Influence of damping models on nonlinear seismic response of a base isolated building

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ABSTRACT

This paper discusses the effect of different damping models on nonlinear seismic response of a 8 storey 2D base isolated (BI) building: (1) Rayleigh damping proportional to the mass and initial stiffness matrices, (2) Rayleigh damping proportional to the mass and effective stiffness matrices, (3) mass proportional damping, (4) stiffness proportional damping using initial stiffness matrices (5) stiffness proportional damping using effective stiffness matrices, (6) constant modal damping through direct integration analysis and the dynamic nonlinear modal analysis method (also known as fast nonlinear analysis (FNA)) and (7) constant modal damping with 0% first-mode damping override.

The results of nonlinear response history analyses using each method is compared for an 8-story base isolated building model assuming linear elastic behaviour of the superstructure. This study shows that selecting an improper viscous damping model in base isolated buildings can lead to the introduction of an artificial viscous damping and underestimating the isolators displacement demands. Based on response results, we conclude that the classical Rayleigh damping model and stiffness proportional damping model using initial stiffness matrices and constant modal damping for all periods are inappropriate. The two preferred damping models are (1) a modified Rayleigh damping model in which the stiffness-proportional term is based on effective stiffness matrices; and (2) a damping matrix defined by superposition of modal damping matrices with a 0% first-mode damping override. It is also found that by considering modal damping, the results of analysis are almost identical for both nonlinear modal analysis and direct integration analysis.

1 INTRODUCTION

The numerical solution of the equations of motion in structural dynamics requires assembling a damping matrix in addition to the standard mass matrix and a stiffness matrix which could be incremental. The standard equations that are solved in nonlinear response history analysis (RHS) are:

$$m\ddot{u} + c\dot{u} + f_s(u) = -mt_x\ddot{u}_{gx}(t) - mt_y\ddot{u}_{gy}(t) - mt_z\ddot{u}_{gz}(t) + P_{gr} \quad (1)$$

where u is the vector of degrees of freedom (DOFs); m and c are the mass and damping matrices, respectively; the vector $f_s(u)$ represents the nonlinear relation between resisting forces and deformations, which includes both material and geometric nonlinearities. (For linear systems $f_s = ku$, where k is the stiffness matrix.) The right side represents the dynamic excitation: $\ddot{u}_{gx}(t), \ddot{u}_{gy}(t), and\ \ddot{u}_{gz}(t)$ in the co-ordinates x-, y- , and vertical components of earthquake ground acceleration, t_x , t_y , and t_z are the corresponding influence vectors, and P_{gr} represents gravity loads.

While in the linear case, except for the conditions at resonance, the effects of damping are typically small. In the inelastic case the details of the inherent damping model have a significant effect on the structural response and, particularly, on the calculated damping forces.

Modelling viscous damping is a challenging task for base-isolated buildings which consist of two subsystems viz. the isolation system and the superstructure, with substantially different energy dissipation properties. It is logical to prescribe viscous damping separately for the isolation system and the superstructure, where the use of effective viscous damping in the isolation system can be avoided by using hysteretic models of bearings to account for all the energy dissipation.

A number of previous studies have illustrated that the traditional use of inherent damping (eg, the Rayleigh approach) in BI buildings can lead to the introduction of an artificial viscous damping to the isolated (first) mode, thus underestimating the first-mode structural responses such as isolators displacement demands. This phenomenon is termed as ‘damping leakage’ in literature.

A few previous studies have proposed solutions to mitigate the problems associated with modelling viscous damping in BI buildings. For example, for BI buildings with a linear isolation system, Ryan and Polanco (2008) showed that using the stiffness-proportional damping approach and computing βk based on the fundamental mode of the fixed-base superstructure would result in a reasonable specification of damping to the isolated mode. For nonlinear isolation systems, Hall (2006) and Pant et al (2013) proposed using variants of the stiffness-proportional damping approach in which the damping matrix was developed based on the post-elastic stiffness of isolators. Kitayama and Constantinou (2018) suggested a solution for cases in which the global damping matrix was constructed based on modal damping ratios. In this method, zero damping was specified to the first mode, a constant non-zero damping to the other modes, and the global stiffness of the system was modified to implement the desired stiffness of the isolators for computing the global damping matrix. The aforementioned works were mostly focused on first-mode dominated responses such as isolator displacement. Only the studies conducted by Pant et al (2013) and Dao and Ryan (2013) in addition to first-mode responses, considered higher mode responses such as short-period floor spectral acceleration responses. These two studies were based on comparisons between experimental data and responses from numerical models. For example, Dao and Ryan (2013) developed numerical models for a full-scale five-story steel moment frame building that was isolated by triple pendulum bearings. They conducted response history analyses under two ground motion records and concluded that using the Rayleigh damping approach along with supplemental damping in the first structural mode resulted in a reasonable match with the experimental data. The lack of studies addressing the effect of modelling viscous damping on the higher-mode responses of BI buildings might be due to the notion that in BI buildings the contribution of higher modes is not significant as their mass participation is relatively low. For example, Chopra (2012) stated that ‘the higher modes [of BI buildings] are essentially not excited by the ground motion—although their [ground] pseudo-accelerations are large—because their modal static responses are very small'. However, as discussed in the early works of Kelly and Tsai (1985) and Tsai and Kelly (1988) the coupling between the equations of motion in BI buildings can lead to the presence of significant higher-mode dominated responses.

A number of different modelling options involving Rayleigh, Caughey and modal viscous damping matrices based on initial or tangent properties have been proposed and warnings about unintended consequences of these choices have been voiced for the last 30 years by a number of authors ((Crisp 1980), (Shing and Mahin 1987), (Leger and Dussault 1992), (Bernal 1994), (Carr 1997, 2005 ,2007), (Hall 2006), (Ryan and Polanco 2008), (Charney 2008), (Petrini et al 2008), (Zareian and Medina 2010), (Smyrou et al 2011), (Jehel et al 2014), (Chopra and McKenna 2015), (Pant et al. 2013)).

The objective of this paper is to investigate the results of using the different damping models, including models based on: (i) initial structural properties, (ii) degraded properties (effective stiffness) on the seismic response of base-isolated structures. In each case, two different estimates of the damping matrix are considered: (1) Rayleigh damping including and excluding the mass proportional term, and (2) modal damping matrix. The effects of different viscous damping models are quantified by numerical inelastic time-history analyses of a multi-story structure subjected to an earthquake excitation. Results in the form of maximum isolators displacements, base shear, and energy dissipated by hysteretic action and inherent damping are presented.

2 NONLINEAR DIRECT INTEGRATION VERSUS FAST NONLINEAR ANALYSIS

Nonlinear direct integration (NLDI) and nonlinear modal time-history analysis, also known as fast nonlinear analysis (FNA), are widely used for the response history analysis of BI buildings. The NLDI approach solves the complete set of equilibrium equations at every time step. In this approach, if a system becomes nonlinear, it may be necessary to reassemble the stiffness matrix for the complete structural system at each time step. Also, iterations may be required within each time increment to achieve convergence and satisfy equilibrium. As such this method is computationally intensive. In the FNA approach, element nonlinear responses are treated as unbalanced forces that are grouped with external loads. This method reduces the large set of global equilibrium equations to a relatively small number of uncoupled second order differential equations. As a result, FNA has the advantage of much faster computational time as compared with NLDI.

With the latest version of the commonly used commercial software ETABS, for both analysis methods, modal damping can be specified which gives the user the ability to assign the global values of viscous damping for all modes, and, at the same time, damping ratios can be manually overridden for a number of modes. Therefore, the damping leakage phenomenon can be readily prevented by specifying a 0% damping to the isolated mode, whereas non-zero damping values are assigned to the other modes. Previously this was only available for FNA.

3 CASE STUDY

The case study is an eight-story 2D steel framed office building located in Wellington, NZ with a seismic coefficient derived from a Hazard Factor (Z) of 0.4 and soil type C. The model configuration is shown in Figure 1. Total seismic weight of 11500kN is considered. The properties of the example building and base isolator are shown in Table 1 and Table 2. Triple pendulum bearings have been used in this study and the isolation system force displacement behaviour are shown in Figure 6.

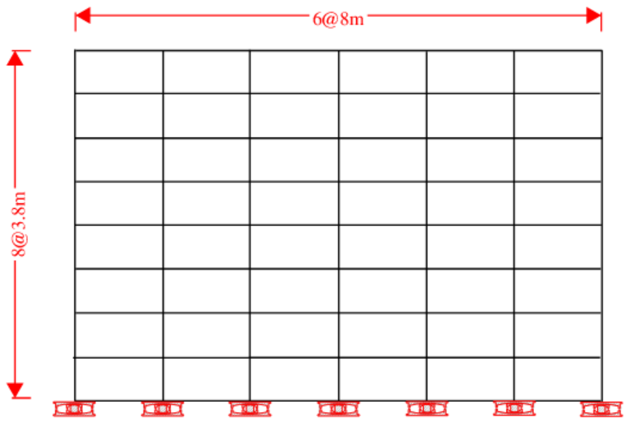

Figure 1: Case Study Model

Table 1: Properties of Example Building

Items	**Properties**
Inter-storey height [m]	3.8m
Bay length [m]	8m
Seismic Weight [kN]	11500 kN
Column dimensions [mm]	950CHS20_ConcFilled
Beam dimension [mm]	800HCB466

Table 2: Properties of Base Isolators

	Outer Top	Outer Bottom	Inner Top/Bottom	
Lower Bound Friction Properties				
Stiffness	0.068 W/0.05"	0.053 W/0.05"	0.055 W/0.05"	kip/in
Friction Coefficient, Slow	0.073	0.063	0.027	
Friction Coefficient, Fast	0.110	0.095	0.040	
Upper Bound Friction Properties				
Stiffness	0.068 W/0.05"	0.053 W/0.05"	0.055 W/0.05"	
Friction Coefficient, Slow	0.090	0.080	0.047	
Friction Coefficient, Fast	0.135	0.120	0.070	
Rate Parameter	1.25	1.25	1.25	sec/in
Radius of Sliding Surface	7.33	7.33	2.33	ft
Stop Distance	24.24	24.24	100.00	in.

Properties based on values provided by Earthquake Protection Systems (EPS)

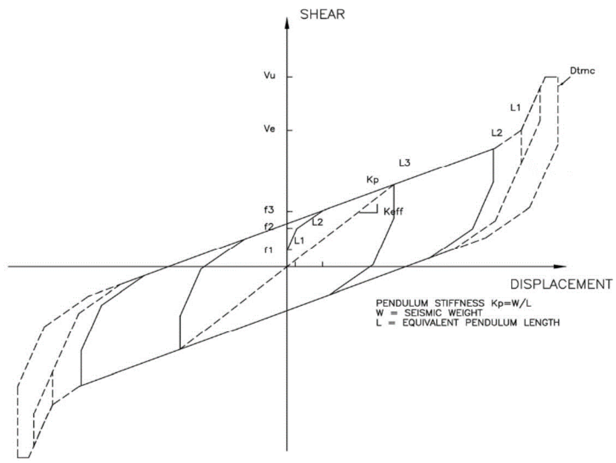

Figure 2: Isolation System Force-Displacement Behaviour

4 MODELLING VISCOUS DAMPING

In this paper, different types of global linear viscous damping models used for nonlinear time history analyses (NLTHA) are considered:

1. Mass and initial stiffness proportional Rayleigh damping, which will be referred to as R_I damping. The damping matrix is a linear combination of the mass and initial stiffness matrices M and K_0, respectively. In the below equation ζ is the damping ratio, ω_n is the circular frequency of the mode and α and β are the mass and stiffness proportional coefficients respectively.

$$C = \alpha M + \beta K_0 \tag{2}$$

where

$$\alpha = {}^{2\zeta\omega_i\omega_n}/_{(\omega_i + \omega_n)} \tag{3}$$

And

$$\beta = {}^{2\zeta}/_{(\omega_i + \omega_n)} \tag{4}$$

2. Mass and effective stiffness proportional Rayleigh damping, which will be referred to as R_{eff} damping. The damping matrix is a linear combination of the mass and effective stiffness matrices M and K_e, respectively.

$$C = \alpha M + \beta K_e \tag{5}$$

The coefficients α and β are computed based on the effective stiffness and are constant throughout the analysis. Studies (Charney 2008, Pant et al. 2013) have considered the case where coefficients α and β are recomputed every time step based on the tangent stiffness. However, this requires a modal analysis at every time step, an expensive and time consuming operation when used for realistic structural models.

3. Mass proportional damping defines as $C = \alpha_0 M$ where $\alpha_0 = 2\zeta\omega_i$ and stiffness-proportional damping defines as $C = \beta_0 K$ where $\beta_0 = {}^{2\zeta}/_{\omega_i}$ can be viewed as special case of Rayleigh damping.

The implementation of both the mass proportional and stiffness-proportional damping models requires assigning a specific superstructure damping ratio to a single mode (generally the first mode), whereas, in the Rayleigh model, specific damping ratios should be assigned to two selected structural modes of vibration.

4. Modal damping. In this method, the damping is equivalent to that used in linear modal time-history analysis. A major advantage of this method is that the modal damping ratio, ζ_n, can be specified for each mode independently. In the modal space, the modal damping matrix is diagonal. The resulting damping matrix for direct integration time history analysis is (Chopra & McKenna 2015, Wilson & Penzien 1972):

$$C = M^T \left(\sum_{n=1}^{N} \frac{2\zeta_n \omega_n}{M_n} \varphi_n \varphi_n^T \right) M \qquad (6)$$

where $\varphi = [\varphi_1, \varphi_2, \dots, \varphi_N]$ is a set of N mode shapes computed for the structure; and M_n, ζ_n, and ω_n are the modal mass, damping ratio, and circular frequency for mode n, respectively. Note that N may be less than the total number of modes for the structure.

5 SYSTEM AND GROUND MOTION

All response results are presented for a variant of an 8-story moment-resisting steel frame base isolated building to qualitatively assess and compare the different damping methods for use in a realistic structural model. It is assumed the superstructure (steel frame) is going to remain elastic and all seismic load is going to be dissipated by base isolations.

All structural modelling and analysis was completed using the commercial software (ETABS CSI v18.1, 2020,) using modal analysis and direct-integration time history analysis. Modal damping ratios are assumed to be 5%. This value is assigned to the first and fourth modes to determine the terms α and β in Rayleigh damping. The modal damping is analysed using FNA with 50 modes generated using accelerations in the X-direction, gravity loading, and link forces as starting vectors for Ritz modal vector generation. 5% damping is applied in all modes with the exception of a 0% first-mode damping override. All response results presented are for the ground motion defined by the El Centro record from the 1940 Imperial Valley earthquake.

6 INFLUENCE OF DAMPING MODEL ON RESPONSE

Table 3 compares the results in the form of peak isolator displacements, base shear, and energy dissipated by hysteretic action and inherent damping.

It shows that selecting stiffness proportional damping model using initial stiffness matrices in a base isolated building can lead to a significant artificial internal damping and should be avoided. Selecting the classical Rayleigh damping model and constant modal damping for all periods in a base isolated building can cause damping leakage problem and an underestimation of the isolators displacement demands.

Considering effective stiffness matrix for both Rayleigh damping and stiffness proportional damping models reduces the damping leakage problem significantly. Also, for modal damping by specifying 0% damping to the isolated modes, the damping leakage issue can be readily prevented. This leads to anincreases in the percentage of the energy dissipation via hysteretic action of base isolation devices which is preferable.

In terms of maximum base shear, all damping models record similar results except stiffness proportional damping with initial stiffness matrix. Using this model, as most of the energy is dissipated by inherent damping not base isolation, the base shear is increased significantly.

It is also found that by considering modal damping, the results of the analysis are almost identical for both nonlinear modal analysis and direct integration analysis.

Table 3: Influence of damping model on nonlinear seismic response

Damping Model	Stiffness Matrix	Max Isolator Displacement (mm)	Max Base Shear (kN)	%Energy dissipated by hysteretic action	%Energy dissipated by inherent damping
(1) Rayleigh damping	initial stiffness	67	2661	76%	21%
(2) Rayleigh damping	effective stiffness	74	2626	84%	13%
(3) Mass prop.	-	74	2650	86%	11%
(4) Stiffness prop.	initial stiffness	21	3950	20%	77%
(5) Stiffness prop.	effective stiffness	71	2665	74%	25%
(6) Modal damping	-	73	2661	82%	16%
(7) Modal damping with 0% 1^{st}-mode	-	79	2638	93%	6%

7 CONCLUSION

This investigation of modelling viscous damping in nonlinear response history analysis of a building with base isolations has led to the following conclusions:

1. Using of the classical Rayleigh damping model and stiffness proportional damping model considering initial stiffness matrices and constant modal damping for all periods in a base isolated building can lead to the introduction of an artificial viscous damping to the isolated modes underestimating the isolated-modes structural responses, such as isolators displacement demands.
2. The two preferred damping models are (i) a modified Rayleigh damping model in which the stiffness-proportional term is based on effective stiffness matrices; and (ii) a damping matrix defined by superposition of modal damping matrices with 0% first-mode damping override.
3. By considering modal damping, the results of analysis are almost identical for both nonlinear modal analysis and direct integration analysis.

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