CHAPTER 1

Introduction and Literature Review

1.1 INTRODUCTION

Damage and collapse of structures as a result of overall lateral drift and residual lateral drift incurred due to large earthquake-induced ground motions is a well-documented hazard. As a result building codes in the United States have incorporated seismic provisions in an attempt to minimize structural damage and loss of life. Specifically, in areas of the Western United States where large ground motions (Magnitude 6.0+) occur at relative small intervals (10 years), seismic provisions have provided stringent guidelines for the design and detailing of structures for many years. However, despite historical evidence of large earthquakes, building codes in the Central and Eastern United States provided insufficient provisions for addressing the risk of future events until the adoption of the International Building Code (IBC), currently in its third edition, International Building Code (2006). The seismic provisions of IBC increased mapped ground acceleration criteria, resulting in greatly increased seismic design forces and drift limitations in locales where large but highly infrequent ground motions have occurred, such as Charleston, SC and The Mississippi River Valley.

Despite the incorporation of seismic provisions to current building codes throughout the United States, the seismic performance of a structure, defined as the condition of a structure subsequent to a design level seismic event, is only addressed in general terms. Performance of a structure is rated as operational, immediate occupancy, life safety, or near collapse after a seismic event. The rating system is primarily based on damage observed in conventional structures in past events. The vast majority of structures currently designed in the United States meet the criteria for life safety or near collapse. In other words, the existing building codes requires that structures resist collapse and loss of life, but may be unusable after the occurrence of a design event.

In an attempt to produce better-performing structures, research as it relates to seismic design of structures has been focused on developing auxiliary components which restrict the amount of damage incurred by a structure during a seismic event. Although still limited, base isolation, passive energy dissipation devices, or a combination of passive energy dissipation devices with base isolation are the most common methods of controlling damage to structures throughout the world. Several devices have been shown to be effective in controlling the amount of damage incurred by a structure during extreme ground motion. Despite demonstrated effectiveness of such devices, their use has been extremely limited. Such systems have been implemented where strong ground motions are common, and in buildings considered to be essential facilities or retrofit projects where alternative solutions are cost-prohibitive. Several factors contribute to the limited use of such devices, including economic feasibility, knowledge required for analysis, and limited code-provisioned benefit resulting from their use. Since the use of such devices is rare in areas of the country where knowledge required for analysis of structures with damping devices is common, it is logical to assume that their limited use is based primarily on cost and a lack of code-provisioned benefits.

This study considers the use of synthetic fiber rope devices as an innovative, cost effective alternative to existing passive control devices for the purpose of controlling maximum and residual drift of new and existing steel moment frame structures subjected to large lateral excursions and dissipating some energy during the response of the structure. Specifically, Amsteel II, a twelve strand, double braided rope produced by Samson Rope Company was considered for the devices. General information regarding the construction of the rope can be found at www.SamsonRopeCompany.com. Specific construction techniques and fiber material used for this rope are proprietary.

Several unique response characteristics of the synthetic fiber rope were considered ideal for the stated purpose. The tensile strength of the rope allows for extremely large loads to be resisted by a relatively small amount of material. When installed in an ideal configuration, the transition from nearly zero stiffness at initial conditions to relatively large stiffness at high levels of strain in the rope provides auxiliary stiffness to the steel moment frame as yielding of the steel moment frame is initiated, without increasing initial stiffness of the overall system. When properly preloaded, prior to installation, the rope devices exhibit a return to nearly zero conditions after extreme levels of rope elongation. Finally, hysteretic behavior of the rope through a complete loading cycle indicates that some energy dissipation occurs within the rope device.

1.2 SCOPE OF RESEARCH

The goal of this research program was to evaluate the viability of the use of synthetic fiber ropes for the purpose of mitigating damage of structures subjected to earthquake-induced ground motions. Specifically the rope devices were evaluated analytically and experimentally for use in two-story steel moment-frame structures. An exhaustive review of developed and experimental devices used for passive control of structures revealed that synthetic fiber ropes have not previously been considered for this purpose. Therefore an extensive experimental program was conducted, which adds to the current knowledge of the dynamic behavior of synthetic fiber ropes from a component level, in which single ropes were studied, to a system level, in which the ropes are installed in scaled structures. The experimental program was augmented with an analytical program in which a model of the rope device was developed for use in a finite element study using DRAIN-2DX, in the NON-LIN PRO (2003) environment, henceforth referred to as DRAIN. To obtain the overall goal, seven specific objectives were addressed with the experimental and analytical software programs.

The first objective was to determine the point-to-point component behavior of the ropes and to model that behavior in a finite element program. The internal mechanical properties of the rope devices were considered in the development of a model. However, it was beyond the scope of this research program to model the strand and fiber interaction within the rope or the eye splice contribution at the rope ends. Instead, the response characteristics of the complete rope as a tension-only element were observed experimentally, and a non-linear element definition was established in DRAIN that was suitable for approximating the observed behavior.

The second objective was to evaluate the energy dissipation provided by the ropes. Preliminary results show that some energy dissipation was associated with nearly elastic stretching of the ropes.

The third objective was to design a method for connecting the rope devices to a moment frame. Considerations for the connection design were efficiency of materials, ease of connection, and length adjustment at the connection.

The fourth objective was to evaluate an optimal installed configuration of the rope device. Over the course of this research project, the optimal initial condition of the rope device was determined to be one in which the structure undergoes some lateral deformation prior to engaging the rope device in tension. This was termed an “initial gap” and was considered one of two parameters that make up the installed configuration for a rope device. Once engaged, the rope devices were observed to exhibit a non-linear, hyper-elastic response in which the rope stiffness increases as elongation increases. The shape of the response curve of the rope was considered the second parameter of the installed configuration of the rope device, and was a function of the cross-sectional area, minimum break strength, and length of the rope. The two parameters of the installed configuration have a significant impact on the dynamic response of a structure in which the devices are installed. It was of interest to vary these parameters in an analytical DRAIN model for the purpose of optimizing frame performance for the frames incorporated in this study.

The fifth objective was to utilize a nonlinear finite-element study, incorporating observations from initial rope experiments, to estimate the effectiveness of a rope device with an optimized installed configuration in reducing the amount of steel required for the moment frames of a two-story prototype structure, while conforming to the strength and drift criteria of the IBC.

The sixth objective was to develop the largest scale model possible for available shaking table testing, based on the prototype frame and rope definitions of objective five. The cost associated with using small scale steel shapes, as was done in Wallace et al. (1999), was not to feasible, limiting the models used to standard steel shapes. Other limitations of the model size included shaking-table geometry, shaking-table servo-hydraulic system capacity. Similitude principles, shaking-table ballast tests, and time-history analyses in DRAIN were utilized to obtain the largest, properly scaled model that could be shaken with desired ground motion. Ultimately, a 1:6-scale model was developed for shaking-table testing.

The seventh objective was to demonstrate the viability of the rope device as an effective means of mitigating total and residual floor and roof drift, while limiting the increase of base shear to an acceptable level, through a 1:6-scale shaking table experiment. Since large excursions causing inelastic deformation of the model frame were of interest, two frames were required for testing. This allowed for a direct comparison of the performance of a scaled structure with rope devices versus one without rope devices. Floor drift, roof drift, and base shear were parameters used to compare the performance of both structures.

1.3 LITERATURE REVIEW

1.3.1 OVERVIEW

The vast majority of analytical research and physical testing of synthetic fiber ropes has been focused on marine towing and deep-water mooring applications. In fact, a review of current literature reveals that synthetic fiber ropes intended to be used for auxiliary control of structures subjected to earthquake ground motion has not been considered aside from preliminary publications related to this research project (Pearson, 2002; Hennessey, 2003). Therefore, a broad range of disciplines was drawn from to create a substantial knowledge base on which an efficient and effective testing and finite element program could be built. The development of current seismic design practice was reviewed to properly understand how the structural engineering design community currently handles seismic design. Development of control devices for structures under seismic loading was reviewed to determine the current capabilities of control devices. Rope mechanics studies were reviewed to gain insight into the internal mechanism of the response of synthetic fiber ropes, which may be useful in developing a finite element model with existing finite element formulations which are not currently used in typical structural applications. Steel wire rope mechanics was also reviewed to determine if parallels exist between the response of steel wire and synthetic fiber ropes. Identifying similarities in the response of steel wire rope, which is currently used extensively in structural applications, and synthetic fiber rope may also be useful in developing predictive methods for synthetic fiber rope. A brief summary of literature reviewed is presented herein.

1.3.2 DEVELOPMENT OF CURRENT SEISMIC DESIGN PRACTICE IN THE U.S.

The International Building Code (IBC) is authored by the International Code Council (ICC), established in 1994 to develop a coordinated and comprehensive building code to replace the three major building codes then being used throughout the United States: the Standard Building Code (SBC), the Uniform Building Code (UBC), and the code authored by the Building Officials and Code Administrators, Inc. (BOCA). The IBC has been adopted with individual state-added amendments as the model building code in its 2000 edition or 2003 edition form in 44 of 50 states in the United States. It will therefore be considered as the standard code in the United States herein. It should be noted that three states with high seismic hazard which have not adopted the IBC are California, Oregon, and Hawaii. These states have adopted the UBC with state-added amendments as their model codes. The primary focus of this study, however, is on seismic design criteria. Since the seismic provisions of both the IBC and the UBC are largely based on the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 2003a), the commentary herein on the seismic provisions of the IBC is applicable for adopted model building code provisions currently being used by structural designers throughout the United States.

The seismic design provisions of the IBC establish a procedure for analyzing a structure based on the maximum force that an structure of calculated or approximate periods of vibration and inelastic rotation capacity would experience. The primary goal of the procedure is to reasonably predict the maximum load demand of the structural components and foundation, and restrict the drift to known, stable limits of the structural system. The secondary goal of the seismic provisions is to provide inherent performance criteria for the structure.

Performance criteria, as they relate to the seismic design of a structure, can be defined as characteristics of the condition of a structure after experiencing a design earthquake. Legal requirements for the anticipated performance of a structure are specified by the IBC in terms of Seismic Use Group designation. Based on the Seismic Use Group designation, the IBC provisions attempt to achieve intended performance goals by regulating the type of structural system allowed, specifying detailing requirements, arbitrarily adjusting the design base shear, and placing limits on the allowable drift. The regulations result in progressively more conservative loading, drift, and detailing to obtain minimum levels of performance which are commensurate with the intended occupancy. The level of anticipated performance for each of three Seismic Use Groups is shown graphically in Figure 1.1, adapted from FEMA (2003b).

The vast majority of the structures that are designed to the standard of the IBC are designated as Seismic Use Group 1 or 2 occupancies. It can be seen in Figure 1.1 that the intended performance of these structures is to provide for life safety and to resist collapse after a design event has occurred. The capability of these structures to provide the specified performance goals is based on their capacity to undergo highly inelastic behavior. The result is extreme damage to primary load-carrying portions of the structure and non-structural components.

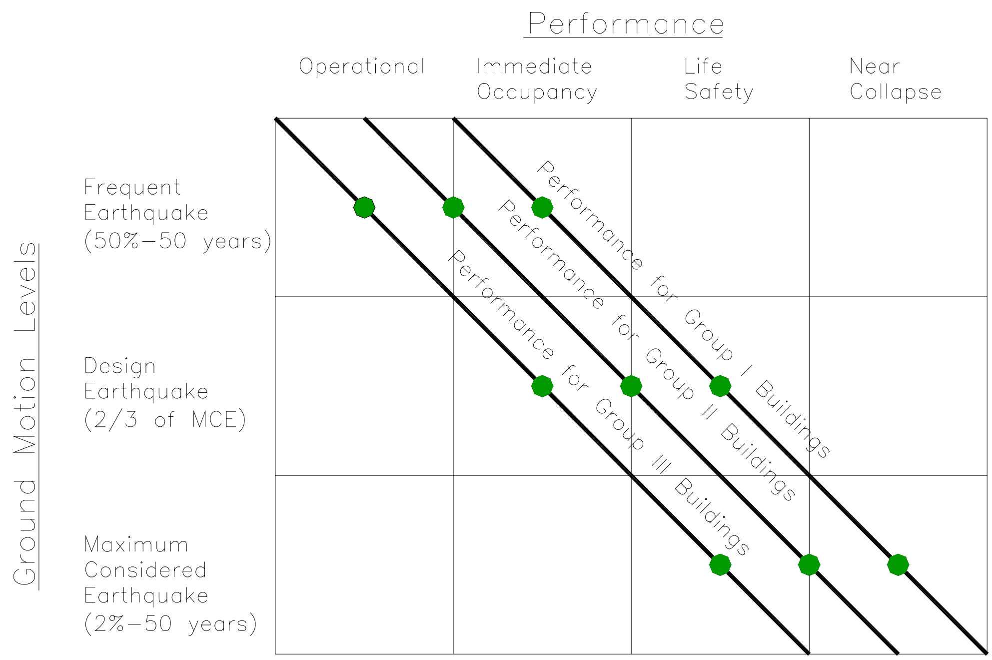

Figure 1. 1: NEHRP Intended Performance of Seismic Use Groups

The seismic provisions indicate that improved performance may be obtained by designing a structure for higher loads and by restricting drift, as is done for Seismic Use Group 3 occupancies. Effectively, the result is stiffer, but less economical systems. However, it should be noted that performance per the seismic design provisions is not checked directly. FEMA (2003b) describes this limitation as follows:

> *Although the Provisions explicitly require design for only a single level of ground motion, it is expected that structures designed and constructed in accordance with these requirements will generally be able to meet a number of performance criteria when subjected to earthquake ground motions of differing severity.*

The inherent assumption in this expectation is that the performance of a structure of a certain type of construction can be predicted based on historical data of performance of structures of a similar construction type. FEMA (2000a) also notes this limitation of performance standards in adopted building codes in more direct language by stating that

> *…the reliability with regard to the attainment of other (than collapse prevention) performance objectives for SUG-1 (Seismic Use Group 1) structures, or for*

reliably attaining any of the performance objectives for the other SUGs, seems less certain and has never been quantified or verified.

Although the current IBC Seismic Provisions were not implemented until 2000-2003, the validity of this type of un-quantifiable performance assumption was challenged in the Northridge Earthquake of 1994, a 6.7 magnitude (M_w) event occurring in the San Fernando Valley, California. This earthquake was determined to be significantly smaller than the building code-provisioned design event. However, extensive structural damage occurred, including collapse of several code-compliant structures, resulting in insurance payments totaling 20 billion dollars and residual financial loss of 800 billion dollars (EERI, 2003).

Over the past ten years, the damage to structures resulting from the Northridge event has been studied closely. The steel industry, motivated by a malfunction of a regularly utilized moment connection detail, began its own investigation. It was soon joined by the federal government in implementing the SAC Joint Venture, culminating in a series of design recommendations for steel moment frame detailing.

One result of these extensive investigations has been provisions implemented by the model design codes and referenced material specific codes. These changes have been largely prescriptive. For example, the IBC has developed seismic provisions with prescriptive redundancy and over-strength considerations. The American Institute of Steel Construction (AISC) has published steel seismic provisions (AISC, 2005b) which prescribe the use of detailing based on tested inelastic rotation capacity of beam-to-column connections, and provide more restrictive specification of welding materials and techniques. The American Concrete Institute (ACI) has developed *Chapter 21* (ACI, 2005) devoted entirely to special detailing requirements of lateral-force-resisting systems, including shear walls, moment frames, coupling beams, and diaphragms.

When followed correctly, these prescriptive measures should result in better-performing structures. However, large amounts of inelastic deformation will still be required to dissipate the energy input from design ground motions. Only a vague understanding of the extent of damage and the consequential cost impact due to that damage is possible. In response, a performance-based criterion for seismic design of building structures has been developing.

Performance-based design is an analysis process for predicting the amount of damage incurred by a particular structure at specific levels of ground motion. The challenge of performance-based methods is translating the non-quantitative performance requirements into measurable criteria that can be calculated. The most current criteria for performance-based design are included in FEMA publications, which are not directly adopted as part of a model code. A basic requirement guideline for the performance of retrofitted structures of all types is found in FEMA (2000d). A more thorough analysis method and acceptance criteria for the performance of new steel moment frames are found in FEMA (2000a).

1.3.3 PASSIVE CONTROL DEVICES

The overriding challenge of a structural response to ground motion remains the ability to dissipate a large amount of energy over a short period of time. Energy is effectively dissipated in conventional lateral-force-resisting systems through damage to the primary load-carrying components. Means of energy dissipation which isolate damage away from, or significantly reduces damage to, primary structural elements may be achieved by adding control devices to a conventional structural system. These devices can be categorized as active or passive. Active devices utilize an external power source to adjust the response of the devices to react to the behavior of a structure in real time and achieve a desired overall response. Passive devices, the focus of this study, do not use an external power source, but provide a velocity, displacement, or combined velocity and displacement-dependent response. Displacement and velocity dependency refers to the amount of resistance contributed by a device. A displacement dependent device contributes increasing stiffness as lateral displacement of the structure increases. A velocity dependant device contributes a zero resistance at zero lateral velocity, and an increasing resistance as lateral velocity of the structure increases.

Currently, provisions for designing a structure to include passive energy dissipation are provided in Chapter 15 of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 2003a). Many control devices have been researched, and several have been implemented in structures located in high seismic locations around the world.

1.3.3.1 FRICTION DAMPING DEVICES

Friction dampers are devices connected across the diagonal of a structural frame. Upon reaching the prescribed load and overcoming the static friction of the device, internal slip in the device occurs between the surfaces, allowing energy to be released in the form of heat created by sliding friction between surfaces. It is important for a stable coefficient of friction to exist between the surfaces to ensure a predictable response. Therefore careful attention is paid to the materials used for the contact surfaces to prevent bonding between the surfaces over time (Constantinou and Symans, 1992). Three devices that have been shown to achieve this outcome are the slotted-bolted friction damper, the self-centering friction spring damper, and the Pall cross-type friction damper.

Levy et al. (2001) examined the use of slotted-bolted friction dampers (Figure 1.2) as passive friction dampers at the ends of, or at the intersections of, steel braced frames. The study presented a two-phase design procedure for steel braces with slotted-bolted friction dampers. The design procedure assumed that the distribution of the nonlinear friction forces in the dampers is proportional to the modal shear forces in the first mode. The procedure is applicable for a response governed by the first mode. The study concluded that the two-phase iterative design developed works well in predicting behavior of a structural system with friction dampers.

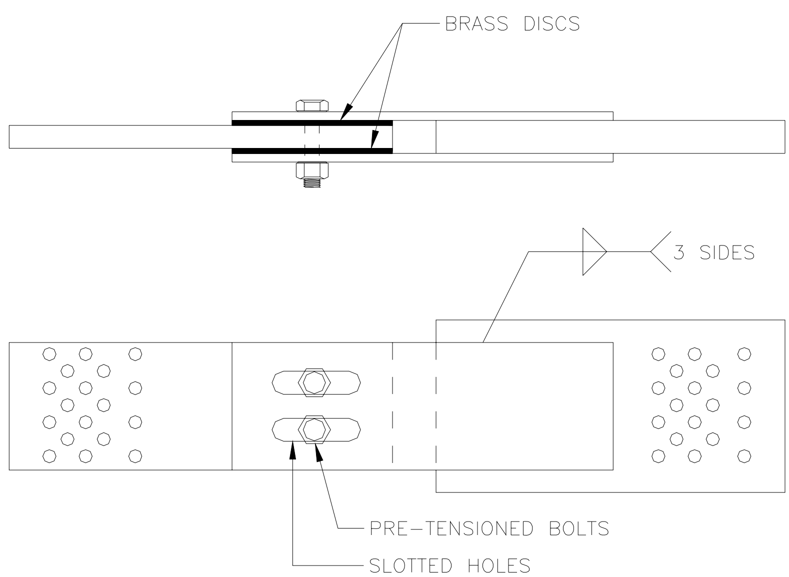

Figure 1. 2: Slotted-Bolted Friction Damper

Kikites et al. (2004) studied the use of slotted-bolted friction dampers and Pall cross-type friction dampers (Figure 1.3). The report reviewed a case study of a six-story steel frame dry-mix cement dispensing facility, which required a seismic retrofit solution. Obtaining an optimal slip load in the friction devices was the primary focus of the study. An optimal bracing layout was determined first. A model was then created in SAP2000 utilizing nonlinear link elements with elasto-plastic behavior in both tension and compression to model the friction devices. A nonlinear time-history analysis was then performed, using scaled 1994 Northridge Earthquake and 1995 Mexico City Earthquake acceleration histories and varying the slip load of the link element. For each analysis, energy input and energy dissipation of the link element were monitored. Optimal slip loads were determined based on overall story drift, percentage of energy dissipated, and base shear. It was concluded that the friction dampers were effective in limiting damage to primary structural elements, as measured by connection rotation. It was also determined that tuning the slip load of the devices was effective in obtaining the optimal response of the overall system.

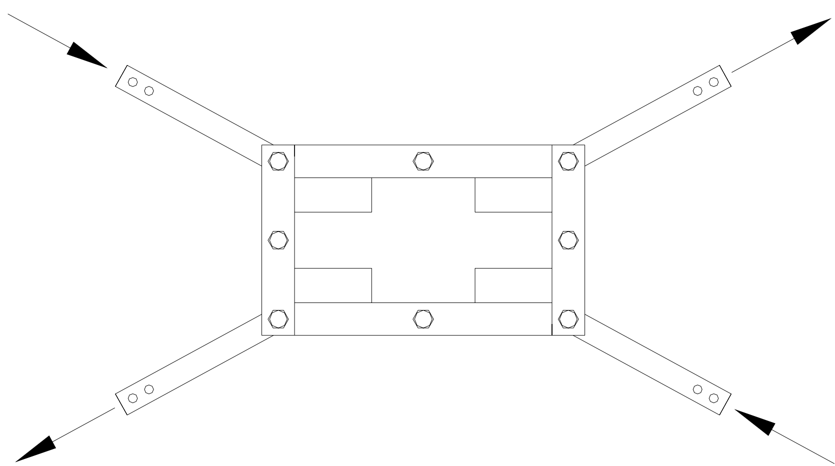

Figure 1. 3: Pall Cross-Type Friction Damper

Filiatrault et al. (2000) performed a physical test program of a self-centering friction spring damper device (Figure 1.4). The test program consisted of component tests and half-scale moment frame tests conducted on a shaking table. Component tests were conducted to determine self-centering properties of the dampers as well to evaluate the damper's ability to perform consistently over time. The tests included cycling the specimen axially at 0.5 Hz and 25-mm amplitude sinusoidal loading over ten cycles to determine the consistency of its stiffness. Frequency dependent tests were then conducted, cycling the specimen axially at 0.05, 0.2, 0.5, 0.1, and 2.0 Hz with a 25-mm amplitude. Earthquake simulation tests were conducted to determine the dynamic performance under irregular loading. The friction spring dampers demonstrated stable hysteretic behavior at all frequencies, which was nearly independent of seismic input. Shaking table tests consisted of (1) low-amplitude varying-frequency excitations to determine the fundamental frequency and damping characteristics of the frame, and (2) seismic tests in which the frame was subjected to 100% and 200% El Centro acceleration histories. The tests were conducted with and without the friction spring dampers to obtain direct comparison of the frame's response. The shaking table test showed that the friction spring dampers reduced the fundamental period of the frame without braces from 0.350 seconds to 0.207 seconds, and reduced the maximum displacement by 50% and 30% for the 100% and 200% El Centro acceleration inputs, respectively. It was shown that 20% and 40% dissipation of the input energy for 100% and 200% El Centro acceleration input, was observed using the devices. DRAIN-3DX was used to simulate the shaking table experiment, and good correlation was shown. It was concluded that the behavior of a friction spring damper provides significant energy dissipation, is stable over time and loading history, and can be modeled accurately using standard finite element software programs.

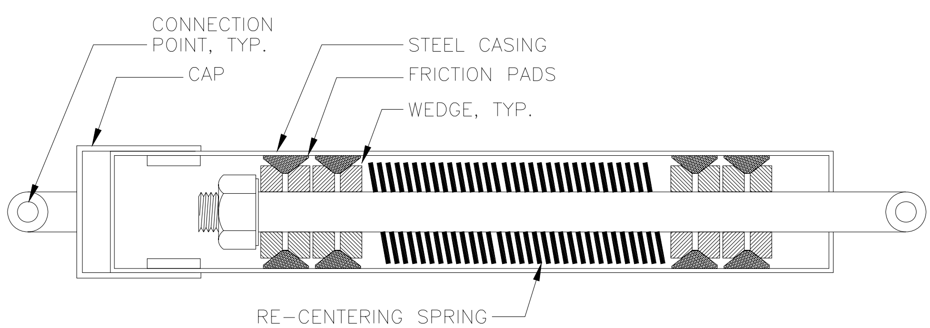

Figure 1. 4: Self-Centering Friction Damper

1.3.3.2 METAL YIELDING DAMPERS

Metal yielding dampers are devices which are designed to provide energy dissipation to a building frame system through stable, inelastic yielding of a mild steel element. Like the friction dampers, metal yielding dampers increase the initial stiffness of a moment frame. When subjected to loading at a level below the elastic capacity of the device, the initial structural stiffness is constant. Upon reaching the elastic capacity of the device, the load in the device remains constant through further inelastic deformation. Two devices which have been installed in structural retrofits or new construction are the Added Damping And Stiffness (ADAS) device (Figures 1.5 and 1.6), and the Unbonded Brace™ damper (Figure 1.7).

Xia and Hanson (1994) studied the use of the ADAS device in steel moment frames. Several characteristics of the frame and their influence on the earthquake response of the building structures were studied. These characteristics included device yield force, yield displacement, strain-hardening ratio, ratio of device stiffness to the bracing member stiffness, and ratio of device stiffness to structural story stiffness without the device. It was concluded that addition of ADAS devices could significantly increase the energy dissipation capacity of a structure, while isolating the dissipation demand away from the beam-to-column frame connections.

Phocas and Pocanschi (2003) presented a study of a variation of the ADAS device to be used in a steel moment frame as part of an original design or a retrofit option. The device consisted of a closed-circuit, tension-only rod-brace system, attached to a multiple steel plate fixture on the bottom center of the beam. The steel plates were expected to yield at a prescribed load, and to provide stable hysteretic behavior thereafter. The researchers assumed that the steel moment frame and the cables remain elastic throughout loading and that the damper exhibits rigid-plastic hysteretic behavior. An approximate design method based on a response spectra approach for a single-degree-of-freedom model with no damping was developed. The method was validated using a SAP2000 model. It was suggested that this damping device be considered for testing and further investigation.

In 2004, Black et al. conducted component testing on Unbonded Brace™ dampers, and evaluated the use of these dampers in building structures. Component tests were used to verify that the inelastic response that was theorized for these dampers was accurate, and to calibrate a model of the brace behavior. The study concluded that the capability of the braces to undergo several cycles while maintaining stable inelastic behavior and limiting drift of the structure make it a reliable and practical means of enhancing earthquake resistance of building structures.

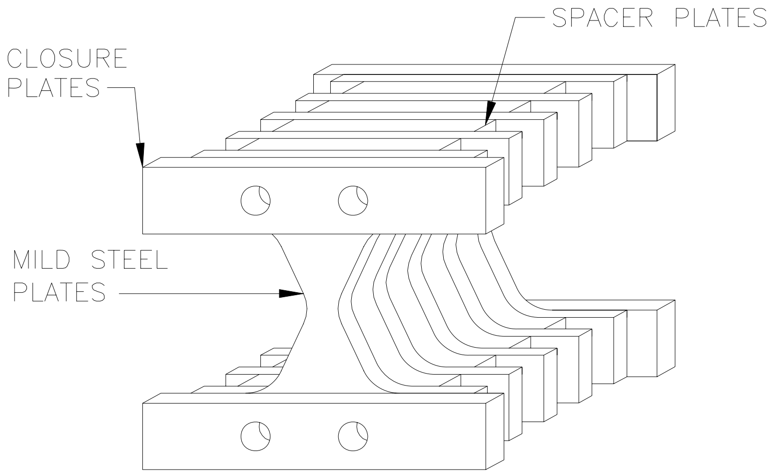

Figure 1. 5: ADAS Device

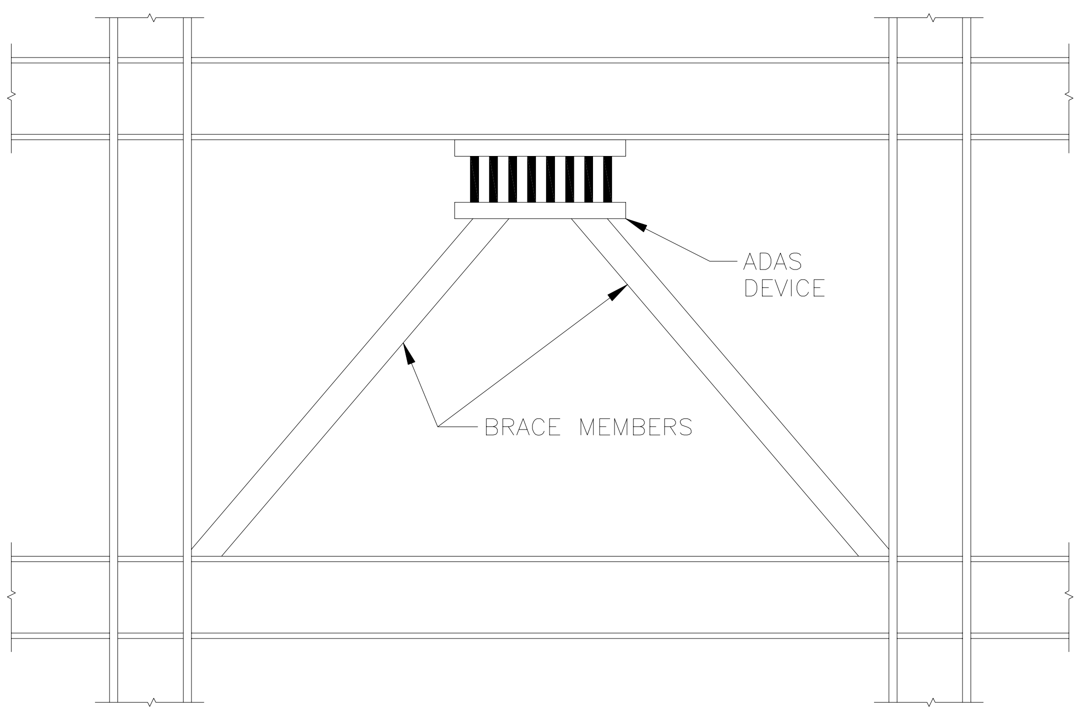

Figure 1. 6: ADAS Device in Frame

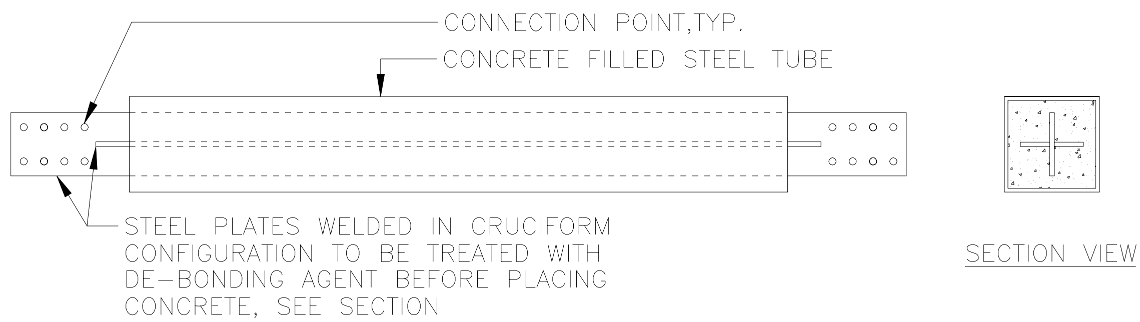

Figure 1. 7: Unbonded Brace Damper

1.3.3.3 VISCOELASTIC DAMPERS

Viscoelastic dampers are constructed of acrylic polymers sandwiched between, and bonded to steel plates, and placed in braced frames (Figures 1.8 and 1.9). These devices were initially conceived to control low-amplitude, high-frequency wind vibrations. The suitability of these devices to control earthquake-induced vibrations was consequently studied in component tests and shaking table tests (Constantinou et al., 1992). Also, several studies have been conducted to determine the most efficient and accurate analysis methods for viscoelastic devices in structures.

Shen et al. (1995) presented results of shaking table tests of a 1:3-scale, three-story, 3-bay by 1-bay concrete frame retrofitted with two different viscoelastic dampers. The concrete frame was initially damaged in tests studying the response of a lightly reinforced concrete frame. The frame was then fitted with viscoelastic damping devices at each level predicted to provide 12% and 20% additional damping in successive tests. The actual damping was determined to be 18% and 30%, respectively, in low-amplitude tests. The frame was then subjected to the Taft Earthquake, scaled to 0.2 g, and the results were compared to the results of the undamped frame subjected to the same ground motion. Base shear and story drift were reduced significantly when dampers were added. Behavior of the damped structure was accurately predicted using a modal strain energy analysis method for predicting equivalent structural damping. This method had previously been shown to work well for steel structures.

A limiting characteristic of viscoelastic dampers has been shown to be a change in response due to mild changes in ambient temperature (Chang et al., 1992). A study to quantify the amount of energy dependence and evaluate methods of predicting behavior of viscoelastic devices under a wide range of ambient temperatures was conducted by Chang et al. (1995). In this research, an extensive experimental study of a 2:5-scale, five-story, single-bay, steel frame structure with viscoelastic damping devices was completed. Three types of viscoelastic devices were studied at three different temperature levels between 77° F and 107° F. The steel frame was subjected to ground motions of increasing intensity, and remained elastic throughout the testing program. It was determined that the effective critical damping ratio was significantly affected by a variation of the temperature over the range studied. Therefore it was concluded that viscoelastic dampers should be designed with maximum temperatures in mind.

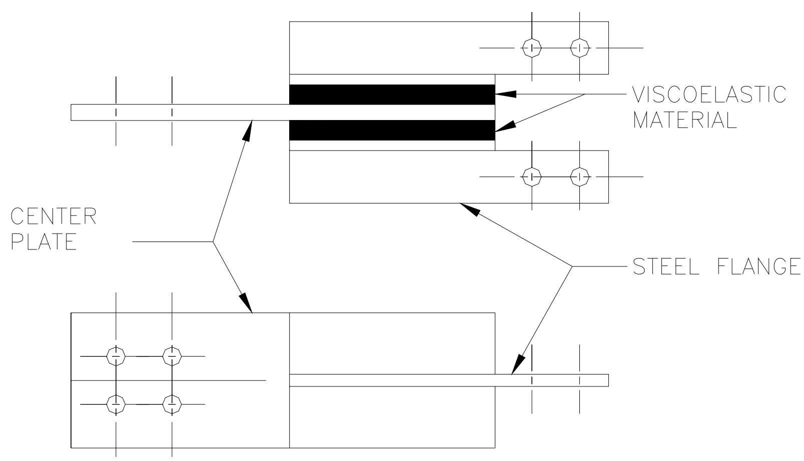

Figure 1. 8: Viscoelastic Damper

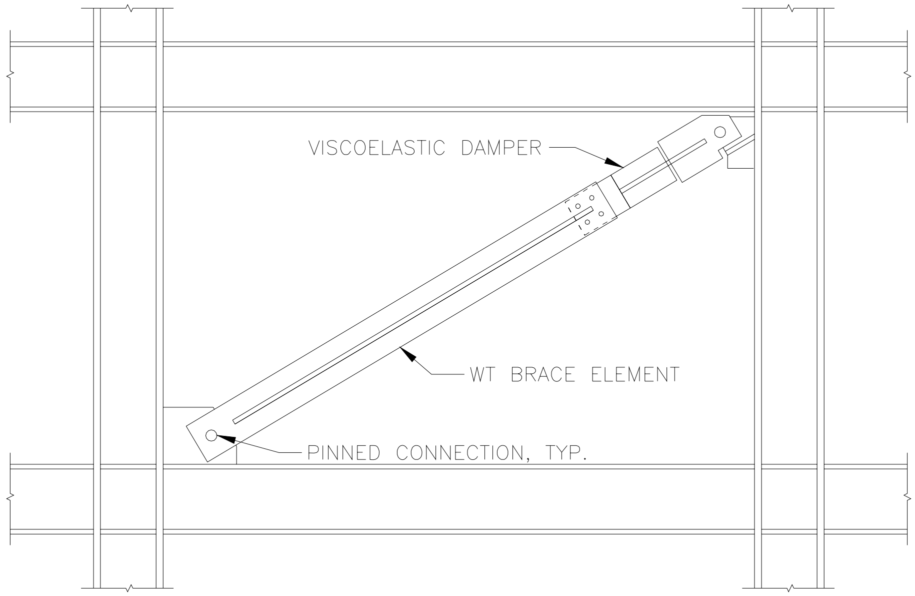

Figure 1. 9: Viscoelastic Damper in Frame

To verify accurate scaling of results from tests conducted by Chang et al. (1995), full-scale tests using viscoelastic dampers were performed by Lai et al. (1995). The test program consisted of a five-story steel frame structure, which was scaled up based on Chang's 2:5-scale frame. The frame was subjected to free vibration and forced sinusoidal loading. It was concluded that the linear viscoelastic theory shown to be accurate in predicting system response for the scaled test was directly applicable to the full damped structure.

Shukla and Datta (2001) conducted an analytical study of the response of building frames with viscoelastic devices using three models for viscoelastic devices: the Kelvin model, linear hysteretic model, and Maxwell model. The responses for each model were obtained by three methods: iterative pseudo-force method, approximate modal strain energy method, and an exact method. The relative displacement and pseudo-acceleration of each model using each method for predicting structural damping were compared. The study determined that analysis conducted with the pseudo-force method provides exact responses for all models. Using the modal strain energy method for predicting structural damping provided exact results when the total system can be considered classically damped. However, the analysis overestimated the displacement and pseudo-acceleration of non-classically damped systems when modal strain energy method was used. The linear-hysteretic and Kelvin models produce nearly identical responses, and the Maxwell model results in somewhat higher responses.

Guo et al. (2002) illustrated a method for conducting a reliability study of a shear beam type structure with viscoelastic dampers subjected to random seismic excitation. Equations of motion of a regular structure with any number of floor levels were developed. The stochastic response of the system was determined, and the response of the system with parameter uncertainties was evaluated. The rational method developed was applied to a ten-story structure with and without viscoelastic dampers, and the conditional failure probabilities during a 50-year service life of the structure and the dampers were determined. Maximum story drift of the structure was the structural limit state, and maximum deformation of the dampers was the damper limit state. The story drift and the failure properties were shown to decrease when the viscoelastic dampers were included if the failure probabilities of the viscoelastic dampers were much smaller than the failure probability of the structure. The results of the study were strictly theoretical and required some assumptions that should be examined more closely before implementing the method.

Lee et al. (2002) examined the use of the modal strain energy method, the complex mode superposition method, and the direct integration method for the analysis of practical structures with viscoelastic devices. A method of condensing the mass, stiffness, and system damping matrices of an arbitrary structure to reduce the number of degrees of freedom, thereby decreasing the computational effort, was presented. A spring in parallel with a dashpot was presented as the model for the viscoelastic damping devices. The spring/dashpot system had two degrees of freedom at each of its two end nodes. It was explained that for compatibility, the rigid diaphragm assumption and the matrix condensation technique also have to be applied to the spring/dashpot before they are added to the condensed system matrix. The three methods of analysis were then applied to the full model and to the condensed models. The resulting story drifts of the analyses were compared. In the eigenvalue analysis, it was concluded that the major vibration modes were mostly preserved when the full model was replaced by the condensed matrix model. Additionally it was shown that the matrix condensation technique resulted in accurate inter-story drifts, regardless of the location of viscoelastic damping devices. The procedure was recommended for use in preliminary design or optimization of viscoelastic damper positioning.

1.3.3.4 FLUID VISCOUS DAMPERS

Fluid viscous dampers (Figure 1.10) are fluid-filled cylinders with two chambers that are separated by a moving piston with directional orifices, and an accumulator chamber. As the head moves longitudinally within the shaft, viscous fluid flows from one chamber to the other. The force in the damper is a result of the pressure differential between chambers, which is a function of the orifices in the piston head and the velocity of the piston head (Constantinou and Symans, 1992).

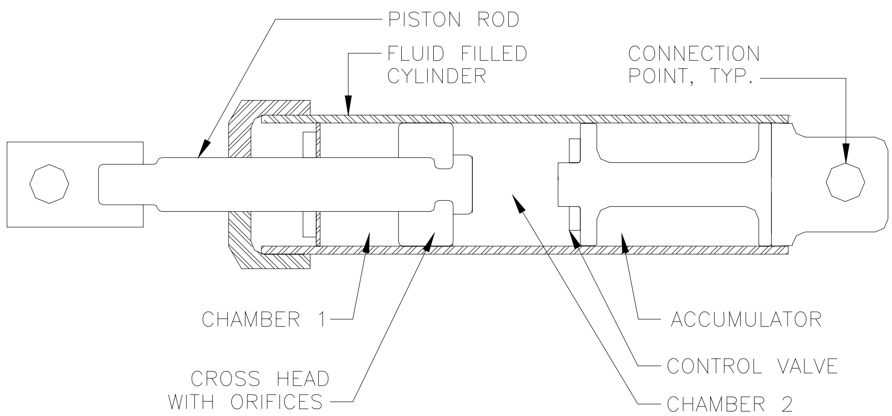

Figure 1. 10: Fluid Viscous Damper (by Taylor Devices, Inc.)

Fluid viscous dampers were first used in classified military applications, beginning in the late 1960's, and eventually found use in the Civil Engineering field in commercial building and bridge structures for earthquake and wind response mitigation in the late 1980's (Taylor, 1999). Consequently, several studies have been conducted on damper components and building frame systems with fluid viscous dampers.

In 1992, Constantinou et al. reported physical tests of fluid viscous damper components and 1:4-scaled steel frames with fluid viscous dampers. The component test program consisted of 58 individual tests in which temperature, and frequency and amplitude of sinusoidal loading, were widely varied. It was concluded that the dampers exhibited stable behavior over a wide temperature range, and that the force in the damper could be expressed by a simple Maxwell model, dependent only on the velocity of the piston head. Sixty-six shaking table tests were conducted on one-story and three-story steel frames, in which the dampers were added in different configurations. The force in the dampers was shown to be 90 degrees out of phase with the peak ground-motion-induced frame loads. Overall base shears and story drifts were shown to be reduced for both one-story and three-story test frames when fluid viscous dampers were added. Comparisons to other passive control devices illustrated superior performance of fluid viscous dampers. It should be noted that the frames remained elastic throughout testing.

Miyamoto and Hanson (2002) summarized a case study of a two-story police head-quarters in Vacaville, CA. The goal of the study was to evaluate the use of a fluid viscous damper to augment a special moment resisting frame (SMRF) system. The moment frame system was first designed without the fluid viscous dampers using the equivalent lateral force procedure provisions of the 2000 edition of *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structure*, which has subsequently been superseded by FEMA (2003a). The same building was then redesigned with viscoelastic dampers using the design procedure in accordance with the provisions of Appendix A of the 2000 edition of *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, which has subsequently been superseded by *Chapter 15* of FEMA (2003a). Adding dampers allowed for a reduction in the required size of roof beams, and elimination of grade beams which were required in the frame without dampers. The performance of both systems using a nonlinear time history analysis with an applicable 475-year return ground motion acceleration history was then analyzed. The study concluded that the FEMA (2003a) procedures were a very effective way to design a structure with fluid viscous dampers. It also showed that the maximum roof drift was reduced by 56%, and the maximum base shear was reduced by 65%, while keeping all SMRFs elastic, when the fluid viscous dampers were added.

1.3.3.5 Newly-Developed Control Devices

All devices described above have been used to augment frames in new construction and retrofits of existing structures. Although many of these devices have been shown to modify the response of structures in desirable ways, each device has associated drawbacks. Therefore, research in this field is ongoing. Many devices have been researched analytically and experimentally, with varying results.

Aguirre and Sanchez (1992) described an investigation of the energy dissipation characteristics of U-shaped mild steel strips. The strips were subjected to cyclic loading, and the viscous effects as well as the fatigue life were investigated. The paper developed a force-displacement model of the damping device. A prototype of several devices used in series was constructed and tested.

Aguirre and Aguirre (2000) discussed the use of oval-shaped steel dampers in steel X-bracing in a reinforced concrete moment frame for the purpose of limiting damage of the structure due to inelastic behavior of the moment frame. The authors suggested that a reinforced concrete moment frame, designed to behave elastically up to the allowable story drift limit, can be augmented with structural seismic dampers, which are detailed in Aguirre and Sanchez (1992). The paper presented a brief study of a six-story, regular reinforced concrete structure conforming to the appropriate building code for Mexico City, RCDF-93, as a general illustration of the design procedure. The paper concluded that the structural dampers make it possible to design an economical reinforced concrete frame structure that will remain elastic when subjected to design earthquake loading.

Peckan et al. (2000a, b) investigated an innovative supplemental bracing design concept to be employed for providing damping for mid-rise structures, in which post-tensioned tendons are installed across the perimeter moment frames of a structure from top to bottom. An “off-the-shelf” self-centering spring damping device was combined with a “fuse-bar”, which is a threaded rod designed to yield at certain load levels. The primary intent of using the tendon/damper system was to provide sufficient damping to prevent unacceptable joint rotations. Additionally, the system was predicted to also prevent significant damage when the structure is subjected to near-field, seismic displacement pulse loads at or near the resonant frequencies of the structure, for which the energy-dissipating ability of a damper-only system is greatly diminished.

In their first study, Peckan et al. (2000a), a 1:4-scale six-story, three-bay moment frame model was built and tested on a shaking table, with several different tendon configurations. A tendon profile that balances the inertial loads, and was proportional to the first-mode shape, was shown to be the most effective in mitigating inter-story drifts. A SAP2000 model of the damping device was also developed. As a result of the shaking table tests, the performance of the supplemental system was shown to be an effective means of reducing ductility demand on beamcolumn connections. The study concluded that further analytical studies should be conducted to quantify and generalize the results of the 1:4-scale testing.

In their second study, Peckan et al. (2000b) used the tendon profile found to be most effective in their first study for further investigation. The profile of the tendons can be best described as piecewise linear over the height of the structure and designed to balance the first-mode inertial lateral loads. The tendons were to be continuous, anchored at the top corner of the structure at one end and to the damping device at the other. The tendons slide through holes in the girders on bearings designed specifically for this purpose. The paper developed an iterative design methodology for this system by first assuming an idealized single-degree-of-freedom system and neglecting supplemental damping for the structure for the preliminary design. Supplemental damping due to self-centering spring damping devices and the fuse-bars was then added and the structure was analyzed dynamically as a multiple-degree-of-freedom structure to determine the optimal layout of the tendons and amount of damping. The total system was then verified by performing an analysis using the maximum considered earthquake force. An existing nine-story steel moment frame structure was then used to demonstrate the effectiveness of the tendon system to provide adequate damping to either eliminate yielding in the structure or allow a specified percentage of plastic hinge formation in the structure under strong ground motions. It was concluded that the system would be useful in retrofitting structures with inadequate inelastic rotation capacity at beam-to-column connections.

Mualla and Belev (2002) conducted a physical testing program to investigate the use of an experimental friction device (Figures 1.11 and 1.12) for dissipating energy of a steel frame under harmonic loading. The friction device was made up of a central vertical plate, two horizontal side plates, and two circular friction discs sandwiched between the vertical and horizontal plates. The device was installed at the connection point of a chevron brace just below a steel beam in a 1:3-scale steel frame. A harmonic horizontal load was applied at the beam level of the frame at a varying frequency between 1 and 7 Hz under displacement control with 4.5-mm peak displacement. Moment-rotation plots were obtained for each frequency and appeared nearly identical, indicating that the hysteretic behavior of the device was almost independent of frequency. Force versus displacement plots showed the hysteretic behavior to be nearly bilinear in the range of testing. Finite element models of the frictional damping device were created using the bilinear characteristics added to a frame modeled in a separate study. The performance of the friction-damping device was then determined when subjected to the acceleration history of the El Centro Earthquake. Different braces with varying levels of pre-stress were evaluated. It was concluded that when the strength and stiffness of the damping system are properly selected, the response base shear and displacement can be reduced significantly, and that the friction damping device is a viable alternative to ductility-based earthquake resistant design.

Corbi (2003) studied tendons made from shape memory alloys (SMAs) for use in improving performance of structures subject to horizontal shaking. Two types of SMAs are of interest; alloys that exhibit "super-elasticity" (the ability to experience extremely large strains while maintaining stiffness and recovering nearly 100% of displacement upon unloading) and alloys capable of recovering inelastic deformations upon the application of high temperatures. Both were considered for control of structures under earthquake loading. An analytical model was established and evaluated. The study concluded that the use of SMA members resulted in improved dynamic response.

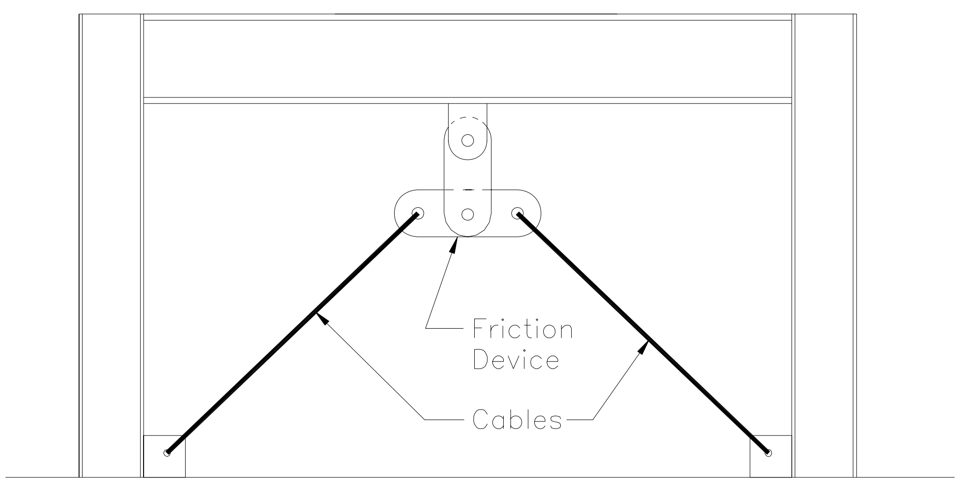

Figure 1. 11: Experimental Friction Damping Device in Frame

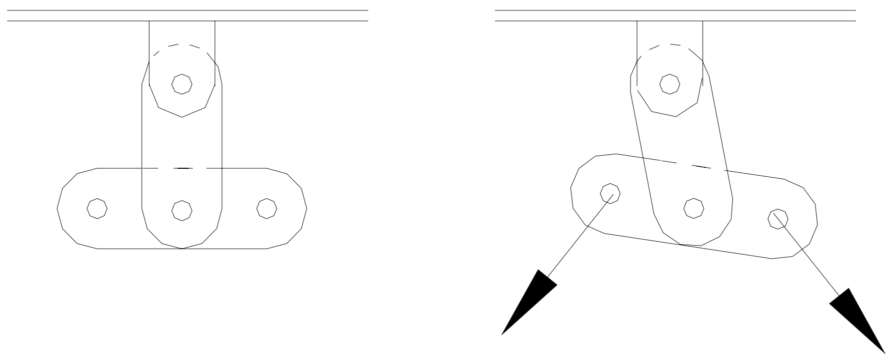

Figure 1. 12: Experimental Friction Damping Device

The analytical results for an innovative, conceptual, elasto-plastic passive energy control device was presented in Ibrahim and Charney (2006). The device consists of viscoelastic rubber compound sandwiched between curved steel plates. The size of the steel plates and the viscoelastic mass were said to be variable, depending on a specific application. In this study the steel plates were separated by 72 in. of viscoelastic rubber in the center, curving to meet at an attachment point at each end. The width of the device was 12 in. and was oriented such that it could fit into a wall that was slightly larger than 12 in. An ABAQUS model of the device was developed. When loaded axially, the analytical model exhibited a hyperelastic response, which included significant damping, due to the axial strain of the viscoelastic rubber. A simplified model for the device was developed using a general-use structural software program. The response of the simplified model was shown to correlate well with the ABAQUS model when subjected to the same loading. The device was incorporated into a 9-story moment frame in a diagonal brace configuration. A response history analysis was conducted on the frame with and without the devices, using two different ground motion records. The device was shown to be effective in mitigating drift of the 9-story structure without increasing the maximum base shear for both ground motions considered.

1.4 SYNTHETIC FIBER AND STEEL WIRE ROPE

1.4.1 MECHANICS OR SYNTHETIC FIBER AND STEEL WIRE ROPE

To better understand failure mechanisms of synthetic fiber rope, Wu et al. (1995) developed an analytical model based on the summation of tension in individual strands of double braided ropes. The geometry of a characteristic strand was modeled as a “helix about a cylinder with sinusoidal fluctuations in the radial direction”, based on three reproducible characteristics of rope, rope diameter, core diameter, and rope period length, measured at a standard rope tension. Compatibility constraints were then discussed. It was explained that the overall strain of the rope is equal to the summation of the individual strains of fibers over their helical period, and that the tension in the rope is constant across its length. Two extreme conditions were considered for the model. In the first, zero friction was assumed to exist between strands of rope, allowing for unrestrained relative slip between strands. In the second condition, infinite friction was assumed and no relative slip was allowed between strands. The load in the individual strand was then evaluated using the nonlinear stiffness behavior determined in axial tests of straight strands. Tensile load was summed for all strands over a range of tensile loads, resulting in a theoretical quasi-static stiffness model for both conditions. Physical rope tests, in which the ropes were cycled ten times between zero load and 50% of break strain and then loaded slowly until failure, were conducted. The results of the quasi-static loading up to failure were compared against the theoretical stiffness models. It was shown that the model stiffness correlates well with the trend of the experimental stiffness, with a slightly consistent higher stiffness across the range of loading. Also, the stiffness model predicted by each condition was identical, up to the ultimate load of the infinite-friction condition. The ultimate load of the zero-friction condition was shown to be 10-15% higher, with the ultimate experimental load falling near or above that of the infinite-friction condition. Other observations of note included an increase of the quasi-static stiffness of the rope when cycled, consistent failure initiation at core layers of the rope, and sudden failure of the ropes at ultimate load.

Leech (2002) detailed the inter-frictional forces and intra-frictional forces developed in polymer fiber ropes. An explanation of the hierarchical structure of the rope was first presented to classify the difference between inter- and intra-frictional forces. It was explained that braided ropes consist of a four-level hierarchy of sub-structures. Fibers or filaments are combined to create the first structural level, yarns. Yarns are then combined to form rope-yarns, which are intertwined to create strands. In braided construction, layers of braided strands often encapsulate a core of parallel strands. In double-braided construction, braiding of each layer of strands is done in two directions to prevent twist of the rope under load. The paper presented four forms of component inter-frictional modes: slip, twist, scissoring, and sawing. These modes occur between transversely discrete rope components. Rope components were defined as transversely discrete if, as a group, they can no longer be considered continuous across the rope structure. For double-braided ropes this occurs between the strands, but not within strands. Within a strand, hysteretic behavior was associated with stretching or squashing. The associated intra-frictional modes are called dilation and distortion, respectively. For each mode, this paper presented a virtual work formulation.

Huang et al. (1996) studied the analytical model of a cable comprised of interacting helical wires, between which slip is allowed, subjected to quasi-static cyclic loading. Load-elongation relationships were obtained for loading and unloading stages, and were used to determine overall energy dissipation. It was shown that the energy loss was proportional to the third power of the forcing amplitude. The study concluded that when inertial properties of a cable can be neglected, a solid model with equivalent stiffness and amplitude-dependent damping can be used. Also, an explicit method for determining an equivalent coefficient of damping was derived.

Flory et al. (1997) explained changes in the Cordage Institute guidelines for testing fiber ropes, which had been updated from the test methods developed in the 1970's. The paper described the difference between the reference force, which was defined as the force at which the standard geometry of a rope is to be measured, and the initial force, which was defined as the low tensile force from which elongation and extension measurements can be referenced. Because fiber ropes usually undergo permanent deformation when subjected to a higher load, a distinction between elongation and extension was made. Elongation was defined as the change in rope length from the initial rope length, prior to cycling. Extension was defined as the change of length of a rope during any particular force cycle. The paper then outlined an updated methodology for measuring a rope and testing the breaking force of a rope.

Labrosse et al. (2000) studied the energy dissipated through pivoting of helically-wrapped strands about a central wire strand in axially-loaded steel wire rope. Pivoting was defined as the local rotation of an exterior wire about the central wire. A hysteretic model of the axial response of a rope as a result of interwire pivoting friction was derived, and a numerical study was conducted considering different lay angles of the exterior strands and different end conditions. It was concluded that energy dissipation due to interwire pivot friction was negligible when compared to slip friction of the wire rope. It was also concluded that the interwire pivot friction is not substantially impacted by the lay angle of the exterior wire strands, or by the end conditions.

1.4.2 Dynamic Studies of Synthetic Fiber Rope for Non-Structural Applications

Ringleb (1957) studied initially-straight elastic cables subjected to oblique-angle impact loading. A practical example of this type of loading case is an arresting cable on the flight deck of an aircraft carrier. The relationship between initial velocity and impact angle of a mass and the stress and motion of the cable just after impact was developed.

Sauter et al. (2002) studied the effect of interstrand friction on the damping characteristics of a steel cable used in Stockbridge dampers. The damper consisted of a short cable attached at its midpoint and free at each end, where steel masses were attached. The cable was not loaded axially. A description of Stockbridge dampers and their uses can be found in Saunter and Hagedorn (2002). Instead the dampers relied on bending of the cable to provide energy dissipation to conductors of transmission lines undergoing wind-induced oscillations. The study developed a dynamic analytical model approximating the hysteretic behavior of the Stockbridge dampers and conducted physical tests on the dampers. The results of the analytical model were compared to the experimental data with good correlation when the damper was excited up to its fundamental frequency. Accuracy of the model was shown to diminish as the excitation approached the second resonant frequency.

Hooker (2000) examined the increased use of high-molecular-weight polyethylene ropes over steel ropes in towing applications. Four key aspects of HMWPE rope were examined and contrasted to steel wire rope. Wire rope was found to be superior with respect to weight, ease of handling, service life, and safety. The drawbacks of the HMWPE rope included susceptibility to local damage, and high stiffness. It was also observed that the weight of HMWPE rope versus its ultimate strength, defined as tenacity, was far superior to that of steel wire rope.

1.4.3 Studies of Synthetic Fiber Rope for Use in Structures

Pearson (2002) studied the transition of high-modular synthetic fiber ropes from slack to taut conditions, when loaded with an impact load. This type of loading was referred to as “snap” loading. Eleven ropes, constructed of one type of fiber, or a combination of different types of fibers, including polyester, high-molecular-weight polyethylene (HMWPE), aramid (Kevlar), and olefin, were tested. Ropes with approximate initial lengths of 7 ft and 9ft and nominal diameters equal to 0.375 in., 0.5 in., and 0.75 in. were tested. Some ropes were pre-loaded to approximately 10% of rated break strength to determine the static elastic properties of the rope for comparison to the dynamic response of the ropes when subjected to dynamic loading. Other ropes were tested dynamically without applying preload. The dynamic loading consisted of attaching a weight to each rope considered and allowing the weight to fall freely until engaging the rope in tension. The number of drop tests per specimen varied from 5 to 16, depending on the rope specimen. The weight, attached at the end of the rope, varied from 25 lb to 145 lb, and the drop height varied from 48 in. to 98 in. Acceleration of the bottom end of the rope, tension at the top end of the rope, and displacement of the bottom of the rope were measured. In general, it was concluded that the ropes dissipated some energy during dynamic loading and that the rope stiffness increased as load excursions increased. Permanent elongation of the ropes was observed throughout testing. The ropes which were preloaded prior to drop-testing experienced much less permanent elongation than the ropes which were subjected to some preload prior to drop testing. In general, permanent elongation experienced by a specimen reduced in successive drop tests of the same height, indicating a stabilization of permanent elongation without increasing the maximum rope force. One particular type of rope, Amsteel-II, appeared to exhibit greater stability related to permanent elongation. The study included eleven types of rope and was widely varied in scope and no specific conclusions about the behavior of one rope type were made. The study did provide insight about which of the eleven types of ropes might be eliminated in a future study.

Hennessey (2003) continued the study conducted by Pearson (2002). In this study, static tests and drop tests similar to those conducted by Pearson (2002) were conducted on 0.5-in.-diameter Amsteel Blue and Amsteel II ropes only, with an initial length of 9 ft. These ropes were chosen to represent two distinct response characteristics of the eleven types of ropes tested by Pearson (2002). Two pre-loaded ropes and ropes without pre-load of each rope type were subjected to a series of drop tests. Drop tests were conducted on each specimen, whereby two drops at each drop height, varying from 56 in. to 2 in. in 6-in. increments. The weight used for all tests was 65 lb. The goal of the experiment was to develop a point-to-point empirical model for the ropes to be used in a finite element program, such as ABAQUS. Three analytical models involving force as a function of displacement and velocity were developed from force, acceleration, and velocity data acquired from the slack-to-taut transition and rebound phase of the drop test, for pre-cycled and new ropes of each type.

1.5 SUMMARY OF LITERATURE REVIEW

The first stage of the literature review was essential in determining current state of design as well as codified design requirements. Specifically, insight into the seismic provisions of general building codes and material specific design codes, allowed for proper application of design principals, and appropriate commentary relating to the building code throughout this study.

The literature provided insight into the state of practice and the development of devices currently being considered for the mitigation of damage to structures due to large seismic events. Strengths and drawbacks relating to such devices in studies reviewed were considered carefully to ascertain the devices which may be considered competitive to rope devices. Criteria for general comparison included reduction in residual and maximum story drift, and increase in base shear as a result of adding devices. The addition of most devices reviewed provided drift reduction at a cost of adding significant base shear, with the exception being fluid-viscous dampers. Although bettering the performance of other devices was not an explicit objective of this study, analytical and experimental results of these devices were considered when evaluating the results presented herein.

Previously conducted studies of synthetic fiber ropes, Pearson (2002) and Hennessey (2003), provided valuable information of the response of different types of synthetic fiber rope. This was essential in quickly determining that the Amsteel II rope was most appropriate for the intended use. The characteristics of the Amsteel II rope, including transition from small initial stiffness to stiffness greater than steel at large strain levels, a return to nearly zero load at zero deformation after extreme loading cycles, and some inherent energy dissipation, compelled the research of the rope devices for the stated purpose. The literature review revealed that rope devices have never been considered for controlling damage to structures due to intense ground motion, while limiting the forces incurred by frame members and the foundation system. Thus, this study was undertaken.