2. Background

2.1 Background of Fluid Viscous Damping

In recent years engineers have developed several approaches to modify dynamic response for the purpose of limiting damage to buildings subjected to earthquake ground motions. These approaches include active control, passive control, and hybrid control. An active control system works by exerting a force on the structure from an external source. In this system, energy can be dissipated and it can also be added to the structure. Passive control systems impart forces that develop in response to the motion of a structure. The passive control devices dissipate energy in the structure, but cannot increase the energy. A hybrid control system is one that incorporates both passive and active devices (Hanson and Soong, 2001).

The current study focuses on fluid viscous dampers, which are used in passive control systems. All structures have the ability to dissipate energy during vibration. This energy dissipation, or inherent damping, can come from several different sources such as internal stressing, rubbing, and cracking. The amount of inherent damping varies between structures. A value of about 5% critical damping us typically used in seismic design. For structures with low inherent damping, it may be advantageous to implement energy dissipation systems. Many different methods have been used to accomplish increased energy dissipation, including use of friction devices, viscoelastic devices, yielding devices, and viscoelastic fluid devices. Viscous fluid damping, which is the method of passive damping utilized in this study, works by transferring the structure's kinetic energy into heat through fluid orificing (Housner et al., 1997).

Fluid viscous dampers were initially used in the military and aerospace industry. They were adapted for use in structural engineering in the late 1980's and early 1990's (Makris and Constantinou, 1990, Constantinou and Symans, 1992). Fluid viscous dampers typically consist of a piston head with orifices contained in a cylinder filled with a highly viscous fluid, usually a compound of silicone or a similar type of oil. Energy is dissipated in the damper by fluid orificing when the piston head moves through the fluid (Hanson and Soong, 2001). The fluid in the cylinder is nearly incompressible, and when the damper is subjected to a compressive force, the fluid volume inside the cylinder is decreased as a result of the piston rod area movement. A decrease in volume results in a restoring force. This force is undesirable and is usually prevented by using a run-through rod that enters the damper, is connected to the piston head, and then passes out the other end of the damper. Another method for preventing the restoring force is to use an accumulator (Symans and Constantinou, 1998). An accumulator works by collecting the volume of fluid that is displaced by the piston rod and storing it in the make-up area. As the rod retreats, a vacuum that has been created will draw the fluid out. A damper with an accumulator is illustrated in Figure 2.1.

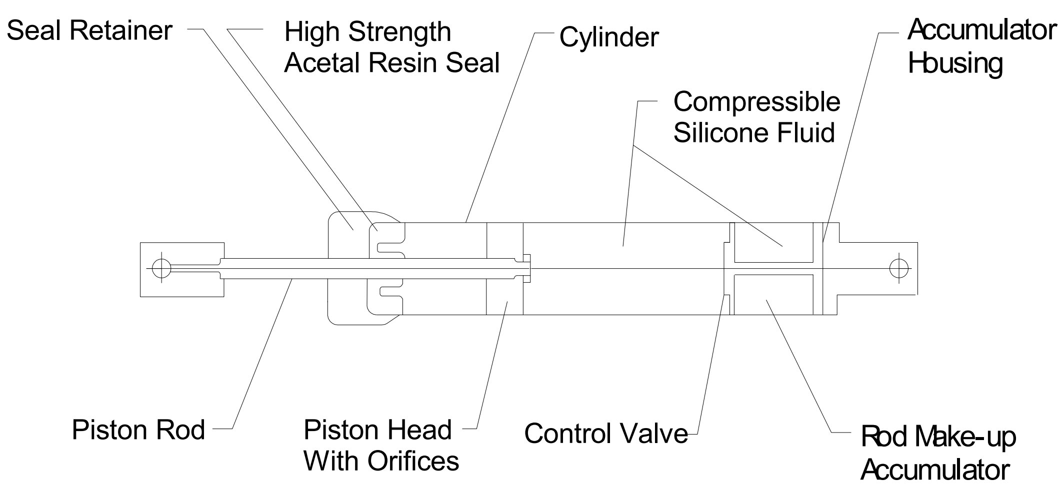

Figure 2.1: Fluid Viscous Damper

Studies by Constantinou and Symans (1993a,b and 1992) were performed to determine the mechanical properties of fluid dampers. The dampers in the experiments were constructed so that temperature of the fluid did not influence the results. To achieve this, the dampers were designed so that the temperature inside the device remained between –40C and 70C. This relatively wide range of temperature is where the mechanical properties were determined to be stable. The dampers in the study were subjected to steady-state harmonic motion at frequencies of 1, 2, and 4 Hz.

The results of these tests showed that when in this range of frequencies, the storage stiffness, K_1 in Equation 2.2, is effectively zero and the damper behavior is virtually linear viscous. However, if the frequency is increased past 4 Hz these dampers begin to exhibit storage stiffness.

Constantinou and Symans (1993a) define storage stiffness in their derivation of the damper force when the damper motion is given by

$$u = u_0 \sin(\omega t) \qquad 2.1$$

where u_0 is the amplitude of the displacement, ω is the loading frequency, and t is time. The resisting force in the device, P, can be described by the following equation

$$P = K_1 u + C\frac{du}{dt} \qquad 2.2$$

where K_1 is the storage stiffness and C is the damping coefficient given by

$$C = \frac{K_2}{\omega} \qquad 2.3$$

where K_2 is the loss stiffness. In Equation 2.2 the first term represents the force due to the stiffness of the damper, which is in phase with the motion, and the second term represents the force due to the viscosity of the damper, which is 90^0 out of phase with the motion. Figure 2.2 (a) plots the force-displacement relationship for the first and second term of Equation 2.2, while Figure 2.2 (b) plots the total force.

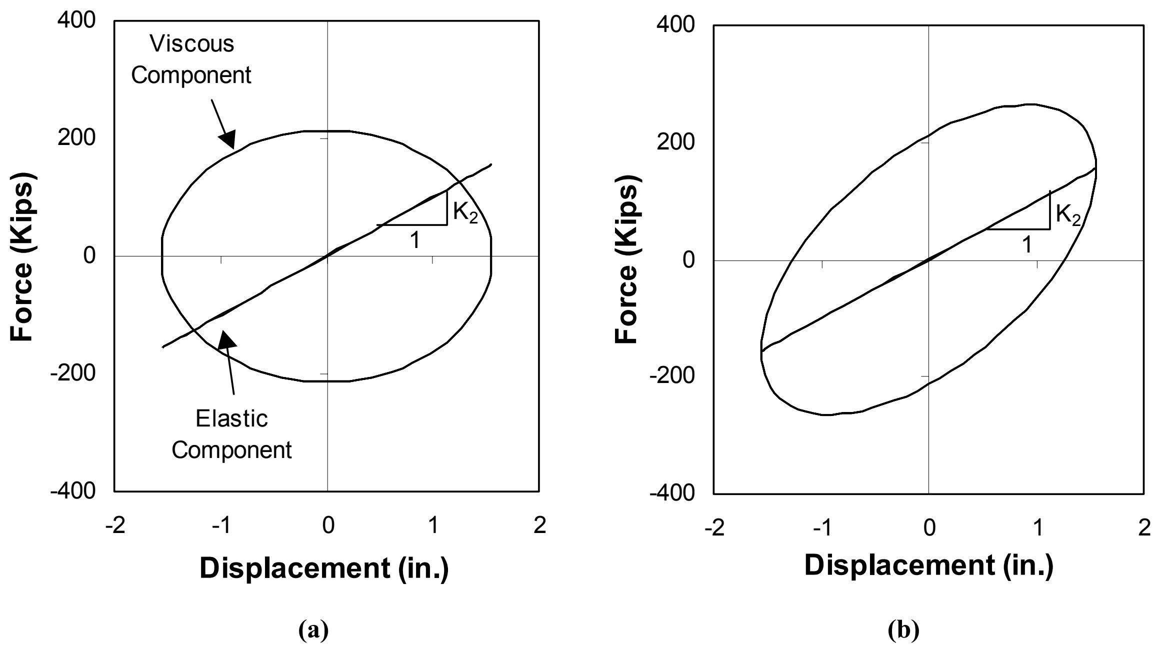

Figure 2.2 (a-b): Force-Displacement Relationship for Viscous Fluid Damper

The loading frequency at which the storage stiffness becomes significant is called the cut-off frequency and can be controlled through design of the damper. The desire of having a cut-off frequency is that if it is greater than the first fundamental frequency, the fundamental mode of vibration will only have added viscous damping, while higher modes will have additional damping and additional stiffness. This can result in the contribution of the higher modes being completely suppressed.

In a study by Constantinou and Symans (1993a), the researchers were also able to conclude that the inclusion of fluid viscous dampers in the experimental structures resulted in reductions in story drifts of 30% to 70%. These reductions are comparable to the reductions found when other damping systems, such as viscoelastic, friction, and yielding steel dampers, are used. However, the use of the fluid dampers was also able to reduce the levels of story shears in the structure by 40% to 70%, while the use of the other damping systems could not result in any significant story shear reductions. Symans and Constantinou (1998) state that the reason for the difference in story shears between fluid dampers and others is the pure linear viscous behavior exhibited in tests, which results in an out-of-phase relationship between the column restoring forces and fluid damper forces. It is Symans and Constantinou's opinion that these findings make fluid viscous dampers the more efficient and effective system. Furthermore, the researchers proceeded to conclude that fluid viscous dampers are more beneficial to use than active control systems because they require less cost and no power, they will last longer, and they are more reliable.

An important note about the Constantinou and Symans experiments is that the structure primarily remained elastic and did not undergo any significant yielding. Later experiments by Miyamoto and Singh (2002) suggest that when a structure behaves inelastically the conclusions about reduction in story shears do not hold.

2.2 Analytical Models

In the current study the dampers were modeled analytically in a steel moment-resisting frame using structural analysis program *RAM Perform 2D* (Powell, 2000). Studies, such as Makris et al. (1993), Makris and Constantinou (1991, 1990), and Constantinou and Symans (1992), were performed to derive an accurate mathematical model of a viscous damper. These studies have also used testing to experimentally determine some of the parameters of this model. The characteristics of the damping systems that are used in this study have all been determined using this model.

Fluid viscous dampers exhibit a viscoelastic behavior. This behavior can be best predicted with the Maxwell model (Bird et al., 1987). Physically the Maxwell model can be described as a spring in series with a dashpot as shown in Figure 2.3.

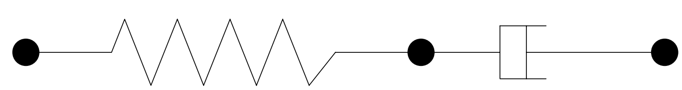

Figure 2.3: Maxwell Model

The model can also be described by the following equation:

$$P(t) + \lambda \frac{dP(t)}{dt} = C_0 \frac{du(t)}{dt} \qquad 2.4$$

where P is the damper output force, λ is the relaxation time, C_0 is the damping constant at zero frequency, and u is the displacement of the piston head with respect to the damper housing. The relaxation time for the damper is defined as

$$\lambda = \frac{C_0}{K_1} \qquad 2.5$$

where C_0 is the damping constant at zero frequency and K_1 is the storage stiffness of the damper at infinite frequency.

Later, Makris and Constantinou (1991) put forth a more general Maxwell model, which can be described by the following equation:

$$P(t) + \lambda \frac{d^r P(t)}{dt^r} = C_0 \frac{d^q u(t)}{dt^q} \qquad 2.6$$

where $\frac{d^r}{dt^r}$ and $\frac{d^q}{dt^q}$ are fractional derivatives of orders r and q, which are based on material properties. Then Makris et al. (1993) examined an even more advanced model of viscoelasticity to study the behavior of fluid dampers. In this model the order of the time derivatives and the coefficients are complex-valued. While the model described by

Equation 2.6 and the model with complex values are more general, it is more practical to use a simplified form.

2.2.1 Linear Fluid Viscous Dampers

The current study focused on both linear and nonlinear fluid viscous damping. The model described by Equation 2.6 can be simplified to obtain a more useful model of linear viscous damping. When $r=q=1$ the model becomes the Maxwell model described by Equation 2.4. The device parameters, λ and C_0, were obtained from experimental tests performed in studies by Constantinou and Symans (1992). If the frequency of vibration is below the cut-off frequency, the second term in Equation 2.6 drops out and the model of the damper can be simplified as

$$P(t) = C_0 \frac{du}{dt} \qquad 2.7$$

where C_0 is independent of the frequency, but dependent on ambient temperature. With this model the damper behaves as a linear viscous dashpot.

As previously stated, C_0 is dependent on ambient temperature. Makris et al. (1998) performed analytical studies addressing the issue of heating of fluid viscous dampers. Their studies concluded that the temperature increase in the fluid for a cycle of steady-state harmonic motion is significant for long-stroke motions. The increase in temperature was found to be proportional to pressure drop and independent of the amplitude of the piston-head velocity. Therefore, if viscous heating is a problem, it can be remedied by reducing the pressure drop in the damper. This can be achieved through the use of piston heads with large diameters. Viscous heating will most likely not be an issue in seismic applications because there may only be a few long strokes for one earthquake.

An important feature of Equation 2.7 is that the damper force is a function of velocity. This is important because if the device is loaded with a sinusoidal function the displacements may be a sine function and then will be out of phase with the velocity and the damper force, which will be a cosine function. This verifies the damping behavior that Constantinou and Symans (1993a) found in earlier experiments. Once again this is beneficial if the structure remains elastic, because the dampers will reduce drifts and shear forces without creating greater column axial forces in combination with the column bending moments (Jinping et al., 1996).

2.2.2 Nonlinear Fluid Viscous Dampers

As stated earlier, the current study used both linear and nonlinear fluid viscous damping. The model represented in Equation 2.7 is specifically for linear fluid viscous damping and therefore needs to be made more general to include nonlinear damping. This generalized form is as follows:

$$P(t) = C_0 \left| \frac{du}{dt} \right|^{\alpha} \operatorname{sgn}\left[\frac{du}{dt} \right] \qquad 2.8$$

where α is a real positive exponent that ranges from 0.1 to 2, and sgn is the signum function. When α is unity, Equation 2.8 reduces to the linear viscous dashpot model described in Equation 2.7.

The force-velocity relationship for dampers with a damping coefficient, C_0, of 50 kip-in./sec and values of α=0.5, 1.0, and 1.5 are illustrated in Figure 2.4. From the plot it can be observed that when α=1.0 the relationship is linear, while the nonlinear relationship varies from $\alpha<1$ to $\alpha>1$. Notice that when $\alpha<1$ the curve has a softening relationship, while when $\alpha>1$ the curve hardens.

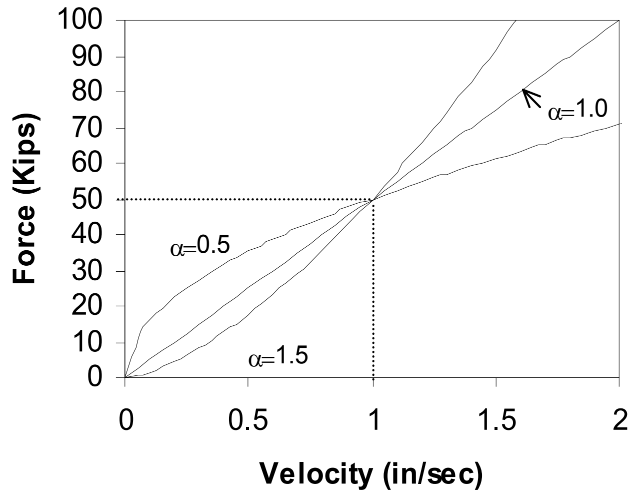

Figure 2.4: Force-Velocity Relationships for Fluid Viscous Dampers

In actual dampers, changing the shape of the orifices in the piston head will vary the damping exponent, α. The use of cylindrical piston head orifices will produce a value of α=2 and the force is proportional to the velocity squared. This type of damping is known as Bernoulli damping. Other values of α can be achieved by altering the shape of the orifices, which will change the characteristics of flow with the fluid speed. According to Symans and Constantinou (1998), Bernoulli damping is unacceptable for seismic energy dissipation due to the very large damper forces that is induced; however, they do suggest that dampers with other values can be useful. They state that a design with a value of α=0.5 can be used for situations when a structure is subjected to extremely high velocity shocks, or near-field earthquakes, because this type of damping limits the peak force in the damper.

2.3 Previous Research

2.3.1 Nonlinear Fluid Viscous Dampers

Symans and Constantinou (1998) attempted to find the optimum damping exponent for the free vibration response of an idealized one-story structure which incorporates a linear or nonlinear fluid viscous damper. In this simple analysis, the single-degree-of-freedom damper systems with α= 0.5, 1.0, and 2.0 were given an initial displacement and then released. From the recorded response histories, it was concluded that the system performed the best when α=0.5, followed by the system with α=1, and the worst performance was when α=2.

A possible shortcoming of these experiments is the method used to determine the damping coefficient for each damper. It was assumed that the dampers were equal if they produced the same maximum force at a certain velocity. By making this assumption, the energy each damper dissipates per cycle will not be equal and the damper that dissipates the most energy per cycle, α=0.5, will yield the best results. For the current study it is considered that it is more useful to compare damping exponents for dampers that dissipate equal amounts of energy per cycle. Therefore, an equal energy method will be used in the calibration of dampers. This method is discussed further in Chapter 3.

The advantage of nonlinear viscous fluid dampers over linear viscous fluid dampers is reduction in base shear. As a study by Miyamoto and Singh (2002) has shown, adding fluid viscous dampers to a structure that behaves inelastically will result in added base shear. However, by using nonlinear dampers, the amount of added base shear can be reduced.

Lin and Chopra (2002) support this notion of reduced base shear in their study of earthquake response of elastic single degree-of-freedom systems with nonlinear fluid viscous dampers. In that study the researchers defined two independent parameters of nonlinear fluid viscous dampers. They are ζ_{SD}, the supplemental damping ratio, and α, the damping exponent, as in Equation 2.8. The supplemental damping ratio ζ_{SD} is the authors' method to characterize the energy dissipation capacity of energy-equivalent nonlinear dampers and is defined with the following equation:

$$\zeta_{SD} = \frac{\beta_\alpha C_\alpha}{2m\omega_n}(\omega_n u_0)^{\alpha-1} \qquad 2.9$$

where m is the mass, ω_n is the natural frequency of the system, u_0 is the amplitude of the harmonic motion, C_α is the damping coefficient, α is the damping exponent, and β_α is a constant given by

$$\beta_\alpha = \frac{2^{2+\alpha}\Gamma^2(1+\alpha/2)}{\pi\Gamma(2+\alpha)} \qquad 2.10$$

where Γ is the gamma function.

From the results of their analytical experiments, Lin and Chopra found that, if ζ was increased and α was held constant, the response of the system (acceleration, velocity, and displacement) could be significantly decreased. Alternately, they found that if α was varied and ζ was fixed, the response of the system did not vary significantly. From these results they were able to conclude that ζ is much better at reducing response than α. However, they did find that if $\alpha<1$ the damper force was greatly reduced. This result strengthens the idea that nonlinear dampers with $\alpha<1$ are more advantageous than linear dampers because the amount of supplemental damping can be increased while limiting the amount of added base shear.

A major consideration regarding the Lin and Chopra (2002) study is that it was limited to elastic systems, and the behavior of systems with dampers may vary as the response changes from elastic to inelastic. This issue has resulted in different conclusions by earlier researchers in regards to damping and base shear forces. Specifically, Constantinou and Symans (1993a) tested structures elastically and found that supplemental damping decreases base shear, while Miyamoto and Singh (2002) analyzed structures inelastically and found the opposite. This problem suggests that any current research should analyze damping systems in a number of structures that undergo a wide range of elastic and inelastic behavior. An analysis procedure that incorporates this ideology is the incremental dynamic analysis, which is discussed further in a later section of this chapter.

2.3.2 Viscous Damping of Structures Subjected to Near-Field Ground Motions

The current study includes consideration for suitability of certain damper types, nonlinear or linear, for buildings subjected to certain earthquake types, near-field or far-field. Damping for near-field earthquakes became an issue in passive control systems in the late 1990's. According to Makris (1997), passive dampers can be successfully used in the mitigation of long-duration excitations that amplify gradually. However, it is more difficult to implement passive control for flexible structures, such as moment-resisting frames, when subjected to near-field ground motions. These motions contain one or two large displacement pulses that range from 1.5 to more than 3 ft with peak velocity of 3 ft/sec or higher. From a study, Makris (1997) concluded that linear viscous damping had little effect in reducing structure response when used alone. The most likely reason for this finding is that linear viscous damping is most effective when the response occurs at the resonant frequency. In near-field earthquakes the response is non-resonant.

Early studies by Filiatrault and his coworkers (Tremblay et al., 1998, Filiatrault and Kremmidas, 1999) attempted to improve structural response, when subjected to near-field ground motions, with the use of friction dampers. The goal of the research was to determine if the damping systems could prevent brittle weld fractures in beam-to-column connections due to excessive curvature ductility demands. The conclusion from these studies was that the friction dampers were ineffective due to the large inelastic rotations that were exhibited in the beams and columns.

A later study by Filiatrault et al. (2001) expanded the previous work to include linear and nonlinear viscous damping. The conclusion of this study was that the viscous dampers reduced acceleration levels better than the friction systems. However, they were ineffective because the braces used to connect them to the structure yielded in tension or would have undergone inelastic buckling, thus limiting the viscous damping system. As a result, the viscous systems by themselves would not prevent the fracture of beam-to-column joints. As discussed in the introduction, there are two possible shortcomings to this study. The first is that the study was limited to only using dampers with damping exponents less than unity. It was expected for the current study that nonlinear dampers with an exponent greater than unity would be the best at reducing structural response. The other shortcoming is that the dampers were deemed ineffective because the damper braces yielded. This conclusion seems to be off because the brace yielding is most likely a result of poor design rather than ineffective damping.

2.4 Incremental Dynamic Analysis

Nonlinear dynamic analysis was used in the current study in order to obtain the most complete understanding of a building response when it is subjected to an earthquake. Presently, computers have the processing power to efficiently perform several analyses in a reasonable amount of time. As discussed in an earlier section, when studying damping in structural systems, there is a demand to include structural behavior that ranges from elastic to inelastic. This can be achieved through the use of the incremental dynamic analysis (IDA) procedure.

According to Vamvatsikos and Cornell (2002), the first concept of IDA was developed by Bertero in 1977. In more recent years, different forms of IDA have been used in several studies. An IDA procedure that uses incremental dynamic analysis curves was originally developed by Cornell and coworkers (Luco and Cornell, 1998) for the SAC project [1]. This procedure has been a popular one and has been used repeatedly in studies by several different researchers, including Bazzuro and Cornell (1994a, 1994b), Lee and Foutch (2002), and Mehanny and Deierlein (2000).

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[1] www.sacsteel.org

As described in Lee and Foutch (2002), this IDA method requires a structural model to be subjected to a selected ground motion acceleration record that is scaled several times so that the level of earthquake intensity increases incrementally. The first three or four records are scaled to a level of intensity low enough that the structure will most likely remain elastic. From these analyses, an elastic baseline of response may be obtained. These are followed by further time history analyses which will be scaled so that they elicit inelastic response. Each analysis will produce a performance measure, such as maximum interstory drift angle or a damage index.

The results of the analyses can then be plotted in curves that relate scale factor for the earthquake record to the response, such as drift of the structure. The global building capacity is reached when displacements become excessive and the structure becomes unstable. If drift is used as a measure of performance, the structure becomes unstable when the drift increases drastically for a relatively small increase in ground motion acceleration or when the equivalent stiffness of the building decreases to less than 20% of the slope of the elastic baseline. An example of an IDA curve in Figure 2.5 highlights the stiffness of the elastic baseline and equivalent stiffness between S_a=1.3 g and 1.4 g where g is equal to the acceleration of gravity. A maximum global capacity is usually determined so that the structure does not become unstable. A reasonable value will be chosen for the capacity. When drift is used, Lee and Foutch (2002) suggest a drift limit of 0.1 is used.

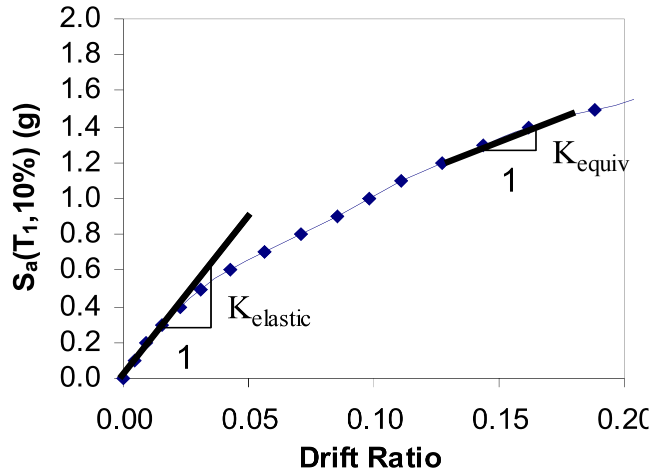

Figure 2.5: IDA Curve for Single Ground Motion Record

According to Vamvatsikos and Cornell (2002), incremental dynamic analysis can be performed for a single ground motion, but a single record cannot give a complete picture of the behavior which a building might experience in a future earthquake. To capture the full range of possible response, the IDA should include a full suite of ground motion records. Then multiple IDA curves can be developed and when displayed together will give a plot similar to Figure 2.6.

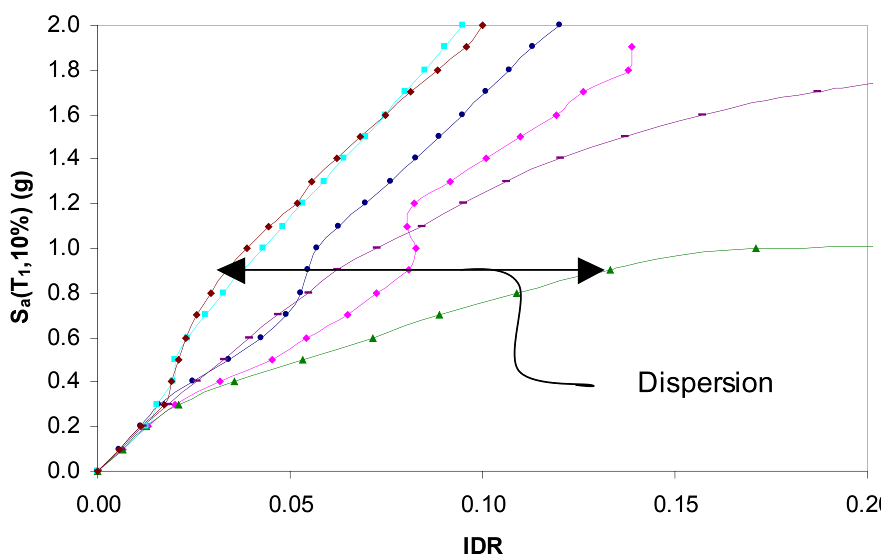

Figure 2.6: Multi-Record IDA Curve

An important characteristic of the multi-record IDA curve to observe is that if all the ground motions are scaled in an identical manner, then their response will be relatively uniform up to a certain level of intensity. After this point, the yield point, there is much variation between the responses. In the current study the variation in IDA curves is referred to as dispersion. This dispersion may represent the randomness of the different earthquakes, i.e., for each earthquake there is a different sequence of peak accelerations. The difference between the magnitudes of peaks, the rate of peak occurrence, and the time of peak occurrence for each earthquake may result in the variation of the sequence by which the structural members yield. One yielding sequence may cause a vital member to fail sooner than in others; as a result, the response for one earthquake may be greater than the response for another earthquake at the same intensity. The differences in the response will then cause dispersion in the IDA curves.

Another possible source of dispersion in the results is residual displacements. Residual displacements occur when the structure experiences permanent deformation at the end of the earthquake. The relationship between IDA dispersion and residual displacements is explained further in section 2.6.

If engineers are able to decrease the amount of dispersion in the response of a structure, then they will have more confidence in that structure to perform at a certain level of earthquake intensity. Therefore, this study focuses on a better understanding of the cause of the dispersion and determining whether a certain type of damping system is best suited to decrease the amount of dispersion.

While the ultimate goal of the IDA procedure is to determine a building's capacity to resist total collapse, Vamvatsikos and Cornell (2002) have identified the following as more specific objectives of an IDA study:

1. Thorough understanding of the range of response or ‘demands’ versus the range of potential levels of ground motion record
2. Better understanding of the structural implications of rarer/more severe ground motion levels
3. Better understanding of the changes in the nature of the structural response as the intensity of the ground motion increases
4. Producing estimates of the dynamic capacity of the global structural system
5. Understanding of how behavior changes from one ground motion to another

The ability of the IDA procedure to meet these objectives makes it a good procedure to use in this study, because the main objective of this study is comprehensive observations of structural response with different damping systems. This procedure allows for determination of the best damping system for any reasonable ground motion intensity.

2.5 Damage Indices

The IDA design procedure is performance-based and the main goal of performance-based design is to build structures that have predictable behavior for a specified range of intensity of ground motions (Mehanny and Deierlein, 2000). There are two important criteria for the performance of earthquake-resistance structures: the structure does not collapse or experience excessive damage from the design earthquake, and damage is limited for earthquakes that occur more frequently than the design earthquake. To assess whether or not a structure meets the criteria, the performance needs to be measured qualitatively. There are several different damage indices that can be utilized in order to achieve this assessment (Mehanny and Deierlein, 2000).

2.5.1 Park and Ang Damage Index

A damage index developed by Park and Ang (1985) was one of the indices used in this study. This index has been used extensively among researchers, mainly because it includes both ductility and dissipated energy by means of a linear combination. The ductility term of the model accounts for the peak deformations, while the dissipated energy term takes care of the cumulative deformations. This index is a favorable choice due to its simplicity. The Park and Ang index is presented in the following form:

$$DI_{PA} = \frac{u_{\max}}{u_{mon}} + \frac{\beta E_H}{F_y u_{mon}} \qquad 2.11$$

where u_{mon} is the maximum deformation capacity of the system under a monotonically increasing lateral deformation, $u_{\max}$ is the maximum deformation, E_H is the irrecoverable hysteretic energy, F_y is the yield strength of the system, and β is a calibration constant. According to Sorace (1998), studies conducted on wide-flange steel specimens gave a mean value of β=0.025.

The values of DI_{PA} range from 0 to infinity where for values of zero the response is elastic and values greater than or equal to one signify total collapse (Park and Ang, 1985). Fajfar (1992) goes on to say that values of DI_{PA}<0.4 represents damage that is repairable, where DI_{PA}>0.4 represents damage beyond repair.

In the current study, the Park and Ang damage index, DI_{PA}, was only applied to the girder hinge in the structural model as the calibration constant, β, is not available for the panel zones and columns. In order to assess the performance of the whole steel moment-resisting frame, the value of each local DI_{PA} needs to be translated to the global scale.

One possible method to achieve this is to use the maximum value; however, a drawback is that if one girder fails it does not necessarily mean that the whole structure will fail. Another possibility is to use a weighted average. The idea is to assign a weight factor to each inelastic element and produce a DI for each story, and then each story is assigned a weight that will be used to find an average for the entire structure. The weighting factors used may involve energy dissipated by a member or the tributary gravity load supported by the member. According to Mehanny and Deierlein (2000) these two approaches give more weight to the lower stories, which is conceptually correct. However, a drawback to both weighting methods is that they fail to identify soft stories, which can also cause failure of an entire structure.

Regardless of the drawback, this study used a weighted average, and the method used for determining the weighing factors is the amount of energy dissipated by each member. The following are the weighted average indices for each story and for the entire structure; a few of the past researchers that have used this weighted average index are Park et al. (1985), Chung et al. (1987), and Kunnath et al. (1992).

$$DI_{story} = \frac{\sum_i DI_i E_i}{\sum_i E_i} \qquad 2.12$$

where DI_{story} is the story damage index, DI_i is the Park and Ang damage index for each member, and E_i is the amount of energy dissipated by member i.

$$DI_{structure} = \frac{\sum_i DI_i^{story} E_i^{story}}{\sum_i E_i^{story}} \qquad 2.13$$

where $DI_{structure}$ is the story damage index, DI_i^{story} is the Park and Ang damage index for each story, and E_i^{story} is the amount of energy dissipated by story i.

2.5.2 Peak Interstory Drift Angle

Peak interstory drift angle has been selected as another damage measure that was used in the current study. Peak interstory drift ratio is the relative displacement between top and bottom of the story normalized by the story height. In this current study, flexible steel moment-resisting frames are subjected to near-field earthquakes and therefore a damage index that characterizes the expected damage is desirable. Hall et al. (1995) performed a study of flexible buildings subjected to near-field ground motions and found that the susceptibility of welded connections to fracture in steel moment-resisting frames increases the possibility of total collapse. In a summary of the SAC case study building analyses, Deierlein (1998) states that in taller buildings, interstory drift ratios correlate well to location of connection failures. Therefore, it seems appropriate to use interstory drift ratio as a damage measure.

2.5.3 Peak Base Shear

As seen in previously discussed studies, such as Constantinou and Symans (1993a), Miyamoto and Singh (2002), and Lin and Chopra (2002), adding fluid viscous dampers to a structure affects the amount of base shear present. An objective of the current study was to determine how different types of dampers can improve structural response in terms of drift and damage, but in doing so it must realize what those improvements costs in terms of base shear. Some of the improvements may be unrealistic because no structure could be economically designed to resist the large forces developed.

2.6 Residual Displacements

Earlier in the chapter residual displacements were suggested as a possible cause for the dispersion observed in the IDA multi-record curves. A study of inelastic systems by Veletsos and Hall (1964) showed that residual displacements can have an effect on the maximum inelastic response of a structure. In that study inelastic response spectra were created for elasto-plastic systems with 2% critical damping for the 1940 El Centro Earthquake. Veletsos and Hall observed that when the periods were between about 0.5 and 3.33 sec the inelastic displacements were very close to the linear elastic displacement for all the different ductility factors. This means that in the IDA plots, when the ductility demand increases with increasing earthquake intensity, the displacements should increase linearly. However, Veletsos and Hall also found that the equal displacement relationship did not hold for systems with smaller periods. They attributed this to systems with smaller periods being more susceptible to residual displacements, which can increase or decrease the total displacement. Therefore, it seems reasonable that the variations of the inelastic responses that cause the dispersion in the IDA plots is also caused by residual displacements (Newmark and Hall, 1982).

Mahin and Bertero (1981) describe another inelastic design method that replaces an inelastic system by an equivalent linear elastic system. The method uses the linear elastic design spectrum directly to determine the inelastic displacements for an earthquake. The equivalent linear elastic system represents the inelastic system with an effective stiffness and some amount of viscous damping. To replace the inelastic system, the effective stiffness and viscous damping coefficients are derived on the basis of a steady state hysteretic loop. According to Mahin and Bertero, the linear system can then be adjusted by certain factors to account for actual ground motions not producing steady state response. If this method works, then, as earthquake intensity increases, the response of the inelastic system, modeled as a linear elastic system, should increase linearly. However, as shown in Figure 2.6, the inelastic responses to most ground motions are not linear.

The reason that the equivalent linear system method does not work is again most likely due to residual displacements. Jennings (1968) performed one of the first studies on equivalent linear elastic systems. In that study, Jennings found difficulties applying the concept when observing response to earthquake motions instead of harmonic motions. It was determined that the cause of these difficulties was residual displacements.

Two different methods of inelastic analysis have been presented. Each suggests that residual displacements are the reason that the relationship between earthquake intensity and displacement is not linear, which is a cause for dispersion in the drift IDA plots.

Therefore, the current study observed when residual displacements occur. Furthermore, the study investigated whether reductions of residual displacement results in reduced IDA dispersion and improved structural resistance to severe damage.

MacRae and Kawashima (1997) have studied residual displacements in their investigation of bilinear oscillators. In that study they found that the parameters of the oscillator's force-displacement relationship affected the amount of residual displacements observed at the end of an earthquake. The least amount was found for the system with positive post-yield stiffness, followed by an elastic-plastic system, and the worst response occurred for the system with negative post-yield stiffness. The authors reason that when the oscillator is at its maximum displacement, the system with the positive post-yield stiffness has the capability to force the system back to the original position of equilibrium.

Before the current study was performed it was expected that a system using nonlinear dampers with α=1.5 is the best for reducing residual displacements, followed by α=1.0 and 0.5. This was expected because as the velocity increases the α=1.5 dampers will pick up a large percentage of the base shear in the structure, which reduces the tendency of the moment-resisting frame to yield.