quality Bolts or E70xx electrode welds.
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 12ga formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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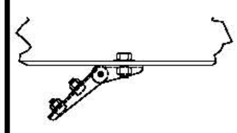
SEISMIC RESTRAINT GUIDELINES	INDIVIDUALLY SUPPORTED SYSTEMS STRUCTURAL STEEL BEAM OR MEMBER A307 BOLT OR 70XX WELD	

Pipe Dia. (mm)	Maximum Weight per Meter (kg)	Maximum Brace Spacing (m) for Specified Seismic 'g' Loads *						Option 1		Option 2		Steel Bolt		Weld Size (mm)
		0.25g		0.5g		1.0g		SCB Size	Cable Dia. (mm)	SSB Size	Minimum Brace Size (mm)	Qty. Req'd	Dia. (mm)	
		Tran.	Long.	Tran.	Long.	Tran.	Long.							
25	4.1	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	3
32	5.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	3
38	6.6	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	3
51	9.2	15.2	24.4	12.2	24.4	12.2	20.7	SCB-1	3	SSB-1	L51x51x6	1	13	3
64	13.5	15.2	24.4	12.2	24.4	12.2	22.9	SCB-1	3	SSB-1	L51x51x6	1	13	3
76	18.0	15.2	24.4	12.2	24.4	12.2	17.1	SCB-1	3	SSB-1	L51x51x6	1	13	3
102	27.3	15.2	24.4	12.2	22.6	11.3	11.3	SCB-1	3	SSB-1	L51x51x6	1	13	3
127	39.5	15.2	24.4	12.2	15.2	7.6	7.6	SCB-1	3	SSB-1	L51x51x6	1	13	3
152	51.8	15.2	24.4	12.2	24.4	12.2	12.5	SCB-2	5	SSB-2	L76x76x6	1	16	3
203	82.0	15.2	24.4	12.2	15.8	7.9	7.9	SCB-2	5	SSB-2	L76x76x6	1	16	3
254	119.3	15.2	21.9	11.0	11.0	5.5	5.5	SCB-2	5	SSB-3	L76x76x6	1	16	5
305	162.2	15.2	24.4	12.2	12.2	6.1	6.1	SCB-3	6	SSB-3	L102x102x6	1	19	5
356	181.5	15.2	21.9	11.0	11.0	5.5	5.5	SCB-3	6	SSB-4	L102x102x6	1	19	5
406	222.6	15.2	17.1	8.5	8.5	4.3	4.3	SCB-3	6	SSB-4	L102x102x6	1	19	5
457	282.7	13.4	13.4	6.7	6.7	3.4	3.4	SCB-3	6	SSB-4	L102x102x6	1	19	5
508	317.7	15.2	24.4	12.2	12.2	6.7	6.7	SCB-4	10	SSB-4	L102x102x6	1	32	5
610	430.1	15.2	19.5	9.8	9.8	4.9	4.9	SCB-4	10	SSB-4	L102x102x6	1	32	5

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.5; for 2:1 brace angle, divide by 2.
Example: For 76 mm dia. pipe at 0.5 g input and 1.5:1 brace angle ratio, the maximum transverse spacing = 12.2/1.5 = 8.1 m.
For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 305 mm dia. pipe, transverse brace spacing = 6.1/0.74 = 8.2 m. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H7 to H12 for the maximum seismic loads.

NOTES:

Anchorage is based on attachment with a Standard ASTM A307 Quality Bolts or E70xx electrode welds.
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 2.7 mm formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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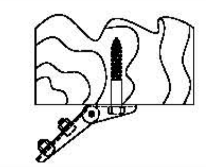
SEISMIC RESTRAINT GUIDELINES	INDIVIDUALLY SUPPORTED SYSTEMS STRUCTURAL WOOD BEAM OR MEMBER LAG BOLT



Pipe Dia. (in.)	Maximum Weight per Foot (lbs.)	Maximum Brace Spacing (ft.) for Specified Seismic 'g' Loads *						Option 1		Option 2		Lag Screws		
		0.25g		0.5g		1.0g		SCB Size	Cable Dia. (in.)	SSB Size	Minimum Brace Size (in.)	Quantity Req'd	Dia. (in.)	Embed. (in.)
		Tran.	Long.	Tran.	Long.	Tran.	Long.							
1	2.8	50	80	40	80	40	80	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
1 1/4	3.8	50	80	40	80	40	65	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
1 1/2	4.5	50	80	40	80	40	55	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
2	6.2	50	80	40	80	40	40	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4
2 1/2	9.1	50	80	40	54	27	27	SCB-1	1/8	SSB-1	L2 x 2 x 1/4	1	1/2	4
3	12.1	50	80	40	40	20	20	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
4	18.3	50	52	26	26	13	13	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
5	26.6	36	36	18	18	9	9	SCB-1	1/8	SSB-1	L2 x 2 x 1/8	1	1/2	4
6	34.8	40	40	20	20	10	10	SCB-2	3/16	SSB-2	L2 x 2 x 1/4	1	5/8	5
8	55.1	24	24	12	12	6	6	SCB-2	3/16	SSB-2	L2 x 2 x 1/4	1	5/8	5
10	80.2	16	16	8	8	4	4	SCB-2	3/16	SSB-3	L2 x 2 x 1/4	1	5/8	5
12	109	12	12	6	6	3	3	SCB-2	3/16	SSB-3	L2 x 2 x 1/4	1	5/8	5

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.5; for 2:1 brace angle, divide by 2.

For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 6" diameter pipe, transverse brace spacing = 10/0.74 = 13 feet. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H13 for the maximum seismic loads.

NOTES:

Anchorage is based on attachment with Lag Screws, 1991 National Design Specification Tables 9.2A & 9.3B.
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 12ga formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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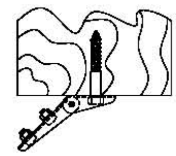
SEISMIC RESTRAINT GUIDELINES

INDIVIDUALLY SUPPORTED SYSTEMS

STRUCTURAL WOOD BEAM OR MEMBER

LAG BOLT



Pipe Dia. (mm)	Maximum Weight per Meter (kg)	Maximum Brace Spacing (m) for Specified Seismic 'g' Loads * 0.25g Tran.	0.25g Long.	0.5g Tran.	0.5g Long.	1.0g Tran.	1.0g Long.	Option 1 SCB Size	Option 1 Cable Dia. (mm)	Option 2 SSB Size	Option 2 Minimum Brace Size (mm)	Lag Screws Quantity Req'd	Lag Screws Dia. (mm)	Lag Screws Embed. (mm)
25	4.1	15.2	24.4	12.2	24.4	12.2	24.4	SCB-1	3	SSB-1	L51x51x3	1	13	102
32	5.6	15.2	24.4	12.2	24.4	12.2	19.8	SCB-1	3	SSB-1	L51x51x3	1	13	102
38	6.6	15.2	24.4	12.2	24.4	12.2	16.8	SCB-1	3	SSB-1	L51x51x3	1	13	102
51	9.2	15.2	24.4	12.2	24.4	12.2	12.2	SCB-1	3	SSB-1	L51x51x6	1	13	102
64	13.5	15.2	24.4	12.2	16.5	8.2	8.2	SCB-1	3	SSB-1	L51x51x6	1	13	102
76	18.0	15.2	24.4	12.2	12.2	6.1	6.1	SCB-1	3	SSB-1	L51x51x3	1	13	102
102	27.3	15.2	18.3	7.9	7.9	4.0	4.0	SCB-1	3	SSB-1	L51x51x3	1	13	102
127	39.5	11.0	11.0	5.5	5.5	2.7	2.7	SCB-1	3	SSB-1	L51x51x3	1	13	102
152	51.8	12.2	12.2	6.1	6.1	3.0	3.0	SCB-2	5	SSB-2	L51x51x6	1	16	127
203	82.0	7.3	7.3	3.7	3.7	1.8	1.8	SCB-2	5	SSB-2	L51x51x6	1	16	127
254	119.3	4.9	4.9	2.4	2.4	1.2	1.2	SCB-2	5	SSB-3	L51x51x6	1	16	127
305	162.2	3.7	3.7	1.8	1.8	0.9	0.9	SCB-2	5	SSB-3	L51x51x6	1	16	127

* Maximum brace spacing for up to 1:1 brace angle from horizontal.
For up to 1.5:1 brace angle from horizontal, divide brace spacing by 1.5; for 2:1 brace angle, divide by 2.

For maximum brace spacing at 'g' forces other than those listed, divide the 1g spacing by the desired 'g' force.
Example: For a 0.74g input for 152 mm dia. pipe, transverse brace spacing = 3.0/0.74 = 4.1m. (Note: Transverse and longitudinal brace spacing shall not exceed those stated in the general notes.)

Special Note: The Structural Engineer of Record shall verify the seismic loads applied by the SCB/SSB to the structure are acceptable. Refer to Pages H13 for the maximum seismic loads.

NOTES:
Anchorage is based on attachment with Lag Screws, 1991 National Design Specification Tables 9.2A & 9.3B.
An increase in SCB or SSB size may be required to accomodate the cross bolt or support rod. (Ref. Page B2)
All SSB brace members tabulated are steel angle. Factory 2.7 mm formed channel strut may be used in lieu of steel angle. (Ref. page X4)

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APPENDIX D

DYNAMIC CHARACTERISTICS AND ACCELERATION DATA

The dynamic characteristics of each system were determined through subjecting the subassemblies to a series of sine sweeps. The acceleration and displacement time histories for the sine sweep can be seen in figures D-1 and D-2. The Fast Fourier Transform (FFT) of each acceleration instrument was taken. The major peaks from all instruments were compared and tabulated to find the natural periods of each subassembly.

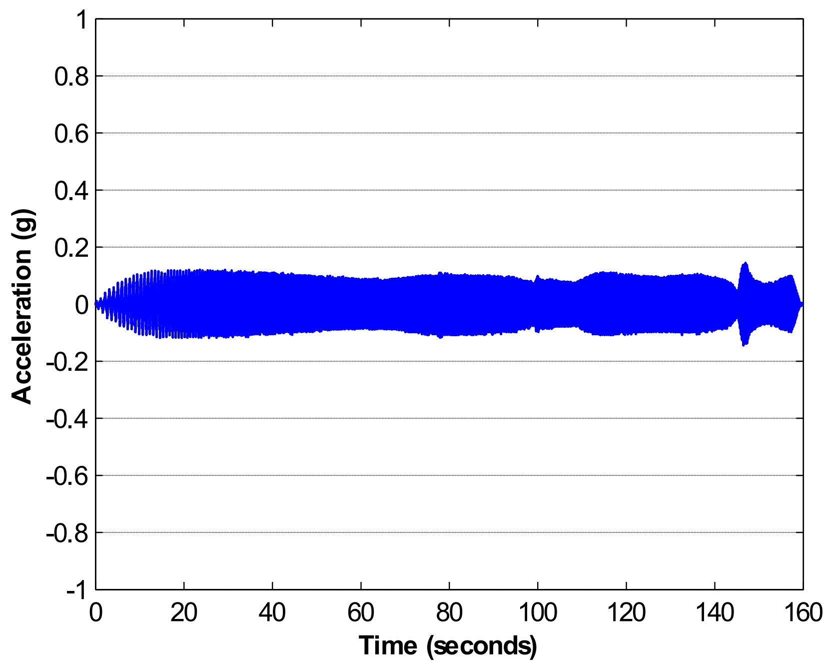

Figure D-1 Sine Sweep Acceleration Time History

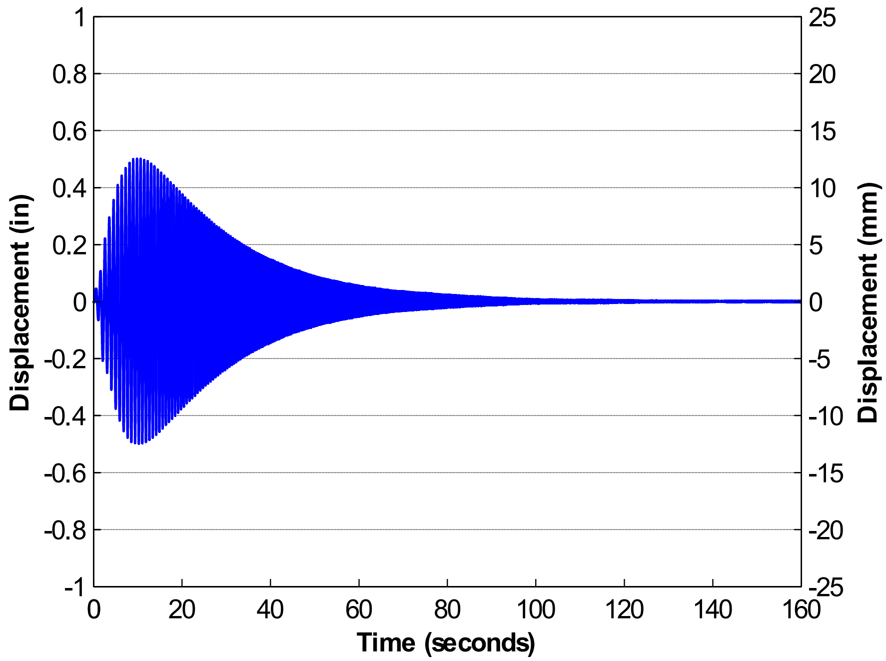

Figure D-2 Sine Sweep Displacement Time History

D.1 Welded Subassembly

The following sections present the experimental results from the braced and unbraced welded subassembly experiments. The welded braced subassembly was subjected to runs 1-68 from the experimental protocol (tables 2-3 and 2-4). The unbraced subassembly was subjected to runs 69-121 (tables 2-5 and 2-6) from the experimental protocol. Only the acceleration and dynamic characteristics are discussed in this Appendix.

D.1.1 Welded Braced Subassembly

D.1.1.1 Dynamic Characteristics

Table D-1 shows the first two periods of the welded braced subassembly. The first two periods of the subassembly were 0.25 and 0.35 seconds. The FFT's of instruments nv3, nv26, nv9 and nv18 are presented in figures D-3 - D-6. These instruments are typical of all instrument responses.

	Mode (seconds)		
	1st	2nd	3rd
Welded Braced	0.25	0.35	NA
Welded Unbraced	0.17	0.43	0.59
Threaded Braced	0.23	0.33	NA
Threaded Unbraced	0.17	0.62	0.98

Table D-1 Description of Natural Periods

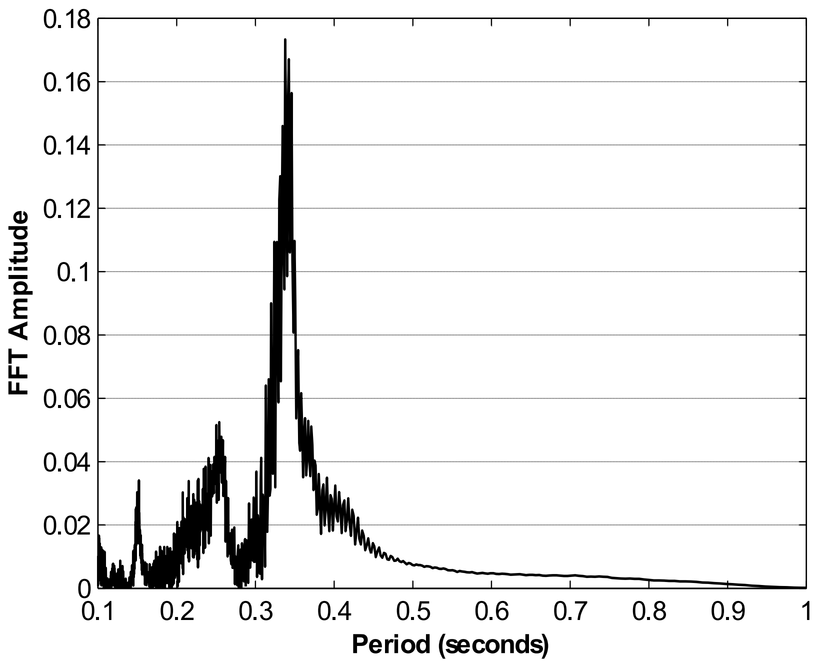

Figure D-3 Welded Braced FFT of nv3

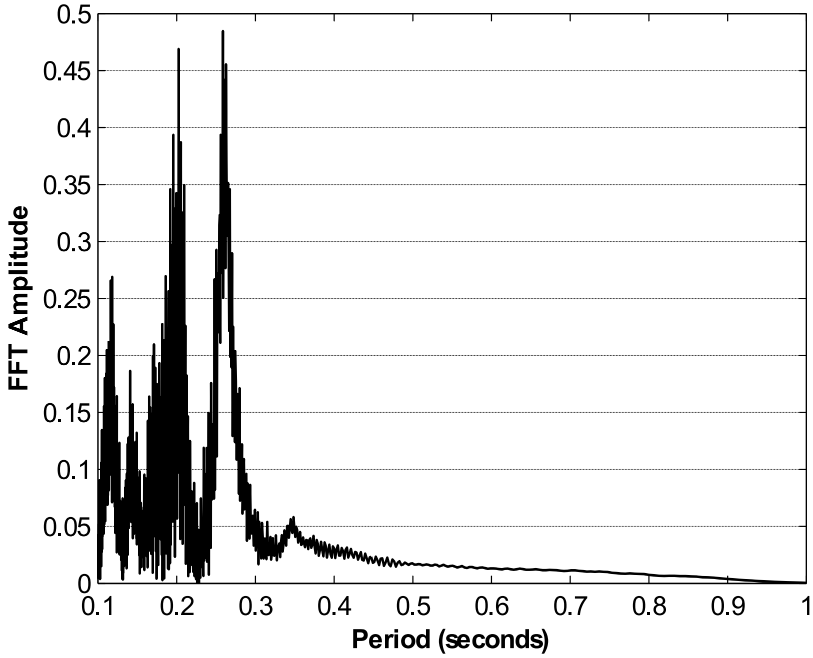

Figure D-4 Welded Braced FFT of nv26

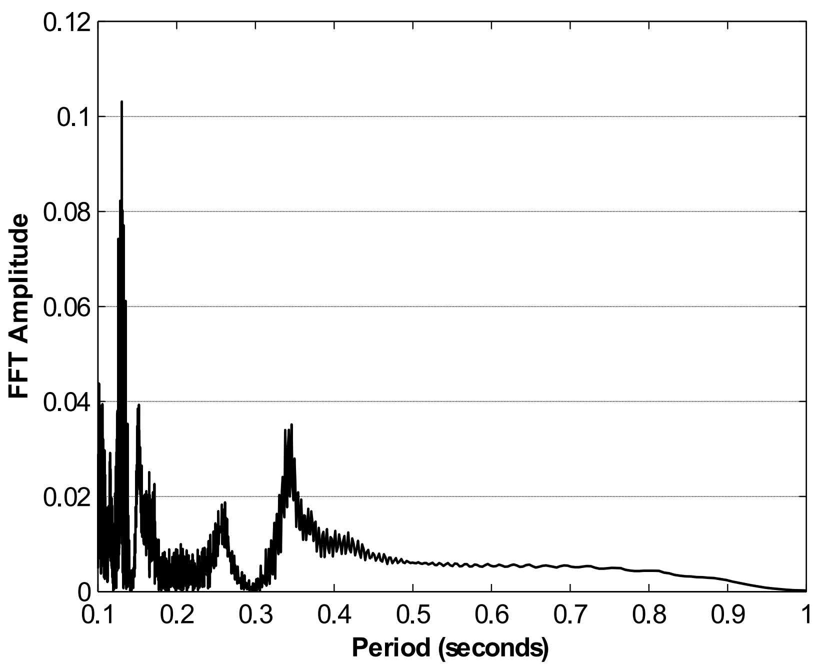

Figure D-5 Welded Braced FFT of nv9

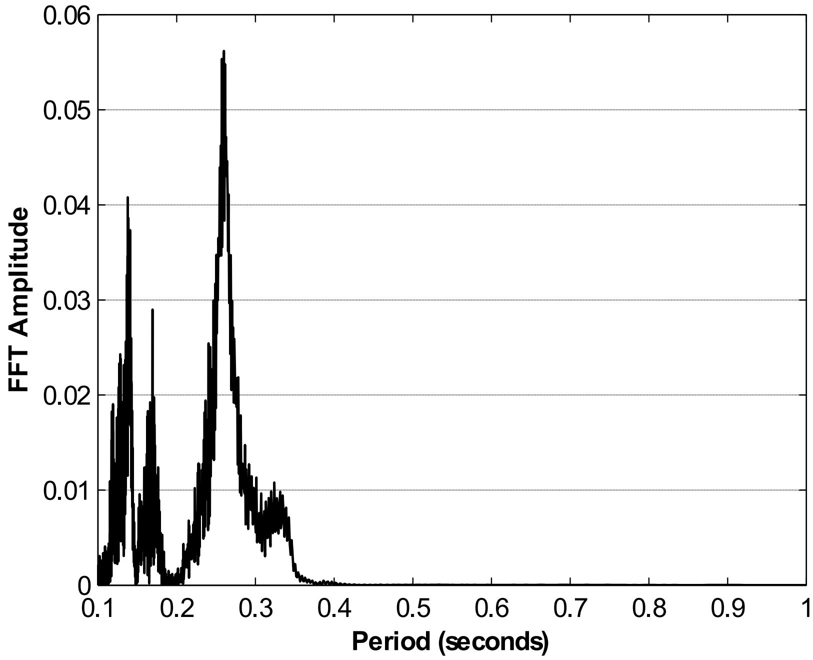

Figure D-6 Welded Braced FFT of nv18

D.1.1.2 Acceleration Response

For run 61, the maximum parallel acceleration of an instrument was 2.65g. This occurred at instrument nv26. The maximum transverse acceleration for run 61 was 2.32g, which occurred at nv4. Instrument nv49 had the highest vertical acceleration at 2.06g. The time histories for these instruments can be seen in figures D-7 - D-9

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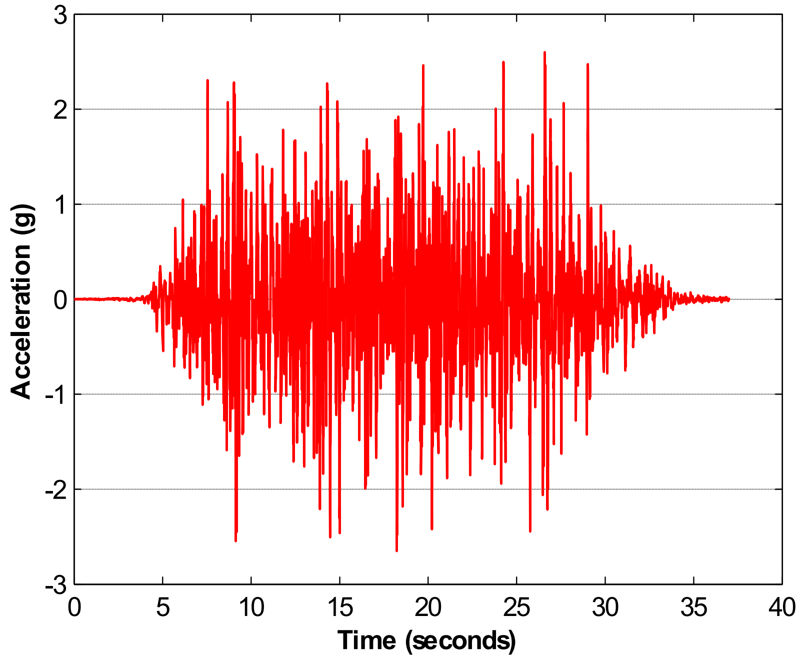

Figure D-7 Run 61 - Acceleration Time History of nv26

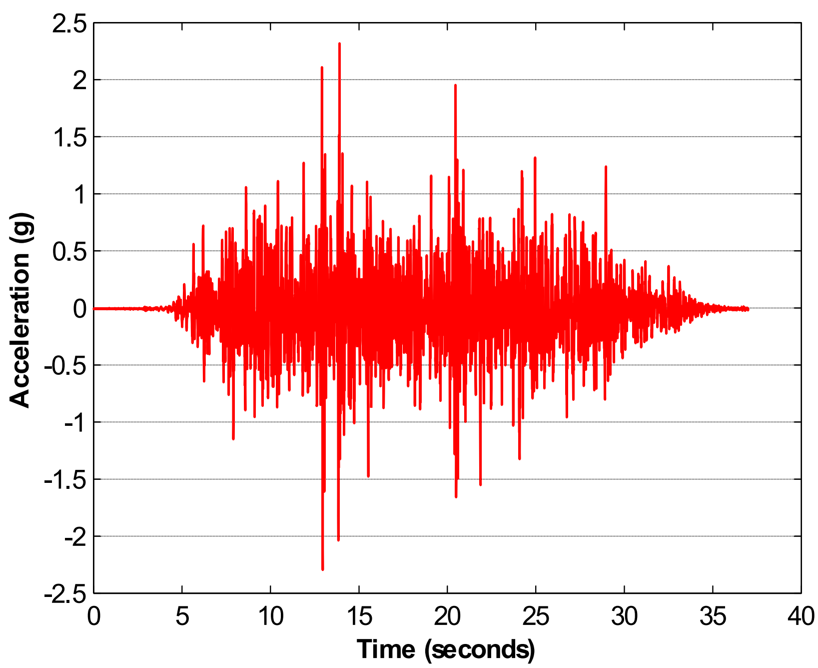

Figure D-8 Run 61 - Acceleration Time History of nv4

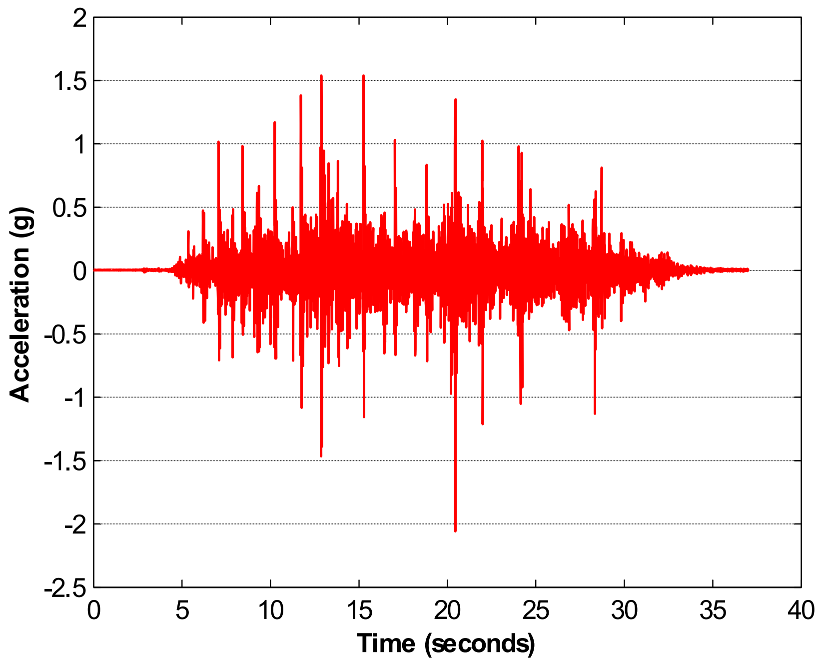

Figure D-9 Run 61 - Acceleration Time History of nv49

For run 62, the maximum parallel acceleration was 2.55g, measured by nv4. The maximum transverse acceleration was 1.77g measured by nv38. Instrument nv49 recorded a vertical acceleration of 1.50g. The time histories for these instruments can be seen in figures D-10 - D-12.

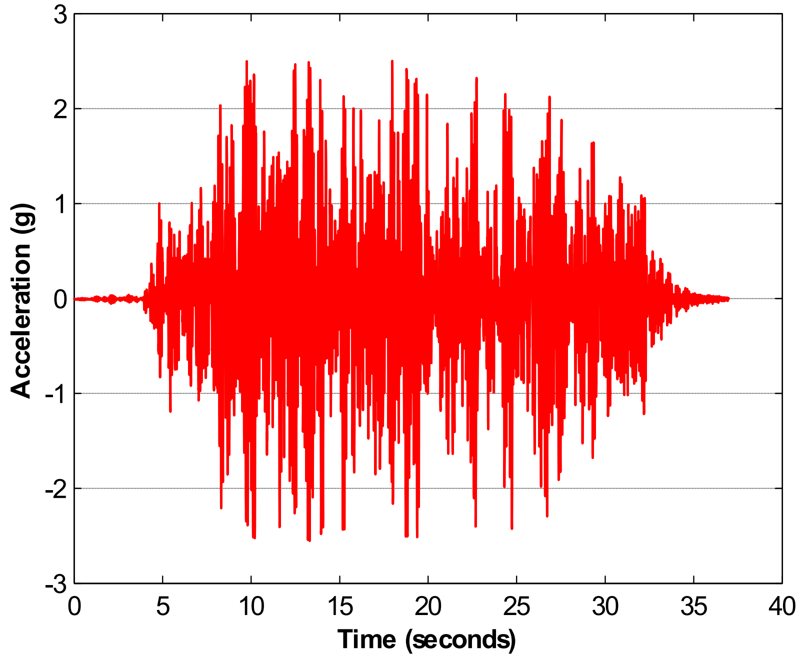

Figure D-10 Run 62 - Acceleration Time History of nv4

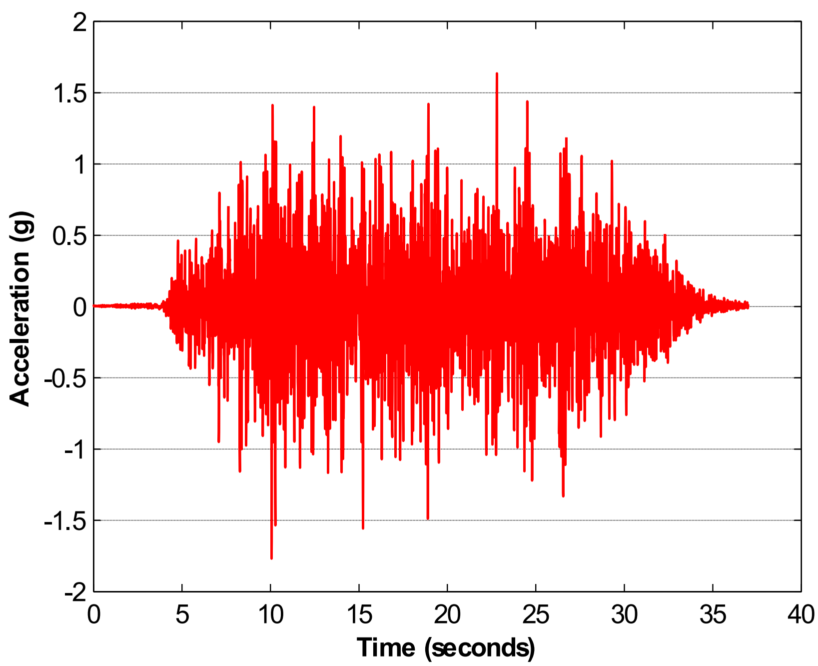

Figure D-11 Run 62 - Acceleration Time History of nv38

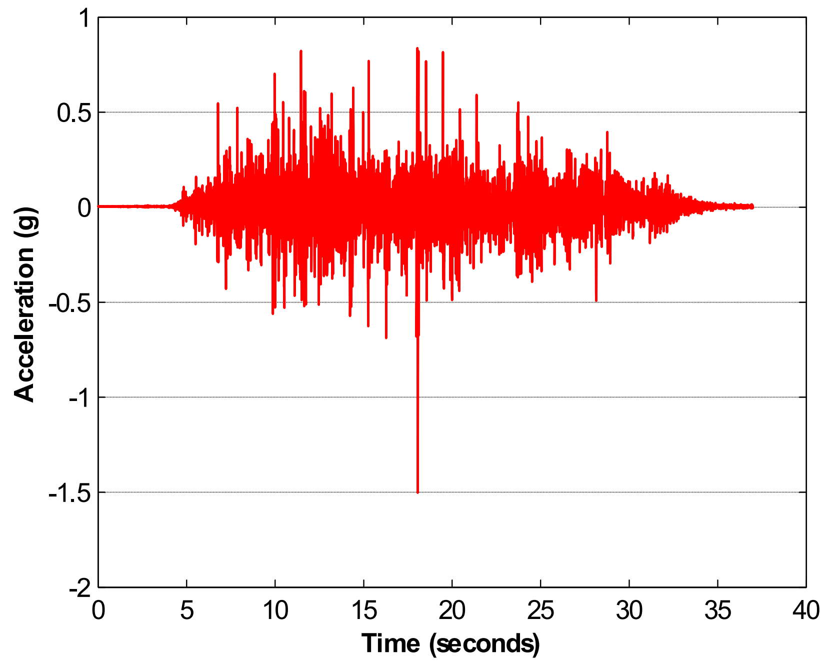

Figure D-12 Run 62 - Acceleration Time History of nv49

Table D-2 shows the maximum acceleration for the selected instruments during runs 61-63.

	Maximum Acceleration (g)		
Instrument Number	**Run 61**	**Run 62**	**Run 63**
nv26	2.65	1.73	2.36
nv8	2.55	0.94	1.54
nv5	2.31	1.29	1.05
nv13	1.67	1.20	1.31
nv4	2.32	2.55	2.51
nv18	1.62	1.55	1.59

Table D-2 Maximum Accelerations of Selected Instruments for Runs 61-63

D.1.2 Welded Unbraced Subassembly

D.1.2.1 Dynamic Characteristics

Table D-1 shows the first three periods of the welded unbraced subassembly. The first three periods of the subassembly were 0.17, 0.43 and 0.59 seconds. The FFT's of instruments nv3, nv26, nv9 and nv18 are presented in figures D-13 - D-16. These instruments are typical of all instrument responses.

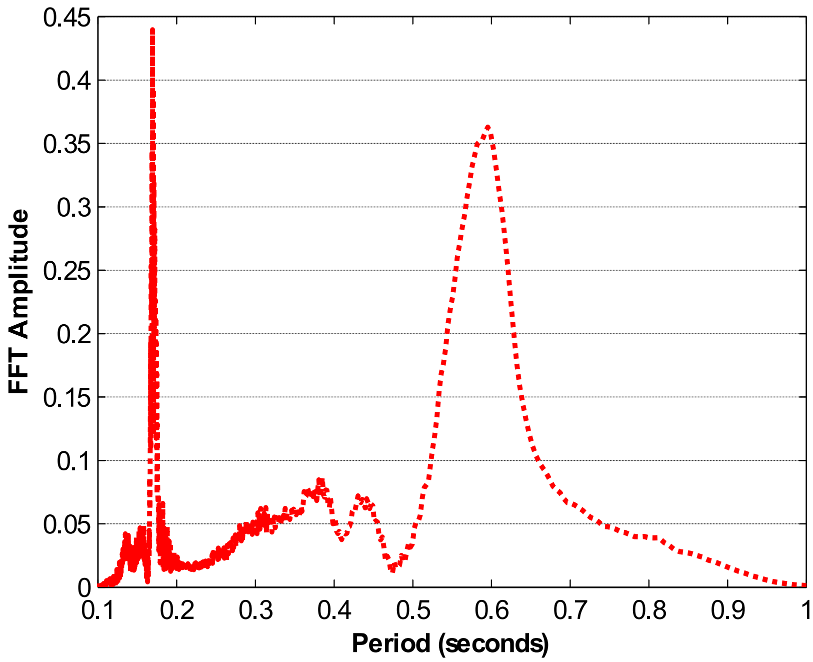

Figure D-13 Welded Unbraced FFT of nv3

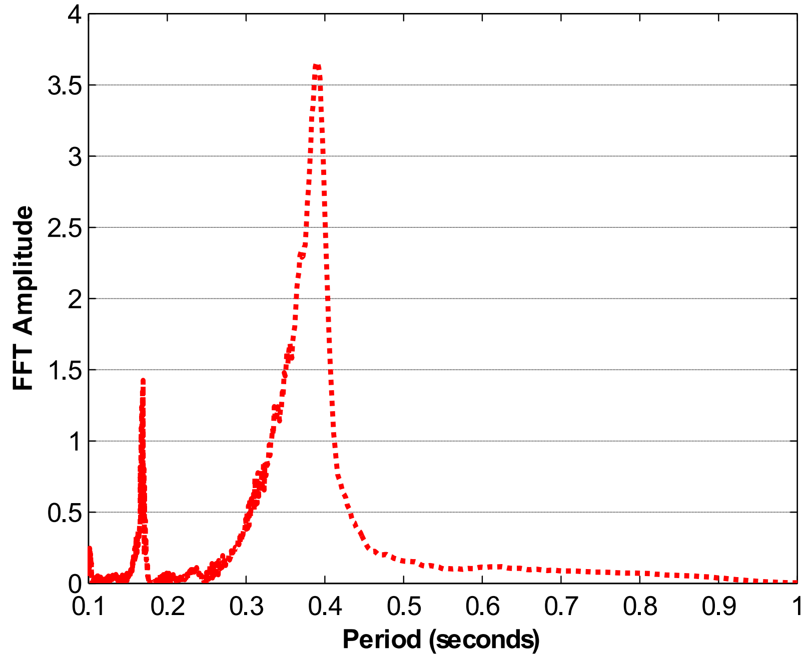

Figure D-14 Welded Unbraced FFT of nv26

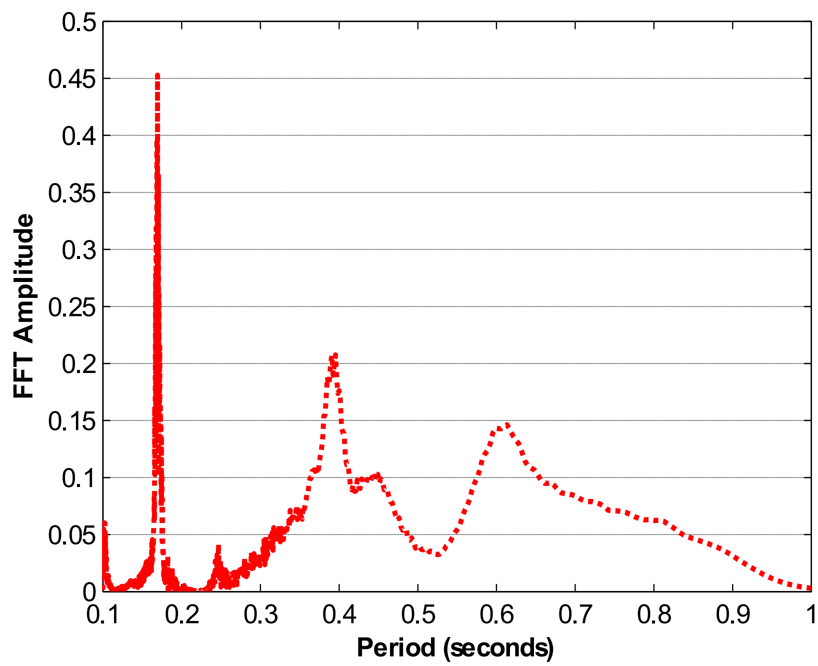

Figure D-15 Welded Unbraced FFT of nv9

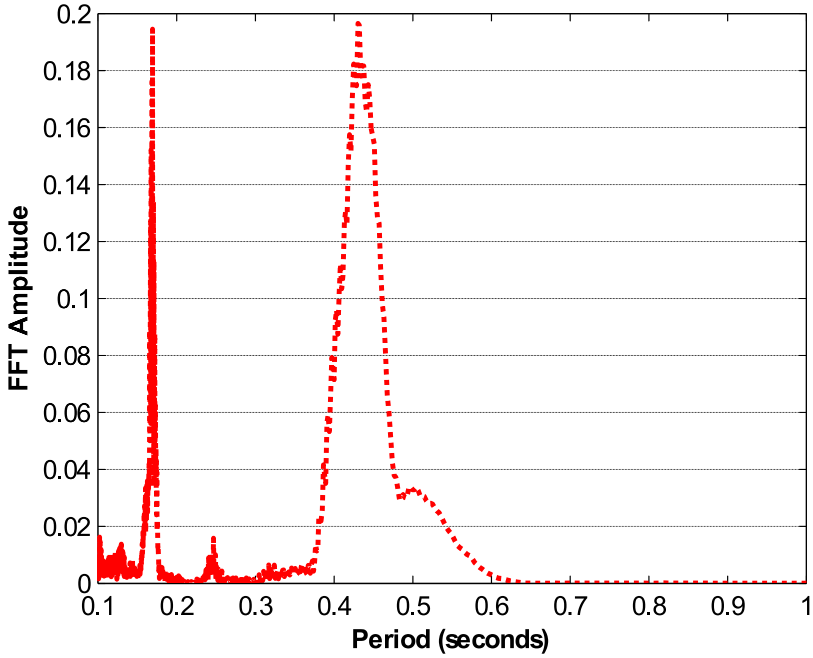

Figure D-16 Welded Unbraced FFT of nv18

D.1.2.2 Acceleration Response

For run 113, the maximum parallel acceleration of an instrument was 2.64g. This occurred at instrument nv26. The maximum transverse acceleration for run 113 was 1.69g, which occurred at nv31. Instrument nv13 had the highest vertical acceleration at 2.27g. The time histories for these instruments can be seen in figures D-17 - D-19.

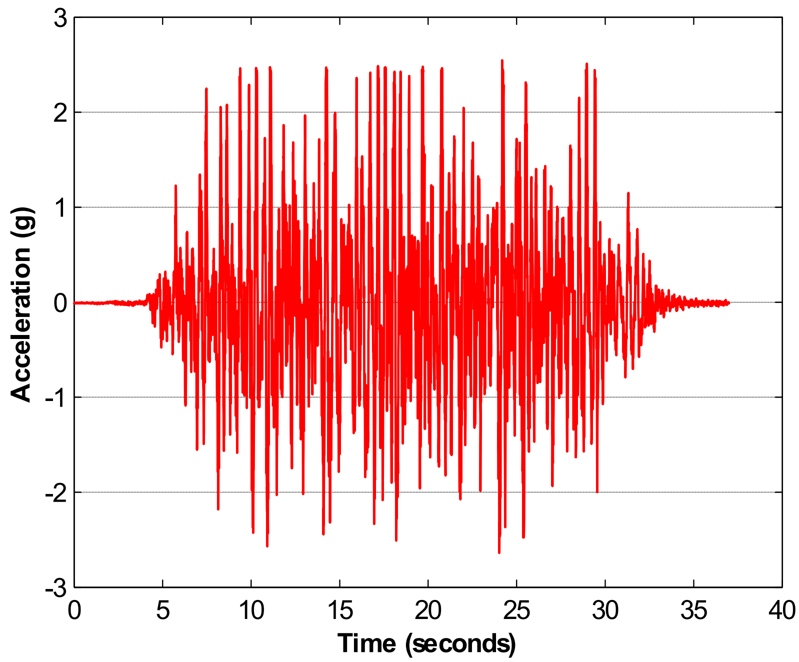

Figure D-17 Run 113 - Acceleration Time History of nv26

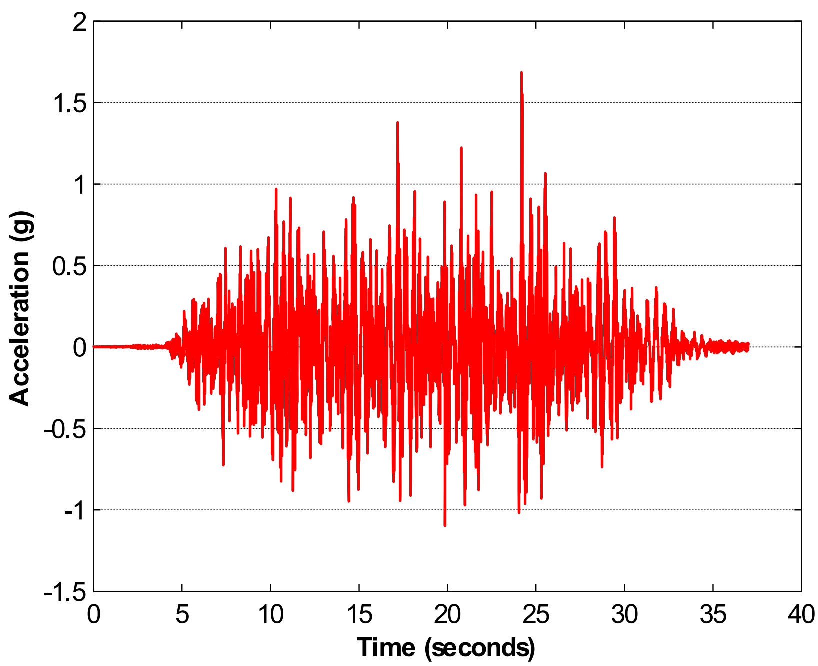

Figure D-18 Run 113 - Acceleration Time History of nv31

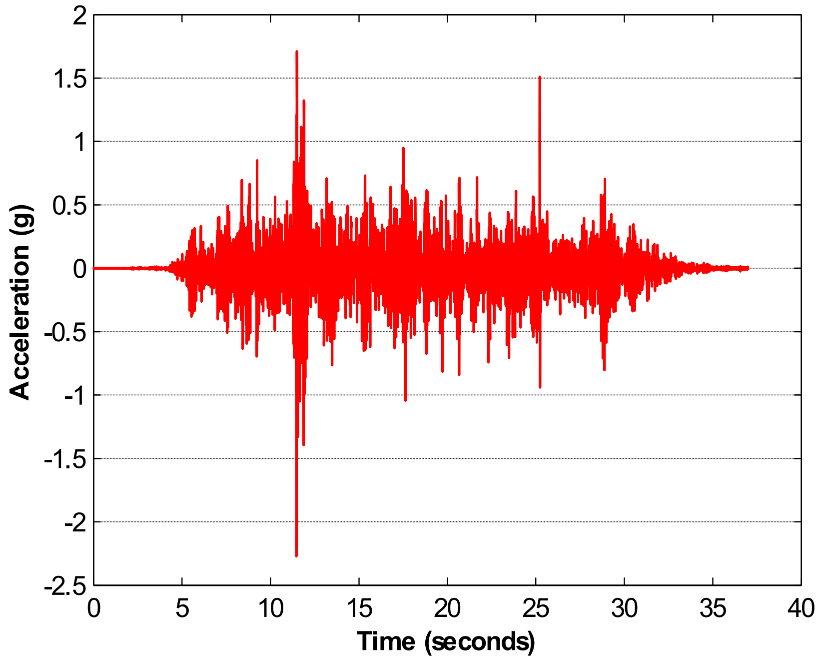

Figure D-19 Run 113 - Acceleration Time History of nv13

For run 114, the maximum parallel acceleration was 2.56g, measured by nv4. The maximum transverse acceleration was 2.47g measured by nv38. Instrument nv13 recorded a vertical acceleration of 1.79g. The time histories for these instruments can be seen in figures D-20 - D-22. Table D-3 shows the maximum acceleration for the selected instruments during runs 113-114.

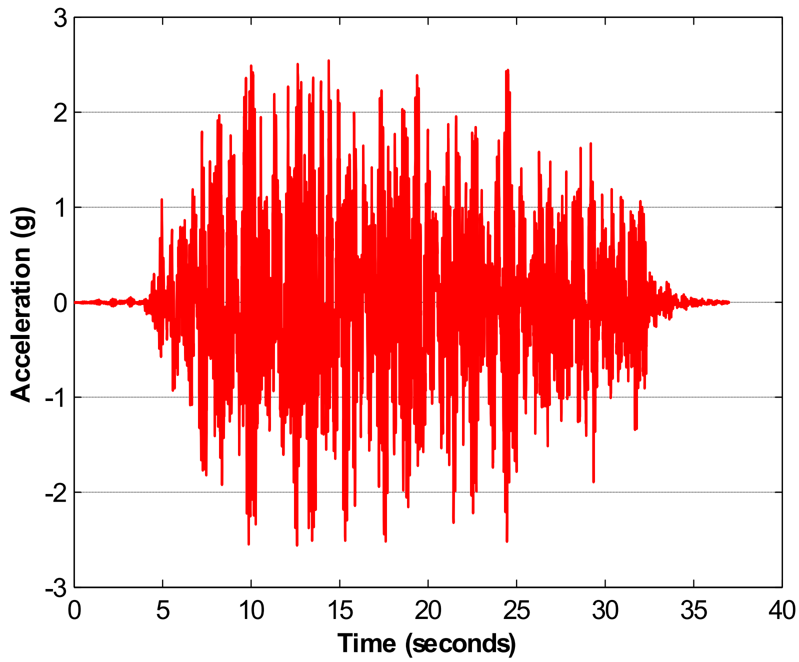

Figure D-20 Run 114 - Acceleration Time History of nv4

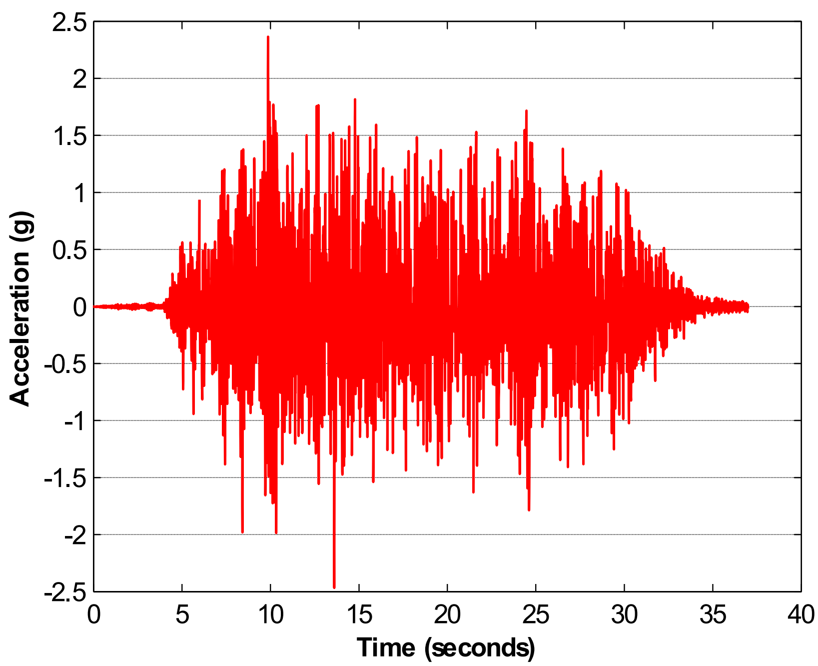

Figure D-21 Run 114 - Acceleration Time History of nv38

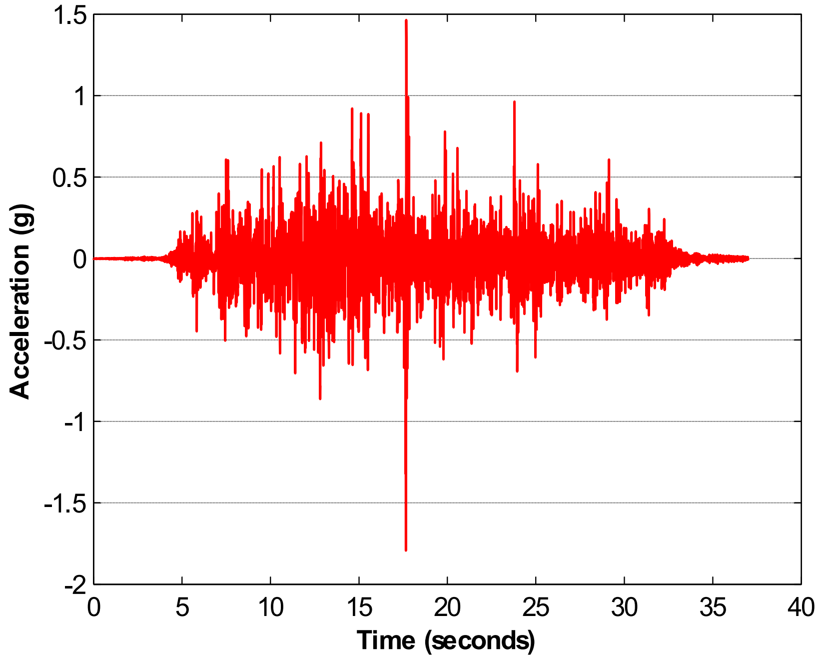

Figure D-22 Run 114 - Acceleration Time History of nv13

	Maximum Acceleration (g)		
Instrument Number	**Run 113**	**Run 114**	**Run 115**
nv26	2.64	2.38	2.53
nv8	2.19	1.69	1.62
nv5	2.21	1.66	2.57
nv13	2.27	1.79	2.13
nv4	1.38	2.56	2.39
nv18	1.48	2.09	1.68

Table D-3 Maximum Accelerations of Selected Instruments for Runs 113-115

D.1.3 Comparison of Welded Braced and Unbraced Subassemblies

This section will present a comparison between the responses of the welded braced and unbraced subassemblies. Existing plots are discussed and new plots are presented in order to make the comparison.

D.1.3.1 Dynamic Characteristic Comparison

Figures D-23 - D-26 show a comparison of the FFT's of nv3, nv26, nv9 and nv18. As seen, the braced subassembly had shorter natural periods than the unbraced. Table D-1 shows the natural periods of both subassemblies. This shorter period response translates to a stiffer system. The unbraced subassembly had a significant response when the table input motion period was above 0.5 seconds. This is seen in figures D-23 - D-26. The braced subassembly, however, had little to no response above 0.5 seconds.

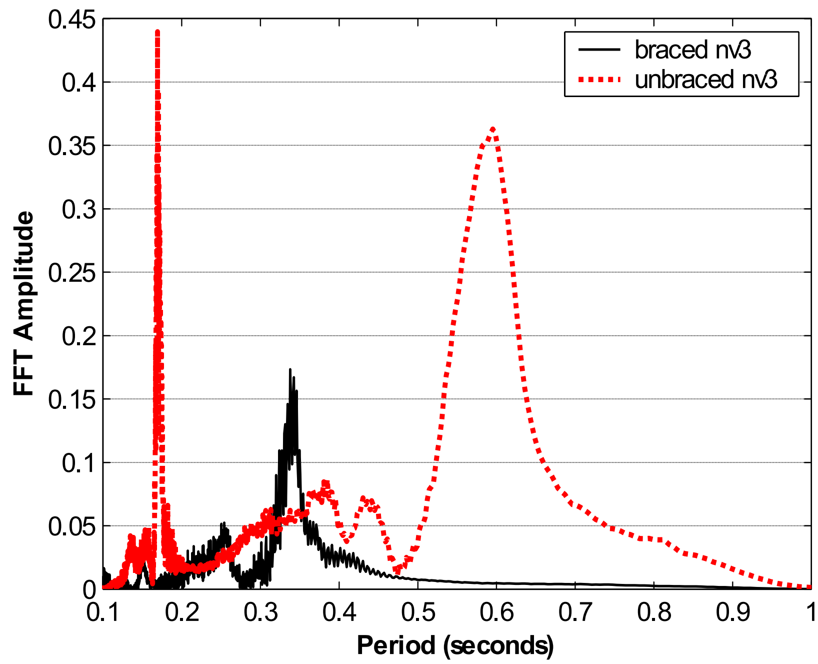

Figure D-23 Welded Braced and Unbraced FFT of nv3

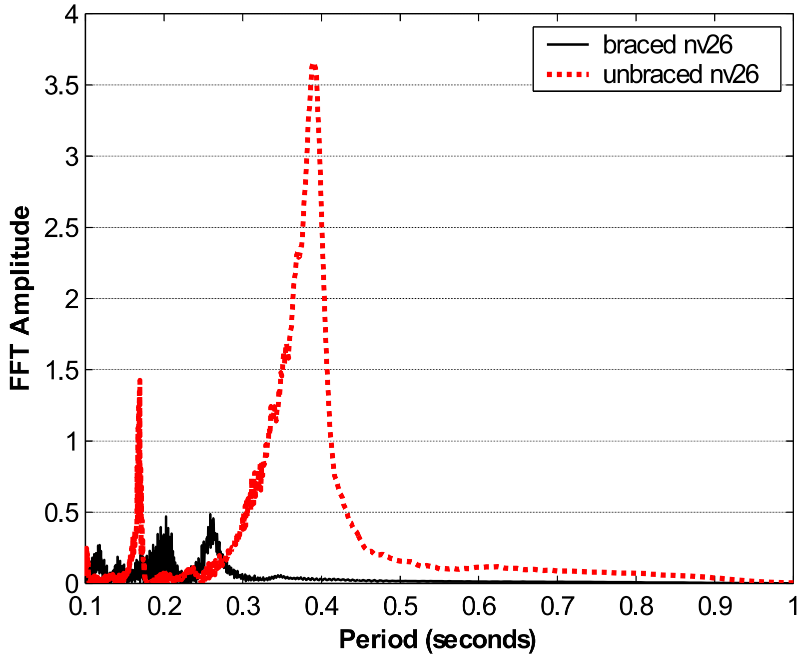

Figure D-24 Welded Braced and Unbraced FFT of nv26

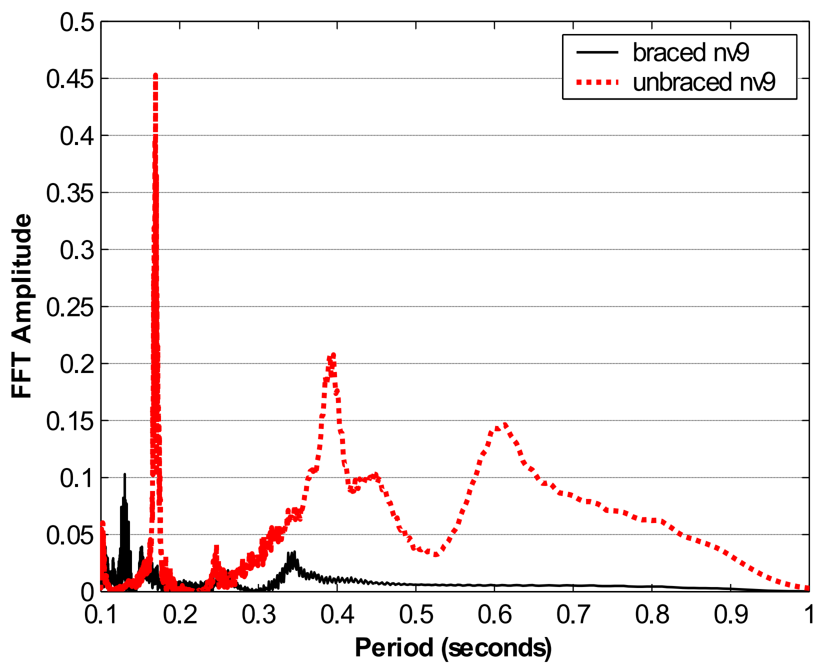

Figure D-25 Welded Braced and Unbraced FFT of nv9

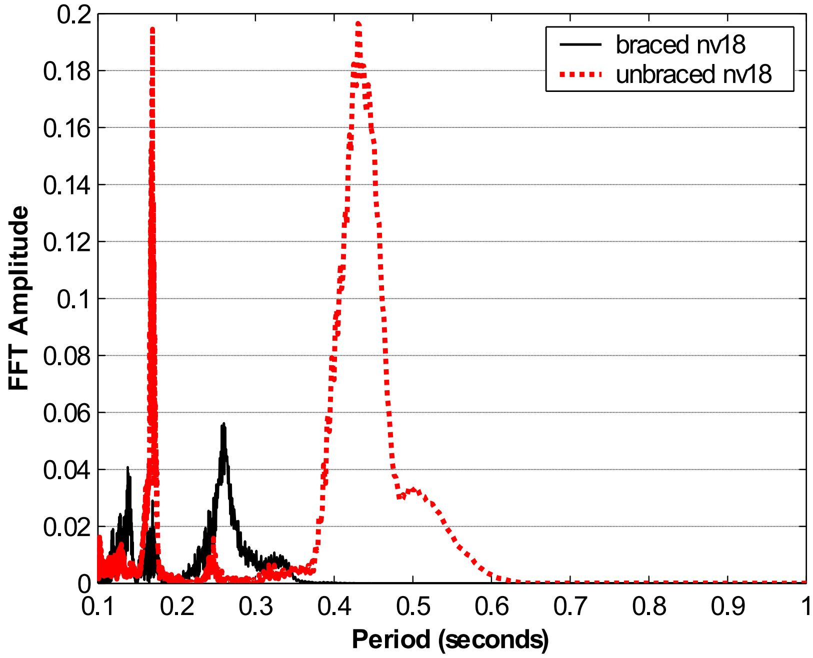

Figure D-26 Welded Braced and Unbraced FFT of nv18

D.1.3.2 Acceleration Comparison

Although the braces limited the displacement of the system, the braces did not restrict the acceleration response. In some cases, the braced acceleration was higher than the unbraced. For runs 61 and 113, the parallel acceleration response for the unbraced case was anywhere from 0.74 times to 1.12 times the braced response. The transverse unbraced response also varied between 0.9 time to 1.98 times the transverse braced response.

For the North/South (runs 62 and 114) excitation, the unbraced parallel and transverse accelerations were always larger than the braced parallel and transverse accelerations. This is in contrast to the East/West excitations in which the maximum acceleration response varied between the braced and the unbraced cases.

The vertical unbraced acceleration response for both excitation cases varied between 0.73 times and 1.49 times the braced cases.

D.2 Threaded Subassembly

The following sections present the experimental results from the braced and unbraced threaded subassembly experiments. The threaded braced subassembly was subjected to runs 122-180 (tables 2-7 and 2-8) from the experimental protocol. The unbraced subassembly was subjected to runs 181-230 (tables 2-9 and 2-10) from the experimental protocol. Displacements, accelerations and observations are discussed for each subassembly. A comparison of the two subassemblies is made as well.

D.2.1 Threaded Braced Subassembly

D.2.1.1 Dynamic Characteristics

Table D-1 shows the first two periods of the threaded braced subassembly. The first two periods of the subassembly were 0.23 and 0.33 seconds. The FFT's of instruments nv3, nv26, nv9 and nv18 are presented in figures D-27 - D-30. These instruments are typical of all instrument responses.

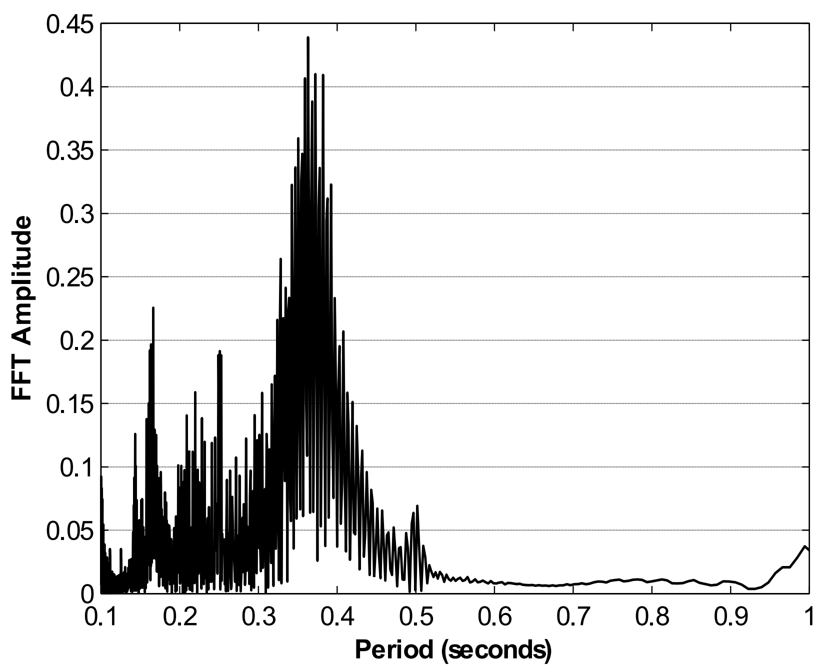

Figure D-27 Threaded Braced FFT of nv3

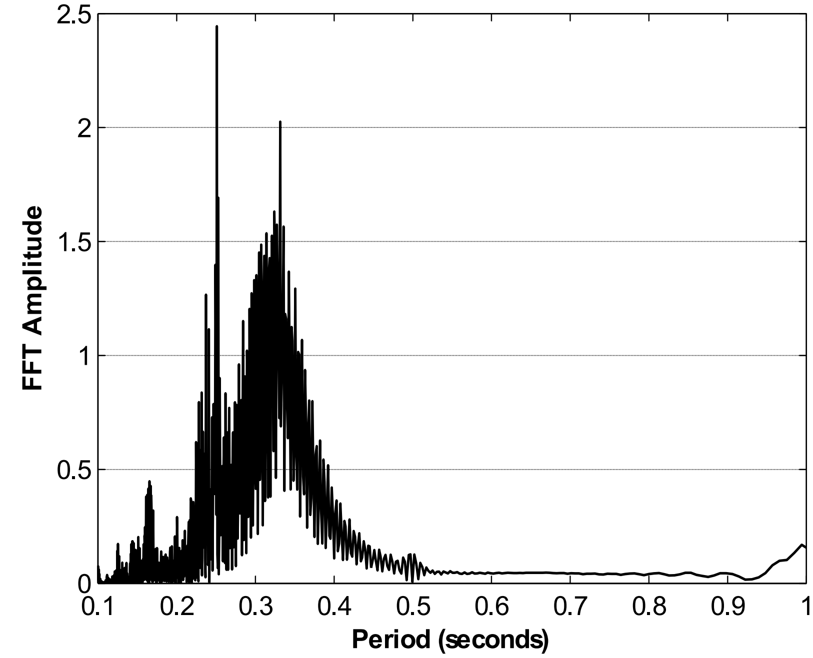

Figure D-28 Threaded Braced FFT of nv26

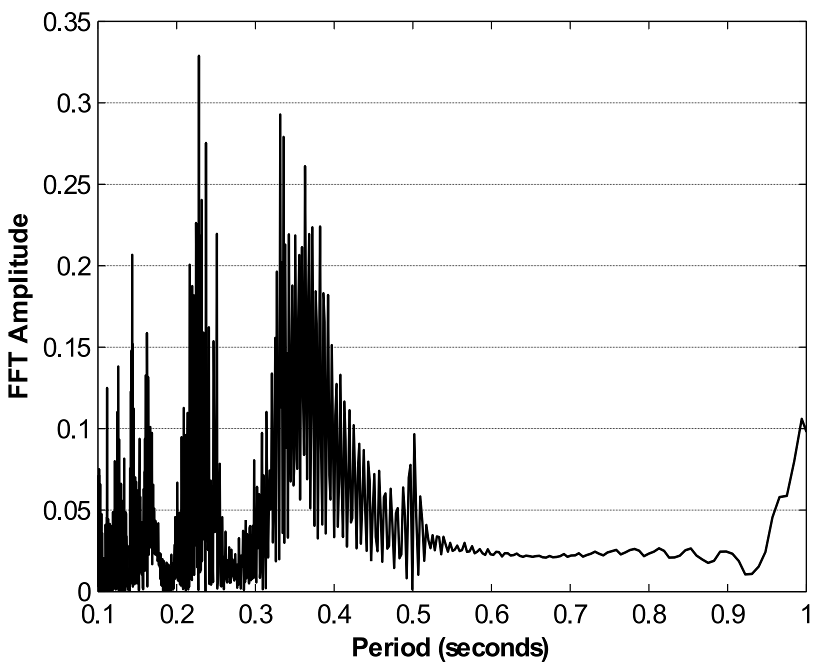

Figure D-29 Threaded Braced FFT of nv9

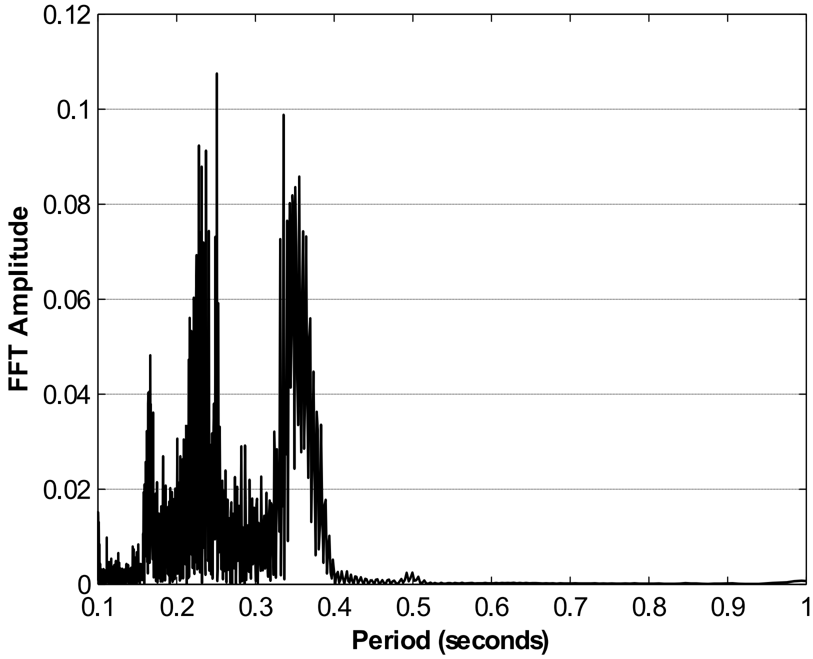

Figure D-30 Threaded Braced FFT of nv18

D.2.1.2 Acceleration Response

For run 180, the maximum parallel acceleration of an instrument was 2.58g. This occurred at instrument nv26. The maximum transverse parallel acceleration for run 180 was 2.23g, which occurred at nv4. Instrument nv49 had the highest vertical acceleration at 1.53g. The time histories for these instruments can be seen in Figures D-31 - D-33.

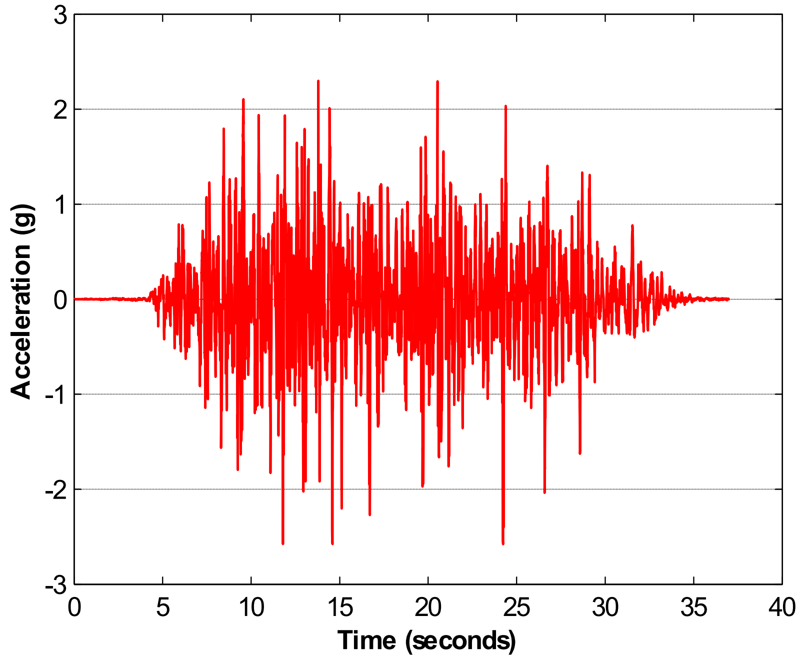

Figure D-31 Run 180 - Acceleration Time History of nv26

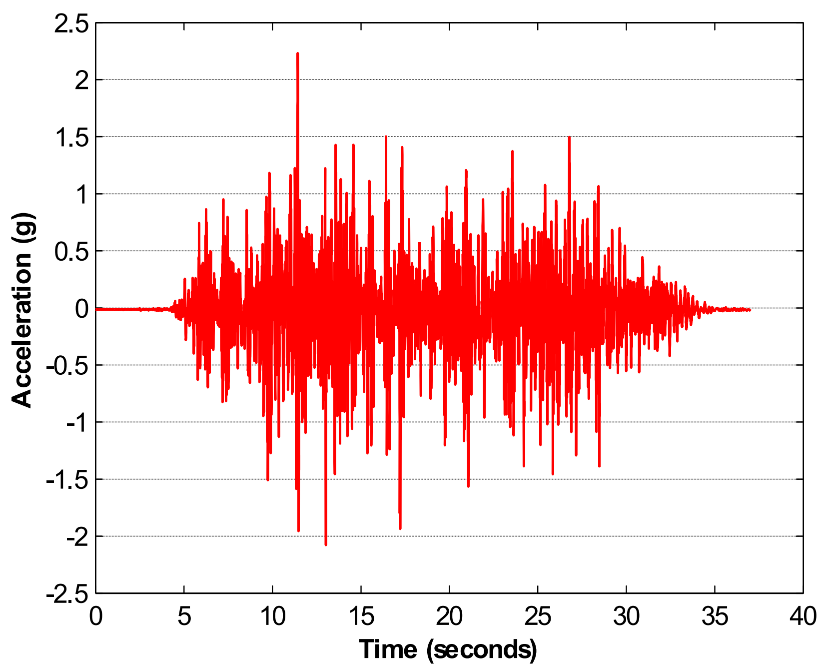

Figure D-32 Run 180 - Acceleration Time History of nv4

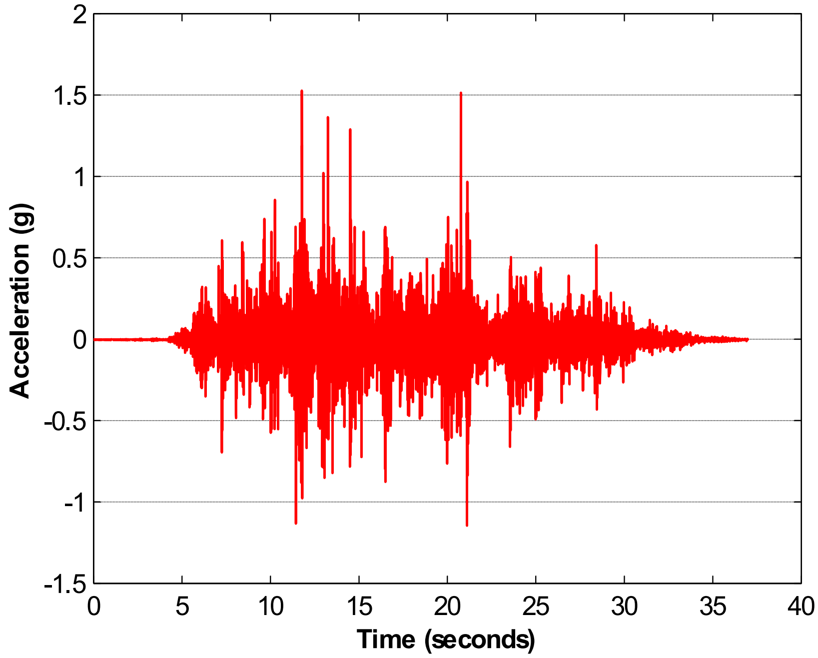

Figure D-33 Run 180 - Acceleration Time History of nv49

Table D-4 shows the maximum acceleration for the selected instruments during runs 179-180.

	Maximum Acceleration (g)			
Instrument Number	**Run 180**	**Run 177**	**Run178**	**Run179**
nv26	2.58	2.59	1.52	2.22
nv8	2.52	2.53	1.20	2.34
nv5	2.01	1.28	0.62	1.03
nv13	1.50	1.36	0.43	1.47
nv4	2.23	2.13	2.26	2.22
nv18	1.18	0.78	1.03	0.98

Table D-4 Maximum Accelerations of Selected Instruments for Runs 177-180

D.2.2 Threaded Unbraced Subassembly

D.2.2.1 Dynamic Characteristics

Table D-1 shows the first three periods of the threaded braced subassembly. The first three periods of the subassembly were 0.17, 0.62 and 0.98 seconds. The FFT's of instruments nv3, nv26, nv9 and nv18 are presented in figures D-34 - D-37. These instruments are typical of all instrument responses.

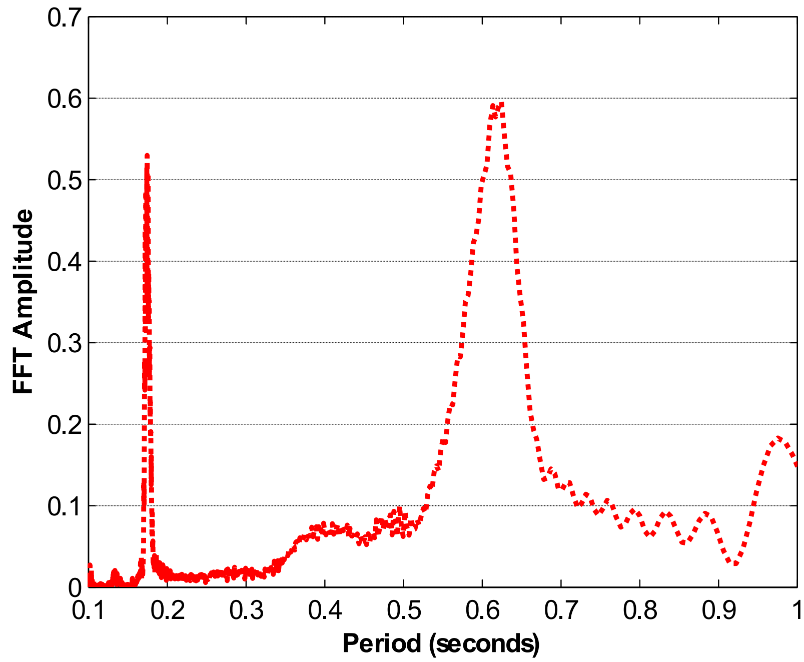

Figure D-34 Threaded Unbraced FFT of nv3

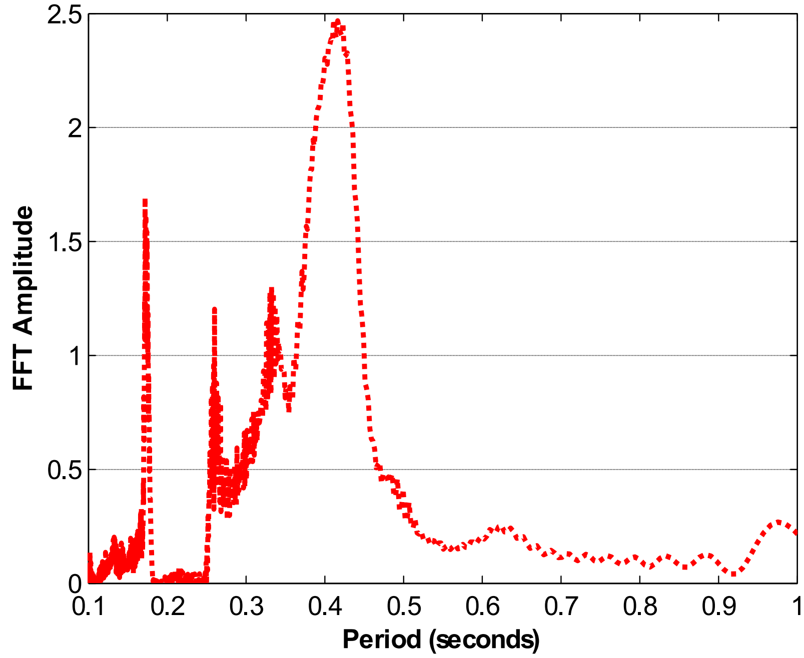

Figure D-35 Threaded Unbraced FFT of nv26

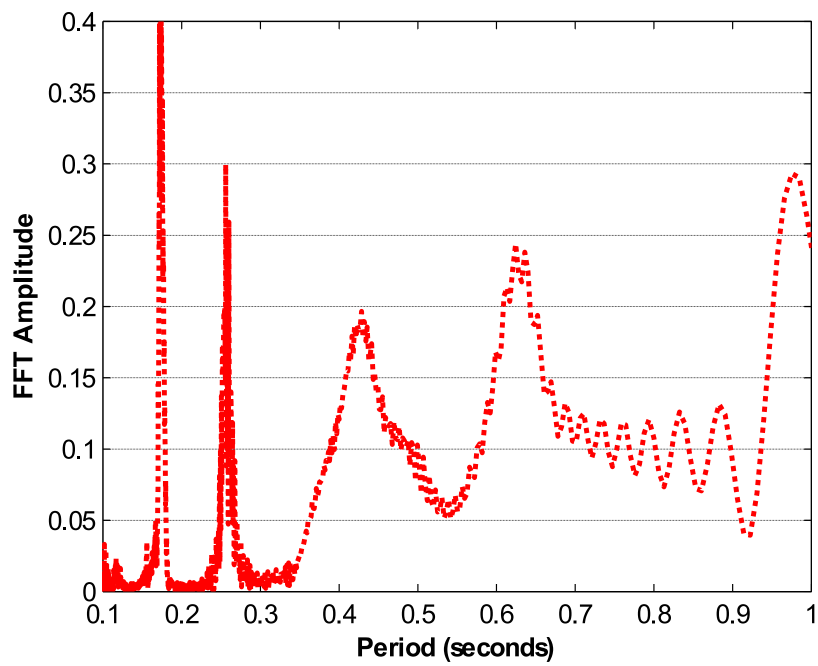

Figure D-36 Threaded Unbraced FFT of nv9

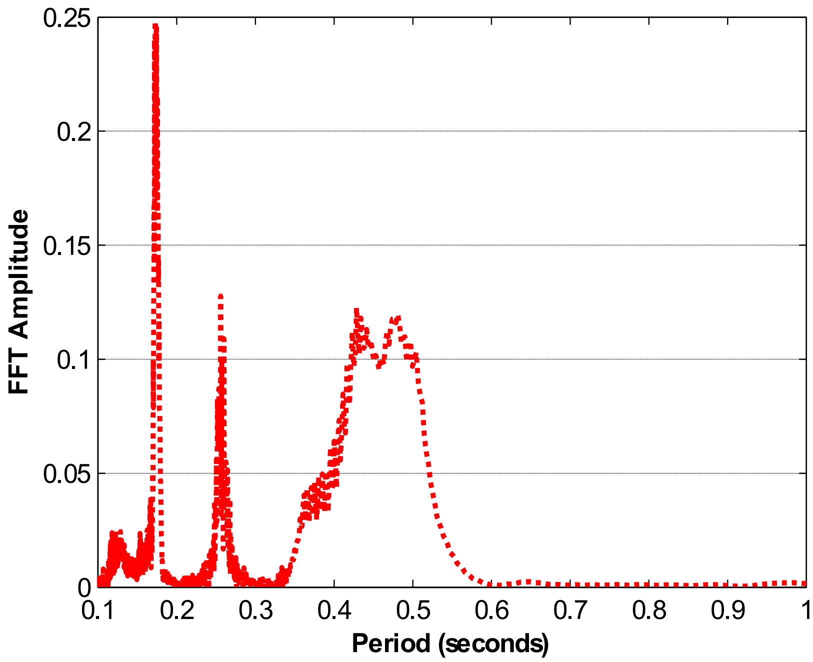

Figure D-37 Threaded Unbraced FFT of nv18

D.2.2.2 Acceleration Response

For run 225, the maximum parallel acceleration of an instrument was 2.60g. This occurred at instrument nv26. The maximum transverse acceleration for run 225 was 2.00g, which occurred at nv4. A malfunction in the accelerometers that measured vertical acceleration prevented accurate results.

For run 226, the maximum parallel acceleration was 2.52g, measured by nv4. The maximum transverse acceleration was 2.51g measured by nv26.

Table D-5 shows the maximum acceleration for the selected instruments during runs 225-227.

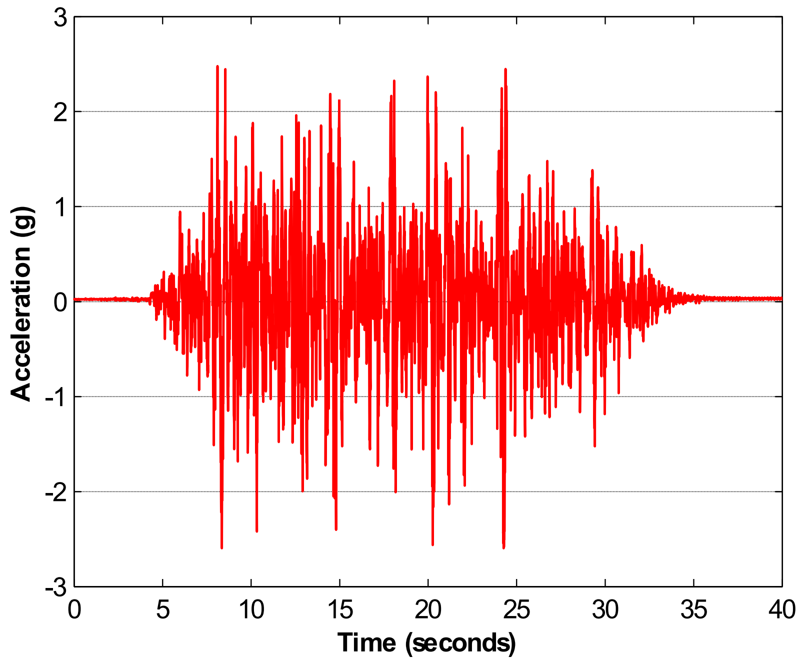

Figure D-38 Run 225 - Acceleration Time History of nv26

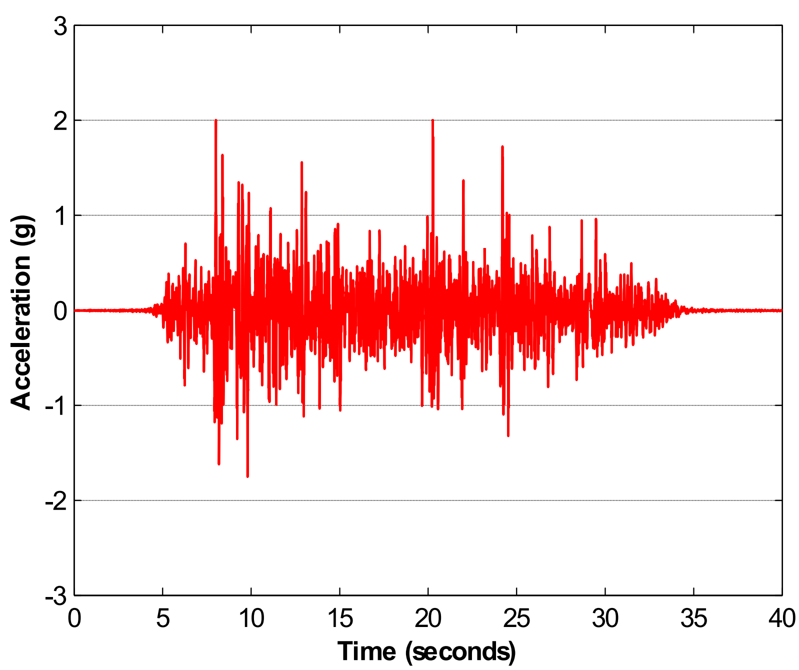

Figure D-39 Run 225 - Acceleration Time History of nv4

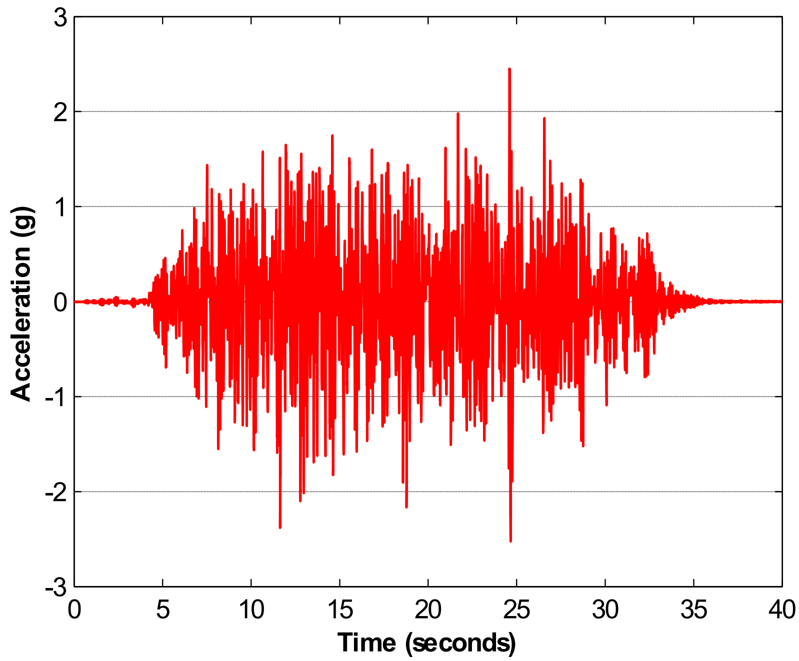

Figure D-40 Run 226 - Acceleration Time History of nv4

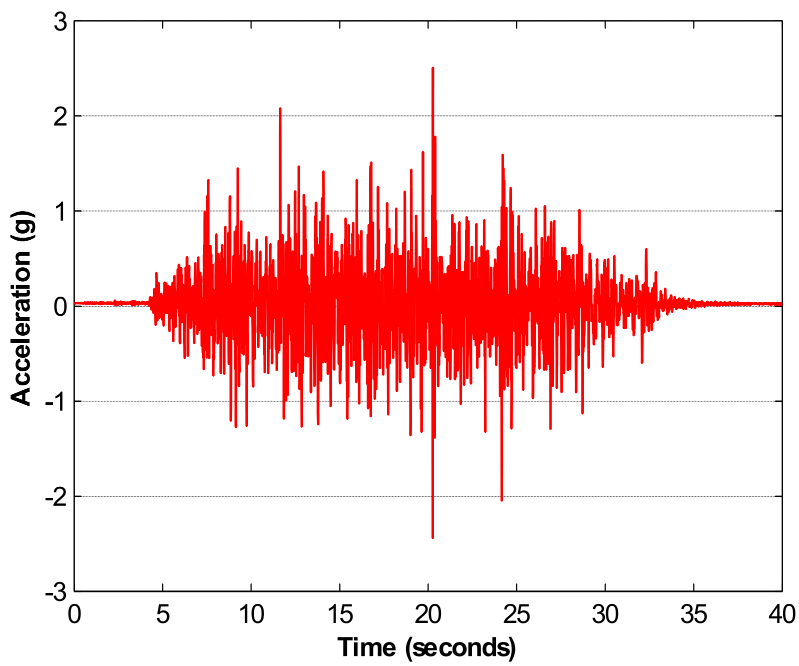

Figure D-41 Run 226 - Acceleration Time History of nv26

	Maximum Acceleration (g)		
Instrument Number	Run 225	Run 226	Run 227
nv26	2.60	2.51	2.31
nv8	1.97	1.15	1.53
nv5	1.53	1.47	0.27
nv13	3.56	2.56	2.05
nv4	2.00	2.53	2.26
nv18	1.14	1.29	1.23

Table D-5 Maximum Accelerations of Selected Instruments for Runs 225-227

D.2.3 Comparison of Threaded Braced and Unbraced Subassemblies

The following section will present a comparison between the threaded braced and unbraced subassemblies. Existing plots are discussed and new plots are presented in order to make the comparison.

D.2.3.1 Dynamic Characteristic Comparison

Figures D-42 - D-45 show a comparison of the FFT's of nv3, nv26, nv9 and nv18. Table D-1 shows the natural periods of both subassemblies. As seen, the braced subassembly had shorter natural periods than the unbraced. This shorter period response translates to a stiffer system. The unbraced subassembly had significant response when the table input motion period was above 0.5 seconds. The braced subassembly, however, had little to no response above 0.5 seconds.

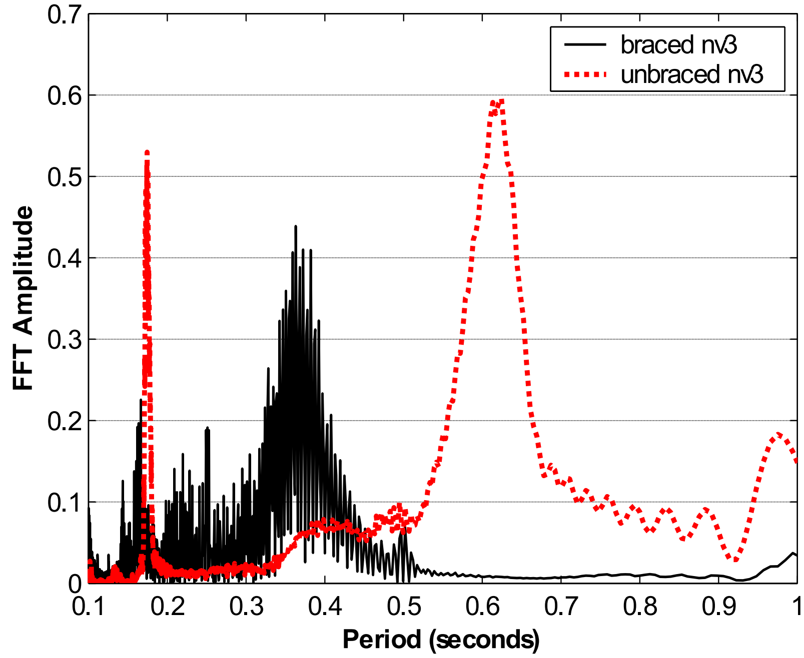

Figure D-42 Threaded Braced and Unbraced FFT of nv3

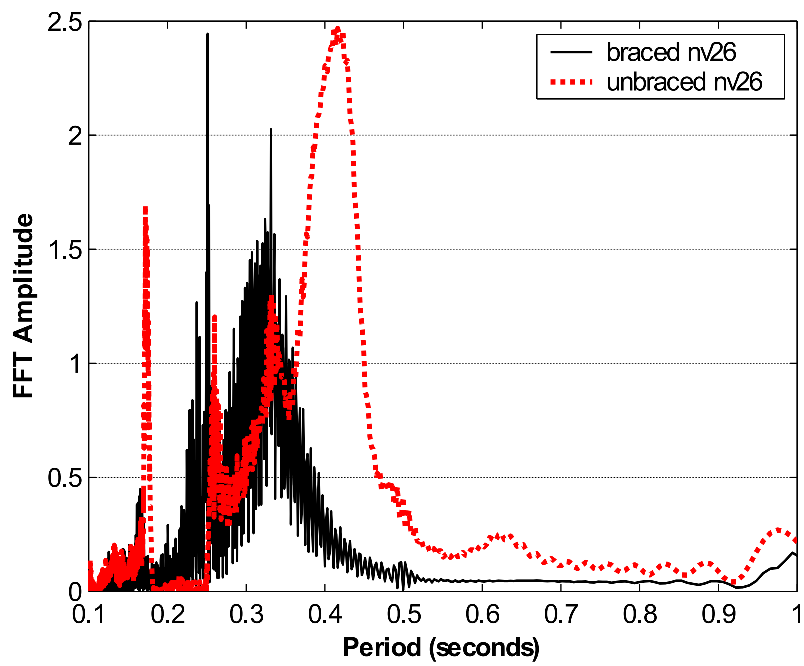

Figure D-43 Threaded Braced and Unbraced FFT of nv26

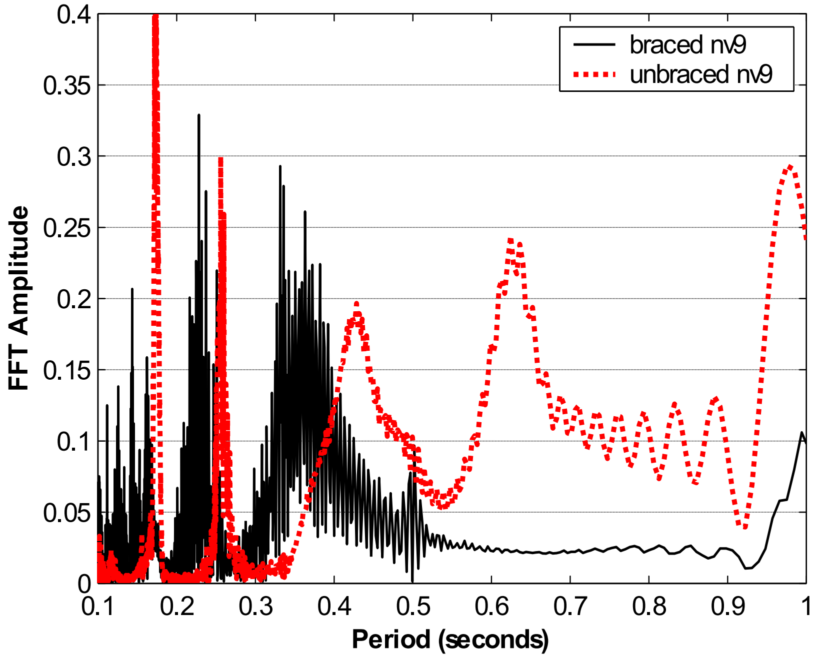

Figure D-44 Threaded Braced and Unbraced FFT of nv9

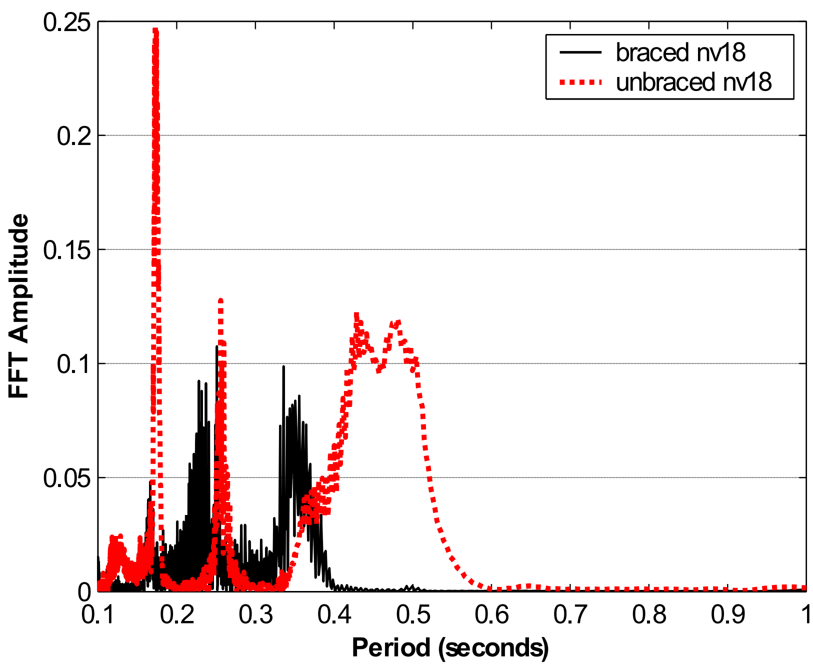

Figure D-45 Threaded Braced and Unbraced FFT of nv18

.2.3.2 Acceleration Comparison

Although the braces were effective in limiting the displacement of the subassembly, the braces did not restrict the acceleration response. In most cases, the braced acceleration response was larger than the unbraced acceleration response. Instrument nv38 was the only instrument to have significantly more acceleration in the unbraced case than in the braced; the unbraced acceleration was 1.48 times higher than the braced acceleration.

During the final experiments, nv49 fell off of the system. The only vertical acceleration recorded was nv13. This instrument is located near the set of valves on the west side of the system. For nv13, the unbraced acceleration was 2.37 times the braced acceleration.

D.3 Comparison of Welded Braced and Threaded Braced Subassemblies

This section compares the performance of the welded braced and threaded braced subassemblies. First, the braced subassemblies are discussed, followed by the unbraced subassemblies.

D.3.1 Dynamic Characteristic Comparison

The braces effectively changed the natural periods of both the welded and threaded systems. The braces forced the periods of both subassemblies to be around 0.25 seconds and 0.35 seconds. Both of the subassemblies had little to no response above 0.5 seconds.

D.3.2 Acceleration Response

The parallel acceleration response for the braced welded subassembly ranged from 0.77 to 1.28 times the braced threaded assembly. Only nv3 had a response that was higher during the threaded experiments than in the welded. The transverse response for the braced welded subassembly was always less than the braced threaded system. For the braced threaded and braced welded subassemblies, the overall acceleration response was very similar.

D.4 Comparison of Welded Unbraced and Threaded Unbraced Subassemblies

This section compares the performance of the welded unbraced and threaded unbraced subassemblies. First, the braced subassemblies are discussed, followed by the unbraced subassemblies.

D.4.1 Dynamic Characteristic Comparison

Without the braces restraining the motion of the subassemblies, the systems were able to vibrate relatively freely, therefore exhibiting their true periods. The welded subassembly had shorter periods compared to the threaded subassembly. Both unbraced subassemblies had a natural period of 0.17 seconds.

D.4.2 Acceleration Response

The parallel acceleration response for the welded subassembly ranged from 0.69 to 1.01 times the threaded assembly for East/West excitation. The transverse response for the unbraced welded subassembly ranged from 0.66 to 1.45 times the response of the unbraced threaded subassembly. The North/South excitation had very similar acceleration results to the East/West excitation.

D.5 Conclusions

This appendix described the experimental results found from conducting experiments on 4 full scale hospital piping systems. Comparative plots and tables were presented that summarized the experimental results.

The periods of the two braced subassemblies were similar. Neither subassembly had a response above 0.5 seconds. This leads to the conclusion that the braces effectively limit the displacement response of the subassemblies.

The response and periods of the unbraced subassemblies were different. The welded subassembly had lower periods than the threaded subassembly. This leads to the conclusion that the welded subassembly is stiffer than the threaded subassembly. The braces did not have a major effect on the acceleration response of either subassembly.

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