The Use of Bracing Dampers in Steel Buildings under Seismic Loading

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ABSTRACT

This study focuses on the modeling of manufactured damper when used in steel buildings. The main aim of the manufactured dampers is to protect the steel buildings from the damaging effects that may result due to earthquakes by introducing an extra damping in addition to the traditional damping.

Only Pure Manufactured Dampers, has been considered in this study. Viscous modeling of damping is generally preferred in structural engineering as it leads to a linear model then it has been used during this study to simulate the behavior of the Pure Manufactured Damper.

After definition of structural parameters of a manufactured damper (its stiffness and its damping) it can be used as a structural element that can be added to a mathematical model of the structure. As the damping of manufactured dampers is generally greater than the damping of traditional materials, then the resulting damping matrix for the whole structure will be classified as a nonclassical damping.

As most of literature on earthquake engineering have been written in terms of terminology related to mode superposition method and as this method is applicable to classical damping only. Then, this study tried to check the accuracy of the mode superposition method when applied to a structure with manufactured dampers. In this checking, approximated results of mode superposition method have been compared with more accurate results of direct integration method. From this comparison, it has been noted that the mode superposition method has different levels of accuracy depending on the relation between the fundamental frequency of the structure and the dominate frequency of the earthmotion. If the frequency of the structure is approaching to a dominate frequency of the earthmotion, then the damping effect will be important and the difference between the direct integration method and the model superposition method is increasing and vice versa.

الخلاصة

هذه الدراسة تهتم بنمذجة دعامة الاخماد الصناعي في البنايات الفولاذية المتعددة الطوابق.

المفهوم ببساطة يتعلق بحماية البناية من التأثير المؤذي للهزة الارضية باستخدام دعامة الاخماد الصناعي بدلا من الدعامات التقليدية. درس في هذا البحث نوع من الدعامات الصناعية وهي (Pure Manufactured Damper)

يفضل عادة استخدام النمذجة المرنة للإخماد في الهندسة الانشائية لأنها نمذجة خطية. تبدأ هذه الدراسة باستخدام النموذج المرن لمحاكاة اداء (Pure Manufactured Dampers) بعد ان تم تعريف الجساءة والاخماد لدعامة الاخماد الصناعي كعنصر انشائي، تم اضافته الى الانموذج الرياضي للبناية. بما ان اخماد الدعامات الصناعية بصورة عامة أكبر من اخماد الدعامات التقليدية فأن هذا يجعل مصفوفة اخماد المنشأ الكلي تصنف على انها ذات اخماد غير تقليدي.

هذه الدراسة تدرس اداء طريقة (Mode Superposition) والتي تستخدم في اغلب المصادر الخاصة بهندسة الهزات الارضية لتحليل الابنية ذات الخمود التقليدي. حيث تم تقصي دقة هذه الطريقة عند استخدامها لدراسة منشأ ذو دعامات اخماد صناعية (غير تقليدي). من خلال مقارنة النتائج التقريبية لهذه الطريقة مع النتائج الدقيقة لطريقة التكامل المباشر تم التوصل الى ان نتائج هذه الطريقة تكون بمستويات مختلفة من الدقة اعتمادا على العلاقة بين التردد الرئيسي للمنشأ وترددات الهزة الارضية. فعند اقتراب أحد ترددات الهزة الارضية من تردد المنشأ فأن تأثير دعامة الاخماد الصناعي يكون اشد ويكون الفرق بين نتائج طريقة التكامل المباشر وطريقة (Mode Superposition) في تزايد والعكس صحيح.

INTRODUCTION:

In the design of most buildings, the primary loads that must be considered are those due to the gravity. These loads are always present and consequently must be resisted throughout the life of the building. Typically, the variation with time is slow as compared with the characteristic times of the structure. As a result, a static idealization is quite appropriate. Furthermore, the magnitudes can be readily determined based on self-weight and occupancy requirements. This combination of factors greatly simplifies building design, and, in fact, allowed the ancestors to design and construct impressive structures prior to the development of rational scientific principles. The simplicity of the problem permits the use of a trial-and-error approach to design, particularly if one is not unduly constrained by material and labor costs (**Soong and Dargush 1997**)

In the recent time, resources are often severely limited. Efficient designs must be sought. Additionally, protection from environmental forces, including winds, waves, and earthquakes, which are neither static nor unidirectional are demanded. For these types of loads inertial effects become important, resulting in dynamic amplification and cyclic response. As compared to gravity loads, the magnitudes are also much more difficult to predict, since the temporal and spatial scales of these phenomena are much smaller.

However, by considering the actual dynamic nature of environmental disturbances, more dramatic improvements can be realized. As a result of this dynamical point of view, new and innovative concepts of structural protection have been advanced and are at various stages of development. Modern structural protective systems can be divided into three groups as shown in Table 1. These groups can be distinguished by examining the approaches employed to manage the energy associated with transient environmental events.

Table 1 Structural Protective Systems (Soong and Dargush 1997)

Seismic Isolation	Passive Energy Dissipation	Semi-active and Active Control
Elastomeric Bearings	Metallic Dampers	Active Bracing Systems
Lead Rubber Bearings	Friction Dampers	Active Mass Dampers
Sliding Friction Pendulum	Viscoelastic Dampers	Variable Stiffness or Damping Systems
	Viscous Fluid Dampers	Smart Materials
	Tuned Mass Dampers	
	Tuned Liquid Dampers	

MODELING OF VISCOUS FLUID DAMPERS AS A STRUCTURAL ELEMENT:

There are two approaches to derive structural characteristic of manufactured dampers (i.e., stiffness and damping). The first approach is based on macroscopic point of view. Where in this point of view, the stiffness is defined based on the slope of the diagonal line of the hysteretic loop and the damping is derived from the hysteretic loop of tested damper. Whereas the second approach is based on a microscopic point of view. Interaction between the different parts of the manufactured damper is used to define the stiffness and damping. First approach had been used in this study to define the structural properties for the manufactured dampers.

MODAL SUPERPOSITION METHOD: BASIC CONCEPTS:

The generalized eigenvalue problem associated with the undamped free vibration of MDOF structure is considered. That is

$$\left(\hat{k} - \omega_0^2 \hat{M}\right)\phi = 0 \tag{1}$$

where ω_o represents an undamped natural frequency of the structure including passive elements and ϕ is the associated mode shape vector. The present undamped system will have N such natural frequencies and mode shapes labeled ω_{oi}, and ϕ_i, respectively, for i = 1, 2, . . . , N. Usually, the natural frequencies are ordered by increasing numerical value, with the lowest (ω_{o1}) referred to as the fundamental frequency. Additionally, the mode shapes satisfy the following orthogonality conditions

$$\phi_i^T \widehat{M} \phi_j = \begin{cases} 1 & i = j \\ 0 & i \neq j \end{cases} \quad (2)$$

$$\phi_i^T \widehat{K} \phi_j = \begin{cases} \omega_{0i}^2 & i = j \\ 0 & i \neq j \end{cases} \quad (3)$$

and form a complete set spanning the N-dimensional vector space. Consequently, this set provides the basis for a suitable transformation that can be applied to the original system defined by Equation (4)

$$\widehat{M}\ddot{x} + \hat{C}\dot{x} + \widehat{K}x = -\widehat{M}\ddot{x}_g \quad (4)$$

In Equations (2) and (3), superscript T indicates vector or matrix transpose.

There are numerous methods that available to solve the generalized eigenvalue problem defined in Equation (1). The choice depends largely on the size and structure of the matrices $\widehat{M}$ and $\widehat{K}$. The SAP computer program contains efficient and robust eigenvalue extraction routines that require little user intervention. Routines are also available in the public domain through the MATLAB implementation. (**Hanselman and Littlefield 1995**)

For notational convenience, the natural frequencies are placed in a diagonal matrix ω_o. The corresponding mode shape vectors form the columns of a square matrix $\boldsymbol{\phi}$, which functions as the transformation matrix. Thus, any relative displacement vector x can be represented by

$$x = \phi y \quad (5)$$

where y is the vector of modal (or normal) coordinates. Utilizing Equation (5), along with Equations (2) and (3), in Equation. (4) leads to the following equations of motion expressed in terms of the modal coordinates,

$$\ddot{y} + \boldsymbol{\Phi}^T \hat{C} \boldsymbol{\Phi} \dot{y} + \omega_0^2 y = -\ddot{y}_g \quad (6)$$

where

$$\ddot{y}_g = \boldsymbol{\Phi}^T \hat{M} \ddot{x}_g \quad (7)$$

In general, Equation (6) still represents a coupled set of ordinary differential equations. The equations uncouple only when the term $\Phi^T \hat{C} \Phi$ is also a diagonal matrix. This occurs for the case of proportional (or Rayleigh) damping, in which

$$\hat{C} = \alpha_0 \widehat{M} + \alpha_1 \widehat{K} \quad (8)$$

contains scalar constants α_0 and α_1. From Equations (2) (3) and (8), one obtains

$$\boldsymbol{\Phi}^T \hat{C} \boldsymbol{\Phi} = \alpha_0 I + \alpha_1 \omega_0^2 \quad (9)$$

which is diagonal. The form of $\hat{C}$ can actually be generalized to the Caughey series

$$\hat{C} = \widehat{M} \sum_{j=0}^{N-1} \alpha_j \left[\widehat{M}^{-1}\widehat{K}\right]^j \quad (10)$$

while still permitting diagonalization. Equation (10) is seldom used to compute $\hat{C}$ for given set of α_j. Instead, modal viscous damping ratios ζ_i are specified, such that

$$\boldsymbol{\Phi}^T \hat{C} \boldsymbol{\Phi} = 2\zeta\omega_0 \quad (11)$$

with ζ representing a diagonal matrix containing ζ_i. values With this assumption, Equation. (6) becomes

$$\ddot{y} + 2\zeta\omega_0\dot{y} + \omega_0^2 y = -\ddot{y}_g \quad (12)$$

Since the equations are now uncoupled, a scalar equation, for each mode i can be written, as

$$\ddot{y}_i + 2\zeta_i\omega_{0i}\dot{y}_i + \omega_{0i}^2 y_i = -\ddot{y}_{gi} \quad (13)$$

Equation (13) has the same form as the SDOF system, consequently, all of the methodology and behavioral patterns of SDOF are directly applicable to Equation (13). The solution of the original problem expressed in

Equation (4) is greatly simplified. Once Equation (12) is solved, the relative displacement vector $\mathbf{x}$ can be determined at any time via the transformation Equation (5).

The major computational task in this whole process is the determination of the natural frequencies and mode shapes. Even this task is not as onerous as it first appears, since for most building systems only a small percentage of the N modes actually participate significantly in the system response. As a result, only the structural modes within a certain frequency range need be calculated.

The price paid for this simplicity is the initial restriction to system matrices with constant coefficients, and the further constraint on the damping matrix specified in Equation (10). If the latter condition is relaxed, it still may be advantageous to use a modal approach as an approximate technique for the following reasons:

- Building systems according to most of building codes (e.g. UBC and IBC) are defined in terms of Fundamental Time Period “T”, and system Damping Ratio ζ and the response of buildings systems in theses codes are expressed in terms of Response Spectra. These terminology (T, ζ, and Response Spectra) all have meanings only with modal superposition method.
- The physical interpretation of system response to dynamic loads can easily be visualized with mode superposition method as compared with other solution methods.
- It is often still possible to utilize a mode shape set much smaller than N, since typically only a small portion of the undamped modes will be excited.

DIRECT TIME DOMAIN ANALYSIS BASIC CONCEPTS:

In the direct integration method, Equation (4**)** is integrated by using a numerical step by step procedure. In essence, the direct numerical integration is based on two basic ideas (**Bathe 1996**):

- First, instead of trying to satisfy Equation (4**)** at any time t, it is aimed to be satisfied only at discrete time intervals Δt apart.
- The second idea on which the direct integration method is based on that a variation of acceleration within the time interval Δt is to be assumed.

It is the form of the assumption on the variation of displacements, velocities, and accelerations within each time interval that determines the accuracy, stability and cost of the solution procedure. Based on this assumption, different integration schemes have been developed. Table 2 below represents a brief comparison between common integration schemes:

Table: 2. Comparison between Different Types of Direct Integration Schemes (Bathe 1996):

Integration Scheme		Explicit or Implicit	Stability	Special Starting Procedure
1.	Central Difference Method	Explicit	Conditionally	Required
2.	Houbolt Method	Implicit	Unconditionally	Not Required
3.	Wilson θ Method	Implicit	Unconditionally	Not Required
4.	Newmark Method	Implicit	Unconditionally	Not Required

Based on the above Table, Newmark method seems to be one of the most efficient integration schemes. Then this scheme will be used in this study. Pseudo code of this method is summarized as follows:

INITIAL CALCULATIONS:

1. Assemble stiffness matrix **k**, mass matrix **m**, and damping matrix **c**.

2. Initialize $\mathrm{u}(0), \dot{\mathrm{u}}(0),$ and $\ddot{\mathrm{u}}(0)$, where $\mathrm{u}(0), \dot{\mathrm{u}}(0),$ and $\ddot{\mathrm{u}}(0)$ are vectors of initial displacement, initial velocity and initial acceleration respectively.

3. Select time step Δt and parameters α and δ and calculate integration constants:

$$\delta \geq 0.5 \qquad \alpha \geq 0.25(0.5+\delta)^2 \qquad (14a)$$

$$a_0 = \frac{1}{\alpha \Delta t^2}, \quad a_1 = \frac{\delta}{\alpha\,\Delta t}, \qquad (14b)$$

$$a_2 = \frac{1}{\alpha\,\Delta t}, \quad a_3 = \frac{1}{2\alpha} - 1, \qquad (14c)$$

$$a_4 = \frac{\delta}{\alpha} - 1, \quad a_5 = \frac{\Delta t}{2}\left(\frac{\delta}{\alpha} - 2\right), \qquad (14d)$$

$$a_6 = \Delta t(1-\delta), \quad a_7 = \delta \Delta t \qquad (14e)$$

4. Form the effective stiffness matrix $\hat{\mathbf{k}}$:

$$\widehat{\boldsymbol{K}} = \boldsymbol{K} + a_0\boldsymbol{m} + a_1\boldsymbol{c} \qquad (15)$$

5. Triangularize $\widehat{\boldsymbol{K}}$:

$$\widehat{\boldsymbol{K}}^{-1} = \boldsymbol{Invers\,of K} \qquad (16)$$

FOR EACH TIME STEP:

1. Calculate the effective load at time $t+\Delta t$:

$$\widehat{\boldsymbol{R}}^{t+\Delta t} = \boldsymbol{R}^{t+\Delta t} + \boldsymbol{m}(a_0\boldsymbol{u}^t + a_2\dot{\boldsymbol{u}}^t + a_3\ddot{\boldsymbol{u}}^t) + \boldsymbol{c}(a_1\boldsymbol{u}^t + a_4\dot{\boldsymbol{u}}^t + a_5\ddot{\boldsymbol{u}}^t) \qquad (17)$$

2. Solve for the displacements at time $t+\Delta t$

$$\boldsymbol{u}^{t+\Delta t} = \widehat{\boldsymbol{K}}^{-1}\,\widehat{\boldsymbol{R}}^{t+\Delta t} \qquad (18)$$

3. Calculate the accelerations and velocities at time $t+\Delta t$:

$$\ddot{\boldsymbol{u}}^{t+\Delta t} = a_0\left(\boldsymbol{u}^{t+\Delta t} - \boldsymbol{u}^t\right) - a_2\dot{\boldsymbol{u}}^t - a_3\ddot{\boldsymbol{u}}^t \qquad (19)$$

$$\dot{\boldsymbol{u}}^{t+\Delta t} = \dot{\boldsymbol{u}}^{t+\Delta t} + a_6\ddot{\boldsymbol{u}}^t + a_7\ddot{\boldsymbol{u}}^{t+\Delta t} \qquad (20)$$

CASE STUDIES FOR A BUILDING WITH MANUFACTURED DAMPERS:

These case studies aim to assess the accuracy of model superposition method when applied to steel buildings with non-classical damping due to the use of manufactured dampers.

Sections for beams and columns of the buildings have been selected based on a preliminary structural design. This selection will make stiffnesses of beams and columns more reasonable and this in turn makes any conclusions on these case studies has more practical value. Based on same reasoning, mass of structure has been computed based on load values similar to that recommended in the building codes.

In these case studies, each building will be defined in terms of number of stories and number of bays (with story height of 3m, and bay width of 5m). For example the building shown below will be referred as a building with three stories and three bays:

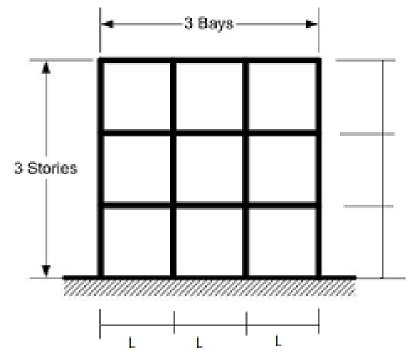

FIG. 1: ASSEMBLE OF BUILDING FRAME

RESULT

Based on the above information, results of these case studies have been summarized in the Tables below:

TABLE: 3.
ASSESSMENT OF MODEL SUPERPOSITION METHOD BASED ON MAXIMUM TIP DISPLACEMENT FOR BUILDINGS WITH A PURE JARRET BRACING:

Case Study No	No of Bays	No of Stories	Fundamental Time Period "T" (sec)	Maximum "u" Based on Model Superposition mm	Maximum "u" Based on Direct Integration mm	Absolute Error In Model Superposition Method %
1.	3	2	0.736	6.88	9.27	25.78
2.	3	3	0.868	9.87	14.9	33.76
3.	3	4	1.04	13.27	20.3	34.63
4.	3	5	1.16	15.38	23.68	35.05
5.	3	6	1.34	18.35	25.87	29.07
6.	3	7	1.50	21.2	27.0	21.5
7.	3	8	1.67	24.9	28.3	12.0
8.	3	9	1.79	28.3	30.4	6.85

TABLE: 4.
ASSESSMENT OF MODEL SUPERPOSITION METHOD BASED ON MAXIMUM BASE SHEAR FOR TYPE PURE JARRET:

Case Study No	No of Bays	No of Stories	Fundamental Time Period "T" (sec)	Maximum "V" Based on Model Superposition kN	Maximum "V" Based on Direct Integration kN	Absolute Error In Model Superposition Method %
1.	3	2	0.736	68.2	91.2	25.22
2.	3	3	0.868	98.2	146.5	32.97
3.	3	4	1.04	120.1	175.9	31.72
4.	3	5	1.16	141	195	27.69
5.	3	6	1.34	144.7	189	23.44
6.	3	7	1.50	145.8	177.1	17.67
7.	3	8	1.67	141.9	158.4	10.42
8.	3	9	1.79	137.5	151.5	9.24

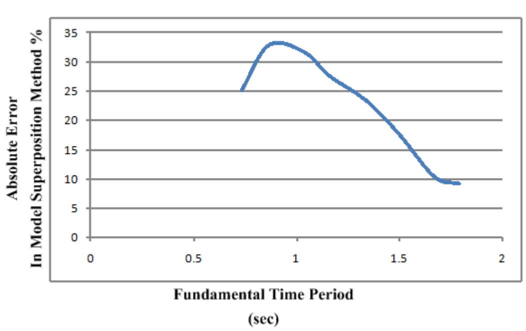

FIG. 2: BASE SHEAR OF PURE DAMPER.

CONCLUDING REMARKS :

The main concern of the above results is that there are different error ratios for different case studies. This can be explained as follows:

1. As discussed in the previous sections, the main difference between the direct integration method and the model superposition method is related to the damping modeling. where for a

structure with non-classical damping, model superposition represents only an approximate method.

2. For harmonic excitation, it is well known that damping effect is not constant for systems that have different time period but it becomes greater when the excitation frequency approach to one of the system frequencies and as shown in Figure (3) below:

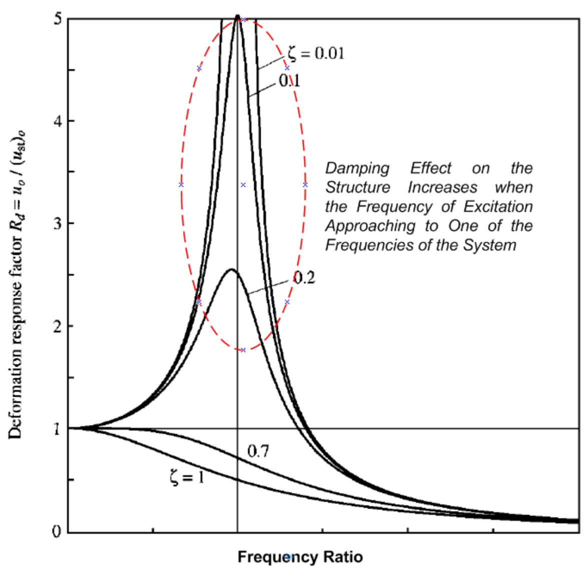

FIG. 3:DEFORMATION RESPONSE FACTOR FOR DAMPED SYSTEM EXCITED BY HARMONIC FORCE (Thomson 1998).

3. It is also well known from the structural dynamics that even nonperiodic functions (like the earthmotion) can be represented in terms of harmonic functions through Fourier transformation technique (**Fertis 1973**). With this approach El Centro earthmotion can be re-represented as shown in Figure (4) below (Spectral Density Function (**Rao 2004**))[1].

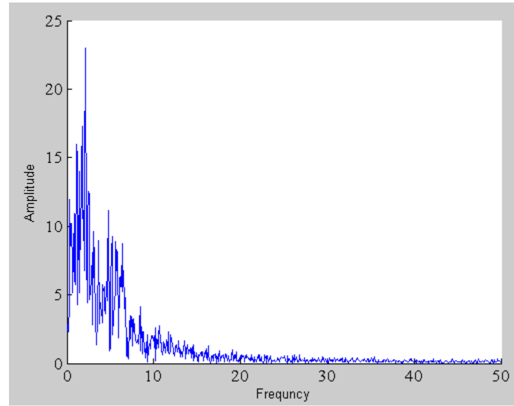

FIG. 4: SPECTRAL DENSITY FUNCTION OF EL CENTRO EARTHQUAKE.

Then, the buildings to be subjected to a large number of harmonic loads with different frequencies and amplitudes can be considered.

4. With the above three facts the results of the case study the fundamental frequency of the structure is either approaching or diverging from one of the dominate frequencies of the earthmotion. If the frequency of the structure is approaching to a dominate frequency of the earthmotion, then the damping effect will be important (as discussed in item 2) and the difference (error) between the direct integration method and the model superposition method is also increasing (as discussed in item 1) and vice versa , Figure (5) below.

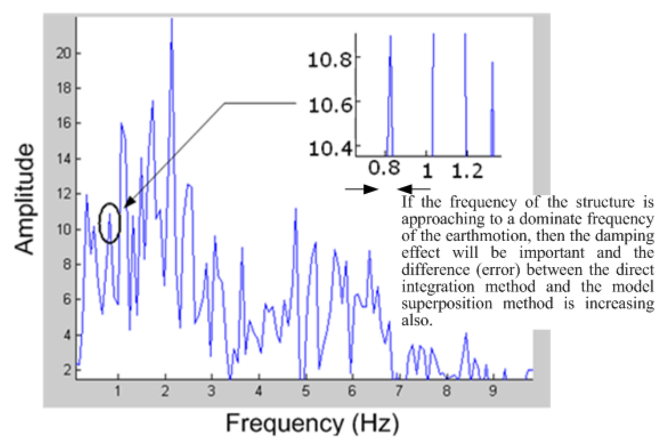

FIG. 5:
EFFECT OF CONVERGENCE BETWEEN BUILDING FUNDAMENTAL FREQUENCY AND ONE OF EARTHMOTION ON THE DAMPING VISCOUS.

[1] Spectral Density Functions through this study have been prepared by the Fast Fourier Transform Algorithm of the Matlab (See **Hanselman and Littlefield 2009** and **Yang, 2005)**)

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NOTATIONS

Δt Time Interval.

C_0 Zero-Frequency Damping Coefficient.

$\dot{u}(0)$ Vector initial velocity.

T Fundamental Time Period.

u Displacement.

v Velocity.

C Damping Constant.

$\boldsymbol{C}$ Damping Matrices.

$\boldsymbol{K}$ Stiffness Matrices.

$\boldsymbol{M}$ Mass Matrices.

$\boldsymbol{u}$ Vector initial displacement.

ω Vibration Natural Frequency.

ω Undamped Natural Frequency.

ϕ Mode Shape Vector.

α Velocity Exponent.

ζ Damping Ratio

ω Load Frequency.

SDOF Single Degree of Freedom System