SIGNIFICANCE OF ACCELERATION IN THE DESIGN OF DAMPERS FOR LIFELINE BUILDINGS IN SEISMIC GROUND MOTIONS

Arun M Puthanpurayil[1] and Dr. Paul Reynolds[2]

[1] *Structural Engineer, Gifford, UK*
[2] *Senior Lecturer, Dept. of Civil and Structural Engineering, University of Sheffield, UK*
Email: Arun.Mankavu@gifford.uk.com, p.reynolds@sheffield.ac.uk

ABSTRACT:

This paper describes an analytical study carried out to investigate the significance of the floor acceleration parameter in the design of viscous dampers aimed at reducing nonstructural damage in lifeline buildings (those buildings that need to be functional after the earthquake, e.g. Hospitals) during a seismic ground motion. A comparative quantitative assessment is carried out between the minimum drift criterion and minimum acceleration criterion for both near field and far field ground motions. The study reveals that significant anomalies are observed if the proportioning of the damper is carried out solely using the minimum drift based criterion, which is more common in practice. Both linear dampers and nonlinear dampers are used for the study; linear dampers for low rise structures and nonlinear dampers for medium and high rise structures. Linear time history analyses were performed for low rise structures equipped with linear dampers and nonlinear modal time history analyses were employed for medium and high rise structures with nonlinear dampers. Both far field and near field ground motions are used for the study.

KEYWORDS: Linear Dampers, Nonlinear dampers, Linear Time History analysis, nonlinear modal Time History Analysis

1. INTRODUCTION

Lifeline buildings in a broader sense can be described as those buildings which need to be fully functional after an earthquake. These include hospitals, fire stations, electrical control substations etc. which need to be functional in order to carry out a successful rescue operation. Hospitals for example equipped with a lot of highly sensitive equipment need to care for a large number of injured people immediately after the earthquake. So the seismic designs of such buildings need to cater for not only the parent structure, but also the contents of the building, so as to render it fully functional. This means seismic design of such structures should reduce both structural and nonstructural damage to the minimum possible extent.

Based on the type of possible damage, nonstructural components can be classified as deformation sensitive or acceleration sensitive. Deformation sensitive nonstructural elements would include the architectural claddings, partitions etc. and acceleration sensitive equipments would include generators, heavy machines rigidly fixed to the floor, archives and normal building contents such as computing devices and shelving. For a lifeline building to be fully operational in emergency situations all these components should be fully functional and intact.

At present this is achieved by the innovative application of mechanical devices in the primary structural system which tend to reduce the detrimental response of the system during a seismic motion. A recent study shows that there is considerable reduction in the seismic responses characterizing nonstructural damage of RC frames when viscous dampers are added to the system (Arun, 2007). A comparative study carried out for the low, medium and high rise structures showed that; linear dampers resulted in considerable reduction in seismic response of low rise structure while nonlinear dampers were required for medium and high rise categories (Arun, 2007). Currently in the design process of viscous dampers for nonstructural damage mitigation, it is a common practice to give more significance to displacement sensitivity than to acceleration sensitivity. This approach can be justified on the basis that most of the costly nonstructural components of the buildings are displacement/deformation-sensitive (Whittaker & Soong, 2003). However this approach would not suffice for lifeline buildings such as hospitals which houses a lot of highly acceleration sensitive contents. This paper emphasizes the significance of acceleration in the design of dampers for lifeline buildings. A set of response curves are presented based on control quantities (drift and acceleration) and a comparative study is carried out on the effect of the damper 'sizing' parameters on these control quantities. The study is carried out for all the three categories of structures i.e. low rise, medium rise and high rise buildings subjected to both near field and far field ground motions.

2. GROUND MOTIONS USED IN THIS STUDY

A set of real earthquake time histories were generated using the software Seismosignal developed by Seismosoft. The time histories were classified as Near Field Earthquake (NFE) and Far Field Earthquake (FFE) based on their frequency content. The details of the time histories are given in table 2.1.

Table 2.1 Frequency content and Peak ground acceleration of the earthquakes used

Earthquakes	Predominant Frequency Content (Hz)	Peak Ground acceleration (g)
Sakaria (NFE)	0.3-9.0	0.556
Loma Priea Corrailitos (NFE)	2.0-3.5	0.689
Chi-Chi Longitudinal (FFE)	1.6-1.7	0.776
Loma Prieta Emeriville (FFE)	0.6-1.0	0.2498

3. METHODS OF ANALYSIS USED IN THE STUDY

Linear viscous dampers are analyzed using linear time history analysis and frames with nonlinear viscous dampers are analyzed using nonlinear modal time history analysis (Fast Nonlinear Analysis method).

3.1 Linear Time History Analysis

Linear time history analysis is an incremental time stepping procedure for evaluating the dynamic response of a structure subjected to an arbitrary loading that may vary with time (Hart & Wong, 2000). The method involves a solution of the complete set of equilibrium equations at each time increment Δt. The dynamic equilibrium equation for a system is given by Eqn.3.1.

$$M\ddot{x} + C\dot{x} + Kx = f(t) \qquad (3.1)$$

Where M is the mass matrix, K is the stiffness matrix, C is the damping matrix. $\ddot{x}$, $\dot{x}$ and x are acceleration, velocity and displacement respectively of the structural system and $f(t)$ is the external load vector. Qualitatively, equation (3.1) can be looked upon as derived from static equilibrium considerations at time *t*. Therefore in short dynamic analysis can be viewed as a static analysis at time t with inertia and damping force included (Bathe & Wilson, 1976). Here in order to solve the equation (3.1), direct integration technique is used. Direct numerical integration attempts to satisfy dynamic equilibrium in specific discrete points in time. The numerical integration is carried out by Hilbur, Hughes and Taylor α method which is a modification of the classic Newmark time integration scheme.

3.2 Nonlinear Modal Time History Analysis (FNA Method)

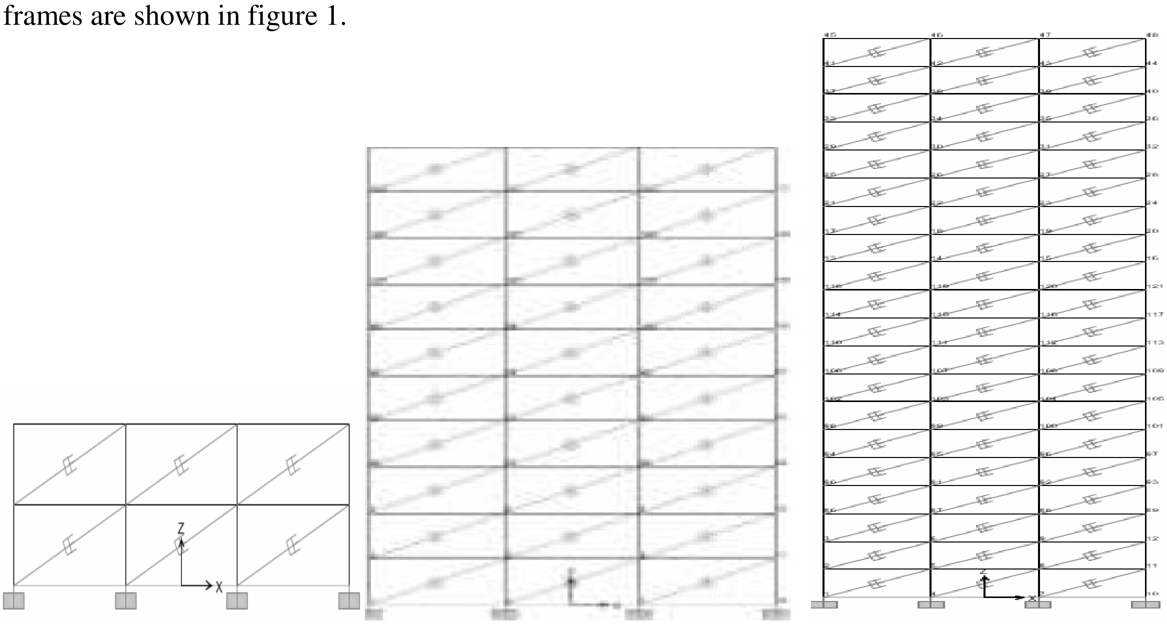
Nonlinear modal time history analysis is carried out using SAP 2000. This method is especially suitable for analysis of structures which are primarily elastic, but have some nonlinear elements intended for undergoing large localized strains (Wilson, 2002). The force equilibrium of a computer model at a time **t** with diagonal non-linear elements incorporated is given by Eqn. 3.2(Wilson, 2002).

$$M\ \ddot{x}(t) + C\,\dot{x}(t) + K\,x(t) + f_{NL}(t) = -f(t) \qquad (3.2)$$

Where M is the diagonal mass matrix, K is the stiffness matrix, C is the proportional damping matrix. $\ddot{x}(t)$, $\dot{x}(t)$ and $x(t)$ are acceleration, velocity and displacement respectively of the structural system and $f(t)$ is the external load vector. The $f_{NL}(t)$ is the global node force vector for the sum of the forces in the nonlinear elements. Here the elastic stiffness matrix K neglects the stiffness of the diagonal nonlinear elements. The mode shapes are determined by Ritz-Vector analysis. Modal analysis by Ritz-vector method spans in the spatial distribution of loading but approximates its frequency content. Ritz vectors are basically constructed from the loading applied (SAP Manual, 2002).

4. DESCRIPTION OF SUPPLEMENTALLY DAMPED STRUCTURAL SYSTEM

The structures analyzed are three reinforced concrete frames; a two storey frame representing the low rise structure, a ten storey frame representing the medium rise structure and a twenty storey frame representing the high rise structure. The frames are designed as OMRF (ordinary moment resisting frame) with no special requirement for ductility. The cross sections are proportioned by considering only the gravity load effects. This is done in order to assess the maximum possible drift and peak acceleration that can happen in the structure. The frames are designed to take a live load of 4kN/m^2. The frames are assumed to be part of a building with a typical grid spacing of 7.5 m x 7.5 m. The dead loads include the weights of all services, a 250 mm thick slab throughout the plan area in each floor and down stand beams of size 300mm x 700mm. The dampers are arranged such that one damper runs diagonally in each bay in every floor. The damper is assumed to be connected to the centre line of the frames. The damper is assumed to be fitted to the structure in such a way that it does not add additional stiffness to the structure but only provides the damping. The analytical models of the frames are shown in figure 1.



(a) (b) (c)

Figure 1 (a) Arrangement of dampers in two storey frame (b) Arrangement of dampers in ten storey frame (c) Arrangement of dampers in twenty storey frame

5. OBSERVATIONS AND DISCUSSIONS:

This section outlines the observations made in the study and also critically draws a qualitative outlook on the effect of acceleration on the damper 'sizing' process. The results are presented in the form of response curves illustrating the effect of the 'sizing' parameters on the control quantities (drift and acceleration) and a comparative study is carried out. As stated earlier linear dampers are used for low rise structure whereas nonlinear dampers are used for medium and high rise structures. The curves for the low rise structure depict the variation of control parameters with respect to damping coefficient *'c'*. In the case of medium and high rise structures each curve illustrates the effect of the velocity exponent η on control parameters for a constant damping coefficient *'c'*. The *'c'* value studied for linear dampers range from 500-2000 kN-sec/m and for nonlinear dampers it ranges from 5000-8000 kN-sec/m. Though higher *'c'* values are available in practice, the limits have been chosen by taking a judicious view considering the economic aspect. An increase in the 'c' value always increases the size and cost of the damper (Taylor, 1999). The η value for linear dampers is unity and for the nonlinear dampers, it ranged from 0.3-1.0 (Kelly, 2001).

5.1 Observations for Near Field ground motions:

A total of 64 time history analyses using near field ground motions are carried out for the development of the response curves presented in figures 2-4.

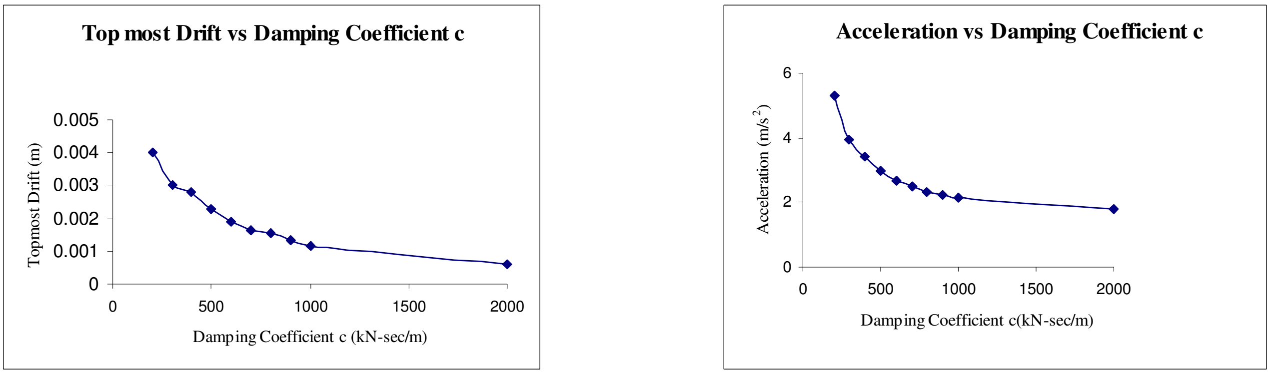


Figure 2 Low Rise Structure (Two Storey Frame)

Figure 2 above depicts the variation of drift and floor accelerations with respect to damper 'sizing' parameters. For low rise structures employing linear dampers, a similar trend is followed for both drift and acceleration; Results suggest that, although there are variations in the optimum sizing parameter to be used, in practical terms there is no significant difference between using either the minimum drift or minimum acceleration as the basis for selection. In short for the given range of *'c'* under consideration it is easy to qualitatively arrive at 'optimum sizing parameters' by giving due consideration for both the control criterions i.e. drift and acceleration.

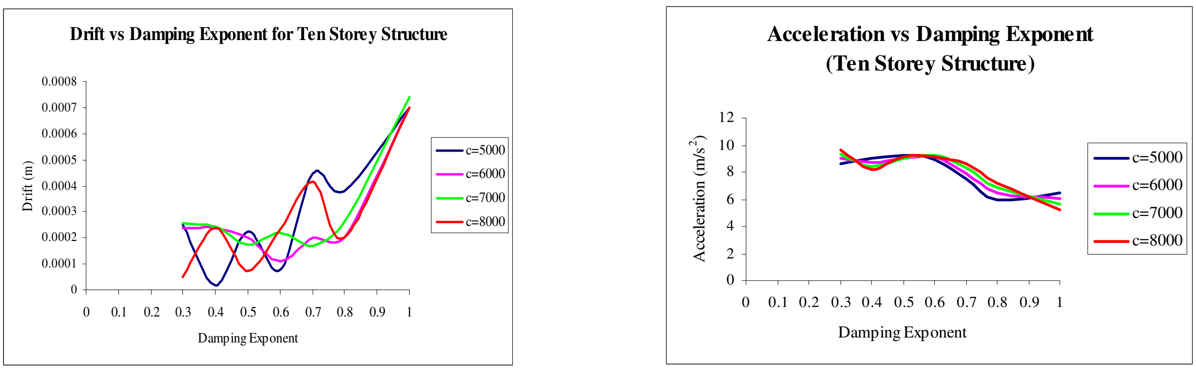


Figure 3 Medium Rise Structure (Ten Storey Frame)

Figure 3 illustrates the variation of control quantities (drift and acceleration) with respect to damper sizing parameters for medium rise structures. Whilst the drift curve obtained is very random in nature with alternating crusts and troughs; On the contrary acceleration curve tends to follow a uniform pattern. It should be noted that if drift alone is taken as the control parameter for damper proportioning, it tends to increase the acceleration. To exemplify this, the value of damping coefficient and velocity exponent for minimum drift is $c \cong 5000$ for $\eta \cong 0.4$; but looking at the acceleration response curves we find that this range is very detrimental for the acceleration criterion as we find the system experiencing peak values and the minimum value for floor acceleration occurs at $c \cong 5000$ and $\eta \cong 0.8$. So this might prove very critical for buildings hosting acceleration sensitive equipments.

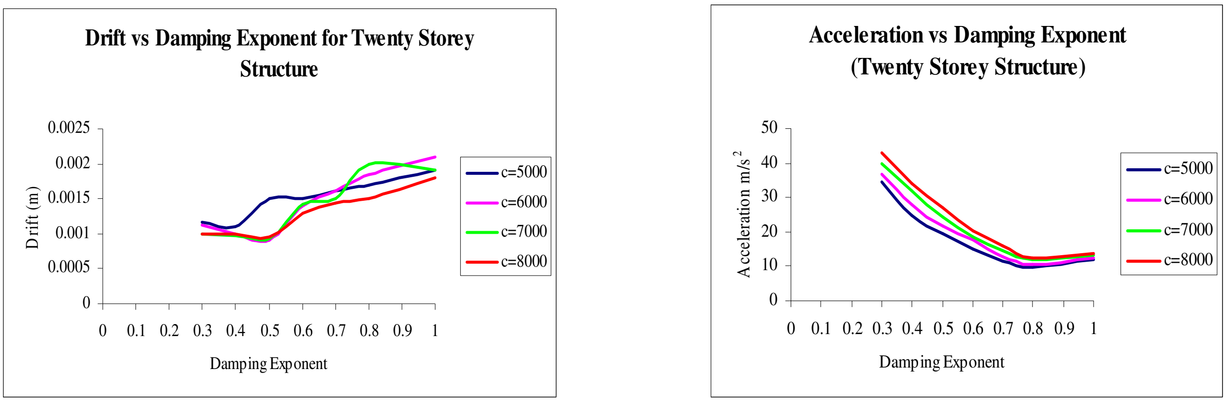


Figure 4 High Rise Structure (Twenty Storey Frame)

Figure 4 presents the relationship of control parameters with respect to damper sizing parameters for high rise structures. The drift curves obtained in general have positive slope, whereas the acceleration curves exhibit a negative slope giving an absolute minimum zone for η in the range of 0.75-0.8 after which it changes the slope. Here also the trend exhibited in the medium rise structure is replicated but with a higher intensity. For example, the optimum range of damping coefficient and velocity exponent for minimum drift design is $c \cong 6000-8000$ for $\eta \cong 0.35-0.55$; this range is very detrimental for minimum acceleration criterion as we find the system experiencing peak values. So this might prove very critical for buildings hosting acceleration sensitive equipments.

5.2 Observations for Far Field ground motions:

A total of 64 time history analyses using far field ground motions are carried out for the development of the response curves presented in figures 5-7.

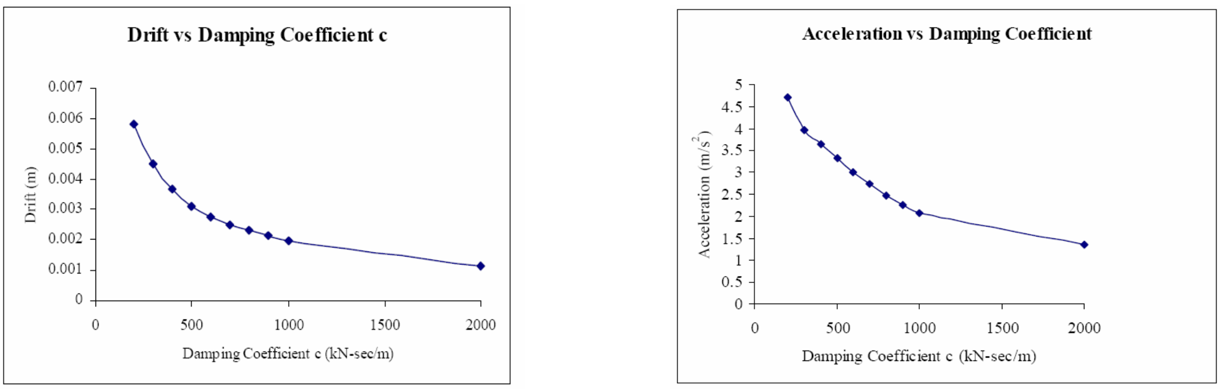


Figure 5 Low Rise Structure (Two Storey Frame)

Figure 5 above depicts the variation of drift and floor accelerations with respect to damper 'sizing' parameters for low rise structures subjected to far field ground motions. Here again the trend exhibited as in the case of near field vibration is repeated, but with a reduced intensity. Both the curves have the same nature of slope qualitatively; though a strict quantification of the parameters would suggest a slight variation in the optimum selection of the sizing parameters based on either of the above two described design criterions. As with the low rise study for near field ground motions, the results suggest that, although there are variations in the optimum sizing parameter to be used, in practical terms there is no significant difference between using either drift or acceleration as the basis for selection. In short the response curves indicate that it is easy to qualitatively arrive at 'optimum sizing parameters' by giving due consideration for both the control criterions.

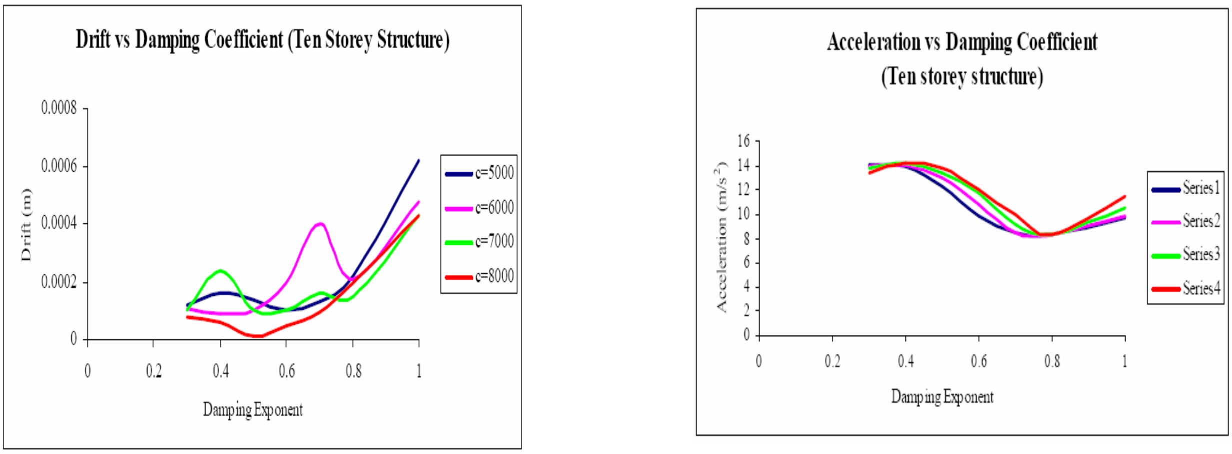


Figure 6 Medium Rise Structure (Ten Storey Frame)

Figure 6 illustrates the variation of control parameters (drift and acceleration) with respect to damper sizing parameters for medium rise structures. The drift curves obtained are very random in nature with prominent alternating troughs and crests; this is in contrast to the acceleration curves which tend to follow a uniform pattern with a negative slope giving an absolute minimum at $\eta \cong 0.8 - 0.85$ and after which the slope changes the nature showing an ascending trend. But evidently here also it is very explicit that if minimum drift solely is taken as the control parameter, it tends to increase the acceleration. For example, the optimum range of damping coefficient and velocity exponent for minimum drift base criterion is $c \cong 7000 - 8000$ for $\eta \cong 0.4 - 0.6$; but this is very critical for the acceleration criterion as we can find that the system experiencing peak values at 0.4 and then decreases slowly when it proceeds to 0.6. So this range might prove very critical for buildings hosting acceleration sensitive equipments.

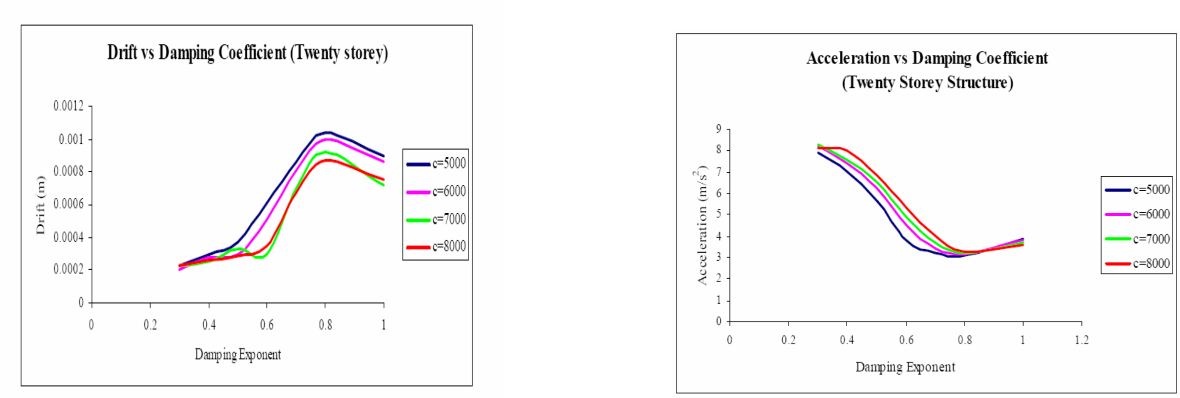


Figure 7 High Rise Structure (Twenty Storey Frame)

Figure 7 illustrates the variation of control quantities with respect to damper sizing parameters for high rise structures. The drift curves obtained in general have positive steep slope up to a specific point at which they drop down, whereas the acceleration curves exhibit a general negative slope. Here also the trend exhibited in the medium rise structure is repeated but with a higher intensity. For example, the optimum range of damping coefficient and velocity exponent for drift base design is $c \cong 6000-8000$ for $\eta \cong 0.3-0.4$; but for acceleration consideration this range of sizing parameters exhibits peak values.

5.3 Discussions:

The observations made in both near field and far field ground motions emphasizes the significance of floor acceleration as an important control parameter in the design of dampers for lifeline buildings. The effect of floor acceleration is found to be more prominent for medium and high rise structures employing nonlinear dampers as compared to low rise structures employing linear dampers. This can be attributed to the fact that the force in the viscous damper is out of phase with the displacement of the frame and maximum acceleration would occur when the force response reaches a maximum. Medium and High rise structures experience larger drift due to the inherent flexibility of the frame. So if minimum drift criterion is given more significance as compared to minimum acceleration in damper 'sizing' process, then it would call for larger force in the damper. This force would induce higher acceleration in the parent structure. So if drift is considered as the sole control parameter there is a high chance that the system might be vulnerable to the initiation of large accelerations. Although quantitatively the results presented in the study are specific to the analyzed structural system, they pose no limitation to qualitatively understanding the significance of floor accelerations in damper design process.

6. CONCLUSION

The effect of acceleration sensitivity in the damper design process is investigated in detail and the results are presented in the form of response curves. A comparative assessment of the effect of the damper 'sizing' parameters on drift and acceleration are presented. The presented results qualitatively illustrate the significance of acceleration in the damper design process for lifeline buildings. Discussions on the observations are also included.

REFERENCE

Arun, M.P. (2007), Optimal Viscous Dampers for Reduction of Non-structural Damage in Seismic Loading, MSc. Thesis, Sheffield University, U.K.

Whittaker, A.S and Soong, T.T. (2003). An overview of nonstructural components research at three US earthquake engineering research centers. *seismic design, Performance and Retrofit of Non-Structural components in Critical facilities* ATC: 29-2 proceedings, 271-280.

Hart, C.G. and Wong, K. (2000), Structural Dynamics for Structural Engineers, John Wiley & Sons. Inc, New York.

Bathe, K. J. and Wilson, E. L. (1976), Numerical Methods in Finite Element Analysis, Prentice-Hall, Inc., New Jersey.

Wilson, E.L. (2002), Three-Dimensional Static and Dynamic Analysis of Structures, Computers and Structures Inc., Berkeley, U.S.A.

SAP 2000 Analysis Reference Manual.(2002), Computers and Structures Inc., Berkeley, U.S.A.

Taylor, D.P. (1999), Buildings: Design for Damping, Taylor Devices Inc., U.S.A

Kelly, E.T. (2001), In-structure Damping and Energy Dissipation: Design Guidelines, Holmes Consulting Group Ltd., Wellington, New Zealand.