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DYNAMIC BEHAVIOR AND SEISMIC DESIGN OF STRUCTURAL SYSTEMS HAVING MULTIPLE HIGH-RISE TOWERS ON A COMMON PODIUM

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ABSTRACT

A building system having multiple high-rise towers on a common podium usually have complicated dynamic behavior. A three-degree-of-freedom model, representing a system having two towers and a podium, has been used to study the dynamic characteristics of such kind of structures. It has been found that the interaction between different components in the system has significant impacts on the dynamic behavior of each individual unit. In most cases, some components can develop a more severe response than that in a situation without multi-unit interactions. Such increased response should be considered in the seismic design practice.

KEYWORDS

high-rise buildings, structural system, dynamics, interaction, seismic response, behavior, design, earthquake

INTRODUCTION

Modern architectural design of mixed use complex has introduced a building system which consists of multiple high-rise towers sitting on a common podium structure. Figure 1 shows an example of such kind of development. The towers are typically of different height and plan configurations in order to accomplish various functional and planning requirements. On the other hand, the podium is a low-rise multi-story structure normally having much larger plan dimensions than the tower structures. Although the height and size of the tower and podium structures are different, the mass and stiffness of each component are often ranged in similar order of magnitude. Because of the interaction between all components, the dynamic behavior of such kind of structures is much more complicated than that of conventional buildings.

In order to develop an understanding of the dynamic behavior and the interaction mechanism of such systems, a three-mass mathematical model (Fig. 2) has been developed. The model represents a simple prototype system having a multi-story podium structure (m_1) connecting two tower structures, namely tower A (m_2) and tower B (m_3). Further simplification is achieved by limiting each mass component having only one translational displacement in a two-dimensional plain, resulting a three-degree-of-freedom (3-DOF) system. A parametric study has been performed using the 3-DOF models.

DYNAMIC BEHAVIOR OF SIMPLIFIED 3-DOF SYSTEMS

Equations of Motion

The dynamic behavior of the simplified 3-DOF system is investigated using the principals of structural dynamics. The equations of motion for an elastic, undamped 3-DOF system as shown in Fig. 2 subjected to earthquake ground motion can be expressed as:

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{Bmatrix} \ddot{u}_1(t) \\ \ddot{u}_2(t) \\ \ddot{u}_3(t) \end{Bmatrix} + \begin{bmatrix} k_1+k_2+k_3 & -k_2 & -k_3 \\ -k_2 & k_2 & 0 \\ -k_3 & 0 & k_3 \end{bmatrix} \begin{Bmatrix} u_1(t) \\ u_2(t) \\ u_3(t) \end{Bmatrix} = -\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix} \ddot{u}_g(t) \quad (1)$$

By solving the eigenvalue problem of equation (1), the frequencies (ω_1, ω_2, and ω_3) and mode shapes (Φ) can be expressed in terms of the mass and stiffness values. Once the dynamic properties of the system are determined, the maximum elastic forces generated in each mass and the total base shear corresponding to the nth mode can be calculated in the form of:

$$\begin{Bmatrix} f_{1n} \\ f_{2n} \\ f_{3n} \end{Bmatrix}_{\max} = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{Bmatrix} \varphi_{1n} \\ \varphi_{2n} \\ \varphi_{3n} \end{Bmatrix} \frac{L_n}{M_n} S_a(\omega_n, \xi_n) \quad (2)$$

and

$$V_{b_n} = [1 \;\; 1 \;\; 1] \begin{Bmatrix} f_{1n} \\ f_{2n} \\ f_{3n} \end{Bmatrix} \quad (3)$$

in which the definition of L_n and M_n are the earthquake-excitation factor and the generalized mass of the nth normal mode, respectively. Detailed definition of L_n and M_n can be found in the literature for structural dynamics (Clough and Penzien, 1993). The total response resulting from all modes can be evaluated using established modal combination techniques (Der Kiureghian, 1979).

It is well understood that developing general solutions for the above equations is not practical. In this study, the solution is obtained for a set of predetermined parameters. In order to maintain the applicability of the solution to general conditions, all parameters are presented in relative values wherever possible. Details of the selection of parameters and determination of their values are discussed next.

Parametric Studies

In order to minimize the number of variables, and at the same time, maximize the flexibility of representing different structural system, a reference mass and stiffness are set as *m* and *k*, respectively.

It is assumed that the podium mass (m_1) equals to *m*, and tower A (m_2) and tower B (m_3) always have the same mass of γm. Theoretically, the value of γ may vary from zero to infinity having the extremes representing situations of having no tower mass or no podium mass. A range from 0.2 to 2 is considered in this study.

Similarly, the stiffness k_1 is assumed to equal to *k*, and the stiffnesses k_2 and k_3 equal to αk and βk, respectively. The values of α and β are always positive, and may vary independently to represent tower A and tower B having different stiffness characteristics. However, the summation of α and β is limited to be less or equal to unity to avoid situations where the lower part of a structure is softer than the upper part. For the purposes of current study, the value of $(\alpha+\beta)$ is limited in the range from 0.1 to 0.9. In addition, the value of α is limited to be smaller than or equal to β implying an assumption that tower A is always softer than tower B, or at an extreme, the two towers are identical (noticed previously that the mass of the two towers are the same). A summary of the basic parameters, the definition and their range of variation can be found in Table 1.

Table 1. Summary of Parameters Used in the Study

Parameter Name & Definition	Range of Variation
$Y = m_2/m_1 = m_3/m_1$	$Y = 0.20$ -- 2.00
$\alpha = k_2/k_1$	$\alpha \leq \beta$ and $\alpha = 0.01$ -- 0.45
$\beta = k_3/k_1$	$\beta \geq \alpha$ and $\beta = 0.10$ -- 0.45

The influence of ground motion characters to the structural response is not included as the main purpose of the current study is to evaluate the dynamic response characteristics of the 3-DOF systems affected by the structural properties. The effects of different ground motions are eliminated by assuming a constant acceleration response spectrum over the entire frequency range. Since different combinations of stiffness and mass of each component may result in the same fundamental frequency for the system, using a constant response spectrum has the advantage of expressing explicitly the difference of the structural response as caused by different combinations of the dynamic properties of each component. Although using such a frequency independent response spectrum limits the applicability of the results to buildings having medium to high frequencies, the results are reasonably representative to buildings having low frequencies.

The response of different systems is monitored in terms of the elastic forces of each mass and the base shear of the entire system. It is not intended in this study to define explicitly the reference mass and stiffness values, all results are presented as dimension-less response factors to emphasize the dynamic response affected by the internal interactions due to different dynamic properties of the each component. Specifically, an elastic force developed at a mass is normalized with respect to the product of the mass and the spectral acceleration. Similarly, the total base shear of the entire system is normalized by the product of the total mass and the spectral acceleration. Using this normalizing approach, an elastic force response factor of unity means a mass responds as it is an independent system. whereas a unit base shear response factor means the total mass of the entire system responds as a single mass system. As no actual mass, stiffness and spectral acceleration values are specified, the displacement response of the system can not be evaluated and is not covered in the study.

RESULTS AND OBSERVATIONS

Elastic Force of Tower Messes

Figure 3 presents the tower mass elastic force response factors for four cases of tower to podium mass ratios ($\gamma=0.2, 0.5, 1.0$ and 2.0). There are two situations that a unit elastic force response factor will be achieved. One case is that the towers have much more mass than the podium such that the mass of the podium become insignificant to the entire system. In this case, if the two towers posses the identical dynamic properties ($k_2/k_3 = 1.0$ in Fig. 3d) each of them will respond independently. The other case is when one of the tower is much softer than the other, the softer one eventually become isolated from the system (when $k_2/k_3 \rightarrow 0$ as shown in Fig. 3a to 3d) regardless of the tower to podium mass and stiffness ratios.

In general, the response of tower structures are amplified by the existence of the podium, and such amplification are greater when the mass of the towers become less dominant, as can be observed in the order from Fig. 3d to 3a. It is also noticed that between the two tower structures, the softer tower almost always has higher response factor than the stiffer one except at the second situation mentioned previously. It can be said that the amplification to the softer tower is compounded by both the podium and the stiffer counter part. The maximum amplification occurs at different tower A and B stiffness ratios (measured by the abscissa) for different tower to podium stiffness ratios (β). For the cases studied, a maximum response factor of nearly two has been observed.

Total Base Shear

Figure 4 shows the variation of base shear coefficient as affected by different combinations of the dynamic properties of the system. The results are presented in a parallel manner as the for the elastic force responses. For each tower to podium mass ratio (γ=0.2 to 2.0 as shown in Fig. 4a to 4d), the total base shear coefficients are depicted for three values of tower B (the stiffer tower) to podium stiffness ratios.

It is clear that the maximum base shear response occurs when the two tower structures are identical ($k_2/k_3=1.0$), and in-phase response of the two towers occurs. As the difference of the dynamic properties of the two towers increase, the phase angle between the peak response of the two towers increase, resulting a decrease in the total base shear response. On the other hand, the base shear coefficients increase consistently when the stiffness of the towers getting closer to that of the podium, as indicated by larger values of β. This can attributed to the fact that more intense interaction within the system can be developed when the dynamic properties of all components become similar.

There is no noticeable relationship can be identified with regard to occurrence of the maximum base shear response and the maximum elastic force response of an individual tower structure. It is apparent that the maximum elastic force response of a tower structure is independent from the maximum total base shear response, which suggests that the maximum response of in individual component should not be gauged by the maximum total base shear response.

SEISMIC DESIGN IMPLICATIONS

The current design practice usually adopts a static or dynamic design approach as outlined in the Uniform Building Code (UBC) (Volume 2, 1994) or other modern seismic design provisions. The equivalent static lateral procedure is intended to define a minimum earthquake lateral load for regular structures having nearly uniform mass and stiffness distribution along the height of the structure. The dynamic approach is to be employed whenever significant irregularities exists in a structure, and is expected to capture the more severe design conditions caused by the abnormal dynamic behavior of irregular structures.

However, the dynamic design procedure sometimes can not be carried out as it is intended due to the lack of well defined site specific response spectra for various design limit states. As part of the current design practice, the results from the dynamic analysis using a normalized response spectrum is scaled such that the dynamic base shear has the same value as that defined in the static procedure. For structural systems as discussed in this study, it is observed that the dynamic base shear can be significantly different because of different dynamic properties of each component, even the system as a whole may have the same total mass and the same fundamental frequency. Obviously, scaling the dynamic results due to different combinations to a single reference will not results in a structural design that provides an even protection for all systems.

Based on the current study, the following recommendations are made for the seismic design of structural systems as described herein.

1. A seismic design based on a true dynamic analysis using site specific response spectrum corresponding to the design limit state under consideration. Due to the complexity of the multi-component interaction and the independence between the peak elastic force response of a tower structure and that of the total base shear, scaling the dynamic analysis results with respect to a single reference base shear may results inconsistent factor of safety for different components.

2. Due to the interaction of multiple components, the dynamic properties of each structural should be assessed in a reasonable range, taking possible variations of mass and stiffness quantities into account in order to cover the most critical design conditions. For reinforced concrete buildings, the effects of cracked section to the stiffness of the structure should be considered.

3. Among the two tower structures, the more flexible has more severe dynamic response in most of the situations, particularly when the dynamic properties of the two towers are close. An important implication of this observation can explained in the following situation. When there are two nearly identical towers which are designed similarly, the one that is somewhat softer may develop a more severe response during an earthquake. The induced damage to the softer structure makes it increasingly softer, which in-turn may cause further increased response. Such condition will be terminated until the apparent stiffness of the softer structure has changed significantly to shift the structure off the frequency range of peak response. The decrease of the fundamental frequency of a structure is often accompanied by increased structure deformation. Therefore, cautions should be taken for the design and detailing of the flexible component to assure adequate deformation capacities.

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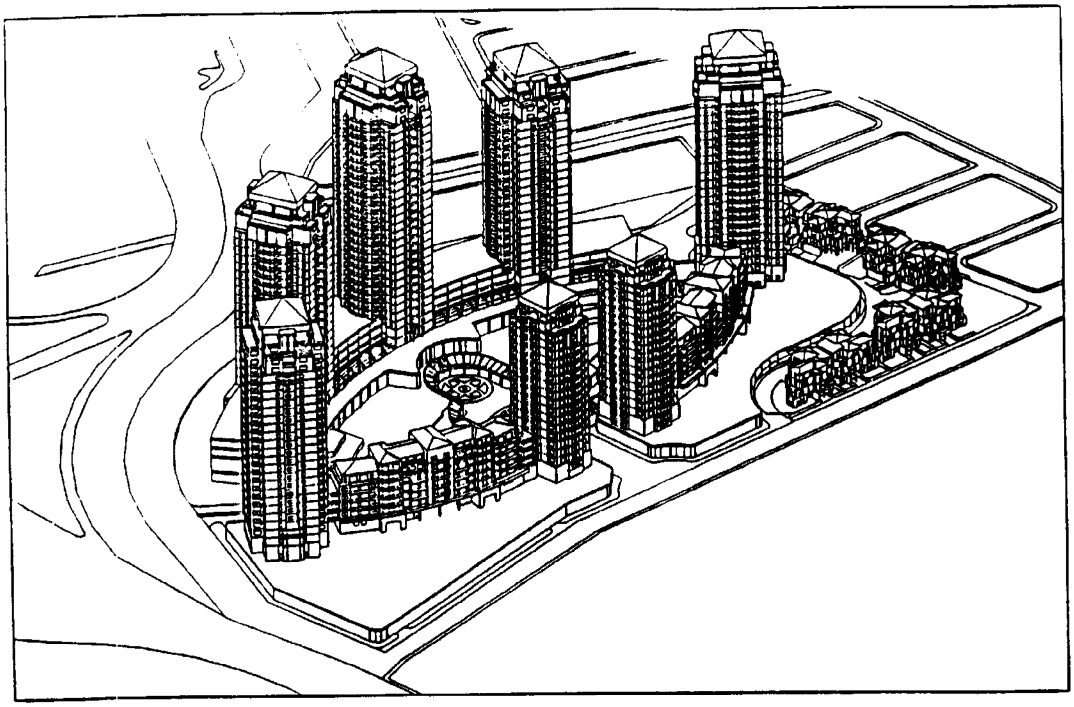

Fig. 1 An Example of Multiple Towers on One Podium Complex

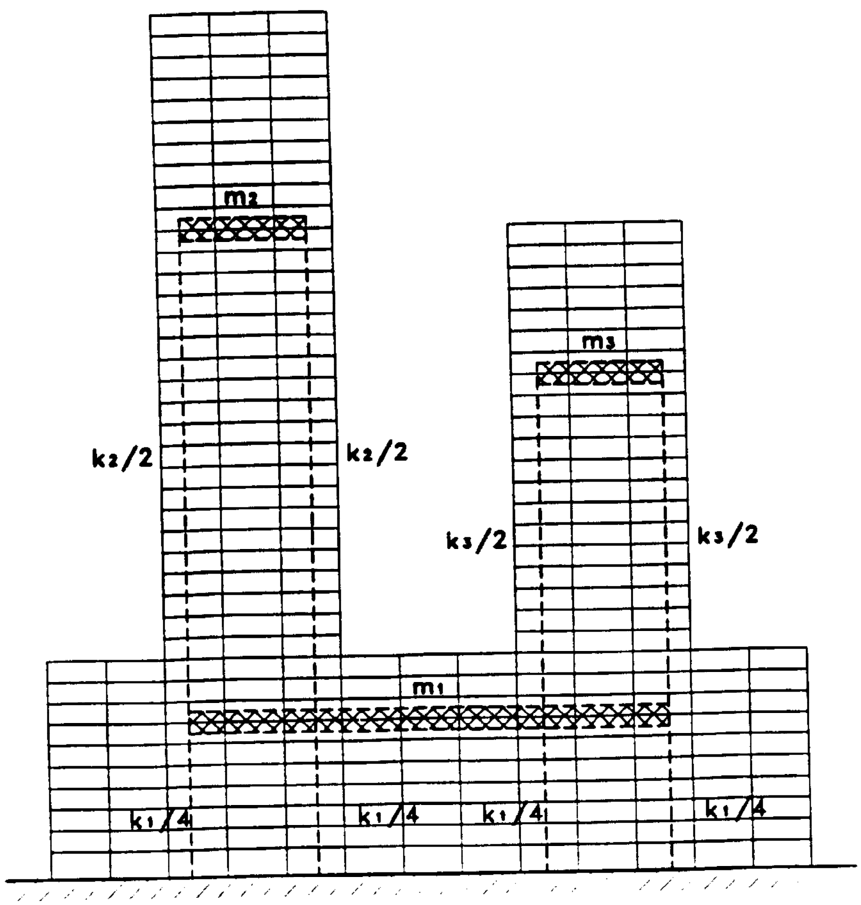

Fig. 2 A Simple Prototype Structure and 3-DOF Model

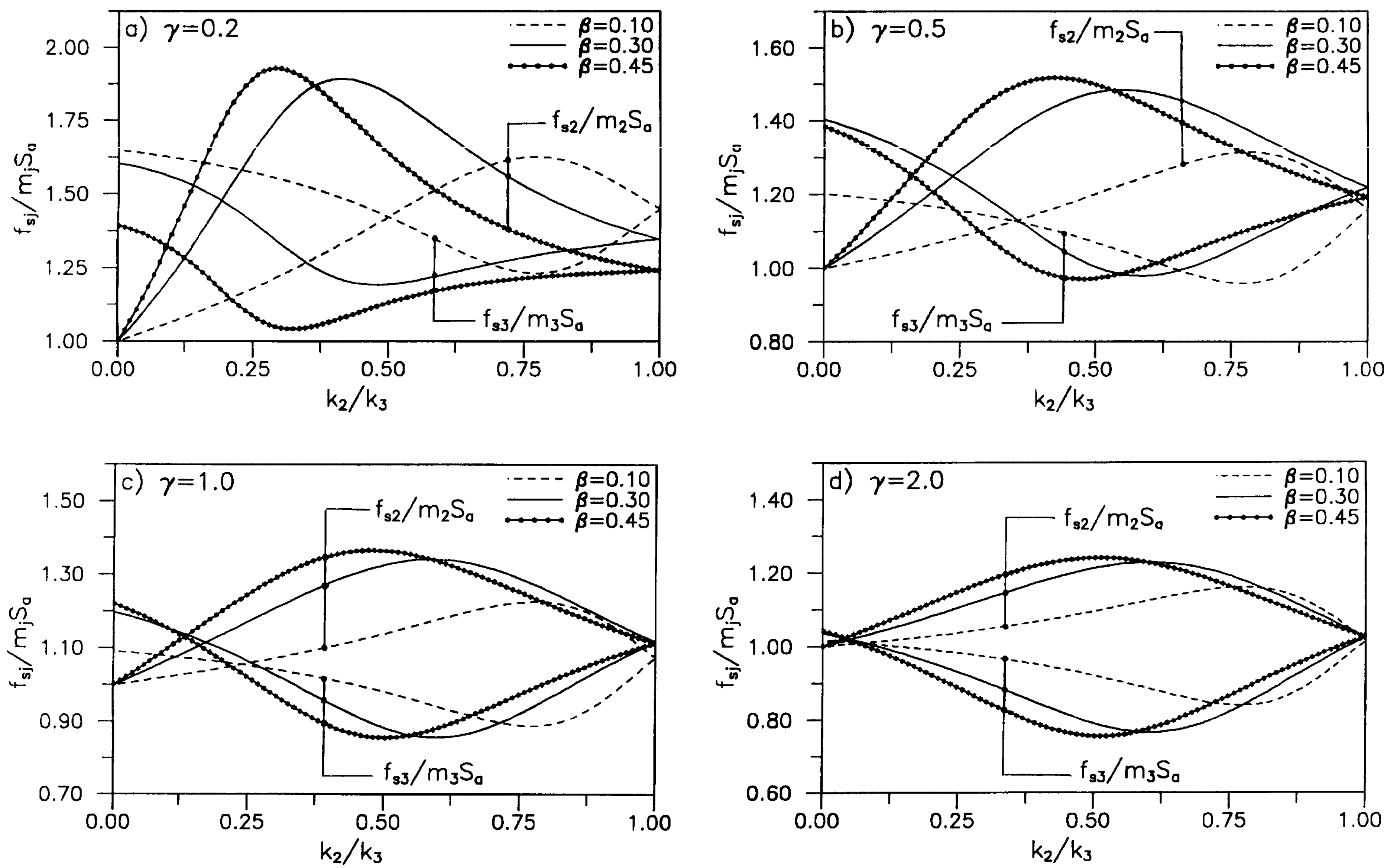

Fig. 3 Tower Mass Elastic Force Response Factors

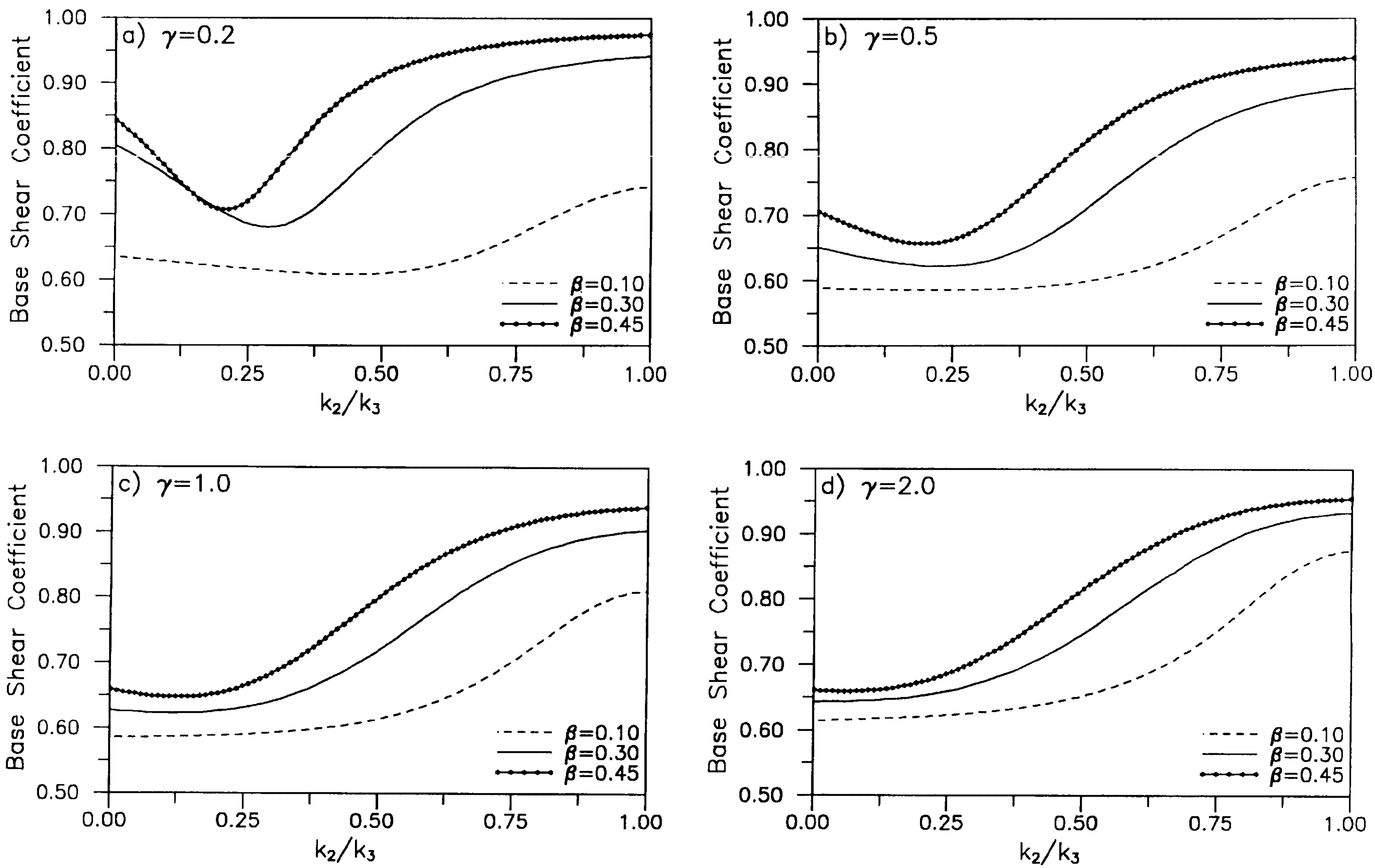

Fig. 4 Total Base Shear Coefficients